G. Manfredi,
M. Dolce
editors

This volume presents the results of the second phase of the largest research program on earthquake engineering ever held in Italy, i.e. Project ReLUIS II. The 2010-2013 ReLUIS research program, which is financially funded by the DPC, improves the knowledge and extends the outcomes of the previous three-year project 2005-2008 ReLUIS.

The 2010-2013 project was aimed at improving the knowledge and developing and integrating the existing regulatory and design tools to manage and mitigate the seismic risk in Italy for what concerns the safety of constructions. The ReLUIS II project mainly comprises the following areas:
– The implementation of a simple yet reliable approaches for the evaluation and mitigation of the seismic risk of existing structures and newly-constructed facilities.
– The evaluation of the existing seismic codes and recommendations for their improvement. Further development includes novel provisions for critical structures, infrastructures and hazardous plants, non-structural components, artistic contents of museums, historical buildings and archeological sites. The latter are new topics that have been successfully investigated during the period 2010-2103.
– The development of new tools and technologies for the health monitoring of structures, the management and mitigation of seismic risk and the post-earthquake rapid response major activities. The training for experts that may assist DPC personnel after moderate-to-major earthquakes is also envisaged.

The project ReLUIS II includes comprehensive experimental research activities carried out primarily in the large laboratory facilities of the Italian ReLUIS network. The experimental tests cover more than 50% of the activities carried out in the 2010-2013 research project. The experimental data (including videos, photos and details of the tests) of the present project have been collected in a repository (web-database), according to sound protocols that have already been used by other similar networks world-wide.

ISBN 978-88-89972-42-7

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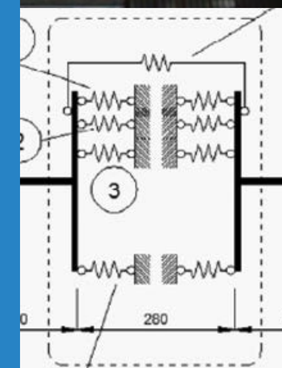


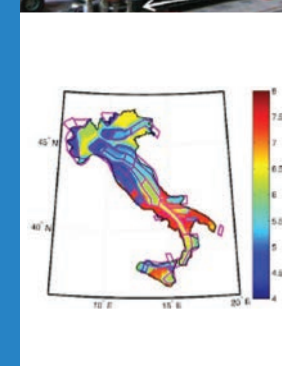


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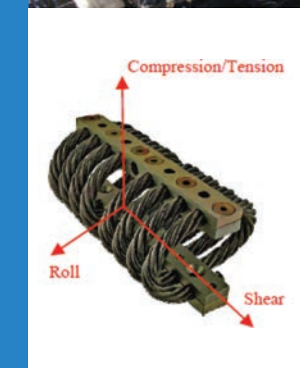



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ISBN 978-88-89972-42-7

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THE CONTRIBUTION OF ReLUIS TO DPC FOR THE TECHNICAL MANAGEMENT OF RECENT EARTHQUAKES AND THE NEW RESEARCH TRENDS FOR THE ADVANCEMENT IN EARTHQUAKE ENGINEERING

Gaetano Manfredi [a], Mauro Dolce [b]

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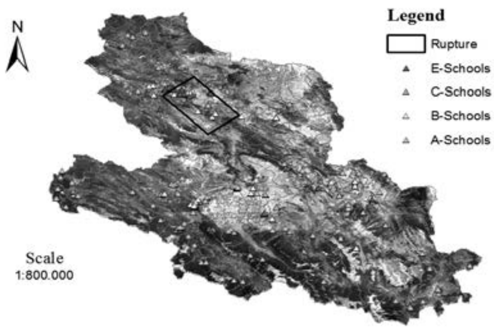
The collaboration of ReLUIS with DPC, besides being related to scientific activities for the improvement of the knowledge finalised to the mitigation of seismic risk, deals also with a strong cooperation in the management of the technical-scientific activities to be carried out in the aftermath of a strong earthquake, as well as in the dissemination of the knowledge on seismic risk to the population, with the same finalisation of risk mitigation.
The recent 2009 L'Aquila and 2012 Emilia earthquakes have shown the importance of post-earthquake inspections aimed at assessing the seismic safety and/or usability of buildings and the evaluation of the functionality of infrastructural systems, e.g. gas and water pipelines, electrical and telecommunication networks. The technical inspections carried out by experts, such as structural engineers, architects and trained technicians, are crucial activities to quickly identify the buildings and the areas with a potential seismic risk for the population. Such post-earthquake inspections are finalized to minimize the impact of the losses on citizens as fast as possible. The ReLUIS (i.e. Rete dei Laboratori Universitari di Ingegneria Sismica) consortium, which is a Competence Centre of the Department of Civil Protection (DPC), had a key role in the post-earthquake emergency of Abruzzo and Emilia earthquakes. The Consortium supported DPC in the technical surveys of buildings, with emphasis on school buildings, strategic industries and sites with historical and monumental heritage. Researchers from several Italian universities have contributed to the surveys in the aftermath of L'Aquila (with more than 600 technical survey teams) and Emilia (employing about 250 researchers) earthquakes.

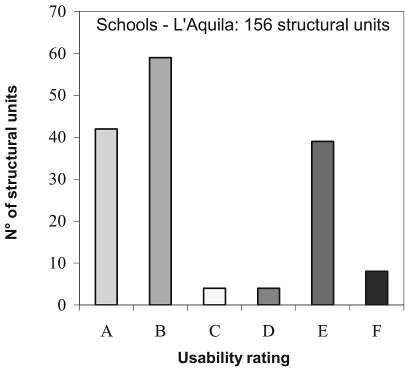
1 THE 2009 ABRUZZO EARTHQUAKE AND THE CONTRIBUTION OF RELUIS TO THE COMMUNITY RECOVERY

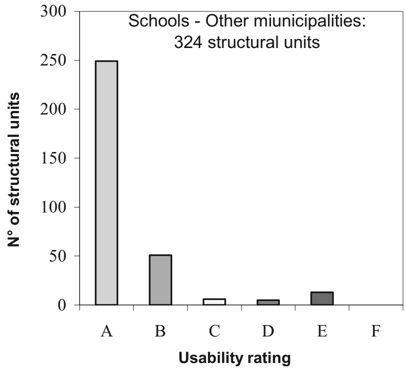
One of the primary goals of DPC after the 2009 L'Aquila earthquake was the assessment of school building damage and the definition of a sound strategy to quickly repair and strengthen those structures which exhibited minor structural damage. The structural safety assessment was performed for schools located in the city of L'Aquila as well as in the surrounding municipalities, namely in the Provinces of L'Aquila, Pescara, Teramo and Chieti. In L'Aquila, 62 school complexes, comprising 156 buildings, were investigated: 53 primary schools (hosting 6300 students out of a total of about 7000) and 9 secondary schools (hosting 4000 students out of a total of about 5000). The inspections on school buildings of municipalities located in L'Aquila Province included 234 primary or secondary schools for a total of 324 buildings, as pictorially reported in Figure 1. The data plotted in the figure show

that 66% of the school buildings of L'Aquila are RC structures (56% RC framed structures and 10% RC shear wall-type structures), 21% of the sample include masonry structures. About 30% of the RC framed structures were assessed with usability rating A (i.e. no significant damage), about 40% B (i.e. no significant damage on structural members) and about 25% E (i.e. significant damage on both structural and non-structural elements). The buildings with a lower level of damage (i.e. A and B) have been built between the '60s and the '90s, while the structures recorded as E were mainly built between the '20s and the '70s. The RC framed structures showed damage primarily on non-structural elements, e.g. partitions and ceilings. The RC shear wall-type structures were assessed only as A (27%) or B (73%); no significant damage on structural members were found.
Additionally technical activities were carried out in the L'Aquila post-earthquake management by the ReLUIS Consortium. It was, indeed, involved in the supervision of the repair and strengthening intervention of lightly damaged school buildings; the activity allowed school buildings in Abruzzo to be ready for the new school year. Once the *in-situ* inspections were ended, several teams with members from ReLUIS and the Public-Works Office of Lazio, Sardinia and Abruzzo were asked to: i) investigate the repair/strengthening measures to be applied to school buildings; ii) define the budget for repair/strengthening works; iii) schedule the repair/strengthening works in order to start the school year on September 2009. Priority was given to interventions on schools with usability rating A and B (see also Figure 1).



(a)





(b) (c)

Figure 1. L'Aquila earthquake: usability rating of school buildings (a); L'Aquila municipality (b); other municipalities (c).

The repair strategies as well as the intervention design were carried out by engineers of municipality or province under the supervision of ReLUIS and the Public-Works Office. The Public-Works Office also managed the bids for the construction works. According to Ordinance 3779 and its commentary, the afore-mentioned interventions were targeted to the repair of non-structural elements and local strengthening interventions on structural members (e.g. strengthening of exterior joints on RC structures, insertion of tie rods and braces on masonry structures). Interventions on non-structural elements (interventions on partitions in order to strengthen them and avoid their overturning, as well as to connect their internal and external faces) were also considered. Before carrying out the repair and strengthening works, non-destructive tests, such as rebound and sonic tests, tests on concrete cores and steel specimens, as well as in-situ compression tests by using flat jacks on masonry structures and load tests on slabs, were carried out. The analysis of several case studies allowed a variety of repair or local strengthening interventions on existing RC or masonry structures to be optimized.

A detailed description of standard repair interventions to repair and strengthen buildings damaged by earthquake loadings has been reported in a document issued jointly by DPC and ReLUIS: "Guidelines for Repair and Local Strengthening of Structural and Non Structural Members". The successful synergy between DPC, ReLUIS, Public-Works Office as well as the involved municipalities and the Province of L'Aquila allowed school buildings with usability rating A to be re-opened before summer, thus the exams at the end of the school year were regularly carried out. Further, repair/strengthening works were carried out on 41 schools (hosting about 8,300 students) with usability rating B, with a total cost of € 27millions, so that the whole stock of school buildings assessed as B could be re-opened within October 5th, 2009.

Furthermore, the ReLUIS consortium has been a key player in the reconstruction of residential buildings outside the historical centres damaged by the L'Aquila earthquake. The primary activity of the Consortium comprised the technical checks and approval procedure for government funding assigned to damaged buildings in the Abruzzo region. The checks carried out by ReLUIS included the evaluation of the consistency between repair intervention and damage. Additionally, the compliance between designed local (or global) strengthening interventions and current seismic code provisions and ordinances issued after the L'Aquila earthquake were also investigated.

To support the designers and practitioners involved in the post-earthquake reconstruction process, technical reports on the earthquake sequences and damage on the structures, handbooks for seismic analysis and design (see for example Figure 2), guidelines and recommendations as well as software have been provided in the immediate post-earthquakes and shared freely on the official website of the consortium (www.reluis.it).

The design tools that have been provided by ReLUIS researchers were validated also on the basis of the outcomes of the experimental and analytical work carried out within the 2005-2008 and 2010-2013 DPC-ReLUIS Projects. The results of the 2005-2008 DPC-ReLUIS Project have been summarised in the volume "The State of art of Earthquake Engineering Research in Italy: the DPC-ReLUIS 2005-2008 Project", edited by the Authors. The latter volume is a companion publication that may be of interest for researchers, practitioners and engineers concerned with the interdisciplinary field of seismic risk evaluation and mitigation. The present volume focuses on the experimental and numerical results of the 2010-2013 DPC-ReLUIS Project.

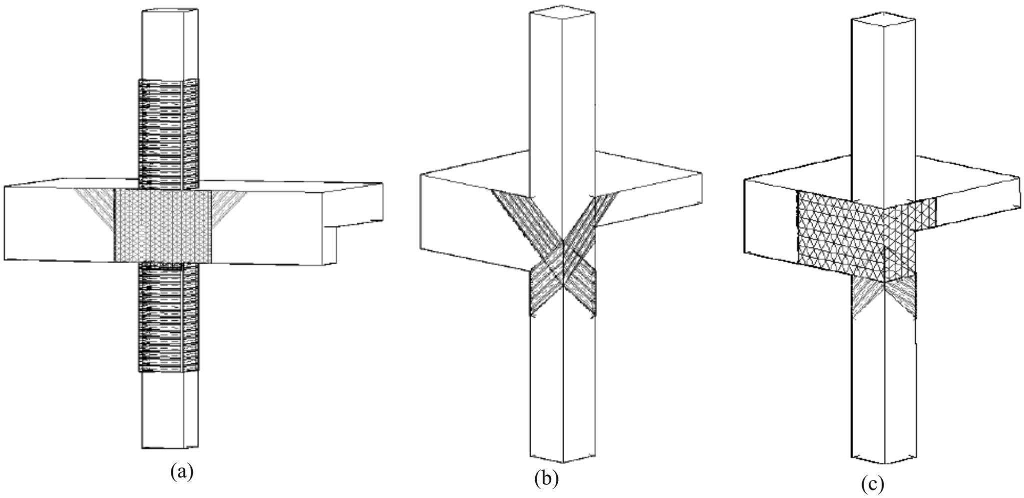


Figure 2. Local interventions on RC members and joints proposed in the ReLUIS design manual: use of FRP for column wrapping (a); use of FPR uniaxial (b) and quadri-axial fibers (c) for beam-to-column nodes.

2 THE 2012 EMILIA EARTHQUAKE AND THE CONTRIBUTION OF RELUIS TO THE COMMUNITY RECOVERY

On May 2012 a sequence of two earthquakes (on the 20th and 29th of May with M_L=5,9 and M_L=5,8, respectively) occurred in Emilia-Romagna region (Northern Italy), causing many injured and homeless people; 27 people died. The maximum horizontal peak ground accelerations recorded vary between 0.26g and 0,31g, while about 0.9g vertical acceleration was recorded near the epicentre during the second shock.

The sequence with the two mainshocks that occurred in Emilia-Romagna Region generated significant damage in structural and non-structural systems. As in the previous experience of 2009, teams of technical experts were coordinated by the Consortium ReLUIS to carry out seismic and usability assessment of public and private buildings, within the general emergency coordination made by the Civil Protection Department. The Emilia earthquake strongly impacted on productivity of primary importance for the local and national economy. Public buildings and numerous historical buildings suffered severe damage; extensive structural and non-structural damage was detected in masonry and RC buildings. Numerous inspections were carried out by ReLUIS on private and public structures. Such inspections were performed on 330 structures, including 200 schools, 15 public offices, 10 hospitals, 5 barracks, and several theatres, libraries, sport centres, etc. The usability rating of the structural units inspected by the ReLUIS Consortium are reported in Figure 3. The Figure shows that about 31% of structures had a usability rating A, 31% B, 6% C, 4% D, 23% E and 4% F.

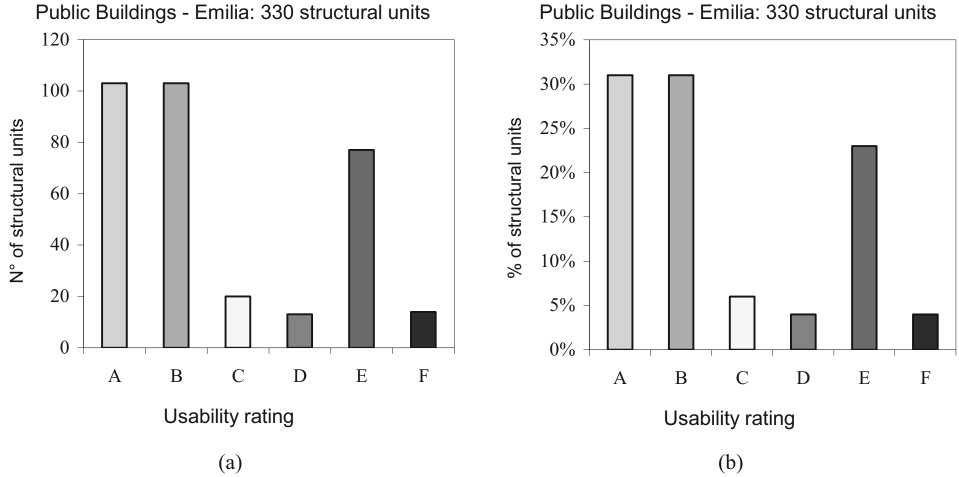


Figure 3. Emilia earthquake: usability rating of public buildings (a); usability rating ratio of public buildings (b).

The damage and/or collapse of numerous industrial structural systems caused huge economic losses, due to both the direct economic damage amounts and the indirect losses generated by the business interruption. Therefore, after the 20th and 29th May seismic events, the actions of the emergency phase were two-fold: (i) protect the life of the local citizens and (ii) restart rapidly the commercial and industrial activities located in the earthquake affected zone. Furthermore, there was an urgent need to provide simple yet reliable design aids to either repair or strengthen existing precast building structures located in the region affected by the earthquake. Thus, a group of Italian researchers, academics and structural engineers formulated the “*Guidelines on local and global retrofitting systems of precast structures*”, a useful reference document for professional engineers and practitioners acting in the post-earthquake assessment and retrofitting of industrial buildings. This document was edited under the supervision of the Italian Department of Civil Protection and with the fundamental collaboration of the ReLUIS consortium, among other institutions. It provides the most advanced knowledge on structural seismic safety and shows the operational process that can be implemented to obtain the seismic usability, according to the current Italian buildings codes (D.M. 14.01.2008, Circolare n.617/2009) and to the ad-hoc issued law (Legge 122/2012).

The document consists of four chapters:

1) Report of the recorded damage in the precast structures;
2) Description of the most commonly used structural types of industrial buildings, designed primarily for gravity loads;
3) Requirements and systems in order to achieve the seismic usability and the seismic safety;
4) Technical aids for the design and the verification of the suggested retrofitting systems.

The contribution of the ReLUIS consortium was essential for the development of the guidelines for precast structures. Numerous structural and non-structural deficiencies were observed during the 2012 Emilia earthquake sequence. The latter deficiencies are documented in details in the guidelines and can be used as a reference for the structural earthquake engineers. The surveyed structures show that the most common deficiency is related to the absence of mechanical devices in the connections between structural elements, causing many collapses due to the loss of support of roof elements (Figure 4(a)) and beams (Figure 4(b)).

(a) (b)

Figure 4. Roof elements collapse due to the loss of support from main beam (a) and loss of support of beam from column (b).

Another important source of vulnerability observed in the aftermath of the Emilia earthquakes is the inadequate connections between the structure and the cladding systems of precast buildings, consisting of precast RC panels. The widespread collapse of these non-structural elements was caused by the failure of the connections and not by inadequate design and/or construction of the panels themselves, as depicted in Figure (a). Furthermore, storage rack without bracings failed with consequent losses of contents and often with consequences for the structure, due to the impact against it, as surveyed in a number of industrial buildings (see Figure 5 (b)).

(a) (b)

Figure 5. Collapse of horizontal precast panels (a) and damage to storage racks (b) with consequent loss of contents, as surveyed in a number of industrial buildings.

Seismic retrofitting strategies are also included in the above mentioned “*Guidelines on local and global retrofitting systems of precast structures*”; they were implemented according to the two-phases process of the retrofitting requested specifically by the Law 122/2012.

The first phase of the intervention process consisted of quick emergency actions aimed at obtaining the positive usability judgment. This goal could be achieved by removing/resolving the main deficiencies: 1) lack of connections between structural elements; 2) infill precast elements not properly anchored to the main structure; 3) not braced storage rack, loaded with heavy materials that can involve in their collapse the main structure. The second phase required the assessment of the structural seismic safety and the seismic retrofitting of the structures, according to the building code in force, in order to obtain the needed performance level.

As far as the retrofitting actions are concerned, several solutions have been proposed for the precast industrial building systems. They include the following:

- Actions to prevent the loss of support of the horizontal elements. The main goal is to prevent the loss of support of the horizontal elements by preventing the relative displacements between the elements without changing the structural response of the existing system. For instance, one of the suggested retrofitting solutions is the introduction of new mechanical devices between the two elements (see Figure 6).

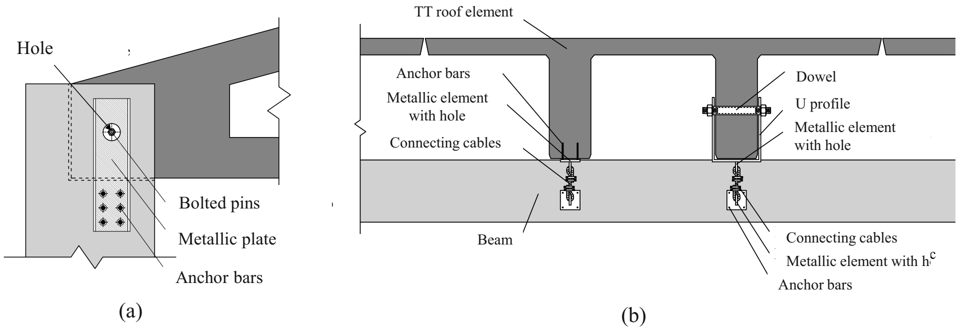


Figure 6. Retrofitting solution for beam-to-column connections (a) and roof-to-beam connection (b).

- Interventions to prevent the collapse of the cladding panels. The retrofitting systems should prevent the overturning of the panels (Figure 7) and/or they should allow the relative displacement between the panel and the structure in order to reduce or remove the interaction under seismic actions.

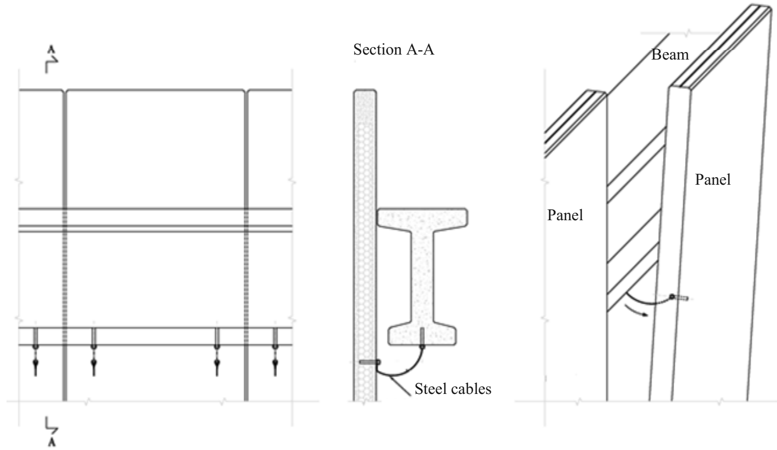


Figure 7. System to prevent the overturning of the cladding panels.

- Interventions on the foundation systems. The retrofitting systems should increase the rotational stiffness in order to ensure the connection at the base of the columns and/or they should increase the strength to the lateral forces, according to a capacity design approach.
- Interventions on the vertical structural elements. The retrofitting actions should increase the stiffness, the strength and/or the ductility of the elements.
- Interventions on storage rack. The retrofitting systems should remove any connections between the racks and the main structure and/or they should reinforce/brace all the levels with systems and devices in order to support the load and to prevent the contents fall.

3 THE DISSEMINATION PROGRAM BY RELUIS-DPC

The seismic emergency following the 2009 L'Aquila, the 2012 Emilia-Romagna and, more recently, also the 2013 Lunigiana-Garfagnana earthquakes have demonstrated the crucial role of all those initiatives aimed at the promotion and dissemination of the knowledge of the effects of the seismic risk in the social and the technical communities. The ReLUIS Consortium has been working closely in partnership with DPC to stimulate measures aimed at verifying and enhancing emergency preparedness. The aim is to build resilient structures, infrastructures and communities, thus minimizing earthquake-losses. To do so, numerous actions have been carried out with a constant and successful collaboration between the interested stakeholders, e.g. scientific community, local and national governmental agencies, civil protection volunteers, etc.

Special attention has been devoted to the implementation at local scale of a national prominent initiative for seismic risk reduction, that is the national campaign "Terremoto, Io non rischio - Earthquake, I don't take risks" (TINR, www.iononrischio.it) sponsored by DPC and ANPAS in collaboration with INGV and ReLUIS. The first edition of TINR campaign was held on 22 and 23 October 2011 in the squares of nine Italian towns located in high seismicity zone. A selected group of volunteers was firstly trained by experts from DPC, ANPAS, INGV and ReLUIS on basic concepts concerning hazard, vulnerability, seismic risk and communication procedures. Then, in turn, they trained other volunteers widening the number of actors to be committed in the process of risk knowledge and awareness improvement in the population. During the first campaign, 120 volunteers distributed information, illustrative material and provided answers to the citizens' questions on possible individual actions to carry out in order to mitigate seismic risk.

As a consequence of the positive feedback and results of the 2011 campaign, a second edition was planned on October 2012 extended to about 100 squares throughout the country (two of the involved locations are in the area affected by the Pollino seismic swarm, i.e. the villages Lagonegro and Rotonda). In the second edition of TINR campaign over 1.500 trained volunteers from 12 different national associations working on civil protection have been involved. The third edition of the campaign took place on September 28 and 29, 2013, in more than 200 Italian municipalities (mainly located in areas classified as high seismicity zones). In the 2013 edition, the trained volunteers involved in the campaign were more than 3.000 units.

Whereas the TINR campaign is designed and performed in "peace-time", an analogous activity thought for the emergency phase following a damaging seismic event was performed after the May 2012 Emilia-Romagna earthquake, with the initiative named "Terremoto - parliamone insieme / Earthquake: let's talk together". It is mainly aimed at transferring information to local technicians, administrators and common people concerning the seismic sequence characteristics, criteria and results of the usability surveys, activities to be carried out to repair damage and strengthen the buildings in the affected area, etc.. This initiative was promoted by DPC, INGV and local administrations, with the contribution of ReLUIS mainly involving local experts being part of the ReLUIS network. The initiative "Terremoto - parliamone insieme" has been also carried out after the M5.2 earthquake occurred on June 20, 2013 in the area of Lunigiana-Garfagnana, Tuscany.

4 THE FRAMEWORK OF THE 2010-2013 RELUIS-DPC PROJECT

The Project ReLUIS II, which is financially funded by the DPC, improves the knowledge and extend the outcomes of the early three-year project 2005-2008 ReLUIS. The new project is aimed at developing and integrating the existing regulatory and design tools to manage and mitigate the seismic risk in Italy. The RELUIS II project comprises the following areas:

- The implementation of a simple yet reliable approach for evaluation and mitigation of the seismic risk of existing structures and newly-constructed facilities. Such platform is intended for a regional scale approach and it is flexible enough to incorporate the outcomes of existing and new methodologies for the evaluation of the seismic vulnerability and the retrofitting measures based on traditional and innovative materials and technologies.
- The revision of the existing seismic codes and recommendations. Further development includes novel provisions for critical structures, infrastructures and hazardous plants, non-structural components, artistic contents of museums and historical buildings and archeological sites. The latter are new topics that have been successful investigated during the period 2010-2103.
- The development of new tools and technologies for the health monitoring, the management and mitigation of seismic risk and the post-earthquake rapid response major activities. The training for experts that may assist the DPC personals after moderate-to-major earthquakes is also envisaged.

The project ReLUIS II includes comprehensive experimental research activities carried out primarily in the large laboratory facilities of the Italian ReLUIS network. The experimental tests cover more that 50% of all the activities that have been carried out in the new 2010-2013 research project. The experimental data (including videos, photos and details of the tests) of the present project have been collected in a repository (web-database), according to sound protocols that have already been used by other similar networks word-wide.
Three research areas have been identified for the ReLUIS II project; they include the following:

- Research Area 1 (RA-1): *Tools to evaluate and manage the seismic risk of the existing structures*

4.1 New aspects in the evaluation of the seismic vulnerability of the existing structures and retrofitting measures. Evaluation of the seismic risk at regional scale;
4.2 Displacement-based methods for the evaluation of the seismic vulnerability.

- Research Area 2 (RA-2): *Advances in seismic codes and novel technologies for earthquake engineering*

2.1 Aspects in the seismic design of new structures;

2.2 Evaluation of the vulnerability and seismic risk of special systems;
2.3 Technologies and innovations in earthquake engineering.

- **Research Area 3 (RA-3)**: *Technologies for the monitoring of the seismic risk and the management of the emergency*

3.1 Development of new technologies for health monitoring and emergency response;
3.2 Actions for the management of the emergency and rapid response.

- **Special Topics**: *Interdisciplinary topics.*

ST1: Engineering issues of the seismic input characterization;
ST2: Near-source seismic demand and effects on structures;
ST3: Evaluation of the usability of buildings, damage evaluation and tools for the reconstruction;
ST4: Capacity models to evaluate the vulnerability of existing structures;
ST5: The influence of the outcomes of the ReLUIS II project on the Italian seismic code of practice.

Additionally, special topics relative to the geotechnical earthquake engineering were developed. Three topics were identified; they include: GA1 (local site response and lifelines); GA2 (superficial and deep foundations), GA3 (rigid and flexible geotechnical structures).
Technical reports were issued yearly and were reviewed by a panel of international experts, which includes: Professor A.S. Elnashai (University of Illinois at Urbana-Champaign USA), Professor P. Fajfar (University of Ljubljana, Slovenia) and Professor K. Pitilakis (Aristotle University of Thessaloniki, Greece).
The deliverables of the 2010-2013 ReLUIS-DPC research project include guidelines and recommendations for the next generation seismic codes of practice, detailed design handbooks, advanced software for designers and practitioners (see also www.reluis.it).

G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 1-49, doi: 10.14599/r101301, © 2015 Doppiavoce, Napoli, Italy

REINFORCED CONCRETE STRUCTURES

Gaetano Manfredi [a], Giorgio Monti [b], Enrico Spacone [c]

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1 INTRODUCTION

Since the adoption of the National Technical Code in 2008 (NTC 2008), methods and procedures thereby contained have been the subject of a continuous and accurate assessment through their widespread application in the professional world. An indisputable merit of this code is that it has significantly shortened the cultural distance between researchers and practitioners. Topics,that once were only heard of by professionals, are nowadays of common use, such as performance-based design, capacity design, nonlinear analysis, fiber-section modeling, along many others that have been produced in the research field of Earthquake Engineering in the last two decades.
In addition, professionals have codes the idea of dealing with a performance-based code, as opposed to the prescriptive codes they were used to work with. This has endowed the designer with more responsibilities than in the past and has also led to a more aware approach towards the use of the Code; this, almost naturally, brought with it a more critical attitude, as well. A new era then started of a deeper and beneficial interaction between academia and professional world, where the former has received suggestions, comments and reactions from the latter, with the aim of improving the applicative part of the Code, while making the theoretical part more accessible and understandable, especially with regards to the background motivations.
This effort has led to the ReLuis research project 2010-2013 on Reinforced Concrete Structures, coordinated by the authors, which has tried to provide answers to doubts that arose in practitioners in the first years of application of the Code, while at the same time opening new fields of research on topics that are still debated at the international level.
The following topics were deemed worthy of further investigation:

- Knowledge of Existing Structures
- Assessment of Nonlinear Behavior of Buildings
- Influence of Infills on Structural Response
- Behavior and Strengthening of Beam-Column Joints
- Behavior and Strengthening of Columns and Beams
- Local Strengthening Methods
- Global Retrofitting Methods
- Behavior and Strengthening of Industrial Structures

Several Italian Universities took part in this effort, with a uniform distribution over the Country, which testifies the constructive interactions that the ReLuis project has ignited among several, geographically spread, researchers.

These are (along with the local Coordinator): Naples (Manfredi), Rome, (Monti), Chieti (Spacone), Catania (Ghersi), Potenza (Masi), Rome 3 (Nuti), Florence (De Stefano), Benevento (Di Sarno), Bologna (Savoia), Bari (Marano and Mezzina), L'Aquila (Galeota), Naples (Nigro and Verderame), Milan (Toniolo), Salerno (Faella), Udine (Russo), Palermo (Papia).

This report is structured in the following sections:
- Background and motivation
- Research Structure
- Main Results
- Discussion
- Visions and Developments
- References

In each section, the topics are treated separately.

2 BACKGROUND AND MOTIVATION

2.1 Knowledge of Existing Structures

Many reinforced concrete (RC) buildings have suffered structural damage in recent Italian earthquakes. These seismic events and recent seismic codes emanated after 2003 have led to a national campaign of seismic vulnerability assessment of existing RC buildings. Such assessments are only possible on the base of verified assumptions on the mechanical properties of the materials. Material characterization of concrete and reinforcing steel bars of aged RC structures has become a technical need of both practical and scientific interest. A relation between mechanical performances and state of conservation, evaluated of old concrete and reinforcing steel bar specimens taken from existing structures and buildings, have been investigated with respect to the structure age, the historical and present use, number of stories, method and phases, structural elements the different samples were taken from (columns, beams, etc.). A validation of the existing formulas to determine the concrete elastic modulus and compressive strength through non-destructive methods (ultrasonic, rebound and SonReb method) was attempted. Moreover, specific analyses were run to evaluate the effects of different assumptions in terms of the material mechanical behavior when estimating the seismic vulnerability of structures. Finally, the disturbances caused by destructive testing techniques (mainly core drilling) on the concrete strength prediction process were studied.
Another important aspect of the seismic evaluation of existing buildings is the significant amount of uncertainty in the structural modeling parameters. These modeling uncertainties can be classified into two groups; uncertainties in the mechanical properties of the construction materials and uncertainties in the structural detailing or defects. In the structural reliability assessment, the uncertainties are propagated in order to assess the structural performance in terms of the probability of exceeding a specified limit state. The main objective of this task is to provide data and methodologies for a more reliable characterization of model uncertainties in existing RC structures for a possible improvement of current code provisions on this aspect.

2.2 Assessment of Nonlinear Behavior of Buildings

Research on performance-based seismic engineering poses many challenges, among them the need for a reliable procedure to predict structural damage and collapse as a function of the earthquake ground motion intensity. A large source of variability in seismic performance assessment arises from simplification in defining earthquake intensity relative to the proper damaging consequences of ground motions on structures. For both assessment and design applications, the expected structural responses should be estimated with a high level of accuracy using the available computational resources. The structural response is usually best assessed via nonlinear dynamic time-history analysis of advanced 2D or 3D computer models subject to real or synthetic ground motions that are "consistent" with the hazard at the building site. The accuracy in response prediction is accomplished by relying on relationships between ground-motion intensity measures (IMs) and measures of structural responses, which in this framework are often called Engineering Demand Parameters (EDPs). The higher the predictive power of the selected IMs, the lower is the uncertainty in the estimated EDP and, therefore, the lower the number of computer runs necessary to achieve the desired level of statistical accuracy. This issue is also essential for the definition of ground motion selection and scaling methods (GMSMs) to be used in nonlinear dynamic analyses, particularly in decreasing the number of records needed for evaluating the buildings response.

2.3 Influence of Infills on Structural Response

The interaction between masonry infills and RC frames is yet to be full understood. The issue is particularly important for existing buildings. The following topics are of specific interest:

- *Modeling of infill panels*. Mesoscale modeling (including the interactions between resisting elements and mortar joints) and macro-models (one or multiple diagonal struts, simulating the behavior of the infills) are considered. For the equivalent struts, it is important to pursue the possibility of using of nonlinear structural models available in the commonly used software packages.
- *Effects of the infill panels on the dynamic characteristics and the seismic response of RC structures.* The following cases are of interest: a) existing structures, designed to frame gravity loads only or designed according to obsolete seismic code provisions; for these structures the main objective is to verify whether the infills provide beneficial or unfavorable effects; b) new structures, designed according to modern seismic code provisions, in order to verify whether the contribution of the infills influences the capacity design rules. The effects of irregular infill distributions are also of interest.
- *Seismic upgrading of existing RC structures using frame-infill connections or dissipative diagonal braces.*

2.4 Behavior and Strengthening of Beam-Column Joints

Field surveys have identified brittle failure modes of poorly detailed beam-column joints as one of the main sources of vulnerability in older RC buildings (e.g. Abruzzo 2009). Modern codes, such as the Italian NTC 2008, provide detailing prescriptions for the joint regions and for the frame elements in order to avoid brittle failures and improve ductility. However, further studies are required in order to improve such procedures and, specifically, obtain more accurate data on the behavior of beam-column joints, whose role on the global behavior of RC frame buildings can be crucial. Experimental studies and analyses must lead to more accurate strength prediction procedures and should develop strengthening techniques applicable to RC beam-column joints that take into account the actual geometry of the subassemblages, e.g., the presence of slab and other framing elements that can prevent an effective arrangement of the retrofitting system.

2.5 Behavior and Strengthening of Columns and Beams

When dealing with the safety evaluation of a large number of buildings, which is the case of post-earthquake damage assessment, it is important to have easily applicable formulas for evaluating yield and ultimate moment and curvature for RC sections, subjected to bending moment and axial load. Moment-curvature relationships have the merit of synthetically representing how the bending stiffness of the cross section varies from pre-yielding to the ultimate stage, but the fact that they are obtained through fiber section analysis and iterative methods discourages their use for intensive risk or stochastic studies. Closed-form approaches are needed for fast application.

Analytical/numerical and experimental evaluation of the confinement effects produced by external steel cages or FRP/FRCM wrapping on RC columns is another area of great interest. The ultimate curvature and bending moment under uniaxial and biaxial bending are the main quantities under consideration, in order to update current design formulas and procedures

2.6 Local Strengthening Methods

Recent earthquakes have clearly shown the high vulnerability of existing RC structures mainly related to poor concrete quality, lack of adequate transverse steel reinforcement at members' ends and on partially confined beam-column joints, poor attention to details, and design for gravity loads only or with reference to obsolete seismic provisions. Demolition and reconstruction of these RC buildings is often not a viable strategy due to social factors or economic issues. An effective strategy for such structures may be to introduce local reinforcements on components that are inadequate in terms of strength or deformation capacity. Several experimental studies confirmed that FRP interventions can be effective to improve the structural seismic capacity. Following the April 2009 L'Aquila earthquake, local retrofit works based on FRP were widely accepted to increase the seismic capacity of public and private buildings. FRP strengthening of partially confined joints (typically exterior) and column ends, to prevent brittle failure mechanisms and to increase structural dissipation capacity, were largely adopted for the school building stock in order to allow their quick reopening and to significantly reduce the seismic vulnerability of these strategic buildings.

Another issue considered in this task relates to shear strengthening. The design tools, used to evaluate the strength of beams retrofitted in shear with C-FRP, are based on conventional flexural experimental tests, where negative bending moments are neglected according to the scheme proposed by national and international codes and guidelines. Procedures are needed for estimating the structural safety of existing beams retrofitted in shear with C-FRP U strips in case of negative moment at the beam ends and in general in continuous beams.

Another task dealt with the repair and retrofitting of seriously damaged RC bridge piers to restore the original resistance and improve the seismic performance in terms of ductility and shear strength. The repair consists of damaged rebars substitution by inox rebars which increase durability followed by concrete restoration by using self-compacting concrete (SCC) that simplifies the casting process. Retrofitting by C-FRP is a quick solution. Experimental and analytical investigations and verifications are required for improving some intervention details and for developing and tuning proper numerical models to predict the seismic response of existing repaired and retrofitted bridges. Pseudo-dynamic test (PSD) is a valid solution to evaluate the effectiveness of the intervention, reducing the test cost.

2.7 Global Retrofitting Methods

Seismic assessment of non-seismically designed RC structures generally points out their structural deficiencies related to a general lack of strength and ductility of either the most engaged members or the structural system as a whole. Retrofitting is generally required for such structures and various technical solutions can be considered for improving their seismic performance according to suitable objectives, basically depending on the building use.
The possible technical solutions for seismic retrofitting of existing buildings can be classified into two broad classes: the member-level techniques and structure-level techniques (fib, 2003). The former ones can be adopted for enhancing the seismic capacity of the most deficient members by means of various technical solutions such as confinement, jacketing and so on; the latter aim reducing demand on the existing structure as a whole by means of new members and substructures, such as shear walls or steel bracings, that work in parallel with the existing one. Although plenty of researches have been carried out to investigate the behavior of under-designed RC members and structures repaired by one of the above techniques, no general strategy has been proposed for choosing and possibly combining those techniques to obtain the most efficient and effective solution for seismic retrofitting. Completely lacking are studies on the seismic vulnerability of tall buildings and suitable retrofitting techniques. Generally tall buildings can be more affected by wind action than earthquake. Nevertheless, comparison of the design spectra provided by design specification in the DM 1996 and those provided by NTC 2008 for RC walls structures (typical of tall buildings) showed no significant differences for high periods of vibration, for good soil (soil type A, as defined by the NTC 2008) and differences larger than 100% for poor conditions soils (for example soil type D). Hence, the need to investigate the seismic vulnerability of tall buildings.

2.8 Behavior and Strengthening of Industrial Structures

The seismic performance of traditional cast-in-place RC frame structures has been largely studied. The same cannot be said for precast structures. In fact, their peculiar characteristics and, more specifically, their response to seismic excitations, have not been thoroughly investigated and completely understood, especially with regard to the behavior of the connections between the different elements and the interaction of the frame structure with the cladding wall panels. For many years ductility of precast columns was considered the main topic to be clarified, since this type of structures is very often statically determinate. On the contrary, recent Italian earthquakes (2009, 2012) have clearly shown the sensitivity of industrial structures to horizontal forces, since their large deformability requires a strong displacement capacity. Recent earthquakes have also shown that cladding wall panels can represent a strong contribution to irregularity in the structural behavior of precast systems.

Precast concrete systems are continuously enhanced to confront new challenges of building industrialization. The need to guarantee speed of construction and to ensure the continuity of the structure, have led to develop new structural systems in which precast components are assembled and then linked by casting connections and slabs. In these systems the nodes between columns and foundation are crucial in order to ensure the redundancy of the structure. Even though some studies can be found on the static behavior of such connections, very few information can be found regarding their cyclic behavior and mechanical stability.

3 RESEARCH STRUCTURE

3.1 Knowledge of Existing Structures

During the first and second year of the research project, in situ sampling and testing were performed to determine the mechanical performance of old concrete and steel bars taken from existing multi-story RC structures built in the 20th century.
Analysis of the collected data was carried out during the third year. The materials' most representative in situ strength values and mechanical behaviors were recognized in combination to the structural main features such as age, use and importance of the considered buildings. Strength and ductility responses expected from different typologies of reinforced concrete columns found in existing buildings were investigated and compared.

More specifically, the objective of this research was the evaluation of the uncertainties in the mechanical properties of the construction materials and in the structural detailing or defects. The research was organized along four tasks.

(i) First, data about mechanical characteristics of reinforcing bars and concrete used between 1950 and 1980 were collected to create a database of cases where in situ inspection and laboratory tests are not available.
(ii) Second, a comprehensive survey of methods proposed in the literature for modeling the spatial correlation in structural modeling parameters was carried out.
(iii) Third, the Bayesian framework was employed for reliability assessment of existing RC structures for adaptive updating of both the structural reliability and the uncertainties based on the results of tests and inspections.
(iv) Fourth, a basis for the characterization of the uncertainties related to the construction details, attributed to the as-built conditions of the structure prior to tests and inspections, was carried out to identify and to characterize the most dominant types of construction defects which may be found in existing RC buildings.

It is worth recalling that RC buildings can require the determination of the in-situ concrete strength during the execution of new structures to investigate low-strength results from acceptance tests and, especially, in the capacity assessment of existing structures. Most structural codes and technical recommendations indicate that in situ concrete strength should be estimated by means of drilled cores (Destructive Test, DT), possibly supplemented by non-destructive tests (NDTs) (Malhotra et al. 2004). For this reason, a specific research activity was devoted to the estimation of the in-situ concrete strength by using destructive and non-destructive (NDT) methods in order to:

(i) investigate the effects of core drilling on the structural members; tests before and after a possible restoration were performed;
(ii) study the role of the main factors influencing the relationship between the "local" strength provided by core specimens and the in situ strength of the structural member as a whole;
(iii) analyze the role of damage possibly occurring during the core drilling on the concrete strength evaluated from compression tests on cores.

3.2 Assessment of Nonlinear Behavior of Buildings

The broad objective of this research work was to find correlations between intensity measures (IMs) and engineering demand parameters (EDPs) that describe the performance of RC new and existing structures, using a comprehensive set of ground-motion time histories for a variety of structures, modeled at different levels of complexity, for predicting different response quantities related both to local and overall structural and non-structural damage. In this context, the influence of the infills on the building response was also of interest. Moreover, as the response of acceleration-sensitive non-structural components in buildings is directly affected by the floor acceleration that they experience during the ground shaking, floor acceleration demands along the height of different RC frame buildings were also investigated. Prediction equations proposed by different seismic codes were also checked.

3.3 Influence of Infills on Structural Response

The research is based on theoretical/numerical analyses and experimental large-scale tests. The starting point was given by the results of the previous ReLuis research program and other available pertinent literature. The Italian NTC 2008 building code and Eurocode 8 contain the reference design provisions. The following were the main objectives:

(i) to deepen knowledge on the nonlinear hysteretic behavior of infilled RC frames under seismic actions;
(i) to provide designers and engineers with analytical methods and instruments for the safety evaluation of existing RC buildings;
(ii) to improve available computer programs for structural analysis by introducing more appropriate models to simulate the infill behavior;
(iii) to verify the new analysis models derived from this research by means of their applications to case studies referring to typical real buildings, and experimental tests of large scale specimens subjected to lateral loading that suitably simulate the seismic action.

On the whole, this research tends to mark a step forward in the interpretation of the behavior of the infills so that future technical codes can include explicit provisions that take into account the structural contribution of these "non-structural" elements.

3.4 Behavior and Strengthening of Beam-Column Joints

The research was organized on three basic branches:

(i) analysis of a wide database containing the results of experimental tests on 224 exterior RC beam-to-column joints carried out in the DPC-ReLuis 2008-2010 project;
(ii) execution and analysis of extensive experimental tests on joint specimens having different characteristics (e.g. poorly detailed beam column joints and joints with a beam depth equal that one of the surrounding slab);
(iii) experimental validation of strengthening techniques based on the comparison of relative performances and application limits.

3.5 Behavior and Strengthening of Columns and Beams

The research activity started with the analysis of the state of the art on the subject topic. Several methods and some simplified formulations were proposed to derive moment-curvature equations. For the sake of conciseness only a few cases will be reported: Pfrang et al. (1964) provided an analytic tool for analyzing RC sections. The proposed procedure, even though clearly summarized in few calculation steps, still requires computer programming to be applied. Carreira and Chu (1986) presented a general method to calculate moment-curvature relationship consisting of solving a nonlinear second degree equation, Kwak and Kim (2002), working on an analytical model for material nonlinear analysis of RC-beams, developed an approximate tri-linear moment-curvature law for RC beams, resorting to section

analysis and iterative methods to find the solution: the translation equilibrium condition is satisfied when the difference between tensile and compressive axial forces acting on the cross section is less than a given tolerance. Such relation was then improved (Kwak and Kim, 2010) by considering constant axial force effect, but still using an iterative approach. However, many additional methods are available from the published literature: Bresler (1960), Pannel (1963), Parme et al. (1966), Monti and Alessandri (2006), Hsu (1989), Hsu and Mirza (1973), Fafitis (2001) and Romao (2004).
The next step of the study consisted in the derivation of closed-form equations and in the evaluation of their accuracy through a comparison with the results obtained from fiber section analyses. An extended parametric study followed.

Regarding the effects of confinement, the following phases were followed:

(i) proposal of an analytical model capable of describing the constitutive law of confined concrete and longitudinal reinforcement, considering also the hardening behavior that can occur in the case of wrapping with carbon fiber sheets;
(ii) definition of ultimate moment-curvature domains of RC sections under uniaxial bending and assigned compression level;
(iii) application of a numerical procedure to link the ultimate bending moment and the curvature components under biaxial bending;
(iv) experimental validation of the theoretical approaches.

Another objective of this research was the evaluation of the flexure and shears response of existing RC elements for both as-built and retrofitted conditions. This part of the research was organized in four basic phases:

(i) an intensive numerical analysis campaign was carried out based on recent comparative studies on different shear capacity models, to propose a practice-oriented tool for the evaluation of shear-flexure hierarchy in existing RC members;
(ii) experimental data on deformation capacity of 32 RC columns with plain bars was collected. Provisions of ASCE/SEI 41 – Supplement 1 for the seismic assessment of older-type RC columns with plain bars were discussed for possible improvements aimed at increasing the predictive capability of current formulations;
(iii) biaxial bending tests on existing RC columns were carried out for comparison with current capacity models and to develop specific biaxial deformation capacity domains;
(iv) experimental tests on EFRP systems were performed and evaluated in order to validate the design formulations provided by the CNRDT200/2004 Guidelines for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures.

3.6 Local Strengthening Methods

The first part of the research was organized in two basic branches:

(i) experimental validation of the use of externally bonded FRP sheets (as external reinforcement on partially confined beam column RC joints) to increase their shear capacity;
(ii) theoretical study to quantify the benefits provided by FRP-based local strengthening interventions on existing RC structures.

The experimental program was carried out on six full scale poorly detailed T-shaped beam-column joints representative of typical existing structures. The study confirmed that the use of FRP may significantly improve the capacity of beam column joints, which are one of the main sources of vulnerability in existing RC structures.

A theoretical study aimed at assessing the structural capacity increase, obtained through FRP local strengthening interventions on nine existing RC school buildings in L'Aquila, was carried out. The capacity increase was computed based on the ratio between the maximum peak ground acceleration sustained by the structure at the life safety performance level, PGA_{CLV}, and the demand peak ground acceleration, PGA_{DLV}.
Moreover, a comparative study of the effectiveness of different methods to retrofit existing RC structures, with specific reference to school buildings designed in the absence of seismic criteria, was performed according to the NTC2008 retrofit criteria.

As for shear strengthening, the research carried out assessed the behavior of beams extracted from old RC existing buildings and retrofitted in shear with CFRP and in flexure with a RC slab, subjected to shear and negative bending moments. The effectiveness of the C-FRP strip anchorages on a cracked concrete surface due to negative moments was evaluated. The results obtained by experimental tests on retrofitted beams up to failure were also studied by detailed 3D FEM analysis by ATENA 3D, using material characteristics obtained from non-destructive and destructives tests, to better understand the inner and local failure mechanisms. The tests were used to evaluate the effectiveness of the reinforcement design tools and of the construction details of the intervention.

Finally, for the local strengthening of RC bridge piers, the influence of the material properties and construction details used for repair and seismic upgrade was clarified with the definition of intervention criteria also for RC columns. Hybrid tests (PSD) on RC piers (and bridges) repaired and reinforced was performed to assess the effectiveness of the proposed local repair and retrofitting technique. The in-house PSD program was calibrated and upgraded to allow the use of the OpenSees program for a more detailed model of the bridges. Numerical analyses using OpenSees fiber models helped understand the experimental results. Tests on materials used for the repair intervention (stainless steel rebars and SCC confined by C-FRP) were performed. The analytical models for concrete and steel were calibrated essentially using OpenSees to improve the results of the numerical analysis.

3.7 Global Retrofitting Methods

The research was organized in three basic branches:

(i) the life cycle costs associated with different rehabilitation schemes that may be utilized for existing RC framed buildings, with particular emphasis on the use of BRBs;
(ii) the study of the seismic vulnerability of tall buildings according to new Standards (NTC2008);
(iii) experimental tests and numerical simulations for gaining knowledge about methods for seismic upgrading and retrofit of existing buildings.

The first activity involved the study of an existing RC frame building built in the late 1930s characterized by a rectangular plan layout. The seismic response of the sample building was investigated through nonlinear time history analyses. Incremental dynamic analyses (IDAs) were used to compute the fragility curves of the case study structure. The modal response quantities were derived for both the as-built and the retrofitted structural systems.
The second activity was mainly devoted to the study of the seismic vulnerability of tall buildings according to new Standards, focusing on the influence of the foundation soil type. More specifically, the case study of a public building in the city of Naples was examined. The building is more than 100 m high, is founded on piles of large diameter, more than 40 m long, drilled in pyroclastic soils, and was designed according to codes and guidelines older than the

NTC 2008. The seismic response was studied via dynamic analysis performed by using both standard spectra and ad hoc spectra, obtained through the analysis of the seismic site response (SSR). For the characterization of the mechanical behavior of the foundation soils, several in-situ and laboratory tests were performed.

The third activity involved experimental tests and numerical simulations to study the effectiveness of different kinds of dissipative steel devices. Both low-cost devices, obtained from conventional steel profiles through ordinary steelwork procedures (Perri et al, 2013a), and produced by means of dedicated industrial processes (Perri et al, 2013b) were analyzed with the aim of describing their low-cycle fatigue response. On the other hand, the global response of RC concrete frames retrofitted by steel bracings was analyzed, with a focus on investigating the consequences of alternative patterns and distributions of bracings on the global response of the retrofitted structure (Faella et al., 2014).

3.8 Behavior and Strengthening of Industrial Structures

The objectives of the research are the mechanical characterization of the connections in precast systems and the development of proper seismic design criteria for precast structures taking into account the role of cladding wall panels. The research included experimental tests carried out on single connection devices and sub-assemblies, and numerical investigations based on nonlinear static and dynamic analyses with special attention paid to the seismic behavior of precast structures with cladding wall panels

The research program was subdivided into five sub-tasks:

- (i) classification of the types of connections of precast structures;
- (ii) numerical investigation on the influence of connections on the seismic behavior of precast structures;
- (iii) evaluation of forces transmitted through the connections for the most diffused precast building typologies;
- (iv) evaluation of the non-linear/cyclic behavior of specific connections;
- (v) definition of design guidelines for the seismic retrofit of existing precast structures.

4 MAIN RESULTS

4.1 Knowledge of Existing Structures

A significant number (thousands) of in-situ samplings and tests were performed - including rebound, ultrasonic and combined indirect tests - and hundreds of concrete samples were taken from existing structures and tested in the laboratories following the 2009 L'Aquila earthquake. Although the experimental campaign involved a large number of buildings, the number of tests wasn't always large enough to observe a significant correlation among the collected data. Results from direct and indirect tests on concrete were found to be well related only when referring to large groups of homogeneous data. Unfortunately, these circumstances are not common in practice (because of the reduced extension of the structures and the variability of other conditions). Moreover, the number of samples, suggested by the current Italian code to achieve given levels of knowledge for existing structures, was found to be largely insufficient to later correlate direct and indirect measurements on concrete.

Significant differences were recognized among mean values of the concrete compressive strength computed from direct tests on concrete samples taken from existing structures and mean strength values computed on the base of the original certificates archived by the Official

Testing Laboratory at the University of L'Aquila. Also, the quality of the steel used in reinforcing bars has also changed in past decades, both in terms of strength and ductility. Finally, the effects of materials change (both as result of durability issues and /or mistake in the construction process) on structural strength and ductility expected from RC elements was investigated. More specifically:

- due to concrete compressive strength reduction, the strength of RC elements under combined axial and bending actions reduces faster (and more than proportionally) in case of transverse cross section characterized by higher depth base ratios (walls rather than columns). This result is independent from the reinforcement steel grade.
- due to concrete compressive strength reduction, sectional ductility reduces faster when higher strength steel is used and is more evident in RC elements whose cross section is characterized by lower depth to base ratios (columns rather than walls).

Data on the mechanical characteristics of reinforcing bars used between 1950 and 2000 were collected in Naples too, using the archive of the steel testing laboratory of the Department of Structural Engineering at the University of Naples Federico II. Considered parameters included reinforcement typology, yield and ultimate strength and ultimate strain (Verderame et al., 2011). A software (STIL, Figure 1) was implemented, providing all information on the collected data through a graphical interface.

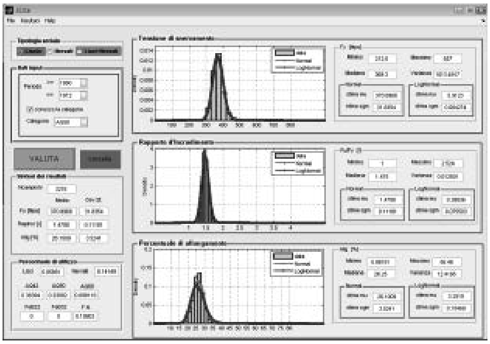


Figure 1. STIL software user interface.

A comprehensive survey of methods proposed in the literature for modeling the spatial correlation in structural modeling parameters was developed. In previous works, Franchin et al. (2010) employed a random field based on Nataf distribution for taking into account the spatial correlation of concrete strength in a structure. Alessandri et al. (2006) proposed a methodology for taking into account the spatial correlation in material properties, where the outcome of the rebar inspection in limited sections of the structure is used to predict the rebar quantity in the rest of the structure. Jalayer et al. (2010a) and (2011) used simplifying assumptions in order to characterize the spatial correlation of the structural modeling properties. Three correlation levels (namely, uncorrelated, systematic correlation throughout the floor, and systematic correlation throughout the structure) are used Depending on which correlation category fits in with the modeling parameters considered, they are divided into subgroups. The parameters belonging to the same subgroup can have a given correlation structure but are assumed to be uncorrelated with the modeling parameters outside the group. Another approach for taking into account the spatial correlation in structural modeling parameters uses the first order second-moment (FOSM) methods in order to link the structural reliability to the statistical properties of the structural modeling parameters (Baker and

Cornell, 2003). A comprehensive report on different methods for taking into account the spatial correlation in structural modeling parameters was prepared.

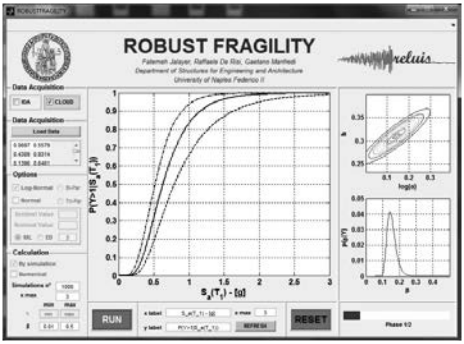


Figure 2. ROBUST FRAGILITY software interface.

Jalayer et al. (2013) developed a Bayesian methodology for efficient calculation of structural reliability taking into account various sources of uncertainty. This methodology - called the robust fragility method - is a fundamental tool for rapid calculation of structural reliability and the propagation of various sources of uncertainty both in structural modeling parameters and those related to the ground motion record-to-record variability. A software (ROBUST FRAGILITY, Figure 2) was implemented, providing the direct application of the proposed methodology.
Finally, a thorough study on a priori probability distributions was performed. This activity provided a basis for the characterization of the uncertainties related to the construction details attributed to the as-built conditions of the structure prior to tests and inspections. The basic idea was to identify and to characterize the most dominant types of construction defects which may be found in existing RC buildings. A survey for professional engineers was prepared in order to characterize these prior distributions in relation to expert opinion (Elefante, 2009). Two pilot surveys intended for professional structural engineers of Avellino and Naples were carried out through the Reluis website. The survey results were processed in order to come up with probability distributions that characterize the structural details (Elefante et al. 2011). A demo of the application was prepared for an existing structure in Avellino showing how prior probabilities based on expert opinion can be implemented in the reliability-based evaluation of seismic performance of an existing RC structure (Elefante et al., under preparation). The preparation of a comprehensive report on the characterization of the a priori probability distributions for uncertain structural modeling parameters and the eventual influence on structural reliability assessment is under way.

With regard to the determination of the concrete properties from core drilling, experimental test results on RC members extracted from typical Italian existing buildings, designed for gravity loads only and scheduled for demolition, were analysed. Tests on columns and on cores extracted from them were carried out. The effect of core drilling and of subsequent restoration on the strength of structural elements was evaluated by comparing performances of as-built columns, drilled columns and drilled-restored columns. Factors influencing the relationship between the local strength from core specimens and the in-situ strength of the structural member as a whole were highlighted.

The role of different factors affecting the estimation of in-situ concrete strength through destructive (core drilling) and non-destructive (rebound number, direct and surface ultrasonic velocity) tests was examined through an experimental program on a beam extracted from an existing RC structure. The experimental results show a low variability of rebound numbers and direct velocity values along the beam, while a high variability was detected for both surface velocity values and particularly drilled core strengths. Also, the results confirm the need to carefully locate measurement points within the structural members. They have to be placed in zones without apparent damage and/or cracking, where stresses due to applied loads are the lowest in the element, and representative of the average conditions of the concrete taking into account casting and ageing effects.

Finally, the role of damage, possibly occurring on cores during the drilling phase, in the estimation of the in-situ concrete strength through compression tests on cores was analysed. Two wide datasets were examined, including: (i) about 500 core specimens extracted from RC existing structures, and (ii) about 600 cube specimens taken during the construction of new structures in the framework of routine acceptance controls. The two experimental datasets were compared in terms of compression strength and specific weight values, accounting for the main factors affecting the concrete properties, that is type and amount of cement, aggregates' grading, type and maximum size of aggregates, water/cement ratio, placing and curing processes, concrete age. The results show that the strength reduction due to drilling damage is strongly affected by the actual properties of the concrete, and is inversely proportional to its strength. Therefore, the application of a single value of the correction coefficient, as generally suggested in the technical literature and in structural codes, appears inappropriate. A set of values is suggested from the drilling damage coefficient as a function of the strength obtained from compressive tests on cores.

4.2 Assessment of Nonlinear Behavior of Buildings

The research plan initially considered the selection of RC frames representative of a large class of existing buildings, in terms of configuration in plan and elevation and resistance to seismic actions. Secondly, two- and three-dimensional models of a sample of RC buildings were subjected to a significant set of strong motion records. Nonlinear analyses were performed on frames modeled with different levels of detailing. Irregular buildings subjected to bi-directional ground motions were also considered. Suitable measures of seismic intensity, expressed in terms of displacements, forces, and energy, within the two- and three-dimensional analysis to be used in correlation with parameters describing the local and global structural response, were investigated. Floor acceleration demand, expressed also in terms of amplification factors, was assessed in order to identify the most meaningful intensity measures influencing the floor response.

The RC buildings designed according to old Italian seismic codes were modeled in the OpenSees software. First, models with regular stiffness and mass distributions in plan and elevation were considered, with different infill wall configurations and designed for different base shear values. The research focused on the performance of various Intensity Measures (IMs) in terms of efficiency and sufficiency for predicting the structural seismic response (expressed in terms of DPS, Engineering Demand Parameters) of fixed base buildings in the framework of PBEE. Two concrete frame buildings, 4-story and 6-story high (Figure 3), were investigated in the study cases.

Three EDPs, namely Maximum Roof Drift Ratio (MRDR), Maximum Inter-storey Drift Ratio (MIDR) and Maximum Floor Acceleration (MFA), were considered. The ground motions were divided into two sets, ordinary ground motions and near-fault pulse-like ground motions,

in order to observe the effects of pulse-like records on the properties (i.e., efficiency and sufficiency) of the IMs.
The investigated IMs are categorized into two groups: 14 non-structure-specific IMs - calculated directly from ground motion time histories - and 13 structure-specific IMs - obtained from response spectra of ground motion time histories depending on the period of the structure. The second set is further sorted into two sup-sets: IMs obtained from the response spectral ordinate at certain periods and from integration of response spectra over a defined period range. More specifically, IMs evaluated by integration of the structural response at a given period range can explicitly account for higher modes effects as well as period lengthening due to structural softening.
The IMs considered in this study, which are classified as the second group of investigated IMs, are: ASI (Acceleration Spectrum Intensity), VSI (Velocity Spectrum Intensity), IH (Housner Intensity), VEIrSI, and VEIaSI (relative and absolute Input Equivalent Velocity Spectrum Intensity, respectively). VEIrSI, and VEIaSI are parameters obtained from integration of the energy response spectra in the period range 0.1-3.0 sec. Modified versions of the second group of structure-specific IMs were also considered. The modified IMs were obtained from the existing ones by changing the period range of integration into 0.2T-1.5T, where T is the fundamental period of the fixed base structures.

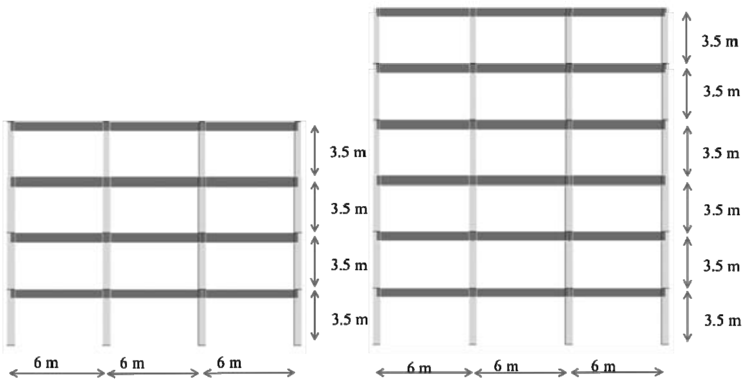


Figure 3. Schematic representation of the frame structures analyzed.

An important finding is that the proposed modified intensity measures are always more efficient than the corresponding spectrum intensity measures in predicting each considered EDP. The reason for this could be due to the fact that the integral period range of the proposed modified intensity is dependent on the structural fundamental period, which could be more representative of the influence of the higher-mode vibrations and the inelastic effects with respect to the spectrum intensity measures with fixed integral period range.
The results from the study on the floor response spectra shed some additional light on the effects of post-elastic non-linear behavior of the supporting building on Peak Floor Accelerations (PFA) and Floor acceleration Response Spectra (FRS). The floor acceleration response was investigated for strong beam-weak column old vintage RC frames, with and without infill walls, using a large set of far-field ground motion recordings. The findings of this study were compared with those of others and with the prescriptions of codes. Eurocode 8, the New Zealand Standard 1170.5, and the FEMA P-750 documents, which represent the

state-of-the-art in seismic design codes and guidelines, predict PFA profiles and FRS ordinates that are somewhat inconsistent with the findings of the present study. These documents do not seem to fully consider the influence on PFA and FRS caused by the level and type of damage experienced by the supporting building. In general, these documents prescribe PFA values at the roof level that are overestimated and peaks of the FRS that are underestimated. It is important to point out that these conclusions are not general but hold for structures similar to those considered in the present study.

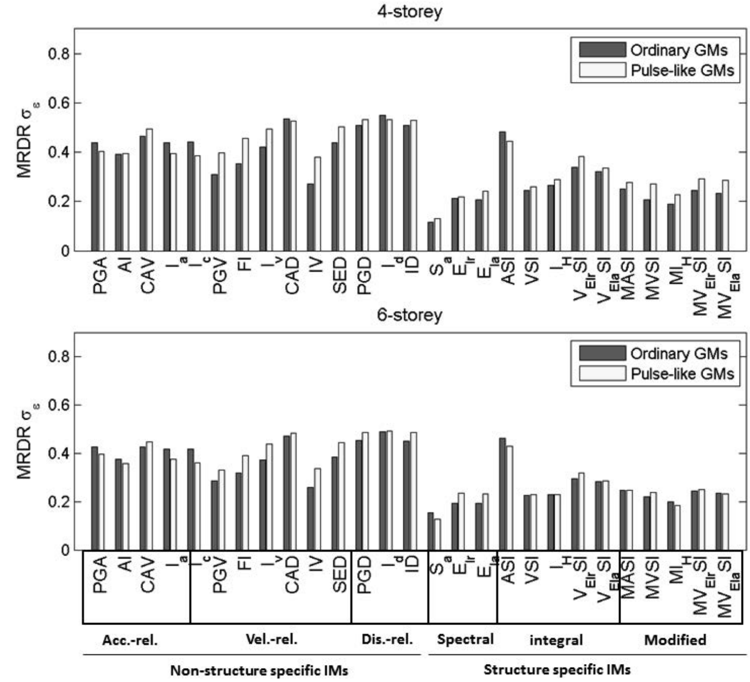


Figure 4. Schematic representation of the frame structures analyzed standard error of residuals σ_ε obtained in the ln(MRDR)|ln(IMs) regression of 4-story frame (top panel) and 6-story frame (bottom panel) subjected to ordinary ground motions and near-fault pulse-like ground motions.

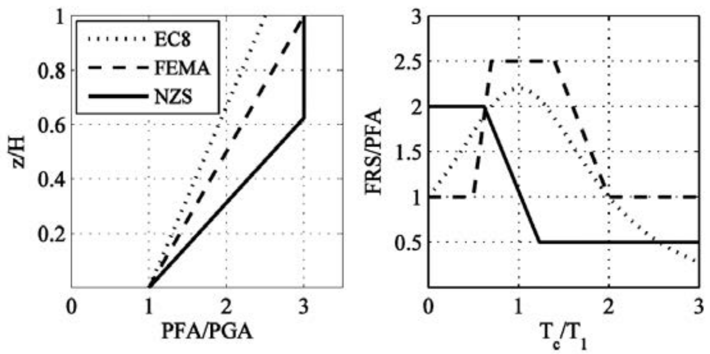


Figure 5. Comparison of estimates of PFA (normalized by PGA) and floor response spectra (normalized by PFA) at the roof level of the 6b frame obtained using the three codes.

4.3 Influence of Infills on Structural Response

The main results were presented in about forty scientific papers, many of which published in international journals. In the following brief summary some of them are cited for each research theme.

Modeling of infill panels.

i) A report collecting all main contributions concerning the different micro- and macro-infill models was produced.

ii) By using the classical single- or multi-strut formulations, sensitivity analyses were carried out considering 2D systems (frame-infill in their plane) and 3D systems (building with infilled frames) with respect to the geometrical and mechanical parameters characterizing the simplified model. The results were obtained by nonlinear static analyses.

iii) A simplified method, based on a mesoscale approach, was proposed, which derives the conventional constitutive law of the diagonal strut equivalent to the infill from a more refined model.

iv) The Dowell et al. (1998) model (for concrete), which is available in many software libraries, was calibrated to reproduce by means of an equivalent strut, the nonlinear hysteretic behavior of an infill panel made of calcarenite ashlars, clay bricks or lightweight concrete blocks, experimentally derived during the previous ReLuis research.

The theoretical/numerical approaches used in this study highlighted the role of the main parameters governing the infill response to lateral actions, suggesting also the analytical or experimental strategy to calibrate them.

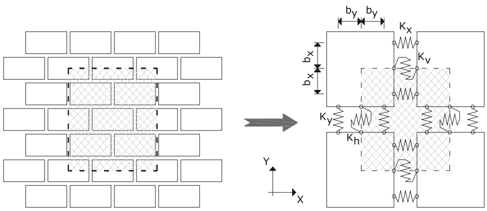


Figure 6. A mesoscale approah for infills with regular brick texture.

Effects of infills.

(i) An experimental campaign was carried out on one-story one-bay infilled frames (scale 1:2) designed for gravity loads (GLD) or according to recent seismic codes (SLD), in order to evaluate the influence of the infills on the global response and on the local behavior, depending on the structural characteristics of the surrounding RC frame. Infills had a 80 mm thickness, and were made of hollow clay bricks and typical mortar used for Italian non-structural masonry. Pseudo static tests were carried out with imposed cycles of increasing lateral displacements. The experimental results showed that the post-elastic behavior of GLD specimens was controlled by brittle failure mechanisms: shear failure at the top of the columns was observed, due to local interaction with the infill panel. By contrast, SLD frames showed the expected ductile behavior, controlled by flexure mechanisms. In both cases the significant contribution of the infills to the initial stiffness and maximum strength was observed.

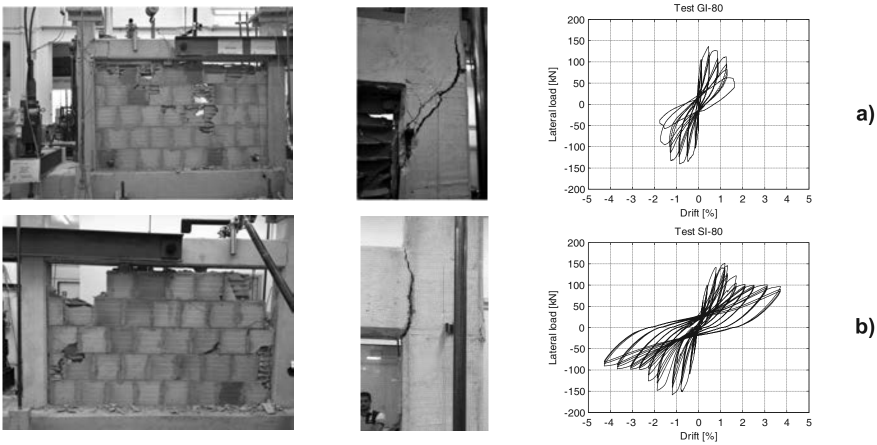


Figure 7. Global and local damage and cyclic lateral response of infilled frames: a) Gravity Load Designed frames; b) Seismic Load Designed frames.

Figure 8. Pushover analysis of building considering bare or infilled RC frames: a case study.

(ii) A large-scale damage scenario for RC buildings during the 2012 Emilia (Italy) earthquake was evaluated taking the infill contributions into account. Building stock characteristics and historical seismic classification were employed for the definition of two benchmark structures, representative of the whole building stock. Damage States were defined according to a mechanical interpretation of the EMS-98 scale, and the damage scenarios obtained appeared to be in agreement with the observed ones.

Figure 9. Pushover analysis of building considering bare or infilled RC frames: a case study. Damage scenario for Emilia region based on census data, and examples of damage observed during post-earthquake field campaigns.

(iii) A simplified method for seismic vulnerability assessment was developed, considering the main characteristics of RC buildings, such as number of stories, global dimensions, type of design, infill presence, and on the assumption of a shear type behavior to evaluate in closed form the nonlinear static response of a building. The seismic capacity was evaluated within the framework of the N2 method. This approach was used to carry out a large scale multilevel seismic vulnerability assessment on RC buildings in the city of Avellino. The results provide a useful procedure for similar analyses.

(iv) Effects of irregular in-plane infill distributions and frame-infill interactions were analyzed through several case studies. The results confirmed the expected behavior and the experimental evidence.

Seismic upgrading.
Two different techniques were considered: a connection of the infills to the surrounding RC frame by means of effective “links” along the top and lateral edges or by the application of plaster reinforced with a regular steel wire mesh. Insertion of dissipative diagonal braces derived by displacement base design, considering both the Damage Limitation Limit State and the Ultimate Limit State. After numerically simulating the use of both these techniques for several case studies, the main conclusion is that the first type of intervention proves to be very efficient under earthquake of moderate intensity. The behavior of the infilled structure tends to become that of a wall, with a appreciable expected damage reduction. The use of dissipative braces is very efficient against the ultimate limit state. For this technique, a simplified design procedure was also proposed for a preliminary evaluation of the characteristics required for the braces.

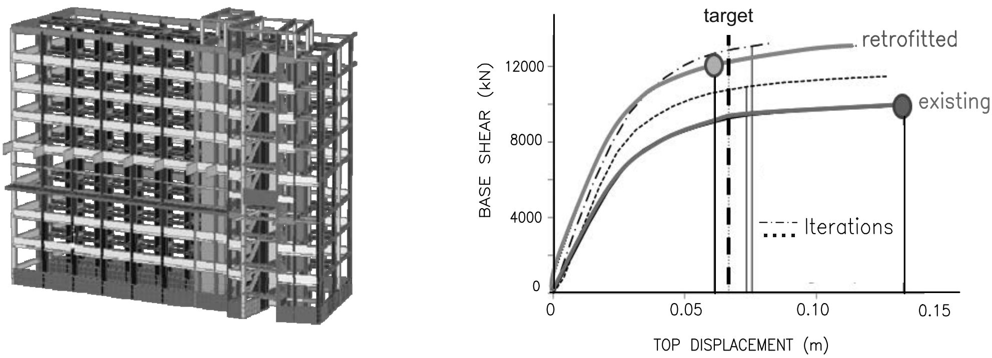


Figure 10. Seismic upgrading by using dissipative braces: a case study.

A further study was conducted by considering the same classes of existing buildings as those used in the study on the Assessment of Nonlinear Behavior of Buildings, using two-and three-dimensional models, where the presence of the infill panels was also considered in evaluating the dynamic response of RC frames and in the correlation analyses between IMs and EDPs.
In order to consider the modified seismic response of frame structures in the presence of masonry infills, proper models must be used. Considering the complexity and the computational feasibility of the problem, an improved hysteretic infill model based on the equivalent strut approach for in-plane and out-of-plane behavior, were employed. It uses the energy-based deterioration law for strength and stiffness degradation in both the in-plane and out-of-plane directions. The out-of-plane hysteretic deterioration can also be taken into account. The proposed strut approach has the advantage of modeling the cyclic degradation

caused by the following four deterioration modes: basic strength deterioration, post-capping strength deterioration, unloading stiffness deterioration, and accelerated reloading stiffness deterioration. It considers the effects of in-plane and out-of-plane strength interaction, together with the influence of prior in-plane damage on out-of-plane behavior. An algorithm that introduces the progressive collapse of the infill was introduced.

A straightforward way to implement this model into Opensees by simply modifying existing beam-column type elements was partially developed. The cyclic in-plane behavior is implemented in the axial behavior of the new element, and the out-of-plane behavior was implemented in the rotation-bending relationship for the node corresponding to the panel mid-node. The adequacy of the model in predicting the seismic response of single/multi-story infilled RC frames was verified through comparisons against experimental results.

Moreover, the effects of the openings on the seismic response of infilled frames was studied and the main parameters involved were identified. The presence of openings in the infills, which leads to significant uncertainty due to the variability of sizes and position of the openings, results in a reduction of stiffness, strength and energy dissipation capacity of the panel.

A simple model that takes into account the openings in the infills was proposed and compared with other models proposed by different researchers. The model is based on the use of a reduction factor to be applied to the single strut model parameters. The reduction factor, which depends on the window size and of the presence of reinforcing elements around the openings, allows the adequate reduction of both lateral stiffness and strength of the frame-infill system due to the presence of openings.

The cyclic model reproduces fairly well the strength, stiffness and energy dissipation capacity degradation due to the presence of openings and represents some characteristic of the observed behavior, such as the force degradation at constant displacement amplitude, the small influence of the infill strength on the energy dissipation and the fact that the influence of the openings is less important in the case of strong infill.

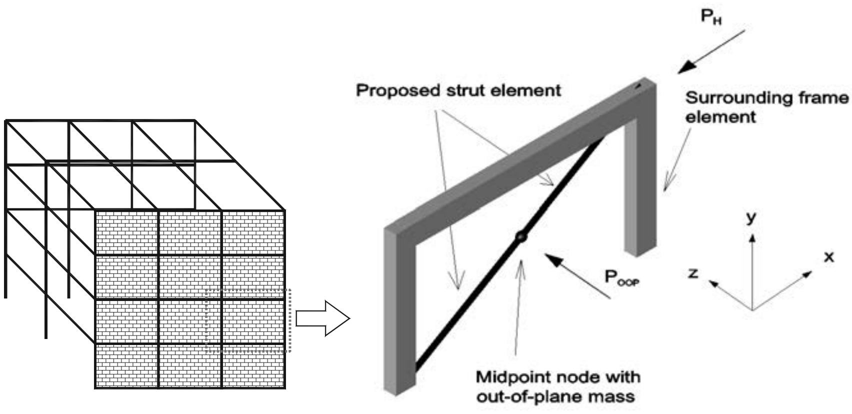


Figure 11. Proposed infill wall model.

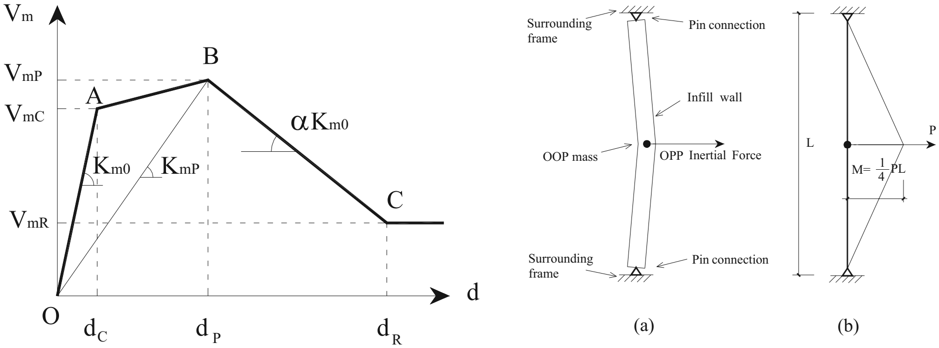


Figure 12. (Left) In-plane lateral force-displacement envelope relationship of the proposed model; (right) the working state of the proposed model in the out-of-plane direction.

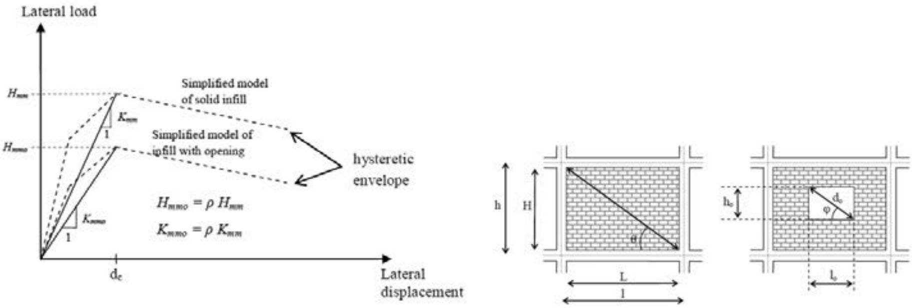


Figure 13. Schematic representation of the infill lateral load-displacement envelope.

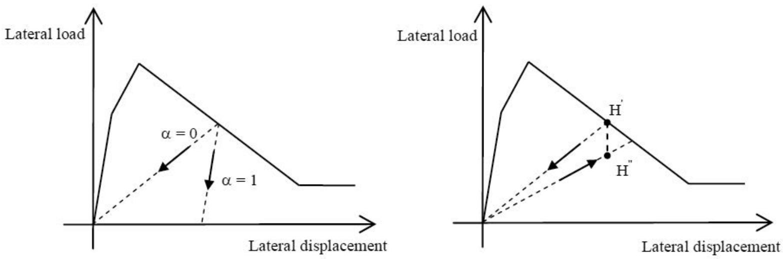


Figure 14. Strut model hysteretic rules.

4.4 Behavior and Strengthening of Beam-Column Joints

The results collected from a wide database were employed to assess the accuracy of a series of theoretical models available in the scientific literature to predict the shear capacity of beam-column joints. Since all models are (partly or fully) based on empirical considerations, their predictive capacity is supposed to be significantly affected by the actual range of variation of the key geometric and mechanical parameters used for the models' calibration. Therefore, with the aim of assessing the accuracy of these models on relevant joint types, the experimental database was partitioned by considering the three following categories:

- EC8-compliant joints;
- under-reinforced joints (namely, joints reinforced by fewer internal stirrups than required by EC8 provisions);
- unreinforced joints.

Figure 15 shows some of the comparisons between experimental and theoretical results. The plots report the experimental value $V_{jh,exp}$ on the x-axis and the corresponding theoretical prediction V_{jh} on the y-axis. They show the general trend of each model in either overestimating or underestimating the actual values of shear capacity obtained from the experimental tests. Moreover, the performance of each model for the three aforementioned categories of joints can be easily understood by analyzing these graphs.

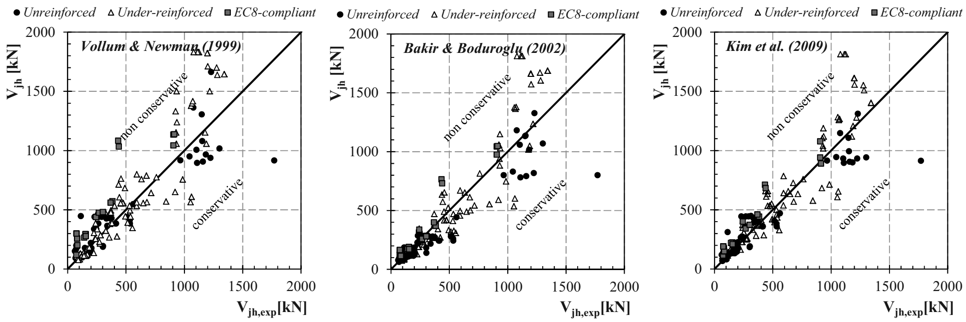


Figure 15. Assessment of the capacity models.

Two experimental programs were carried out on poorly detailed specimens and beam-column joints having the beam depth equal to that one of the surrounding slab.
In the first experimental program a total number of six subassemblies were designed to reproduce subassemblies with weak column and strong beam and to achieve shear failure in the joint panel before yielding of both beam and column reinforcements under simulated seismic actions. Three specimens were tested in the as-built configuration, and three specimens were strengthened to investigate the benefits provided by different FRP layouts. The main experimental results provided by tests on poorly detailed as-built specimens show that the comparison between several strength capacity models available today and the experimental results indicate an absolute maximum scatter of 10%. The smallest difference was obtained by using the principle stress approach suggested by Priestley, The use of the shear at first cracking to compute the joint shear capacity, as suggested by EN 1998-1, provided very conservative predictions (i.e. 30% lower than experimental maximum strength capacity). The accuracy of the joint strength capacity expressions provided by ASCE/SEI 41-06 and AIJ 1999 strongly depends on the proper evaluation of the confinement benefits given by transverse beams on exterior joints. Suitable capacity models of the joint shear deformability should be specifically calibrated for poorly detailed beam-columns joints. Existing prediction models, mainly based on a limited number of tests related to joints and conforming to current seismic codes, may lead to significant underestimation of the joint shear stress and strain.
A second experimental program was carried out on several wide beam-column joints having the beam depth equal to that of the surrounding slab; the tests were performed on specimens with different Earthquake Resistant Design (ERD), that is both specimens designed only to

gravity loads (NE) and specimens designed with seismic criteria. As for the latter specimens, some were designed for very low seismicity (Z4, PGA=0.05g) and some for moderate seismicity (Z2, PGA=0.25g). Moreover, a test on a beam-column joint with stiff beam – first damaged and later repaired with the DIS-CAM system – was performed in order to check the effectiveness of this strengthening technique (Dolce et al. 2006).
First of all, it was possible to outline the influence of the ERD level on the joints' behavior. The joints with higher ERD level suffered a heavier damage to the beam-column intersection with faster strength degradation. This can be mainly ascribed to the larger amount of beam reinforcement outside the column width, which is poorly confined without the positive contribution of the axial load. This reinforcement induced a higher level of tensile principal stresses not mitigated by the axial load, thus triggering early cracking. However, in accordance with theoretical considerations, joints with higher ERD level showed higher performances in terms of strength. On the other hand, the comparison between the Z4 joints tested with different levels of axial load (0.15 and 0.30 of the ultimate axial load) showed quite different behaviors. The joint tested under the larger axial load delayed the occurrence of damage to the beam-column intersection with respect to the joint tested with the lower axial load. This, in turn, allowed the first one to have lower strength degradation in the post-peak phase of the force-drift envelope, resulting in a larger deformation capacity. In fact, the joint tested with higher axial load, despite reaching the same horizontal force values, showed a deformation capacity increase of about 20% compared to the other joint. This is an unexpected result, given that most of the damage occurred in the beam-column intersection outside of the column and, thus, not subjected to the axial load confining effect. Further analyses will be necessary to better interpret this result.

4.5 Behavior and Strengthening of Columns and Beams

The main result of this task was the derivation of closed-form equations for yield and ultimate moment and curvature formulas for RC sections subjected to uniaxial bending moment and axial force.
The moment-curvature relationship was obtained by studying two strain profiles: at yield of the tensile bars and at crushing of the compressive concrete, and by imposing equilibrium. They represent, respectively, the yield and ultimate point of the moment-curvature law.
The simplest yet sufficiently accurate moment-curvature relationship for RC sections is described by a bilinear law, with the fist branch going from the unloaded condition up to the point corresponding to tension steel yielding (yield point), and the second one going from this point to that corresponding to compressive concrete crushing (ultimate point). The latter condition occurs if one assumes an indefinitely plastic law for steel (EN 1992-1-1), see Figure 16 and Figure 17. The equations are summarized in Table 1.

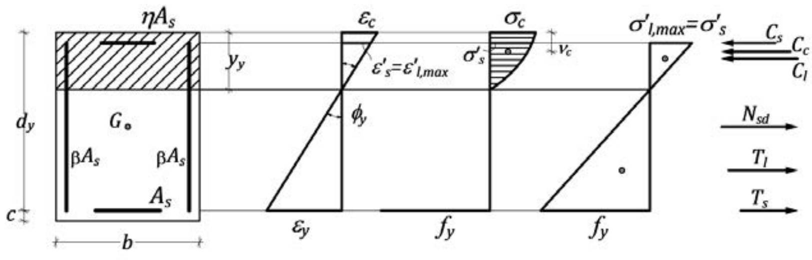


Figure 16. Cross section: strain and stress profiles and resultants at yielding of tensile steel.

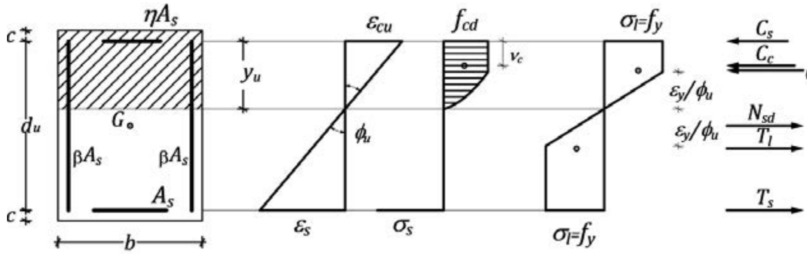


Figure 17. Cross section: strain and stress profiles and resultants at ultimate.

Table 1. Summary of closed-form equations.

Yield Curvature	$\phi_y = \frac{\varepsilon_{yd}}{d_y} \frac{1}{1-\xi_y}$
Yield Moment	$m_{yd} = \frac{1}{2}\left\{ \mu_{sy}\left(1+\eta\frac{\xi_y}{1-\xi_y}\right) + \bar{\varepsilon}^2 \xi_y \left(\frac{\xi_y}{1-\xi_y}\right)^2 \left(1-2\nu_c \xi_y\right) + \frac{1}{3}\beta\mu_{sy}\left(\frac{1}{1-\xi_y}\right) \right\}$
Ultimate Curvature	$\phi_u = \frac{\varepsilon_{cu}}{d_u} \frac{0.8+4\beta\mu_{su}}{n_{Sdu}+\mu_{su,tot}}$
Ultimate Moment	$m_{Rd} = \frac{1}{2}\left\{ \begin{array}{l} n_{Sdu} + 2\mu_{su}\left(1+\beta\right) + \\ -\left[0.8^2 + 4\beta\mu_{su}\left(1+\frac{1}{3}\bar{\varepsilon}^2\right)\right]\left(\frac{n_{Sdu}+\mu_{su,tot}}{0.8+4\beta\mu_{su}}\right)^2 \end{array} \right\}$

The results of some of the parametric analyses, with the comparison between the fiber section analysis (dotted lines) predictions and the proposed closed-form equations (solid lines) are shown in the following figures: **Figure 18** and **Figure 19** show how yield curvature and moment, and ultimate curvature and moment, respectively, change for different axial force levels. Figure 20 and Figure 21 depict moment-curvature relationships for different combinations of the four considered variables and Figure 22 shows the estimate of ductility.

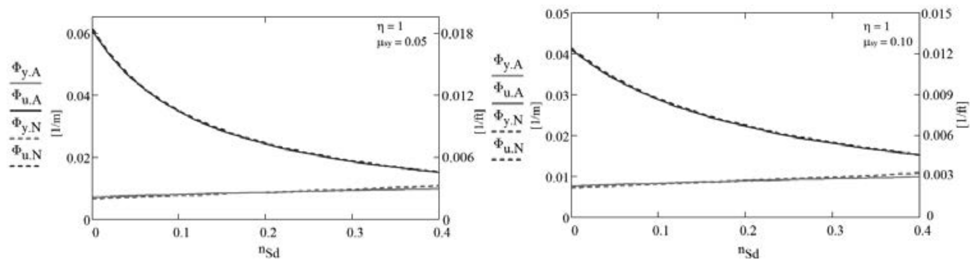


Figure 18. Yield and ultimate curvature: fiber section analysis (dotted lines) and proposed closed-form equations (solid line).

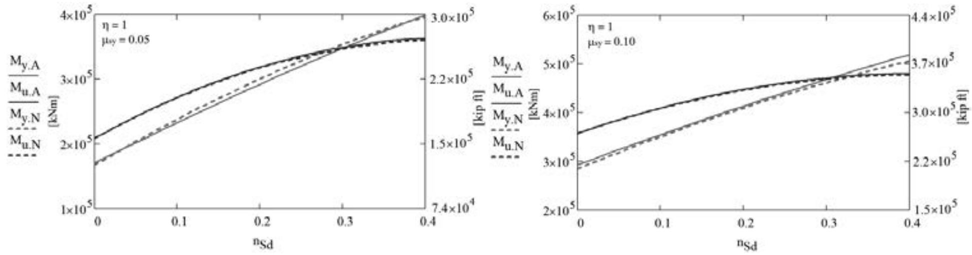


Figure 19. Yield and ultimate moment: fiber section analysis (dotted lines) and proposed closed-form equations (solid line).

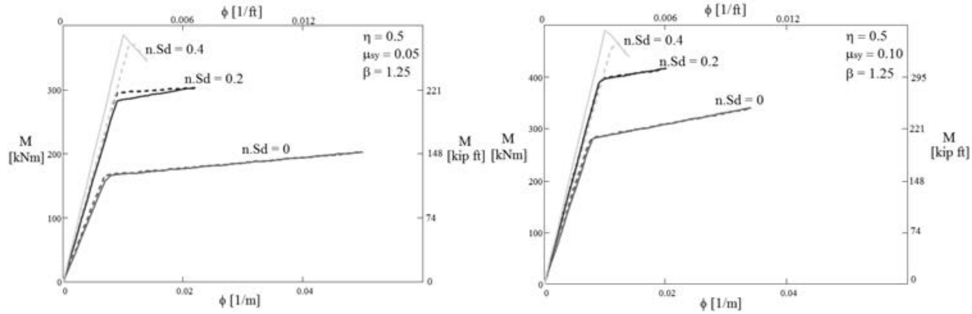


Figure 20. Moment-curvature relationship: fiber section analysis (dotted lines) and proposed closed-form equations (solid line) for three different values of normalized axial force and η=0.5.

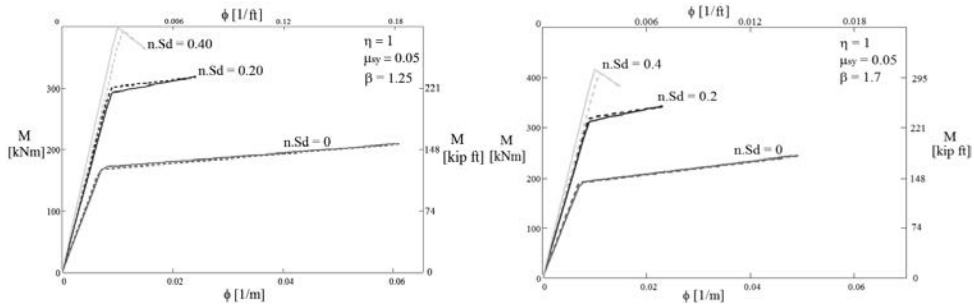


Figure 21. Moment-curvature relationship: fiber section analysis (dotted lines) and proposed closed-form equations (solid line) for three different values of normalized axial force and η=1 and μ_{sy}=0.05.

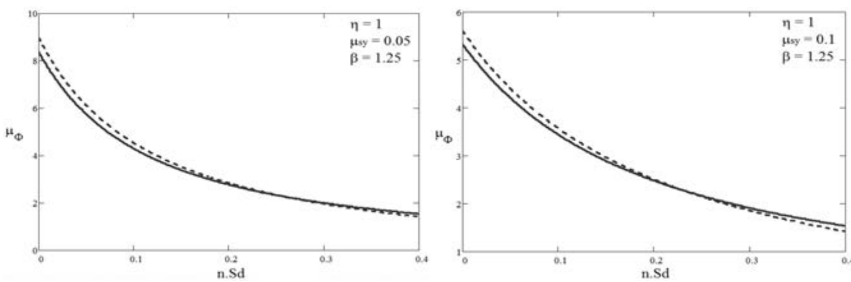


Figure 22. Estimate of ductility: fiber section analysis (dotted lines) and proposed closed-form equations (solid line).

Regarding the effects of confinement, with reference to RC columns having rectangular cross-section, the following main results were obtained:

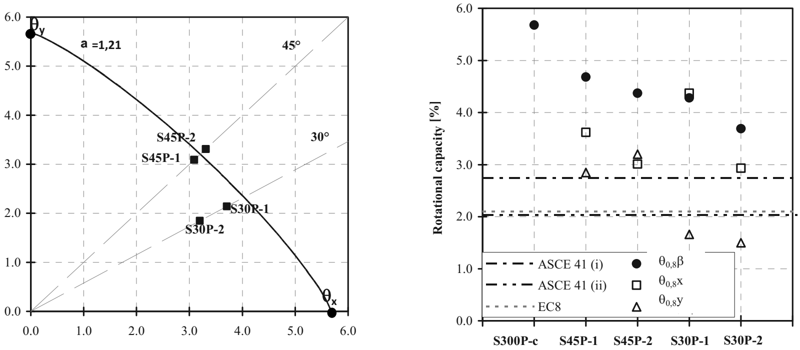
- a dimensionless analytical procedure was proposed to define the strength and curvature domains;
- sixteen 200x300x1300 mm RC columns, manufactured with normal or low-strength concrete, were subjected to uniform or eccentric compression, without or with external cage in order to evaluate the improved performance produced by this confinement technique. The confinement effects of FRP/FRCM sheets was derived by tests of uniform monotonic or cyclic compression on thirty-two prismatic concrete specimens 600 mm height, with circular, square or rectangular cross-section. The results of this experimental investigation allowed to calibrate suitable parameters that govern a generalized stress-strain model for confined concrete.

The analytical formulations related to both confinement techniques can be used for approximate evaluations of the improved available ductility and strength due to their use. Further experimental results and a comparative analysis of all the results available in the published literature would be necessary in order to propose code requirements.

Regarding the effects of EFRP strengthening, an intensive numerical analysis campaign was carried out based on recent comparative studies on different shear capacity models, employed as basic references for this subtask. A simplified tool for the fast evaluation of the occurrence of brittle failures in existing RC elements was provided considering different shear capacity formulations (De Luca et al. 2013). This tool requires only some basic information about the elements, which can be easily obtained by relevant practice or incomplete construction drawings.

In a parallel study, experimental data on the deformation capacity of 32 RC columns with plain bars were collected. Provisions of ASCE/SEI 41 – Supplement 1 for the seismic assessment of older-type RC columns with plain bars were discussed. The applicability of ASCE/SEI 41 models to the deformation capacity prediction of older RC columns with plain bars was evaluated, and possible improvements were proposed. Plastic deformation capacity predicted by ASCE/SEI 41 – Supplement 1 could be increased, on average, by 40% to fit the target safety level selected by code, or by 16% to fit the same safety level actually observed on columns with deformed bars. An overestimate of effective stiffness for columns with axial load ratio lower than 0.50, consistent with the indications provided by ASCE/SEI 41 – Supplement 1, was observed, and possible improvements were then proposed aimed at increasing the predictive capability of the effective stiffness model, particularly for columns with low axial load ratios (see Di Ludovico et al. 2013a).

In order to analyze the effects of biaxial bending on the seismic performance of existing RC columns with design characteristics non-conforming to present day seismic codes and practices, the activities also focused on experimental tests on full scale RC columns under both axial load and uniaxial or biaxial bending. Four tests were carried out on full scale square columns reinforced with plain bars, subjected to constant axial load and biaxial cyclic actions. Two different cyclic displacement paths were investigated: horizontal displacements with an inclination angle of 45° or 30° with respect to the cross section principal axes. The influence of biaxial bending actions on the global behavior of existing RC columns in terms of stiffness, strength, and deformation capacity as well as the effects of different horizontal displacements orientation on their performances were analyzed. The results were compared with experimental outcomes provided by uniaxial bending tests on companion specimens (see Figure 23). Experimental findings showed that biaxial bending actions affect columns rotational capacity more than strength. The rotational capacity reduction owing to biaxial bending actions, even if more significant than the flexural capacity reduction, is not currently taken into account in seismic guidelines related to existing RC buildings (Di Ludovico et al. 2013b).



(a) (b)

Figure 23. Rotational capacity: (a) best fitting of experimental points; (b) comparison between experimental values and guidelines predictions (Di Ludovico et al. 2013b).

Another research activity was carried out with the aim to validate the design formulations provided by CNRDT200/2004 Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures. The end-debonding phenomenon, which strongly influences the FRP shear reinforcement efficiency, was examined. The phenomenon was studied by experimental investigations (monotonic and cyclic bond tests on concrete elements reinforced with CFRP plates and sheets). A comprehensive database based on experimental data (more than 400 bond tests) reported in the literature (e.g., Nigro et al., 2011) was also drawn up and used to calibrate design formulations according to the procedure provided by Eurocode 0 (EN1990, 2002) for the "Design by testing", see Figure 24.
Enhancements to CNR-DT200/2004 were proposed. The updated code is available at http://www.cnr.it/sitocnr/IlCNR/Attivita/NormazioneeCertificazione/DT200_R1_2012.html.

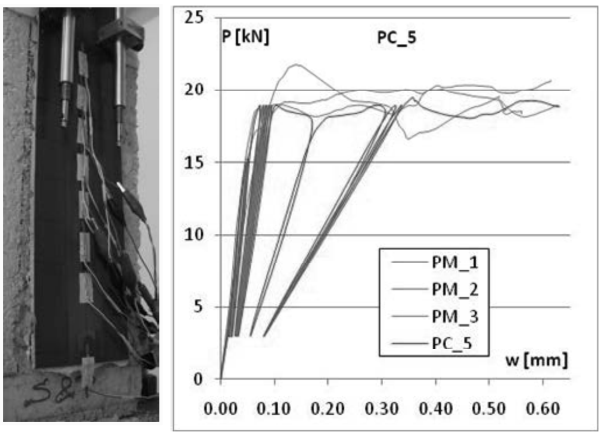


Figure 24. Experimental results of cyclic bond tests on EFRP RC elements (Nigro et al., 2011).

4.6 *Local Strengthening Methods*

In order to quantify the benefits provided by local strengthening solutions on partially confined beam column RC joints, theoretical analyses were performed on six school buildings to determine their deficiencies and the safety increase provided by FRP strengthening solutions. The analyses confirmed that the use of FRP based local strengthening interventions significantly increase the global structural seismic capacity. It has been shown that in most cases the local strengthening of exterior joints may increase the safety index level up to achieving about 60% (that is the minimum requested on existing buildings according to Italian Ordinances 3790 and Ordinance 3907 after L'Aquila earthquake) or higher. In some of the case studies, a safety index of 60% can be attained by simply increasing the shear capacity of a few columns. In most cases, the structure displacement capacity was the governing failure mode. It should be noted that the increase of the beam column joint shear capacity against the local effects of strong infills, along with interventions to avoid out-of-plane collapse of masonry infills, may be strongly required. This results in a supplemental lateral strength of the building that is not accounted for in the analyses and may considerably increase the seismic capacity of the buildings.

Based on the results of the vulnerability analysis aimed at the assessment of the effectiveness of different retrofitting methods, carbon FRP wrapping of columns or bracing through RC walls were also analyzed based on the safety levels set by current codes (NTC 2008). The results highlighted that an analysis of the foundation structure is always required due to demand increases on the foundations. However, the retrofit strategies based on the increase in capacity, may require only minimal interventions in the foundation, whereas retrofit strategies with bracing structures require foundations retrofit due to the increase and redistribution of forces. The cost analysis, with reference to cost of demolition and reconstruction of finishes, structure retrofit intervention and foundation retrofit intervention, showed that the total cost of the FRP retrofit depends almost exclusively on the material cost, while intervention with RC walls is strongly affected by the foundations' retrofit.

The C-FRP shear strengthening effectiveness of RC members was also studied through testing. The elaboration of experimental data related to retrofitted beams and the beams materials extracted at the end of beam failure tests, allowed a better interpretation of the experimental results thanks to more detailed 3D FEM analysis by ATENA 3D. The code formulations for the retrofitted beam shear strength evaluation were analyzed considering the results of the tests on materials and beams. The negative effect of construction defects was evaluated: the lack of connectors and longitudinal steel reinforcement along the entire RC slab reduced the shear strength of the beams as flexural beam end collapse happened. Test results allowed revisions of code prescriptions concerning C-FRP shear reinforcement details in the presence of negative moments.

Finally, for the strengthening of RC bridge piers, a computer code was developed in LabVIEW. This program is easily interfaced with OpenSees for a better numerical modeling of the tested structure for PSD test. The program can take into account asynchronous seismic input, important for extended structures such as bridges, but synchronous application was carried out. The PSD tests on the repaired and retrofitted bridge show that the proposed repair and retrofitting intervention is effective. The original pier strengthened was recovered without the original shear failure mechanism. However, construction details concerning rebar connection systems should be improved. Experimental results have been interpreted using refinements of concrete and steel models.

4.7 Global Retrofitting Methods

The first activity allowed the estimation of the maximum interstory drifts both for the as-built and the retrofitted structural schemes. The comparisons between results demonstrate that the lateral displacements are significantly lowered after the retrofitting intervention on the existing structure. Seismic fragility analyses were carried out using the multiple-stripe analysis method (Jalayer, 2003). At the same acceleration level, the retrofitted structure has a 68% and a 24% probability of exceedance the damage measure of 0.5% and 2.0%, respectively. Therefore, the effectiveness of the retrofitting strategy was shown both under moderate and high magnitude earthquakes.

The results obtained with the case study on the public tall building confirmed that if the building is based on poor quality soils the new code requirements may not be respected. Nevertheless, the definition of the design seismic actions based on the analysis of the seismic site response (SSR) allow the retrofitting strategy to be better calibrated. Indeed, the accurate reconstruction of the subsurface model allowed identification of an important and favorable effect of a relatively shallow layer of peat on the ground shear wave velocity profile. The RC members that are actually most vulnerable were finally identified through the structural analysis performed with the ad hoc seismic response. Therefore, the reinforcement technique has been directed towards a local reinforcement.

As for the use of dissipative steel devices, the cyclic response of steel devices easily obtained by conventional steel profiles through simple steelwork operations, such as cutting and welding was investigated in order to evaluate the possibility of producing low-cost components to be employed as dissipative elements in steel bracings for seismic retrofitting of RC frames. More specifically, the performances of the so-called Steel-Slit and Short Link devices were compared. It was demonstrated that the Steel-Slit solution is characterized by a significantly faster degradation, in terms of both strength and ductility, with respect to the short link devices. The results of the aforementioned cyclic tests were elaborated to obtain low-cycle fatigue curves, which showed how the equivalent number of available cycles is significantly higher for “short-links” than for “steel-slit” devices. Finally, it is worth noting that a much better performance was achieved by the Torsional Damper in terms of both maximum displacement capacity and equivalent number of cycles.

4.8 Behavior and Strengthening of Industrial Structures

Experimental and analytical activities were carried out to investigate the seismic behavior of different connection types commonly found in precast structures. Monotonic and cyclic experimental tests were performed on floor-to-beam, roof-to-beam, column-to-foundation, beam-to-column and panel-to-panel connections. Figures 25a-e show the setup for some of these tests. The results provided information on the seismic behavior of connections in terms of strength, ductility, dissipation, deformation, decay and damage (Figure 25f).

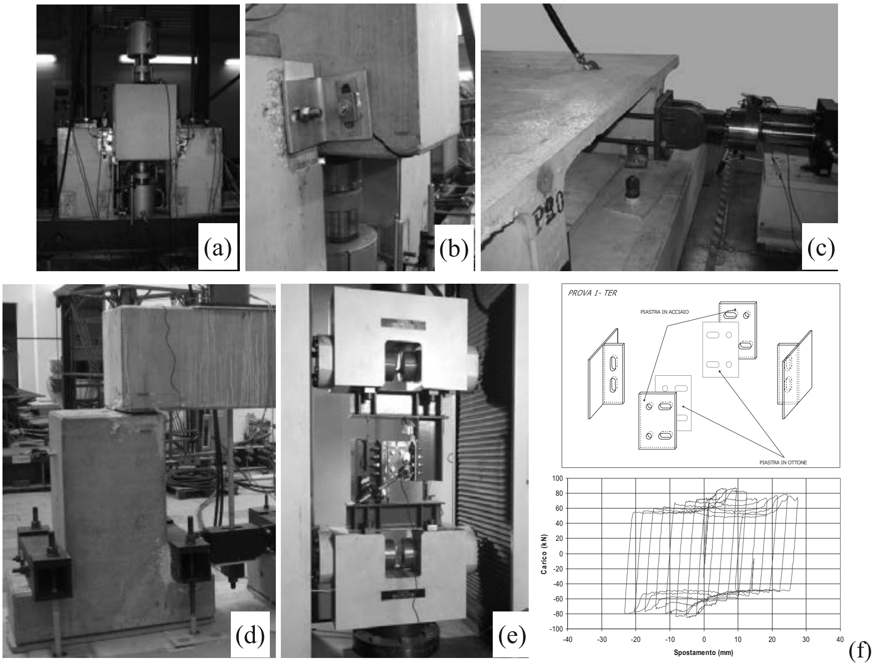


Figure 25. (*a-e*) Set-up of experimental tests on connection between structural elements; (*f*) innovative connection.

Analytical studies investigated the role of panel-to-panel connections in precast structures with cladding wall panels, considering both vertical and horizontal cladding panels. The results showed very high forces in the fastening devices, which create serious problems for the connectors' design. These forces can be reduced or eliminated by using dissipative devices or statically determinate panel-to-structure connection systems. In this context, an innovative connection device aimed at ensuring a de-coupled behavior between frame structure and panels was experimentally tested to check its sliding capacity under combined out-of-plane and in-plane forces, providing also information about the friction coefficient. Analytical studies on the role of panel-to-panel connections and of the diaphragm action in multi-span precast frame structures were carried out. The results highlight that the intensity of the forces arising in the floor connections of semi-rigid diaphragms is significantly higher for integrated connection systems than for statically determinate systems. Relevant distortions were also found for strut-type floor members. Finally, guidelines for seismic design and retrofit of connections in existing industrial buildings were drafted. These guidelines include criteria for the retrofit of different connections types, as well as a "defense line" for the cladding wall panels realized by means of steel cables connecting each panel to the frame system to prevent it from falling, after the failure of its existing connections. Innovative panel-to-panel connections are also proposed to achieve large energy dissipation.

A test program was also developed to study the bond between reinforcement bars and mortar, inside steel tubes used as foundation anchorages of rebars protruding from columns, or between steel tubes and surrounding concrete. Tests were carried out with different anchorage lengths and different steel box types. Cyclic forces were applied both on the longitudinal and transverse directions (with respect to the bars). Results show the effectiveness of the system, even for short anchorage lengths, especially for corrugated tubes. The test program aimed at understanding the behavior of the connection between prefabricated columns and foundation was completed with full scale tests. Three types were tested: conventional cast in place, with square steel boxes and with corrugated steel tubes. Results show a good capacity of developing inelastic deformations, reaching drifts higher than 4%. Others tests were conducted on full scale 3 and 4 ways prefabricated column - beam nodes completed in situ, showing full capability of transmitting moment and shear. Numerical models were developed to simulate the experimental test results. Special attention was given to the modeling of bond between bars and concrete inside the beam - column node to verify the effectiveness of the bars' anchorages. Finally, other tests and numerical analyses were carried out on precast roof-to-beam and on innovative beam-to-column and panel-to-column connections in order to investigate their displacement and energy dissipation capacity. Results showed that dissipation of the beam-to-column connections can strongly reduce the maximum transmitted force at the column base, the most critical cross-section. The connection proposed to retrofit existing cladding panels was able to disconnect the displacement of the column from that (much smaller) of the panel, thus realizing the mechanical independence usually considered according to old design criteria.

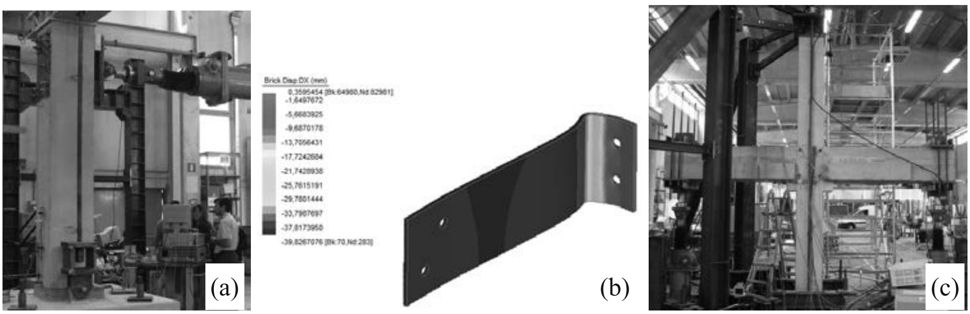


Figure 26. (a) column-foundation, (b) floor-beam and (c) beam-column connections.

The above experimental and analytical results were used for the preparation of Guidelines for seismic design and retrofit of connections in existing industrial buildings published after the 2012 Emilia Romagna earthquake. Moreover, the research team members helped the Civil Protection Department in disseminating this information to the industrial world, thus increasing awareness of the problem and of its possible solutions.

5 DISCUSSION

5.1 Knowledge of Existing Structures

The main objectives of the research program were achieved. Given the big effort in software development, the activities could be considered as the first step in reducing the gap between research and the professional world in the field of structural reliability.

As for the concrete properties, the results highlighted by the research allowed to better understand: (i) the effects of core drilling on the member strength, (ii) how and where to perform the NDTs in order to make sure that the results are not influenced by previous damage and cracking of the investigated member and, finally, (iii) how to correct the core strength in order to take into account the damage occurred during the drilling operation. Thanks to this research, information is now available to better approach the estimation of the concrete strength in existing structures as well as to be aware of the effects of the core drilling on members.

5.2 Assessment of Nonlinear Behavior of Buildings

All the research objectives foreseen at the beginning were fully attained.

5.3 Influence of Infills on Structural Response

The results obtained are considered satisfactory; they contribute to increase knowledge on the behavior of the infills and show that neglecting their presence under seismic action is often a non-conservative assumption. Collection of these results in a research report, similarly to what was done in the previous ReLuis research in this field, could show the remarkable progress achieved in the last three years.

5.4 Behavior and Strengthening of Beam-Column Joints

Test results on wide beam-column joints show the critical role of the bond of the beam longitudinal bars on the whole subassemblage behavior. In fact, the larger the amount of longitudinal reinforcement in the beam, the earlier the occurrence of the diagonal cracking of the joint panel out of the column width. This explains the sudden force drop after the peak value (in the shear drift envelope) especially found in seismically designed joints that have a larger amount of longitudinal beam reinforcement, causing higher values of the tensile principal stress. For this reason, bond conditions deteriorate, weakening the flexural strength of the beam and, then, threatening the response of the entire joint specimen. The sudden degradation of strength is, in turn, the cause of the decrease of the conventionally evaluated deformation capacity. However, the experimental peak strength is very close to that computed assuming hinging of the beam. Finally, from the test results it appears that the major problem of wide beam-column joints can be the poor bond condition also caused by the lack of space to develop the full anchorage length, due to the smaller height of wide beams compared to stiff ones.

5.5 Behavior and Strengthening of Columns and Beams

The completed research has led to an analytical tool that provides moment-curvature relationships with a very low computational effort for the case of RC sections subjected to uniaxial bending and axial forces. The accuracy of the proposed formulations was verified through a comprehensive parametric study, by varying the four parameters involved in the formulations. The comparison demonstrates that the closed-form equations are able to capture with a bilinear law the trend of M-ϕ relationship obtained from fiber section analysis. The application of the formulations to the analysis of real cases could definitely help validate and improve the proposed method.

The study concerning the confinement effect produced by FRP/FRCM wrapping, based on a high number of experimental tests, allowed considering the influence of many geometrical and mechanical parameters, so that in this field the objectives of the research program can be considered to be reached. On the other hand, the objectives related to the study of columns strengthened by steel angles and strips were reached only partially, because the number of tests was insufficient, and the test equipment did not allow high values of load eccentricity.

5.6 *Local Strengthening Methods*

The main objectives of the research program were achieved and data collected during the research program allowed further development and benchmark studies for code provision enhancements.

In the study of shear strengthening, experimental tests up to failure were carried out on existing beams reinforced in shear with C-FRP U strips negative moment regions. Numerical studies were carried out for a better interpretation of the experimental results. The effectiveness of the retrofitting was shown in case of shear and negative moments but the collapse of the beams is due to C-FRP detachment localized at the strips anchorages. Test results allowed a critical analysis of code prescriptions concerning C-FRP shear reinforcement details in presence of negative moments. However, while it is clear that the anchorage of shear U strips reinforcement negative moment regions cannot be treated as in the case of simply supported beams, a comprehensive solution is not yet available and further experimental and numerical tests are needed.

In the research on bridge piers, experimental PSD tests proved the effectiveness of the proposed repair and retrofitting intervention on seriously damaged RC bridges. The software developed for the PSD tests on repaired bridges and piers, was upgraded for considering asynchronous seismic input too and a better structural model using OpenSees. Some of the repaired pier specimens were not tested due to the large amount of data processed for the first cases, while some evident changes should be considered for bar substitution. These latter should be the subject of future studies. Numerical analyses using OpenSees allowed the interpretation of the experimental results. Experimental tests on SCC confined by C-FRP and stainless steel rebars, used in the repair intervention, allowed the calibration of existing analytical models for these materials, essentially using OpenSees to improve the results of the numerical analyses. The proposed repair and retrofitting intervention proved to be a rapid and valid solution that reduces costs and increases the structures' durability.

5.7 *Global Retrofitting Methods*

The research results show that retrofitting of existing RC framed buildings with BRBs generally requires: the estimation of the optimum parameters for the dissipative braces, which may be conveniently performed by using simplified analysis methods; the application of capacity design checks for all members of the structure under the expected ultimate force induced by the dissipative braces, e.g. the yielding force of the BRBs; the compliance with the design performance requirements, which is preferably carried out through nonlinear time history analyses.

A new assessment of the seismic action for tall buildings, consistent with the new codes, should lead to a good behavior of the buildings, unless the conditions related to the type of soil on which the building is founded are extremely unfavorable. In these cases a seismic site response evaluation is strongly recommended for the proper assessment of the seismic safety and the accurate choice of the retrofit strategy.

For the seismic retrofitting of RC frames, experimental evidences showed that low-cost components can be employed as dissipative elements in steel bracings.

5.8 *Behavior and Strengthening of Industrial Structures*

The main objectives of the research program were achieved and data collected during the research program allowed further development and a better tuning of design rules for code provision enhancements.

6 VISIONS AND DEVELOPMENTS

6.1 Knowledge of Existing Structures

Notwithstanding the significant work produced on this topic, further research and application on more case studies are still needed. Updated and more complete software versions (also considering the tests on the concrete strength) are in preparation. As possible future development, the database of available cyclic test results on RC members can be used in order to provide a probabilistic characterization of the hysteretic behavior in RC members. This information can be used in order to achieve a more comprehensive characterization of uncertainties taking also into account the uncertainty in RC members' components in existing buildings.

As for research on material testing, data search concerning historical certifications of construction materials (concrete and steel), issued and recorded by the Official Testing Laboratories in L'Aquila, is an extremely time consuming activity. Cooperation with other research groups experienced in the use of direct and indirect test methods for testing concrete, as well as in "archiving search of data" would speed up research. Concerning in situ test methods, more effective techniques should be developed in order to properly investigate the behavior of reinforcing steel bars, thus reducing the need for sampling them from the structures. Regarding existing software already proposed by ReLuis (such as "Biaxial") some optimizations would be suitable. In fact, mechanical response of RC elements would be better carried out if a larger variety of non-linear stress-strain models could be implemented for both concrete and reinforcing steel bars.

As far as the determination of concrete properties, one of the possible developments of the research is the extension of previously reported research to RC members provided with reinforcement details quite similar to those suggested by modern codes. Therefore, the evaluation of the effects of core drilling on the member strength and of the influence on the results of NDTs of the location along the element could be also studied for those RC members having a larger average concrete strength and different reinforcement arrangements as well as ribbed bars in place of smooth ones.

6.2 Assessment of Nonlinear Behavior of Buildings

In this topic a significant research effort should be devoted to extending the knowledge to more sophisticated structural models, from two-dimensional to three-dimensional buildings subjected to bi- or three-directional earthquake ground motions, taking also into account the influence of in the plan and elevation irregularities and the possible presence of infills. Moreover, in order to consider the modified seismic response of framed structures in the presence of masonry infill, proper models should be formulated considering the complexity and the computational feasibility of hysteretic infill models based on the equivalent strut approach for in-plane and out-of-plane behavior.

6.3 Influence of Infills on Structural Response

The following possible developments are advisable:

- carry out further experimental and numerical investigations on frame-infill interaction;
- develop further analyses of case studies of realistic Italian existing structures;
- thoroughly investigate the problem concerning the out-of-plane stability of the infills, possibly considering combined in-plane and out-of-plane loading;

- extend the experimental investigations by considering large-scale models of infilled frames with openings.

6.4 Behavior and Strengthening of Beam-Column Joints

Additional tests could be necessary in order to investigate the effects of biaxial actions and of the infill panels interacting with the beam-column joints. Further research should also address modeling issues for static and dynamic nonlinear analyses.

6.5 Behavior and Strengthening of Columns and Beams

Starting from the results of the developed research, the next step might be the extension to the case of RC sections subjected to biaxial bending moment and axial forces. Moreover, with the aim of extending the study to the analysis of bridge piles, the proposed formulation should be generalized to the case of RC hollow sections.

Possible developments of the research carried out regarding confinement effects are the following carry out: compressive tests on RC columns wrapped by FRP/FRCM sheets in order to verify the reliability of the constitutive law derived from the concrete elements tested plan; further experimental tests on columns strengthened with steel angles and strips. In both cases a collection of all results available in the published literature and their comparison in relation to common parameters would be desirable.

Also, further research is still needed to better investigate the experimental behavior of elements characterized by shear and flexure-shear failure modes. Furthermore, additional biaxial tests would be necessary for a more robust calibration of biaxial deformation capacity domains.

6.6 Local Strengthening Methods

In light of the fact that local interventions produce an improvement in the global behavior of existing buildings, which can also be significant, and that current guidelines do not require global structural analyses to quantify this improvement in each individual case, the development for easy and preliminary quantification of global seismic capacity increase due to local interventions can become an important instrument for the selection of the appropriate structural strengthening solutions.

Regarding shear strengthening, new experimental tests and studies will be useful to define more detailed code specifications on: a) C-FRP shear configuration and b) anchorage length definition in case of shear in negative bending moments regions. A Design by Testing method should be proposed for evaluating the partial coefficient taking account of the test results on materials and structures for assessing shear and bending capacity of existing beams before and after the reinforcement. Coefficients to be used in the verification formulas need further calibration.

As far as the study on bridge piers, the repair of piers damaged by an earthquake requires complex interventions in the plastic hinge areas, with replacement of the longitudinal and transverse steel and concrete damaged. More specifically, the longitudinal reinforcement represents a complex technological problem since the connection between new and original rebars must be made in the critical zone ensuring the development of the ductility demand. Further studies on the improvement of the construction details such as the connection between longitudinal rebars and technical solution to assure the plasticization in plastic hinge zone on the substituted rebars, will be conducted to increase the effectiveness of the seismic upgrade. Experimental tests on new rebar portions (carbon and stainless steel types) and connection systems will be carried out. Numerical cyclic models will be calibrated for new stainless steel rebars (produced in Italy after 2008) and used to simulate repaired and seismic upgraded bridge and pier behaviors.

6.7 Global Retrofitting Methods

Additional research is needed to better investigate different possible innovative techniques to increase the global seismic capacity of existing buildings. As for tall buildings, the effects of the interaction between soil and structure in the presence of seismic action cannot be negligible. Nevertheless current codes provide only partial information on this issue. Thus the identification of structural types for which this interaction is not negligible could be important for a proper calibration of retrofit techniques.

6.8 Behavior and Strengthening of Industrial Structures

Starting from the results of the research carried out in this project, the next step might be the extension of the investigation to a larger number of possible connections, more specifically those introduced after the Emilia Romagna earthquake. Many of them, in fact, were used almost without design process relying only on the argument that they were made of steel. The proposed further investigation will allow the completion of the design rules and guidelines for these types of local connections. The problem is very important since hundreds of thousands of RC industrial building still have an insufficient safety levels against the design earthquake.

Another important aspect to be further developed is the definition of simple numerical models capable of taking into account the role of the cladding panels and their interaction with the RC frames.

Finally, for those construction techniques allowing the precast system to be completed on-site (by means of casting or prestressing) in order to obtain a certain degree of redundancy, further analyses and tests are required in order to obtain a better perspective of their real ductility capacity.

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EVALUATION OF THE VULNERABILITY OF MASONRY BUILDINGS, HISTORICAL CENTRES, CULTURAL HERITAGE

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1 INTRODUCTION

Masonry buildings, whether ordinary or monumental, still constitute one of the most vulnerable classes of structures. Despite the numerous studies that were carried out in the recent decades, still almost countless open problems exist regarding assessment methods, strengthening strategies and techniques, availability of adequate standards and codes of practice for design, assessment and strengthening.

This Task of the ReLUIS project has envisaged two main thematic areas, one related to the improvement of the methods and models for structural analysis and assessment of the vulnerability of existing structures, and the other to the design of experimental techniques and to the development of tests for the validation of strengthening methods, technologies and strategies.

The issue of the methods of analysis and assessment of existing masonry structures is a fundamental one, that has gained even further relevance after the publication of new code provisions such as the NTC 2008, since such codes have enhanced the role of nonlinear analysis and have introduced the necessity of assessing local mechanisms mainly associated to out-of-plane response. Within the first ReLUIS 2005-2008 framework project, the research project no. 1 "Evaluation and reduction of the vulnerability of masonry buildings" had tackled the different topics with the aim of defining the state of the knowledge in Italy, with an intense confrontation among the several Research Units (RU) and with the attainment of several important results. Regarding modelling and assessment methods this second triennial project has started from the standpoint of a general knowledge of the problems and focused on specific issues on which it is felt that research is strongly needed.

The second thematic area encompasses in-situ and laboratory experimental applications aiming at: a) building a better knowledge of the existing structures; b) the development and assessment of strengthening techniques and strategies. In this area, several activities constitute a continuation and completion of experimental researches that had been developed in the first triennial framework project, but also new topics have been introduced. In short, three main fields of investigation where research is needed have been envisaged: 1) the experimental assessment of the mechanical properties of the different existing masonry typologies and of the effectiveness of techniques that aim to improve the quality of masonry; 2) floor and roof diaphragms, timber substructures, masonry vaults and arches and their interaction with vertical substructures and 3) experimental assessment of the behaviour of structural systems or subsystems.

2 BACKGROUND AND MOTIVATION

The vulnerability assessment of masonry buildings, in particular those aggregated in historical centres and the cultural heritage structures, is of fundamental importance for the mitigation of seismic risk, both because they represent a high percentage of the built environment and they are very vulnerable. This topic has become even more relevant after the issue of OPCM 3274, in 2003, and of the subsequent NTC 2008, as these standards have introduced the use of new advanced nonlinear methods for the seismic analysis and verification, in terms of displacements/deformations demands and capacities. This approach is proposed both for the global analysis, which consider the in-plane response of masonry walls, and the local mechanisms that may involve the out-of-plane behaviour.

Within the ambit of the previous ReLUIS project, Task 1 (Vulnerability of masonry buildings - Coordinators: Sergio Lagomarsino and Guido Magenes) had tackled the different issues with the aim of making a state-of-art of the wide knowledge achieved in Italy on this topic in the previous 30 years. Many research units (RUs) were involved, in order to make a comparison and promote the discussion among people that worked in this field. This approach has turned out to be very effective, as a lot of information had been collected and organized, as well as new results were achieved.

Considering that we started from a deep and consolidated knowledge of the problem and from an updated state-of-art, the Task "Vulnerability of Masonry Buildings, Historical Centres and Cultural Heritage", in this second ReLUIS project, was organized by selecting some critical issues and topics, which have been investigated, in most of the cases, by one single RU. However, collaborations between RUs took place, with interesting and valuable results.

3 RESEARCH STRUCTURE

The new project is articulated in two main subtasks, each one divided in working packages (WPs) and relevant activities. A total of 32 Research Units were involved, three of them without financial contribution. The Research Units and the corresponding scientific coordinators participating into the project are the following:

RU	Institution	Scientific Coordinator
POLIMI-a	Politecnico di Milano	Giuliana Cardani
POLIMI-b	Politecnico di Milano	Maria Adelaide Parisi
ROMA1-a	Università di Roma "La Sapienza"	Luis Decanini Domenico Liberatore
ROMA1-b	Università di Roma "La Sapienza"	Giorgio Monti
ROMA2	Università di Roma "Tor Vergata"	Ugo Ianniruberto
ROMA3	Università di Roma Tre	Gianmarco de Felice
UNIBG	Università di Bergamo	Giulio Mirabella Roberti
UNIBO *	Università di Bologna	Marco Savoia
UNIBS	Università di Brescia	Ezio Giuriani
UNICAS *	Università di Cassino	Elio Sacco
UNICT	Università di Catania	Ivo Caliò
UNIFE	Università di Ferrara	Antonio Tralli
UNIFI	Università di Firenze	Andrea Vignoli
UNIGE-a	Università di Genova	Sergio Lagomarsino
UNIGE-b	Università di Genova	Stefano Podestà

UNINA-a	Università di Napoli "Federico II"	Nicola Augenti
UNINA-b	Università di Napoli "Federico II"	Bruno Calderoni
UNINA-c	Università di Napoli "Federico II"	Antonio Formisano
UNINA-d	Università di Napoli "Federico II"	Beatrice Faggiano
UNINA-e	Università di Napoli "Federico II"	Claudia Casapulla
UNINA-f *	Università di Napoli "Federico II"	Antonello De Luca
UNIPD-a	Università di Padova	Claudio Modena
UNIPD-c	Università di Padova	Maria Rosa Valluzzi
UNIPG	Università di Perugia	Antonio Borri
UNIPI	Università di Pisa	Mauro Sassu
UNIPV-c	Università di Pavia	Guido Magenes
UNIRC	Università di Reggio Calabria	Vittorio Ceradini
UNISR	Università di Catania – sede di Siracusa	Caterina Carocci
UNITN	Università di Trento	Maurizio Piazza
UNITS	Università di Trieste	Natalino Gattesco
UNIVE/IUAV	Università IUAV di Venezia	Paolo Faccio

* RU without financial contribution

The organization in subtasks, work packages and activities is given in the following. The RUs involved in each activity over the triennium are also listed next to each activity. Due to the high number of Research Units and the limited number of pages of this chapter, the description of the activities and main results over the triennium will be given very briefly in the next section.

SUB-TASK 1a - Modelling, seismic analysis and assessment of masonry structures - Coordinator: Sergio Lagomarsino.

WP1: Knowledge of the structure, evaluation of the sources of vulnerability, definition of the structural models.

a. Identification of the structural units in a building aggregate. Damage interpretation and identification of possible mechanisms. Survey. (UNISR)

b. Critical issues in structural modelling of masonry structures. (UNIFE)

c. Methodologies and procedures for the seismic vulnerability assessment of buildings aggregates in historical centres. (IUAV)

WP2: Analysis of the local mechanisms and relevant assessment criteria.

a. Validation of different numerical models (also with reference to experimental results). (UNIGE-a)

b. The role of masonry quality. The influence of model uncertainties. (ROMA3)

c. Deformation and dissipation capacity in out-of-plane mechanisms. (ROMA1-a)

d. Dynamic behaviour of out-of-plane mechanisms. (UNINA-e)

WP3: Modelling of the global response.

a. Simulation of shake table tests (UNIPV-c)

b. The role of masonry spandrels and relevant strength criteria (UNINA-b)

c. The role of diaphragms and relevant modelling criteria (ROMA2) *d. Assessment of irregular buildings. Response of buildings with flexible diaphragms (UNIGE-a)*

e. Modelling and assessment criteria for buildings in historical aggregates (ROMA1-b)

f. Nonlinear analysis of building aggregates in historical centres (UNINA-c)

g. Modelling of monumental buildings (churches) (UNINA-f)

h. Modelling of strengthening interventions (UNICAS)

SUB-TASK 1b - Masonry structures: experimental research, testing, evaluation of the effectiveness of strengthening interventions - Coordinator: Claudio Modena.

WP1: In-situ/experimental evaluation of the masonry quality and of the effectiveness of masonry strengthening/improvement techniques.

a. Knowledge of the structure, in-situ survey, in-situ and laboratory tests and diagnostics aiming to assess the masonry quality (POLIMI-a, UNIPG, UNIFI)

b. Experimental evaluation of the in-plane behaviour of double-leaf stone masonry with different degrees of transversal connection (UNIGE-b).

c. Experimental evaluation of the effectiveness strengthening/improvement techniques:
c1. grout injections and applications of FRPs (POLIMI-a,UNIPD-c, UNIBO)*
c2. innovative strengthening techniques (fibre reinforced masonry, "reticolatus"...) (UNIPG)
c3. innovative strengthening techniques (fibre reinforced plasters) (UNIFI)

d. In-situ testing of shear strength of masonry:
d1 in-situ shear test with flat jacks (UNICT, UNIBG)
d2 in situ shear test with the mutually contrasting panels technique (UNIPI)

e. Optimization of the diagnostic strategies and techniques (UNIPD-a)

WP2: Floors, roofs, timber substructures, masonry vaults and their interactions with vertical structures

a. Timber substructures
a1. Study of the elastic and post-elastic behaviour of traditional screwed connections in wood elements strengthened with metallic elements (UNITN, POLIMI-b)
a2. Study of alternative timber floor strengthening techniques (UNITN, POLIMI-b)
a3. Analysis of the response of timber structures and elements in the L'Aquila earthquake (POLIMI-b).
a4. Analysis of the most common sources of vulnerability in masonry buildings associated to the timber roof and floor substructures and proposal of interventions (UNITN, POLIMI-b).

b. Non-destructive diagnostic techniques for timber (UNINA-d)

c. Arches, vaults, diaphragms and their interactions with vertical structures:
c1. Experimental study of the rocking of arches subjected to horizontal loading and of the effectiveness of stiffening of thin vaults with lightweight ribs (UNIBS)
c2. Experimental and analytical study of the cyclic behaviour of wall-to-floor and wall-to-roof connections (UNIBS).
c3. Experimental study of the effectiveness of innovative solutions for floor stiffening (natural lime reinforced with glass fibre meshes, fibre reinforced concrete) (UNIBS).

WP3: Testing and modelling of masonry structural systems and subsystems

a. Full-scale testing and modelling of masonry spandrel beams (UNIPV-c, UNITS, UNINA-a)

b. Tests on masonry walls before and after strengthening with innovative materials (UNINA-a)

c. Reconstruction of historical masonry (UNIRC)

d. Development of provisional interventions for cultural heritage buildings (UNIPI)

4 MAIN RESULTS

4.1 SUB-TASK 1a - Modelling, seismic analysis and assessment of masonry structures Coordinator: Sergio Lagomarsino

WP1: Knowledge of the structure, evaluation of sources of vulnerability, definition of the structural models.

a. Identification of the structural units in a building aggregate. Damage interpretation and identification of possible mechanisms. Survey.

UNISR

Activities were focused on the following topics: (i) defining the contents of a procedure for the identification of Structural Units in the aggregate, (ii) the preparation of a document-guide for the identification of the activated damage mechanisms activated through the use of synthetic “abaci”.
A short description of results follows:
(i) Analysis of the historical centres in Emilia damaged by the earthquake in May 2012. The methodology for the identification of the aggregates in historic urban fabric already suggested by the analysis carried out in a previous research (smaller historic towns in the area of L’Aquila, the historic centre of Faenza), is enriched by different remarks arising from specific configurations of aggregation, such as the presence of "ambitus" in the early allotment.
(ii) Also with regard to this topic, the activities clarified some particular seismic response - not highlighted before - in a peculiar configuration of buildings located in an aggregate (for example buildings containing porch as in the case of Crevalcore). A more complete description of damage mechanisms that can affect the building located in an aggregate was achieved.

b. Critical issues in structural modelling of masonry structures.

UNIFE

A study of the out-of-plane behaviour of masonry walls has been performed.
In particular a F.E.M. formulation has been extended to analyze the effects of second order geometrical nonlinearities. Within this formulation it is possible to correctly perform the nonlinear kinematic analysis, defined in the Italian code NTC2008, also in presence of out-of-plane collapse mechanisms in which the overturning of the wall does not occur around a horizontal line (Rondelet Mechanisms), and torsional effects are present.

As a consequence of the seismic sequence that stroke Emilia in May 2012 the RU has been directly engaged in the evaluation of damage to masonry buildings. The results of this activity are presented in a series of case studies that were treated in collaboration with the RU UNIGE-a.

c. Methodologies and procedures for the seismic vulnerability assessment of buildings aggregates in historical centres.

IUAV

The activities carried out within the research project have implemented and enhanced a methodology for the seismic vulnerability evaluation of closely connected buildings, i.e. aggregates. Such an approach was applied and tested to the case study of “Civita di Bagnoregio”.
The analysis of the aggregate starts with the critical examination of the main documents concerning the growth and transformation of each property (e.g. land register and historical reports documentation), as well as with the application of the stratigraphy survey of the constructions. The aim is the identification of the transformation-growth phases of each building, the construction relationships between elements, underlining macro-stratigraphic relationships (Faccio and Brogiolo, 2013), (Zamboni, 2013a, 2013b), (Scaramuzza, 2013). Such an approach gives the main elements for the evaluation of seismic vulnerability through simplified numerical approaches able to define the parameters and the constructive characteristics which influence the numerical models.
The possibility of comparison between the analysis of damage and the constructive phases (identified both with direct methods and documental sources) carried out on the aggregate (Campanini 2013a, 2013b), with the response of the numerical/analytical modelling contributes to the definition of rapid analyses protocols and to the development of vulnerability assessment of historical centers, usable at a territorial level but also preparatory to the definition of higher level methods.
During the research, the need of combining the quantitative evaluations (carried out using a simplified approach applicable at territorial level for the whole historical center) with a qualitative vulnerability matrix was highlighted. Such a vulnerability matrix includes all the elements obtainable from a rapid investigation supported by stratigraphic analysis, transformation phases identification and land register and historical reports documentation.
In particular, for the definition of the vulnerability matrix it is necessary to perform:

- Analysis of the land register and archivist documentation;
- Study of seismic chronology
- Rapid stratigraphic analysis;
- Study of geological documentations;
- Drafting of the Masonry forms.

Starting from the data obtained during the phase of knowledge acquisition, a seismic index for each aggregate was evaluated taking into account the mechanical characteristics of masonry elements, obtained with micro-modelling technique (both based on homogenization technique, Milani and Cecchi, 2013, and on compatible identification approaches) which considers the variability of geometrical dimensions of blocks and mortar joints, of the masonry texture and of the deterioration state. Such a seismic index was obtained by applying simplified approaches based on the LV1 method proposed by the Italian code “Direttiva 2011” for isolated buildings, as well as on a modified method which accounts for different techniques for the force distribution between the elements of the aggregate (depending, for

example, on the stiffness of horizontal deck). The adoption of modelling techniques allows to limit as much as possible the direct tests on masonry.
At a glance, the main aim of such an approach, which, as previously said, combines:

- the implementation of a qualitative matrix summarizing the main vulnerability of each aggregate belonging to an historical center;
- the evaluation of a seismic index with rapid methods (LV1 approach, modified LV1 approach)

is to carry out the seismic vulnerability assessment with the related drawing up of the risk ranking (*vulnerability list for comparison*) of all the aggregates belonging to the same historical center. Such a ranking represents one of the instruments available to the public administration for pointing out the need of further investigations and for the intervention planning for seismic risk mitigation.

WP2: Analysis of the local mechanisms and relevant assessment criteria.

a. Validation of different numerical models (also with reference to experimental results).

UNIGE-a

Within the scope of WP2 - topic a (*Validation of procedures for analysis and verification, also with reference to experimental data*), experimental tests have been made on three masonry panels, of different thickness and slenderness, subjected to out-of-plane actions (Degli Abbati et al. 2014).
Panels were made of a double leaf irregular masonry, without transversal connections, sometimes with a small inner core made of smaller stones. The shape of the stones was irregular and roughly square. Lime mortar had very poor mechanical properties (compressive strength is around 1.5 MPa) and was present in large quantity, due to the irregular shape of stone elements, which did not allow a good mechanical interlocking.
The three panels had a width equal to 90 cm and the other dimensions are:

- Panel 1: thickness 22 cm, height 110 cm (slenderness 5)
- Panel 2: thickness 30 cm, height 90 cm (slenderness 3)
- Panel 3: thickness 30 cm, height 150 cm (slenderness 5)

All panels have been built over a basement made by the same masonry, but with a slightly higher thickness, in order to address the position of the overturning hinge. Each test was characterized by two phases:

a) Static overturning test, by applying controlling displacements and a horizontal action at 2/3 of the panel height, in order to evaluate the capacity curve;
b) Free vibration dynamic test, by releasing the panel after the static test and measuring the period of oscillation in the nonlinear range and the damping, due to impacts.

The above-mentioned tests have been repeated for different levels of displacement.
Results confirmed the reliability of non linear kinematic analysis, showing: a) the need to consider masonry quality (by shifting the position of the hinge towards the interior of the wall); b) the initial elastic stiffness which determines the behaviour (better represented by a dynamic bi or tri-linear model, rather than the Housner model); c) the sharp increase of damping with the displacement, from the initial value of 4% till to around 10%.
It is worth noting that in one of the panels the hinge did not appear at the base but a little bit above, where the stone blocks were not well connected; the result was a deterioration of the behaviour, with a significant reduction of the capacity, both in terms of strength and displacement.

Moreover, some in situ experimental campaigns have been performed in collaboration with the research units of UNINAc and UNIRC for the characterization of the in plane and out-of-plane masonry behaviour (Candela et al. 2013).
Finally, a detailed and comprehensive procedure for the assessment of rocking masonry structures through the displacement-based approach has been developed (Lagomarsino 2015).

b. The role of masonry quality. The influence of model uncertainties.

ROMA3

1-Multi-leaf masonry: seismic capacity and strengthening
A modeling strategy allowing the simulation of the influence of masonry morphology, reproduced from photographic surveys, on the seismic behavior of multi-leaf walls loaded out-of-plane has been developed (de Felice, 2011). For a given sample the methodology provides the pushover curve, including an estimate for the displacement capacity. A wide number of numerical simulations have been carried out on a sample consisting of 40 real masonry sections, by referring to two boundary conditions (free standing wall and wall constrained at the top) and by applying accelerations with both positive and negative polarities (Mauro and de Felice 2011 and 2012). The results allow the evaluation of the capacity reduction induced by the lack of connections throughout the thickness of the walls and the definition of some correlations with semi-empirical indexes provided in the literature to quantify masonry quality. Eventually, the sample of walls collected so far has been adopted for studying the effects of strengthening interventions made with injections of mortar and with tie-bars along the thickness of the walls, respectively. The pushover curves then obtained, when put in comparison against the unstrengthened walls, provide an estimate for the efficacy of the retrofitting interventions analysed (de Felice *et al.*, 2013).

2- Prediction of the local mechanism and influences of walls' connections
Aiming at investigating the influence of the connections among masonry walls, a methodology based on the Distinct Element Method has been developed which makes it possible to reproduce masonry textures surveyed on real structures. This approach has been applied to a sample of religious buildings (single nave churches) that have been damaged by the L'Aquila 2009 earthquake (de Felice and Mauro, 2010). For each case study, pushover analyses have been carried out with the purpose of evaluating *i)* the ability of the proposed approach in reproducing the actual local mechanism triggered by the earthquake, *ii)* the increase of capacity provided by the connections with transverse walls with respect to the case of isolated façade, and *iii)* the reliability of the methodologies proposed by the national code. The results collected from the sample of case studies highlight the effectiveness of the classical approach adopted for evaluating the out-of-plane seismic behavior of masonry walls, based on rigid block assumption, since the decrease in strength related to the morphology of the wall section is balanced by the increase in strength provided by connections with transverse walls.

c. Deformation and dissipation capacity in out-of-plane mechanisms.

ROMA1-a

Pushover experimental tests on freestanding parapet walls have been interpreted (Decanini et al., 2013a). Hence, an optimal curve and a range of variation have been suggested, related to the caementitious and the pozzuolanic mortar. The optimal curves have been selected minimising an error function between numerical and experimental free-vibration time histories. With respect to a bilinear nominal curve, the displacement capacity is approximately 88-93% (84-91%) for the walls with caementitious (pozzolanic) mortar. Similarly, the strength is 79-88% (56-79%) of the nominal one. Guidelines on the estimation of the coefficient of restitution have been prepared for three mechanisms: parapet wall undergoing two-sided rocking, parapet wall undergoing one-sided rocking, vertical spanning strip wall. The Guidelines are divided into two sections: the first part is dedicated to the closed-form formulation of the coefficient of restitution; the second part is focused on the experimental calibration of the analytical coefficient of restitution. The ratios between experimental and analytical coefficient of restitution are 0.95, 1.05, 0.90 for the three mechanisms previously mentioned. For the two-sided rocking mechanism a linear regression between coefficient of restitution and mortar compressive strength has been proposed.

Several thousands numerical time histories have been computed for the three mechanisms, for several wall geometries, using 20 pairs of horizontal components recorded in Italian and International seismic events (Decanini et al., 2013b). These have been used to evaluate the relevance of a parametrically varied coefficient of restitution on the vulnerability assessment. The correlation with the intensity measures of the excitations is very weak, while it is stronger with the type of mechanism. The parapet wall undergoing two-sided rocking and the vertical spanning strip wall are sensitive to variation of the coefficient of restitution; therefore, this needs to be carefully addressed. On the contrary, the parapet wall undergoing one-sided rocking is much less sensitive because it dissipates much more kinetic energy at impact. A second group of time-history analyses have been computed, considering only Italian records, in order to compare them to equivalent-static code procedures. Two skeleton curves of the static model have been taken into account: bilinear and trilinear. In the first case, for strength verifications, the percentage of conservative cases varies strongly with the boundary conditions. Similar percentages are obtained only if the behaviour factor is varied: 2 for the vertical spanning strip wall, 3 for the two-sided rocking, 5 for one-sided rocking. The displacement procedures for the three mechanisms involve closer results, although still rather conservatives. Such conservative cases increase if a trilinear skeleton curve is assumed.

Experimental data on clay brick and tuff masonry, published in approximately 80 papers, has been interpreted (Decanini et al., 2013c). The focus has been the estimation of the shear strength, evaluated through eight code procedures. Within such framework it was possible to highlight the conditions that grant similar results under a diagonal cracking approach or Coulomb approach. The collected experimental data has been compared to the tables in Italian codes, which usually are rather conservative. Some possible revisions have been suggested.

d. Dynamic behaviour of out-of-plane mechanisms.

UNINA-e

Within the framework of a new proposed strategy of analysis for the rocking response of a masonry rigid block under seismic actions, the research activity has focused on the validation of its first phase. The simplified equation of motion, which represents a uniform accelerated motion of the block between two subsequent impacts on the ground, was compared to the approximated equation first proposed by Housner (1963). Both formulations provide solutions in closed form of the stabilized block response to the resonance input. Good agreement between the results was observed, with slight discrepancies for very small and slender blocks (the Housner's results appear to be more conservative than the others). However, the proposed formulation takes the great advantage of providing very simple expressions for the maximum rotation angles and stabilized periods. Another check of validity was carried out by

means of the analysis of the response of the classical linear elastic SDOF oscillator to an analogous sequence of pulses and the comparison with the results for the rigid block obtained with both formulations. By using the empirical equation for the equivalent viscous damping ratio provided by Makris and Konstantinidis (2003), the results obtained reaffirm that the amplitude resonance for the block is much more intense than that for the SDOF oscillator and that, even in this case, the Housner's results are more conservative than those obtained using the proposed formulation (Casapulla 2013).
The second field of research is the limit-state analysis of masonry structures with frictional joints with the aim of defining reliable solutions of the ultimate load factors for the most recurrent in-plane and out-of-plane local mechanisms. The new proposed procedure of analysis based on macro-block modeling has been validated for the in-plane (Casapulla et al. 2012, 2013a) and out-of-plane mechanisms of masonry walls through the comparison against micro-block modeling and experimental evidence existing in literature. The micro-block model used for comparison is based on a recently proposed strategy, which involves solving a series of LP problems with successively modified failure surfaces and associative friction. This micro-modeling strategy has been further developed and extended to account also for the possibility of the cracking failure of micro-blocks (Portioli et. al. 2012). On the other hand, a first attempt to extend the procedure to three-dimensional limit analysis is based on the use of linear and linearized yield functions for rocking, sliding and torsion failure, whilst simple yield conditions were defined to take into account interaction effects of shear force with torsion and bending moment (Casapulla et al. 2013b, Portioli et. al. 2013). The collaboration of this RU with the University of Sheffield (UK) has recently been resumed to provide further developments on the 3D limit analysis of more complex masonry structures with non-associative frictional joints.

WP3: Modelling of the global response.

a. Simulation of shake table tests

UNIPV-c

The activities have been oriented to the numerical simulation of the experimental tests performed on three full-scale stone masonry prototypes at the University of Pavia and at EUCENTRE (the construction details of the masonry buildings and shaking table testing procedures are described in detail in Magenes *et al.* (2010c; 2012; 2013; 2014).
Simplified but effective numerical models have been created following an existing equivalent frame macro-element approach implemented in the program TREMURI (Lagomarsino *et al.*, 2013, Penna *et al.*, 2014), to replicate the experimental response of the prototype buildings by means of nonlinear static analyses and nonlinear dynamic analyses. The models have been calibrated to reproduce the damage pattern and the failure mechanisms activated in the shaking table tests, as well as the response of cyclic quasi-static tests of single structural members. The numerical simulation showed a significant consistency of the nonlinear static analysis results with the experimental response of the building (the experimental and numerical results relative to the building prototype with stiffened diaphragms are presented in Magenes *et al.* (2013) and Senaldi *et al.* (2014).
As concerns the nonlinear static analyses, the discretization of the equivalent frame model is meant to reproduce the damage pattern experienced by the building prototypes during the experimental campaign, hence assuming the height of piers close to that of the adjacent opening. The mechanical properties assigned to structural elements are based on the average

values from the masonry characterization tests (Magenes *et al.*, 2010a; 2010b). A force distribution proportional to the first mode was applied during the nonlinear static analyses to represent the actual distribution of acceleration occurred during the shaking table tests.
The numerical simulations, by means of nonlinear static analyses, showed a good consistency of the numerical results with the experimental response of the building, when applying a macro-element modelling approach. When compared with the envelope of the experimental seismic resistance curves, the macro-element model reproduces with sufficient approximation the global response of the prototypes, also in terms of damage mechanisms activated and of deformations experienced. It has also been shown that the numerical simulations based on elasto-plastic equivalent frame assumption tend to produce a rather conservative prediction of the strength capacity of the systems while, at the same time, they do not always reproduce satisfactorily the damage mechanisms activated during the last stages of the experimental tests. It is worth to mention on this regard that in the bilinear elasto-plastic approach the interaction between axial and shear forces as well as axial forces and moments are governed by the strength criteria rather than representing continuously the damage evolution and degradation of strength and stiffness, as implemented in the macro-element approach. Furthermore, in the elasto-plastic models issues are related to the role of spandrels in the redistribution of applied forces within the structural members (Da Paré et al., 2013).
The global behaviour was further simulated by means of nonlinear dynamic analyses, trying to reproduce the hysteretic behaviour of the strengthened prototypes at levels of acceleration similar to those of the shaking table tests. The numerical models were calibrated to capture also the evolution of the exhibited damage pattern at each stage of the dynamic testing. Sensitivity analyses were carried out to investigate the influence of several factors on the modelling of the global response of the building, in particular considering the following:

- discretization/geometry of the equivalent frame model;
- dispersion of the mechanical properties obtained experimentally from masonry characterization tests and in-plane cyclic tests;
- selection of an appropriate value of damping.

Based on the outcomes of the sensitivity analyses, the nonlinear dynamic analyses of the macro-element models showed a good consistency of the numerical results with the experimental response of the building, with a fair approximation of the hysteretic behaviour of the strengthened buildings, as presented in Figure 1.
The outcomes of the research are presented in deliverable UNIPVc-04 (Magenes et al. 2013b)

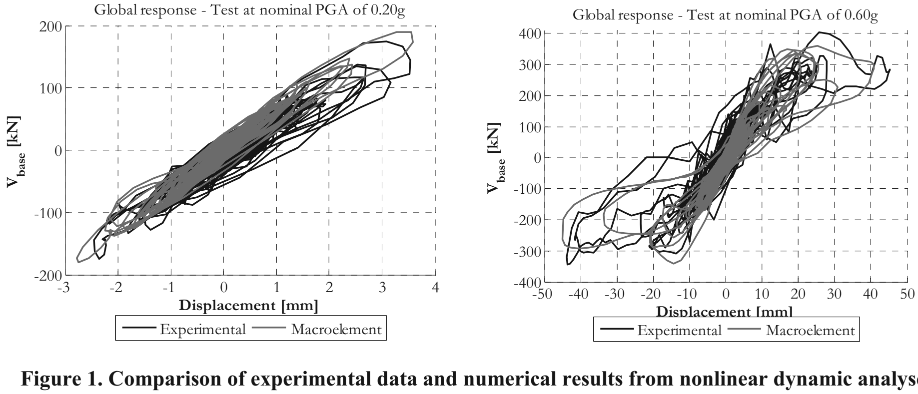

Figure 1. Comparison of experimental data and numerical results from nonlinear dynamic analyses of Building 3. Test at nominal PGA of 0.20g (left) and of 0.60g (right).

b. The role of masonry spandrels and relevant strength criteria

UNINA-b

A simplified theoretical model ('the arched strut'), useful for evaluating the structural behavior of masonry spandrels subjected to shear, has been proposed (Calderoni et al., 2012b). It is defined on the basis of both several numerical analyses and experimental testing results. In particular it allows to consider in a comprehensive way all the mechanisms of shear failure observed experimentally. More specifically, in a former model the "effective" zone of the spandrel was defined only on the basis of the results of the performed numerical analyses. In the present formulation it is obtained by applying theoretical compatibility and equilibrium conditions.
The improved model and the corresponding resistance criteria have been then validated by means of experimental results of former testing campaigns performed by the authors.
In general a satisfactory correspondence between experimental and theoretical results has been found, even if the difficulty in defining the actual tensile strength of masonry affects the validation of theoretical models (Calderoni et al., 2013b).
A new experimental campaign on H-shaped portions of masonry wall, formed by both a spandrel and part of the two adjacent piers, was carried out (Calderoni et al., 2012a; Calderoni et al., 2013a). The specimens (1:10 scale) were made of homogeneous material (reinforced with FRP strips and unreinforced) or ordinary block masonry and were tested by means of already existing testing equipment, which had been purposely modified. In this way the influence on spandrel behaviour of the masonry panel zone has been analyzed, in order to better simulate (by calculation) the seismic behavior of actual walls. The new experimental program has been performed on specimens presenting three different slenderness ratios, so defining slender, intermediate and stocky panels. Monotonic tests have been carried out on panels made of homogeneous material, while both monotonic and cyclic tests on those made of ordinary masonry and in reinforced homogeneous material. Different failure mechanisms have been observed: horizontal sliding (on slender or intermediate panels in ordinary masonry), diagonal cracking (on all stocky panels) and toe-crushing (on slender panels of all type of masonry and on all specimens in reinforced homogeneous material). Mixed failure mechanisms occurred too, particularly on intermediate panels.
The experimental results (in terms of shear strength) have been compared with the values given by Italian code (NTC08) prescriptions.
Finally, in order to evaluate the influence of the resistance of spandrels on seismic capacity of the masonry wall, a non-linear static analysis on a masonry wall was carried out. During the analysis the resistance and stiffness of spandrels were varied. It was observed that, unlike the piers, for spandrels the resistance of the material greatly affects the seismic capacity of the masonry. Furthermore it was observed that often the capacity that is obtained by considering the effective resistance and stiffness of the spandrels give a PGA less than that which obtained considering weak spandrels.

c. The role of diaphragms and relevant modelling criteria

ROMA2

After the selection of representative masonry structures, dynamic linear and non-linear analyses were performed. The result of the analysis show that a reduction of the in plane stiffness of the diaphragm is non always related to a reduction of the out-of-plane wall-floor interaction forces and therefore the best structural solution has to be chosen case by case. If a diaphragm with low masses is adopted a reduction of the shear forces at the in-plane wall-diaphragm connection is obtained.

d. Assessment of irregular buildings. Response of buildings with flexible diaphragms

UNIGE-a

The research activity has benefitted from models and procedures developed in the previous years, with the aim of validating and proposing changes to the verification criteria adopted by NTC 2008, in the case of irregular masonry buildings, complex and with flexible diaphragms.
To define the various Limit States (up to failure) it was necessary to formulate multilinear constitutive laws for masonry panels (piers and spandrels), to be used in the equivalent frame models; those models also consider the cyclic hysteretic response, in order to perform nonlinear dynamic analysis. An interesting validation has been made, through simulation of the seismic damage in a building in San Felice sul Panaro, after the earthquake of May 2012 (Cattari and Lagomarsino, 2013).
A multiscale approach is proposed for the definition of the limit states, which performs checks on the evolution of damage at three different scales: 1) structural elements, 2) *macroelements*, 3) global response. This allows obtaining reliable results even in the presence of complex buildings with flexible diaphragms, for which a certain performance cannot be fulfilled due to damage at a single wall or in a limited number of elements.
In the presence of complex buildings with flexible diaphragms the participant mass in the first mode can be very low. It is proposed in these cases to combine, with the SRSS rule, all significant modal shapes in a given direction, considering those that do not show sign reversal at the different levels, in such a way as to define a sort of "first modal shape" (and resulting pattern of forces), to be used for the pushover analysis. This "modal shape" generally has a participating mass greater than 75% and does not differ much from the simplified triangular shape.
In the presence of buildings irregular in plan, for which torsional effects and a certain degree of coupling of the response in the two orthogonal directions can occur, it may be necessary to consider the bidirectionality of the motion.
In the presence of buildings irregular in elevation (and when the number of floors is greater than four) higher modes can become important. It is suggested in these cases the use of multi-modal analysis.
Moreover, the problem of incomplete knowledge in the seismic assessment of masonry buildings was faced, by proposing the sensitivity analysis as a tool for planning investigations and defining a proper value of the confidence factor (Cattari et al., 2015).

e. Modelling and assessment criteria for buildings in historical aggregates

ROMA1-b

The current NTC-08 code allows for a simplified comprehensive verification for building clusters, allowing to analyze each structural unit individually, though obliging to carry out the study in the non-linear field, through a pushover analysis. The need of facilitating the vulnerability checks, as well as the description of the structure, was pursued in this research, with the implementation of a program dedicated to the analysis of building clusters, developed under Visual C#. The use of a GUI facilitates the input data phase, together with the graphic representation of the geometry of the different structural units. The vulnerability checks are conveniently accompanied by tables and online help, which support the user in interpreting the results. The opportunity finally to repeat the tests with ease, if necessary, by changing the values assigned to variables, makes the calculation tool a valuable means of investigation to support the study of alternative scenarios, thus helping to define the most effective intervention strategy.

Within building clusters, it is common to find masonry arches, which must be subject to security checks against lateral forces. To address the problem, the analysis procedure limit provided for by the NTC-08 and its Commentary (Circular no. 617/2009), for the evaluation of the level of safety of building structures against possible local mechanisms, has been applied to arches with circular profile. The equations obtained allow to perform rapid assessments of the level of seismic safety of existing arches and designing possible interventions to strengthen their upper surface. Starting from a case study, this RU has developed a sensitivity analysis of the risk of the considered arch, due to the variation of the main geometrical parameters representative of the shape and texture of the arch. The results provide a set of qualitative indications on the seismic response, which are in full agreement with the physical interpretation of the phenomenon.

f. Nonlinear analysis of building aggregates in historical centres

UNINA-c

After the last Abruzzi and Emilia Romagna seismic events (Formisano, 2012; 2013), forecast of seismic behaviour of of masonry building aggregates is becoming a pressing need. Seismic damages detected within the L'Aquila districts of Poggio Picenze and San Pio delle Camere historical centres, whose large scale vulnerability and damage have been examined (Formisano et al., 2013a; 2013b), have been addressed, where seismic behaviour of a series of building aggregates of San Pio delle Camere has been evaluated (Fonti et al., 2013).

Results of two different structural analysis programs have been compared with those deriving from LV1 approach of the Italian Guidelines on Cultural Heritage, which underestimate of about 50% the base shear of the aggregate structural units. Therefore, a shear increasing factor dependent on pier slenderness has been proposed in order to achieve in less conservative way more refined analysis results. By combining such results with hand calculation techniques able to assess yielding and ultimate displacements of masonry buildings, a rather precise simplified curve to predict the non-linear response of building aggregates has been plotted. Some damage curves of structural units intended both as isolated and as a part of the masonry compound have been drawn for different earthquake grades. The achieved curves have shown a damage reduction of “head” units (i.e. peripheral) greater than the internal units one due to the beneficial effect of the aggregate condition. This occurred for rectangular shape aggregates especially in the direction of walls in common among structural units (transversal direction). The seismic safety factor calculated according to the mentioned Guidelines underestimates of about 50% the one obtained using the 3Muri computer program. Such a gap could be eliminated by defining a new relationship of the participating mass ratio related to the first vibration mode for irregular buildings.

Finally, with reference to a case study, seismic retrofitting systems based on reversible and sustainable metallic systems have been implemented and applied, they conferring a better seismic behaviour to the masonry compound.

g. Modelling of monumental buildings (churches) (UNINA-f)

UNINA-f

The UNINA-f research unit carried out the following activities:

- Critical discussion about the results obtained from the evaluation of seismic action on churches through dynamic modal analysis with response spectra;
- Application of pushover analysis to obtain the "in plane" seismic capacity of masonry walls and to evaluate the strength increment provided by inserting of steel ties;
- Employing of Limit Analysis as an easy tool to check the results obtained through more complex non linear static analysis methodologies

Discussion on the dynamic behavior of the churches

From the results obtained applying the modal analysis applied to the 3D models of the four sample churches it is clear that this type of buildings is characterized by a special dynamic behavior. This behavior is rather different from that of ordinary buildings; it is characterized by the absence of principal vibration modes and by the existence of a considerable number of vibration modes that energize very small participating masses.

The consequence of this is that, to describe the dynamic behavior of the building, it would be necessary to consider at least the first 100 eigenmodes, so as to take account of at least 80% of the total mass of the building.

It was then proposed a possible procedure for checking the safety from local collapses; this procedure consists in identifying the possible mechanisms of local collapse starting from the "critical" mode shapes, once the mechanisms are known, it is possible to determine the seismic capacity associated with each of them and then the capacity is compared with the seismic action (spectral ordinate).

Evaluation of in-plane capacity of masonry walls by Pushover and Limit Analysis

A number of non-linear static analyses have been carried out on a sample of four buildings (placed in Napoli and L'Aquila) characterized by different geometric ratios. The software packages used for the analyses are: ABAQUS, 3Muri and 3DMacro. Three different approaches were adopted because: for the analyses with ABAQUS the walls were modeled using plate elements, in 3Muri the classical "equivalent frame" model is implemented while in 3DMacro the walls are modeled as assemblies of macro elements configured as pinned frames with diagonal springs (shear behavior) and perimeter springs (flexural behavior).

From the comparison between the results of the analyses, expressed in terms of F-δ curves, it can be first noted that there exists a good agreement between the results obtained through the three different modeling approaches (although with ABAQUS there are problems in assessing the displacement capacity).

The results obtained through Limit Analysis (LA) are perfectly aligned with those obtained using the softwares, thus demonstrating how LA represents a reliable method to control the results.

Finally, it was evaluated the effect of steel ties on the "in plane" seismic capacity of walls loaded. It resulted that, if the collapse of the spandrels is not considered, it is theoretically possible to achieve large increases in strength but, actually, those increments are quite impossible because of the weakness of the spandrels.

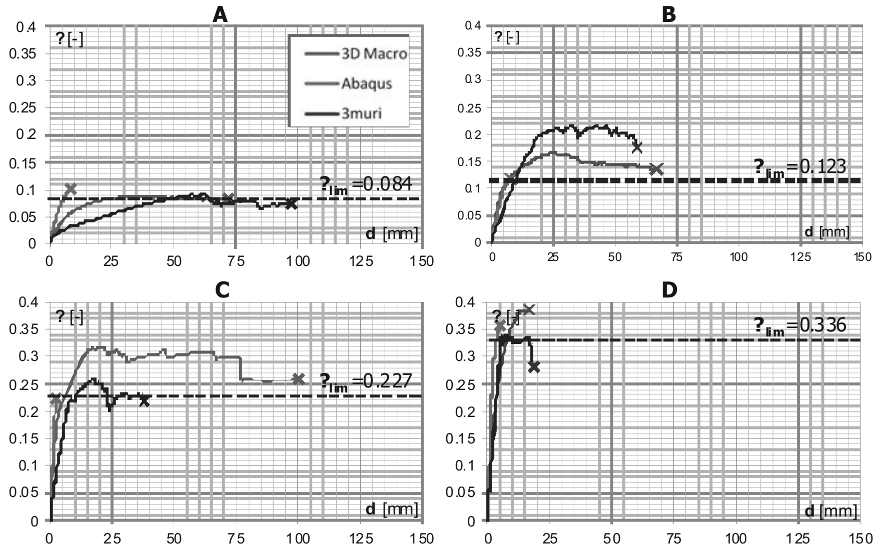

Figure 2. Comparison between the results of the pushover analyses and the Limit Analysis applied to the four sample buildings.

4.2 SUB-TASK 1b - Masonry structures: experimental research, testing, evaluation of the effectiveness of strengthening interventions - Coordinator: Claudio Modena.

WP1: In-situ experimental evaluation of the masonry quality and of the effectiveness of masonry strengthening/improvement techniques.

a. Knowledge of the structure, in-situ survey, in-situ and laboratory tests and diagnostics aiming to assess the masonry quality

POLIMI-a

A new approach to measure of the displacements during a double flat jack test was calibrated and a system of elaborating the results determined.
This new optical system is totally non invasive and can be used together with the traditional displacement transducers (LVDTs, deformometers, etc.) or a removable extensometer applied to the masonry surface, as suggested by the codes. They are used to measure relative displacements at each stress increase. This method can be correctly applied both to the regular brick masonry and to the irregular stone masonry, when local anomalies, rotation of stones and different stress flows can influence the traditional test results and reliability. After the laboratory calibration carried out on some stone masonry specimens realised in the labs of RU UNIPD, other tests were carried out in situ on historic masonry buildings.

UNIFI

The RU UNIFI has worked on processing and organization of data, concerning in-situ tests and laboratory tests carried out on masonry typologies commonly found in Tuscany, in order to achieve a regional masonry abacus.
The considered data include:

- experimental campaigns performed since the 80s and new experimental campaigns;
- tests on masonry panels (compression tests, diagonal tests, compression-shear tests, double flat jack tests), tests on resistant elements (compression after coring), tests on mortar (mineralogical and petrographic analysis, DRMS);
- data available by the Tuscany Region were acquired;
- tests on masonry typologies not included in Table C8A.2.1 of C.M. 2009 were carried out – brick blocks masonry with over 45% holes ("occhialoni", Annex UNIFI-III-01) and concrete blocks masonry ("cantoni", Annex UNIFI-III-02), providing to complete the Panel's Sheet (format ReLUIS Project 2005-2008 revised by RU UNIFI) and to calculate the correspondent IQM (according to RU UNIPG procedure).

With reference to the categories of Table C8A.2.1 (eventually corrected using the coefficients of the Table C8A.2.2), ultimate objective is:

- to identify possible categories/subcategories recurring in Tuscany;
- to determine specific values of its mechanical properties;
- to find a correlation between mechanical properties and IQM;
- to create a guideline document for the assignment of the masonry categories/subcategories.

At the same time a database with GIS interface is being built, through which all gathered information will be presented in a georeferenced tool of view and manage.

UNINA-a

In the framework of WP1, the research group UNINA-a analyzed experimental data on brick masonry assemblages gathered from *Database Murature UNINA-DIST* (URL: http://www.reluis.it/dbuninadist/), which is an experimental data archive produced within the research project ReLUIS-DPC 2005–2008. Data sets were compared to ranges recommended by the Italian Building Code Commentary (i.e., Circ. Min. 02.02.2009 n. 617), in view of its next updating (Parisi and Augenti, 2012a). Statistics of mechanical properties of masonry assemblages were taken into account to investigate uncertainty in seismic capacity of masonry buildings with box-type response (Parisi and Augenti, 2012b; Parisi, 2013). The former database was enlarged, released in English and renamed as *MADA* (*MAsonry DAtabase*) (Augenti et al., 2012). *MADA* is the deliverable UNINA-a-04 and can be accessed from the *Design* page of the ReLUIS website at:

- http://www.reluis.it/index.php?option=com_mada&Itemid=160&lang=it (Italian);
- http://www.reluis.it/index.php?option=com_mada&Itemid=160&lang=en (English).

b. Experimental evaluation of in-plane behaviour of double-leaf stone masonry with different degrees of transversal connection.

UNIGE-b

Historical masonry walls are often characterized by different layers, which are eventually connected through transversal header elements (*diatoni*). The structural behaviour of multi-leaf masonry panels is deeply influenced by the presence of these transversal connections, which can lead, or not, to a monolithic behaviour of the wall. The research is, in fact, aimed to the study of the role of the transversal connections ("*diatoni*") and numerical, analytical and experimental studies were carried out.

Starting from the results of an in-situ test-campaign carried out by the authors in the past, comprised of 27 in-situ diagonal compression test, it has been possible to notice some discrepancies between the experimental results and the suggestions of Italian Technical Code, about the influence of transversal connections in multi-leaf masonry panels.

During the first year of the project the research has been focused on the literature review related to the in-plane shear behaviour of masonry panels: test methodologies for the evaluation of shear strength and stiffness were analyzed, together with numerical results.
The different approaches proposed by International codes or Guidelines have been followed: in particular the actual Italian Technical Code for Constructions suggests to take into account some constructive details for characterizing the mechanical properties of each kind of masonry. In the case of having good transversal connections, the Italian Code proposes to increase the strength parameters but not the stiffness values.
During the second year an analytical model was developed to estimate the influence of transversal connection in double-leaf masonry panels. Subsequently 8 double-leaf masonry panels were built to validate the analytical model by diagonal compression tests. The tests were performed in the Structural Laboratory of the University of Genova. The classical test setup was modified to better understand the behaviour of leaves.
During the third year other 4 double-leaf masonry panels were built and were tested as done previously.
The results show that if the leaves are very similar, the transversal connections are able to increase the resistance of the panel (Bozzano et al., 2013; Brignola et al., 2013). In fact a double-leaf masonry panel, with a very high numbers of *diatoni*, is similar to a monolithic panel, with a higher value of strength than that without *diatoni*. If the leaves are very dissimilar, the transversal connections cannot increase the ultimate load because when a leaf breaks, the connections transfer the load, as an impulse, to the other leaf, causing the failure of the panel. The tests confirm the factor 1.3 proposed by the Italian Code to increase the strength of masonry and the absence of correlation with deformability parameters. Furthermore, the correction factor should not be applied when the leaves were significantly different. This aspect is in contrast with what assumed in the analytical model and with the common thought.

c. Experimental evaluation of the effectiveness strengthening/improvement techniques of masonry.

POLIMI-a

A research program on the durability of different composite materials applied on different masonry typologies was started in collaboration with UNIPD. The composite materials are nets of carbon fibres applied with lime mortar and cementicious mortar and carbon and glass fibres applied with epoxy resin. The used masonry is part of full scale models in full brick masonry and sandstone masonry built in an outdoor laboratory in Milan. In such environment it is possible to evaluate the influence of different aggressive environmental conditions, such as temperature, humidity and, mainly, salt crystallization (due to Sodium sulphate).
The same tests were carried out in an indoor laboratory on single units and small masonry assemblages, evaluating in a controlled environment, the effect of humidity, of thermal cycles on the delamination of the composite materials from the masonry support and salt crystallization tests. The bond between the composite materials and the masonry substrates was evaluated with the pull-off tests, as an easy and fast test method.

UNIPD-c

The research of Unit UNIPD-c was mainly devoted to the experimental investigation of behaviour and effectiveness of composites materials externally applied on masonry and timber structures. Common reinforcements like carbon or glass fibres embedded in epoxy

resin have been adopted, as well as emerging products, like steel fibres coupled with inorganic matrices or bio-composites (hemp fibres). Activities ranged over local (bond behaviour) and structural scale (masonry vaults and timber floors).
The first research topic was focused on the bond behaviour of Fibres Reinforced Polymers (FRP) applied on masonry, in the case of actions normal to the surface. The second research topic was aimed at investigating the bond of FRP applied on solid clay bricks in the case of reinforcement axially loaded: forty shear tests were performed to study the debonding phenomenon, mainly from a methodological point of view; this work was part of a Round-Robin activity of the RILEM Technical Committee TC 223-MSC (Masonry Strengthening with Composites materials), which involved twelve European Institutions (Universities and Companies). Moreover, bond of FRP applied to five-course masonry samples was investigated through thirty additional shear tests, aimed at highlighting the possible influence of mortar joints.
The third topic involved the in-plane reinforcement of timber floors by means of carbon and steel reinforced polymers, as well as hemp textiles embedded in epoxy or vinyl matrix. Results have been compared, in terms of stiffness, strength and ductility, to other techniques such as double planking, steel diagonals, etc.
Finally, the fourth topic faced the strengthening of solid clay masonry vaults. Static tests were carried out on eight 3m span masonry barrel vaults, one unreinforced and the other strengthened using bidirectional basalt nets and unidirectional steel fibres coupled with inorganic matrix, as well as unidirectional carbon and steel embedded in epoxy resin. Two specimens were strengthened by additional transversal stiffening walls (*frenelli*). Load, except for two monotonic tests, was cyclically applied at both the quarters of the span using two hydraulic jacks alternatively working, in order to simulate the effects of a seismic motion. Test results allowed comparing the performance of various composite reinforcement systems – epoxy based or applied with inorganic mortars – in terms of failure modes, global strength and displacement capacity.

UNIPG

The main achieved results are the following:
1) Masonry Quality Index (IQM): the procedure to evaluate IQM has been updated for the case of masonry bricks or blocks; new experimental data have been inserted on the correlation diagram between IQM and τ_0.
2) Experimental investigation on masonry column: 19 uniaxial compression tests were conducted on model masonry columns. The proposed strengthening system has then been applied to the design process of the retrofitting intervention for a masonry column, belonging to a cloister portico in the city of Spello (Italy).
3) Experimental investigation on masonry walls: 13 diagonal compression and shear-compression tests have been carried out on masonry walls strengthened with a mortar coating reinforced with a GFPR grid.
4) Experimental investigation on tile arches: 8 brickwork arches, strengthened by overlapping different layers of tiles and laminates, embedded within an hydraulic mortar, were tested under a monotonic vertical load applied at the keystone;
5) Experimental investigation on panels reinforced with artificial diatoni: both laboratory and in situ test (overturning) were carried out on masonry panels reinforced with diatoni of different types.

6) Determination of shear strength of masonry panels through different tests: a new procedure has been defined to explain the shear-compression tests so that the results agree with those of diagonal compression tests on the same walls.

d. In-situ testing of shear strength of masonry:

UNICT

For the seismic assessment of an existing masonry structure it is necessary to provide mechanical parameters able to characterise the nonlinear material response of the masonry media, consistently with an adopted numerical model. In this context in-situ non-destructive and semi-destructive tests can play a fundamental role since their interpretation could allow a correct calibration of the computational method adopted in the numerical simulations. To this purpose, the double flat jacks test introduced by Rossi in 1982, has been successfully used, in the last three decades, for a reliable estimation of the masonry compressive strength and its elastic modulus. However in order to estimate the seismic vulnerability of an existing building it is also needed to obtain a reliable evaluation of the structural parameter which govern the nonlinear shear behaviour. The main goal of the research program performed by the UNICT unit was to propose, verify and validate non-destructive or low-destructive in situ tests aiming to estimate the mechanical parameters that govern the shear-sliding and the shear-diagonal collapse behaviour of masonry walls and to correlate and numerically simulate the results obtained by different in-situ tests procedures.
In particular UNICT considered the application of in-situ shear-sliding and shear-diagonal tests by means of flat jacks according to low-destructive procedures, if compared to the traditional in situ tests. During the first Reluis project, the Catania Unit had developed a shear-sliding test by means of single flat jacks. This methodology has been applied several times and has been calibrated experimentally through a comparison with more a destructive tests, (Annex UNICT -01). Further applications of this innovative shear-sliding test have been performed with reference to different masonry typologies with the aim to enrich the database and to evaluate the limits of applicability. Some numerical simulation of a shear-sliding tests, performed on a brick masonry wall, by means of nonlinear finite element platforms (LUSAS and ADINA) have been also performed. In this second Reluis project a shear diagonal test based on the use of flat jacks has been proposed, the masonry portion under investigation being subjected to the in situ vertical load. The test is generally performed by using a single non standard flat jack and possesses the great advantage to be less invasive if compared with the traditional in situ shear-compression tests. This low-invasive approach has been applied in some different masonry typologies and the results have been collected in a report (Annex UNICT-02). The proposed in situ shear-compression test has been also numerically simulated and compared with a standard diagonal-compressive test (Annex UNICT-03). A first database and comparison between the in situ tests based in the use of flat jacks is reported in (Annex UNICT-04)

UNIBG

The focus was devoted to the numerical analysis of testing arrangement, in order to detect the stress (and strain) distribution in the masonry panel consequent to the applied pressure in the vertical cuts due to flat jacks.
The testing set up was modelled according to the test specimen designed in the first year of the research and not yet built: three-headed brick masonry wallets were chosen, allowing a partial cut involving only external leaf instead of passing-through cuts, in order to perform a

less destructive in-situ test. The panel aims to represent a typical situation of masonry panel of medium thickness subjected to moderate vertical loads. The results show a very clear and uniform shear stress concentration in the horizontal bed joint near the surface of the panel, allowing for an easy determination of masonry shear strength based on in-situ test.

UNIPI

At the "Livi" masonry building (rubble stones with chaotic texture) in the public hospital of Volterra (PI), first penetrometric PNT-G tests on mortar and double flat jack tests have been performed, obtaining compression strength on mortar of 0,87 +/-0,03 MPa and of 0,80-1,10 MPa, respectively. Diagonal compression tests performed tensile strength (RILEM, Brignola et al., 2009)) of 0,05 MPa. The twin panel tests (TPT) on identical panels (120x120cm) have also been performed, acting on the ratio between vertical force (constant) and horizontal force (unilateral cycles). The tensile strength obtained results equal to 0.05 MPa and the shear stress in absence of compression results equal to 0.05 MPa, assuming a friction coefficient of 0.4. The TPT provided horizontal force-displacement laws for several levels of vertical action. It has been moreover possible to obtain the stress-strain curves in vertical direction.. The TPT then provides a greater number of experimental results and information about the mechanical behaviour of panels, involving similar costs and destructivity levels than the diagonal compression test ones.

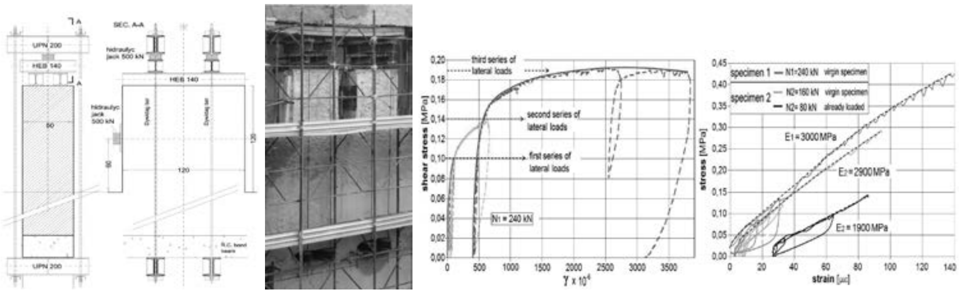

Figure 3. TPT test (a) scheme, (b) the test,(c) shear stress-strain, (d) compression stress-strain.

Together with the UNICT RU shear-diagonal tests with flat jacks were carried out on the same building. The test governs the horizontal action in unilateral cycles (the compression is given by the in situ vertical load). The advantage of this method is that it is less invasive in comparison with the traditional shear-compression tests; it also requires the same equipment of the classic tests with flat jacks (pumps, straingauges).
The set of tests allowed to build a collapse domain (shear - compression), useful for the calibration of macro-elements for numerical simulations.

e. Optimization of the diagnostic strategies and techniques

UNIPD-a

The contribution of the RU UNIPD-a included a series of static tests on three-leaf stone masonry walls, in plain conditions or strengthened with grout injections. Wall specimens were designed and built according to the original constructive techniques representative of common types of historical masonry buildings found in Italy. The performed static tests were essentially simple compression and shear-compression in-plane cyclic tests, carried out on both reinforced and unreinforced stone masonry panels. Such tests allowed characterizing the mechanical behaviour of tested samples, integrated with laboratory tests on masonry assemblages and components, namely stones, natural lime mortars and injected grouts. Moreover, several in-situ diagonal tests were performed on two-leaf stone masonry walls, typical of the Abruzzo region (Italy). Test specimens were obtained from buildings found in the towns of Onna, S. Eusanio Forconese and Tempera (Abruzzo, Italy), severely damaged after the 2009 earthquake. As done in the case of laboratory specimens, the experimental activity was integrated by destructive tests on component and reinforcing materials.

Starting from the end of the second year, numerical simulations were performed to give an adequate insight in test results, allowing a better understanding of failure modes, strain and stress distributions. Based on calibrated models, a numerical parametric assessment was carried out to define critical mechanical parameters, to highlight the limits of proposed intervention technologies and to assist the task of optimizing design.

A further topic involved definition and selection of real case studies, aimed to develop and calibrate local and global monitoring techniques (SHM, Structural Health Monitoring). Applications were designed according to integrated methodologies to minimize interventions also by using proper monitoring strategies. Static and dynamic monitoring systems were applied before, during and after the execution of strengthening interventions. Numerous case studies have been analysed during the research project, from the initial knowledge phase, to the execution of specific diagnostic investigations and, through the construction of numerical models, until the implementation and real application of state-of-the-art monitoring systems on selected monuments.

In this framework, designated case studies cover different types of problems concerning specific objectives and applied methodologies. The different identified problems are a consequence of the significant variability shown by case studies with regards to construction typologies, seismicity of the sites, quality of the construction materials and techniques, damage conditions, effects of past or recent earthquakes and effectiveness of possible performed strengthening interventions. Italian CH buildings, currently equipped with SHM systems, that have been selected as pilot projects within the present research are: (i) Roman Arena of Verona; (ii) Cansignorio stone tomb in Verona; (iii) Spanish Fortress in L'Aquila and (iv) Civic tower of L'Aquila.

Thanks to the implementation of a large number of monitoring systems to CH structures, it was possible to define a methodology of application according to the final aim of monitoring within each selected case history. Following this approach, monitoring represents an essential step in the assessment and protection processes of historical constructions, regarding, in particular, the following problems:

1. increasing the knowledge on the structural behaviour using SHM to assess strengthening needs and avoid the execution of unnecessary interventions (case study of the Arena of Verona);
2. applying an incremental approach to the execution of strengthening interventions using SHM before, during and after the implementation, validating eventually their effectiveness (case study of the Cansignorio stone tomb);
3. post-earthquake controls on severely damaged buildings using SHM to control the evolution of damage and verify the effectiveness of provisional strengthening measures (case studies of the Spanish Fortress and of the Civic Tower in l'Aquila).

WP2: Floors, roofs, timber substructures, masonry vaults and their interactions with vertical structures

a. Timber substructures

UNITN

With reference to the issue of mechanical characterization of the in-plane behaviour of existing timber diaphragms, a joint project between the UNITN RU and the research group of the University of Auckland, New Zealand, led by J. Ingham, has been undertaken. As a result, an extensive *in-situ* experimental campaign was conducted on the timber floor of a two-storey clay masonry building located in Whanganui (New Zealand). Quasi-static cyclic tests and dynamic snap-back tests were performed on both as-built and retrofitted specimens (Giongo et al. 2013a,b). Part of the campaign was dedicated to study the connections between the timber diaphragms and the masonry skeleton. Such connections, made of epoxy-grouted steel rods, were subjected to shear cyclic tests.
As regards the assessment of existing timber structures, research activities were also carried out in the field of ND test techniques, as reported in the references. Some of these activities were conducted, in co-operation with the POLIMI-b RU, in order to calibrate synthetic methods for the diagnosis of timber structures and for the evaluation of their seismic vulnerability (Riggio et al., 2013)

POLIMI-b

The POLIMI-b RU investigated the seismic behavior of timber elements, and particularly of roof structures. The tasks assigned to the unit comprised: collecting information on various cases and typologies of damage to timber roofs in the L'Aquila earthquake; the development of criteria for assessing the seismic vulnerability of these structures, and the characterization of the behavior of carpentry joints.
The activity has been devoted primarily to detail the procedure for the assessment of seismic vulnerability of timber roof structures. The procedure comprises (1) a visual exam of the structure with the aim at collecting data according to a standardized format and (2) a subsequent evaluation of a set of vulnerability indicators, from which critical elements are pointed out and a general classification of the structure can be expressed. In the final form, the indicators have been set to four, namely: the structural typology, the roof constraint system, the carpentry joints conditions, and the state of the structure, which includes different aspects like maintenance state and whether the structure had been subjected to some form of strengthening interventions. The indicators related to structural typology and quality of carpentry joints have been developed in depth, and the work in the latter stage of the research has dealt with (1) the problem of assessing the effect of interventions and (2) the definition of criteria for the constraint system. On point (1), the interventions carried out at different times on the roof of S. Michele in Vimercate have been examined, giving the opportunity of modeling and analyzing different situations, which have finally brought to a favorable vulnerability reduction. From this and from the cases previously seen, considerations for evaluating the corresponding indicator were drawn.
For point (2), beyond the albeit common case of collapse from loss of support, the interaction between the roof truss and the supporting wall may give rise to a positive or a negative seismic response, depending on the characteristics of the two elements. Considering the possible collapse mechanism within a limit analysis approach, a simple formula has been developed which permits to estimate the limit acceleration as a function of parameters like

weights, forces, and geometry of the two components. On this basis, the vulnerability level may be assessed, with computations that may be easily performed referring to parameter values derived from visual inspection. Various examples have been developed making reference to structures for which detailed analyses were available and could be used for comparing results.

b. Non-destructive diagnostic techniques for timber

UNINA-d

The RU has applied the methodology for the mechanical identification of timber members through Non Destructive Techniques, which were set up in the first two years and in the framework of previous experimental campaigns (PROHITECH 2004-2008; PRIN 2006-2008). The performed activities have essentially consisted in the following items:

- Extension of samples population, through the selection of 24 squared specimens with defects (4x4x76cm) and of 36 defect-free ones (2x2x40cm) for the execution of non-destructive (NDT; sclerometric and resistographic) and destructive tests in bending (DT);
- Analysis and statistical elaboration of all experimental results;
- Determination of NDT-DT correlations by means of both simple and multiple linear regressions.

The results of destructive tests emphasize that the bending strength of clear wood is about two times the one of structural timber. Furthermore, the main collapse modes are identified in function of extension and position of material defects. Through the regression analyses developed by using the whole database of the investigation campaign, good relationships between NDT parameters and density were found. In particular, the combined method sclerometric-resistograph (R^2_{adj}=0.68) offers greater reliability and significance of the results. It was also possible to establish that the density is well correlated with the mechanical properties of strength and elasticity of the material. The final result of the whole activity is represented by guidelines and recommendations that can define, in addition to the mechanical properties of the ancient chestnut, a procedure for the practical in situ evaluation of DT parameters through the use of NDT techniques.

c. Arches, vaults, diaphragms and their interactions with vertical structures

UNIBS

1) Roof and floor diaphragm

- Innovative solution for floor diaphragm.

The feasibility of floor diaphragms obtained with a thin slab made of natural lime mortar was assessed by means of numerical, analytical and experimental studies on reduced scale specimens.

- Roof diaphragm deformability and connection to the perimeter walls.

The influence of the floor diaphragm flexibility on the diaphragm-arch abutment rocking response was studied. Numerical studies highlighted the beneficial effect of adopting flexible, dissipative elastic-plastic roof diaphragms in reducing the seismic vulnerability of historic churches, by controlling the amplitude of the abutment rocking response and the magnitude of the seismic action transferred to both the façade and the triumphal diaphragm arch. The study was focused on the deformation demand to the roof diaphragm and on the proposal of strengthening solutions for the roof diaphragm in order to obtain the desired seismic performance (Preti et al. 2012, Bolis et al. 2013). Further experimental tests were carried out on stud connections, transferring the shear forces gathered by the roof diaphragms to the resisting walls. In order to increase the connection capacity, which is otherwise jeopardized by the reduced shear resistance of the crowning masonry caused by the lack of vertical load, a thin lime mortar slab, stiffened with plaster mesh, was cast on top of the masonry walls. The thin slab spreads the stud point load into a small shear flow acting along the interface with the masonry crowning wall, thus avoiding any connection anticipated brittle failure.

2) Rocking and the differential rocking of neighbouring diaphragm arches
A large-scale test on a diaphragm-arch under horizontal cyclic loading was carried out. The geometry of the experimental specimen reproduces a typical diaphragm arch of a single nave historical church. The test showed large ductility capacity associated to limited damage of the test specimen. Several cycles of increasing displacement were applied, up to a drift of 3%. The response curve showed a stable force-drift relationship with the typical trend of the rocking bi-linear capacity curve. The test confirmed the central role played by the tie in the system response. For increasing drift, the tie experienced a significant over-tension, up to 3 times the ideal arch thrust in "at rest" conditions. Such increment was mainly due to three phenomena: (i) the horizontal force redistribution between the abutments, (ii) the ideal arch shape variation during rocking and (iii) the relative rotation of the central portion of the arch against the abutments, which is limited by the tie stiffness.
<u>- Strength and ductility of plain stone masonry walls: role of lateral confinement</u>
Aim of this part of the research was to evaluate the deformation capacity of rubble stone masonry walls and diaphragm arch piers undergoing rocking at the base sections. A theoretical formulation of the compressive strength of both unconfined and confined stone masonry wall was proposed (Giuriani 2012). The stone masonry experimental minimum ultimate strength was evaluated on ideally confined and unconfined specimens. Further experimental tests were carried out on regularly confined walls, in which confinement was obtained with horizontal transverse bars stretched through the masonry cross section. The confined masonry ductility improved its deformation capacity up to 3 times that of the unconfined masonry. Results are in good agreement with the theoretical predictions.

3) Single leaf vaults
The vulnerability of single leaf vaults was analytically analyzed and an experimental study was carried out on a vault subjected to cyclic unsymmetrical loads. The results were used to validate a simplified analytical model based on the limit analysis (Ferrario et al. 2012): abaci and equations were proposed as a practitioner-oriented tool for the seismic vulnerability assessment of single-leaf barrel vaults with respect to the "direct" differential bending mechanism. In addition to the lightweight extrados rib technique, tested during in the first part of the project, lightweight plywood lateral spandrel elements were proposed as a further extrados strengthening solution. The effectiveness of the solution was verified by means of a full-scale experimental test. By unilaterally constraining the outward rotations of the vault, the lateral spandrel elements enforce a new four-hinges collapse mechanism characterized by a much higher strength with respect to the mechanism associated with the un-strengthened vault. The proposed retrofitting solution can be regarded as a light, reversible and cost-effective intervention that preserves the original structural role of the vault. A simple analytical procedure was proposed for the design of the lateral spandrel elements.

WP3: Testing and modeling of masonry structural systems and subsystems

a. Full-scale testing and modelling of masonry spandrel beams

UNIPV-c

The main subject of the research activity of UNIPV was the study of seismic behaviour of stone masonry spandrels. These structural components were studied both experimentally and numerically. The experimental study aimed at investigating the seismic behaviour of full-scale stone masonry spandrels, both in the presence and in the absence of a horizontal tie rod (with different lintel configurations). For this reason, an experimental apparatus had been designed in order to test full-scale masonry spandrel specimens. Information about the displacement response of the structures during the tests was provided by a high definition video acquisition that allowed to monitor all the deformation field of the specimens. This facilitated and will facilitate the calibration of numerical models.

Two slender spandrel specimens were constructed and tested. The specimens are H shaped (made of one spandrel and portions of the adjacent piers) in order to account the effects of deformability of the node zones and the interlocking phenomena that occur at the edges of the spandrel. The specimens were built with a double leaf stone masonry, the same technique of the spandrel specimens tested in the previous triennial Reluis-DPC project and of 3 full scale buildings previously dynamically tested on shake table. One of the spandrel specimens had a well-connected timber lintel. The spandrel of the second specimen was supported by a very slender timber lintel, a 25 mm thick board simply supported on the adjacent piers, whose bending stiffness was meant to be negligible compared to the masonry spandrel.

Based also on the experimental observations from past tests carried out at UNIPV and at ETH Zurich, the main damage mechanisms and strength/deformation characteristics were indentified. A numerical model was then calibrated, making use of the TREMURI software, based on a 2-node macro-element model (Lagomarsino et al., 2013; Penna et al., 2014). A simplified modeling strategy for spandrels is proposed, trying to interpret and simulate the behaviour of spandrels in terms of maximum strength, initial stiffness and inelastic response (hysteretic behaviour). Results from this process have been extended to the prediction of the seismic performance of two case-study buildings. The aim of this study was to suggest modelling strategies that are consistent with the experimental results, in order to better represent the contribution of spandrels to the structural response. The outcomes of the research are presented in Graziotti et al. (2013) and Da Paré et al. (2013).

UNITS

The results of experimental tests carried out over the first and second year of the project on samples of brick and stone wall with openings were analyzed. The tests were designed to study the behavior of spandrel beams. The samples considered in the tests are composed of two piers connected by a spandrel beam at half height of the piers so to form a H shape. The analyses of the test results consider the 4 tests conducted on samples of brick masonry and the 2 tests performed on samples of stone masonry. The three couples of tests concern the execution on the same type of masonry of tests with "weak" spandrels (non-reinforced) and "robust" spandrel (reinforced with different techniques). In particular, the spandrels of the samples of brick masonry, which have the same texture, were reinforced with two different techniques: steel angle applied at one side of the wall and joined to it by means of injected threaded bars, CFRP strips, glued on both faces close to bottom and top of the spandrel. The spandrel of stone masonry was reinforced with a steel angle.

It was then possible to compare the effectiveness of the two different reinforcement techniques considered for the same type of masonry (brick masonry 380 mm thick) and highlight the benefits of the same strengthening technique (steel angle) applied on two masonries with very different characteristics (stone and brick). In the first case, a considerable increase in shear strength of the spandrel reinforced with CFRP strips was noted (almost tripled compared to the specimen unreinforced), while the increase is much smaller in the case of the steel angle (about 1.3 times). In the case of reinforcement with CFRP strips, however, the resistance reduces very quickly after the peak with the collapse of the spandrel in correspondence of a drift equal to 3%, due to the debonding of CFRP strips from the support. On the contrary, the sample reinforced with the steel angle, after the peak value showed a very slow reduction in resistance up to reaching the collapse; in fact, before the collapse the resistance was equal to 80-85% of the peak value. The maximum displacement at collapse was equal to 6% of net length of the spandrel. The reinforcement of the stone masonry sample, made with the steel angle, showed a behavior similar to that in the sample of brick masonry but with a more marked increase in the maximum resistance compared to that of the unreinforced sample (more than doubled).
The experimental results allowed to develop a numerical model able to simulate the actual behavior of the spandrel through equivalent nonlinear springs. The model considers rigid beams with elastic-plastic springs in the middle, which simulates the shear behavior. The springs were calibrated with cyclic non-linear character. The elastic-plastic element represents the shear hysteretic cycle based on the model proposed by Tomazevic and Lutman in 1996, with a trilinear envelope curve symmetric about the origin. Its use requires as input the elastic stiffness in shear and the strength capacity. The resistance is evaluated on the basis of an adequate collapse mechanism hypothesized for the specific case. The model is able to represent the degradation of stiffness and strength and can be used in nonlinear static and dynamic analysis. The numerical simulations of the cyclic behavior describe with good reliability the behavior observed during the experimental tests.

b. Tests on masonry walls before and after strengthening with innovative materials

UNINA-a

Quasi-static cyclic tests on full-scale tuff masonry walls with an opening were performed to assess the influence of spandrels on nonlinear behavior up to collapse. The specimens had different characteristics compared to that tested during the previous research project ReLUIS-DPC 2005–2008, both in terms of spandrel configuration (i.e., with masonry arch above the opening, with both masonry arch and reinforced concrete bond beam running along the spandrel) and strengthening system applied over the spandrel. The strengthening system was based on a composite material with inorganic matrix, basalt fiber grid and vertical fiber ties inserted within the spandrel panel above the opening. The main results of the experimental program were published in Augenti et al. (2011b, 2011c); Parisi et al., (2012a). The characterization of tuff masonry strengthened with the composite system applied over spandrels was based on diagonal compression tests, which were carried out in cooperation with the research group UNINA-UR3 (Prota) (Parisi et al., 2012b; Parisi et al., 2013). Experimental results reached during the period 2010–2013 are discussed in the *deliverable* UNINA-a-02. Such results are also compared to those reached during the period 2005–2008 within a joint *deliverable* in cooperation with UNIPV-c (Magenes), UNITS (Gattesco) and two research groups from Lausanne (Switzerland) and Auckland (New Zealand).

From an analytical standpoint, first of all the research group UNINA-a investigated the mechanical behavior of running bond masonry through $\sigma-\varepsilon$ constitutive models derived in the first year (Augenti and Parisi, 2010) and $\tau-\gamma$ constitutive models derived in the second year (Augenti and Parisi, 2011). Then, the closed-form integration of $\sigma-\varepsilon$ constitutive models allowed us to define: *evolutionary* strength domains in the bending moment-axial force plane (Parisi and Augenti, 2013b); 3D strength domain under eccentric compression and shear (Parisi and Augenti, 2012c); novel moment-curvature diagrams with softening (*deliverable* UNINA-a-01). After that a smeared crack and plasticity macro-element was developed, the following activities were carried out: (1) the development of 3D nonlinear macro-element models of masonry buildings, which were implemented in a new computer program named *RAN code* (Augenti et al., 2011a; *deliverable* UNINA-a-03); (2) the evaluation of the seismic capacity of unreinforced masonry walls with irregular layout of openings (Parisi and Augenti, 2013a); and (3) the simplified multi-risk assessment of a cultural heritage building subjected to both seismic and volcanic hazards (Parisi and Augenti, 2012d).
From a numerical standpoint, the research group UNINA-a carried out nonlinear finite element simulations of the experimental tests on full-scale masonry walls with openings, by means of DIANA code (rel. 9.4). Those simulations allowed us to assess the role of the strengthening system (Parisi et al., 2011b), rocking behavior of piers (Parisi et al., accepted for publication) and fracture energies to be assumed in smeared crack models (Parisi et al., 2011a). Rocking behavior was also discussed along with sliding mechanisms in a paper concerning the simplified seismic assessment of artworks (Parisi and Augenti, 2013c).

UNITS

Two shear-compression tests were carried out on a full-scale sample of stone masonry to highlight the effectiveness of a reinforcement technique consisting in the application of a reinforced mortar coating on both sides of the wall. The reinforcement of the coating was made with a mesh of GFRP fiber-reinforced composite (glass fiber reinforced polymer). The tests relate to an original sample and one reinforced with the described technique. In parallel were also carried two diagonal compression tests on the same type of masonry and with the same type of reinforcement. The results of these experimental tests have shown the importance of the connectors passing throughout the wall linking together the two reinforced plasters. This was particularly emphasized by the shear-compression test, where, in the masonry, the compression due to shear forces is added to that due to vertical loads. The horizontal confinement ensures the cooperation between masonry and reinforcement even after the appearance of the diagonal cracks, with significant increase of both the resistance and the ductility. The diagonal compression tests, showed no signs of detachment of the plaster before reaching large post cracking deformations. On the contrary, in shear-compression tests the separation of the reinforcement occurred for load values lower than the maximum load and this has resulted in a maximum shear resistance smaller than expected on the basis of the results of the diagonal compression tests and a limited ductility.
The numerical simulation of the shear-compression tests carried out with ABAQUS allowed to better highlight the significant role of the connectors between the two reinforced coatings. The calibration of the numerical model and the definition of the parameters to be used in the numerical simulation were performed with reference to the experimental results obtained with the shear-compression and diagonal-compression tests.

c. Reconstruction of historical masonry

UNIRC

The in-situ experimental activity that was originally planned at the beginning of the project was not carried out because other opportunities for in-situ experimentation have arisen. As part of the experiments of the RELUIS triennial project (2011-2013) three cycles of in situ tests were carried out, prepared by the M.A.RE. University Laboratory of the Department PAU, Mediterranean University of Reggio Calabria. The first aim of the tests was to analyze the stability of a wall under the variation of the constraints, to which it is subject (from the beams of floor, the chains, the curbs and the roof). Second purpose was a quality control (also in the construction phase, in the first experimental case) of door and window lintels, that are the first elements to suffer damage in the mechanisms of the first and second mode, and whose failure, especially out-of-plane, is almost always the first cause of damage for building and consequently of its collapse. The in situ test, implemented in the course of three year research, was a static incremental type and was performed with a simple instrumentation, consisting of a steel reaction frame and a hydraulic jack to apply a horizontal force. The displacements and deformations on the walls were measured and monitored by an integrated data acquisition system. After the first experiments, the data acquisition system has been simplified, having noted that during the experiments the monolithic behavior of the walls.
The main feature of this in situ mechanical test (classifiable as a non-destructive, if performed in the construction or restoration phases) is the low cost and ease of execution. A house in Gallico (Reggio Calabria, 2011), a home-warehouse of Reggio Calabria (2012) and a house in San Calogero (Vibo Valentia, 2013) were subject to testing. In each of these three cases the forces were imposed on the lintels of doors or windows.

d. Development of provisional interventions for cultural heritage buildings

UNIPI

The UNIPI RU carried out numerical simulations of preventive provisional interventions applied to a couple of case studies of churches in the district of Pisa: San Frediano in Pisa and San Verano in Peccioli. They consist in introducing steel ropes as chains or horizontal St. Andrew's cross-bracings to reduce the relative motions of the column capitals and walls at the abutments of arches and vaults. Classical linear dynamic analysis and kinematic collapse evaluations have been also performed. Thus introducing a macro-model for the simulation of cross vaults throughout diagonal rods, nonlinear dynamic analyses have been performed (accelerograms within NTC 2008 of Pisa area); such analyses have shown an increase of seismic reliability with fast and cheap provisional interventions, reversible and with reduced visual impact. The aim is to protect and temporarily use a large real estate set of cultural heritage buildings in seismic risk with limited investments, pending definitive consolidation works.

5 DISCUSSION

In this section some issues related to the development of the activities and difficulties or deviations from the initial research program are reported.

UNISR

The activity has been devoted to prepare the document entitled "Guidelines for the identification of the seismic damage mechanisms through the use of simplified abacuses".
The RU succeeded to achieve only partially the above-mentioned results. In fact, for both the objectives of the three-year program, despite having collected a considerable amount of information and examples from many historical urban aggregates, the RU has not come yet to a meaningful synthesis in the form of a guideline. This partial mismatch in the program was caused by the activities generated by the earthquake of 2012 on which the Research Unit of Syracuse was concentrated in the last year.

ROMA3

The purpose of ROMA3 RU was to address the issues related to the seismic assessment of local mechanisms in masonry structures. The activity stemmed from a critical analysis of the assessment procedures proposed in the national code and dealt with the following issues:

1. Definition of the local mechanisms;
2. Definition of the capacity curve;
3. Evaluation of seismic demand.

Concerning the first issue, a modelling approach has been developed for the simulation of local mechanisms which makes it possible to derive a distinct element model from the survey of masonry morphologies on existing structures (de Felice and Mauro, 2010). A wide number of numerical simulations have been carried out by referring to a sample of real connections between façade and transverse walls and by comparing the results obtained with the actual damage induced by seismic events (de Felice *et al.* 2013).
Regarding the second issue, pushover curves of multi-leaf masonry walls have been derived together with a correlation of the seismic capacity with semi-empirical indexes proposed in the literature for quantifying masonry quality. Eventually, the effectiveness of strengthening interventions currently used in practical applications has been evaluated (Mauro and de Felice, 2012; de Felice *et al.* 2013). In addition, the effects of externally bonded composite systems on the seismic response of masonry arches have been investigated. The constitutive relation for the reinforcement has been calibrated by referring to experimental tests (Valluzzi *et al.,* 2012; de Felice and Malena, 2012; de Felice and De Santis, 2010), included in a modeling strategy based on fiber-beam section and finally adopted for performing numerical simulations (De Santis and de Felice 2012, 2013).
Concerning the third issue, the seismic demand/capacity ratio for local mechanisms has been evaluated by means of non-linear time-history analyses carried out by adopting two modeling strategy calibrated on the basis of shake table tests results (Al Shawa *et al.* 2011, 2012). The models account for the impact of the façade with transverse walls and for the effects of imperfections. The results obtained make it possible to derive estimation for the safety level related with the assessment procedures proposed in the national code. Eventually, the use of non-dimensional rocking envelopes has been investigated as an alternative to the SDOF approaches currently adopted for seismic assessment of local mechanisms in masonry structures, de Felice *et al.* (2013).

ROMA1a

All three deliverables have been prepared as planned.
The “Report on the experimental tests and guidelines for the estimation of the coefficient of restitution” comprises a plot of all experimental pushover curves and the response parameters of all free-vibration time histories. The step-by-step analytical procedure for the calculation of the closed-form coefficient of restitution is presented within the guidelines.
The report on the “Role of energy damping varying boundary conditions and comparison between time-history analyses and code static procedures for the assessment of local collapse mechanisms (Circolare 02/02/2009, § C8A.4)” presents the intensity measures of all records used in numerical time histories, the response parameters of all dynamic analyses and all the comparisons to code procedures.
The “Report on the estimation of strength parameters of masonry types” is referred to clay brick and tuff masonries, which are the most investigated in the technical literature. In the manuscript all the literature data are presented in thematic tables.

UNINA-e

The activities carried out fall within the two initially proposed study fields (Casapulla 2013):

1) Dynamic analysis of the rigid block under seismic actions.
2) Limit analysis of in-plane and out-of-plane local damage mechanisms of masonry structures with frictional behaviour.

POLIMI-a

Experimental techniques development and experimental assessment of the mechanical properties and of strengthening techniques of the different existing masonry typologies is one of the goals of the project, following the provisions of the new code NTC2008.

UNIFI

With respect to the initial program, it was considered appropriate to prolong the activities conducted during the second year, in order to increase the sample data available. Therefore a proposal for an amendment of the masonry typology of C.M. 2009 and of its mechanical properties (always with reference to Tuscany) could be formulated. The study of reinforced masonry panels using traditional techniques (reinforced concrete) and/or innovative techniques (fibro-concrete plaster, FRP) is currently pending. The experimental campaign initially scheduled on the walls of the town of Castelnuovo, a hamlet of the San Pio delle Camere (AQ) municipality, is subject to implementation of the Reconstruction Plan drawn up by RU UNIFI, at present under approval.

UNIBG

The activity of the third year was strongly conditioned to the gap in the activity of the Principal Researcher, due to health reasons. The experimental testing was suspended, and only the numerical verification has been completed.

UNIPI

The time-table of activities has been substantially respected, providing a technical report on the use of innovative compression-shear tests (TPT and vertical jacks) with UNICT and a document of guidelines for preventive provisional interventions in churches.

UNITN

The activities fulfil the expected results regarding the use of reversible technologies for the strengthening of timber floor/roof structures for in-plane and out-of-plane actions and, partially, the in-plane modelling of timber diaphragms.
The research activity in the field of assessment of timber elements and sub-structures resulted in a conclusive document, which also contains the outcomes of the research work (especially regarding the evaluation of seismic vulnerability of existing timber structures) carried out in close co-operation with POLIMI-b.

POLIMI-b

For what concerns the initial programme, the activities fulfil the expected results regarding the investigation on the seismic behavior of timber elements, particularly of roof structures, and the assessment of their seismic vulnerability.
The final work has been reported in Parisi (2013) and Parisi et al. (2013a, 2013b). Additionally, an abstract has been proposed for the 10th US-National Conference on Earthquake Engineering of 2014 (Parisi and Chesi, "Seismic vulnerability of traditional buildings: the effect of roof-masonry walls interaction", paper 1074, 2014) and an article intended for an international Journal is in preparation by the same authors, with tentative title "Seismic vulnerability of timber roofs: criteria, method, and examples". Finally, the report "Guide for evaluating the seismic vulnerability of roof structures" is the deliverable product prepared for the completion of the project.

UNINA-d

According to the program, the following main goals have been reached:
- Mechanical characterization of old chestnut timber and clear wood by performing compression and bending tests;
- Evaluation of defects influence on the mechanical behaviour of timber;
- Idea for a practical procedure aimed at the estimation of density together with strength and elasticity characteristics of chestnut timber, based on the use of non-destructive simple and combined methods;
- Definition of guidelines for practical applications that provide criteria for diagnosis and mechanical in situ identification of timber members.

UNICT

The activities carried out are in accordance with the provisions in the initial programe.
The comparison between different shear tests and the discussions with other research units highlighted some interesting questions that need further research activities.

UNIBS

All activities were carried out as initially planned. The only exceptions are listed below:
- no full scale in-plane shear test was carried out on natural lime mortar floor diaphragms; the corresponding deliverable "Innovative floor diaphragm" was therefore not produced;
- an additional solution for the strengthening of single leaf vaults was proposed and its efficiency was experimentally assessed through a full scale test;

- a theoretical formulation for the evaluation of the compressive strength of both unconfined and confined stone masonry wall was proposed and verified through experimental tests performed on both confined and unconfined specimens.

6 VISIONS AND DEVELOPMENTS

The seismic assessment of existing masonry structures is a topic of relevant importance due to the significant vulnerability and the high number of these buildings, particularly in Italy. It is a complex and wide issue, in which many problems are still open in the field of seismic analysis, verification procedures, techniques for the strengthening interventions and availability of reliable and accurate standards or guidelines.

The scientific knowledge and the operational methodologies developed so far, including the results of the first DPC-ReLUIS project (2005-2008), still require further developments, both in terms of experimental and numerical validation of methods of analysis and actual applicability to the different geometric and typological configurations of the historical built environment.

Many complex configurations, as for example mixed masonry and reinforced concrete buildings, aggregated units in historical centres, cultural heritage specialistic types (churches, towers, fortresses, etc.), can now take advantage of new advanced methods for the seismic assessement, if compared with ten years ago. However, still there are many open issues.

The recent experiences in the field of strengthening intervention, in particular after destructive earthquakes (like in L'Aquila 2009 and Emilia 2012), show that all this knowledge is not well consolidated and learned by practitioners and stakeholders. Methods of analysis, as well as strengthening techniques, are often used and applied in a wrong way, and these facts can induce ineffective or even negative results.

In addition to the topic of existing buildings, the problem of designing new masonry structures and elements is also very important, both in the case of using modern masonry for structural elements and when this material is adopted for non-structural elements, like cladding or partition walls. In particular this last application, both for residential, commercial or industrial buildings, is widely spread for a number of reasons, related to cost, availability, durability, thermal-acoustic performance. The damage observation after the last earthquake in Italy and all over the world has shown that damage in partition and infill walls is a serious problem not only for the life safety performance level but, in particular, for the direct and indirect losses.

Another very important topic is to face the problem of risk mitigation by a vulnerability assessment at territorial scale; in particular it is necessary to improve:

- the knowledge on the structural characteristics and performance of the different typologies of masonry buildings, considering the Regional and local specific features;
- the reliability of fragility curves and methods for the forecast of seismic damage, in particular for buildings belonging to the highest vulnerability classes (in which local out-of-plane mechanisms usually take place), even with reference to low to medium seismic intensity actions.

In the case of masonry buildings, these issues are even more complex due to the presence of aggregated buildings, in which each unit interacts with the adjacent ones. Post-earthquake damage assessment data, in particular after L'Aquila (2009) and Emilia (2012) earthquakes, could be very useful to improve the knowledge and the methodologies of seismic assessment of sigle buildings or risk assessment at territorial scale.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 99-141, doi: 10.14599/r101303, © 2015 Doppiavoce, Napoli, Italy

STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES

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INTRODUCTION

In recent years, the research in the field of seismic engineering applied to steel structures has produced a huge amount of results and advances in the state of knowledge that, opportunely integrated, allow the maintenance and review of the technical codes for design and construction. In this context, the main objective of Line 1 "Aspects in the Seismic Design of New Buildings" is the formulation of proposals for updating the Italian standard code for construction, NTC 08, with regard to the structures made of steel and composite steel - concrete in seismic zone. In particular, this line is proposed to amend and/or supplement the current design rules with reference to types of structures already present in the current regulations as well as to innovative structural types.
In the field of steel structures the new Italian code (NTC 2008), which is very close to European EN1998-1, is liable to be updated and improved in several aspects. With this aim Task 2.1.2 is devoted to deepen the main open issues related to seismic design of steel and steel-composite, that are: material overstrength, local ductility, design rules for connections in dissipative zones, behaviour factors, capacity-design rules, design of concentrically braced frames, dual structures, new structural types, bridges.
Therefore, the research is developed by 10 Research Units (RU), each one composed by people belonging to 10 Universities and engaged on different topics. The research units involved and the specific related tasks are indicated in Table 1.
All the RU participating, have a relevant background of both theoretical and experimental knowledge, needed for achieving the task objectives.
In general the RU activities are articulated in three main jobs: 1) State of the art; 2) Numerical and experimental investigation campaigns; 3) Design guidelines.
Hereafter the research activity of each unit is summarized, describing the main aspects developed and outcomes obtained.
The following report of the research is divided in two parts: Part I devoted to steel structures and Part II devoted to Steel-concrete composite structures. In particular Part I gathers the contribution of the research carried out by the units UNINA-ING, UNITN, UNINA-ARCH, UNISA, UNICH, UNINA2, UNIPI, specifically dealing with the bare steel structures; besides Part II gathers the contribution of the research carried out by the units UNISANNIO, UNITS, POLIMA, specifically dealing with the concrete - steel composite structures.
Of course it is to be underlined that the research activities are carried out on the basis of a complementary effective collaboration among all the RUs, also stimulated by the planned coordination meetings.

Table 1. Research units and related tasks.

Research unit	Affiliation	Research Coordinator	Task
UNINA-ING	University of Naples Federico II - Engineering	Federico M. Mazzolani Beatrice Faggiano	Concentric bracing systems
UNITN	University of Trento	Riccardo Zandonini	High strength steel in seismic zone
UNINA-ARCH	University of Naples Federico II – Architecture	Raffaele Landolfo	Steel members
UNISA	University of Salerno	Vincenzo Piluso	Steel connections
UNICH	University G. D'Annunzio of Chieti	Gianfranco De Matteis	Structures equipped with shear panels
UNINA2	Second University of Naples	Alberto Mandara	Behaviour factors for moment resisting steel frames
UNIPI	University of Pisa	Walter Salvatore	Analysis of the effects of material mechanical properties
UNISANNIO	University of Sannio	Maria Rosaria Pecce	Concrete-steel composite members
UNITS	University of Trieste	Claudio Amadio	Concrete-steel composite connections
POLIMA	Marche Polytechnic University	Luigino Dezi	Seismic behaviour of bridge decks with concrete-steel composite cross section

RESEARCH PROJECT
DPC - RELUIS
2010-2013
LINE 1. ASPECTS IN THE SEISMIC DESIGN OF NEW BUILDINGS
TASK 2. STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES

PART I - STEEL STRUCTURES

I 1. CONCENTRIC BRACING SYSTEMS [UNINA ING]

I.1.1 Background and motivation

The structural design in seismic areas is a constantly and rapidly evolving theme. With regards to steel construction, the variety of possible structural typologies represents a specific feature. From one side, this is a richness, since an assortment of design solutions are offered, from the other side it represents a challenge for the development of a comprehensive and reliable code, since many different issues should be addressed and solved. In recent years, the research in the field of Earthquake Engineering, also applied to steel structures, has provided many advances in the state of knowledge. Based on these, the current Italian code for constructions (NTC 08 and Circular 2009; M.D., 2008; M.C., 2009), largely inspired to Eurocode 8 (CEN, 2005), is susceptible to undergo significant changes and/or additions, especially concerning the bracing systems, aiming at considerable improvements.
In the perspective of a critical review of the current Italian technical code for the seismic design of steel structures, the attention is focused on concentric braced frames. The analysis of the design requirements is performed, with the purpose to evaluate the efficiency and consequently to suggest some possible modifications that better reflect the actual behaviour of study structures.

I.1.2. Research structure

The task objective is the optimization of the seismic design criteria for steel bracing structures. The goal is achieved through a preliminary careful study of the technical/scientific background of the current code, followed by ad hoc numerical and experimental analyses aimed at filling the gap of knowledge. The design guidelines able to provide additional criteria and/or amend the current rules are the main result.
The first part of the planned activity is a literature review for the collection of technical/ scientific data in order to identify the weaknesses and critical points of the standard code, examining the recommendations that affect the choice of structural elements. Static and dynamic non-linear structural analyses are carried out on selected concentrically braced frames purposely designed for the performance evaluation, in order to both emphasize deficiencies and address potential improvements to the code. As final result, for the examined seismic-resistant typologies, indications aimed to simplify the design procedures, even assuring the adequate level of safety under seismic actions, are provided.
Going more into details, it is known that within braced structures the dissipation of seismic energy is entrusted to the brace diagonals, whereas the other structural elements (columns, beams and connections) must remain elastic. In non-linear field, first, compressed diagonals undergo buckling phenomena, then, yielding of tensile braces may occur. It is obvious that yielding of tension braces can be achieved only if the other structural elements have a sufficient resistance to remain stable and elastic. Even if the design objective is the simultaneous yielding of all the diagonals of the building, frequently concentration of damage in the braces located at one or few storey levels occurs.
In the Italian code NTC 08 at the ultimate limit state of design only tensile diagonals are considered as active. An exception is represented by V-bracing, where also buckling of compression braces should be checked. The previous proposed code OPCM 3274 (M.D., 2005) considered two limit situations: a) in the elastic field of behavior both tension and compression braces were considered in the analysis; b) at the ultimate limit state, the compressed braces were assumed to have buckled, while the tensile brace were able to make equilibrium to the seismic forces.

The goal of NTC 08 to have all the tensile braces in the plastic range, thus both maximizing the dissipation capacity and having uniform damage distribution, would be achieved by limiting the difference between the maximum and the minimum overstrength coefficient of braces, it being the relationship between the tensile plastic resistance and the applied load, within 25%. However this requisite almost always leads to very large overstrength of braces at the upper stories of the buildings, especially in case of V-bracing systems.
With these premises the critical analysis of the NTC 08 design methodologies is carried out in two phases. The first one consists in the design of typical steel braced structures, chevron and X-braced, by means of linear analyses, both static and dynamic. This phase is intended to identify the critical points of the prescriptions given by code, with particular reference to the operative applicability of the proposed procedures and the actual possibility to select braces sections. In some cases, the examined structures are also designed according to alternative design procedure purposely proposed. The second phase is the evaluation of the seismic response of structures designed both according to NTC 2008 and proposed alternative procedures. To this aim, the behaviour factors, the failure mechanisms, the effectiveness of capacity design criteria, the non-dimensional slenderness factor of bracing elements and the over-strength factor of structural members are evaluated by non-linear static analyses.

I.1.3. Main results

The study structures with typical chevron and X-braces have 3, 6 and 10 floors and they are designed for high seismicity zones (a_g = 0.35g). Each geometry is designed by both linear static and dynamic analyses. In case of chevron braced structures, the design is carried out by considering two solutions for braces profiles, such as circular hollow sections and HE sections; whereas, for X-braced structures only HE sections were taken into account.
With reference to the purpose of the research, the influence of different prescriptions provided by NTC 08, such as limitations on the normalized slenderness of the diagonal braces, capacity design criteria and rules for uniform dissipative behaviour in elevation, is evaluated.
The results of the design of chevron systems with HE braces showed that the values of the overstrength factor Ω are particularly high and, sometimes, greater than the behaviour factor of this typology for high ductility class (2.5). Therefore, the design of non-dissipative elements (beams and columns) is excessively penalized. In addition, due to the low demand at the upper floors, the rules for the uniformity of the overstrength factor in elevation implies great difficulties in the selection of the diagonals profiles. To this end, the following alternative design procedures are introduced: (A) evaluation of the coefficient Ω with respect to the brace buckling resistance, (B) exclusion of the top floor from the check of the uniformity of the overstrength factor, (C) upper limitation of the Ω factor to the behaviour factor in the application of capacity design criteria. The design according to the three different approaches allows to limit the values of the Ω factor, with a consequent reduction (up to 25% for approach A) of the structural weight. Also for X-braced systems, the limitations of the normalized slenderness together with the uniformity of Ω factor imply significant difficulty in the selection of the diagonals profiles, especially in the case of 10-storey structures.
The response of the designed structures, evaluated through nonlinear static analyses able to take into account the non-linear behaviour of diagonals (Georgescu, 1996, Tremblay, 2002), showed that the prescription provided by the NTC 2008 does not allow to obtain global mechanisms. In particular, in the case of chevron systems, the collapse always occurs for the premature failure of the beam due to the buckling of the compression diagonal. Therefore, it could be assumed that the design rules for beams, which consists in considering yielded the tension diagonal and buckled the compressed one with a residual strength equal to 30% of its tensile strength, is not conservative. As a result, the exhibited collapse mechanism implies

behaviour factors, calculated as defined in (Uang, 1991), in the range from 2.3 to 7.0. In particular in the case of 10-storey structures, designed according to the NTC 08, the behaviour factor values are smaller than the one proposed by code. Concerning the alternative design procedures, the approach A provides very small (1.7÷6.2) behaviour factors, in many cases smaller than the one given by code. On the other hand, approaches B and C give behaviour factors similar or slightly smaller (B: 2.1÷6.0, C: 2.3÷6.6) than those obtained for the structures designed according to NTC 08.

In case of X-braced systems, even if a local mechanism always occurs, the code prescriptions seem to provide better results. In facts, the collapse occurs after the yielding of the majority of diagonals (>60%). The obtained values of behaviour factors (4.5 to 11.7) are always greater than the one provided by the code with high overstrength contributions (2.12-3.88).

I.1.4. Discussion

The research purposes, as the assessment of the prescriptions provided by the NTC 08 on Concentric Bracing Frames (CBF), such as the limitations on the normalized slenderness of the diagonal braces, the capacity design criteria and the rules for uniform dissipative behaviour in elevation have been achieved, consistently with the proposed research program.

I.1.5. Visions and developments

Further developments of the research on the seismic behaviour of concentric St. Andrew's cross (X) and chevron (V) bracings should be:

(CBF-V)
- Assessment of the influence of the compression brace on the beam behaviour;
- Analysis of the calculation model with tensile brace only without limitation on the normalized slenderness;

(CBF-X)
- Optimization of the relationship for calculation of the fundamental vibration period;
- Introduction of specific design rules for top storeys of multi-storey frames;
- Analysis of the influence of the compression brace on the system behaviour;
- Introduction of a new calculation model based on both bracings;
- Study of the connection type influence on the system behaviour;

(CBF-X and CBF-V)
- Evaluation of the q-factor on the basis of Incremental Dynamic Analysis;
- Definition of new slenderness limits for compression braces.

I.1.6. Main references

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Macillo V., Castaldo C., Fiorino L., Formisano A., Faggiano B., Mazzolani F. M. (in stampa). "Evaluation of the italian seismic code for the design of concentrically x-braced steel structures". *7th European Conference on Steel and Composite Structures EUROSTEEL 2014*, Napoli, Italia, 10-12 Settembre 2014.

I 2. HIGH STRENGTH STEEL IN SEISMIC ZONE [UNITN]

I.2.1 Background and motivation

The use of Circular Hollow Sections (CHS) and steel-concrete Composite Filled Tube (CFT) sections recently has had a significant development both for the excellent structural and architectural properties and the rapid development of end-preparation machines. Despite of this, the use of high strength steel (HSS) circular hollow sections (CHS) are still limited in the construction industry. Moreover, although Eurocode 3 Part 1-12 (CEN, 2007), extends its scope to steel grades up to S690/S700MC, restrictions in the application exist at the material, structural and design levels.

Therefore the research aims to promote new products, HSS-CHS, in order to cover the gaps and uncertainties in both Italian (NTC, 2008) and European (Eurocode 3, 4 and 8) codes, in view of new market opportunities.

I.2.2. Research structure

The project aims at developing performance-based design approaches, for extending the capacity design to HSS-CHS structures to prevent collapse under earthquake loading. To this purpose both analytical and experimental know-how are intended to be gathered. The ambitious targets are to increase the structural performance of steel structures, to reduce weight and construction costs for buildings subjected to exceptional load.

The investigation will be both experimental, analytical and numerical through advanced finite element simulations, in order to make full use of high strength steel ranging from S500Q/S500MC to S690Q/S700MC according to the new Eurocode 3 Part 1-12 (CEN, 2007), for structural tubes ranging from 2in to 24in, with D/t > 30, which nowadays represents an upper limit for structural applications.

The research program is articulated in the following phases:

Phase 1: State of the art review- i) experimental test on HSS, brace-beam-to-column joints with HSS-CHS columns and elements subject to earthquake; ii) design procedures, like the capacity design and displacement based design for HSS joints.

Phase 2: selection, design and numerical modelling of brace-beam-to-column joints with HSS-CHS columns and elements to be tested, they being part of a purposely designed reference building.

Phase 3: i) mechanical characterization of materials of the specimens; ii) planning, preparation and execution of tests on HSS columns, brace-beam-to-column joints with HSS-CHS columns and elements subjected to monotonic, cyclic and random loads.

Phase 4: i) calibration of 2D-3D local and global numerical models; ii) development of parametric numerical analyses; iii) definition of design rules and proposal for both the Italian and European codes.

I.2.3. Main results

The preliminary activity carried out is the collection of the state of the art according to the planning above described.

Then the specimens are selected from “actual” study cases, in order to test in laboratory realistic elements and or substructures (i.e. brace-beam-to-column joints). The reference building is a steel-concrete composite structure for offices and meetings, with concentric diagonal bracings placed in both longitudinal and transversal directions. Columns, HSS-CHS, made of HS S690 steel, have variable diameter and thickness along the height.

Once designed the reference building both nonlinear numerical analyses and experimental tests are carried out. In particular, both non-linear static (pushover) and dynamic incremental analysis (time history) aim at evaluating the behaviour factor and the inter-storey drifts at the formation of plastic hinges in the braces at the top floors. Besides, the experimental program (Table I.2.1) consists in 7 tests: 5 tests on specimens with standard braces (UPN180) and 2 tests on specimens with improved braces (UPN 180 - Dog Bone). Moreover, two different constant amplitude loads, equal respectively to 25% and to 50% of design buckling resistance ($N_{b,Rd}$), are applied on the top of the columns: $0.25N_{b,Rd}$ corresponding to the collapse in the diagonal bracing in tension, $0.5N_{b,Rd}$ corresponding also to column buckling. All the test results are shown and compared in Figure I 2.1.

Table I.2.1. Experimental program.

	STANDARD SPECIMEN	IMPROVED SPECIMEN
1	Monotonic – S – $0.50N_{b,Rd}$	
2		ECCS – I – $0.50N_{b,Rd}$
3		Random – I – $0.50N_{p,Rd}$
4	Random1 – S – $0.25N_{b,Rd}$	
5	ECCS – S – $0.50N_{b,Rd}$	
6	Random2 – S – $0.50N_{b,Rd}$	
7	Constant-amplitude $D=8e_y$–S–$0.50N_{b,Rd}$	

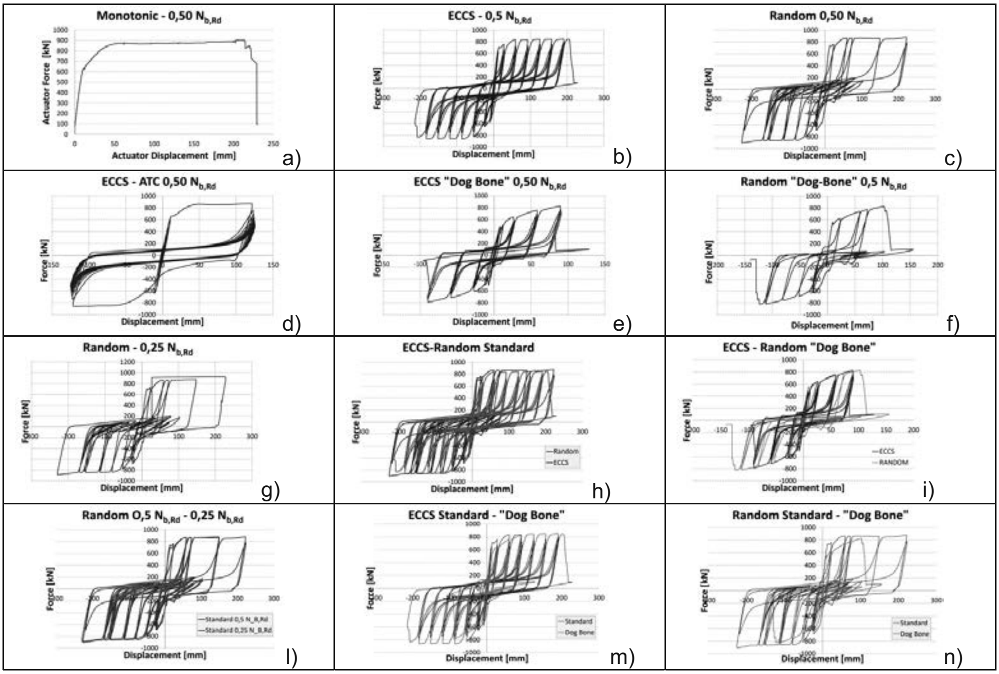

Figure I.2.1. Results of experimental tests.

The monotonic test shows a tri-linear trend with loss of stiffness when the applied force is approximately 600kN due to slipping of bolted connection.

In all experimental tests the collapse is due to the cracking of diagonal bracing in tension. Only in the tests with $0.5N_{b,Rd}$ a residual deformation of 3mm is recorded in the columns.

Hysteresis loops are characterized by the classical pinching phenomenon due to buckling of brace (Figs I 2.1 b-g). With regards to Random tests, the “standard” specimen curve shows a plateau at about 850kN after yielding, while the "improved" specimen curve shows a lower plateau at about 600kN, owing to the weakening of the section, and a subsequent hardening, due to the plastic redistribution in the braces in tension, until collapse at about 850kN. Moreover Dog Bone specimens reach only about 120mm maximum displacements, while standard specimens about 230mm (Figs. I 2.1 m,n). Definitely the energy dissipated by standard specimens is greater than for Dog Bone specimens. In both specimens collapse is due to crack of the net cross-section at fasteners holes. Maximum values of forces and displacements (Figs. I 2.1 h-i) are similar in both Random and ECCS tests. The normal force on columns does not affect the overall behavior of the system (Fig. I 2.1 ℓ).
Experimental tests are simulated by the OpenSees software with the aim to calibrate a model for a numerical analyses campaign on the reference building (Fig. I 2.2 a).

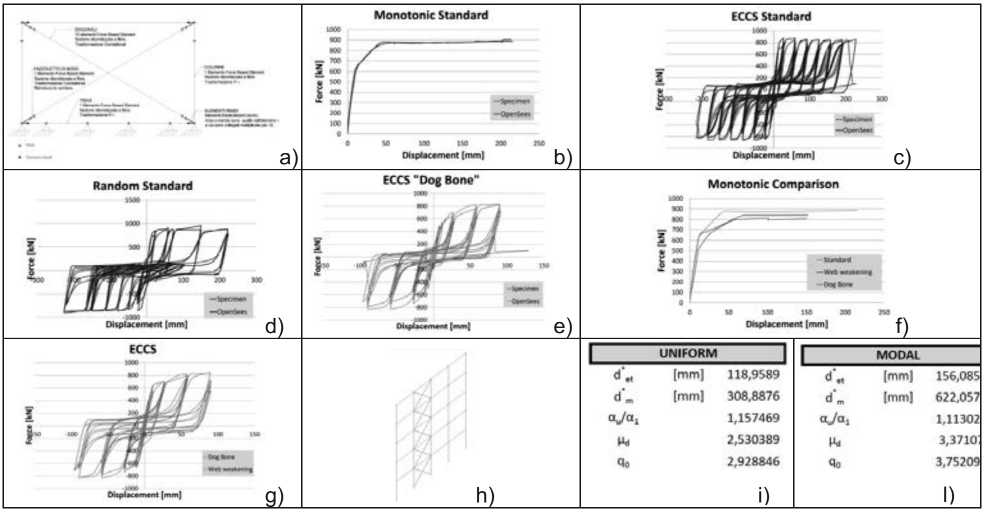

Figure I.2.2. Experimental vs numerical results.

Braces, beams and columns of the specimens are modeled with force-based (FB) fiber beam-column elements that permit spread of plasticity along the element. Moreover the geometric second order effects are taken into account. Figures from I 2.2 b to e show the comparison between numerical and experimental results. It is possible to see how the global behavior of all tests were reproduced satisfactorily by OpenSees. Given the unsatisfactory performance of Dog bone specimens a different kind of weakening, localized in the web of braces, is numerically studied. Figure I 2.2 f shows the monotonic response of the new numerical model that results more rigid than the model with Dog Bone. The cyclic test of the model with web weakening is better and the energy dissipated is higher compared to the Dog Bone weakening (Fig. I 2.2 g). Subsequently, a model of a braced frame of the building is performed in OpenSees (Fig. I 2.2 g, h). The pushover analyses, with both uniform and modal distribution, show that the maximum displacement of the SDOF, d^*_m, is higher than the target displacement, d^*_{et}, for both distributions of forces. The behaviour factor q_0 is equal to 2.93 for uniform force distribution and 3.75 for modal distribution (Figs. I 2.2 i, ℓ). Both values are close to the factor q of 3.2 assumed in the structural design.

I.2.4. Discussion

The work completed in the research project, according to the plan of the activities, has allowed increasing the knowledge about the seismic behavior of HSS-CHS bracing structures. An overall assessment of the experimental and numerical results indicates that the performance-based design approaches can be extended to concentrically braced frames with tubular high strength steel columns, when the collapse under earthquake is suitably prevented.

I.2.5. Visions and developments

The studies regarding bracing frames with HSS-CHS columns, carried out in accordance with the capacity design philosophy, show that seismic performance of concentrically braced frames are also excellent in case of the use of HSS in the non-dissipative zones; in addition, if properly designed, these structural systems have good ductility and failure modes that meet capacity design criteria. Therefore, HSS can increase the performance of steel-concrete composite structures and reduce both weight and construction costs.

Further developments will be undertaken as part of the research line:

- Detailed FE models of bolted brace-to-beam joints of the frame with the aim to investigate stress and strain distribution, as well as, the magnitude of slippage occurred in the experimental step.
- Parametric analysis of HSS-CHS column with the objective to formulate a new classification of sections. In fact, an important problem of HSS section, owing to the high yield strength, consists of respecting the classification limits imposed by Eurocode 3-1-1. Several studies and tests have shown that slenderness limits in EC3-1-1 are, probably, too conservative for both mild steel up to grade S460 and for HSS, especially for circular hollow sections (Beg et al, 1996; Elchalakani et al. 2002). There are significant differences in slenderness limits recommended in various codes for circular hollow sections (CHS) under bending (Elchalakani et al. 2002).
- Experimental tests on specimens with web weakening for understanding its behavior and confirm the satisfactory performance already obtained from numerical analysis.

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I 3. STEEL MEMBERS [UNINA ARCH]

I.3.1 Background and motivation

The cold-formed steel (CFS) structures are an attractive alternative to the traditional structural systems in high seismic areas, due to the good structural response together with other important features as lightness, rapid on-site erection and capability to obtain high standards in terms of safety, durability and eco-efficiency. For this reason, the seismic behaviour of CFS structures needs to be assessed.

The interest in CFS building for the housing development is significantly increasing in the last years and the commercial deployment of such systems is not followed by an evolution of the current codes at both national and European levels. As a result, accurate standard recommendations are necessary for seismic applications. The first step to obtain the development of specific design guidelines for the application of such systems in seismic zones is a seismic performance evaluation devoted to define seismic capacity and demand.

The design under seismic horizontal loads is a delicate issue, already being object of several studies carried out at University of Naples Federico II (Della Corte et al. 2006, Landolfo et al. 2006; Fiorino et al., 2007; Landolfo et al. 2010; Landolfo 2011; Fiorino et al. 2012a, 2012b, 2014; Iuorio et al. 2014). In fact, when the building is subjected to a horizontal load, floors and roofs have to be able to resist and transfer the loads to the walls, which, in turn, have to resist these loads and transfer them to the foundations. Therefore, the global lateral response of the building is strongly connected to the structural behaviour of floors and walls under in-plane actions. The in-plane resistance of these structures can be achieved either using steel bracing (usually X-bracing) or taking into account the sheathing-to-frame interaction. Therefore, it is possible to identify two different design approaches named “all-steel design” and “sheathing-braced design”. When the “all-steel design” is selected, then the in-plane resistance is assured by X-bracings, in which the diagonal elements are generally made of steel straps. In floors and roofs, steel straps are connected to the bottom flanges of joists while, in walls they are connected to the external faces of studs. As an alternative to resist to seismic loads, the effects of sheathing-to-frame interaction can be taken into account, this is the case in which the “sheathing-braced design” is used. In this case the interaction of steel framing, sheathing and their connections represents the real lateral resisting system. When this approach is adopted, floor and walls can be considered as diaphragms and the structural response depends on their elements and relevant connections.

I.3.2. Research structure

The research deals with the seismic behaviour of strap-braced CFS structures, following the “all-steel design” approach.

In the first phase of the study, appropriate assumptions are made for the design of different case study buildings. In the second phase of the research, the global response is evaluated by means of full scale tests on main seismic resistant elements, represented by strap-braced CFS stud walls. In addition, the experimental activity is completed by means of tests on material and main wall components, in order to assess the local response. The final step of the research Is the comparisons between initial design hypotheses and obtained experimental results.

I.3.3. Main results

The main structural components of strap-braced CFS stud walls are the steel frame composed by studs and tracks, diagonal bracings, diagonals-to-frame connections and connections between steel framings and external structures. In particular, steel straps are used as braces and, being very slender, they are considered active only in tension. Therefore, the lateral load applied on a wall is absorbed only by the diagonal in tension, which transmits a significant axial compression force to the ends of wall. Thus design of members and connections at wall ends is important, especially for end studs, diagonal connections and tension anchors.

In order to define the wall configuration to be tested, three residential buildings are considered as case studies, with 1, 2, 3 levels, respectively. The design of these buildings is carried out according to the Italian Construction Technical Code (NTC 2008) and, for the aspects not covered by NTC 08, to EN 1993-1-3 (2006). Two seismic zones are assumed, middle-low and middle-high. Both elastic (behavior factor q=1) and dissipative design are carried out (behavior factor q=2.5 according to AISI S213-07/S1-09, 2009). As preliminary result, it is apparent that, in case of middle-low seismic zones, both elastic and dissipative designs are quite simple, while, the elastic design for middle-high seismic zone can be very expensive in terms of technical details.

Finally, 3 strap-braced CFS stud walls are designed: a wall referring to the elastic design of a single-story building in middle-low seismic area (elastic light wall: ELW), a wall referring to the dissipative design of a single-story building (dissipative light wall: DLW) and a wall related to 3-story building in middle-high seismic zone designed according to dissipative analysis (dissipative heavy wall: DHW). The experimental program includes 17 tension tests on steel, 8 shear tests on simple connections, 28 shear tests on joints, 12 shear tests on walls.

Regarding tests on materials, specimens made by S350GD+Z (for the main structural wall framing components and diagonals of ELW) and S235 (for diagonals of dissipative walls) are tested, at standard (0.05mm/s) and higher rate (50mm/s), for evaluating the effects of strain rate. Results show an about 7% increase of yield and ultimate strength for the higher rate.

Regarding the diagonal strap-steel framing connection tests, “simple” joints, consisting in lap shear tests with only one screw, and “complete” joints, reproducing the actual connection in the walls, are considered. Also two different strain rates (standard and higher) are foreseen.

Regarding tests on full scale walls, the monotonic tests are articulated in two phases (pull and push). After each phase, the wall is unloaded and taken to the initial condition. Tests are performed under imposed displacements. Results reveal a reduction of maximum strength in the pushing phase with respect to the pulling phase, while the stiffness decreases in the pushing phase, due to the occurrence of local damages of some wall components in the previous pulling phase. For the ELW configurations, the collapse is governed by the net section failure of diagonal straps, while DLW and DHW specimens show the brace yielding without reaching the rupture, in accordance with the maximum stroke of the actuator. Results highlight strengths variations up to 9% and stiffness variations ranging between 8% and 47% between the experimental and theoretical values.

The cyclic tests are carried out by adopting a loading protocol known as “CUREE ordinary ground motions reversed cyclic load protocol” developed for wood walls by Krawinkler et al. (2001) and modified for strap-braced walls by Velchev et al. (2010). Results show that the strength and stiffness recorded for the two loading directions generally have maximum differences of 4% and 18%, respectively. The observed collapse mode is generally the net section failure of diagonal straps. The ratios between the average experimental and theoretical values highlight that the experimental strengths are higher than the theoretical predictions with maximum difference of 14%, while the measured stiffness values are lower than the predicted parameters with a variation up to 14%. The comparison between monotonic and cyclic test results reveals that the average experimental shear strength and stiffness values registered under monotonic loads are generally lower than the one recorded in cyclic tests with maximum variations of 8% and 16%, respectively.

Finally, starting from the comparisons between the adopted design assumptions and obtained experimental results, a critical analysis is carried out in order to give a preliminary

contribution for the future development of guidelines for the seismic design of strap-braced CFS structures. The behaviour factor and the criteria of capacity design are the most important issues covered by the critical analysis.
The behaviour factor of ELW obtained by the experimental results is equal to 2, which is greater than the AISI S213 value (1.6 in case of elastic design). Besides the behaviour factor obtained experimentally for dissipative walls (DLW and DHW) and for different interstorey limits (q=8-6 for 7%, q=4-3 for 2%, q=3-2.5 for 1.5%) shows that the AISI S213 value (2.5 in case of dissipative design) is a lower bound limit.
With reference to tensile verification of the strap and in order to avoid the net section failure, the NTC 2008 expression adopted to design the tensile parts in dissipative zones can be used for the design of diagonal-frame connections.
The ductile collapse mechanism (yielding of tensile diagonal) can be guaranteed using the NTC 2008 relationship for the design of connections in dissipative zones, which is able to ensure adequate overstrength of the other possible collapse mechanisms.
Finally, both NTC 2008 and AISI S213 provide no prescription aimed to avoid brittle behavior of connections. Therefore, it is considered appropriate to transpose the EN1993-1-3 prescription, which ensures an adequate overstrength respect to screws shear failure.

I.3.4. Discussion

All the research goals are reached according to the triennial plan. In particular, on the basis of prescriptions given by the AISI S213 for CFS structures and those provided by NTC 2008 for traditional X-braced steel frames, a possible seismic design method for CFS strap-braced structures is proposed to be implemented in future guidelines. The experimental results allowed the validation of assumed design hypotheses. The force modification factor values provided by AISI S213 are widely confirmed by the experimental tests and, the code values represent lower limits of the one obtained experimentally. In addition, the requirements concerning the capacity design given in the NTC 2008, for traditional X-braced steel frames, are also reliable, with some modifications, for the CFS diagonal strap-braced stud walls.

I.3.5. Visions and developments

The future development aims at transforming the seismic design method for strap-braced CFS stud walls developed during the Reluis 2010-13 research project in basic principle, provisions on materials, behaviour factors, dissipative mechanisms and capacity design rules to be implemented in future guidelines and national seismic code.

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I 4. STEEL CONNECTIONS [UNISA]

I.4.1 Background and motivation

The seismic behaviour of steel structures is significantly affected by the cyclic rotational response of beam-to-column and column-base connections. In particular, the connection design criteria govern the location of dissipative zones. In case of full-strength connections, plastic hinges develop at the end of connected members so that their plastic rotation supply can be completely exploited provided that connections are designed accounting for the maximum overstrength that members are able to exhibit before the occurrence of local buckling. Conversely, in case of partial strength connections, the dissipation of the earthquake input energy occurs in the fastening elements in relation to the connection typology. Nevertheless, depending on the degree of overstrength with respect to the connected member, also intermediate behaviours can be exhibited, where part of the dissipation of seismic input energy is developed at the end of the connected member and part occurs within the fastening elements. In other words, the member end and the joint can be regarded as two structural elements located in series, constituting the beam-joint system, whose behaviour depends on the relative resistance of the two components.

Even though this design issue is not covered by modern seismic codes, it could be faced by means of the component approach. In addition, it affects the degree of ductility supply leading to an innovative distinction to be made: full-ductility connections (i.e. connections assuring that the plastic rotation supply of the beam-joint system is not less than the one of the connected member) and partial-ductility connections (i.e. connections leading to a plastic rotation supply of the beam-joint system less than the one of the connected member).

Despite beam-to-column and column-base joints play a role of paramount importance in the seismic response of Moment Resisting Frames (MRFs), only limited design information is given in Eurocode 8 and in D.M. 14/01/2008. Therefore, the development of more detailed design rules based on the component approach, already codified in Eurocode 3 for monotonic loading conditions, is a pressing need.

I.4.2. Research structure

The research activity is devoted to the definition of design rules to control the local ductility supply in steel framed structures, i.e. the local ductility supply of the member-joint system regarded as structural components in series, such as beam-to-column and column-base joints. In particular, mechanical models predicting the plastic rotation supply of connections through the component approach are set-up. The same approach, but including a semi-analytical modelling of the joint components, in order to account for stiffness and strength degradation and for pinching phenomena, is also applied aiming at the prediction of the cyclic rotational behaviour of connections. Finally, innovative connections types that improve the energy dissipation capacity are examined.

The project is organized in the following three phases, planned in three years: 1) experimental testing and modelling of traditional beam-to-column joints; 2) innovative beam-to-column joints and column-base joints; 3) definition of design criteria and new code provisions.

The research activity is articulated according to the following tasks:

Task 2.1.1: Experimental analysis of the ultimate behaviour of beam-to-column connections and column-base connections under cyclic loads;

Task 2.1.2: Set up of new design criteria, based on expected ultimate behaviour, for beam-to-column and column-base connections;

Task 2.1.3: Modelling of beam-to-column and column-base connections by the component approach for predicting the ultimate plastic rotation supply and the cyclic behaviour;

Task 2.1.4: Improving code provisions by means of new design criteria and new rules to control and predict the ultimate behaviour of connections.
In particular, Task 2.1.4 represents the final goal of the research activity, i.e. the improvement of the whole framework of code provisions dealing with the design of beam-to-column and column-base connections in seismic resistant structures.

I.4.3. Main results

With regards to the cyclic rotational response of beam-to-column connections an experimental program is carried out, focusing on the identification of the joint components. Both the cyclic moment-rotation curve of the joint as a whole and the cyclic force-displacement curves of all the joint components are evaluated. The comparison between the sum of the energy dissipated by each joint component and that dissipated by the joint as a whole confirms that the extension of the component approach to the prediction of the cyclic behaviour of beam-to-column joints is feasible, provided that the joint components are properly identified and modelled. With regards to the prediction of the cyclic response of bolted beam-to-column joints, a mechanical model is developed within the framework of the component approach already codified by Eurocode 3 for monotonic loadings. The model is calibrated on experimental results. The obtained results encourage the possibility of extending the component approach to the prediction of the cyclic response of bolted connections.
With regards to the column-base joints, three monotonic tests on real scale joints are carried out. On the basis of the obtained results and a significant number of additional test results collected from the technical literature, the reliability of the model proposed by Eurocode 3 for predicting the rotational stiffness and the flexural resistance of base plate joints is analysed. The EC3 approach results to provide a sufficiently accurate prediction of flexural resistance, while an overestimation of the flexural stiffness.
With regards to innovative connection types, two alternative approaches for improving the hysteretic behaviour of traditional partial strength joints are proposed. The first approach is based on the application of the concepts usually adopted for the development of ADAS hysteretic dampers to the T-stubs, the second one is based on the application of friction dampers located between the T-stub web and the beam flange of double split tee connections. In both cases experimental tests are carried out both with reference to the dissipative joint component and on real scale beam-to-column joints. In case of T-stubs equipped with friction pads, cyclic tests are carried out on different interfaces (steel-steel, brass-steel, rubber-steel) in order to determine the static and dynamic values of the friction coefficients and three tests on real scale double split tee joints equipped with friction pads are carried out. The attention is focused on the cyclic moment-rotation curve of the joint as a whole and on the cyclic force-displacement curves of the new joint component, i.e. the friction damper. The obtained experimental results evidence the good performance in terms of energy dissipation of the joints equipped with such damping devices. The obtained results are very encouraging about the application of the proposed approaches in order to obtain highly dissipative joints or damage preventing joints for seismic-resistant steel MRFs.
Seismic design criteria based on the application of hierarchy criteria at the joint component level are defined. In addition, design guidelines for beam-to-column connections to be used in seismic-resistant moment frames are developed for the two cases of full-strength connections, where the primary aim is the control of the location of the plastic hinge by properly accounting for beam overstrength, and of partial-strength connection where hierarchy criteria at the level of single joint components are established to control the weakest joint component and additional design formulations are provided to control the plastic deformation capacity of the weakest joint component, governing the plastic rotation supply.

I.4.4. Discussion

Dealing with traditional beam-to-column connections, the new experimental results obtained during the research activity confirm the possibility to extend the component approach, currently codified in Eurocode 3 with reference to monotonic loading conditions only, to the case of connections subjected to cyclic loads. It is demonstrated that the energy dissipation provided by beam-to-column joints under cyclic loads can be obtained as the sum of the energy dissipation offered by the single joint components, provided that the components are properly identified and the cyclic behaviour is properly measured.

Starting from the above result, mechanical models extending the component approach to the problem of predicting the cyclic moment-rotation response of beam-to-column joints are developed and their accuracy is compared both with the experimental results of the tests performed during the project and with test results of independent researchers.

Another important result outlined by the theoretical and experimental activity carried out during the research project is the possibility of using the component approach as a powerful tool to design beam-to-column connections by means of local hierarchy criteria, which aim to control the weakest joint component and, as a consequence, the main source of energy dissipation capacity. In particular, design criteria are identified to control the location of the weakest joint component and, in case of partial strength connections, to control the plastic deformation capacity of the weakest joint component with reference to the components usually modelled as an equivalent T-stub.

Regarding column-base connections, both the monotonic and the cyclic response are investigated with reference to base-plate connections with anchor bolts. The interaction between axial force and bending moment is investigated and some improvements to the codified approach for predicting the rotational stiffness of such connections are proposed. In addition, also the cyclic response and the ductility are investigated. However, in case of base-plate connections additional studies are needed to extend the component approach to the prediction of their moment-rotation cyclic response, because of the additional difficulties due to the interaction with the axial force.

The obtained results are in line with those forecasted during the preliminary planning of the research activity. In addition, with respect to the initially planned research activities, also innovative connections equipped with friction dampers are conceived and tested. The preliminary experimental results are very encouraging about the possibility to develop beam-to-column connection able to accommodate without any damage the rotation demands occurring even in the case of destructive earthquakes.

I.4.5. Visions and developments

The possibility of further developments for the research activity in the field of beam-to-column connections, dealing with traditional connections, mainly regards the development of design guidelines for both full-strength and partial strength connections. In this context, some codes like ANSI/AISC 358-10 already introduce the concept of prequalified connections, whose performance in terms of plastic rotation capacity is checked by experimental tests. However, despite these design rules constitute a useful reference, they cannot be easily applied with reference to the European context, because they are based on U.S. practice and code provisions. In addition, ANSI/AISC 358-10 provides recommendations for full-strength connections only. Therefore, the development of design guidelines aiming to the identification of prequalified connections is a challenging task for the future research activity.

Regarding column-base connections, the state-of-knowledge has a gap in comparison with beam-to-column connections. Therefore, additional experimental activities should be carried

out to provide accurate and reliable design guidelines to be applied to column-base connections subjected to cyclic loads, like those occurring under seismic actions.
Finally, with reference to the innovative connections tested during the research project, i.e. bolted double split tee connections equipped with friction dampers, additional experimental tests are needed to identify the best material for the friction pad and to improve the structural detail. These are challenging tasks for the future development of the research activity, considering that the tested beam-to-column connections are able to withstand severe rotation demands without any structural damage by simply predicting the maximum stroke which the friction damper has to exhibit under the maximum credible earthquake ground motion.

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I 5. STRUCTURES EQUIPPED WITH SHEAR PANELS [UNICH]

I.5.1 Background and motivation

The research activity aims at evaluating the seismic behaviour of steel frames passively protected by means of special devices based on the use of metal shear panels.

I.5.2. Research structure

The research is mainly articulated in the following tasks:
1) Development of new devices based on the use of metal shear panels;
2) Definition of the inelastic structural behaviour of steel frames seismically protected by means of metal shear panels having a stiffening and/or a dissipative function;
3) Code proposal of the q-factor for steel frames protected by means of metal panels;
4) Development of design methodologies.

Several innovative shear panel types, namely slender, semi-compact and compact, are studied. They are applied on a number of steel frames characterized by different beam-to-column joint details, geometry, storey and bays number. Time history analyses of frames protected by shear panels run by using a FEM non-linear numerical model, in order to evaluate the seismic response in the inelastic field. The obtained results on one hand allow to define the meaningful design parameters, such as the q-factor, which can be proposed for Codes; on the other hand, they allow to delineate the most convenient design strategy to be pursued.

I.5.3. Main results

Hereafter the results of the main activities are summarized.

Review of the state of the art

The current progress provided by literature concerns the following topics:
1) Design criteria for retrofitting existing structures;
2) Set-up of capacity design principles to be applied to steel shear walls;
3) Identification of the elastic strength reduction factor of metallic shear wall;
4) Alternative strategies other than stiffeners, to obtain dissipative shear walls.

Experimental test on a new prototype of dissipative buckling inhibited shear panels

An alternative and innovative strategy of obtaining dissipative shear panels, also when these are thin (so to favour a quick activation of their hysteretic capacity) is developed by inhibiting the principal buckling modes of the base plate by means of steel bands laid on the plate diagonals. The used bands are conceived in order to allow relative displacements in the plane of the plate and preventing the out-of-plane displacements.

A first preliminary cyclic experimental test is performed by using a specimen made of pure aluminium. The test specimen type 1 and the obtained results are shown in the Fig. I 5.1. As it is possible to observe, the hysteretic response is characterized by large cycles, which evidence a suitable dissipative capacity of the system.

Definition of the q-factor of steel frames with metal panels

Incremental dynamic analyses are carried out on different frames equipped with compact aluminium shear panels in the bracing type configuration. The main outcome of these analyses consists in the definition of a realistic q-factor, determined by applying the Ballio-Setti procedure, it ranging between 5 and 9, depending on the imposed record.

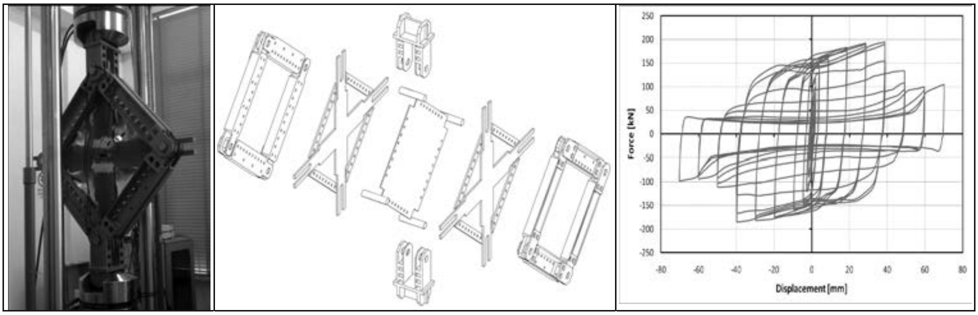

Figure I.5.1. Test specimen type 1 and experimental results.

Experimental activities on both steel and aluminium shear panels equipped with devices devoted to partially or totally inhibit buckling phenomena

An evolution of the above system is dealt with. It is characterized by devices able to totally restrain the out of plane of the whole system. Thus, the new shear panel is able to avoid also the higher critical modes, resulting in a more performing response from the dissipative point of view. The obtained outcomes form experimental tests on this new panel type 2 are described in Figure I 5.2.

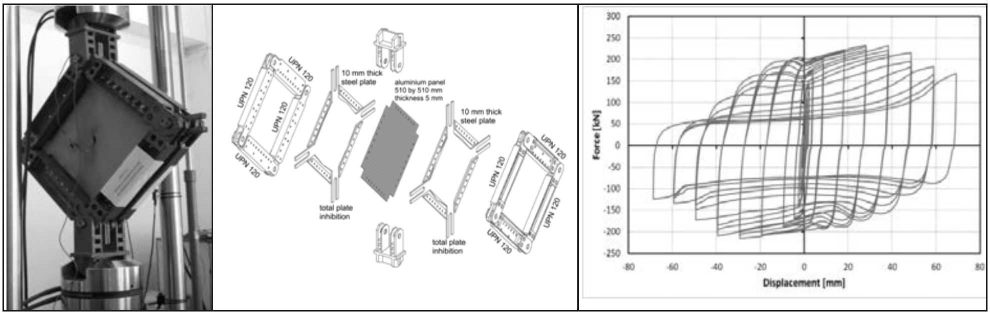

Figure I.5.2. Test specimen type 2 and experimental results.

The buckling inhibition devices used for aluminium plates are applied also on shear plate made of steel characterized by different thicknesses. The obtained results highlight some criticisms when the “gap” between the inhibition system and the base plate increases. Nevertheless, they confirm the effectiveness of the proposed technology.

Time history analyses on dual frames seismically protected by compact shear panels.

The numerical activity, aimed at determining a q-factor for simply hinged frames with compact shear panels, is extended to dual frames designed according to capacity design criteria. The q-factors are evaluated with several procedures (Ballio-Setti, Energy based method, etc), for frames with and without panels. A multiplier of 2-3 for the q-factor given by Codes for frames without panels is evaluated.

Incremental dynamic analyses are implemented on 4-8-12 storey frames, obtained by means of proper design criteria, in which shear panels are arranged in a bracing type configuration (Figure I 5.3a). Two sets of natural records (one for the lower-rise frame and the other for the 8-12 storey frames) are adopted and scaled up to the attainment of a failure condition (member collapse, global buckling, storey drift of 3%, etc). Moreover, for each configuration,

the above analyses are repeated by considering shear panels whose hysteretic cycles resulted negatively affected by different levels of pinching, so to determine the effect of these detrimental phenomena on the global dissipative response.
This allowed to detect the simple following expression of the behavior factor:

$$q_{\eta} = q_0 \cdot \omega(\eta)$$

where η is an energy efficiency factor, which indicate the detrimental effect level to be associated to the considered hysteretic cycles due to buckling phenomena (η=0 for shear panels that does not contribute to the structural response, whilst η=1 for fully dissipative shear panels), q_0 is the behavior factor of the steel frames without shear panels e $\omega(\eta)$ is the increment of the behavior factor to be ascribed to the shear panels themselves (Figure I 5.3b).

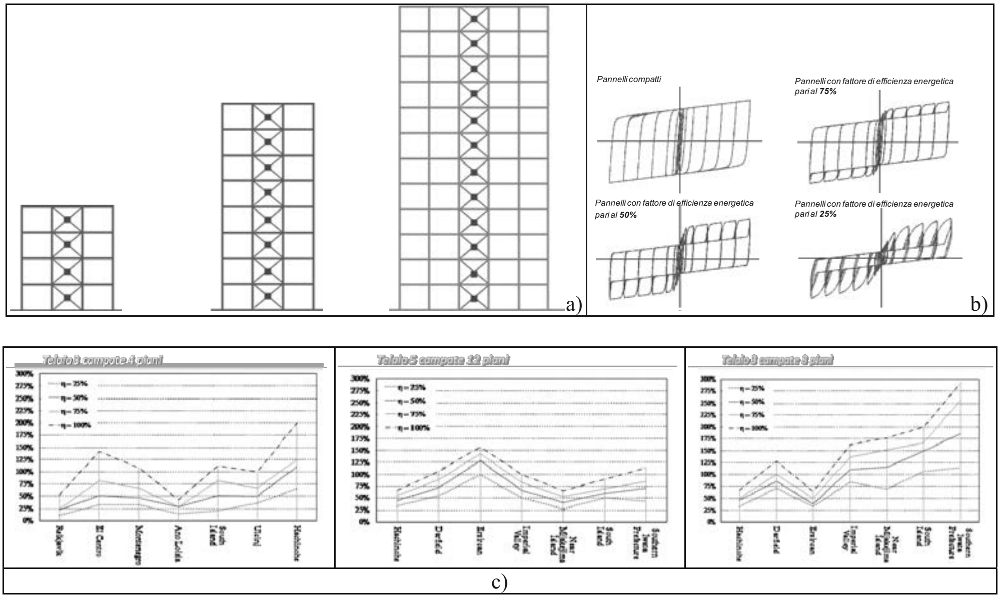

Figure I 5.3: Time history analyses on dual frames equipped with compact shear panels

I.5.4. Discussion

The main objectives of the research activity, such as the definition of the structural behavior of steel frames protected by metal shear panels with stiffening or dissipative function, the evaluation of the inelastic capacity of structures with metal shear panels and definition of the relative behavior factors to be proposed for Codes and Guidelines, with regard to new and existing steel buildings in seismic areas, the proposal of design rules for the use of shear metal panels for the protection of steel structures, actually, are achieved without significant discrepancies. Only the initial experimental program is modulated in a different way. In fact, the results obtained during the first part of the experimental campaign as it was preliminarily planned provided a large quantity of information, suggesting the choice to avoid the implementation of further tests. On the other hand, more efforts are devoted to the set up of numerical models, calibrated on the basis of the obtained experimental results. Therefore a wider parametric analyses is carried out with respect to the initial intention.

I.5.5. Visions and developments

Based on the obtained results, the following two main developments of the implemented activity are necessary:

1) Proposal of additional types of shear panels which are properly weakened in order to accomplish, in a more convenient way, the capacity design criteria for the protected structure. These can be obtained by applying on the base plate a suitable quantity of slits or holes, with an arrangement conceived in order to minimize the detrimental effects of possible buckling phenomena.

2) Proposal of additional design rules for code implementation able to put into evidence the structural interaction between the protected frames and the shear panels, even considering the possibility of adopting semirigid/partial strength beam-to-column joints.

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I 6. BEHAVIOUR FACTORS FOR MOMENT RESISTING STEEL FRAMES [UNINA2]

I.6.1 Background and motivation

Steel moment resisting frames are expected to be able to sustain large plastic deformations in bending and shear. However, structural damage and collapses during recent earthquakes evidenced some critical aspects in the seismic behaviour of steel structures even when designed according to the current codes. The ductility and the capacity design criteria may be not effective to obtain a global plastic mechanism and to avoid that damage may far outweigh the cost of the structural system. More advanced design procedures based on the second order plastic analysis proved to be effective to ensure a global plastic mechanism. However, they require a great overstrength of steel members. Furthermore, the design strength of the structure is independent by the seismic intensity level. Finally, the ultimate limit state verification is not sufficient to ensure the verification for the other limit states.

Most seismic codes use the concept of scaled design response spectrum to approximately estimate the response of inelastic multi-degree-of-freedom (MDOF) systems accounting for the nonlinearity associated with the material, the structural system and the design procedures. The reduction factor is called behaviour factor (q-factor) in the European Code and response modification factor (R) in the American Codes. In SEOAC Guidelines (1999) R is termed "structural quality factor" or "system performance factor". The behaviour q-factor definition is based on the maximum capacity of structure to dissipate energy during the plastic deformations corresponding to ultimate limit state criterion. Some considerable differences in the numerical values of the behaviour factors specified in various codes for the same type of structure may be found: behaviour factors adopted in American codes (NEHRP and UBC) presuppose the existence of significant amounts of overstrength in the structures, which however can be relied upon without any check, as opposed to the Eurocode 8 procedure for steel structures. However, a direct code comparison between EC8 and US provisions is not consistent if only the level of force reduction is considered. A reliable comparison should also involve the full design procedure including the partial safety factors used in each code for material resistances and applied loads. The ductility-dependent component of the behaviour factor is generally estimated on the basis of inelastic spectra. The overstrength-dependent component is connected to the design procedure and it is generally estimated with static and dynamic inelastic procedures. ATC-63 (2008) introduces a separate factor relating to the structure's redundancy. NEHRP defines an empirical R factor to account for both damping and ductility inherent in a structural system at a displacement approaching the maximum one. Eurocode 8 (2004) defines the behaviour factor for steel structures explicitly accounting for the effects of ductility and redundancy and member overstrength.

The main motivation of the research is, on one side, the necessity to predict the basic features of the inelastic response of steel structures without performing complex time history dynamic analyses, on the other side, the convenience to consider modifications and/or integrations to the national seismic code as far as the design of steel structures is concerned.

I.6.2. Research structure

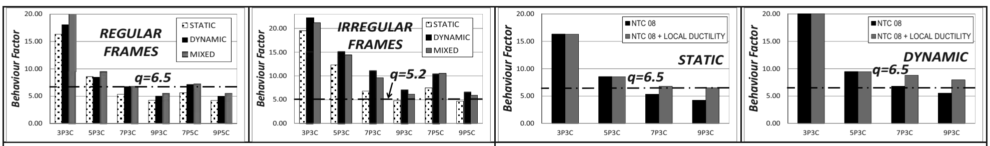
The research activity focuses on analytical and numerical aspects over a wide range of topics involving the seismic response of steel structures, they being in particular the assessment of the inelastic seismic behaviour, response and performance of typical ductile steel moment resisting frame (SMRF) structures. The objective is to estimate the effectiveness of design procedures, through the estimation of the global behaviour of the structure in terms of lateral displacement with nonlinear static and dynamic analyses. A numerical verification of the

behavior factor adopted in seismic design codes in Italy is carried out by comparison with existing methods for determining the q-factor of SMRFs, also in comparison with other structural types like framed reinforced concrete structures. Finally, the research is aimed at gaining more accurate formulations of the q-factor to be used in Italian seismic code.
In particular the analytical and numerical activities aim at evaluating the effectiveness of linear elastic analyses with q-factor to approximately estimate the response of inelastic MDOF systems. Therefore, static and dynamic methods are applied to both regular and irregular in elevation multi-storey SMRFs. The effects of storeys, spans and regularity in elevation of frames on the behaviour factor are considered. On the basis of results, modifications to the design rules given in the Italian Structural Code (NTC08) are proposed, in order to account in a more effective way for the actual ductility properties of the structure.
Therefore the research programme is organised around the following topics:

1) Accuracy assessment of nonlinear static procedures for estimating the seismic performance of steel frame structures (multimodal pushover analyses, force-based and displacement-based adaptive pushover procedures, adaptive capacity spectrum methods);
2) Development of displacement-based approaches to assess and design SMRF structures (acceptance criteria, design displacement spectrum, design procedures).
3) Assessment of q-factors for seismic design of moment-resisting steel frames starting from the study of inelastic seismic performance (parametric analysis, comparison with other structural types, design rule definition process).

I.6.3. Main results

The effectiveness of the behaviour factor proposed in Italian Code to predict the nonlinear response of SMRFs is investigated. Results show that the overstrength reduction factor recommended by EC8 and Italian Code for multi-bay multi-story frames is conservative. Contrary, the ductility response modification factor and, consequently, the behaviour factor may be not conservative. This result derives from the effect of axial force that reduces the plastic moment capacity of the first-story columns in high-rise steel frames, whose compression failure limits the ultimate displacement capacity of the structure. Therefore, a local ductility criterion based on a limit of the axial force ratio is proposed to control the ductility of columns and so ensure that the recommended behaviour factor is conservative. This criterion may be usefully introduced in the Italian Code for improving the coherence between the initially adopted and the real behaviour factor defined from nonlinear analyses.



a) b)

Figure I.6.1. Behaviour Factors a) regular and irregular steel frames; b) frames designed with or without the local ductility criterion (N/N_{PL}<0.3).

Both static and dynamic non linear analyses are carried out on both regular and irregular steel frames designed according to NTC08. In particular, the non linear modeling is defined assuming two different ultimate rotations: 1) θ_u=0.03 rad; 2) θ_u defined according the FEMA 356, as a function of both geometrical and mechanical features of cross sections. Some values of the q-factor obtained by applying the static, the dynamic and the mixed method are given in Fig. I 6.1a.

It can be observed that only for 3- and 5-storey frames the value of the q-factor given by the code is conservative as compared with that coming from the analyses. Furthermore, the reduction of ductility resulting from the excess of axial load is emphasized, which reduces the global dissipative capability of the structure and, hence, the q-factor. For this reason, in order to improve the local ductility, a proposal is made to limit the value of the axial load acting in the columns to 30% of the plastic load. Such limitation, which is similar to that given for r.c. structures (§7.4.4.2.2 NTC08), generally interests the columns at the first level of the 7- and 9-storey frames. In this way, with a little increase of the structural weight it is possible to get a significant improvement of structural ductility. This leads to values of the q-factor in better agreement with values assumed in design.

I.6.4. Discussion

SMRFs covering a wide range of structural characteristics are designed and analysed. Both static pushover analyses and nonlinear incremental dynamic analyses are performed and the effects of some parameters influencing the response modification factor (including regularity, number of spans and number of storeys) are investigated. As a significant outcome of the activity, a local ductility criterion based on the control of the axial load level is proposed to implicitly ensure that the columns exhibit a more ductile behaviour and that the recommended value of behaviour factor given in the Italian seismic code is conservative.

The advantages of both adaptive and multimodal pushover procedures over conventional pushover methods are verified through an extensive comparative study involving regular and irregular steel moment resisting frames subjected to a comprehensive set of input ground motions. The results are of interest in evaluating the accuracy and applicability of the mentioned pushover-based methods in predicting seismic demands and, in particular, the peak inelastic roof and interstorey drift response.

In conclusion, based on the results of a wide parametric analysis, the main objectives of the research are reached. Nevertheless, some ideas for further investigation arise.

I.6.5. Visions and developments

Because of both width and complexity of the problem, further research is still required.

First of all, the results obtained are valid under the assumption of full-strength connections. The characteristics of connections, specially the column base-to-foundation connections may influence the inelastic behaviour of the structure under seism. Consequently, more research is needed for other types of connections.

With regards to the advanced non-linear static procedures for the evaluation of the overall seismic response, adaptive load distributions should be examined, in order to reproduce in a more faithful way the ongoing degradation of the structures as long as the external load increases. Such procedures could be, for instance, the Improved Displacement Coefficient Method (DCM), the Modified Modal Pushover Analysis (MMPA), the Extended N2 Method and the Adaptive Capacity Spectrum Method (ACSM). Their accuracy in the prediction of the seismic behaviour and in particular in the evaluation of the response factors should be checked against the results coming from a more refined incremental dynamic analysis.

Another specific subject that could be developed is the relationship between the common rules for seismic design and the provisions adopted to mitigate the progressive collapse potential of structures. There is a general feeling, in fact, that the two design approaches may lead to contrasting results, particularly in terms of beam and column sizes. This aspect may be very interesting since the specific characteristics of progressive collapse are very different from the global collapse mechanism usually sought under seism, as for example the initiation by relatively localized damage against the evolution time to the global collapse, as well as robustness design against normal design. First, in progressive collapse design the applied actions cannot be specified explicitly and local damage scenarios are postulated. Second, progressive collapse design requires structural strength at large deformations and acceptability is based on comparison of the maximum ductility demand against the ductility capacity in various parts of the structures. Finally, the capacity design concept followed in seismic design may give smaller margin of safety to accommodate column-removed conditions. In this context, the directions for future research could be:

a) Critical survey and comparative analysis of design procedures for seismic design and progressive collapse with the purpose of harmonization between the two design approaches;

b) Assessment of progressive collapse resisting capacity of SMRFs subjected to column failure;

c) Proposal of structural optimization techniques for cost-effective design of seismic steel moment resisting frames with enhanced resistance to progressive collapse.

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I 7. ANALYSIS OF THE EFFECTS OF MATERIAL MECHANICAL PROPERTIES [UNIPI]

I.7.1 Background and motivation

According to actual standards for constructions (EN1998, NTC2008) structural systems are designed following the criteria of capacity design, in order to dissipate seismic energy through the development of global collapse mechanisms. Eccentrically Braced Frames (EBF) steel structures, in particular, are designed localizing the dissipative zones in specific elements (links), while columns, braces and other beams are oversized to remain elastic, using special coefficients that take into account both possible phenomena of material overstrength (γ_{ov}) and differences between plastic resistance adopted in the design and real forces acting on elements (Ω). This design technique is necessary for avoiding gaps between what assumed in the design and the real situation (i.e. between nominal and real values of material properties), leading to the alteration of collapse modes assumed in the design. In the common practice, linear analyses, considering the ductility of the buildings through behaviour factors depending on the geometry, material and structural type, are generally used. The problem related to this approach is that, frequently, P-Δ, II order effects or the respect of stiffness limits become more conditioning than seismic assessments, leading to oversize elements' sections. All this phenomena can modify the efficiency of seismic design according to actual standards.
In this context the research focuses on the influence of material variability on the seismic response of EBF, for providing design indications for new buildings in high and medium seismic areas. The work is also developed in the framework of the research project OPUS (Optimizing the seismic Performance of steel and steel - concrete structures by standardizing material quality control) founded by the European Fund for Coal and Steel (RFCS).

I.7.2. Research structure

Aim of the research is the investigation of the influence of both material properties and seismic input variability on the dissipative behaviour of EBFs, evaluating the effectiveness of actual standards in the definition of overstrength coefficient (γ_{ov}) and finally individuating the most suitable values of behaviour factors q for the proper seismic design of structures.
To this purpose, different steel buildings are firstly designed according to actual European standards and then analysed through non-linear Incremental Dynamic Analyses (IDA), considering the variability of both mechanical properties of materials and seismic input, in order to achieve a complete probabilistic characterization of the mechanical response of the system. The results obtained using the nominal and the real values of mechanical material properties, provided by the European steel producers, partners of the project, are compared in terms of activation of collapse criteria, analyzing the effective level of structural safety and the ability of the structure to redistribute the plastic demand imposed by the earthquake.
In such a way, it was possible to assess:

a) The influence of material properties' variability on the effective seismic behaviour of buildings designed according to actual standards (q factor);
b) The necessity of applying an upper limit to the yielding stress, actually not present;
c) The influence of the application of γ_{ov} on the actual design standards.

The work is organized in 5 parts, leading to the statistical analysis of the structural behaviour of EBF systems with regards to the variability of material properties and seismic input:

1) Detailed statistical study of the actual European structural steel productions, for finally providing a probabilistic model of mechanical variables (yielding and tensile stress, elongation ad fracture) considered with their probabilistic interdependencies.
2) Design of EBF case studies, with different geometry, materials and seismic intensities.

3) Characterization of seismic hazard according to EN1998, generation and selection of 7 seismic inputs for each seismic area for the non-linear analyses.
4) Incremental Dynamic Analyses (IDA) on EBFs, evaluating PGA of selected seismic inputs that activate collapse modes.
5) Probabilistic techniques for assessing the structural response of case studies as function of seismic input and mechanical properties variability.

I.7.3. Main results

Three case studies are designed (Fig. I 7.1a), considering both European and Italian standards for constructions (EN1998, NTC2008), satisfying both seismic and lateral stiffness requirements. The analysis of all the possible collapse mechanism is carried out and preliminary 2D fibre models are analyzed. The actual European steel production is investigated, comparing results with the nominal values generally assumed in the design (Fig. I 7.1b) and a model for the generation of samples according to real production is developed.

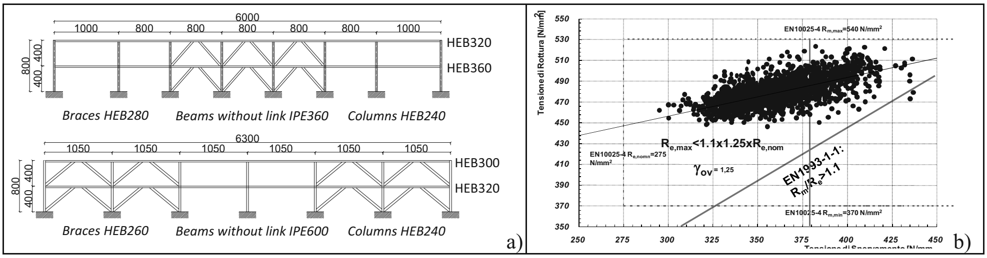

Figure I.7.1. a) Car park with shear links, b) Investigation of mechanical properties of materials.

A detailed investigation of the seismic hazard is performed; in particular, in order to generate artificial earthquake time histories, the program SIMQKE is used. Two types of seismic intensities are considered: for high seismicity, PGA equal to 0.25g and type 1 spectrum for soil B; for low seismicity, PGA equal to 0.10g and type 2 spectrum for soil C. The accelerograms duration is equal to 15s and 20s for PGA equal to 0.10g and 0.25g respectively. The accelerograms adequacy is checked by determining the elastic response spectra. IDA simulations are performed by scaling generated seismic inputs at different PGA levels. In particular, for each plane structure, different excitation levels are individuated according to the collapse modes that can be activated by increasing PGA. For each case study, collapse criteria are analysed for each considered PGA level, through IDA adopting alternatively the 7 artificially generated accelerograms. Monte Carlo Method is applied to each analysis generating 500 samples of mechanical variables and running IDAs for each of them. In particular, to be adherent to the real assembling of steel structures, all beams and braces members are considered as probabilistically not dependant, while columns of two subsequent floors are considered as characterized by the same probabilistic variables.
In order to generate samples of mechanical properties, a log-normal model is assumed for each of them, yield strength $R_{e,H}$, ultimate strength R_m and elongation A, so that their distribution resulted multivariate with the three variable inter-correlated. The generation procedure is based on the adoption of an equivalent multi-normal probabilistic distribution (Tamast 1977) obtained from the original multivariate log-normal model. In such a way, for each case study 3500 numerical simulations are carried out (i.e. 7 quakes × 500 material samples) for each considered PGA level and each considered collapse criterion.

Results are analysed in order to individuate the failure probability (P_f) associated to the significant collapse criteria, through the application of a probabilistic procedure. Moreover, the effective behaviour factor (q) is evaluated, to be compared to the design one.
The necessity of applying an upper limitation to the yielding strength in correspondence of dissipative zones is also evaluated: to do this, the probabilistic procedure previously elaborated is applied to a reduced set of materials characterized by a fixed limit of f_y, once again individuating the corresponding probabilities of failure.
The efficacy of the capacity design approach and the importance of the over-strength factor in the mitigation of overstrength phenomena is widely investigated, individuating the probability of obtaining equal or higher values of internal forces in protected members in relation to the assumed design values (for different levels of PGA and γ_{ov}). The probabilistic procedure evidences that the seismic behaviour of EBFs is not degraded by the material properties variability, which can be considered consequently mitigated by the capacity design approach and by the expressions presented in EN 1998-1:2005 and in NTC 2008. The probability of failure for the different collapse criteria is in general lower than the fixed limit equal to 10^{-4}.
The introduction of an upper limitation to the yielding strength in correspondence of the dissipative zones produces in general a decrease of the P_f associated to non-dissipative members, while, on the other hand, the P_f of ductile modes generally increases.
Fragility curves evaluated for different values of the overstrength factor indicate that design standards and capacity design approach assure the required protection of non-dissipative members, characterized by probability of failure lower than the ones commonly accepted. Moreover, buildings characterized by materials with overstrength factors higher than 1.25 present a behaviour completely in line with what assumed during the design, despite the presence of elements oversized due to interstorey drift limits, second order effects and others.

I.7.4. Discussion

The investigation of the influence of material properties on the ductile behaviour or EBF steel structures was developed through a multi-step procedure, based on an accurate statistical investigation of the actual European steel production aiming at the elaboration of a probabilistic model that will be used for the generation of samples to adopt the numerical simulations. In particular, the scattering of steel mechanical properties was represented by a multi-variable model in which the yielding strength $R_{e,H}$ (f_y), the tensile strength R_m (f_t) and the elongation at fracture A (ε_u) were considered with their probabilistic interdependencies.
Different EBF structures (including buildings with different geometrical configuration, different steel grades, different design ductility class and seismic hazard) were designed according to EN 1998-1:2005; the design was optimized in order to avoid the oversizing of structural elements due to gravitational loads and preliminary numerical simulations, including non-linear static and dynamic analyses, were executed in order to assess the effective structural behaviour in terms of dissipation of energy, development of ductile mechanisms and activation of significant collapse criteria (i.e. maximum rotation of links, reaching of the maximum interstorey drift, buckling of members in compression) and for the evaluation of the effective behaviour factor. Preliminary IDA, executed adopting the mean values of real mechanical properties, allowed the individuation of the PGA levels activating significant collapse criteria for each of the considered EBF buildings. After those preliminary simulations IDAs were executed adopting 500 different material samples (in terms of $R_{e,H}$, R_m, ε_u) for the PGA levels generally activating significant collapse criteria and for 7 different artificial accelerograms. In particular, seismic action was modeled adopting the hazard model proposed by Eurocode 8 and calibrated according to design parameters associated to ULS.
The numerical results coming from dynamic analyses were finally analyzed adopting a

statistical procedure providing fragility curves and yearly threshold exceedance probability (P_{fail}) associated to the relevant collapse modes previously selected for each case study.

I.7.5. Visions and developments

Investigations were mainly devoted to the assessment of the influence of material properties' scattering on the seismic response of EBF steel structures, to the analysis of the efficiency of the capacity design rules adopted by EN1998-1-1 for the optimization of seismic design of EBFs and to the following values of reliable behaviour factors.

The influence of material properties' scattering on the failure probability of structural members was estimated as marginal. The P_{fail} associated to all relevant collapse modes were often lower than the fixed acceptance threshold, confirming the effectiveness of the capacity design approach proposed by Eurocode 8, despite the unavoidable oversizing of the non-dissipative members mainly due to gravitational loads. Moreover, the analysis executed on capacity design equations evidenced that the assumption of γ_{OV} as material overstrength is, in general, too conservative and can lead to an excessive structural members' oversizing.

Results obtained shall be translated into useful indications for designers and technicians especially for what concerns the design of EBF steel structures and the optimization of their ductile properties, using for example specific values of the behaviour factors calibrated on the base of numerical results for different levels of ductility and seismic intensity. Moreover, guidelines and background documents related to the individuation of the effective overstrength factors (both γ_{ov} and Ω) shall be elaborated.

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RESEARCH PROJECT
DPC - RELUIS
2010-2013
LINE 1. ASPECTS IN THE SEISMIC DESIGN OF NEW BUILDINGS
TASK 2. STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES

II - STEEL CONCRETE COMPOSITES STRUCTURES

I 8. CONCRETE-STEEL COMPOSITE MEMBERS [UNISANNIO]

I.8.1 Background and motivation

The steel-concrete structures present many advantages due to the collaboration of the two materials, however in the field of seismic design the approach is not well assessed and moves on the rules of steel structures. In Italy code provisions for composite constructions in seismic zone have been introduced with OPCM 3274 in 2003 and nowadays they are available in the current DM 14/01/2008 (NTC2008), similarly to Eurocode 8. In case of Moment Resisting Frame (MRF) the performance of composite structures are assumed as equal to that of steel structures considering the same dissipative capability and q-factor, neglecting the composite effect. Experimental results and numerical analyses on the dissipative capacity of composite elements are still lacking; furthermore the choice of the type and the identification of the behaviour of joints (beam-column and base-column) are complex problems. Therefore the design procedure of MRF is not reliable and there are not adequate information about ductility (plastic rotational capacity) of composite elements for carrying out reliable non-linear analyses with concentrated plasticity models.

The present research focuses on defining the aspects that govern the design of MRF and the consequent global performances of structures; at the same time the non-linear behaviour of beams and columns is studied in detail in order to evaluate the local ductility that contributes to the global ductility of the structure. The purpose is to find formulations for evaluating the plastic rotation capacity of beams made of different materials and shear connection devices, identifying the influencing parameters, to be used in non-linear analyses of the seismic-resistant frames by lumped plasticity approach. Therefore, the structural non-linear models of a base column connection for composite columns and composite beams under hogging moment are developed. For the column base connection a new type realized by embedding the steel column in the foundation block is considered.

The study provides practical provisions for the seismic design of steel-concrete composite structures.

I.8.2. Research structure

The main objectives of the research are the following ones:

- outline the critical points of the code provisions for the design of steel-concrete composite MRFs and highlight possible modifications;
- identify the parameters that govern the plastic deformability of composite beams and base column joints.

The organization of the project considers the following phases: 1) development of the state of the art and design examples of buildings with MRF; 2) implementation of numerical models of base column joints characterized by a socket type connection and composite beams subject to hogging moment in order to perform a parametric study; 3) definition of the local behaviour of the elements to be introduced in the global analysis of MRF buildings obtaining practical provisions for the design.

The activities of the research program are listed hereafter:

- examination of the seismic performances of composite frames and design applications according NTC2008 and EC8;
- characterization of the post-elastic deformability of composite elements;
- implementation of non-linear FEM models of composite beams and columns;
- experimental tests on composite beams for investigating the critical zone at the beam-column joints;

- definition of the procedure and formulation of design criteria for the base-column joint using a socket type system and partially encased columns.

Most of the activity is carried out in cooperation with the research group UNITS, with particular regard to the analysis of beam-column composite joints.

I.8.3. Main results

The technical literature is scarce of studies and experimental results. Some research are carried out on the non-linear behaviour of composite beams especially through experimental tests and a few number of numerical models. The most lacking data are related to the base-columns joints. In general, the high dissipative capacity of composite frames, designed according to Eurocode 8, is evidenced, even overcoming the capacity expressed by the q-factor adopted by the code, since beams and columns are characterized by a great plastic rotation capacity. The experimental studies point out the efficiency of realizing the dissipation in the beam-column joint, as allowed by Eurocode 8 but not by NTC2008. The design aspects according NTC2008, with some references to Eurocode 8 too, are examined by means of design examples of plane and 3D frames. Some uncertainties in applying the code procedures, especially for defining the effective width of beams and the calculation of the connection between the concrete slab and the steel profile, are apparent. The non-linear analysis is implemented too, acquiring data about the influence of the effective width definition on the global strength and ductility of the structure; furthermore the effective q-factor results much higher than the one adopted for the design.

The FE models of composite beams and columns are carried out aiming at parametric analyses to calibrate simplified formulations for evaluating the deformability of the joint in the elastic and post-elastic fields to be implemented in lumped plasticity models. Particularly the base column connection realized by embedding the steel profile in the concrete block, showing high ductility in experimental tests previously carried out, is analyzed. The FE 3D model implemented by software DIANA is calibrated using the experimental results available on partially encased columns. The structural scheme is a simply supported beam loaded by a force in the middle span in order to have hogging moment i.e. the slab in tension. The parameters considered in the analysis are the percentage of reinforcement in the slab, the mechanical ratio R (the ultimate strength of the reinforcement area/ the ultimate strength of the steel profile area), the type of constitutive law for the structural steel (elastic-plastic, elastic-hardening), the role of the critical stress (i.e. the effect of local buckling), the shape of the steel profile and the effect of the connection between the two components.

The parametric analysis on the 3D model allows to define the embedment length to attain the full strength and develop the plastic deformation of the column.

As a result, the mechanical limitations of the steel reinforcement amount in the concrete slab in order to obtain a ductile crisis of the section are identified; furthermore an approach to evaluate the moment-curvature relationship, which takes into account the buckling phenomena through a critical stress value, is developed. These relationships can be multiplied by an equivalent length of plastic hinge, obtained through the parametric analysis, allowing to get the rotational capacity. This represent a simple tool for the nonlinear analysis of composite frames, in which the dissipation of the seismic energy is addressed at the beam ends, as typically occurs in case of framed structures. These formulations are then used to carry out non-linear analyses of composite frames for calculating the behavior factors, which result greater than those indicated by the Italian standard code NTC2008, proving the benefit of the structural type in seismic areas.

Finally two full-scale composite beam-column joints are constructed, they being a welded and a bolted connection, to be tested under cyclic loads. The aim is to characterize the response of the nodal zone in the elastic and post elastic fields.

I.8.4. Discussion

The specific objectives of the research program, like the appraisal of the state of art about the efficiency of non-linear analysis of composite frames by lumped plasticity models, the identification of parameters that influence the plastic hinge length for the composite beams under hogging moment, the implementation of the numerical models of the composite beams and columns, have been achieved. However the numerous results obtained are not enough for assessing final formulations to be applied for the design of new structures or the study of existing composite frames through non-linear analyses or to validate the behaviour factor of such structural systems. Further numerical analyses by a detailed FEM and comparisons with experimental results are needed. The same critical aspect can be highlighted for the innovative base-column joint, socket type that is defined in detail for a specific type of column (partially encased), but further analyses are necessary to generalize the results extending the model to other types of columns. Nevertheless the design of composite joints to be tested has been developed, albeit not planned in the research project, since the influence of beam-to-column joints on the overall response of the structures, both in the elastic field than in the plastic one, is well recognized and has been highlighted as a topic subject.

I.8.5. Visions and developments

In the field of steel-concrete composite constructions the knowledge about the seismic performances of frames is a paramount subject since this lack determines the limited employment of this structural type. As respect to steel or RC structures few experimental results are available and numerical models are more complex due to the presence of steel, RC and their interaction; furthermore joints are particularly articulated due to the connections of the two materials and the strong non-symmetric behaviour under hogging and sagging moments. A better understanding of the local mechanism of the RC slab in the nodal zone is needed and the study of new detail solution is fundamental for improving the dissipative response. In fact, considering the approach of the seismic design based on the resistance hierarchy the main topic is the non-linear behaviour of beams and joints that can be individuated as dissipative zone, while the column has to be more resistant. The study of a clever and balanced dissipative mechanism involving the beam and the joint that does not compromise the elastic response of the column is an interesting challenge.

Another important aspect is the definition of the effective field of application of the composite frames evidencing the benefits as respect to RC frames or steel braced structures through comparative examples of structures designed according to the code provisions.

Finally the effect of considering the composite beams in the seismic analysis of braced steel structures should be investigated. In fact usually the concrete slab is connected to the steel beam, realizing a composite beam, but the seismic analysis of the building is carried out considering only the steel members i.e. neglecting the contribution of the composite beams on the dynamic behaviour and the distribution of the seismic action.

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I 9. CONCRETE-STEEL COMPOSITE CONNECTIONS [UNITS]

I.9.1 Background and motivation

The actual Italian code is lacking in the suggestions for the realization of a full strength steel-concrete composite node. Some instructions are given only with regard to the welded connections. There are also considerable deficiencies with regard to the structural modelling of a composite structure. Actually the evaluation of the behavior factor for a composite structure is very similar to that of a steel structure. It is therefore of considerable importance to assess more accurately the real seismic performances of the composite typology and therefore to establish appropriate criteria for the evaluation of performance points of a composite structure, varying the limit states, in a pushover analysis.
The main objectives of the research are listed hereafter:
1) Definition of suitable new simple criteria for the design and modelling of an internal or external node and a whole steel-concrete composite structure;
2) Realistic evaluation of the behavior factor of a building realized by using steel-concrete composite frames;
3) Definition of simplified criteria for the evaluation of performance points on the capacity curve obtained by a pushover analysis.

I.9.2. Research structure

The research program is articulated in the following steps:

- Development of non-linear numerical models for the study of beam-column nodes and the entire building composed by moment resisting frames, in order to perform reliable static and dynamic analyses. The models are calibrated on experimental data available in literature and the results obtained by the research activity of the Sannio Unit;
- Development of new simple criteria to design the node and the whole composite frame, based on results obtained by an accurate numerical non-linear modelling;
- Extensive parametric analysis for an accurate assessment of the behavior factor.
- Determination of simplified criteria to identify the real performance points on the capacity curve of the composite structure, obtained for the limit states provided by codes in a pushover analysis.
- Draft of guidelines providing practical indications with reference to the above points.

Project activities are made on the basis of a complementary and coordinated collaboration with the Sannio Unit.

I.9.3. Main results

An extensive research on the state of the art for cyclic and monotonic modelling of the composite joints is performed. New models for more important components of the composite joint are implemented through the software Abaqus.
Some preliminary studies on the node and the entire composite frame are made in order to develop suitable models of calculus for next parametric analyses.
Two types of composite full strength joints, a welded and a flanged joint, are analysed in detail, applying a complete model based on the component approach, implemented into the Abaqus code and a simplified model, based on the Eurocode 3 and 4 suggestions. Both joints responses are validated on the base of literature experimental tests, with good results. A preliminary study on the influence of the node deformability on the whole response of a composite frame is developed for both types of joints. First results show a relevant influence of the joint for both the initial stiffness of the frame and the maximum deformability at

collapse. A parametric analysis is in progress to evaluate the influence of the node deformability on the evaluation of the behavior factor of the frame.
A procedure for modeling the composite joint and the plastic zone in the beams within a whole frame is developed. The aim is to deliver guidelines able to provide the designer with all the elements useful for a correct design of this complex structural typology. In particular, specific information are provided to the designer, in all design phases, for the modeling of two types of full strength composite node, one welded and one flanged. By using a model based on the use of concentrated plasticity, rigid-plastic hinges or non-linear link elements are also developed for the modeling of the plastic zone of the beams inside the frame, by referring to the indications currently provided by FEMA 356.
Finally, criteria for the identification along the pushover curve of points corresponding to limit states prescribed by standard are provided.
In the final part of the research a lot of comparisons and validations based on the results of literature and a 3D modeling are carried out using the computer code Abaqus for a single node and a sub-structure representative of an interstorey.
At last, as a case study, a steel-concrete composite frame of four floors is analyzed comparing the response varying the type of modeling, the length of the plastic hinge and the stiffness of the joint. On the basis of a parametric analysis the importance of various parameters on the seismic response of this type of frames, such as: length and rotational capacity of the plastic hinge, stiffness, strength and deformability of the joint, is highlighted.
It is found that values provided by standards for the behavior factor are close to those found numerically, though not always precautionary.

I.9.4. Discussion

On the whole, the research carried out allows to reach the main proposed objectives such as:
1) Definition of the nodal modeling of steel-concrete composite joints using the components method.
2) Modeling of the nodal zone and the plastic zone of the composite beam in a steel-concrete composite frame.
3) Evaluation of the influence of main parameters that affect the non-linear response of a steel-concrete composite frame.
With reference to point 1, for both welded and bolted composite joints, all components are defined in terms of resistance and stiffness. For point 2, the main objectives are achieved, although further investigations are necessary for the rotational capacity of joints and beams. For point 3, analyses are limited to plane frames and they should be extended to 3D frames.

I.9.5. Visions and developments

Steel concrete composite frames are potentially very efficient structures in seismic zone, but their design requires special attention for the complexity of the interactions between the component elements. For these reasons, in general, the designer finds strong difficulties in applying the directives of current regulations, which are frequently complicated. The development of a very detailed guideline, with accurate design explanations regarding various aspects such as: the modeling with conventional loads, the long term effects, the seismic modeling in an elastic design, the seismic modeling in a non-linear phase, is very opportune. For each of these aspects the development of simple and detailed explanation of principal design rules could be opportune. The development of adequate examples of calculation is also appropriate. Theoretical, numerical and experimental analyses on rotational capacity of composite joints and beams are an issue that needs further developments.

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I 10. SEISMIC BEHAVIOUR OF BRIDGE DECKS WITH CONCRETE-STEEL COMPOSITE CROSS SECTION [POLIMA]

I.10.1. Background and motivation

Steel-concrete composite (SCC) bridges are a common, economical and efficient type of highway bridges, especially in the range of short and medium span-lengths. These bridges are usually characterized by a continuous SCC deck resting on reinforced concrete piers, the latter providing the main seismic energy dissipation source. According to modern anti-seismic codes, the superstructure, the bearings and the abutments should remain in the elastic range during the seismic action. For this reason, the accurate evaluation of the seismic demand and of the corresponding capacity of all the resisting components of SCC bridges is very crucial for both the seismic design and the assessment of bridges. The design and verification of the SCC deck is of particular interest, due to the complex modelling issues and the nature of the acting loads (creep, shrinkage, vertical loads, seismic loads). The seismic assessment of SCC bridges becomes even more critical in presence of a transverse restraint between the deck and the abutments, often introduced for avoiding the use of expensive bi-directional joints, inducing a complex dual-load path behaviour in the bridge, not fully studied yet.

I.10.2. Research structure

The main objective of the research is to define simplified methods for evaluating the seismic response and capacity of multi-span continuous bridges with a SCC deck. In particular, the research aims at providing guidance to the modelling of the superstructure in terms of stiffness and resistance and to the analysis of local issues related to the transfer of forces from the superstructure to the substructures, at assessing the accuracy of the analysis method based on a response spectrum reduced by a strength reduction factor for the design of SCC bridges with transverse abutment restraint, finally, at the application of displacement-based method for the design of the particular class of bridges considered.
The research program consists of the following activities:

- survey of the most diffused typologies of SCC decks and bridge static schemes in order to highlight typical problems related to seismic response;
- analysis of local problems related to the transfer of longitudinal and transverse forces between concrete slab and steel girders and connection at abutments and piers top;
- analysis of the influence of construction sequence, creep and shrinkage on the transverse strength of the composite section;
- evaluation of the reliability of response spectra reduced by a behaviour factor for the analysis and design of SCC bridges with dual load path;
- definition of simplified methodologies for preliminary assessment and design of SCC bridges with abutment transverse restraint.

I.10.3. Main results

Three activities are carried out: 1) survey of recurrent configurations of SCC sections; 2) analysis of the seismic response of bridges with abutment restraints and evaluation of the reliability of response spectrum analysis; 3) proposal of simplified approaches for the seismic assessment and design of SCC bridges with dual load path behaviour.
A set of recurrent sections of SCC bridges with dissipative piers and a steel-concrete composite (SCC) deck restrained at the abutments is individuated. The modelling issues and the aspects concerning the resistance of the sections components (deck slab and girders) are

investigated. Deck sections are designed according to Eurocodes and composed by reinforced concrete slab and two steel girders, symmetrical as respect to the deck centerline.
A parametric study aims at investigating the changes in the seismic response of a 3-span continuous SCC bridge with dual load path behavior due to a variation in the relative deck-to-pier stiffness, described by the ratio *H/D* between the piers height *H* and cross-section diameter *D* (Fig. I 10.1). Furthermore, the elastic response of the bridges is compared with the inelastic response, for evaluating the strength reduction factor. The study highlights the importance of accounting for deck yielding in the response of SCC bridges with transverse restraints. Moreover, a simplified analysis approach based on linear elastic analysis and a global unique strength reduction factor, while often accepted in modern seismic codes, may provide inaccurate results for the specific system showing dual load path behavior.

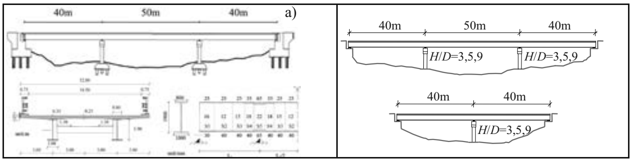

Figure I.10.1. Three-span SCC bridges analysed.

A simplified analytical model capable of describing the important aspects of the seismic behavior of the bridges is proposed. It consists of a 2D beam pinned at the abutments resting on intermediate discrete nonlinear hysteretic (elastic-perfectly plastic) supports representing the pier-bearing systems. By applying the Lagrange-D'Alembert variational principle and assuming a sinusoidal transverse displacement shape, the properties of a nonlinear single-degree-of-freedom (SDOF) system equivalent to the bridge is obtained. The SDOF system approximation can be employed for design purposes, but it is also used to obtain a simple approximate expression for the relation between the bridge global ductility capacity μ_{eq} and the piers ductility capacity μ_p in case of elastic deck and elastic-perfectly plastic homogenous piers. The derived expression is function of two non-dimensional parameters: α^2, as the ratio of the deck to pier stiffness, and β, describing the piers distribution along the bridge and inversely proportional to the number of piers. Both parameters can also be used to assess the accuracy of the sinusoidal shape in approximating the transverse motion of the bridge. It is observed that μ_{eq} can be significantly reduced with respect to μ_p in consequence of the contribution to elastic strain energy provided by deck bending. This is an important aspect because the global ductility capacity is related to the response modification factors (R-factors) currently used in seismic codes for the design of bridges with a reduced response spectrum.
Moreover, the influence of SCC deck yielding on the global dissipative capacity of the bridges is investigated. A set of non-dimensional geometrical parameters is derived for identifying the critical bridge configurations in which deck yielding occurs before the piers attain their ultimate ductility capacity. In these configurations, the dissipation capacity of the piers cannot be fully exploited and the global bridge ductility capacity is reduced.
In general, transverse yield curvatures and moments of a SCC deck vary along the length, due to different sections used and the influence of non-seismic loads. In fact, seismic transverse action induces a bending deformation on the horizontal plane and the relevant bending moment acts on composite sections subjected also to vertical bending due to permanent loads, shrinkage and creep. Moreover, construction phases must also be considered in determining stresses because dead loads act on steel girder only. A procedure for the transverse moment-curvature analysis is developed by accounting for the initial state of stress due to permanent

loads, shrinkage, creep and construction phases. It permits to determine the value of the transverse yield curvature Φ_{dy} even yielding occurs in the girder or in the slab rebars. The results of incremental dynamic analysis (IDA) applied to refined nonlinear FEM of bridges demonstrate that the proposed model is effective in unveiling the peculiarities of the behavior of the class of bridges analyzed and in particular its capability in accounting for the influence of deck yielding on the seismic capacity.

Concerning the design approach, the force-based design is contemplated in current seismic codes, and it entails using a response spectrum reduced by a behavior factor. According to the NTC 2008, the maximum value of the behavior factor that can be assumed in the analysis is equal to 3.5 in case of ductile piers and to 1.5 in case of abutments connected to the deck. Therefore, in case of bridges with dissipative piers and deck transversally restrained at the abutments, it is not clear what value of the behavior factor should be considered in the analysis. In order to clarify this aspect, the case studies are re-examined, in order to investigate a wide spectrum of seismic behaviors. IDA are carried out by considering natural earthquakes representative of different soil categories (A, B, C e D) and different values of seismic intensity increased until failure of the piers or of the deck is attained. The reduction factors, defined as the ratio between the forces measured on the elastic model and those evaluated on the inelastic model for the earthquake intensity corresponding to the ultimate conditions of the bridge, are calculated for the piers, the abutments and the deck. The results of the study show that the reduction factor for the piers (r_p) decreases with the increase of the ratio H/D. This trend is explained in part by the reduction of the pier ductility for increasing pier height, but mainly by the fact that the deck yields for value of $H/D>5$, thus preventing the full exploitation of the energy dissipation capacity of the piers. The reduction factor for the abutments (r_{ab}) assumes a value close to unity in all the considered cases. This is due to the redistribution of forces from the piers to the abutments following the plasticization of the first and due to the influence of higher modes on the abutment reactions. The reduction factor of the transverse bending moments in correspondence of the central span ($r_{M,sag}$) is always greater than 1, while that of the moments at the support ($r_{M,hog}$) can assume values less than 1. This is due to the change in shape of the diagram of the bending moments as a result of pier yielding.

Finally, a displacement-based method for the preliminary design of SCC bridges with abutment transverse restraint is proposed. The procedure is based on a sinusoidal approximation of the transverse displacement field and on the linearization of the hysteretic behavior of the piers, modeled by a spring and a damper in parallel equivalent in terms of secant stiffness and dissipative capacity. By assuming a model consisting of a beam on visco-elastic supports for the bridge and by applying the Lagrange-D'Alembert principle, the properties of a single degree of freedom (SDOF) system equivalent to the bridge are obtained. This system is used to estimate the seismic demand based on a comparison with the seismic response spectrum, reduced to take account of the equivalent damping of the system. First the target transverse displacement to be achieved under the seismic action, compatible with the piers and deck capacity, is selected. Subsequently, the amount of longitudinal reinforcement to be placed at the base of each pier is defined. The longitudinal reinforcement not only controls the maximum moment at the pier base, but also its stiffness, and thus the overall bridge stiffness and dissipation capacity. Obviously, the amount of reinforcement must be such that the seismic demand and the seismic capacity intersect in correspondence of the performance point, as identified in the ADRS plane.

The proposed design procedure is applied to a series of case studies consisting of 3 and 4 span bridges. Non-linear finite element models of the bridges are also developed and time-history analyses are carried out to test the proposed design procedure. The analysis results show that the proposed design method provides useful information for the preliminary design of the piers and permits to control the seismic demand on the various bridge components.

I.10.4. Discussion

Main objectives of the research are: 1) set up of simplified methods for the study of transverse behaviour of decks, considering the evolution of the state of stress in the concrete slab; 2) evaluation of the field of application of the linear analysis with response spectrum through the definition of appropriate behavior factors for the components.
With reference to the first objective, a strategy for modelling and analyzing the capability of composite concrete-steel decks is set up. It accounts for the complex state of stress due to vertical loads, shrinkage and erection phases of decks. Moreover, a simplified formulation for the transverse yielding curvature of the deck is determined.
With reference to the second objective, the analysis of several case studies evidences the limits of the design method based on response spectra reduced by the global behavior factor for piers and abutments, in case of bridges transversally restrained at the abutments and dissipative piers.
Definitely, the objectives are satisfactorily achieved

I.10.5. Visions and developments

The analyses carried out provide contradictory results as respect to indications given by the Italian codes about the behavior factor to adopt in case of decks transversally restrained at the abutments and dissipative piers. The use of a single structural factor leads to a non-conservative design of the abutments and restraint devices. Further deepening aimed at the calibration of simplified formulations for the estimation of two different structural factors for the design of piers and abutments through the force based approach are required.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 143-172, doi: 10.14599/r101304, © 2015 Doppiavoce, Napoli, Italy

TIMBER STRUCTURES

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1 INTRODUCTION

Wood is a building material that can be used for different applications, ranging from its implementation as structural component to interior design. In particular, if we refer to the residential market in Italy, a non-negligible percentage of new buildings is currently being constructed using a timber structure.

Modern timber construction systems allow the building of a structure in a very simple and competitive way, especially in relation to the time required for construction operations. Wooden buildings, traditionally associated with the idea of the mono- or multi-family single dwelling, can now be associated with the idea of a residential complex, with a notable area in plan and with a remarkable number of storeys. This is a new and more general use of wood as structural material, also thanks to modern assembly techniques and to the presence on the market of highly engineered components and elements (CLT panels, GLT glulam elements, LVL panels, other wood-based materials).

Wood is considered to be a material particularly suitable for earthquake-resistant structures, due to its low density, with a resistance to mass ratio similar to that of steel, and much more effective than other traditional materials, i.e. concrete or masonry. In short, this means that the stresses acting on a timber building in the event of an earthquake, being proportional to the mass of the building itself, are much lower. On the other hand, timber structures are generally more flexible than similar structures built with reinforced concrete or masonry: this proves to be a further advantage, as a flexible structure is typically less 'sensitive' to the consequences of seismic excitation.

However, timber has some critical characteristics related to the intrinsic brittleness of the material, at least in the case of tensile and bending stresses. Nevertheless, it should be noted that the timber building is never a ‘monolithic’ frame, but it is formed by several elements (beams, walls, floors), joined together through mechanical connections. Those connections, if correctly designed, can give a very positive contribution to the overall behaviour of the building, thanks to the plastic (or, generally, non-elastic) deformation of the metallic elements and to the friction between the contact surfaces, allowing for dissipation of the energy released during an earthquake. This was proved both by recent experimental research carried out on modern timber buildings, and by the survey done on existing structures after recent earthquakes. Many timber buildings, in Japan and China, withstood several earthquakes during centuries of life. A famous example can be cited, among many others: the Pagoda of Sakyamuni, in the county of Yingxian (Shanxi province), with more than 950 years of life, 67 meters high, entirely assembled with carpentry joints.

Referring to modern timber buildings, the most commonly used construction typologies involve the utilization of prefabricated wood panels, both light-timber framed walls or massive cross-laminated timber panels (CLT), assembled at each storey (the so-called platform frame construction process). In these systems, the prefabricated elements can

simultaneously perform as structural elements and as elements of the building envelope. The timber walls and floors can easily be prefabricated at the factory, and then assembled on site by means of steel mechanical connections.
With regard to light-timber-frame houses, they are very popular especially in the English-speaking world, primarily North America, New Zealand and Northern Europe. The wood timber-frame walls are based on equally spaced vertical studs and on the horizontal bottom and top plate: this ‘wood frame’ is closed with thin wood-based panels (OSB, plywood etc.), connected to timber elements using simple connections, like nails, staples or screws. The wood-based panels are used both as the bracing system in the plane of the wall and as closing system of the frame. The construction system resorting to the use of solid wood panels, typically cross-laminated timber panels (CLT), was recently introduced in Europe, but recently it was also adopted in other countries, i.e. Canada and the United States. The prefabricated planar elements are used both as floor and wall structures: the CLT elements are obtained by superposition of crossed arranged layers of wood boards, joined together by gluing. Timber buildings produced with one of these construction systems generally have remarkable seismic-resistant characteristics, when properly designed and assembled. In addition, possible post-earthquake interventions are highly simplified and quite easy as they consist in replacing the damaged parts of the connections.

The aim of the work done is to provide the basis for developing guidelines to be used for the design of earthquake-resistant timber buildings. The contents are mainly oriented to the designers, to Companies and, generally, to professionals who are actively involved in the construction process. Due to lack of space, references are inserted in the text to specific documents and papers produced within the framework of RELUIS 2010-2013 research program. Some observations are also reported, which can be used as background for modifications, clarifications or, simply, amendments to improve the Italian 'Technical Standards for Construction', Chapter 7 (CS.LL.PP., 2008), and Eurocode 8, Chapter 8 (CEN, 2004).

2 BACKGROUND AND MOTIVATION

The Italian market covering the residential construction area has multiplied by five the number of houses built with timber structure systems, between the beginning of 2006 and the end of 2010. From a recent Italian survey conducted on behalf of the ‘FederlegnoArredo’, in short FLA (Gardino, 2011), it is clear that timber buildings will further increase by 50% over the next 5 years starting from the data recorded in 2010. The main construction systems used to build houses are typically based on panel construction elements, whether they are made of light weight wood frame panels or cross-laminated timber panels. In these systems, structural panels can simultaneously develop the bearing function and be considered as building envelope. Such systems can be easily built using prefabricated and modular elements, produced in factory, and subsequently joined on site using mechanical connections. The simplicity of installation of timber construction systems, in addition to their sustainability and biocompatibility, makes this solution increasingly competitive for buildings if compared to buildings assembled using traditional materials.
Recent studies conducted by the National Institute of Geophysics and Volcanology (NIGV) confirmed that Italy is a country exposed to a medium-high seismic hazard. The NIGV has implemented the seismic hazard map for our area, which can be used to evaluate the probability of occurrence of an earthquake of a given intensity as function of the time range

and the local geographical coordinates. The problem of seismic risk mitigation, namely the reduction of adverse consequences of earthquakes on people, is the main problem of earthquake engineering, as well as one of the worst problems a national civil protection system has to deal with. The Italian construction regulations in response to this need have always been affected by updates, especially in the last decade, in order to incorporate all of the latest developments in the seismic design field. However, this updating process has not always been accompanied by an adequate and effective response by designers. Sometimes, the seismic design procedures have been proved excessively exposed to the opinion and decision of the single designer.

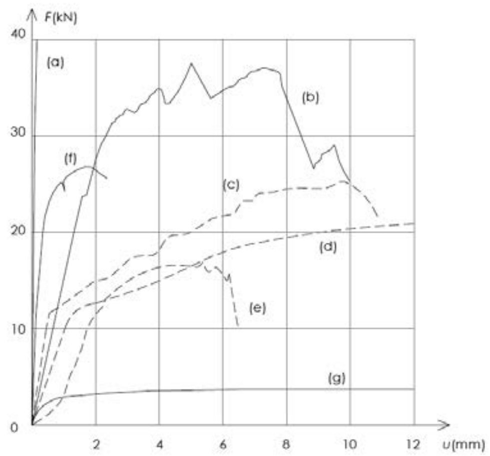
This research aims at studying several aspects about the design of earthquake-resistant structures for residential use. The main objective of this research program is to provide some rules for the design of new timber buildings in seismic areas. Design rules, accompanied by a series of design examples to use when calculating earthquake-resistant timber structures, are the target product of the research. The objective of these rules is to guide designers in the realization of timber houses, taking into account structural and technical problems. Such document aims at providing design procedures to be applied complying with the directives of recent seismic standards, Eurocode 8 (CEN, 2004) and the Italian Technical Standards for Construction, NTC 2008, (CS.LL.PP, 2008), integrated with suggestions for the design of cases not covered by those standards. Dimensioning models from recent developments, presented in documents of proven reliability, as well as advanced knowledge and experience gained in the international context, will be inserted within this document. This document can be considered as the first attempt towards the establishment of guidelines for the design of timber buildings.

3 RESEARCH STRUCTURE

The seismic performance of timber buildings is generally adequate thanks to their light weight and structural uniformity. In areas of high seismic risk, such as Japan and China, there are ancient buildings that prove the effectiveness of construction techniques adopted to withstand earthquakes. New timber buildings are built with various techniques and construction typologies. Many systems have evolved, with the progression from traditional (carpentry joints) to modern methods (engineered joints; e.g. mechanical connections or glued-in metallic joints) for building element assembly and from the use of solid wood elements to the use of engineered wood elements. Some of the house building technologies and construction systems in use in Europe were developed in areas where the occurrence of earthquakes is negligible. However, these construction systems are often employed with excessive optimism in Italy and even in areas of high seismic risk. In this work we intend to define a strategy for the design of earthquake-resistant timber buildings for residential use.

Wood is a building material that offers high load capacities in relation to its weight. The strength characteristics of wood are influenced by its anisotropy and its rheological behaviour. The strength and stiffness of a timber element varies with any defects present and with the orientation of the applied load with respect to the fibres.

The stress-strain curves (σ-ε) of a timber element show a behaviour that is markedly fragile, except for the elements compressed perpendicularly to the grain. Failure mechanisms due to bending or shear are brittle and must therefore be avoided in seismic zones. To obtain a ductile response of the structure, the design of the connections must respect the Capacity Design rules (CD rules), which ensure that the connections are the weakest links between timber elements.



CONFIGURATION OF THE CONNECTION

(a) Glued joints (A=12500 mm^2)

(b) Split-ring (d=100 mm)

(c) Double sided toothed-plate (d=62 mm)

(d) Dowel (d=14 mm)

(e) Bolt (d=14 mm)

(f) Punched plate (A=100x100 mm^2)

(g) Nail (d=4.4 mm)

Figure 1. Experimental load-slip curves for joints in tension parallel to the grain (modified from Racher, 1995).

The ductility of the system is thus achieved through the correct selection and the careful design of connections. Figure 1 shows the monotonic load-slip response (*F*-*u*) and the level of ductility obtained for a series of timber connections. The dissipative capacity of connections, under repeated loading, is related to the strength of the materials and to the geometric configuration of the joints. We simply show that only certain types of connection give the desired ductility level and hysteretic behaviour. The connections normally used in modern timber constructions are elements or metal devices that can transmit forces among structural elements.
A set of timber structures designed to ensure ductile behaviour, used in the European housing construction market, is shown in Figure 2.

(a) (b) (c)

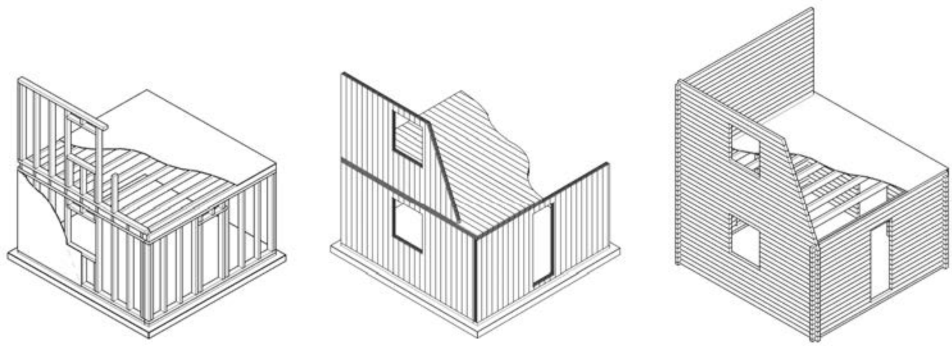


Figure 2. Timber house systems: (a) timber frame panels; (b) cross-laminated timber panels; (c) log-house.

If well designed, the construction systems of Figure 2 provide the desirable ductile behaviour. These first considerations highlight the importance of studying the dynamic response of different timber structure systems, with various types of connection and installation. The research was divided into tasks with specific objectives for the various structural types. The study of each structural type covers some specific aspects (Table 1): (i) the research on the state-of-the-art construction types investigated, (ii) experimental tests and (iii) non-linear static and dynamic analyses. For each structural type, we intend to develop a set of case studies on full-scale buildings with available static or dynamic tests on the elements or the whole structure.

For each specific structural type we define the evaluation criteria for structure element strength and deformation capacity, as well as the dynamic characteristics needed to define the seismic response (damping, dissipative capacity and deformed shape of the structure), also related to the structural analysis method adopted. Laboratory tests to define the mechanical properties of the materials, their use within buildings and their installation are expected from all Research Units (RU). We also include tests on some structural elements for each structural type. Some tests were carried out at the university laboratories involved in the research project. The results of this work are also supported by the experience gained during the post-earthquake reconstruction in the L'Aquila area in Italy. Some experimental data are available from the tests performed during the reconstruction phase and kindly offered us by the builders.

Table 1. Research topics and main activity of each Research Unit.

Research Unit	Research Topic	University of	Project leader
RU1	General aspects of timber construction systems	Trento	Prof. M. Piazza
RU2	Cross-laminated timber panel systems	Brescia	Prof. E. Giuriani
RU3	Solid wood wall systems	Naples	Prof. B. Calderoni
RU4	Timber frame construction systems	Trieste	Prof. N. Gattesco
RU5	Timber frame panel construction systems	Udine	Prof. A. Gubana
RU6	Development of numerical models for timber systems	Sassari	Prof. M. Fragiacomo

In conclusion, this work has been organized so as to develop a structured and consistent tool, experimentally and numerically validated, that allows us to define some rules of calculation and experimental verification for timber structures, with particular attention to their seismic behaviour. The research has been divided into tasks, with specific objectives for each construction system. However, we intend to develop a series of design rules for each of the timber construction systems. Some Research Units have been involved in laboratory work, performing full-scale tests on the structural timber elements currently employed in European construction systems. Other Units have worked on the development of numerical models that describe and reproduce the load-deformation curves found experimentally and that simulate the energy dissipation capacity of the various structure types.

The main results obtained by the Research Units during this three-year project will be presented in the following Sections.

4 MAIN RESULTS

Research Unit 1 (RU1) was responsible for summarizing the current state-of-the-art regarding the design and construction of timber buildings in Italy, also considering the recent post-earthquake reconstruction work at L'Aquila (Italy). The RU of Trento coordinates and guides all the other Research Units activities. The Unit has held discussions with researchers and has promoted the liaison with the Italian producers involved in the Project. The acquisition of all the results obtained from the individual Units, whether obtained experimentally or by numerical simulation, complete the work carried out during the three-year research project. Each Research Unit has specifically investigated some peculiar aspects of earthquake-resistant buildings, here intended as construction systems designed to ensure suitable performance in relation to the seismic intensity level. The current structural systems used to build timber buildings we make reference to in this task have been classified as follows: *cross-laminated timber panel construction systems* (Figure 3) and *light wood frame construction systems* (Figure 4).

The research activity of RU Trento was focused on an experimental campaign on full scale specimens of timber walls for their mechanical characterization, in terms of stiffness and resistance (Figure 5), whose main results can be found in (Piazza *et al.*, 2013). Research results and other useful design instructions were included within each annual technical report, in order to make it as clear as possible to the designers and engineers involved in timber engineering.

The research activity of RU Brescia has focused its research on three main topics: the drafting of guidelines for the design of CLT timber buildings with design case study, a theoretical and numerical study on the influence of floor diaphragm stiffness on the distribution of the seismic action among the shear walls, and theoretical and experimental studies on Platform Frame walls.

The research work of the Unit of Brescia has been devoted to investigating the deformability influence of diaphragms (due to the connections among the elements) on the distribution of the seismic action among the resisting walls. The work has been developed by means of numerical analyses as well as analytical approaches. The results show that, for both inclined screws and nails, the forces of a statically undetermined distribution tends to the solution of a diaphragm on simply supported beams, independently from the distance and the diameter of the connectors employed (Figure 6).

The Brescia RU has worked on the evaluation of the ductility and of the dissipative behaviour of timber Platform Frame walls (which basically consist of particleboard sheathing nailed onto a glulam timber framework). The study aims at assessing the behaviour factor q used when the Capacity Design criterion is adopted for the design process. Firstly, the attention was focused on the evaluation of the ductility and dissipation capacity of the local connections (sheathing-to-framing joints, hold-down and shear angle brackets); secondly, the experimental campaign focused on the full-scale shear wall behaviour.

Concerning the local behaviour of the connections, the experimental results pointed out that, depending on the type of nails (smooth or ring), the initial stiffness and ultimate strength can assume different values: smooth nails guarantee higher initial stiffness, whereas ring nails, by enhancing the axial-withdrawal capacity, guarantee higher ultimate capacity (Figure 7). So, the surface feature of the nail may be a crucial parameter, which in turn may affect shear wall behaviour.

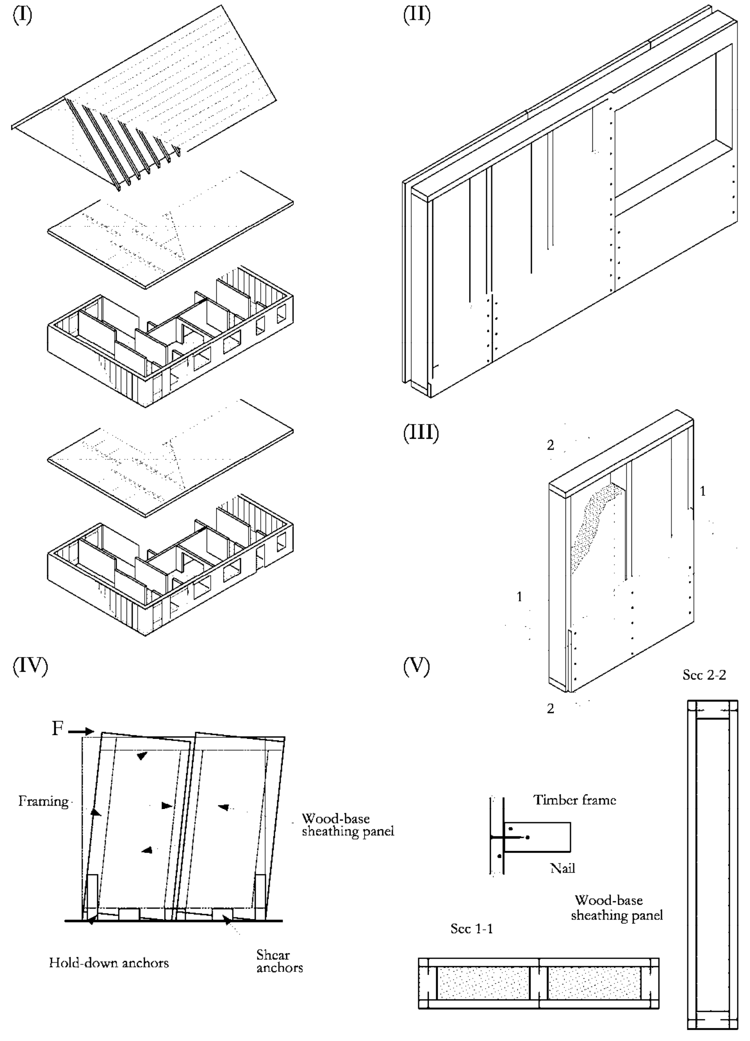


Figure 3. Building constructed using timber frame panel technology: (I) axonometric view of the structure; (II) wall configuration; (III) individual shear wall element; (IV) mechanical connections to prevent lifting and overturning of shear wall elements; (V) vertical and horizontal section of the wall and typical connection between frame and sheathing panels (Loss *et al.* 2013a).

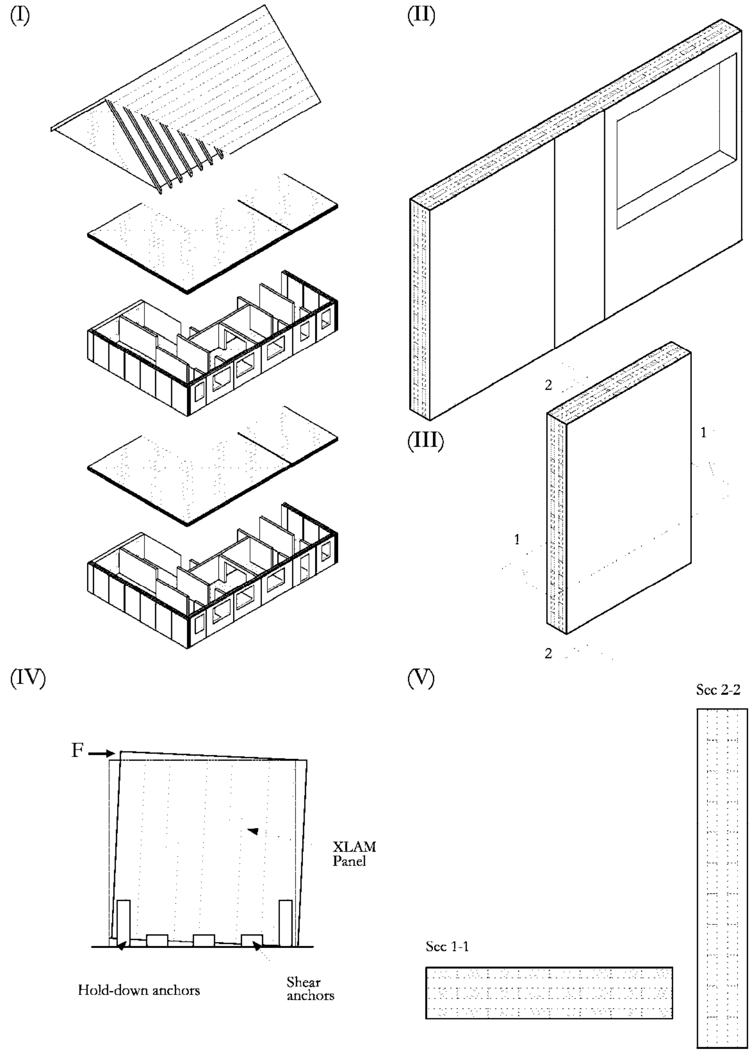


Figure 4. Building constructed using cross laminated timber panel technology: (I) axonometric view of the structure; (II) wall configuration; (III) individual shear wall element; (IV) mechanical connections to prevent lifting and overturning of shear wall elements; (V) vertical and horizontal section of the wall (Loss *et al.* 2013a).

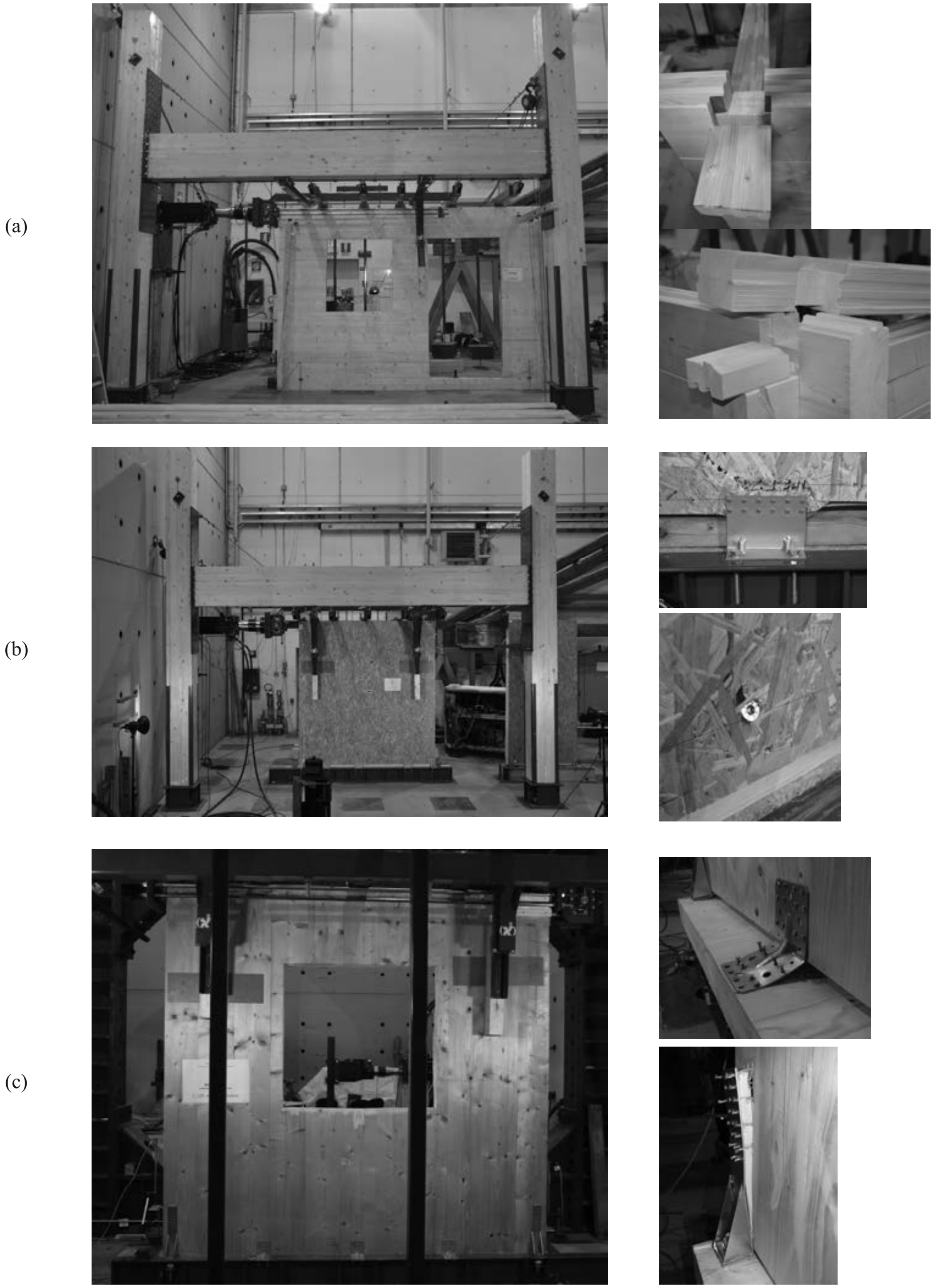


Figure 5. Test on log-house walls (a), timber frame walls (b) and cross laminated timber (CLT) walls.

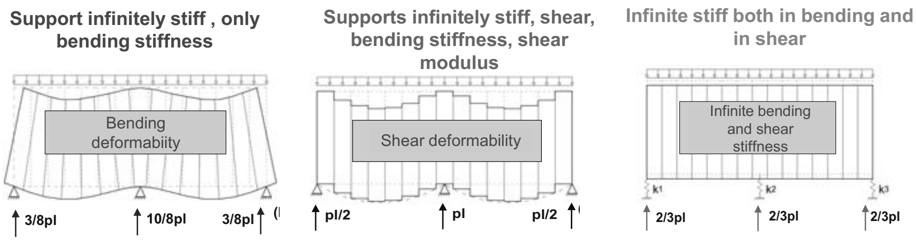


Figure 6. Distribution of the shear actions among shear walls.

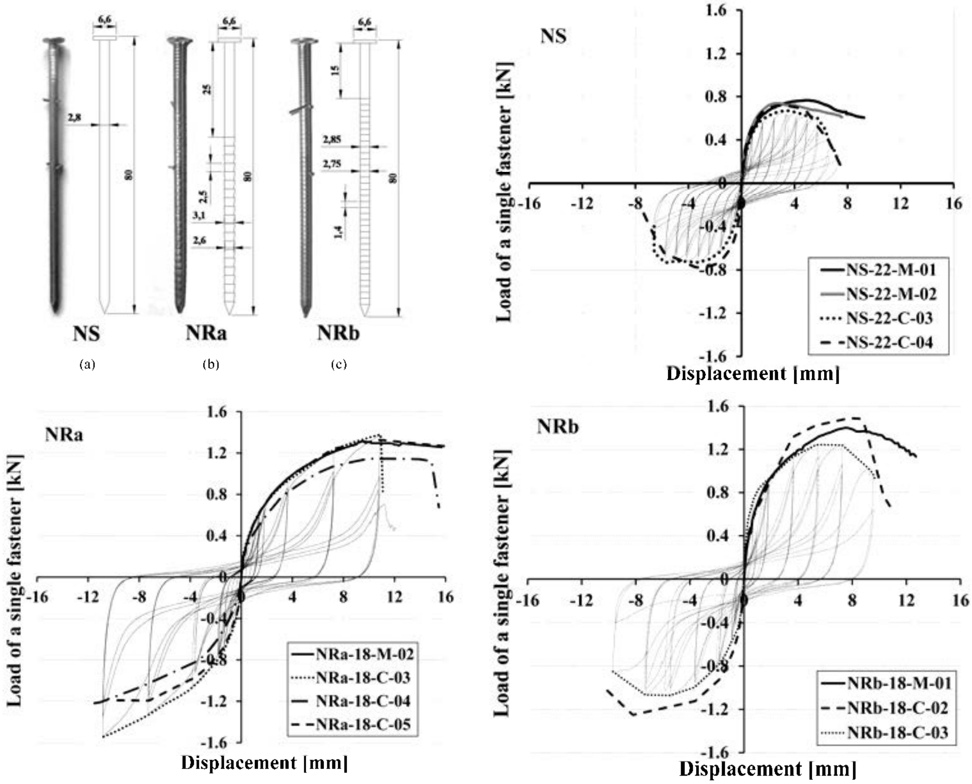


Figure 7. Experimental results of cyclic and monotonic tests on nailed connections. load vs. displacement curves for a single fastener: smooth nails (a), ring nails NRa (b) and NRb (c) (Germano *et al.*, 2012; Germano *et al.*, 2013b).

The experimental results were compared to the maximum strength and stiffness estimated according to the main European standards, such as EN 1995, DIN 1052 and CNR-DT 206. The Standards overestimate the strength of smooth nails by about 13% without taking into account the “rope effect”, which for EN 1995 and CNR-DT206 may be up to 15% of the connection resistance calculated according to Johansen's theory, by considering the formation of two plastic hinges in the nail shank, as experimentally confirmed. For ring nails, characterized by one plastic hinge, an underestimation of more than 30% may be observed, which points out their important withdrawal capacity. Regarding stiffness, the Standards' formulation provides an initial stiffness value in good agreement with the experimental results for smooth and NRb ring nails with a difference ranging between 18 and 32% depending on the code considered. The experimental stiffness of NRa nails resulted 75% lower than the calculated one, because the Standards do not consider the effect of shank surface which may create an initial clearance in the timber connected elements.

Furthermore, on the basis of the test results, according to the requirements of Eurocode 8 and Italian Standard for dissipative zones, only smooth nails and ring nails with a limited surface roughness of the shank (type NRb in Figure 7) can be employed for structures of High Ductility Class (DCH). NRa nails should be used for Medium Ductility Class (DCM) only, since they showed a strength reduction greater than 20% for a displacement equal to 6 times the yield displacement v_y.

A test bench was designed to study a full-scale shear wall (Figure 8), which was tested under cyclic reverse loading displacement of increasing amplitude. The 2500x2500 mm walls consisted of a timber frame with 5 glulam studs hinged to the upper and the bottom rail. The frames were sheathed on both sides with two vertically oriented 1250x2500 mm particleboard panels (P5 type according to EN 312 having a thickness of 18 mm). The panels were fastened to the frame by NRb ring nails with a constant spacing of 100 mm along the edges of the particleboard panels and 150 mm along the inner stud.

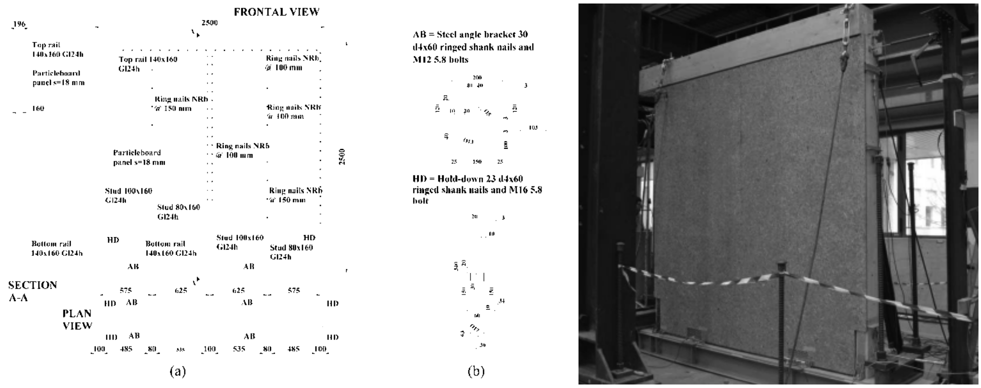


Figure 8. Light timber frame walls tested by the Brescia RU (Germano *et al.,* 2013a,b).

Two different levels of vertical load were applied. The first shear wall (SW-0) was tested without a vertical load, while the second wall (SW-100) was also subjected to a vertical load of 100 kN (equivalent to 40 kN/m), which may represent the gravity load carried by a ground floor wall of a three-storey building. The cyclic behaviour of the shear walls was not affected by the vertical load, since hold-downs and shear brackets, which were designed with an adequate overstrength factor (equal to 1.5), efficiently restrained the uplift and the sliding of the wall (Figure 9a).

The shear walls collapse at about 2.5% of drift due to low cycle fatigue fracture of the nail connection between sheathing panel and timber frame. High strength steel nails, commonly used in wood frame structures, are characterized by low ultimate tensile strain (ε_{su}<3%) resulting in a limited capability to dissipate energy for high drift values and a large number of cycles.

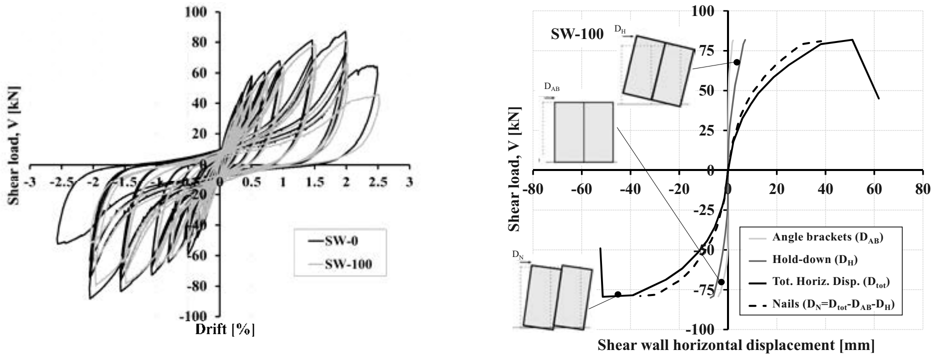


Figure 9. (a) Experimental results on Platform frame wall: shear action against the inter-story drift; (b) contribution of each type of connection on the horizontal displacement of the SW-100 shear wall (Germano *et al.*, 2013a,b).

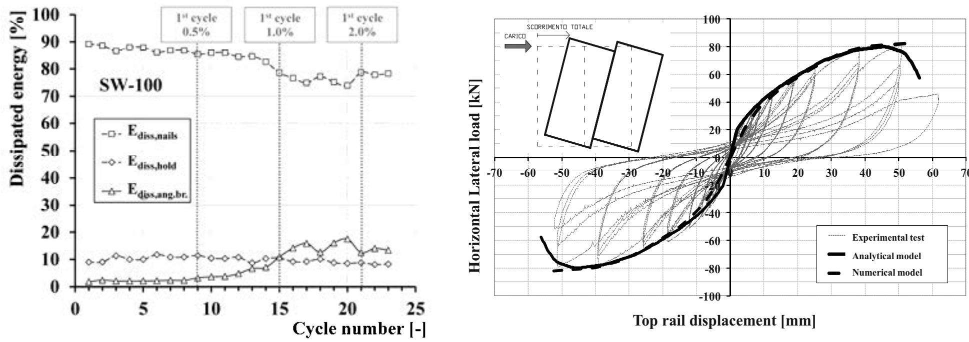


Figure 10. (a) Energy dissipated by each connection (Germano *et al.*, 2013a,b); (b) comparison between experimental tests and analytical/numerical ones (Germano *et al.*, 2013c).

Moreover, the dissipation capacity and the top displacement of the shear wall strongly depend on the sheathing-to-frame joints, whereas shear angle brackets and hold-down connections do not affect the global behaviour of the shear wall, as they were designed with an adequate overstrength, according to the CD approach (Figure 9b). Consequently, the wall damage resulted evenly spread in the several nailed connections along the wall frame, which dissipated about 80% of the total energy (Figure 10a). Furthermore, a ratio of about 4.5 was experimentally measured between the sheathing-to-frame relative slip and the shear wall horizontal displacement due to panel rotation. This result is consistent with the value that can be analytically calculated by means of the formulations found in literature.
Moreover, the q factor deduced from the experimental tests is only about 2.5. It should be noticed that this value may be consistent with a DCM structure, even though the particleboard-to-frame nailed connections satisfied the requirements of Italian Standards and Eurocode 8 for the dissipative zones of DCH structures, which may be designed with a behaviour factor up to 5.
Finally, an analytical approach is proposed in order to simulate the global behaviour of the shear walls. Timber frame and sheathing are assumed to be elastic together with the shear angle brackets and the hold-down connections, while only the sheathing-to-frame connections are able to develop a non-linear behaviour. The analytical results match very well the experimental and numerical outcomes (Figure 10b).

Numerical and theoretical studies were developed by the Brescia RU concerning the role of the connections between XLAM panels in the seismic response of structural systems. Two characteristic layouts of the walls were studied: (a) multi-storey walls made with overlapping single-storey XLAM panels (Figure 11a); (b) multi-storey walls with coupling action between adjacent XLAM panels (Figure 11b). Non-linear analyses pointed out that while in structures with overlapped XLAM walls the displacement demand is generally lower than deformation capability guaranteed by the nailed connections, in the case of the coupled walls the ductility both of the connections between adjacent panels and of the hold-down connections at the wall base is not sufficiently adequate to guarantee structure deformation capacity greater than the displacement demand. As a result, for the design of XLAM buildings according to the CD approach, a behaviour factor not greater than 1.5 is strongly recommended (Metelli *et al.*, 2012).

(a) (b)

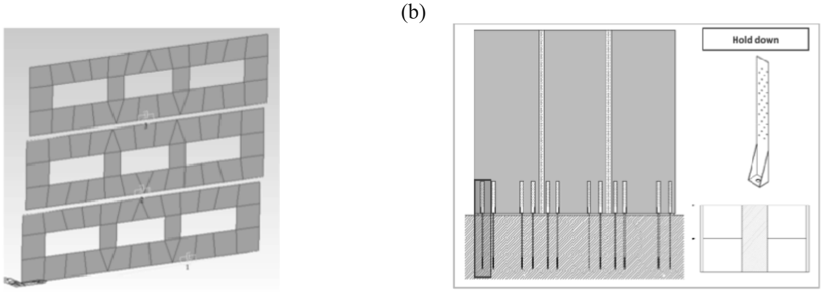


Figure 11. Overlapped walls (a) and coupled walls (b) (Metelli *et al.*, 2012).

The Research Unit of Naples has worked to develop simplified models for the design and verification of buildings built with cross-laminated timber panel construction systems. These models are based on the outcomes obtained in preceding years, concerning the calibration of complex numerical finite element models, in which panels and mechanical connections at various floors and at the base, and their non-linear behaviour, have been properly taken into account. In such models, plasticity is concentrated in the contact and anchoring area of the panels. From the results obtained using complex models, a simple mechanical model has been defined which considers the non-linear behaviour of the panels concentrated in sections close to the adjacent planes and at the foundation.
Specifically, starting from the results obtained with the complex FEM model, the Research Unit of Naples proceeded to:

I. define buckling verification criteria for the CLT panels used within the load-bearing walls;
II. calibrate a simplified equivalent frame model for structural analysis.

To determine the ultimate bending moment of the panel-to-connections system, researchers started from the analogy of a common reinforced concrete section, in which the reinforced elements are formed by mechanical connections, while compressive strength is ensured by the panel-to-panel (and panel-to-foundation) contact area. According to this hypothesis, the compressive strength of the wood to be used is the one perpendicular to the fibres.
The procedure for the determination of the resistance bending moment firstly considers the connection systems and the bond of wood in the compression orthogonal to the fibres. In this work, a parabola-rectangle diagram of wood is assumed, while for connection systems a

stress-strain relationship of steel or connectors can be assumed, depending on the hierarchy rules employed: the plasticization of the steel plate or the ‘yield’ of connectors. Then, the position of the neutral axis can be determined using an iterative procedure, based on translation and rotation equilibrium equations. The ultimate bending moment is the one that balances internal and external forces.
In the definition of the ultimate bending moment and in the construction of the M-N domain of the section (Figure 12), the ultimate strain of the wood in the direction orthogonal to fibres has significant influence (common values up to 10% and over). The analyses performed confirmed that the higher the value of the ultimate compression strain, the higher the ultimate bending moment and the curvature ductility of the section. In this work, an ultimate strain between 1% and 5% has been considered.

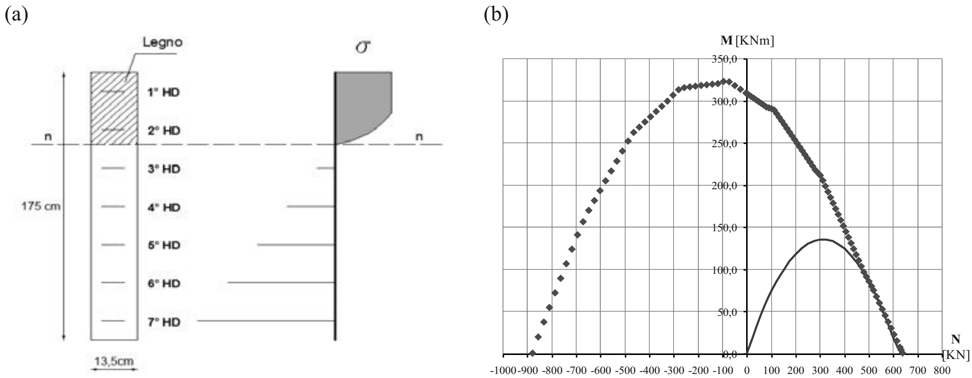


Figure 12. (a) Analytical model of the cross-section; (b) M-N Domain.

In order to validate the procedure, the ultimate bending moment value obtained from the simplified model was compared with the one provided by the FEM model with implemented system-to-panels connections at the collapse state. In all the recorded cases, the deviation of the results is less than 5%. The analytical M-N domain is preparatory to the definition of a simplified equivalent frame model with concentrated plasticity, which can be used to perform non-linear analyses.
The analyses performed with the FEM model have highlighted the lack of efficiency offered by lintels in the coupling of the piers with respect to seismic actions. Thus, the most suitable computational model of the wall to be used in design and verification is the one with full cantilever beam connected at each floor by pendulums. Therefore, even in the definition of the simplified model, the equivalent frame model has been implemented with macro-elements connected with pendulums: the uprights elements follow the centreline of piers while pendulums follow the centreline of lintels. A first set of non-linear static analyses were performed with this model, using flexural plastic hinges in upright elements (with known axial force) and with ultimate bending moment estimated with the procedure described earlier, and neglecting the behaviour of mechanical connections.
Comparison between the pushover curves extracted from the refined FEM model and the equivalent frame model has demonstrated that the latter behaves with higher elastic stiffness. Indeed, in the first case the intrinsic deformability of connections (hold-downs and steel brackets) has been considered negligible. On the other hand, a good agreement between the ultimate strength estimated by the simplified and refined model confirms the reliability of the

method used for the calculation of the ultimate bending moment. However, this shows that it is better to take into account the behaviour of mechanical connections in the evaluation of the final response of the structure.
In the simplified model we have introduced several springs with an equivalent elastic stiffness evaluated from the force-displacement curve of the mechanical connections used. In particular, we have introduced a rotational spring (k_{ϕ}) to model the flexural stiffness of the panel-to-connections systems (evaluated considering bending moment and axial force action), and linear springs (k_t) which simulate the shear stiffness of connections.
Comparison between the curves obtained from the new equivalent-frame model (with springs) and the refined FEM model highlights a good approximation in the evaluation of the global deformation of walls. Within the framework of the simplified model, the next step will be to define a reduced elastic stiffness to be assigned to the vertical structural elements (wooden piers), which takes into account the intrinsic deformation of connections.
The research work of RU Trieste has concerned the study of the dynamic behaviour of multi-storey wooden buildings (Figure 13).

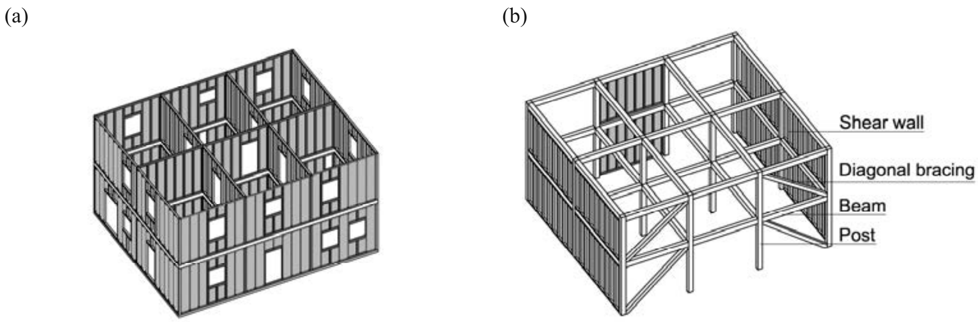


Figure 13. Timber frame panel systems (a) and timber frame systems (b).

In this structural type, the walls are made with wood-based panels nailed onto a framework of wooden studs and joists (Figure 14). The walls, which have the task of bracing the structure, are connected to a timber frame made with continuous columns and beams, with the task of withstanding the vertical loads.

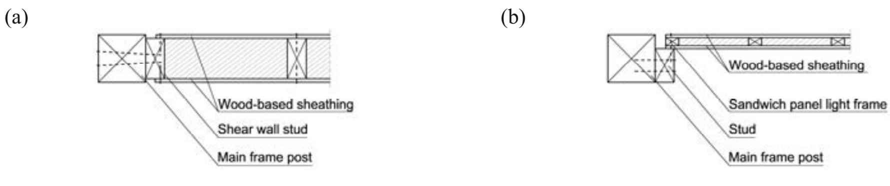


Figure 14. Nailed connections between shear walls and posts: simple shear wall panels (a) and sandwich panels (b).

Some experimental tests were performed on two wooden walls 2105x2870 mm in dimension, with the purpose of assessing the effective capacity and deformability in the plane. The lateral studs have a 160x200 mm section in one sample and a 140x160 mm section in the other. The panels of wood particles, arranged on both sides of the frame, were fixed with 2.8/70 nails, arranged at a distance of 50 mm on the outer studs and the top/bottom joists and at a distance of 100 mm on the interior studs; the thickness of the panels is 20 mm for the first sample and 15 mm for the other.

The tests were conducted under displacement control and the samples were subjected to a sequence of symmetrical cycles, with increase of the maximum horizontal displacement imposed at the top of the wall (cyclic shear test). The experimental results have shown that the resistance is slightly higher in the thicker wall (128 kN), compared to the thinner one (119 kN). On the contrary, the stiffness in the second sample was greater (5.5 kN/mm) than that of the first one (4.2 kN/mm).

The results of the two tests were compared with those of the five cyclic shear tests conducted in the course of the project to highlight the differences in behaviour and response in function of the different characteristics of the samples (different sizes, different spacing among the nails connecting the panels to the frame, different application of the hold-down, different type of connection at the wall base, presence of openings, etc.). All tests have identified some of the main shortcomings of the system in terms of response to seismic actions. In particular, the influence of the following aspects was evaluated:

I. positioning of the connections hold-down: the placement above the panels causes shear failure of the latter in the area close to the hold-down; instead, by fixing the hold-downs directly to the studs of the frame, the nailing connection determines the crisis of the wall;
II. restraints at the base of the wall: the shear slip at the bottom of the wall, which is negligible in the case of connections to ground via threaded rods, becomes appreciable in the case of connections with nailed steel angles;
III. size of the wooden elements of the framework: the shear resistance of the wall increases by very little when passing from elements with 140x160 mm cross section to 160x200 mm elements;
IV. spacing of the nails connecting the panels to the framework: when halving the spacing of nails, a slight increase in the stiffness of the wall and a considerable increase in resistance (almost proportional to the number of the nails) is observed;
V. panel material and thickness: very small differences in terms of stiffness and strength were noted between particle boards and OSB panels;
VI. perforated walls: the presence of window openings in the wall, if not too extensive, do not appreciably affect the strength of the wall, whereas stiffness is significantly reduced in comparison to walls without openings.

On the basis of the experimental results (Figure 15), a theoretical study was carried out to define an analytical method that allows to correctly estimate the stiffness of the walls to be considered in the evaluation of the horizontal actions shared among the resisting elements (shear walls).

A simplified model of the walls has been proposed using two-dimensional elements or extensional diagonal springs, whose stiffness characteristics are measured taking into account the contributions of the shear and flexural deformability of the wall, the deformability of the connections of the wall to the base and to the columns and of the panel to the framing (Figure 16). In order to consider the deformability of the base connections of the columns (strong angles or hold-down), the use of tensile extensional springs placed at the bottom of the frame was suggested. These theoretical studies were also extended to cases of walls made with multiple segments and walls with openings, proposing simplified procedures.

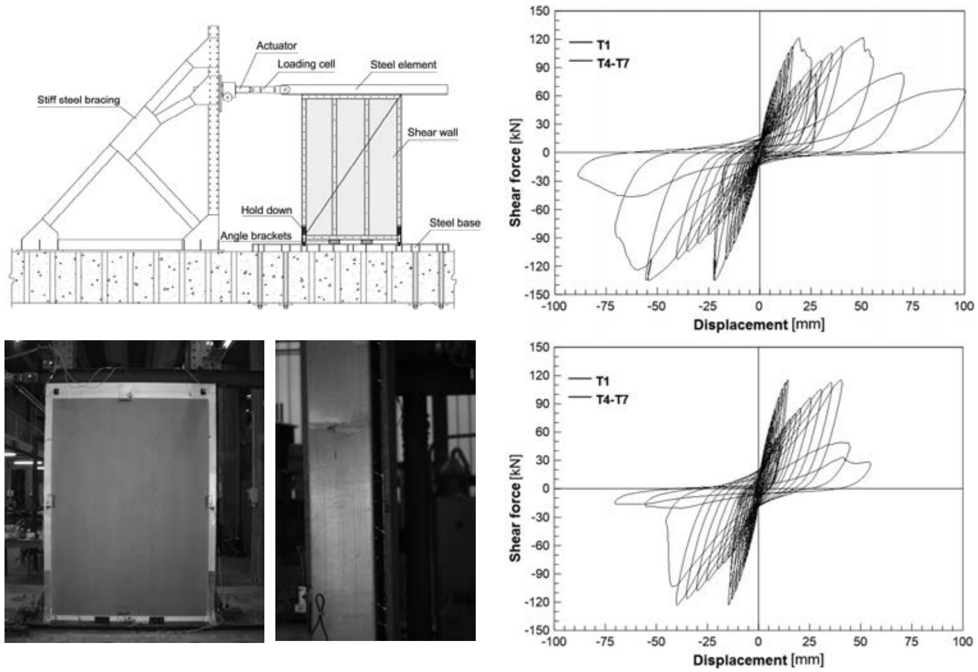
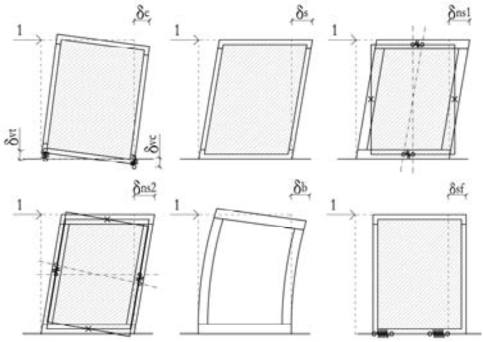


Figure 15. Experimental shear tests on timber shear walls (Gattesco and Franceschinis, 2012).



δ_c deflection due to the rotation at the base of the shear wall;

δ_s deflection resulting from shear deformation in the sheathing

δ_{ns} deflection due to nail slip in the perimeter connections between sheathing and framing, resulting by summing the displacements δ_{ns1} (due to sheathing-joists connections) and δ_{ns2} (due to sheathing-studs connections)

δ_b flexural deflection of the timber frame as a cantilever

δ_{sf} deflection of the shear connectors at the base of the wall

Figure 16. Contributions to shear wall deflection.

The relationships proposed to model the behaviour of the walls were compared with experimental results and provided good estimates of actual behaviour. The comparison with the relations proposed by foreign legislation (New Zealand, United States and Canada) gave comparable results. However, it should be noted that in such standards the contribution of the hold-down is based on a known deformability of the hold-down. This information, however, is not currently accessible for products available in Italy and therefore it is necessary to perform experimental tests on hold-downs before starting with the wall design.

A numerical simulation was carried out to evaluate the behaviour of multi-storey frame structures. In particular, the behaviour of a two-storey residential building was studied (plan dimensions 16x10.5 m, inter-storey 3.0 m), consisting of 4x4 columns connected with beams. The frame was modelled through one-dimensional elements with linear elastic behaviour, hinged at the intersections. The bracing walls were modelled with two-dimensional elements with orthotropic behaviour and were connected to the frame with equivalent non-linear springs able to simulate the actual behaviour of the connections. The characteristics of the springs, involved to transfer the shear stresses among elements, were calibrated through refined numerical models able to simulate, in a realistic way, the interaction between the connectors (nails, screws, bolts, dowels, etc.) and the wooden elements. Each floor was considered as rigid in its plane.

The analysis allowed to evaluate the capacity curve of the structure through a non-linear static analysis and to assess the value of the behaviour factor to be used in seismic design through a linear static analysis. The results showed that the maximum value of the behaviour factor allowed by Eurocode 8 and by DM 14/01/08 for this type of structures may be unsafe.

The experimental tests, the case studies and the numerical simulations provided important knowledge and were very helpful in the development of rules for the seismic design of timber structures.

In the first two years of the Project, the UR of Udine searched the literature on traditional timber truss building in seismic regions in and near Europe. Technical papers on modelling timber trusses and the evaluation of seismic behaviour were collected. The aim of the first part of the research work was to implement a simple procedure for nonlinear structural analysis, running on commercial FEM software, for use by structural designers. The first applications have been tested on experimental test results for a timber truss and we find good correspondence between numerical results and experimental data.

The scheduled work was completed during the third year of research activity. In particular:

I. some of the rules for the design of timber frame structures were discussed and prepared;

II. an example of a timber frame building design was completed, experimental tests and numerical simulation were done on CLT panels subjected to shear loads and on Timber to CLT composite section beams.

Some preliminary documents regarding the analyses and the study of some aspects of the calculation and numerical simulation of timber frame structures were processed in order to draft some guidelines accompanied with a case study.

A new series of shear tests was completed on only 3-layer CLT panels suitable to create the diaphragm effect in restoration and retrofit interventions on existing buildings (Figures 17, 18).

The experimental results were compared with the outputs of numerical models. Panel shear strength values are compatible with stress levels induced in the building diaphragms subjected to horizontal seismic forces (Figure 19).

Four Timber-to-CLT composite section beams were tested. The CLT panels were made only of 3 layers for a total height of 6 cm. Timber screws were used to connect the panel to the beam. Linear and non-linear models of the beams were analysed by means of FEM simulation and the shear-slip screw correlation was calibrated on force-slip experimental tests.

(a) (b)

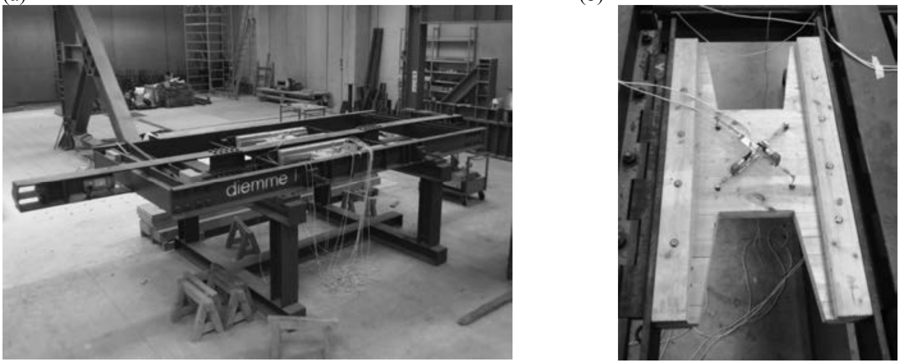


Figure 17. Shear test rig (a) and undeformed panel (b).

(a) (b)

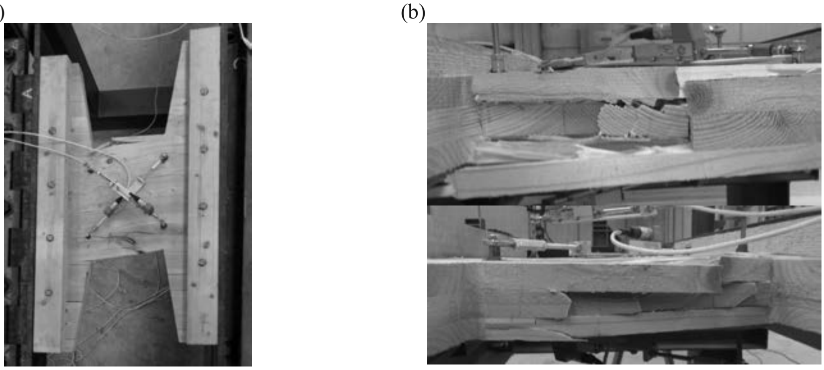


Figure 18. Failure of panels: overhead view (a) and two lateral views (b).

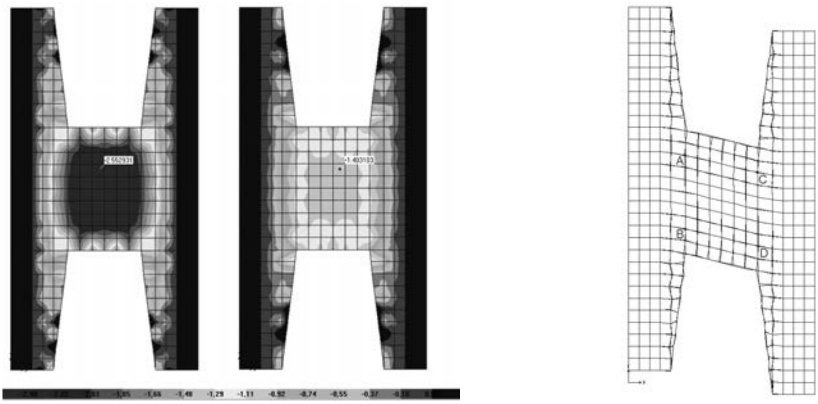


Figure 19. FEM model of the shear test performed on a panel.

The numerical model developed by the UR of Sassari (Rinaldin *et al.* 2013) has been used to simulate the behaviour of CLT and light-frame buildings. Two different buildings were modelled:

I. a CLT single-storey building tested by IVALSA at the University of Trento;

II. a light-frame building tested by Fischer *et al*. (2001) in the US.

CLT building

A single-storey CLT building was tested at the University of Trento by CNR-IVALSA. This building is composed by an assemblage of CLT panels connected to the foundation with angle brackets and hold-downs. The connectors used, with the same number of nails, had been singularly tested previously. The building is 7×7 m in plan and 3.1 m high. The walls are 85 mm thick and the floor is 142 mm thick with a central opening (Figure 20a).

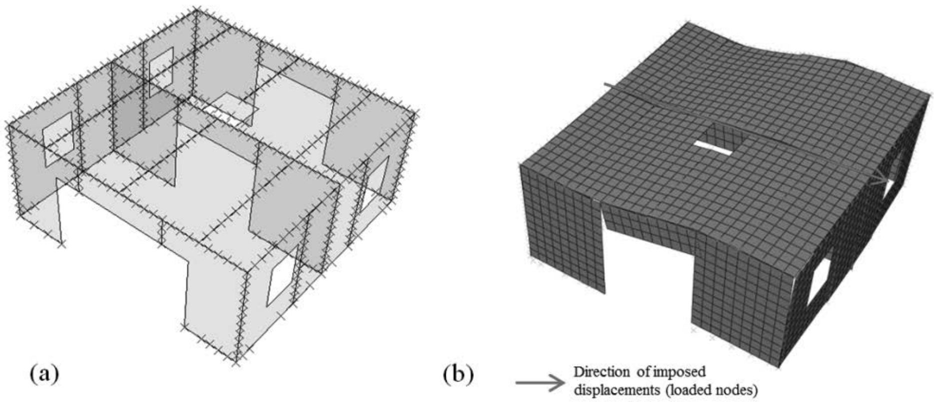
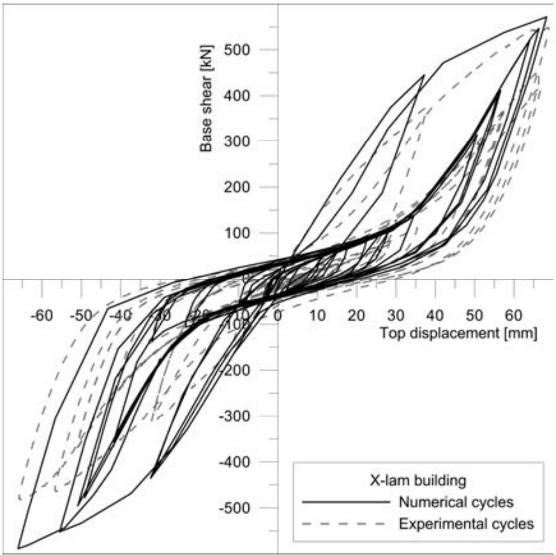


Figure 20. (a) 3D views of the single-storey CLT building model with springs marked with crosses; (b) deformed shape with 20 times amplification factor (Rinaldin *et al.*, 2014).

Pseudo-dynamic tests were performed in three different configurations obtained by varying the width of the door openings. In this work, the configuration with a non-symmetric layout with respect to the direction of the actuator (Figure 20b) was investigated. This configuration has two external openings, 4.0 m and 2.25 m wide, respectively. All the wooden parts have been modelled in Abaqus with elastic shell elements (S4R). The size of the adopted mesh is a convenient value to ensure that the connections are placed as close as possible to their real position and that the elements are regular 5304 springs used and placed with the help of an automatic mesh-maker developed on purpose; their location is visible in Figure 20a.
Every spring has the parameters obtained by calibrations made on experimental data of single connectors (Gavric *et al.,* 2011 and 2012); all wall-to-wall joints have the properties of LVL spline connection tests (Gavric *et al.,* 2012), while every single floor panel is connected to the adjacent ones with half-lap screwed joints (Gavric *et al.,* 2012).

(a)



(b)

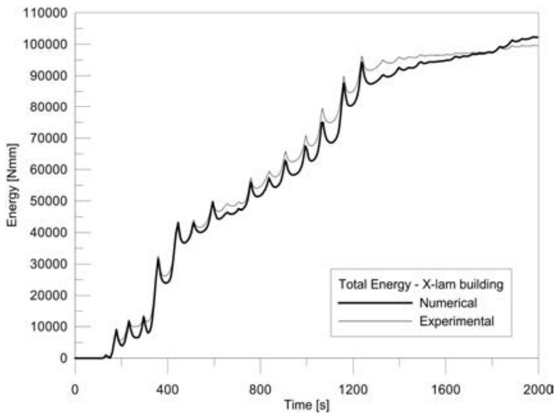


Figure 21. Experimental-numerical comparison of the hysteretic cycles (a) and time-history of total energy (b) of the single-storey X-lam building tested by CNR-IVALSA at the Univ. of Trento, Progetto *Sofie* (Rinaldin *et al.*, 2013a).

The vertical connections between perpendicular walls were modelled using springs with the same properties as the springs used for in-plane wall-to-wall connections.

The building was subjected to a pseudo-dynamic test that simulated the earthquake of Kobe JMA 0.5g. A non-linear static analysis was performed, imposing the seismic displacement at the top of the building. A typical deformed shape can be seen in Figure 20b, while results in terms of total base shear force vs. top displacement are presented in Figure 21a. An overall good accuracy of the experimental behaviour can be observed; the CLT walls did not deform considerably as most of the deformation was concentrated in the springs.

The numerical model slightly overestimates the backbone response of the building. In Figure 21b, numerical and experimental responses are compared in terms of total energy.

The final numerical value of the total energy is 2.85% higher than the experimental one, due essentially to some approximations of the narrow inner cycles that the model predicts at low displacement amplitude (Figure 21a).

Light-frame building

A light-frame building tested by Fischer *et al.* (2011) was modelled and the results were compared with the experimental data. The 2-storey building was made with light-frame walls as displayed in Figure 22a. The building is 4.88×6.11 m (16×20 ft) in plan and 2.44 m high.
The building was modelled using a couple of diagonal springs for each light-frame wall, arranged as in Figure 22b. The same wall arrangement as in the numerical model developed by Du (2003) was used. Du's model used the cyclic results of the 2.44×2.44 m light-frame wall tested by Dolan (1989) to characterise the building. For this reason, the geometry of the building was slightly modified to accommodate the different properties of the wall tested in Dolan (1989).
Each wall was modelled using two diagonal springs and rigid truss elements pin connected to each other, representing the perimeter frames of the wall. The diagonal springs were calibrated upon the experimental results reported in Dolan (1989).
The model was restrained with simple supports at the base and the kinematic hypothesis of rigid floor diaphragms was applied. A modal analysis was performed, returning a natural vibration frequency of 5.25 Hz, which is fairly close to the value of 5.03 Hz found by Du (2003).
A dynamic analysis was then performed, using the Northridge ground motion (Newhall station, 90° component, 1994). The analysis was stopped when the first storey reached a drift equal to 3% of the building height, which is regarded as an upper limit corresponding to the attainment of the collapse limit state in Du (2003).

(a) (b)

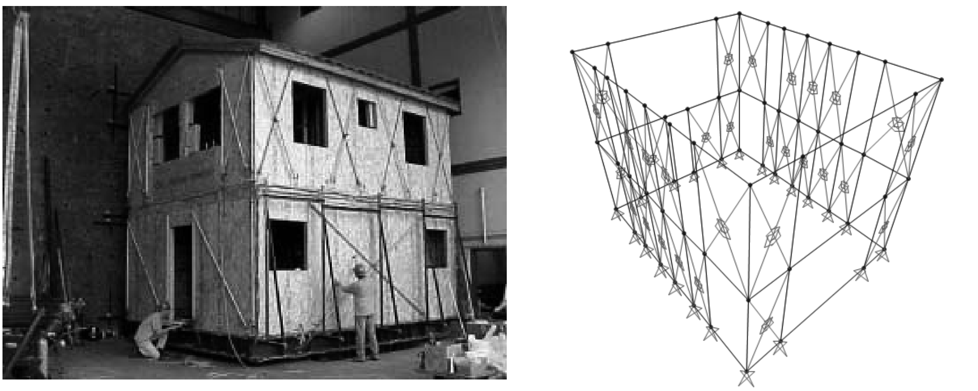


Figure 22. (a) Photo of the tested building (Fischer *et al.,* 2001) and (b) layout of the resisting walls (Rinaldin *et al.,* 2013b).

5 DISCUSSION

The Research Units (RUs) involved in this task of the RELUIS project have dealt primarily with the study of the seismic behaviour of timber buildings. The goal of the work team was to define simple design guidelines based on the results obtained from the experiments and numerical analyses carried out.

With particular reference to the tests performed on full scale elements, it has been possible to evaluate the non-linear behaviour of the elements also and especially in relation to the connecting devices used. The collapse mechanisms have been defined for structural elements, energy dissipation capacity, ductility and other performance parameters (Figure 23). The tests helped to deepen the state of knowledge on the current construction techniques, on the construction practices and on the materials used in the construction of timber modern buildings. In addition, starting from the tests, it has been possible to verify some limits of applicability of the rules and models included in the current versions of the standards, in particular the “Technical Standards for Constructions 2008” (CS.LL.PP., 2008) and “Eurocode 8” (CEN, 2004), for the part related to timber constructions.

(a) (b)

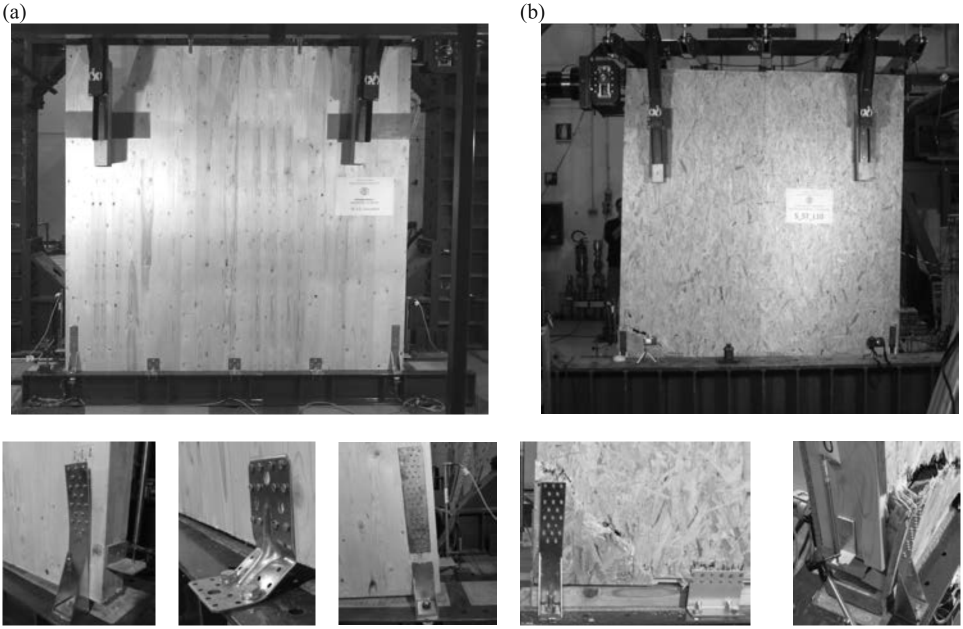


Figure 23. Experimental tests carried out at the Materials and Structural Testing Laboratory of the University of Trento: on CLT walls (a) and timber framed walls (b). At the lower side, some failure mechanisms observed.

The part of work that focused on the numerical analyses has been crucial to clarify the role of FEM models in the assessment of the global response of timber buildings. The implementation of appropriate models for this purpose is a process that focuses on the knowledge of the real behaviour observed experimentally on elements or structural components. In case it is needed to implement non-linear analyses, it is necessary to consider and adequately describe the behaviour of connections with force-displacement links, calibrated on the experimental results. The accuracy level of the FEM model must be

compatible with the quality of data available to describe the behaviour of the connections and the building elements. The work carried out by the Research Units has also proposed equivalent macro-element models (Loss *et al*. 2013b), sufficiently reliable for a simplified evaluation of the seismic resistance of buildings.
This work, which points out some of the most important aspects regarding the seismic design, namely calculation criteria, dimensioning rules, analyses, constructive specifications and rules for the implementation, may be used in the future for the preparation of a handbook for the design and construction of timber buildings.

6 VISIONS AND DEVELOPMENTS

The current "Technical Standards for Constructions" (NTC 2008), together with "Eurocode 8", are regulation documents updated to the modern philosophy of "performance-based" seismic design. A "performance" planning should permit full freedom and operation to the designer in identifying the most suitable tools to size the structure. The focus of the legislator is then directed to the levels of security to adopt and to the calculation principles to follow. Specifically, the designer is given greater freedom in the search for optimal structural solutions, then verifying the achievement of the safety levels established by the legislator in respect of the protection of human life and structural collapse.
The work deriving from this task considered the seismic dimensioning of timber buildings, especially in relation to the possibility of creating earthquake-resistant buildings with a dissipative structural behaviour. In this context, some limits of applicability of the rules contained in the aforementioned documents have been identified, especially considering the current construction technologies and assembly techniques. If we refer to the dissipative structural behaviour of the structure, the NTC 2008 and, similarly, the Eurocode 8 provide guidelines on the materials to be used, the connections, the ductility and the structure factor (q), expressed as the maximum value and defined as a function of the construction typology. The choice of the most appropriate structure factor (q) is a very delicate and not easy operation and, unfortunately, the current NTC 2008 only establish the upper limit value for each structural typology. The use of a given factor q supposes *a priori* a well-defined dissipative structural behaviour, which is expressed in the ability of the structure to provide defined levels of ductility and energy dissipation capacity, as well as resistance to the acting loads. As shown in the carried out work, these features can be guaranteed only by a suitable definition of the connections. The use of a determined value for the parameter q means properly dimensioning the connections, especially in relation to the application of the capacity design. Therefore, it is acceptable to reduce the level of design seismic action, provided that the construction system is able to accept certain levels of inelastic deformation - the latter mainly governed by the capacity offered by the connections.
The inelastic behaviour of the two structural types of reference for the task - CLT panels and framed panels - proved to be significantly different, especially if we consider the collapse failure mechanism. In Figure 24 it is possible to distinguish the two wall mechanisms: (i) a shear deformation mode (sliding) that involves the connectors placed between the covering panel and the frame for framed panel systems and (ii) a rigid rotation deformation mode, where the rotations are triggered by the deformation of the basic connections (failure of hold-down or similar devices) for CLT panel systems.
In accordance with the current version of the NTC 2008, there will be two distinct maximum values of the factor q_0 equal to 5 and 2, respectively for framed and CLT panels. However, the choice of the value to be used in the design phase is a function of the ductility class of the

system ('A' or 'B' according to NTC 2008, and 'H' or 'M' according to EC8). In the case of framed panel systems, the shear mechanism is generally ductile according to the failure mode triggered at the level of the single connector (typically the nail). The failure mode for the nail can be controlled by adopting a sufficient slenderness (ratio between the thickness of the connected elements and the diameter of the connector). The standard defines some limits on the maximum diameter (d) to be considered for the connectors, on the density of some types of panels and on the minimum slenderness (λ_d) to ensure (e.g. d=3.1 mm, ρ_k=650 kg/m^3 for particle panels defined in EN 312, λ_d = 4 for CD 'A' class). The deformation is generated by the sliding of the connectors placed at the edges of the frame. Normally, given the static redundancy of the *shear wall* element, it is easy to reach levels of ductility of about 2÷3 and beyond. In particular, this is possible by appropriately controlling the slenderness of the individual nail/connector and the strength of the steel used, as mentioned in Eurocode 8 (CEN, 2004).

However, to ensure the formation of this mechanism, it is necessary to sufficiently dimension the basic connections (e.g. shear angle brackets and hold-down) so as to ensure the development of the wall element capacity. Failures in the basic connections (Figure 25) are to be avoided before the development of the maximum 'diaphragm' capacity of the walls.

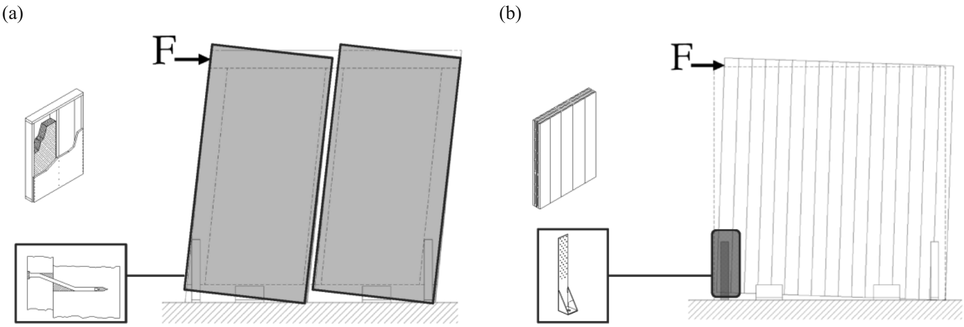


Figure 24. (a) Mechanism of inelastic deformation of a light framed wall with shear deformation; (b) mechanism of inelastic deformation of a CLT wall following the collapse in the ground anchoring devices (Loss et al. 2013a).

Some of the tests performed within this research have highlighted premature failures in ground anchoring devices, resulting in a reduction of the inelastic-dissipative effective capacity and the subsequent reduction of the maximum acceptable value for the structure factor q. In fact, the standard, as it is written, seems to focus exclusively on the local behaviour of the connectors, defining rules on geometry and materials, and forgetting the role played by the ground connecting devices which are normally used in the assembly of structural components (hold-down, metal brackets, perforated metal strips, bars and other mechanical devices). In the absence of specific assessments, it is worth limiting the value of the factor q to a maximum value equal to 3, fully respecting the rules expected for the structures with a high ductility class. If high q factors are used (in the range of 4÷5), then it would be worth justifying this assumption in the planning stage through experimental tests or scientific documentation of proven reliability.

(a) (b)

Figure 25 (a) Brittle failure of the hold-down at the net section level; (b) tear of nails of the base hold-down to reach the maximum capacity.

In the case of CLT panel buildings, a category not clearly identifiable in the current version of the standard (NTC 2008), the deformation mechanism is triggered at the level of the basic connections and, if properly dimensioned, of the wall-wall connections among the panels. In this case, the inelastic deformation capacity is localized in a few points and is mainly guided by the behaviour of the individual connection device. The structure factor to be used in the analyses is therefore a function of the number, distribution and capacity of the installed basic connections. These features can significantly vary from one building to another. In case the structure factor is assumed to be equal to 2 or more, it is worth demonstrating the design choices made, in particular the expected ductile behaviour of the devices. Even in this case, phenomena that may trigger brittle failures in the basic connections are to be avoided (Figure 26).

(a) (b)

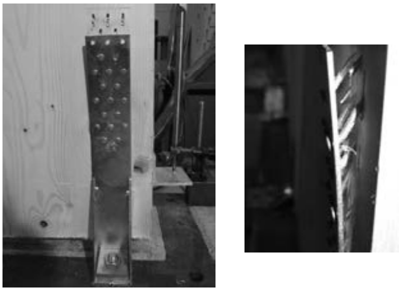
Figure 26. Brittle failure of the hold-down, respectively at the net section (a) and at the stiffening ribs (b).

When it is difficult to determine the actual ductile behaviour of the CLT panel system, or in the absence of available data on the inelastic behaviour of the connection devices used, for the structure factor it is recommended to adopt the minimum value among those proposed, that is to say, 1.5. The use of the structure factor *q* as defined in the NTC 2008 during the design phase supposes a control on the properties of the materials and the connection means employed. The rules defined in the NTC 2008 or the similar provisions of Eurocode 8 derive from the experience gained even with the tests on components/building elements or full scale

structures, and are provided to ensure an appropriate response to low-cycle fatigue effects of both the system and its connections.
In the updating phase of the regulation, it will be necessary to discuss the role that the local behaviour of the connections takes for the characterization of the global seismic behaviour of the building system.
A part of the research activity carried out has focused on the application of *capacity design* (CD) methods. The *capacity design* is a procedure that must be followed in the dimensioning of the elements of a structural system, understood here as a link between individual timber elements and connections, both at a local and global level. The *capacity design* is a planning method that allows controlling the resistant mechanism of the structure and the individual connections. It is implemented by dimensioning the ductile elements with the stress characteristics resulting from the analyses, and oversizing the brittle elements through the application of appropriate overstrength factors (γ_{Rd}).
At an operational level, it is necessary to act first of all (i) identifying the elements and ductile and brittle mechanisms, both locally and globally, then (ii) properly dimensioning the dissipative and ductile behaviour area and finally (iii) implementing measures to prevent the triggering of brittle failure mechanisms. More specifically, this means verifying that the capacities in the ultimate conditions of the elements/brittle mechanisms at a local and global level are higher than the capacity of the elements/ductile mechanisms inside the building system. In order to ensure this condition, the effective capacity of the elements/ductile mechanisms is increased by an appropriate coefficient γ_{Rd}, called "overstrength factor". Starting from this higher capacity, the capacity of the elements/brittle mechanisms is dimensioned.
In case of timber buildings, whether they are constructed through CLT or framed panels, the application of the *capacity design* must act primarily at the level of the various connections used. According to the different wall and failure mechanisms in the connections, the inelastic behaviour assumed in the design phase must be guaranteed in any way. The resistance hierarchy must be applied to each one of the different levels of structural detail, starting from a single connection. For example, referring to the calculation of the bearing capacity of the hold-down connection, considering the possible failure modes of the device, nail side or plate side failures can be triggered. A ductile behaviour can be obtained when the plastic deformation occurs in the connectors (usually nails) used to join the device to the CLT panel (Figure 27a), otherwise the brittle failure caused by tearing of the steel blade (Figure 27b) is triggered. The rules of the *capacity design* avoid brittle mechanisms by simply oversizing the metallic element compared to the maximum capacity that can be transferred by the connectors. By applying this procedure, it is possible to control the failure mechanism of the device, making sure that it shows a ductile failure.
The same rules can be applied to "force" the failure of some connections and preserve others.
In the distribution of the plane forces on the individual vertical bracing elements (*shear-wall*), the floor plays a key role. Depending on the stiffness and strength of the horizontal elements, there can be two limit behaviours: flexible diaphragm or stiff diaphragm. They refer to the distribution of the horizontal forces according to the area of influence or to the stiffness of vertical bracing elements, respectively. According to the NTC 2008, the building system must appropriately respond to the possible torsional effects that may accompany the seismic action. To this end, the horizontal elements of the buildings must have a stiffness and strength so as to allow the exchange of forces among the different resistant vertical systems. In general, in the dimensioning of the horizontal wood elements, the adequate capacity of redistribution of actions on the bracing vertical elements must be considered. Regarding this, in future regulation revisions, it would be desirable to include a part related to the dimensioning of the

connections among the individual modular elements of the floor in order to ensure an adequate transmission capacity of the actions and to control the global deformation of the floor in its plane.

(a)



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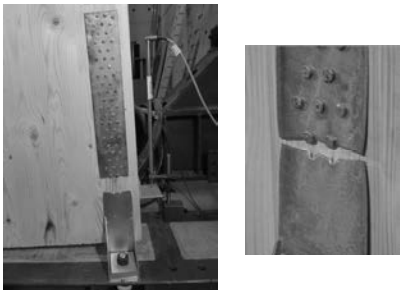

Figure 27. Tensile failure of the ground anchorage devices (hold-down): ductile mechanism with deformation in the nails (a) and brittle mechanism with failure in the net section (b).

In conclusion, it is worth considering the evolution still going on for timber construction systems that is leading to the construction of buildings with a considerable vertical development. Especially for medium-high residential buildings, it becomes essential to also consider the problem of structural 'robustness'. To that end, an adequate capacity to redistribute any stresses along alternative load paths compared to the project paths must be given to the structures, in case a structural element loses its resistant capacity due to unexpected events. The structures must therefore possess an adequate degree of redundancy (but ensuring ductility to the connections) and/or systems able to receive the actions in the event of failure of the structural elements must be defined so as to prevent the generalized collapse of the structure. In any case, the structure must possess such capacity in order to avoid excessive damages (i.e. collapse) compared to exceptional actions (e.g. fire, earthquake, explosion, shocks or consequences of human errors, etc.).
All the problems here mentioned will represent interesting starting points for new research developments, which will be useful in the updating activities of the technical rules, both national (NTC) and European (Eurocodes).

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SEISMIC SAFETY ASSESSMENT OF SPECIAL SYSTEMS

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GENERAL

The Task 2.2 – Seismic safety assessment of special systems is part of the ReLUIS-DPC three-year (2010-2013) funded project. Work was carried out by several Research Units (RUs) disseminated nationwide and coordinated by the University of Naples, Federico II, Department of Structures for Engineering and Architecture. The task was articulated in the following sub-tasks:

Sub-Task 2.2.1: Existing concrete dams;
Sub-Task 2.2.2: Health care facilities;
Sub-Task 2.2.3.1: Pipelines;
Sub-Task 2.2.3.2: Industrial plants;
Sub-Task 2.2.4: Critical infrastructures;
Sub-Task 2.2.5: Nonstructural components;
Sub-Task 2.2.6.1: Statues;
Sub-Task 2.2.6.2: Monuments and museums.

The above eight sub-tasks had active interactions with the RUs dealing with masonry, reinforced concrete (RC), steel structures and the geotechnical engineering.

Sound and versatile approaches for the seismic safety assessment of RC dams, hospitals, industrial plants, lifelines and monumental structure are by virtue interdisciplinary and require continuous interactions and expertise of different disciplines which are under the umbrella of earthquake engineering.

A brief description of the background and motivation, research structure, main results and the discussion of the outcomes obtained in each of the above sub-tasks is included hereafter. The coordinators of each sub-task have also emphasized the needs for further development.

The references relevant for each sub-task are provided along with the publications that were produced during the ReLUIS-DPC three-year funded project.

SUB-TASK 2.2.1: EXISTING CONCRETE DAMS

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1 INTRODUCTION

In Italy there are about 500 dams, most of them were built more than 60 years ago. Few of these have experienced high intensity earthquakes. None was designed using modern design criteria. When considered, the horizontal seismic forces were introduced as the 7-10% of dam's weight. Generally speaking there is a lack of information on seismic response of existing dams against strong earthquakes. The response of dam to past earthquakes are summarized in the 2001 bulletin of the International Commission On Large Dams (ICOLD) [ICOLD bul.120, 2001]. Some additional experiences were collected during the 2008 "Wenchuan" Chinese earthquake and the 2011 "Tohoku" Japanese earthquake. The dam's stakeholders are now convinced that dams must be designed for high level of seismic horizontal forces with which uncontrolled release of reservoir water have to be guaranteed.
The project wanted to identify the state of art of Italian knowledge on seismic safety of dams. The state of the modern codes and the comparisons between them was showed. During the work it was possible to focus on methods of assessment of seismic safety and their application. Both static and dynamic analysis were used to study the response of the structure. Simplified methods, useful to have a quick estimation of the vulnerability were also proposed. Thanks to the planned activities, an important step towards the evaluation of seismic safety of existing concrete dams was done. At the same time it was possible to evaluate the effectiveness of the new Italian legislation on safety of dams identifying the points to improve.

2 BACKGROUND AND MOTIVATION

The most important studies on the seismic response of dams began in the eighties when Professor A.K. Chopra and his colleagues focused on dynamic linear response of dams. In the nineties nonlinear phenomena like the sliding at the base of the dam were introduced.
Simplified method based on planar analysis or other elementary static scheme, are very useful for preliminary analysis. One of the most famous was developed by Westergaard for the evaluation of hydrodynamic pressure during earthquakes [Westergaard, 1933]. This method is especially useful when dams can be considered rigid. The deformability of the dam was then introduced together with the interaction effects with reservoir and rock foundation [Fenves e Chopra, 1986]. Simplified nonlinear analysis also exist for the evaluation of residual base displacement [Chopra and Zhang, 1991; Danay and Adeghe, 1993; Nuti and Basili, 2009] and of concrete cracks [Leclerc et al., 2003]. Recently, methods that take into account the joint effects on response of concrete gravity dams were also introduced [Furgani et al., 2011].
Accurate analysis are performed when no simplified methods exist or when more reliability is needed, these can be divided into linear and nonlinear analyses. The first are quite common for dams [ICOLD bul.52, 1987]. Fluid structure interaction (Akkas, 1979; Wilson, 1983; Aslam, 2002) and soil structure interaction (Wolf, 1990) are today problems well known and

solved using special purpose FEM program. Today the challenge lies in nonlinear analysis for existing dams.
In these years Italian dams are being reassessed with modern methods and updated seismicity. In this context, it is necessary to give some information to engineers on the correct and optimal use of numerical methods.

3 RESEARCH STRUCTURE

The study on dams seismic assessment was treated by the Research Unit of the University of "Roma Tre" led by Prof. Nuti and largely taken care of by the engineers L. Furgani, G. Fiorentino and S. Imperatore. The main activities (Task 3.1) were divided into four sub-tasks.
Starting from the new Italian code and referring to other guidelines (specially from USACE and ICOLD associations) the principles of modern legislations on seismic safety of concrete dams were described.
The second and more consistent sub-task was dedicated to methods of analysis. After a deep study of the main methods of analysis today available, some new simplified methods were also proposed. Simplified and accurate analyses made with FEM method, were then applied to some case studies to describe their effectiveness.
The input parameters needed for the dynamic nonlinear analyses as well as the mechanical features of concrete dams, were defined by means of researches on technical literature. The seismic parameters are also treated using different approaches.
The last activity was devoted to summarize the experiences of the research and the results obtained for the case studies analysed.

4 MAIN RESULTS

4.1 New Italian national code and seismic actions

With reference to the seismic input to apply to the dams, particular attention was directed to the comparison between the design code for ordinary structures (NTC), the actual Italian Technical Code on Dams (TCD) [Cons.Sup.LL.PP, 2008] and the ICOLD Bulletin 148 [ICOLD bul.148, 2010]. The comparison showed little differences concerning the Limit State definitions and the return period used.
Concerning the Collapse Limit State (SLC), associated to the uncontrolled release of water, the TCD refers to a return period of 2500 years for new dams. Considered the level of risk associated to dams, in high seismicity sites a deatiled seismic hazard assessment (SHA) must be performed. For areas with low seismicity the response spectra parameters available in NTC can be used. There are two main methods to perform a seismic hazard assessment, the probabilistic approach (PSHA) and the deterministic approach (DSHA). The ICOLD Bulletin prescribes a 10 000 years return period for the earthquake computed with the probabilistic approach. Comparisons between response spectra showed that increasing the return period always brings to response spectra with higher accelerations. Because of this for high return periods unrealistic values of local intensity parameters can be obtained. A possible solution is to introduce upper bounds to truncate the GMPEs.
Different procedures to select the seismic parameters for SLC earthquakes was applied in a paper that is going to be published in the proceedings of the Second European Conference on Earthquake Engineering and Seismology hosted by Turkey [Fiorentino et al., 2014]. In this study the 4 Italian dam sites listed in "Table 1" were considered. "Figure 1" shows a

comparison between response spectra obtained with different approaches. The PSHA spectrum (computed truncating the GMPE to 3σ) is slightly smaller than the NTC spectrum. It can be observed that deterministic response spectra can be higher than the probabilistic response spectra, depending on which upper bounds are chosen.

Table 1. Dam sites considered for seismic action evaluation.

Site	Location (Region)	PGA for Tr=475 yrs	PGA for Tr=1950 yrs
A	Piemonte	0,15g	0,25g
B	Toscana	0,22g	0,35g
C	Abruzzo	0,26g	0,42g
D	Calabria	0,27g	0,46g

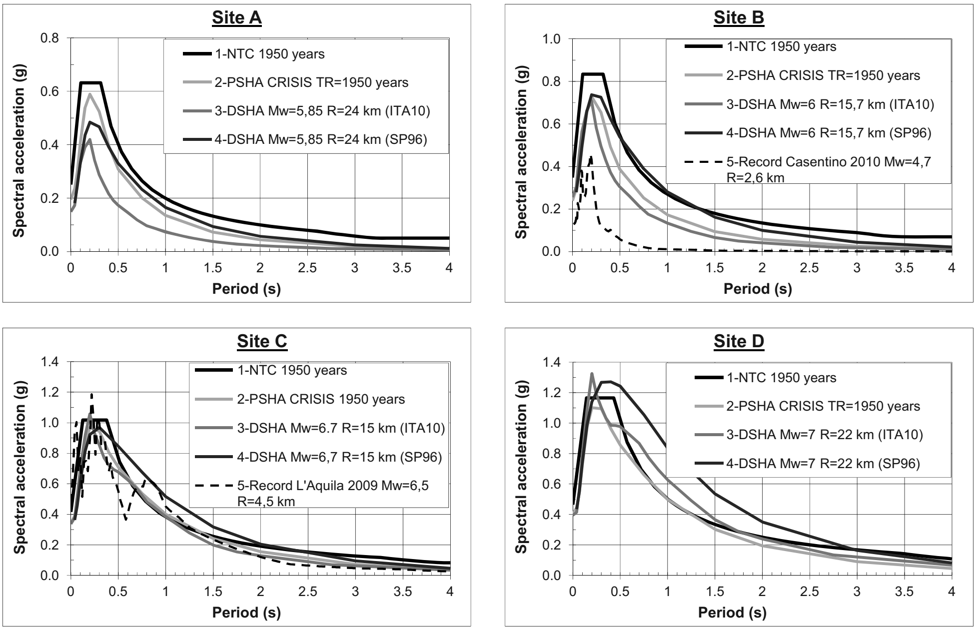

Figure 1. Seismic probabilistic and deterministic spectra for 4 Italian dam sites.

Another important aspect regards the selection of natural and generation of synthetic accelerograms used to perform the dynamic analysis. In all these cases, the spectrum compatibility has been guaranteed and the actual correspondence between the spectral response and maximum response over time is verified.

4.2 Simplified methods available for dams preliminary assessment

With reference to the literature review, the state of the art highlights that analytical studies on the seismic behavior of concrete gravity dams often start from the dynamic analysis of a single section of the dam. The issues related to the three-dimensional effects are often neglected.

Concrete gravity dams may have a very long planar extension, with blocks separated by vertical joints. The dams are characterized by monolithic sections, founded on rocks with angles of friction (concrete-rock contact) typically lower than the concrete ones. In these cases, sliding of the base may occur. In previous studies [Nuti and Basili, 2009] the correlation between μ, the ratio between the maximum elastic top dam displacement and top displacement when the base start sliding, and the ratio β between sliding response acceleration and acting spectral accelerations has been obtained. The law, very effective when β is in the range 0.5÷1.0, allows the assessment of the residual displacement. Some parametric studies were carried out in order to evaluate the parameters that influence the seismic response of a concrete gravity dam. Different methods are also used for 46 natural strong motions signals confirming that simplified methods are reliable for β values greater than 0.5 (see "Figure 2"). The Nuti and Basili method appears the more conservative approach to be used.

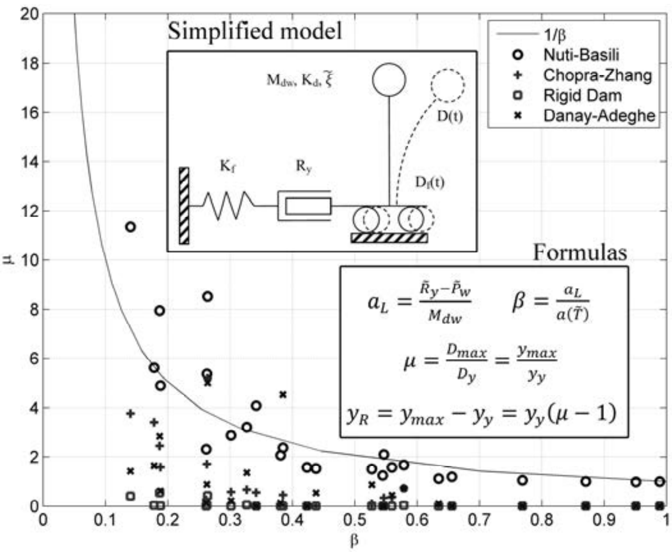

Figure 2. Comparisons between different methods to evaluate the sliding response of a 87 m high gravity dam for a rock-concrete friction angle of 45° and 46 natural earthquake signals in terms of β –μ point.

Additional studies were made to evaluate the influences of seismic parameters selection. The dynamic nonlinear responses of the simplified model represented in "Figure 2" are estimated for different set of accelerograms representing the seismic scenarios described in "Figure 1" for four Italian sites. The results obtained confirm the great influence of the approaches used for: spectra evaluation, signal selection and adaptation [Fiorentino et al., 2014]. The simplified method, can be conveniently used also to choose the sets of signals best representative of the dams site used in more advanced analyses.

Three-dimensional effects can be preliminarily considered by means of the evaluation of the interaction between the blocks of the dam [Furgani et al., 2011]. Adjacent blocks may be connected or not. In the latter case the computation of the three-dimensional effects depends on the different deformation of adjacent blocks (for blocks of different height). These effects were considered statically adding to each block the mutual forces arising from relative displacement. The proposed method was also compared with FEM analysis, with good correlations. As demonstrated by the results and from the past studies on this field three-dimensional effects can reduce the stresses up to 30% in the highest block increasing the stresses of adjacent shorter ones. These effect are particularly important as the valley are narrow and the height of the blocks variable. Detailed 3D FEM model are necessary for more vulnerable structures to reproduce nonlinear forces developed by vertical joints.

Table 2. Effects produced by three-dimensional effects measured on the highest block (see "Figure 3") of the dam through simplified and advanced FEM program.

Description	Ur [mm]	σm [MPa]
Simplified method no 3D effects	40.90	+3.23
Simplified Pseudo 3D Method	37.30	+2.77
FEM Linear Analysis	37.80	+2.88
FEM 3D Linear Analysis	35.10	+2.30

4.3 FEM method applied to dams for advanced studies

After applying the simplified methods, very useful for preliminary analysis as well as for screening problems purpose, detailed finite element analyses are conducted. The finite element analysis, carried out in 2D and 3D, focuses on the most important phenomena that may affect the seismic behavior of the dam body. All the research activities are listed below:

1. Pre-seismic stage and its tensional state;
2. Dynamic interaction with the reservoir;
3. Dynamic interaction with the rock foundation;
4. Nonlinear analysis of sliding at the base of the blocks;
5. Dam crack propagation analysis;
6. Three-dimensional nonlinear dynamic interaction between vertical blocks of dams.

This work is the subject of the article presented at the "World Conference of Earthquake Engineering" in Lisbon [Furgani et al., 2012].

4.3.1 Pre-seismic state.

The first part of the seismic assessment is the evaluation of the stress state generated by the history of the actions and co-actions preceding the earthquake as well as by the change in time of the mechanical properties of the concrete. In this regard, the static forces that characterize the normal operation must be applied, then: the weight, the hydrostatic, uplift pressures of the water and the pressure of reservoir deposits if presents. In addition to these it is necessary to evaluate the effects produced by thermal coactions, by shrinkage and viscosity. Despite the presence of construction joints positioned between the vertical segments of dams, designed also to avoid thermal effects, by monitoring data it can be deduced that during the summer the dam blocks expand arising mutual compression stresses. Using the assumptions and procedures which are best suited, it was concluded that, especially for dams in which three-dimensional phenomena can occur, temperature play an important role. In these cases it is important to determine a pre-seismic relative to the summer season and one related to the winter season. These analyses should be done considering nonlinear behavior of contraction joints in order to reproduce the different responses associated to interacting and disconnected vertical blocks.

4.3.2 Fluid- Structure Interaction

In dam's engineering, the dynamic interaction of the dam with the reservoir was one of the first phenomena to be studied. The simplified methods as those proposed by Westergaard in 1933 and Chopra in 1986 cannot consider three-dimensional problems and non-linear behavior. In these cases the most general way to solve the problem is to perform FEM

analyses. The use of acoustic finite elements are actually the most used to treat the fluid medium. In these case special considerations are needed to account no reflection at the infinite boundary and partial absorption due to deposits (Chopra, 1985).
From the results of dynamic FEM analyses and simplified analysis (based on the concept of added mass or static equivalent hydrodynamic pressure), it was observed that it is fundamental to take into account the effects of the interaction but, for ordinary dams (high less than 100 m), the results obtained with different approaches have the same order of magnitude.

4.3.3 Foundation- Structure Interaction

There are two main ways to solve soil interaction problem: the direct methods and those associated with the concept of substructure. In the first case, used for applications, a significant part of the foundation is modeled. This needs a strategy to avoid reflection of mechanical waves on the contours placed at the boundary of the foundation. Special infinite elements able to eliminate the problem of reflection of the waves as theorized by Lysmer and Kuhlemeyer (1969) were tested. Another added step of this method concerns the deconvolution of the seismic signal. Considering the uncertainty in the soil parameters, mass-less foundation models are used, this avoids the characterization of soil damping, one of the parameters that needs more investigation efforts.
From the results obtained, structure-foundation interaction evaluation is fundamental to consider the lengthening of the period of the system. Modeling the foundation in FEM allows also to better describe the tensions at the base of the dam avoiding unrealistic stress concentrations due to numerical constraints.

4.3.4 Sliding ad the base

One of the most important aspects for the safety of dams is the sliding stability, measured through the sliding safety factors. This parameters can indicate whether or not the dam slides but cannot tell us anything about the entity of residual displacement. As showed before, in literature are available different simplified methods which allow to evaluate the sliding at the base. In this paragraph the sliding has been assessed using nonlinear FEM analyses taking into account the effects of the interaction with the reservoir and the foundation.
The results of these analyses carried out in 2D space have shown that simplified methods should be too much conservative for high intensities earthquakes. Considering these and the definition of β parameter previously discussed, it is suggested to preliminary evaluate the β parameter with simplified methods. If the spectral acceleration associated to the highest block of the dam is greater than the half of the limit acceleration (associated to sliding resistance), equivalent to $\beta<0.5$, more advanced FEM analyses are necessary. Introducing sliding at the base of the dam takes into account the friction dissipation that, also for little value of slip, reduce the stresses in the dam body. This suggest the great importance of introducing these damping effects in the most critical assessments of existing dams.

4.3.5 Crack analyses

In Italy most of the existing dams is found to have been designed using seismic forces lower than those currently required for verification. In light of this, for areas of high seismic activity, it is difficult to exclude the development of cracks in the dam body. The goal of the analysis will be to determine the extent of the size of the slots. One of the most well-known approach is that associated to "damaged plasticity model" developed by Lee and Fenves [Lee and Fenves, 1998]. Thanks to this it is possible to consider both the reduction of the stiffness and the plasticization of concrete.

The results obtained using this method for 2D problems, confirm its effectiveness in relating damages and the limit state definitions. Additional analyses were conducted considering also the non-linearity at the base.

4.3.6 Dynamic interactions between the vertical blocks of the dam

The analyses of gravity dams, consisting of segments separated by vertical construction joints, are often considered as a planar problem. This simplification decays, however, in cases where three-dimensional effects may be relevant. A typical example is that of the gravity dams in narrow valleys. A simplified method is introduced in the previous paragraphs. If finite element analyses are used instead, the problem can be solved through contact elements or interface properties between the segments. This approach is always needed to evaluate the real response of arch dam against earthquakes. In the specific case "surface to surface interaction" procedures are used to reproduce the axial and tangential behavior at the interface. In the analyses it is assumed that the joints do not resist to traction and that can open. Conversely, if in contact, they can transfer shear forces according to the Coulomb frictional criterion.
The results of the analysis confirms that the mutual forces are related to the effects of thermal coercion. With this analysis it is possible to assess what is the response of the dam in winter and summer conditions. The different behaviors have significant implications for the crack patterns.

4.4 Examples and comparisons

This paragraph collects some results from the first two years of study and the seismic assessment of some case studies. Part of these are published in the proceedings of the “9th symposium of the ICOLD European Club” held in Venice on April 2013 [Furgani et al., 2013]. The analyses are performed using a "step by step" approach increasing the level of detail progressively. With simplified methods and with commercial finite element softwares it was possible to comply with the provisions of TCD. Generally speaking the improvements of analyses take to lower stresses and more objective evaluation of the real response of the structure. The seismic assessments for two case studies are briefly described in the next paragraphs. A typical concrete gravity dam and an arch-gravity dam are taken as examples (see “Figure 3”).

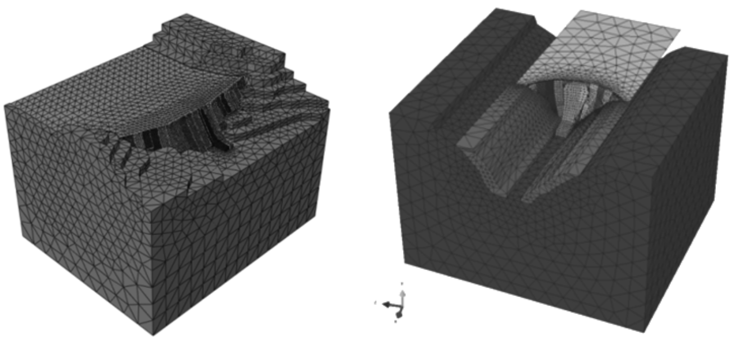

Figure 3. Dam case studies: 87 m high concrete gravity dam (left) and 100 m high arch-gravity dam (right).

The gravity dam, 87,00 m high, composed by 19 vertical blocks 20,00 m width, is assessed for both the Site A and D previously defined. Materials parameters are taken from literature. From preliminary analyses on the planar section of the dam (considered as a cantilever) using static equivalent seismic forces, some vulnerabilities are detected, in particular the tensile stresses on up-stream wall exceed the resistance. For this structure three dimensional effects must be considered. To study all these aspects in detail, more advanced dynamic finite element analyses using spectrum compatible accelerograms are carried out. At the beginning, only the contact nonlinearities (joint residual deformations) are modeled to evaluate the interaction between adjacent blocks. Because some parts of the dam was still over stressed, complete nonlinear dynamic analyses are performed to evaluate the damages produced. The damaged areas where the tensile stresses overpass the strength of the concrete are reported in Figure 4. These are related to the most seismic Italian site (Site D).
Another case study was an arch-gravity dam composed by 11 vertical cantilevers with a maximum width of 21,00 m, separated by grouted joints. The maximum height of the dam is 100,00 m. Since it was not possible to use simplified models, the dam was modeled with 3D finite elements. Also for this case, analyses with increasing complexity level were made. The first model was monolithic and with a linear behavior. With this model spectral response analyses were performed. Regarding stresses, in some regions of the upstream face the maximum stresses exceed tensile strength. For this reason it was necessary to perform time history analyses. These analysis confirmed the emergence of tractions, and allowed to evaluate the time in which the tensile strength was exceeded. On the basis of the response evaluation criteria proposed by Ghanaat [USACE, 2007] it became necessary to use a non-linear analysis. This allowed to evaluate the entity of the damage of the dam.
In the next figure are compared the response of these two different concrete dams to the same Site D earthquake actions.

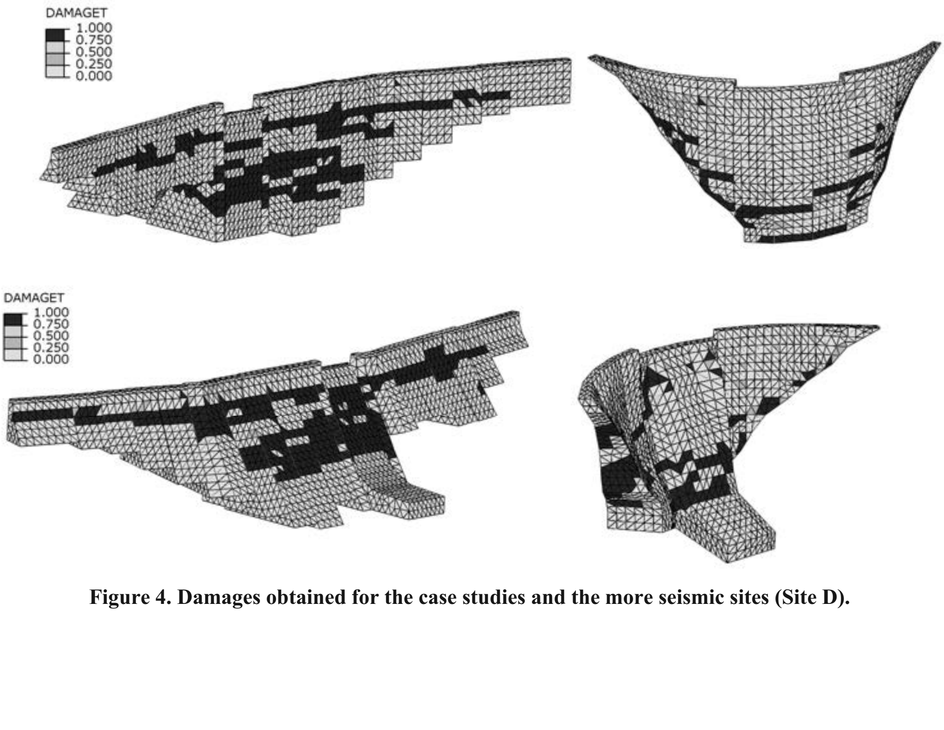

Figure 4. Damages obtained for the case studies and the more seismic sites (Site D).

From the results obtained it is possible to conclude that both dams can suffer damages for high intensity earthquakes. From past experience and numerical results, arch and arch-gravity dams, seem to be less vulnerable.

5 DISCUSSION

The activities carried out by the working group are aligned with the program for the part concerning the application of the simplified and accurate analyses. The experimental evaluation of the concrete properties related to the seismic assessment analysis is substituted with a study based on technical literature.

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SUB-TASK 2.2.2: HEALTH CARE FACILITIES

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1 INTRODUCTION

Moderate-to-large earthquake ground motions may generate severe and widespread damage to health care facilities and their components thus affecting detrimentally the functionality of the system as a whole (e.g Lang et al., 2009; Di Sarno et al., 2011; Masi et al., 2013, Jacques et al., 2014, among many others). Such detrimental response is exacerbated for hospital buildings either designed for gravity loads only or without adequate seismic details. Recently, provisions implemented world-wide in the design standards and guidelines (e.g. DM 2008; ASCE 7, 2010) are targeted at ensuring adequate seismic performance for the hospital structural and non-structural components; nevertheless, they do not provide definite yet reliable rules to design and protect the building contents, e.g. medical components. The latter components may impair the medical services, especially in surgery and emergency rooms and, in turn, they cause high economic and untolerable social losses. It is estimated that 97% of earthquake related injuries occur within the first 30 minutes following the main shock, thus it is essential that hospitals remain operational and continue providing fundamental health services following the disasters. The first 72 hours are a crucial time range for a satisfactory post-event response. Consequently, resilient health care facilities should prevent the disruption of their functionality during post-disaster emergency (Bruneau and Reinhorn, 2007).

The seismic risk evaluation of health care facilities requires sound and complete methodologies based on interdisciplinary approaches encompassing civil and mechanical engineers as well as social science experts, health planners and decision makers. Portfolios of structures can be assessed, at regional scale, with empirical and rapid screening methods based, for example, on appropriate checklists. Such checklists should account for structural, non-structural components, services and functional requirements of the health care system as a whole. Additionally, accurate global response analysis of hospitals requires the identification of adequate response parameters of medical components that need to be monitored at different limit states. However, the existing knowledge is still scarce and further experimental and theoretical investigations are deemed necessary.

The research carried out for the ReLUIS-DPC 2010-2013 project has dealt with the estimation of a reliable seismic safety index for hospitals, the characterization of adequate seismic performance criteria for health care structures, either new or existing ones. On one hand, a simplified and efficient methodology has been developed from previous studies carried out by the Pan American Health Organization (PAHO) and World Health Organization (WHO). A safety index has been developed by convoluting the three fundamentals components of the seismic risk equation, i.e. hazard, vulnerability, and exposure. The proposed index can be conveniently utilized at regional scale either for prioritizing seismic retrofitting interventions or for evaluating the performance in the aftermath of earthquakes. On the other hand, comprehensive experimental shake-table tests were carried out on typical examination (out patients consultation) room unit equipped with typical architectural finishing, freestanding furniture items, desktop computer and medical equipment. Fragility relationships based on a systemic approach (Pinto et al., 2011) have been derived from the experimental test results. Qualitative and quantitative limit states have been defined and suggested for the assessment of the seismic performance of hospital sub-components, e.g. consultation rooms (Cosenza et al., 2014). Simplified finite element models of the experimentally tested components have also been implemented; they can be conveniently utilized to perform reliable parametric analyses that are of interest for researchers and practitioners.

2 BACKGROUND AND MOTIVATION

The evaluation of the seismic risk for hospitals is a very complex task. It requires the definition of an adequate level of knowledge of the material properties, the system layout, the structural and non-structural details, the mechanical and electrical services and the operational layout of the facility, e.g. emergency preparedness plans after an earthquake. The most vulnerable non-structural components that may impair the functionality of hospitals are (PAHO, 2008): emergency exit systems, fire alarm devices, electrical systems, water-supply and medical gas apparatus and communication systems. Their response cannot be neglected in a performance-based framework. However, assessment procedures based on detailed approaches are prohibitive, especially for applications at regional scales. Advanced nonlinear analyses, either static or dynamic, may prove cumbersome in practice, especially for preliminary, post-earthquake estimations; hence, simplified methods can be conveniently utilized. The hospital safety index (HSI), initially formulated by the Pan American Health Organization (PAHO), can be utilized to perform quick and sound evaluation of the earthquake performance of hospitals, before and after an earthquake ground motions. The early-version of the HSI was revised and improved by considering the outcomes of recent studies and post-earthquake reconnaissance reports on hospitals, e.g. Lang et al., 2009; Elnashai et al., 2010; Di Sarno et al., 2011; Masi et al., 2013).
A number of experimental studies were initiated to assess the seismic performance of a variety of furniture items, medical appliances and service utilities of typical hospital buildings and pharmacies. Full scale tests were carried out on shake table tests conducted on large scale sample structures, e.g. a base-isolated four story RC hospital structure (Sato et al., 2011; Kuo et al., 2011; Furukawa et al., 2013), composite hospital rooms at the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University of Buffalo (UB) in the USA, using the Nonstructural Component Simulator (UB-NCS) (Mosqueda et al., 2009) and on the 5-story building, at the outdoor UCSD-NEES shake table facility in San Diego, California (Chen et al., 2012). Experimental shake table experiments have also been carried out on realistic medicine shelves and contents placed in pharmacies (Kuo et al., 2011) and freestanding medical laboratory components, such as low-temperature refrigerators, heavy incubators, freezers, microscopes and lighter computer equipment located on desks and shelves (Achour, 2007; Konstantinidis and Makris, 2009). The above experimental studies were targeted to derive the fragility curves for earthquake loss estimation and formulation of retrofitting measures (Porter et al., 2007). However, such fragilities did not account for the interaction between the components of the system. Additionally, modal response quantities, i.e. natural periods and equivalent viscous damping, and finite element models that can be used in numerical simulations were not adequately investigated. Threshold limits for the assessment of operational limit states also require further investigations, both analytically and experimentally. Towards this aim, a comprehensive shake table experimental campaign was carried out on a full-scale examination (out patients consultation) room unit equipped with typical architectural finishing, freestanding furniture items, desktop computer and medical equipment. The present study introduces the novel systemic approach to evaluate the seismic fragility of the components included in the consultation room. Such an approach accounts for

the convolution of the fragilities of the single components analyzed experimentally on the shaking table.

3 RESEARCH STRUCTURE

The research task has focused on the evaluation of the seismic performance criteria for critical hospital buildings, either new or existing ones. The activities were carried out by a team of academic working groups coordinated by the University of Naples, Federico II, Department of Structures for Engineering and Architecture. The additional groups include the research units (RUs) from the University of Sannio, Benevento and University of Salento, Lecce. The latter RUs were devoted to the development of reliable, yet simplified and quick checklists for the evaluation of the seismic risk of hospitals. Such checklists were developed from existing format by PAHO (2008) and quantify the HSI, which includes all facets of the seismic risk: hazard, vulnerability and exposure. A number of existing hospitals located in areas with low and high seismic hazard were utilized as case studies. Comparisons between the HSIs are provided and possible improvements identified. The checklist used for determining the HIS comprises three main sections: the first section is related with structural elements, the second with non-structural elements and facilities and the last section takes into account the organizational aspects. The relationships adopted for the evaluation of the HSI is as follows:

$$SAFETY\ INDEX = VULN * \frac{HAZ}{4} * \frac{EXP}{3} \tag{1}$$

In eqn.(1), Hazard (HAZ) is a function of the seismicity of the area where the structures are built and the soil type. Exposition (EXP) is a function of the importance of the sample structure. For hospitals, the exposition is related with the type of hospital departments located in the structures. Vulnerability (VULN) is evaluated considering the vulnerability of structural and non-structural elements as well as the organizational aspects. The VULN can be derived by combining the so-called primary indexes, i.e. the index for structural elements (I_{STR}), the index for non-structural elements (I_{NSTR}) and the index for organizational aspects (I_{ORG}).

The value of primary index is estimated through the answers recorded in the questionnaires, separately for each section of the checklist to compile. For each question, three answers are provided, as a function of the level of risk: low, medium and high. Different values express different levels of risk. Each question assumes a different importance in the calculation of the primary index, this value are called Unit Risk Index.

Experimental tests on shake table and advanced numerical analyses were employed to derive performance criteria for typical medical equipment. Fragility curves based on systemic approaches were also derived for an examination (out patients consultation) room unit equipped with typical architectural finishing, freestanding furniture items, desktop computer and medical equipment. In so doing a series of comprehensive full-scale shaking table tests were carried out in the laboratory of the Department of Structures for Engineering and Architecture of University of Naples Federico II, Italy. The system consists of two 3 m x 3 m square shake tables. Each table is characterized by two degrees of freedom along the two horizontal directions. The maximum payload of each shake table is 200 kN with a frequency ranging between 0 and 50 Hz, acceleration peak equal to 1 g, velocity peak equal 1 m/sec and total displacement equal to 500 mm (±250 mm). A single shake table is utilized for the present experimental campaign. A steel single-story framed system was designed with the purpose of simulating the seismic effects on the medical contents of a typical hospital room. The set-up of the sample frame is shown in Figure 1.

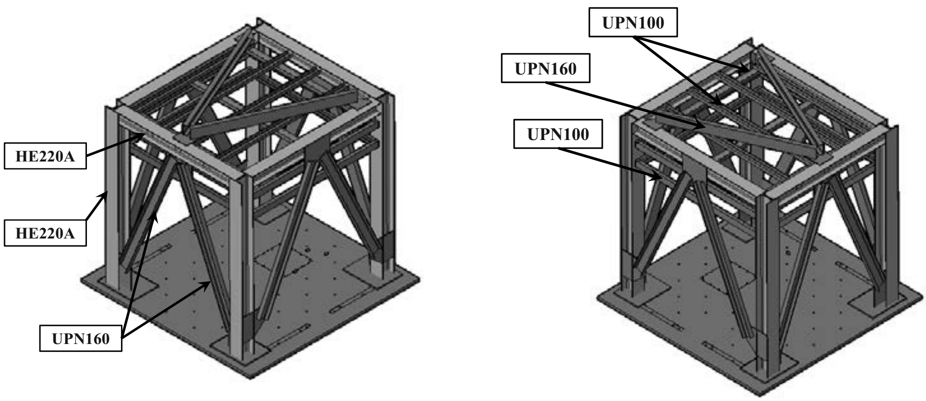

Figure 1. Global perspective of the sample frame used for the shake table tests.

To simulate the effects of the earthquake at different floors on a hospital building, namely the peak floor accelerations (PFAs), the test frame was designed to prevent the onset of the resonance. As a result, the steel frame possesses a large lateral stiffness. The layout of the model consists of a 2.42 m x 2.71 m x 2.72 m test fixture of S275 steel material with concentric V-bracings. The sample frame employs H-shaped columns (HE220A profile) and beams (HE180A profile); the connections are bolted. Two configurations were selected for the shaking table tests. Vulnerable freestanding components and medical appliances were identified on the basis of survey questionnaires and simplified evaluation forms compiled by hospital staff for numerous healthcare facilities world-wide (Achour et al., 2011; McIntosh et al., 2012; Aiello et al., 2012). Examination rooms are departments that are critical to the functioning of healthcare facilities (e.g. Myrtle et al., 2005; OSPD, 2007). Thus, such rooms were selected as representative layouts for the experimental seismic performance assessment of the core units of hospital buildings. Different configurations were analysed and relevant limit states identified for each component and the whole room unit. Acceleration time histories with increasing amplitudes were used to derive seismic fragility curves for the whole medical room, according to a systemic approach. The building contents used for the examination room include: (a) a hospital medicine cabinet made of cold formed sheet and having double moving glass doors with locker and four mobile glass shelves; (b) a hospital medicine cabinet made of cold formed sheet and having single moving glass door with locker and four mobile glass shelves; (c) a desktop computer (monitor, case, keyboard and mouse); (d) a desk made of a steel pipes frame and a wooden desktop and having two drawers with locker. To investigate the seismic behavior of the hospital room and its components, unidirectional horizontal accelerograms were selected to match a target response spectrum, provided by the ICBO-AC156 (2000). Different variables, related to the arrangement of the contents on the different shelves and to the position of the cabinets with respect to the wall behind, are considered. A few variables are investigated in the six test groups of the undertaken test campaign, as summarised in Table 1. Further details on the experimental testing program can be found in Cosenza et al., (2014).

Table 1. Test program definition.

Test group	Plan configuration	Cabinets contents	Cabinet-to-wall distance [cm]
100	1	Equivalent mass uniformly distributed along the height	2
110	1	Equivalent mass uniformly distributed along the height	10
120	1	Equivalent mass uniformly distributed along the height	15
200	1	Equivalent mass non uniformly distributed along the height	2
300	1	Typical glass contents uniformly distributed along the height	2
400	2	Equivalent mass uniformly distributed along the height	2
500	2	Equivalent mass non uniformly distributed along the height	2
600	2	Typical glass contents uniformly distributed along the height	2

4 MAIN RESULTS

The application of the methodology based on the evaluation of the HSI to the case studies of the two sample hospital buildings located in low- and high-seismicity areas has shown that the use of the section of the checklist dealing with non-structural elements and services was not straightforward. The difficulty stems from the needs to survey all facilities in the buildings. The most important deficiencies are related to anchorages of equipment and pipes. A I_{NSTR} value of 0,74 and 0,49 was founded for the hospital in high and low seismic hazard regions, respectively; these values correspond to a high and medium level of risk. For all primary indexes the hospital located in the high seismicity area exhibits higher levels of risk than the counterpart system in the low seismicity area. The highest value of risk is related with I_{STR} (0,78); this follows from the irregularity of the structure, both in elevation than in plan, and in the lack of structural seismic details. The index I_{STR} for the hospital in the low seismicity region assumes a medium value because the structure was designed to sustain not only gravity load but also seismic action according to the Italian seismic classification of that time (medium level of seismicity). The index related to the organization of emergency is the index with minor level of risk. In both structures emergency plans are present even not directly related with seismic emergency. Figure 2 summarizes the outcomes of the application of the HSI to the sample hospital building located in the high seismicity area.

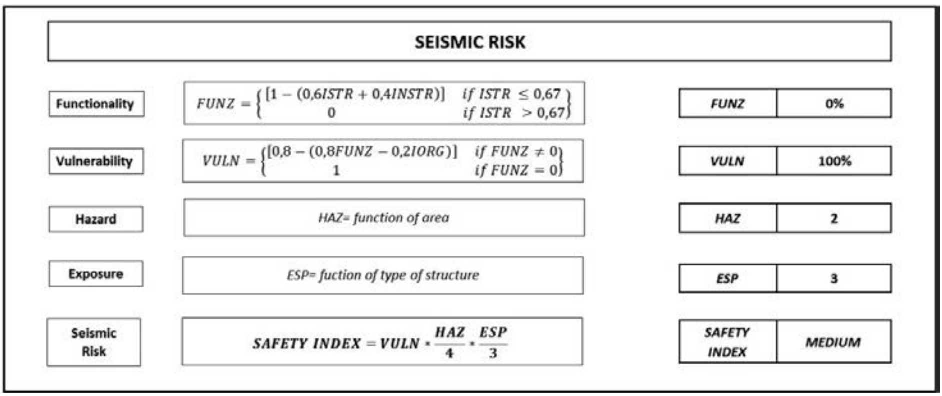

Figure 2. Evaluation of the hospital safety index (HIS) for a building located in high seismic risk area.

Random vibration excitations are performed in order to dynamically identify the different medical components analysed experimentally on the shake-table of the Department of Structures for Engineering and Architecture, University of Naples Federico II. Before each test, different random excitations at different intensity levels are performed. The transfer curve method is adopted to evaluate the natural frequency of the different components. The method is applied for the two cabinets, the desk and the monitor. When evaluating the natural frequency of the monitor, the acceleration time history recorded on the desk is used as input and the acceleration time history recorded on the top of the monitor is used as output in the transfer curve method. Block averaging and Hanning windowing techniques (e.g. Proakis and Monalakis, 2007, among many others) are also adopted. The length of each block, i.e. NFFT, defines the resolution of the transfer curve. Moreover, a 50% block overlap is also utilized. The length of each block is adequately selected to define a fairly regular transfer curve. The method is applied for the sample cabinets and the desk. Typical transfer curves are plotted in Figure 3.

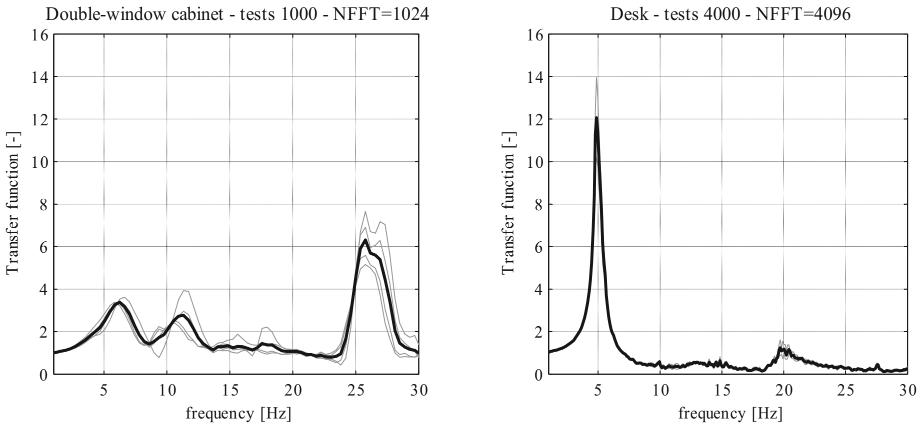

Figure 3. Transfer curves for double-window cabinet (left) and desk (right).

The NFFT is selected equal to 1024 and to 4096 respectively for the transfer curves of the cabinets and the desk. Considering that the sampling frequency of the accelerometers is equal to 400 Hz, the frequency resolution of the transfer curve is 0.391 Hz for the cabinets and 0.098 Hz for the desk. An average transfer curve is evaluated for each test group from the gray curves corresponding to each single test. The peak in the mean transfer curve denotes the natural frequency associated to one of the vibrational modes of the tested components. The transfer curves in Figure 4 emphasize the presence of multiple modes of vibration. In the case of the cabinets, the high frequency peaks seem to be related to modes that involve only a limited portion of the component, e.g. window natural mode. The natural frequencies for the different random tests are summarized in Table 2. It is worth noting that the monitor is identified only during tests 1000. The results related to the first three test groups should be investigated separately from the results related to the last three test groups, due to the different input motion direction on the components.

Table 2. Natural frequency of the tested components for the different random test groups.

Test group ID	double-window cabinet	single-window cabinet	Desk	Monitor
1000	6.25 Hz	7.03 Hz	20.31 Hz	7.03 Hz
2000	5.08 Hz	6.64 Hz	20.31 Hz	-
3000	6.25 Hz	7.03 Hz	20.70 Hz	-
4000	4.69 Hz	7.03 Hz	5.08 Hz	-
5000	5.08 Hz	8.20 Hz	5.08 Hz	-
6000	4.30 Hz	7.81 Hz	5.08 Hz	-

The damping ratios associated to the first mode of the two cabinets and the desk are experimentally evaluated based on the dynamic identification tests. The half-power bandwidth method is applied to the computed transfer curves. The damping ratio of the desk, which varies between 5.1% and 6.0%, is significantly smaller than the damping ratio of the two tested cabinets, which ranges between 14.8% and 28.2. The direction of the seismic input on the components does not significantly influence the damping ratio values.

The tested components are also modelled through finite element models using the Sap 2000 computer program (CSI, 2012). The results of the finite element models were compared to the experimental data measured during the full-scale shaking table tests. The natural frequencies of the numerical model of the desk match the experimental results. The numerical model implemented is capable to simulate the differences of natural frequency as provided by the experimental tests, in case the component is shaken along the two horizontal directions. The natural modes of the two tested cabinets are adequately simulated by the numerical models.

Finally, the fragility curve is evaluated according to Porter et al. (2007). According to this procedure, the fragility parameters are computed as:

$$x_m = \exp\left(\frac{1}{M} \cdot \sum_{i=1}^{M} \ln r_i\right) \tag{2}$$

$$\beta_{\text{mod}} = \sqrt{\frac{1}{M-1} \sum_{i=1}^{M} \left(\ln\left(r_i / x_m\right)\right)^2 + \beta_u^2} = \sqrt{\beta_{fit}^2 + \beta_u^2} \tag{3}$$

where M is the number of the tested specimen, r_i is the PFA at which a given damage state occurs in the i-th specimen and β_u, equal to 0.25, takes into account that the specimens are subjected to the same loading history and the number of the tested specimen is less than 5 (Porter et al., 2006).

A damage scheme is defined in order to correlate the visual damage to the achievement of a given damage state. Thus, fragility relationships can be derived. Three damage states are defined, i.e. Damage State 1 (DS1), Damage State 2 (DS2) and Damage State 3 (DS3). The damage state definitions are strictly related to the loss that a given damage state would cause, as indicated in Table 4.

Table 4. Damage scheme for the correlation of the visual damage to the damage state.

		Damage state 1	**Damage state 2**	**Damage state 3**
Component	**Damage typology**	*Operational interruption*	*Need to replace damaged part of the components*	*Need to replace the whole component and/or threat for life safety*
Cabinet	**Residual displacement**	Displacement larger than 2cm	-	-
	Collapse	Screw loosening	Collapse of one support	Collapse of more than one support
		Residual displacement in shelves less than L/500	Permanent displacement in shelves larger than L/500	Shelves collapse
		Window opening	Window locking	Window collapse
	Overturning	Rocking	Hammering (with damage)	Overturning
Desk	**Residual displacement**	Displacement larger than 4cm	-	-
	Collapse	Screw loosening	Collapse of one support	Collapse of more than one support
		Drawer opening	Drawer slipping out of rail	Desk collapse or overturning
Content	**-**	Displacement	Collapse (less than 10%)	Collapse (more than 10%)

The fragility curves that fit the experimental data (dotted thick lines in Figure 4) are clearly highlighted with respect to the ones with the larger dispersion (solid thick lines in Figure 4). The latter also take into account the logarithmic standard deviation βu. As expected, βfit is very small, since it includes only the variability due to the different mass configuration.

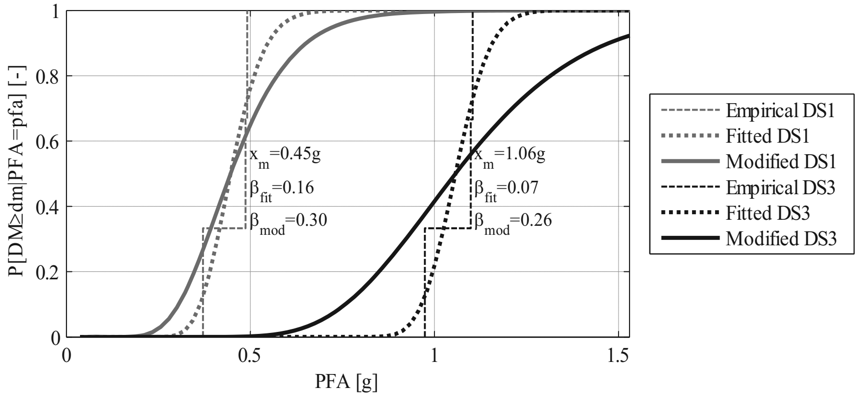

Figure 4. Fragility curves for the damage states 1 and 3 considering mass variability.

5 DISCUSSION

The application of the simplified approach based on the HSI showed a satisfying agreement between the structural index evaluated by the simplified methodology and the index obtained by the push-over analysis. The results of the applications to case studies of hospital structures located in areas with different seismic hazard show that an improvement of the HAZ parameter is, however, deemed necessary. Similarly, the performance of non-structural components and hospital medical components should be further investigated as the response criteria are not reliably defined quantitative. Preliminary results derived from the comprehensive experimental laboratory tests carried out on a full-scale consultation room have shown that the earthquake response of the medical component can be affected by a number of parameters and mechanisms. It is found that the distribution of the mass along the height assumes a key role to evaluate the natural frequency of the cabinets in case they are shaken along their transversal direction.

The transfer curve method emphasizes the occurrence of multiple modes of vibration of the components. The desk is characterized by a significantly different natural frequency along the two horizontal directions. The damping ratio is estimated through the half-power bandwidth methodology. The damping ratio of the desk varies between 5.0% and 7.7%, whereas the damping ratio of the two tested cabinets, which ranges between 13.4% and 26.9%, is significantly high. The natural frequencies of the numerical model of the desk, as implemented in computer platform SAP 2000 (CSI, 2012), match the experimental results. The numerical model implemented is capable to simulate the differences of natural frequency as provided by the experimental tests, in case the component is shaken along the two horizontal directions. The natural modes of the two tested cabinets are adequately simulated by the numerical model. The multiple peaks observed in the transfer curves are associated to a mode shape. Considering the low level of accuracy of the model with respect to the complexity of the components, the outcomes of the numerical analyses provide a close approximation of the experimental ones. The main finding of the present analytical study is therefore that simple models are able to adequately simulate the dynamic properties of the tested cabinets. The rocking mechanism in the two tested configuration of the sample frame initiates for a PFA that ranges between 0.37 g and 0.61 g; instead the overturning of the cabinets occurs for PFA slightly larger than 1.00 g. The accelerations recorded at the top of the different components are strictly related to rocking phenomenon, that induces spikes in the recorded time-histories. It is noted that as the peak floor acceleration exceeds the 0.5g value, the desk slides on the floor reducing the acceleration recorded on the component.

The fragility analysis has emphasized that the cabinet-to-wall distance increase can significantly reduce the seismic performance of the cabinets. This confirms that the higher the wall-to-cabinets/bookcase distance, the higher the probability of the overturn of the component.

6 VISIONS AND DEVELOPMENTS

The research is being further developed in a ReLUIS-DPC funded project by applying the checklist formulated during the previous project to a large population of hospital buildings located nationwide, in areas with different seismic hazard. The results obtained with the HSI can be compared with analytical studies carried out with appropriate numerical simulations. Such comparisons can be employed to evaluate the reliability of the HSI-approach and improve the basic equations of the formulation, where appropriate. Furthermore, a digital user-friendly platform can be implemented to facilitate the collection on site of the relevant data for the critical structures. Such platform can also be powered to visualize the regional distribution of the seismic risk for hospitals. It is envisaged to include, in the checklist, specific sections dealing with intervention remedies that may effectively increase the value of HSI. Thus, the latter index can be utilized to prioritize and select the intervention remedies.

Bidirectional shake table tests are required to assess the seismic performance of the examination (out patients consultation) room unit equipped with typical architectural finishing, freestanding furniture items, desktop computer and medical equipment. The response should be evaluated in the as-built and retrofitted configuration. Retrofitting measures may include either simple connections between the freestanding components and the room walls or the use of isolation devices with or without supplemental damping.

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SUB-TASK 2.2.3.1: PIPELINES

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1 INTRODUCTION

The activities of the UR University of Molise (UniMol) concerned the seismic vulnerability of selected process, transmission and storage equipment present in industrial plants. Critical aspects and the limit states of these structures were analyzed based on an observational approach and an extended review of available post-earthquake reconnaissance reports. Structural, geotechnical and industrial aspects of the problem were analyzed, so that the assessment of interaction mechanisms involving the structure, the surrounding soil and the fluid inside were taken into consideration. In particular, RU UniMol developed basic tools for the seismic vulnerability characterization of industrial components to be used in the framework of Na-Tech risk assessment. Therefore, observational fragility curves and cut-off threshold values were defined. For sake of brevity, in the following some issues related to the seismic performance of pipelines are reported, even if some work has been also carried out in the field of wastewater treatment plants and reported elsewhere (Panico et al., 2013 for details).

2 BACKGROUND AND MOTIVATION

Lifelines and Industrial Plants are key components of the economic and social system of a modern country. A primary requirement for these systems and networks consists of their structural safety, especially when large amount of toxic and flammable substances are stored or manipulated. Among others, the analysis of risks related to the interaction between natural catastrophic events such as earthquakes and civil and industrial installation is becoming a basilar topic in the design of these structures (NaTech risks) (Salzano et al., 2009; Krausman et al., 2011).

Each lifeline is composed by a system of structures and elements: in order to evaluate the seismic vulnerability of the whole system, it is necessary the evaluation of the vulnerability of each component. In this report, the seismic behaviour of pipelines is considered and analyzed. It is worth noting that the seismic response of these structures is, in all cases, quite complex due to dynamic interactions involving three different components (Lanzano et al. 2013a): i) the soil around the structure that offers a lateral confinement; ii) the structure itself, depending on geometric and material features; iii) the fluid inside with its specific properties. However, according to EC8-4 (EN 1998-4,2006), the hydrodynamic effects could be neglected "when H/R (the ratio between filling level H and section radius R) exceeds 1,6" and the pipeline "should be assumed to behave as if it were full, i.e., with the total mass of the fluid acting solidly".

On the contrary, the seismic performance of the (buried) pipeline is always strongly related to the geotechnical effects. Based on experience and data collected during past earthquakes, these dynamic effects can be divided in two categories (O'Rourke and Liu, 1999): the strong ground shaking (SGS), which is the transient deformation of the soil due to wave passage

(Figure 1a), and the ground failure (GF), that is the permanent deformation of the surrounding soil due to co-seismic failure phenomena. The ground failure mechanisms appear only in specific geotechnical conditions; hence they are site dependent and may be summarized in: active fault movement (Figure 1b), liquefaction (Figure 1c) and earthquake-induced landslides (Figure 1d).

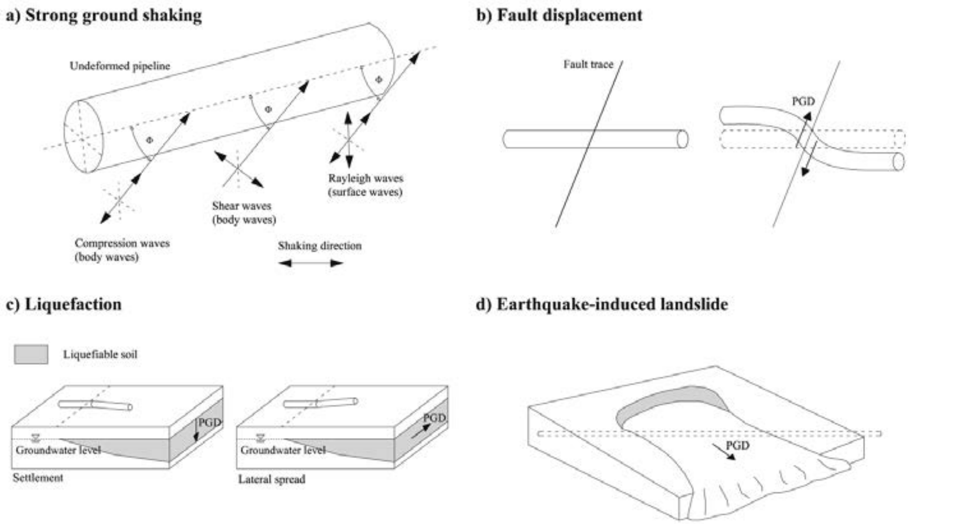

Figure 1. Summary of strong ground shaking and ground failure interaction mechanisms (Lanzano et al. 2013b).

3 RESEARCH STRUCTURE

The seismic damages to pipelines are generally described through curves reporting a performance indicator as a function of a seismic intensity measure. These curves are mainly derived as best fitting of the observational data coming from post-earthquake reconnaissance reports. In general, the performance indicator is the repair rate, RR, which gives the numbers of repairs after an earthquake for a unit length of pipeline.

An alternative description of pipeline seismic performance is considered whenever the attention is paid to the consequence of failures differently from extension of repair interventions and related costs. This is the case of quantitative risk analysis of industrial facilities, which requires an estimation of released hazardous content. In such context, an option is the construction of pipeline fragility on the analogy with the seismic damage estimation of above ground industrial structures developed by Salzano et al. (2003; 2009) and Panico et al. (2013). The method (Lanzano et al. 2013b, 2013c) requires the creation of a thoughtful database of pipeline failure for different class of construction and the definition of Damage State (DS) and Risk State (RS) indicators. The Damage State refers to type of structural damage to pipeline (Table 1), whereas the Risk States (RS) were qualified on the basis of the possible negative effects on the external environment or population, i.e. on the possible harmful effects derived from the release of content from the damaged pipe (Table 2). Quite clearly, the indicators have been distinguished on the basis of the transported fluid. In order to correlate the structural damage to the release of contained fluid, an equivalent diameter Φ of a crack in the pipelines has been defined.

The RS levels for pipelines transporting gas, vapour and liquefied gas were organized in order to match the corresponding damage states. The RS0 corresponds to DS0, in which the damage type, even if severe, did not cause any loss of containment. The RS1 was formulated considered a very limited amount of loss, however distinguishing between toxic and flammable substances. Finally, the RS2 has the highest level of risk and accounts for the release of large amount of fluid in a very short time interval. Similar risk states RS were formulated for liquid transporting pipelines. Differently from the previous classes, the RS0 level allows very limited loss of liquid. The RS1 accounts limited release, but time-distributed, whereas the RS2 involves the cases in which the pipe section has completely failed.

Table 1. Damage states DS for pipelines.

States	Hazard	Patterns (structural damage)
DS0	Slight	Investigated sections with negligible damage; pipe buckling
DS1	Significant	Longitudinal and circumferential cracks; compression joint break.
DS2	Severe	Tension cracks for continuous pipelines; joint loosening in the segmented pipelines.

Table 2. Risk States RS for pipelines (Φ = equivalent diameter).

States	Hazard	Patterns (loss of containment)	
		Gas/Vapour/Liquefied Gas	**Liquid**
RS0	Null	No losses	Limited loss
RS1	Low	Very limited losses: - Toxic ($\Phi < 1$ mm/m) - Flammable ($\Phi < 10$ mm/m)	Limited, time-distributed loss of hazardous substance: multiple losses ($\Phi < 10$ mm/m)
RS2	High	Non- negligible losses	Large loss (e.g. entire tube surface) or multiple losses ($\Phi > 10$ mm/m)

Based on the collection of literature data and the given definition of damage states DS and risk states RS, a uni-modal distribution of the damage states versus the intensity measure has been observed. The data have been fitted by a cumulative log-normal distribution:

$$P(DS \geq DS_i \text{ or } RS \geq RS_i) = \frac{1}{2}\left[1 + \operatorname{erf}\left(\frac{\ln IM - \ln \mu}{\beta\sqrt{2}}\right)\right] \tag{1}$$

where μ and β are respectively the median and the shape parameter of the best-fitting distribution.

Following the procedure given by Salzano et al. (2003; 2009), in order to obtain univocal threshold values both for PGV and PGA with reference to RS states, the seismic vulnerability of the pipelines has been evaluated by using the classical probit analysis. The probit variable Y is expressed in the Eq. (2), as a dose-response model: Y is the measure of a certain damage possibility as a function of a variable "dose" IM (Intensity Measure).

$$Y = k_1 + k_2 \ln IM \tag{2}$$

Details on this classical statistical method can be found elsewhere (Finney, 1971). The value of Y, providing a value of the dose equal to 2.71, corresponds to zero probability and then could be considered as a threshold value.

4 MAIN RESULTS

The collected data were referred to 22 different earthquakes from 1906 to 2012. The damage cases were about 400. All the data were divided into 5 classes, which are considered significant for fragility curves construction, accounting for technological, geotechnical (see section 1) and structural aspects: a) above-ground pipelines (AP, 16%); b) buried continuous pipelines under strong ground shaking (CP-SGS, 33%); c) buried continuous pipelines under ground failure (CP-GF, 21%); d) buried segmented pipelines under strong ground shaking (SP-SGS, 7%); e) buried segmented pipelines under ground failure (SP-GF, 23%).

From a structural point of view, pipelines were divided in two categories in terms of damage patterns and, indirectly, of transported fluid: continuous pipelines (CP) and segmented pipelines (SP). The main features, in terms of materials, joints and damage patterns, are showed in Table 3. A similar approach has been already adopted in the context of HAZUS (FEMA, 1999), where the pipelines are divided in brittle (SP) and ductile (CP), on the basis of the seismic performance in terms of pre-failure deformations.

Table 3. Structural aspects in the seismic behavior of pipelines.

Pipelines	Use	Materials	Joints	Damage pattern
Continuous (CP)	Natural Gas Oil Petroleum Water	Steel Polyethylene (HDPE) Polyvinylchloride (PVC) Ductile Iron	Welded joints Mechanical or flange joints Special seismic joints	Tension cracks Compression cracks Local buckling Beam buckling
Segmented (SP)	Water Wastewater	Asbestos cement Reinforced Concrete Polyvinylchloride (PVC) Vitrified clay Cast iron	Caulked joints Bell end Spigot joints.	Axial Pull-out Crushing of bell end Crushing of Spigot Joints Circumferential Failure Flexural Failure.

The importance of geotechnical aspects (Figure 1) reflected the choice of seismic reference parameters for fragility construction: according to Newmark (1967) approach, the strong ground shaking caused transient deformation and could be related to peak ground velocity (PGV); the permanent displacement induced by ground failure phenomena could also, directly or indirectly, be linked to PGA (Lanzano et al. 2013b).

Results of the RS fragility for the four classes relative to buried pipelines (b, c, d, e) are reported in Figure 2. Tables 4 and Table 5 reports the corresponding calculated values of the median μ and shape parameters β. The calculated cut-off value of the intensity measure parameters, IM0, has also been indicated.

Table 4. Fragility coefficients and thresholds for pipelines under SGS. IM = PGV expressed in cm/s.

Structural aspects	Class	Fragility		Threshold, IM_0
	Risk state, RS	μ (cm/s)	β	(cm/s)
CP	≥ RS1	45.22	0.39	17.05
CP	= RS2	71.16	0.20	26.58
SP	≥ RS1	21.80	0.26	5.50

Table 5. Fragility coefficients and thresholds for pipelines under GF. IM = PGA expressed in g.

Structural aspects	Class	Fragility		Threshold, IM_0
	Risk state, RS	μ (g)	β	(g)
CP	≥ RS1	0.58	0.17	-
CP	= RS2	0.56	0.18	0.20
SP	≥ RS1	0.37	0.23	-
SP	= RS2	0.37	0.19	0.14

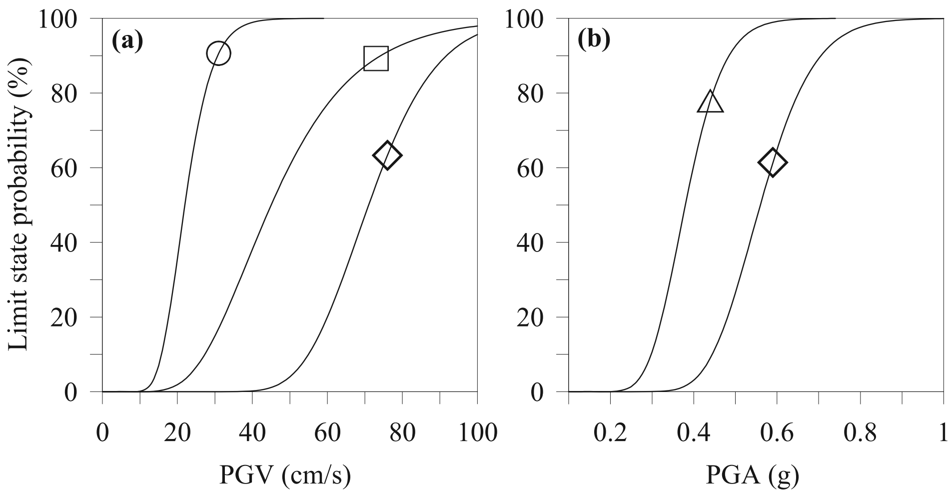

Figure 2. Fragility curves for buried pipelines, under Strong Ground Shaking (a) and Ground Failure (b), in terms of limit state probability (%) for the RS state.

In the case of segmented pipelines under strong ground shaking the sample data were small and the correlation coefficients are lower compared to those retrieved from other groupings; in particular, for the cases RS=RS2, a reasonable cumulative distribution was not obtained. Furthermore, the fragility curves obtained for the ground failure are practically coincident between the RS≥RS1 and RS=RS2 datasets: it means that the RS1 data are not significant for fragility estimation. This consideration, however, is in agreement to the HAZUS indication on ground failure damage, which is almost totally related to high damage (breaks). Also, coherently to seismic performance of segmented pipelines, the value of median μ is lower than the corresponding values of continuous pipelines. Finally, the value of β is higher for SGS classes, compared to GF. The threshold value for SP under SGS is quite low (5.5 cm/s) compared to corresponding value of CP (17 cm/s). In this case, the difference between the two risk threshold levels is about 10 cm/s. The threshold for ground failure cases increases from 0.14 g, for the SP, to 0.2 g for continuous ductile pipelines.

As regards the dataset of continuous pipelines under SGS, which includes the largest amount of data, other additional classes of fragility curves were accounted: small diameters (D < 400 mm) and large diameters (D ≥ 400 mm). The results are given in Table 6 and Figure 3 (Lanzano et al. 2013c).

Table 6. Fragility parameters and threshold values for CP under SGS accounting for diameter, D.

Diameter D	Risk state RS	Fragility μ (cm/s)	β	Threshold, IM_0 (cm/s)
<400 mm	≥ RS1	37.21	0.29	15.45
<400 mm	= RS2	63.25	0.12	25.72
≥400 mm	≥ RS1	50.14	0.23	20.95
≥400 mm	= RS2	49.43	0.41	-

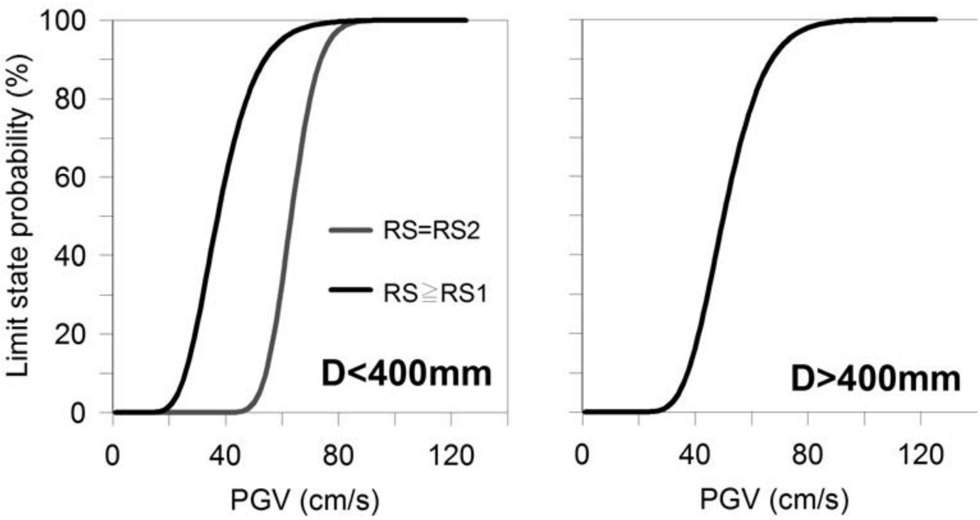

Figure 3. Fragility curves for CP under SGS for different diameter ranges.

The fragility for transmission pipelines (D≥400mm) and RS≥RS2 were less reliable compared to the other classes, because of the limited amount of data. Then, the threshold value was not obtained and it is recommended to refer to the fragility curve derived for all diameters (Table 4).

The difference in the threshold PGV values for RS ≥ RS1 and RS2 is about 10 cm/s both for all the dataset and D < 400mm classes. The threshold values for all diameters are coherently averaged between small and large diameter; the corresponding PGV increase is about 5 cm/s.

5 DISCUSSION

The study of seismic behavior of buried pipelines was carried out through a multi-disciplinary analysis of a collected database of damages occurred during recent documented earthquakes. The available damage cases were analyzed according to geotechnical and structural relevant topics, in order to evaluate the soil/structure interaction and estimate the pipeline response under the seismic loadings. The final goal of the research is the construction of specific fragility formulations for this class of industrial components, accounting the multidisciplinary nature of this research. Fragility and probit curves for pipelines are given: a novel performance indicator was proposed, compared to the existing fragility formulations for pipelines which are based on a global repair rate and not on each single damage mechanism analysis.
Finally significant threshold values for seismic intensity parameters are given in the framework of Quantitative Risk Analyses. This value could be considered as a strength intrinsic parameter, significant for the pipeline structural performance and for the soil/structure interaction during the seismic events; the physical meaning of IM0 is defined as the limit value of the seismic parameter, to above which a certain level of damage should be considered in the Risk Analyses of the pipeline network.

6 VISIONS AND DEVELOPMENTS

The research in the field of seismic protection of industrial facilities is a very stimulating challenge, since it requires the involvement of concurrent skills. The integration of different technical aspects represents the key issue in the accomplishment of the objectives and in the definition of effective and reliable tools for safety management of industrial sites. Future developments of this research could be addressed as follows: a) the construction of other new fragilities, based on empirical data, in order to describe the response of industrial structures under specific co-seismic phenomena, as the liquefaction or the fault displacement; b) the construction of fragilities for different types of pipelines, as the aboveground pipelines, or specific curves depending on the construction material and diameter; c) the validation and extension of the tools through numerical non-linear analyses; d) the application of these tools to real case studies for the validation and calibration of integrated structural and industrial risk assessment procedures.

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SUB-TASK 2.2.3.2: INDUSTRIAL PLANTS

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1 INTRODUCTION

The current worldwide situation of industry concerning natural events, e.g. earthquakes, is particularly critical. This is clearly demonstrated by the consequences of serious accidents caused by natural events to industrial plants (Na-Tech events), particularly in the chemical and oil processing industries. Consequences include the release of the hazardous materials (fires, explosions), human injuries and the increase of overall damage to nearby areas, proving this to be a key emerging risk issue (Cozzani et al 2010, Krausmann et al. 2011). In fact, chemical accidents triggered by natural events like earthquakes have been recognized to be about 5% of all accidents involving the release of hazardous substances (Campedel 2008). Earthquakes can cause severe damages to industrial plants, initiating major accidents, as clearly shown in several events (Sezen & Whittaker, 2006, Suzuki, 2006, Hatayama 2008, Krausmann et al., 2010, Nishi, 2012). The main reason is that chemical plants are complex systems, and this complexity, due to numerous connections and components renders them particularly vulnerable to earthquakes. In fact, in the case of a seismic event, the earthquake can induce simultaneous damages to different apparatus, whose effects can be amplified because of the failure of safety systems or the simultaneous generation of multiple accidental chains. In addition, activities carried out in process plants are often arranged in series. Consequently, the "failure" of a single element may result in the "failure" of the entire system.

In a chemical plant, an earthquake can cause many human losses as a consequence of component collapses, similarly to buildings, along with indirect effects such as economic losses, downtime, environmental damages due to releases of dangerous substances, damages due to explosions, fires and the release of toxic substances. Therefore, the usual safety requirements applied to civil buildings for ultimate and serviceability limit states and the consequences of exceptional actions are generally unsuitable for structures belonging to industrial plants. As a matter of fact, critical damages that could cause even a modest release of inflammable substances, such as a flange opening or a weld breaking, of reduced to negligible significance from a structural point of view, might actually cause considerable accidental chains. Consequently, for process industry it is unavoidable to associate indirect consequences of accidents due to seismic events to direct structural damages. Therefore, many authors have suggested methodologies for a quantitative risk analysis (QRA) of the main chemical plant components for the calculation of their fragility curves and risk indexes, useful for the assessment of possible reference scenarios triggered by seismic events (Antonioni et al 2007, Campedel et al 2008). They have shown the general high vulnerability of chemical plant components and the need for suitable protection systems e.g. Early Warning (EW) (Salzano et al. 2009). An EW system activates interlock systems and fast shut-off valves to prevent loss of content and consequent accidents.

A different solution is represented by passive control systems (PCT). They have been developed during the last 40 years and are based on the concept of reducing the seismic action instead of increasing the structural strength (Spencer & Nagarajaiah, 2003). For civil constructions, these techniques are nowadays considered a consolidated alternative design solution for new or existing structures in seismic-prone areas. Unfortunately, PCT is not easily applicable to industrial structures, for at least two reasons: 1) the large variety of structural and geometric configurations of plant components, 2) the different design objectives and working conditions, closely related to the consequences of possible accidents. As a matter of fact, until now, PCT have been used for a very limited number of industrial applications; for example, in Europe the isolation technique has been adopted only in a few cases: the seismic protection of Petrochemical LNG terminal of Revythousa, Greece (Tajirian, 1998) and of ammonia tanks, at Visp, in Switzerland, by means of elastomeric isolators (Marioni, 1998). Friction Pendulum devices were also used for the seismic isolation of an elevated steel storage tank of the petrochemical plant of Priolo Gargallo in Sicily (Italy) (Santangelo et al., 2007). In Korea a couple of LNG tanks have been isolated using high damping rubber bearings (Koh, 1997). Two large liquefied natural gas (LNG) tanks for the Melchorita facility (Perù) have been seismically protected with Triple Pendulum bearings. The facility is located in an area with high seismic hazard. Use of seismic isolation in these LNG tanks resulted in an economical tank design with a reduced footprint, while providing the most reliable mechanism for accommodating the large seismic displacements that occur during an earthquake.
Nevertheless, analytical and experimental tests have clearly demonstrated the effectiveness of isolators in reducing the seismic response of storage tanks (Chalhoub and Kelly 1990, Calugaru and Mahin 2009, De Angelis et al 2010). Other applications of PCT to industrial components have been proposed in the past for the seismic protection of piping systems using yielding or friction-based bearings (Bakre et al., 2004), or semi-active dampers (Kumar et al., 2012).

2 BACKGROUND AND MOTIVATION

From the above framework it is clear that the investigation on the current seismic design approaches of industrial plant components is needed and consequently the analysis of the most suitable PCT appears necessary.
For these reasons the UR Roma3 after a historical survey of the structural behavior and the relevant damages suffered by oil refinery components during strong earthquakes, a structural classification of the analyzed components has provided. An overview on the most suitable protection strategies, based on passive control systems, is also proposed for each of structural types.
Although the identification of the most important damage states was already provided in literature, especially to identify possible loss of content (LOC) phenomena and consequences, the activity developed by the University Roma Tre concerns the same issue but with a particular attention paid to structural aspects only; the goal is to explain the close relationship between structural typology and passive control technologies (base isolation, energy dissipation, TMD, etc..). In addition, some qualitative information on how PCT can help in avoiding possible hazard accidents is provided and discussed, leaving the identification of the relationship between the structural and hazard benefits provided by PCT systems to further works.

For the above reasons, the main objectives of the three-years activity were to analyze the seismic behavior of the most frequently adopted industrial components under seismic actions and to show the applicability of PCT in industrial plants; these results could be profitably used for a further quantitative evaluation of the seismic risk of a plant in presence of response mitigation systems.

3 RESEARCH STRUCTURE

The main objective of the present research activity is to fill the gap concerning the seismic analysis and design of the main chemical industrial components together with the identification of the main countermeasure to decrease their seismic vulnerability, more in particular passive control solutions, typically employed in civil engineering. Consequently the research was devoted to the following objectives: a) identification of the most common structural typologies of process industry, b) evaluation of the criticality of the most common equipment (storage tanks, piping systems and support structures, etc..), analysis of their seismic behaviour during recent seismic events, c) analysis of the most recent codes and standards on this matter (EC8, API650, ASCE-07, etc..) analysis of the important aspects and drawbacks, d) analysis of the most significant limit state conditions of chemical industrial components, especially piping systems and tanks, e) validation of the current modeling and analysis of industrial components in seismic prone-zones, f) analysis of the most suitable seismic passive protection systems, with particular attention paid on base isolation and dissipation techniques. The research is articulated in the following three phases:

Phase 1: <u>State of the art of the seismic behavior of industrial structures:</u>
Phase 2: <u>Seismic analysis of industrial equipment</u>
Phase 3: <u>Use of the most suitable passive control systems for the seismic protection of industrial equipment and issue of design guidelines for the seismic analysis of piping systems:</u>

4 MAIN RESULTS

The main results of the research activity developed by the UR Roma Tre are herein described and commented in the light of the above objectives, identifying the innovation aspects and the weak points. The results will be described following the research structure presented in section 3.

4.1 State of the art on the seismic analysis of industrial plants and typical damages of industrial components

This activity has been devoted to the analysis of the current seismic design approach of industrial plants and typical seismic damage states of the most vulnerable equipment, including tanks, piping systems, furnaces and support structures. As result, main industrial components of an oil refinery were collected in a limited number of structural classes, identifying their main defects. The relevant results are summarized in (Paolacci et al 2012, Paolacci et al. 2013). The analysis confirmed that tanks and piping systems represent highly vulnerable components of an oil refinery in seismic-prone areas; therefore, numerical analyses were specifically dedicated to this kind of structures and treated in the following sections, even though other typologies has been analysed as well.

4.2 Analysis of the seismic response of industrial components

Given that the UR Roma Tre developed in the past numerical and experimental analyses of above-ground steel storage tanks with and w/o isolation systems (De Angelis et al. 2010), only the analysis of elevated isolated tanks has been performed. In addition, the analysis of piping systems and the relevant support structures have been studied in the light of the modern performed-based approach.

4.2.1 Elevated tanks on short columns

Elevated tanks on short column are particularly vulnerable to earthquakes. This is demonstrated by recent seismic events and confirmed by many contributions present in literature. Typical structural configurations are realized with steel storage tanks supported by steel or reinforced concrete columns. For slender support columns this kind of tanks can show a natural capacity to filter the seismic action. On the contrary, in case of short columns the positive filtering effect of the support maybe limited. Moreover, in case of reinforced concrete supports, the high shear stiffness may induce premature shear failure in columns, as shown in recent seismic events. For example during Itzmit earthquake (1999) in Turkey, a series of elevated tanks for the storage of liquefied oxygen were subjected to serious damages or collapsed. On the basis of the above depicted framework, the work of Roma Tre has been devoted to address the problem of elevated tanks with particular attention paid on steel storage tanks placed on short R.C. columns. The dynamic problem of seismically excited tanks has been described and formalized. In particular, the response in fixed base configuration has been widely analysed, discussed and applied to the case study illustrated in Fig. 1. This latter collapsed, (Fig1a), during the Itzmit earthquake (1999) in Turkey, due to the limited shear strength of the R.C. support short columns.

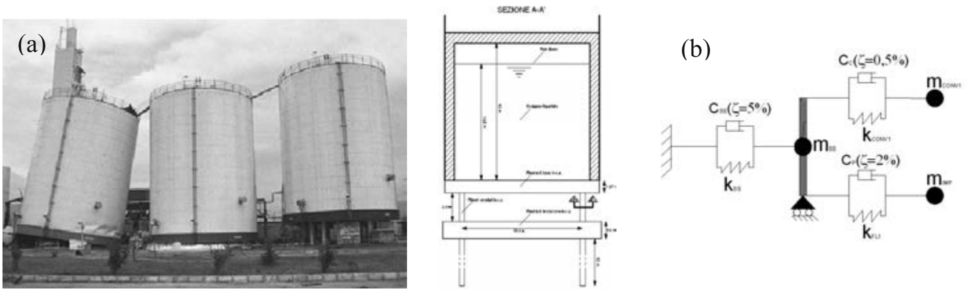

Figure 1. (a) Case study: elevated steel storage tanks on short columns (b) 3DOF model.

For the evaluation of the seismic response both refined and simplified models were used for calculating the base shear components (convective, impulsive, total) and liquid pressure on the tank wall (Fig. 2a, 2b).

For the lumped mass model (see Fig.1b), the fluid-structure interaction was modeled using the Velestos and Yang approach, which considers, as participating mass of the first mode, the entire impulsive mass. To solve the problem, governed by the dependency of the wall displacements from the pressure, an iterative approach was adopted.

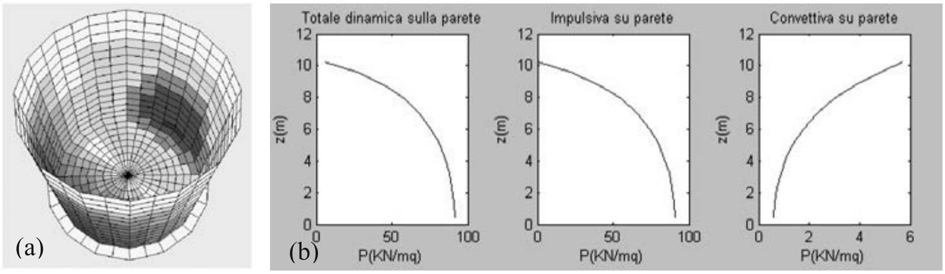

Figure 2. 3DOF model: (a) wall pressures, (b) Total, impulsive and convective pressures along the height.

As expected, the modal analysis of the 3DOF model showed that the convective motion (T_c=3.75 s, MPA=22%) is practically independent of the other two vibration components (impulsive pressure (T=0.56 s, MPA=64%) and the mode associated with the flexible columns (T_c=0.16 s, MPA=14%)), which in turn present a slight interaction.
Figure 3 shows the base shear components due to the wall pressure (impulsive, convective), the inertia component of the tank base and the total base shear. It is evident that: 1) the convective motion is practically independent of the others, 2) the impulsive components is predominant c) after the input end the motion is practically coincident with the convective motion. The value of the response indicates that a serious deficiency in shear strength of the columns can compromise the safety of the structure, as it actually happened during the Itzmit earthquake in 1999. In addition, the strengthening of the R.C. columns does not represent a good retrofit solution because the tank wall could suffer elephant foot buckling phenomena. The previous observations suggested as retrofit strategy the use of base isolation systems.

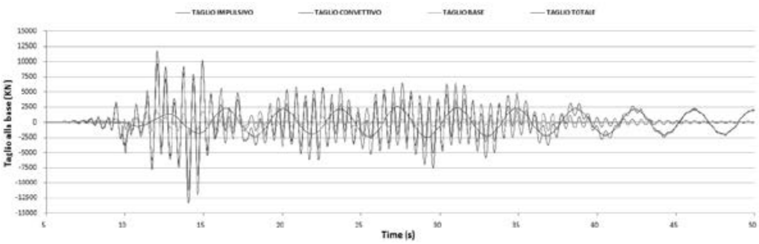

Figure 3. Base shear components of the elevated tank – fixed base configuration.

4.2.2 Industrial piping systems and support structures

In petroleum industries, especially in refineries installations, hundreds of miles of pipes are installed to transfer raw and refined material (fluid and gas) from a point to another of the plant, connecting all the components involved in the transformation process (tanks, distillations columns, furnaces, etc..). Therefore, piping systems represent key structures that deserve particular attention. In addition, a limited number of contributions that clarify their behaviour in seismic-prone areas are present in literature (Paolacci et al 2011, Paolacci et al 2013).
For these reasons, the seismic analysis and component design of refinery piping systems has been analysed and discussed. A review of the current approaches imposed by European (EN13480:3) and American (ASME B31.3) standards was illustrated by using a proper case study regarding a piping system on a pipe-rack (Fig. 4a). The piping system here analysed belongs to a typical petroleum refinery. Pipes with 8” of diameter are supported by a steel

structure that works as moment-resisting frames in the transversal direction and truss structure in the other direction. The model of the entire structure is shown in Fig. 4b where the pipes where modelled using beam elements, whereas the elbow are modelled using shell elements.
For the analysis of the seismic response both response spectrum and time history (T-H) analyses were used. In the first case classical and floor spectrum analysis were used considering the structure as elastic. Generated floor spectra were adopted and compared with the floor spectra suggested by European and American codes, allowing to discover a certain discrepancy in the response, due to the dynamic coupling between pipe and support structure, usually neglected in a floor response spectrum analysis. A parametric study with T-H analysis on typical piping systems and support structure allowed to confirm the rule suggested by ASCE7 code that imposes considering the dynamic coupling only when the pipe to pipe+support weight ratio exceeds25%, as in the analysed case study. The results of dynamic analysis highlighted a general overdesign of such a kind of structures and the possibility to relax it, allowing a certain level of plasticisation in the support structure and pipes.

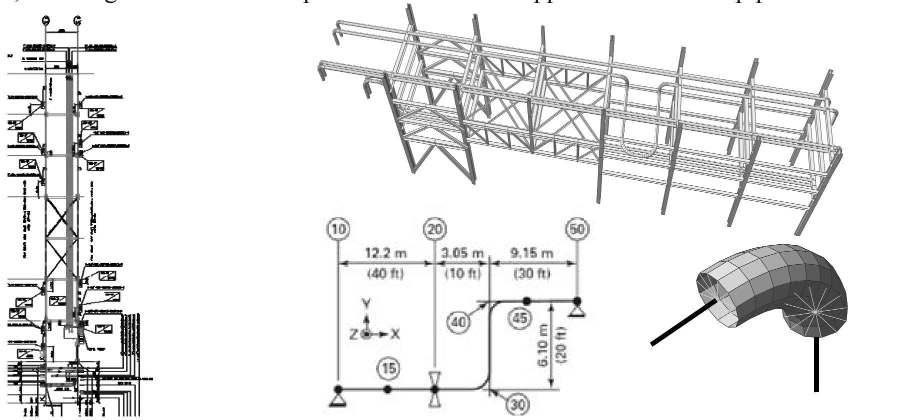

Figure 4. (a) Case study of Piping systems analysed, b) numerical model.

This was confirmed evaluating the behaviour factor of typical piping systems and support structures. In particular, dedicated non-linear analysis on the piping system of Fig.4 allowed to identify a q-factor of 1 to 2 in the support structure against a value of 3 to 4 suggested in both the European and American codes.
For pipes q=1 was instead obtained, even though values of 6 to 12 are suggested in some codes (e.g. ASME B31.3). This high level of conservatism suggests to accept a greater level of deformations in the structure, provided that limit states in the pipes are not overcome (leakage, buckling, etc..). This part is still under investigation and will be concluded during the third year.
The same level of conservatisms was detected applying the piping stress analysis for which allowable stress approach is usually adopted. This seems to be in contrast with the modern performance based-design approach, for which a certain level of yielding in the structure can be admitted, according to a specific performance. The approach, proposed in literature to overcome this problem, consist in a modification of classical allowable stress equations, even though a modern Performance-based approach would be highly recommended. Unfortunately, experimental data for the necessary identification of the main limit states are still limited and advancement in this direction is deemed necessary (Bursi et al 2012, Bursi et al 2014, Reza et al 2014).

4.3 Use of passive control techniques (PCT) for the seismic protection of industrial plants

Innovative seismic control systems belong to the world of the vibration control techniques of structures, which include passive, semi-active, active and hybrid systems (Spencer & Nagarajaiah, 2003). The experience acquired during experimental activities and worldwide applications have indicated passive control techniques as one of the most suitable solutions for the seismic protection of structures. As already experienced in civil engineering applications, they can be subdivided in three different typologies: a) Seismic isolation, b) Energy dissipation, c) Tuned mass damper (TMD).

Table 1. Seismic damages of industrial process components and passive control techniques.

Structural typology	Critical equipment	Typical seismic observed damages	Other possible damages	Passive control techniques
Slim vessels	Columns Reactors Chimney Torch	• Leakage of fluid in flanged joints • Yielding of anchor bars	Overturning	Dissipative coupling
Above-ground squat equipment	Big broad tanks with fixed and floating roof	Failure of wall-bottom plate welding Elephant foot buckling Diamond buckling of tank wall Settlements of ground	Uplifting	Base isolation
		Impact of floating roof to tank wall.	Overtopping Torch fire	Dissipative spacers between roof and wall, TMD
Squat equipment placed on short columns	Spherical tanks	Collapse of structure due to shear failure of columns		Dissipative bracings Base isolation Dissipative coupling
	Process Furnaces	Collapse of structure due to shear failure of columns	Leakage from pipes;	Base isolation
		Collapse of the chimney Detachment of internal pipes	Increase of temperature of Furnace wall	Dissipative bracings
		Detachment of the internal refractory material		TMD
	Cryogenic tanks	Collapse of structure due to shear failure of columns		Base isolation
Piping systems and support structure	Steel or R.C. frames	Collapse for excessive stresses	Damages to supported equipment (pipes, tanks,...)	Dissipative bracings Dissipative coupling Non-conventional TMD

The above techniques cannot be indifferently applied to each type of equipment, because their effectiveness depends on the dynamic characteristics of the structure to be protected. The experience provided by the observation of seismic damages suffered by industrial plant components allows the most vulnerable components and proper passive control techniques to be recognized.

As a matter of fact, Table 1 summarizes typical observed damages and consequences due to earthquakes in major-hazard industrial equipment together with the most suitable passive control techniques. To better understand this proposal, in the following, the effectiveness of the several PCT has been acknowledged and discussed for each category. For the suck of brevity, in what follows the results of a couple of analysed case studies are presented; more details can be found in (Paolacci and Giannini 2012, Paolacci et al. 2013).

4.3.1 Slim vessels

For slim vessels, the most likely damage in the case of an earthquake is the yielding of anchorage bars at the foundation level and the leakage of fluid due to failure of flanged joints caused by excessive displacements. This behavior suggests base isolation and energy dissipation as the most proper PCT solution for reducing seismic response of slim vessels. Actually, energy dissipation devices are to be preferred for high damping capability and the inner capacity of reducing displacements and thus accident consequences (e.g. failure of flange joints or pipe elbows). An interesting example of application of the energy dissipation technique is presented in (Nielsen et al., 1988). An interesting alternative is represented by dissipative coupling between vessels and adjacent structures. In this case the two structures are linked with special dissipation devices, which, because of the different dynamic behavior of the coupled structures, dissipate a high amount of energy. Experimental tests confirmed the high effectiveness of dissipative coupling (Basili et al. 2013). The result of an interesting

application of dissipative coupling and proposed in the present research project has been summarized in (Paolacci et al 2013).
In brief, a reduction of more than 50% of base shear and moment with respect to no connection, and more than 60% regarding rigid connection. Similar results are obtained for the displacements. Some beneficial effects are also present in the seismic response of the column whose shear, moment and displacements are reduced of about 10-20% with respect to no connection case and 50-60% concerning rigid connection.

4.3.2 Steel storage tanks

Many literature contributions recognize base isolation as the most suitable protection system against earthquakes, for above ground squat equipment, especially for storage tanks (Malhotra, 1997; Shrimali & Jangid, 2002). Unfortunately, few practical applications have been proposed and a limited number of experimental activities have been carried out (Calugaru and Mahin 2009; Maekawa, 2012). Given the high effectiveness of base isolation, the potential applicability to tanks could be very high, especially in using Friction pendulum bearings, given that dynamic characteristics of an FPS-isolated tank remain unchanged regardless of the storage level as well as the effectiveness of the isolation system. Unfortunately, isolation systems slightly affect the sloshing motion of liquid. Consequently accidents associated to a floating roof could not be avoided. A possible solution to reduce the effects of the impact between a floating roof and tank wall is represented by damper spacers placed between roof and wall or by inserting a TMD system into the roof (e.g. Tuned Mass Damper Column) (Sakai & Inoue, 2008). As a matter of fact some results from an experimental campaign on a base isolated big broad tank (De Angelis et al., 2010) have been presented and briefly discussed (see Fig. 5 and 6).

Figure 5. Mock-up of the above-ground tank.

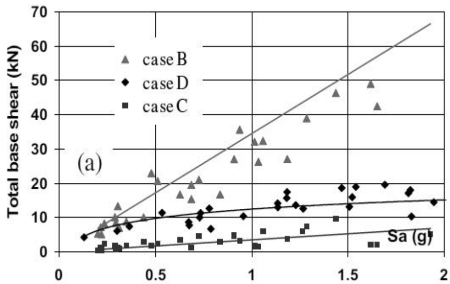

Figure 6. Total base shear versus spectral acceleration of the impulsive motion.

Concerning the elevated tank analysed in section 4.3.1, the base isolation using two isolation systems was analyzed: HDRB and Sliding Surface Bearings (SSB). The comparison between isolated and non-isolated cases showed the high performance of both isolation systems with a remarkable reduction of the impulsive pressure on the tank wall and the base shear. Moreover the high level of displacement requested to the isolators suggests using SSB bearings. An example of seismic response of the tank equipped with SSB devices is shown in Fig. 7. More details can be found in (Paolacci 2014, Paolacci et al. 2014)

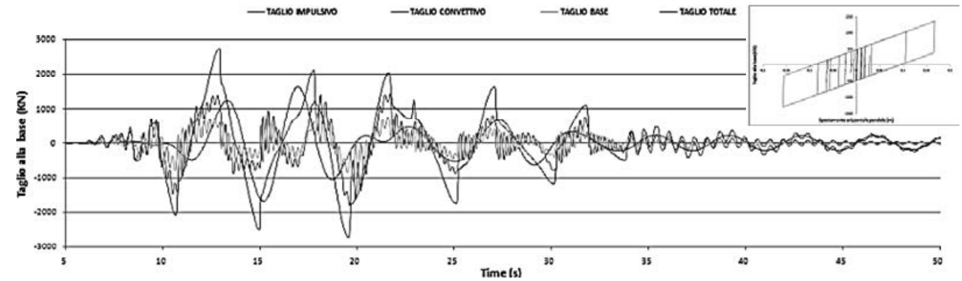

Figure 7. Base shear components of the elevated tank – isolated base configuration.

4.4 Proposal of guidelines for the design of industrial metallic piping systems

Given the general inadequacy of proper seismic analysis and design rules for a piping system and its components, specific guidelines for the design of metallic industrial piping system have been presented in a draft version, which consist of a concise document compatible with the existing Eurocodes design framework (EN1998:4, EN13480), which includes some useful indications on seismic analysis of piping systems. In particular, the basic idea is to use the basic principles of earthquake engineering expressed in (EN1998:1), extending the existing rules to industrial piping components and systems. The proposal, based on a previous work (Paolacci et al 2013) has the following goals:

- Provide in a concise manner a guide to the seismic design of metallic industrial piping systems, typical of process industry; information about nuclear power plant piping systems, excluded here for evident reasons, can be found elsewhere.
- Provide the single steps necessary for the seismic qualification of new metallic piping systems
- Propose a seismic qualification standard to be used as integration of the (EN1998:4).

The guideline is organized in 10 chapters, including preliminary design rules, modeling, seismic analysis methods, definition of input, definition of limit states and possible verification formats. The use of these guidelines is fully explained through the example analysed in section 4.3.1.

5 DISCUSSION

The activity performed during the three years of the research project can be seen as a first attempt to provide a general overview of the currents approaches in designing industrial plant components in seismic-prone areas. The lack of a general framework in which the designer should operate strongly motivated the activity. Consequently an important effort was firstly devoted to identify, under structural point of view, the most vulnerable equipment, typically installed in chemical/petrochemical plants, the most diffused plant. This could considered a useful tool for designers involved in designing/retrofitting industrial plant components, given that the weak structural aspect have been identified and the most suitable seismic protection solutions have been suggested.

To better understand all aspects of the problem, several case studies have been analyzed, whose seismic behavior has been assessed using the current European and Extra-European codes prescriptions. This permitted to identify the major drawbacks of the codes and suggest improvements. In particular, given the general inadequacy of proper seismic analysis and design rules for a piping system and its components, specific guidelines for the design of metallic industrial piping system have been proposed. This can be considered an operative tool for designers involved in seismic analysis of industrial metallic piping systems, typically installed in Chemical/Petrochemical plants.

The high vulnerability, demonstrated by most of the surveys on seismic damages suffered by the main industrial equipment, suggested to investigate suitable seismic protection systems. Literature results demonstrate that PCT are certainly adequate for reducing the seismic response of industrial plant components; this has been confirmed by analysing, during the three years of the project, several case studies (tanks and piping systems).

In conclusion, the activity is in line with the original proposal even though the analysis of a high number of case studies together with an extensive experimental activity of critical components/parts would be certainly useful

6 VISIONS AND DEVELOPMENTS

The present research project, dedicated to the seismic analysis and design of industrial plant components, has been strongly motivated by the inadequacy of the current seismic standards and codes. For this reason it was mainly devoted to the study of the important aspects of the problem, from the definition of the input to the proper modeling and analysis methods based on real case studies. The outcomes could be used to amend the current seismic design codes introducing specific aspects of the problem, often neglected or marginally treated. EC8-part 4 can be certainly considered a corner stone in defining the design aspects of typical industrial components under seismic action. Unfortunately, it has been demonstrated to be incomplete under several points of view. For example, no mention is made on the seismic design of piping systems, whereas seismic analysis of tanks, even though completely described, it appears of difficult application. In addition, no mention is made on traditional and innovative seismic protection techniques that instead appear to be a good solution for improving the seismic performance of industrial plant equipment. Along this line, the study of the several case studies, analyzed within the project, can be considered a good starting point for designers involved in seismic design/retrofitting of industrial plant components. In addition, the design guidelines of metallic industrial piping systems, proposed by the UR Roma Tre, can be considered a good starting point to amend EC8, also about this important aspect.

Given that UR Roma Tre is currently involved in other research projects on the same topic, the results could be synergically used for a radical change of vision on this topic. For a better impact of the results on the scientific and technical community specific experimental tests should also be performed, especially in view of the more modern approached as the performance-based earthquake engineering, that currently is under investigation also for industrial plant structures.

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Sub-Task 2.2.4: Critical Infrastructures

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1 INTRODUCTION

Though a general consensus has not been reached, by the terms lifelines and infrastructure systems it is meant a number of distinct systems, mostly of the network type, that are becoming increasingly interdependent and that collectively provide/convey essential (critical) services and goods to satisfy societal demands. For this reason these systems are often denoted as critical infrastructure(s), or CI (PCCIP, 1997)(Rinaldi et al, 2001).

The modelling and analysis of these systems falls within what is by now accepted as a third mode of science (and engineering), complementing theory and experimentation, i.e. computational science. The latter is based on the idea that facts can be discovered and understanding be gained not just by positing theories and physical experimentation but also through computational simulations. These can sometimes pursue the consequences of hypothesised theories beyond the current reach of mathematical analysis. Thus computational science consists of constructing mathematical models to use computers to solve scientific or engineering problems. In civil structural engineering this is a well-established commonplace, with finite element analysis being used beyond the limits of analytical solutions as an everyday tool by most engineers worldwide. Another typical problem domain for computational science is that of the analysis of complex networks such as infrastructure ones.

Among the many issues related to the topic of critical infrastructures, the research project aimed at advancing knowledge towards the establishment of a near-real-time decision support system for the purpose of civil defence activity and emergency management. In particular, such a system would be one that allows prediction on the state of the CI to be made and be updated quickly based on incoming information from the field (visual inspection, remote sensing, etc). Updating predictions for incoming evidence can be approached in different ways, the most obvious one being using a Bayesian framework. A secondary related topic investigated in the project was that of sensitivity of reliability predictions to system parameters. Thus, in sum, the activity consisted of two tasks:

- Compilation of a state of the art on performance measures for CI;
- Development of a sensitivity analysis method conceived for use in conjunction with a computational simulation-based framework for the analysis of CI performance;
- Development of a triangulation algorithm for the Bayesian network (BN) associated with the mentioned CI analysis framework, algorithm that constitutes the first step of many exact Bayesian inference algorithms.

2 BACKGROUND AND MOTIVATION

The need for tools to analyse and reliably predict the performance of CI has been recognized ever since the end of the ‘90s. At the international level the problem was first posed in the US, where a document, the report to the President’s Commission on Critical Infrastructure

Protection (PCCIP, 1997), can be seen as marking the turning point in the political interest towards the need for protecting the functionality and reliability of these systems that provide essential services to society. Protection cannot but be based on understanding, which is gained through modelling and analysis. The core of protection is the identification of vulnerabilities and correction measures. As it is often the case, knowledge of vulnerabilities can be exploited for the good (protection) or for the bad (attack), as reminded in (Kennedy, 2003).
The problem is one that will still take some time to be mastered, given that complexity of the studied system of systems and the number of different man-made and natural hazards to which it is subjected. CI are indeed a system of systems, in the sense that while each and every lifeline or infrastructure (e.g. water supply network, electric power network, gas network, etc) can be studied in itself, they are all interconnected and dependent (Rinaldi et al, 2001) and increasingly prone to failures that propagate from one to the other in what is referred to as a cascade (Dueñas-Osorio and Vemuru, 2009).
In terms of hazards, it appears that seismic one is the best studied, for reasons that are easily traceable in the more advanced state of understanding and modelling capabilities of this hazard with respect to other natural hazards. In particular, it can be said that modelling of seismic hazard to a spatially distributed system is well understood, and that issues of spatial and cross-correlation between different measures of intensity have been studied by many researchers quite in depth (Goda and Hond, 2008)(Jayaram and Baker, 2009)(Esposito and Iervolino, 2010)(Weatherill et al, 2014).
Current efforts in the US, at almost twenty years from the PCCIP report, are aiming at regulating these systems and producing guidelines together with the network owners/operators under the umbrella of the Technical Council for Lifeline Earthquake Engineering of the American Society of Civil Engineering (ASCE/TCLEE).
More recently, interest at the European level has also increased towards CI, with directives being issued (EU, 2008) and calls being published on the issue of resilience of these systems (this happened after the end of the project described herein). It can be safely said that the best proof of how much the issue of infrastructure systems is the undergoing normative efforts going on worldwide, that testify the attempt to regulate mitigation measures, upgrade interventions and in general activities related to protection and improvement of these systems (Brunner and Suter, 2008). On the other hand all these efforts do not seem to have led to a consolidated codified approach so far. This is mostly due to the complexity of the problem that is due to the following factors:

- Regulatory challenges arise since the actors are multiple and not necessarily used to interact, but the systems are interdependent, and even within the same system different portions are usually physically interconnected but operated by different entities, possibly in different countries (cross-border, trans-national issues)
- Regulation anyhow cannot but follow the acquisition of feasible and sound methods for analysis of the performance of these systems and of the effect of measures
- Such methods and the underlying models are still under development, especially since the infrastructure systems are subjected to multiple, possibly interacting hazards

In sum, the state of development of science lags behind the needs and wishes of the decision-makers, even if the literature in the field is way too large to be reported even succinctly in this paper. Selected relevant works are reported in the next sections 4.1 and 4.2, where appropriate.
The above is the international context to the research reported in this paper. The basis for the research is instead in the work carried out within the SYNER-G research project (SYNER-G, 2012), where a framework and model for the probabilistic performance assessment of a

system of interconnected infrastructure systems has been developed (Franchin and Cavalieri, 2013) (Franchin, 2014).

3 RESEARCH STRUCTURE

The research was carried out entirely within the research unit, with the three tasks identified in Section 1 Introduction. The latter two tasks have a one-to-one correspondence with the expected results, as described in the following sections 4.1 and 4.2.

4 MAIN RESULTS

4.1 Sensitivity evaluation within a Monte Carlo simulation framework for CI

A method was developed and validated, for the evaluation of first-order sensitivities (derivatives) of selected performance metrics employed in the analysis of lifelines to parameters of the probability distribution of uncertain components.

The analysis of network systems can be carried out according to either a basic connectivity or a flow-based approach. The more common former case is entirely based on graph theoretic foundations. The second approach allows refined assessment of network performance, but requires setting-up and solving physical flow equations (the survival of a connection between a source and a sink does not guarantee in any way that the sink or demand node receives a satisfactory service level, especially with low-tolerance systems such as the electric power network).

While for connectivity based analysis refined non-simulation analysis methods exist and in particular the Matrix System Reliability Method (MSRM) is available (Song and Kang, 2009), the only option currently available for probabilistic assessment of lifeline networks with flow-modelling are simulation methods.

If the results of the risk analysis are to be used in an optimization framework (e.g. retrofit resource allocation), it is convenient to have performance-derivatives available in order to employ the more efficient gradient-based methods.

The next sections describe the developed sensitivity enhanced simulation-method, summarize the comparison method (MSRM) and present the results of the validation.

4.1.1 Sensitivity-enhanced simulation

Given a system with uncertain parameters it is possible to evaluate performance sensitivities with respect to two classes of quantities: the system parameters or the parameters of the probability distribution of those system parameters that are modelled as random quantities. To give an example, one may be interested in evaluating the sensitivity of connectivity loss (a system-level performance metric for connectivity based analysis, see first-year report) of a transportation network to the seismic capacity (expressed e.g. in terms of peak intensity that can be survived) of a bridge, or to its median value (one of the two parameters of the assumedly lognormal fragility function of the bridge).

The former problem is more difficult and has not been dealt with. The second one, solved here, is of great interest for retrofit optimization (the median capacity of a bridge to withstand ground motion is one meaningful parameter that can be affected by retrofit actions).

Evaluation of performance sensitivities (first-order derivatives) by simulation proceeds as follows. Let $\mathbf{x}$ represents a vector of random variables describing uncertainty in the analysed system, and $f(\mathbf{x})$ its joint PDF. Any deterministic performance metric Y can be evaluated once a realization of **x** is known:

$$Y(\mathbf{x}) \tag{1}$$

Y can be for instance a Boolean variable expressing absence of a path between any two nodes in the network. The expected value of Y over the domain of **x** is then the probability of disconnection:

$$y = E_f[Y(\mathbf{x})] = \int Y(\mathbf{x}) f(\mathbf{x}, \boldsymbol{\theta}) d\mathbf{x} \tag{2}$$

Often the expectation is taken with respect to a distribution different from the original one, called sampling density *h*, and this forms the basis of variance-reduction techniques collectively denoted as Importance Sampling:

$$y = \int Y(\mathbf{x}) \frac{f(\mathbf{x}, \theta)}{h(\mathbf{x})} h(\mathbf{x}) d\mathbf{x} = E_h\left[Y(\mathbf{x}) \alpha(\mathbf{x}, \theta)\right] \tag{3}$$

where α is called likelihood ratio and accounts for the different probability content in **x** between original *f* and sampling *h* densities. Note that *h* does not depend on the distributional parameters $\boldsymbol{\theta}$ of *f*, neither does the performance Y which depends only on the value of **x**. This means that the derivative of the performance (e.g. probability) *y* can be readily obtained within the same simulation (no need to re-evaluate the system) simply by computing a modified likelihood ratio α':

$$\nabla_\theta y = \int Y(\mathbf{x}) \frac{\nabla_\theta f(\mathbf{x}, \theta)}{h(\mathbf{x})} h(\mathbf{x}) d\mathbf{x} = E_h\left[Y(\mathbf{x}) \alpha'(\mathbf{x}, \theta)\right] \tag{4}$$

If, as it is the usually the case, the joint PDF of **x** is given as the product of the joint probability densities or mass functions (herein denoted with the same symbol π) of *N* statistically independent sub-groups:

$$f(\mathbf{x}, \boldsymbol{\theta}) = \prod_{i=1,N} \pi_i(\mathbf{x}_i, \boldsymbol{\theta}_i) \tag{5}$$

the modified likelihood ratio takes the form:

$$\alpha_i'(\mathbf{x}) = \frac{\nabla_{\boldsymbol{\theta}_i} f(\mathbf{x}, \boldsymbol{\theta})}{h(\mathbf{x})} = \frac{f(\mathbf{x}, \boldsymbol{\theta})}{h(\mathbf{x})} \frac{\nabla_{\boldsymbol{\theta}_i} \pi_i(\mathbf{x}_i, \boldsymbol{\theta}_i)}{\pi_i(\mathbf{x}_i, \boldsymbol{\theta}_i)} = \alpha(\mathbf{x}) \frac{\nabla_{\boldsymbol{\theta}_i} \pi_i(\mathbf{x}_i, \boldsymbol{\theta}_i)}{\pi_i(\mathbf{x}_i, \boldsymbol{\theta}_i)} \tag{6}$$

which involves only evaluation of the ratio between the probability density derivative and the density itself, for the i-th group or element in the system. Notably the ratio $\nabla \pi / \pi$ can be obtained in close-form for many distributions and certainly for the Lognormal one which is the most widely used to model structural fragility functions.

4.1.2 Validation of simulation-based sensitivities

In order to validate the simulation-based method for sensitivity evaluation, a literature example has been chosen (data courtesy of Kang and Song). The system is a transportation network connecting 8 cities with 12 bridges, and is shown in Figure 1. As an example the event that city 5 is disconnected from city 1 where the hospital is located can be described as the failure of a parallel system of the 6 paths existing between the two cities, each path consisting of a series system of the involved bridges. Figure 2 shows the two bridge typologies (single-bent and double-bent) and the corresponding fragility functions.

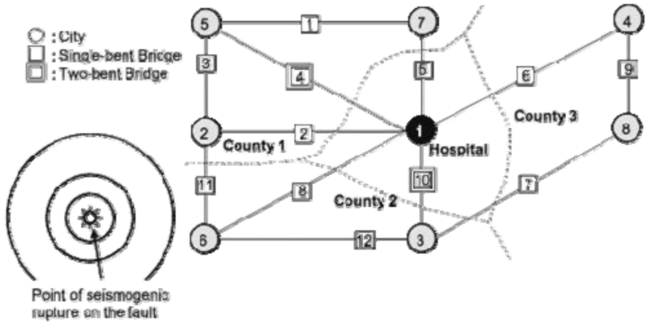

Figure 1. Network configuration from the example application in Kang et al 2008.

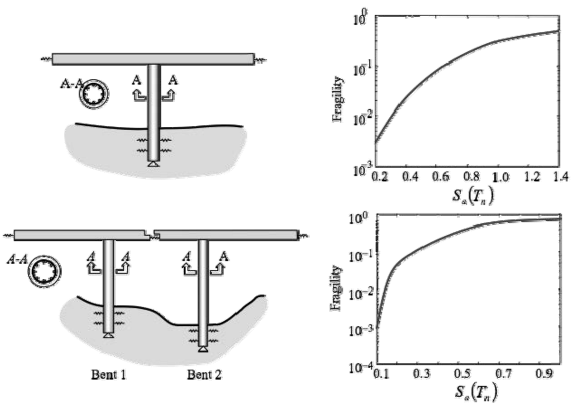

Figure 2. Bridge typologies and corresponding fragility functions from the example application in Kang et al 2008.

The original application had a very simple distributed seismic hazard characterization, which considered just a scenario event with no uncertainty on position and a single random variable to model magnitude (the single random variable introducing statistical dependence, see second year report), described by the truncated Gutenberg-Richter model. Also, in order to keep a single random variable, for the sake of simplicity, the authors have employed a deterministic GMPE at the bridge sites, which simply maps magnitude and distance in A_H (horizontal acceleration). The same hazard characterization has been used for the simulation-based solution, though by no means this is a limitation of the code developed which can be employed with a full distributed seismic hazard model of spatially cross-correlated intensities, including geotechnical hazards, such as that included in the framework developed within the SYNER-G project (SYNER-G, 2012) (Weatherill et al, 2014).

Figure 3 shows the comparison of the probabilities of a number of simple events (disconnection of each of the city from 2 to 8 from city 1, which hosts the hospital) and of a system event (disconnection of at least one city, series system). One can see how both Monte Carlo and Importance Sampling solutions are very close to the exact ones, with an obvious difference in the required sample size.

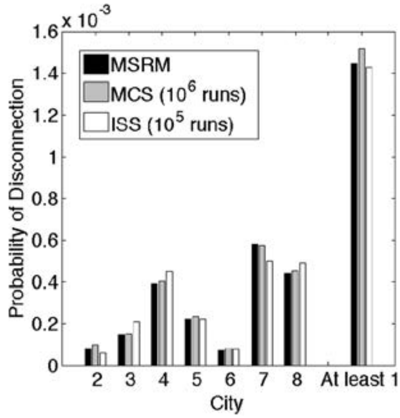

Figure 3. Comparison of probability of simple and system events.

Figure 4 shows instead the comparison between the sensitivities of the last event probability (disconnection of at least one city from the hospital) with respect to both the mean and the standard deviation of the bridge fragility for each of the twelve bridges. The sensitivity to the means, as already pointed out, can be of direct use in optimization procedures for the allocation of retrofit resources. The second derivative is a useful indicator for directing research/modelling and inspection budget, showing which bridge should be characterized in more detail (reduced sigma).

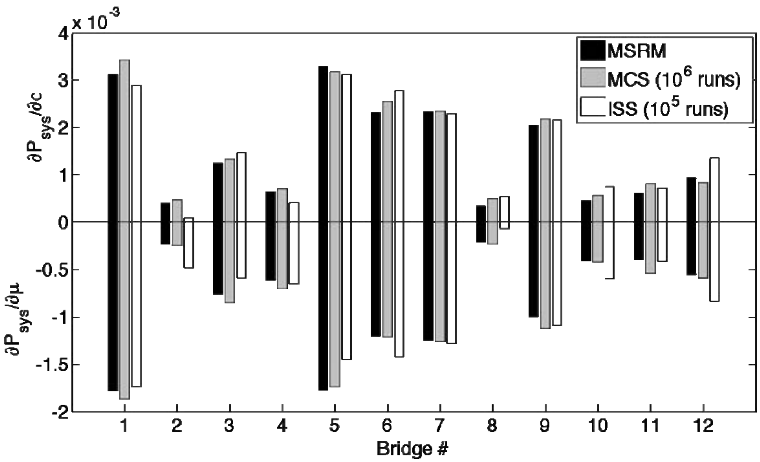

Figure 4. Comparison of sensitivities of the probability of the (series) system event with respect to both the mean (bottom) and standard deviation (top) of the bridge fragility of each of the 12 bridges in the region.

The results, as expected, show that increasing the mean capacity of bridges reduces the disconnection probability (negative sensitivity) and that decreasing the uncertainty on capacity characterization also decreases this probability. Both set of derivatives indicate the same ranking of the bridges. What is important is that simulation results are in excellent agreement with exact MSRM ones.

4.2 Bayesian inference within a Monte Carlo simulation framework for CI

As already stated, a framework for performance simulation of Critical Infrastructures (CI) under seismic hazard, at the regional or urban scale, has been developed (Franchin and Cavalieri, 2013)(Franchin, 2014) and constitutes the basis for research carried out in the project. The model can be used to directly assess the impact of regional seismic hazard on CI or a community, e.g. in terms of social loss such as displaced population (Cavalieri et al, 2012). Within this framework uncertainty is described in terms of random variables and their probabilistic dependencies; the framework, moreover, provides a complete Bayesian Network of the modeled systems. However, so far, the full power of the BNs has not been exploited yet, and the network is just the representation of a single run in a forward simulation: the only methods currently employed are, indeed, the plain Monte Carlo or Importance Sampling variants.

A BN (Jensen and Nielsen, 2007) is a directed acyclic graph in which nodes represent the problem variables (usually random variables with a finite number of possible states); and the edges represent relationships of relevance between the joined variables. To perform efficient inference in Bayesian networks, the network graph needs to be triangulated. Note that the quality of this triangulation largely determines the efficiency of the subsequent inference, but the triangulation problem is unfortunately NP-hard (Wen, 1990), and there is a rich literature proposing heuristic methods, as discussed in the survey of Kjærul (1990).

Thus, motivated by the need of efficient inference in the CI analysis framework, research has focused on the analysis of the generic BN built by this framework, to provide an explicit triangulation of the corresponding moralized graph; the results of this research, therefore, forms the basis for the explicit construction of the junction tree, that is the auxiliary graphical structure needed by the most typical exact (non heuristic) inference engines (Jensen and Nielsen, 2007).

Figure 5 shows a simple system of two infrastructure systems, as modeled within the framework. The figure shows six grid points and a seismic source. Events are generated from the latter (and more in general from all sources affecting the region of interest) and intensity is predicted (probabilistically) in the points of a regular grid (the six points in the example). Intensity at the components is obtained from the grid points' value by interpolation.

The BN associated with the system in Figure 5 is shown in XXX. This BN is a "superset" of that considered for this research, where the third layer, i.e. the System of Systems functional model, has been treated as deterministic.

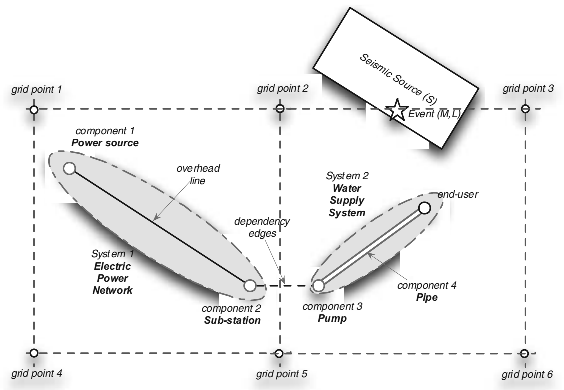

Figure 5. Two interconnected infrastructure systems, each with two components. The power networa sub-station (component 2) provides power to a pump (component 3) that inputs water into a pipe (component 4) connected to the end-user.

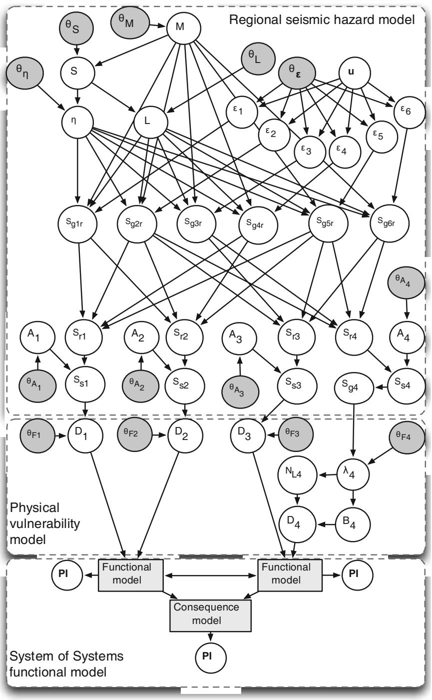

Figure 6. Graphical representation of the BN associated with the system in Figure 5 and generated by the model: circles represent random variables, rectangles represent "analytical/numerical models", arrows represent statistical dependence among random variables or input/output to/from a model.

An explicit triangulation algorithm, explained in detail in (Franchin and Laura, 2014) has been devised for the BN in Figure 6. Here only the result, in terms of the triangulated graph, is shown in Figure 7. The algorithm, however, works for any BN which is generated from the model, i.e. for system of any size and complexity, as long as they are modeled within the framework.

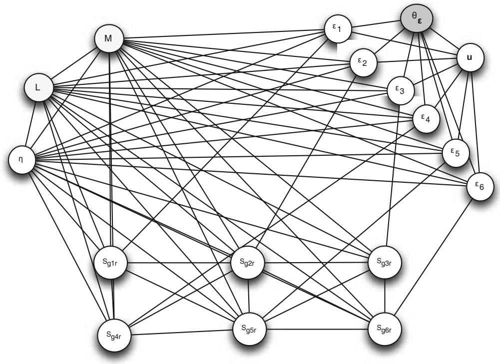

Figure 7. Graphical representation of the BN associated with the system in Figure 5 and generated by the model: circles represent random variables, rectangles represent "analytical/numerical models", arrows represent statistical dependence among random variables or input/output to/from a model.

5 DISCUSSION

The results reported in Section 4.1 show excellent agreement between simulation-based and exact sensitivity for the connectivity-based case. Results not shown for the sake of brevity (see second year report) highlight, however, how the derivative stabilization (with number of simulation runs) is slower than that of the differentiated quantity. This drawback needs to be further investigated in that it would weaken the statement about the derivatives being obtained "within the same" simulation. Actually, in order to grant the same degree of confidence, the actual cost of the simulation may need to be increased by an order of magnitude, thus leading in reality to a larger computational effort. On the other hand, other results, not shown, where simulation-based sensitivities are employed for a flow-based analysis of the same network (with a made-up origin destination matrix, and measuring performance with the Driver's Delay metric - see first year report), show that the stabilization of the metric and its sensitivities is in this case much faster. At present it cannot be ruled out that the larger required effort is an ineludible fact. Further work is thus needed.

6 VISIONS AND DEVELOPMENTS

The performance analysis of critical infrastructures is a fundamental building block for its risk and resilience assessment, with the long-term goal of reducing the former and increasing the latter. Importance of critical infrastructures cannot be understated, and efforts worldwide within the research community as well as in regulatory bodies are directed at producing sound and reliable methods of analysis and underlying models, that are feasible for application for risk mitigation and resilience enhancement. The final goal is the production of guidelines for use by network operators under the umbrella of coordination bodies that ensure effective implementation of measures on a physically interconnected but administratively fragmented system immersed in a multi-hazard environment.

Future research based on this project will employ the sensitivities results in an optimal upgrade resource allocation algorithm, and will test the triangulation algorithm within alternative inference algorithms to assess the practical feasibility and computational requirements of a real-time decision-support system for an infrastructure of realistic dimensions.

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TASK 2.2.5: NON-STRUCTURAL COMPONENTS

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1 INTRODUCTION

Non-structural components are those systems and elements housed or attached to the floors, roof, and walls of a building or industrial facility that are not part of the main or intended load-bearing structural system. These secondary structures may be classified into three broad categories: (1) Architectural components, (2) mechanical and electrical equipment and (3) building contents.
The evaluation of the seismic capacity of some nonstructural components is a main objective of the research study. Since these components are typically not amenable of traditional numerical analyses, the seismic fragility of several components is typically pursued through the experimental method.

2 BACKGROUND AND MOTIVATION

Experiences of past earthquakes have shown that the failure of nonstructural components may critically affect the performance of vital facilities such as fire and police stations and hospitals.
2009 L'Aquila earthquake widely confirmed the importance of nonstructural components behaviour related to the global seismic behaviour of the building. Indeed, most of the evacuated buildings exhibited no damage to structural components while heavy damages on the nonstructural ones, as shown in Figure 1 (Magliulo et al., 2009).

Figure 1. Hospital "San Salvatore dell'Aquila": ceilings failure.

It is also recognized that damage to secondary structures may seriously impair building function, and may results in a major direct and indirect economic losses. About the economic

impact caused by the failure of nonstructural components, evidence from past earthquakes has repeatedly shown that costs associated with the loss of the nonstructural components themselves, the loss of inventory and the loss of business income may easily exceed replacement costs of the building that houses those nonstructural components.
In Figure 2 an estimation of the costs related to typical buildings is shown, highlighting the different costs of the nonstructural and structural part (Taghavi and Miranda, 2003).

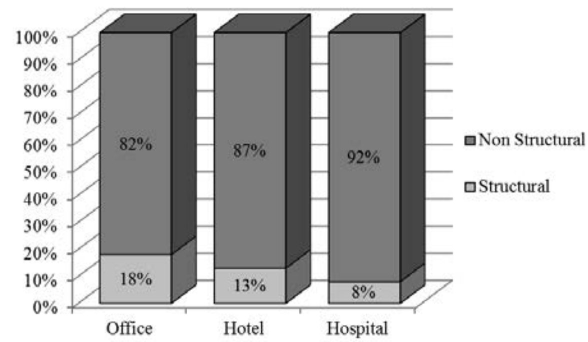

Figure 2. Typical cost distribution of three different types of buildings.

The failure of nonstructural components can cause injuries or deaths; for instance, the failure of cladding panels in precast structure was the main cause of fatalities in the 2012 Emilia earthquake (Northern Italy).
The threatening to the life safety due to nonstructural components increases if it is considered that suffocation is the most common cause of death due to an earthquake. The 64% of the fatalities caused by 1995 Great Hanshin Earthquake was due to the compression (suffocation) of the human body (Ikuta and Miyano, 2011). Such a phenomenon could be caused by the damage to nonstructural components, that may also obstruct the way out from the damaged building.

3 RESEARCH STRUCTURE

The aim of this task is the seismic qualification and the fragility evaluation of some nonstructural components, i.e. plasterboard ceilings and partitions, in order to allow performance analyses of new and existing buildings. Some innovative solutions concerning these components are also expected, in order to enhance the seismic performance of the tested nonstructural components.
Experimentation with shake table system plays an important role within the research project development. Indeed, analytical methods are not usually appropriate to study some nonstructural components, i.e. plasterboard ceilings, and past earthquake database are not suitable to evaluate nonstructural components fragility. Hence, experimental methods are the most appropriate technique to evaluate nonstructural components fragility curves.

4 MAIN RESULTS

The research was related to the investigation of the seismic behaviour of plasterboard and brick partitions via shake table tests. The tested plasterboard partition is an innovative partition typology, while the hollow brick partition is a standard partition, commonly used in Italy. Moreover, a quasi-static test campaign is performed in order to qualify standard plasterboard partition in industrial buildings.

The research firstly focused on the design of the shake table tests to perform for testing nonstructural components. The design phase was first related to the definition of the input to reproduce on the tables. Then, the design of two steel test frames, necessary to insert and test the nonstructural components, was performed.

The input to the table is provided through time histories of acceleration representative of expected/target ground motion on the nonstructural component and acting simultaneously along the two orthogonal directions of the platform simulator. These time histories are defined in order to match the target response spectrum of nonstructural components, clearly defined in ICBO AC156 "*Acceptance criteria for seismic qualification by shake-table testing of nonstructural components and systems*" (ICBO, 2000). The matching procedure provides the spectrum of the generated accelerograms to be included in a predefined range (0.9 times to 1.3 times the target spectrum) over the frequency range from 1.33 to 33.3 Hz (see "Figure 3").

Moreover, the accelerograms are filtered in order to not exceed the shake table limits in terms of velocity (±100cm/sec) and displacement (±25cm) (Magliulo et al., 2012). Finally, the accelerograms are scaled at increasing shaking levels.

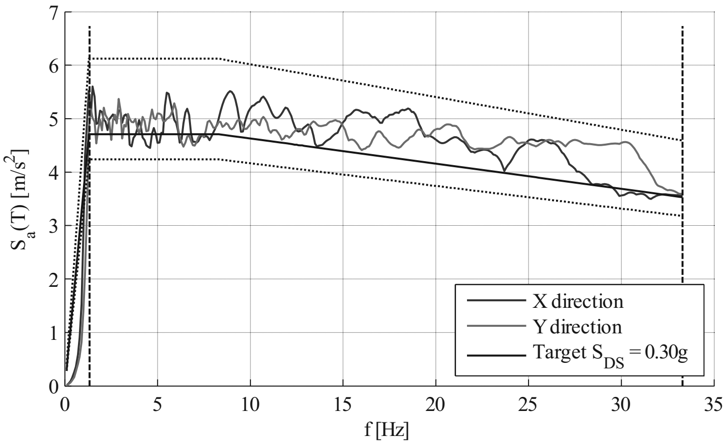

Figure 3. Input time histories and spectra for S_{DS} equal to 0.30 g: (a) acceleration, velocity and displacement time-history - X direction (blue) and Y direction (red); (b) input accelerogram spectra, RRS (bold line), upper and lower limits (dashed line), matching frequency range (vertical dashed line).

A flexible steel structure is designed to test the partitions "Figure 4", which are mainly drift – sensitive components. This structure simulates the behaviour of an ordinary structural floor, exhibiting a 0.5% drift for an earthquake characterised by a return period equal to 50 years. In Figure 4 the test setups for the two above mentioned test campaigns are depicted.

In particular, bidirectional shaking table tests are performed in order to investigate the seismic performance of hollow brick partitions, subjecting the partition simultaneously to interstory relative displacements in their own plane and accelerations in the out of plane direction.
The tests points out an excellent seismic performance of the plasterboard partition system with respect to the brick one. The innovative plasterboard partition system exhibits a good seismic behavior: a minor damage state is attained for 0.58% drift level, while a moderate damage state is attained for 0.98% drift level. The dynamic identification procedure and the experimental evidence shows that the tested plasterboards partitions do not contribute to the structural lateral stiffness (Magliulo et al., 2014). Indeed, no variations in terms of stiffness and structural period are recorded after introducing the partitions within the test frame; moreover, the partitions implies a damping increase, resulting in a beneficial effect in relation to the earthquake.

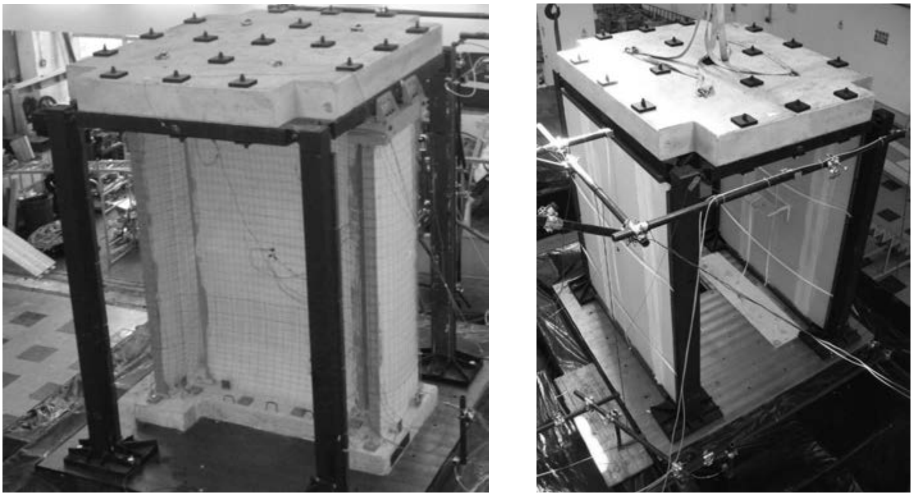

Figure 4. Test setups for seismic qualification of internal partitions.

A numerical model of the test setup, i.e. the test frame and the partitions, is defined and subjected to the recorded base acceleration time-histories through the OpenSees program. The test frame is modelled as a single degree of freedom system. The hysteretic curve comparison shows a good matching in terms of dissipated energy, while the comparison of displacement time histories shows the excellent matching between experimental and numerical results.
The shake table tests executed for the different intensity levels on the brick partitions point out several damages on the system that compromised its integrity. The dynamic identification procedure and the experimental evidence shows that the tested partitions contribute to the structural stiffness. The tests point out several damages on the system that compromised its integrity. The hollow brick partition is subjected to interstorey drift up to 1.0%. It exhibits minor damage for 0.2% interstorey drift, moderate damage for 0.34% interstorey drift and major damage for 0.97% interstorey drift (Petrone et al., 2014).
Standard methods for the dynamic identification of the test setup are used in order to evaluate the influence of the hollow brick partitions on the steel test frame. The change in the natural frequency and the damping ratio during the different seismic tests clearly evidence the damage recorded in the specimen.

Quasi-static tests are performed on eight 5 m tall plasterboard internal partitions, representative of typical partitions used in industrial and commercial buildings in the European area. A steel test setup “Figure 5” is designed in order to transfer the load, provided by the actuator, to the partition. The testing protocol provided by FEMA 461 (2007) is adopted for the quasi-static tests.
The specimens typical failure mode is the buckling of a steel stud, that involves the boards attached to the buckled stud. The buckling failure usually concentrates across plasterboard horizontal joints. The hysteresis loops evidence first a frictional behavior for low demand levels, and then a pinched behavior for moderate-to-high demand levels.
The interstory drift ratios (IDRs) required to reach a given damage limit state are evaluated through a predefined damage scheme. Based on the experimental data the fragility curves for three different damage states are estimated. The fragility curve yields median IDR values equal to 0.28%, 0.81% and 2.05% and logarithmic standard deviations equal to 0.39, 0.42 and 0.46 for DS1, DS2 and DS3, respectively.

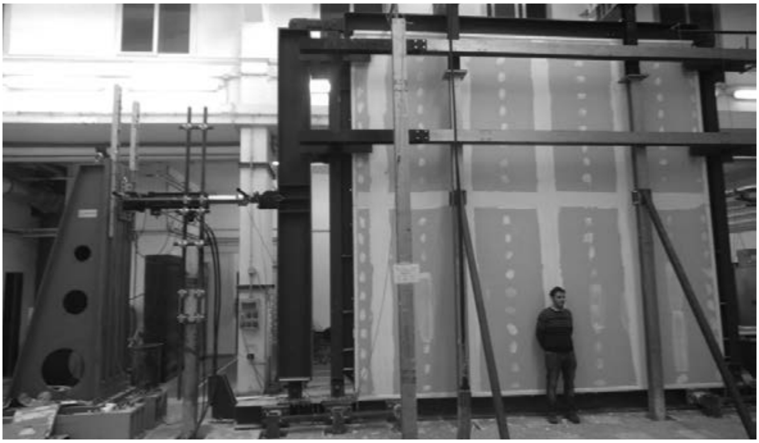

Figure 5. Test setup for quasi-static tests on 5 m tall plasterboard internal partitions.

5 DISCUSSION

The research work is in line with the objective of the research project. Both an innovative plasterboard partition system and a “standard” brick partition are seismically qualified through a shake table test campaign. The fragility curve of 5 m high plasterboard partitions is also evaluated, through quasi-static tests.

6 VISIONS AND DEVELOPMENTS

The research study can be developed by using the experimental data for the definition of a numerical model of the tested components. In particular, the proposal of a micro-modelling technique of these components may allow to investigate the influence of the different geometrical and mechanical parameters on the seismic response of the components. Such a definition would allow to generalize the experimental results to a wide set of specimens.

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SUB-TASK 2.2.6.1: MONUMENTS AND MUSEUMS

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1 INTRODUCTION

In Italy the high seismic hazard can be added to the considerable wealth of artistic heritage and this situation creates a high risk for the same artistic heritage.
In the work developed for ReLUIS we intend to investigate a general methodology for analyzing and evaluating the seismic risk of art objects (with particular but not exclusive reference to the statues) in their place of exhibitions (museums, galleries, etc.).
The goal is to encode a real test protocol, through which the authorities, responsible for the preservation of these artworks, can assess the risk level of protected artistic objects.
The proposed methodology is quick and simple to apply and it is specially coded to allow even unskilled staff to carry out expeditious screening on a large scale of sculptural heritage.

2 BACKGROUND AND MOTIVATION

The recent earthquakes in Italy have again highlighted the high vulnerability of the statues and art objects preserved in museums and the need to recognize and quantify the level of vulnerability of these statues to establish priorities for action in the protection of museum artistic objects. In particular, recent earthquakes emphasized how there were a lot of museums which, although reporting earthquake damage, are not collapsed, while the statues contained within them are irretrievably lost due to lack of attention to their position (Figure 1).

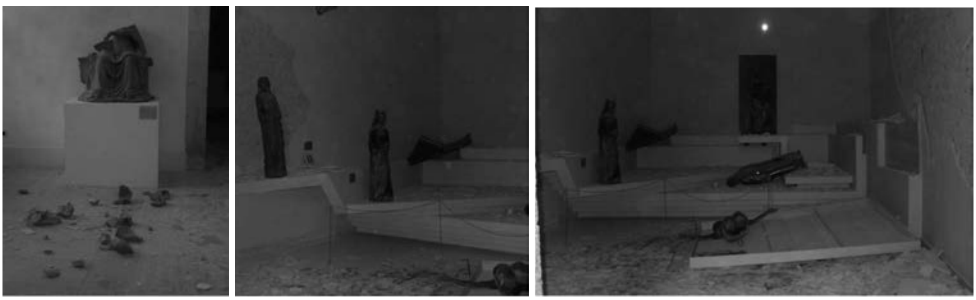

Figure 1. Examples of severe damage to the statues (sometimes irreparable) exhibited in buildings which have been damaged but have not collapsed.

Such an issue was addressed in some Italian and international works (Augusti et al, 2000; AA.VV., 2005; Ishiyama, 1982). Some of these works, however, proposed solutions based on risk assessment procedures particularly onerous and conceived to be applied only by technical staff. The Italian current Code for cultural heritage in seismic zones does not elaborate on this issue and focuses more on buildings.
The amplitude of the Italian sculptural heritage instead suggests the opportunity to do a self-evaluation of seismic risk through immediate application of tools that can be used even by

non-technical staff, and then use the finest tools only in cases where it is really needed (for instance because it was highlighted by the self-evaluation procedure).

3 RESEARCH STRUCTURE

The main objective of the research is to develop a tool which is quick and usable by non-technical staff to identify and quantify numerically the vulnerability to overturning of statues and museum objects.
During the first year it was developed a quick vulnerability index (I_s) derived from an analysis that compares the statue / object exposed to a rigid body susceptible to tipping under seismic conditions. The I_s index depends on the site of the museum, on the storey of exhibition of the statue and on the shape of the statue. The statue has been considered as a rigid block with infinite resistance, the sliding between the statue and the support has been excluded. In fact, we are only interested in analysing the conditions that lead to the permanent loss of the statue caused by overturning for seismic action. In the case of the statues or rigid objects it is extremely simple to find the multiplier of activation for overturning α_0.
We will consider two configurations:
Case A: statue without base;
Case B: statue resting on a base structurally separated from the statue
In case A it is possible only a collapse mechanism (mechanism 1): the overturning of the statue on a horizontal hinge O_S placed at the foot of the statue (Figure 2).
In case B it is possible to have two mechanisms of collapse (Figure 3):
Mechanism 1: the overturning of the statue on a horizontal hinge O_S placed at the foot of the statue.
Mechanism 2: the overturning of the whole system formed by the statue and the base on a horizontal hinge O_B placed at the foot of the base.

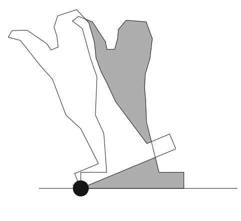

Figure 2. Case A: mechanism 1

The choice of the overturning preferential direction is determined by minimization of the horizontal arms of the stabilizing weights. It is worth noting that the overturning of a statue with a base could take place in a different direction from that of the overturning of the statue and basement system.
When it has been determined the activation multiplier for the mechanism, it must be transformed in the corresponding activation acceleration a^*_0 considering only the fraction of the participating mass involved in the mechanism.

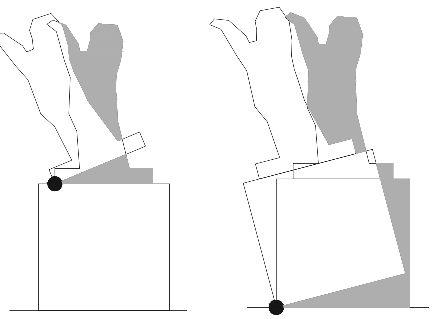

Figure 3. Case B: mechanisms 1 and 2

To determine the seismic vulnerability index of the statues is necessary to distinguish the case of the statues at the ground level from the case of the statues at high floor. Therefore we have two possible cases:
1) Statue at the ground level.
In the case of statues placed at ground level there is no amplication due to the "filter" effect of the structure and then the acceleration to consider is directly a_g. Therefore, according to Italian and European Codes, we can say that:

$$I_{S(SUOLO)} = \frac{a_g S}{a_0^* q}$$

where:
a_g is the site acceleration at the limit state of interest;
S is the amplifying factor due to stratigraphic and topographic effects.

2) Statue at a level higher than the ground floor.
We obtain the following expression:

$$I_{S(QUOTA)} = \frac{\psi \gamma a_g S F_0}{a_0^* q}$$

This expression takes into account:
- Shape and weight of the statue and of the base (through a^*_0);
- Exhibition floor of the statue (through ψ);
- Number of floors of the building (through γ);
- Acceleration of seismic site (a_g);
- Stratigraphic and topographic amplification (S).
In this case, the vulnerability index will be equal to the greater of the vulnerability index of the statue at the ground level and at the higher level.
To determine the vulnerability index of a statue one can proceed according to two approaches:
a) quick approach, in which the overturning activation acceleration a^*_0 is calculated on the basis of geometric macro-characteristics of the statue. With this approach the survey of the statue is not necessary and it is possible to determine the so-called “quick vulnerability index I_s” by non-technical staff as well.

b) exhaustive approach, in which the overturning activation acceleration a^*_0 is calculated by considering the real three-dimensional shape of the statue. This approach is very difficult and requires a laser scanning and must be carried out by technical staff. The result is the so called "real vulnerability I_r index" evaluated for the effective geometry of the statue.
To define the quick approach ("a" type), it is necessary to choose several parameters which can be fundamental in the outcome of the final assessment and divide them into classes of variability easily identified by non-technical staff.
Such parameters can be used to describe any type of configuration that a statue can assume and the I_s quick index can be preliminary calculated for each configuration. In this way non-technical staff will only recognize the particular fields in which the statue can be set and it will be immediately clear the value of the vulnerability index I_s referred to the statue.
The are five parameters able to define the statue configuration: slenderness (S), shape (F), hanging out (A), exhibition floor (P) and total number of floors of the building (N). Each of these parameters can assume conventionally three (or four) values which modify the configuration of the statue and consequently the I_s index.
During the second year the procedure to determine the I_s index was computerized using a worksheet of immediate use (Figure 4). In addition, we performed the measurements with laser scans of some statues of the National Gallery of Umbria and we proceeded to return in CAD of some of these laser scans.
During the third year it has been finished the return in CAD of all the laser scans and have been conducted calculations and evaluations aimed at validating the procedure for estimating the vulnerability with the I_s index (see below).
Particularly, during the third year, starting from the actual characteristics of the scanned statues it has been evaluated their susceptibility to overturn determining the effective acceleration of activation of the mechanism and deducing from this the real vulnerability I_r index.
The scanned statues are ten, three of which are rigidly connected to form a single group of sculptures. Four of these statues are of marble, while the remaining four are made of wood. Six statues are provided with pedestal susceptible to tipping while two statues have a pedestal rigidly attached to the floor.

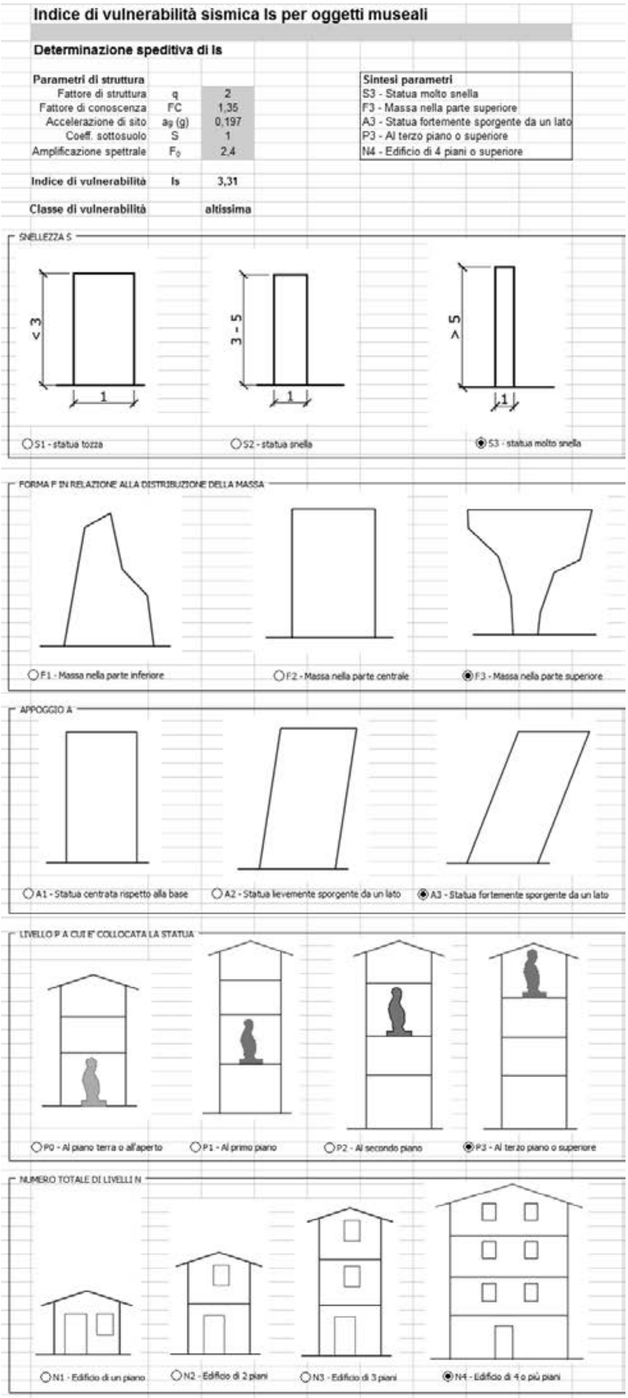

Figure 4. Electronic worksheet to calculate quick index Is.

In order to have a greater number of data based on existing statues, it has been changed the site of the statues (Perugia or L'Aquila) and the exhibition storey (ground or second and third floor) evaluating four configurations for each statue, and thus reaching a total of 56 patterns.
In the Figures 5 – 11 you can see some examples of geometric surveys by laser scanner, modeling and analysing statues and bases.

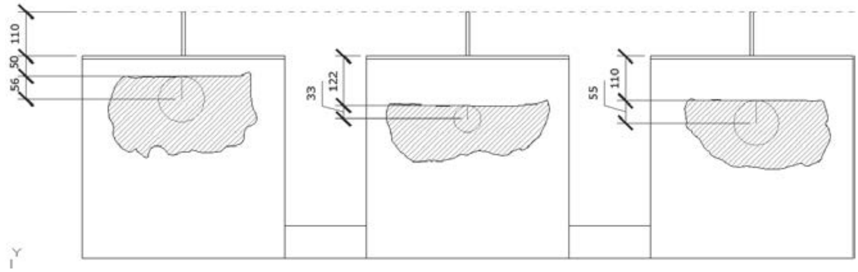

Figure 5. Patron Saints of Perugia. Reconstruction of the coordinates of the centers of gravity of the three statues.

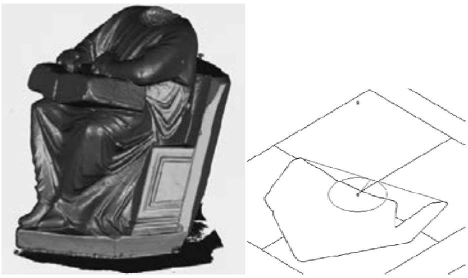

Figure 6. Headless Jurist by Arnolfo di Cambio. Model derived from laser scanning and reconstruction of the coordinates of the center of gravity and of the overturning axis at the base.

Figure 7. Patron Saints of Perugia. Models derived from laser scanning.

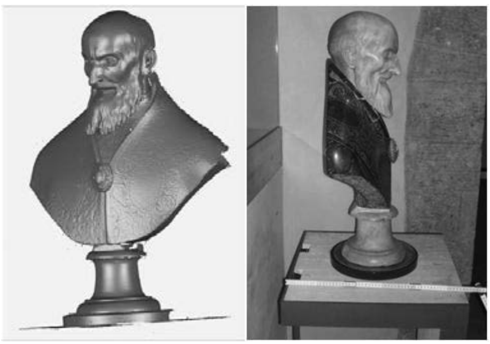

Figure 8. Bust of Pius V by Leonardo Sormanni. Model derived from laser scanning and statue.

Figure 9. Bust of Marcantonio Eugeni by Francesco Mochi. Finding on the model from laser scan of the rest areas on the base.

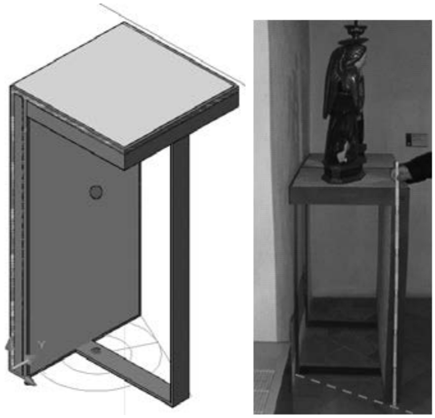

Figure 10. Model of a type of base, with the position of center of gravity and its projection on the ground and highlighting of the axis of rotation of the system composed of statue and base.

Figure 11. Schematic model to show the position of the center of gravity of the Pietà in wood. The dark base is rigidly connected to the statue and it is fundamental to its stability.

4 MAIN RESULTS

The results show a good correspondence between the two methods of estimating the vulnerability to overturn, the quick method based on the I_s index and the method based on the laser scanning of the statues and the real vulnerability index (I_r) (Figure 12).
It is possible to conclude that the method based on the quick I_s index, used even by non-technical staff on the basis of the compilation of a simple data sheet with the main features of the statues, provides results in accordance with the method based on laser scanning of statues and pedestals.

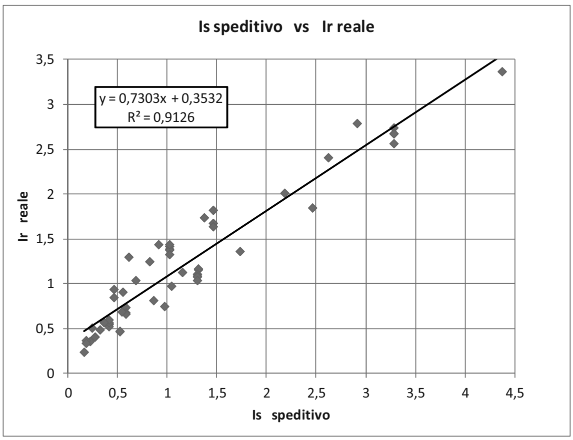

Figure 12. Comparison of indices of vulnerability. In x-axis we can see the quick index Is evaluated on the basis of direct observation of the characteristics of the statue. In y-axes we can see the real index Ir evaluated for the cases of study thanks to the determination of the actual shape of the statues, their sections of rotation and their bases detected by laser scanning.

The method of seismic risk assessment which is here proposed can be used as a first quick self-evaluation and assessment of seismic risk that can also be done by the staff of the museum. This does not preclude a subsequent deeper analysis carried out by technical staff and engineers for those cases that should manifest a particularly high value of I_s. In this way it would be possible to optimize resources and perform analysis of seismic risk and made anti-seismic provisions where it is really needed. In order to suggest modifications and evolution of the technical Codes and of professional practice, the recent Italian earthquakes have shown that it would be very appropriate to protect museum objects from earthquakes higher than those considered in the safety assessment of museum-buildings. This is a point that is different from the current approach of the Italian current Code for cultural heritage in seismic zones. The reason of such a proposal is that while a building at Limit State of Life Safeguard SLV still possesses resistance to seismic action, we cannot say the same for museum objects that are relatively easier to model and for which you can predict with relatively greater accuracy the seismic behaviour.

5 DISCUSSION

The objectives set at the beginning of this work have been achieved: the development of an index for the quick evaluation of the seismic risk of museum objects and its validation with respect to the most reliable and time-consuming procedures based on laser scanning of the statues and museum objects. Further tests are desirable in order to further validate the proposed procedure. It is also desirable a future development of the quick risk assessment procedure that can consider also the problem of sliding of the statues, although this phenomenon is rarer and more easily controllable in terms of anti-seismic provision with respect to seismic overturning.

6 VISIONS AND DEVELOPMENTS

The research on vulnerability and seismic risk of museum objects should be taken forward by proposing quick assessment indices for other types of objects such as paintings, paintings on wood, etc .

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SUB-TASK 2.2.6.2: MONUMENTS AND MUSEUMS

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1 INTRODUCTION

Italy has a wide archaeological cultural heritage that is not negligible with respect to architectonics assets and deserves a suitable protection against earthquakes. Today there are a lot of procedures in Codes to analyse normal structures but for archaeological finds there is a lack of a knowledge path and analysis strategies to protect them in the course of time.
The large number of finds present in archaeological areas, often with recurrent typological characteristics, need the development of strategies to define the most vulnerable typologies and the evidences with high seismic risk within them.
Why cannot we consider the age of the archaeological evidence as a test of seismic safety? With reference to the archaeological finds in the Roman Forum or in the Palatino Area, in Rome, it seems evident that this assumption can be disregarded for two main aspects. On the one hand the physiological deterioration can lead to the reduction of the structural safety of architectonical elements because of the natural decay of the mechanical properties of the materials. This aspect characterizes, in particular, the archaeological finds comprised of a single structural element. The eventual mortar deterioration, or the surface degradation caused by pollution phenomena, can lead to local collapses if the basic maintenance rules have been ignored. Moreover many archaeological finds remained underground and they have been excavated only in the last century. On the other hand, for the archaeological finds that can be assimilated to buildings, it is not possible to assume the a-thousand-year-old history as a structural efficiency tests, because of many transformations occurred during last centuries.
Therefore excavations and transformations can be considered as a reset of the seismic history and so the seismic performance had to be analysed.
The importance of seismic risk mitigation of cultural heritage has increased in the last years, today it highlights the problem of conservation of archaeological sites. It is necessary to define tools able to evaluate the seismic vulnerability for a great number of elements.
Many archaeological finds have a structural behaviour that can be analysed with the macro-block model as columns, obelisks, triumphal arches, trilithons, and freestanding old masonry walls. This aspect, with the purpose to develop a tool that can be applied to a large number of evidences, leads to the use of the limit analyses with reference to the kinematic one. This approach allows to define a value of the seismic risk and requires a few input parameters to be applied. The output is a synthetic index that, when compared with that of the other finds, allows compiling a priority list, needed to schedule additional inspection and analysis.

2 BACKGROUND AND MOTIVATION

The study of archaeological assets, from a seismic point of view, is a new issue that recently has led a large number of works in both national and international sphere. The prevalence of works are concentrated on the study of the dynamics of the oscillating single rigid body [Housner, 1963], in order to determine closed-form rigorous or simplified solutions that could provide, considering sometimes simple seismic actions, an estimate of a parameter which is able to describe the response [Makris and Roussos, 2000; Dimitrakopoulos and DeJong, 2012; DeJong and Dimitrikopoulos, 2013; Lagomarsino, 2014]. Another topic that is highly studied concerns the seismic response of structures having more than one block as multidrums columns or temple portions. The complexity in defining an analytical model to consider systems composed of several blocks [Psycharis, 1990; Spanos *et al.*, 2001], led the authors to study these systems, from a numerical point of view, using, in the majority of cases, (discrete) distinct element models [Papantonopoulos *et al.*, 2002; Psycharis *et al.*, 2003; Argyriou *et al.*, 2007; Dimitri *et al.*, 2011; Scandolo, 2014]. The computational effort of these tools often leads to a single case study, without that generalization which a very extensive archaeological heritage needs.

For these reasons a tool has been developed in this project to define quantitatively, albeit with only a preliminary level of investigation, the seismic vulnerability of a large number of archaeological finds and hence it is applicable on a territorial level.

3 RESEARCH STRUCTURE

The aim of the research is connected to the definition of a methodology for the evaluation of the seismic safety of the archaeological finds at territorial scale. To achieve this purpose, firstly, it is necessary to classify the archaeological finds. The structural behaviour of the most common types (freestanding old masonry wall, single column, trilithon, etc.) can be analysed using the kinematic theorem of the limit analysis once the most probable failure mechanisms have been identified. In particular, the authors will propose the limit domains that can be used to evaluate the seismic safety level of the structural system analysed. Nevertheless the high number of structural types that can be observed, only freestanding old masonry wall or monolithic columns have been analysed.

The high number of structural elements compounding the archaeological heritage needs to define appropriate methodologies. Regarding this aspect, a pre-analysis approach will be developed for identifying the most vulnerable elements that will be the focus of following studies. The methodology will be applied at territorial level; for this reason, it has to be based on few parameters, easy to be surveyed. The method, that can be developed for the most common typologies of archaeological heritage is based on the equilibrium limit analysis as suggested in the paragraph C8A.4 in the commentary No. 617 02/02/2009 - Instructions for the application of the Italian Technical Building Code (M.D 14/01/2008) for the evaluation of the seismic safety for existing masonry buildings related to local collapse mechanisms.

Applying the limit analysis, limit domains will be defined. These domains will allow us to identify the threshold values to consider the macro-element appropriate to support the design seismic action. From these parameters, it will be possible to identify the archaeological finds characterized by the highest seismic vulnerability.

For the elements with the highest seismic risk, an increase of the knowledge through additional inspections and tests (stratigraphic analysis, diagnostic tests, in-depth survey of the construction details, etc.) will allow the definition of the safety level of the structure through deeper analyses.

This approach would significantly reduce the number of elements to be studied through complex analysis, by providing a list of priority in terms of seismic risk.

4 MAIN RESULTS

The kinematic approach also allows for the determination of the horizontal force evolution that the structure is progressively able to withstand meanwhile the mechanism evolves. Having defined α as the ratio of the horizontal forces applied to the corresponding weights of the structural masses, such an evolution can be represented by a curve of α multiplier as a function of the displacement d_k of a reference point in the system. The curve, determined up to the annulment of any capability of sustaining the horizontal actions (α=0), can be transformed into a capacity curve of an equivalent single-degree-of-freedom system, for which the ultimate displacement capacity of the local mechanism can be defined and compared to the displacement demand requested by the seismic action.
In order to get an evaluation of the horizontal loads multiplier α_0 that leads to the activation of the local mechanism, it is necessary assigning a generalized virtual displacement to the generic block of the kinematic chain, one can determine, as a function of the rotation and geometry of the structure, the displacements components of application points of the various forces applied in their respective directions. The multiplier α_0 is then obtained by applying the Principle of the Virtual Work and equating the work done by the internal and external forces applied to the system acting through the virtual displacements:

$$\alpha_0\left(\sum_{i=1}^{n} P_i\delta_{x,i} + \sum_{j=n+1}^{n+m} P_j\delta_{x,j}\right) - \sum_{i=1}^{n} P_i\delta_{y,i} - \sum_{i=1}^{o} F_h\delta_h = L_{fi} \qquad (1)$$

where:

- n is the number of all the forces applied to the various blocks of the kinematic chain;
- m is the number of forces not directly acting on the blocks, generating horizontal forces on the element of the kinematic chain;
- o is the number of external forces, not associated to masses, applied to the blocks;
- P_i is the vertical force of the generic block;
- P_j is the generic vertical force, not directly applied to the blocks;
- $\delta_{x,i}$, $\delta_{x,j}$ are the horizontal virtual displacements of the points of application of the forces P_i and P_j;
- $\delta_{y,i}$ is the vertical virtual displacement of the point of application of load P_i;
- F_h, δ_h are, respectively, the external force applied to a block and the displacement of the application point;
- L_{fi} represents the work of the internal forces.

In order to evaluate the displacement capacity of the structure, up to its collapse, in the considered mechanism, the horizontal load multiplier α_0 can be determined not only with reference to the initial configuration, but also to modified configurations of the kinematic chain, representatives of the evolution of the mechanism and defined by the displacement d_k of a system reference point. The analysis must be carried out up to the configuration corresponding to the annulment of the α multiplier, corresponding to a displacement $d_{k,0}$. For any configuration of the mechanism, the value of α_0 can be determined from Eq. (1), properly rewritten by referring to the modified geometry. The analysis can be carried out with the use of graphic methods, by defining the system geometry in the different configurations up to its collapse, or with analytical-numerical methods, by considering a set of virtual displacements and rotations to be progressively updated based on the system geometry evolution.
The d_k - α curve can be converted in the capacity curve of the system applying Eqs. (2):

$$d^* = \frac{d_k}{F_C} \frac{\sum_{i=1}^{n+m} P_i \delta_{x,i}^2}{\delta_{x,k} \sum_{i=1}^{n+m} P_i \delta_{x,i}}; \quad M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{x,i}^2}; \quad a^* = \frac{\alpha \sum_{i=1}^{n+m} P_i}{F_C M^*} = \frac{\alpha g}{F_C e^*} \tag{2}$$

where:

- M^* is the mass of the structure participating in the mechanism
- $\delta_{x,i}$ is the horizontal virtual displacement of the centroid of the *i*-th block calculated with reference to the initial configuration of the system;
- g is the gravity acceleration;
- F_C is the confidence factor;
- d_k is the finite horizontal displacement of the reference point and $\delta_{x,k}$ is its virtual horizontal displacement.

This approach is used in the developed tool to define the vulnerability of archaeological evidences, described in the following section.
The tools is developed on the basis of the Excel file SPETTRI-NTC ver. 1.0.3 released by High Council of Italian Public Works in order to have the hazard of the Italian areas directly build in the tool, without resort to other software.
The tool is a sequence of three windows (phases) and an introduction (Figure 1).

Figure 1. Introduction of the tool.

The first phase (Figure 2) defines the localization of the area of interest either through specification of the geographic coordinates or by selection of the district. It is possible to choose two methods of interpolation of the seismic grid.
The second phase (Figure 3) define the returned period of the action, the characteristic of geological site, the damping and the Limit State (LS).

The last phase (Figure 4) is the more interesting one. It is divided in two parts. On the left side the verification is performed according to the linear and nonlinear kinematic analysis of a rectangular block with dimension b and h for width and height respectively, even considering the hinge shifting inward to account for shortcomings or a non-infinite material compressive strength.

Figure 2. Phase 1 of the tool: location of the site.

Figure 3. Phase 2 of the tool: definition of the seismic input.

The definition field of the displacement ULS as a fraction of the displacement for which the load multiplier reaches the zero value, is very interesting. This ratio is conventionally taken equal to 0.4 but can be modified by users to consider also systems with a number of blocks greater than 1, as multidrum columns, although in a simplified way [Scandolo, 2014].
The non-linear check can be obtained considering the spectral displacement referred to secant period (as suggested by the Instructions for the application of the Italian Technical Building Code) or referred to the maximum spectral displacement ($T_S=\infty$).
A graphical plot of the geometry of the mechanism is shown in the chart, that it can be updated with a single click on it.

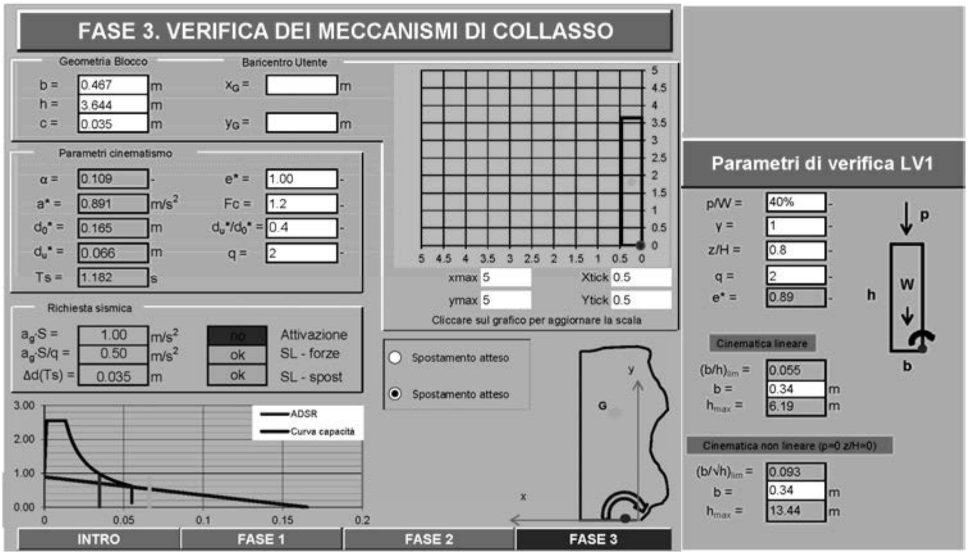

Figure 4. Phase 3 of the tool: definition and check of the mechanism.

On the right side there is the model to perform a territorial level analysis with the linear or the non-linear kinematic approach. In the first case the block is rectangular with dimension equal to b and h (width and height respectively) and it is possible to define a load *p* on the top of the block (in the middle of the width). It is possible to take into account the overturning of a block with a hinge that is not at the ground, so an amplification of the seismic input can exist. For these reasons the modal participation factor and the ratio between the height of the hinge and the height of the construction should be defined. Considering $T_B \leq T_1 \leq T_C$ the spectral coordinate is independent from the natural period T_1.
Equating the spectral acceleration that brings to the activation of the mechanism with the seismic demand is possible to obtain the limit domain Eq. (3):

$$\left(\frac{b}{h}\right)_{\lim} = \max\left\{\frac{1+2\overline{p}}{\left(1+\overline{p}\right)}\frac{a_g S e^* F_C}{gq};\quad \frac{1+2\overline{p}}{\left(1+\overline{p}\right)}\frac{a_g F_O S \psi \gamma e^* F_C}{gq}\right\} \tag{3}$$

where:

- $\overline{p}$ is the ratio between the load *p* and the self-weight W of the block;

- $e^* = \dfrac{1+4p+p^2}{1+4p+p^2}$ is the fraction of participating mass;
- a_g, S, F_O are the seismic parameter that represent the hazard of the site and the LS;
- q is the behaviour factor and conventionally equal to 2;
- γ is the modal participation factor
- Ψ is the modal shape and can be assumed linear equal to z/H

If the width of the block is defined it is possible to obtain the limit height of the block, geometrical parameter easy to survey.
If p and z are zero, a limit domain can be also obtained with reference to the non-linear approach, in an analytical way. The capacity curve is described by Eq. (4) and evaluating spectral displacement with Eq. (5) at the secant period the seismic demand is obtained, with the hypothesis that $T_C \leq T_S \leq T_D$. If the capacity displacement (conventionally equal to 0.4 d_0^*) and the displacement demand are balanced, the limit domain can be obtained Eq.(6).

$$a^* = a_0^*\left(1 - \frac{d^*}{d_0^*}\right) \tag{4}$$

$$S_d\left(T_S\right) = \frac{F_O S a_g T_C}{4\pi^2} 2\pi \sqrt{\frac{0.08h}{0.84g}} \cong 0.016 \cdot \sqrt{h} F_O S a_g T_C \tag{5}$$

$$\frac{b}{\sqrt{h}} \cong 0.08 F_O S a_g T_C \tag{6}$$

Also in this case if the width is chosen it is possible to find the maximum height of the block over that the non-linear check with kinematic approach is not verified.
Then with this tool is possible to perform a single check of a overturning block (or a multidrum columns) or defining the limit geometrical parameter that need to perform an analysis at territorial level.

5 DISCUSSION

The final result of the research involved a software that is able to evaluate the seismic vulnerability of archaeological finds from a territorial point of view. The tool has been developed taking into account, directly inside it, the seismic hazard of the Italian territory, without having to interface with other programs. The software, as described previously, consider only some type of archaeological evidences, but can be easily implemented for other types of finds.

6 VISIONS AND DEVELOPMENTS

The development of a tool that is able to quickly provide an index on the seismic vulnerability of an archaeological asset is very important. This work has focused on archaeological structures whose structural behaviour is associated with the rocking of monolithic block, like

isolated columns, obelisks, statues or fragment of walls. A few instructions are supplied to study also multidrum columns.
This tool can be developed to take into account systems such as triumphal arches and trilithons (i.e. systems whose kinematics are characterized by the formation of 4 hinges when you consider the in-plane behaviour). With this addition the tool will be able to quantify, although at a first level of analysis, the seismic risk of a large number of archaeological finds.

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TECHNOLOGICAL INNOVATIONS IN SEISMIC ENGINEERING

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1 INTRODUCTION

A summary of the research activities developed by Task 3.1 of the ReLUIS DPC 2010-13 Project on the "*Development and analysis of innovative materials for seismic consolidation*" is presented in this paper.
Task 3.1 was organized in the following six research lines and involved twenty four research units, whose topics are listed below with the indication of the corresponding coordinators.

LINE A: *REINFORCING OF MASONRY STRUCTURES*

- *Plane Structures Subjected to Alternate Loads.*

	Research Unit	Coordinator
1	Polytechnic University of Milan	Prof. Carlo Poggi
2	University of Naples "Federico II"	Prof. Gaetano Manfredi
3	University of Perugia	Prof. Antonio Borri
4	University of Sannio	Prof. Francesca Ceroni
5	University of Bologna "Alma Mater Studiorum"	Proff. Marco Savoia and Claudio Mazzotti

- *Arches, Vaults and Domes*.

	Research Unit	Coordinator
6	University of Naples "Federico II"	Prof. Alessandro Baratta
7	University of Bologna "Alma Mater Studiorum"	Prof. Andrea Benedetti
8	University of Florence	Prof. Silvia Briccoli Bati

- *Computational Methods for Interventions of Seismic Reinforcement*.

	Research Unit	Coordinator
9	University of Cassino and Southern Lazio	Proff. Raimondo Luciano and Elio Sacco
10	University of Naples "Federico II"	Prof. Luciano Rosati
11	University of Salerno	Prof. Fernando Fraternali

LINE B: *REINFORCING OF CONCRETE STRUCTURES*

- *Overlapping Areas, Buckling of Longitudinal Bars, Beam column joints.*

	Research Unit	Coordinator
12	University of Naples "Federico II"	Prof. Andrea Prota

- *Use of Pre-stressed Lamina.*

	Research Unit	Coordinator
13	University of Rome "La Sapienza"	Prof. Giorgio Monti

LINE C: *NON-TRADITIONAL COMPOSITE MATERIALS*

- *Mechanical Systems of Anchoring.*

	Research Unit	Coordinator
14	University of Salerno	Proff. Ciro Faella and Roberto Realfonzo

- *Reinforcement with non-traditional fibres and/or matrices.*

	Research Unit	Coordinator
15	University of Salento (Lecce)	Prof. Maria Antonietta Aiello
16	University of Calabria	Proff. Renato Olivito and Giuseppe Spadea

LINE D: *FRP PULTRUDED MATERIALS FOR THE IMPLEMENTATION OF TEMPORARY STRUCTURES FOR THE ESSENTIAL ACTIVITIES RELATED TO CIVIL PROTECTION*

- *Bolted and adhesive joints.*

	Research Unit	Coordinator
17	University of Salerno	Prof. Luciano Feo
18	University of Rome “Tor Vergata”	Prof. Franco Maceri

- *Numerical modelling.*

	Research Unit	Coordinator
19	University of Ferrara	Prof. Ferdinando Laudiero

LINE E: *FIBER REINFORCED CONCRETE*

	Research Unit	Coordinator
20	Polytechnic University of Milan	Prof. Claudio Di Prisco
21	University of Brescia	Prof. Giovanni Plizzari
22	University of Rome “Tor Vergata”	Prof. Antonio Grimaldi

23	University of Bergamo	Prof. Paolo Riva

LINE F: *STRUCTURAL CRYSTALS*

	Research Unit	Coordinator
24	University of Parma	Prof. Gianni Royer Carfagni

2 TOPICS OF THE RESEARCH LINES

2.1 Line A

The main aim of Line A has been the study of the use of FRP (Fibre Reinforced Polymer) materials for the retrofitting and the seismic consolidation of masonry structures. In fact, a great number of existing masonry structures, originally designed without taking into account any type of seismic criterion, are currently undergoing a severe process of structural strengthening in order to improve their strength, ductility and, in general, their capacity to withstand seismic horizontal forces. In particular, the recent seismic events of last years have shown that Italian architectural heritage presents high seismic vulnerability.

2.2 Line B

The main aim of Line B has been to study how to prevent the brittle collapse mechanisms related to the failure of the reinforced concrete structures strengthened with FRP materials due to the loss of bond in steel overlapping areas into columns, as well as the failure due to buckling of longitudinal steel bars into the columns, and due to tensile stresses on the beam-column joint. Furthermore, Line B has dealt with the aspects related to the use of both pre-stressed FRP materials and Near Surface Mounted (NSM) elements.

2.3 Line C

The main object of the Line C has been the analysis of non-traditional techniques for the reinforcement of concrete and masonry constructions. In particular, the effectiveness and reliability of composites realized with cementitious matrices (FRCM) has been investigated as well as that of composites made of natural fibres. Furthermore, the use of the Mechanically Fastening FRP (MF-FRP) technique has been investigated from a theoretical and experimental point of view.

2.4 Line D

A fundamental task for the Civil Protection Service consists in the fast realization of provisional structures. The use of FRP materials for realizing all-composite modular structures can be undoubtedly considered as an innovative and unconventional solution. Within this framework Line D has developed several theoretical and experimental aspects dealing with: i) the characterization of the mechanical behaviour of bolted joints between structural elements made of composite materials; ii) the evaluation of the strength and stiffness characteristics of open-web pultruded profiles; iii) the mode II resistance of bonded joints between GFRP adherents; iv) the numerical modelling of the buckling behaviour of pultruded profiles; v) the experimental identification of the sectional properties of such elements.

2.5 Line E

Line E has dealt with the development and application of High Performance Fibre Reinforced Concrete (HPFRC) for the strengthening and retrofitting of existing R.C. or masonry structures in seismic zones.
The technique is based on the application of a thin jacket made in HPFRC material, characterized by high strength in compression and hardening behaviour in tension.
The Line has developed both analytical and experimental analyses dealing with the use of such technique.

2.6 Line F

Observation of the damages occurred during recent earthquakes has evidenced critical elements that, while not participating to the gross structural stability, nevertheless may cause severe damage to people and property after collapse. These are generally “non-structural” elements, unfortunately not sufficiently well analysed with verification calculations, but at which the utmost attention must be paid. Among these one must undoubtedly consider the glazing systems, which are particularly critical because the glass is the brittle material *par excellence* and, in case of an earthquake, it does not allow any type of plastic adaptation as other building materials. To obtain adequate safety it is in general convenient to laminate glass with polymeric interlayers, in such a way that the fragments remain attached to the interlayer after glass rupture. Therefore, the main objective of the research has been the characterization of the mechanical behaviour of laminated safety-glass, both in pre- and in post-glass-breakage phase.

3 RESEARCH STRUCTURE

3.1 Line A - Research Unit no. 1

The research explored different aspects related to the consolidation of masonry structures. Systems with inorganic matrices (FRCM) were studied in order to solve the drawbacks of reinforcement with epoxy matrix (FRP) (poor fire resistance, low water vapour permeability, non-reversibility of the intervention, poor compatibility with masonry).

3.2 Line A - Research Unit no. 2

The research aimed at investigating the benefits provided by FRCM systems to increase the in plane behaviour of masonry walls. In particular, uncoursed stone masonry panels (representative of existing masonry buildings of L’Aquila) have been tested under diagonal compression tests in the original or strengthened configuration and data of previous tests carried out on other masonry typologies have been collected and analysed.

3.3 Line A - Research Unit no. 3

The objective of this research unit has been to investigate the effectiveness of innovative techniques for strengthening masonry structures. In particular, the following activities have been developed:

- experimental tests on prototypes of masonry columns strengthened by the application of small diameter stainless steel cords able to provide overlapping hoops in

correspondence of mortar joints. Furthermore, a first application of the proposed strengthening system has been analysed;
- experimental in-situ tests on masonry panels strengthened with a mortar coating on both surfaces reinforced with GFRP grid;
- experimental tests on prototypes of brickwork arches strengthened with an innovative technique based on the historical evolution of an ancient system of reinforcing tiled vaults belonging to the ancient constructive Spanish tradition;
- experimental tests on reinforced panels with an innovative technique of consolidation that uses artificial diatoni pre-stressed.

3.4 Line A - Research Unit no. 4

The research program has comprised the study of the adherence of FRP sheets on masonry supports as well as the capacity of masonry walls reinforced with FRPs subjected to cyclic horizontal actions.
In particular, for what concerns this last topic, several experimental tests have been carried out on masonry panels with a reinforcing system made by covering the external surfaces of the masonry walls by means of layers of mortar containing a fibre reinforced polymeric mesh. The walls considered in the study were characterized by having a regular arrangement of blocks and mortar joints, namely adobe, solid burned and unburned clay bricks. The influence of the strengthening on the global capacity in term of increment of strength and ductility was investigated.
Then, the unreinforced masonry walls and reinforced walls have been modelled by means of finite element method in order to investigate the effect of some mechanical parameters of masonry (compressive strength, tensile strength, fracture energy) on the non-linear behaviour of the reinforced elements. The calibration of non-linear parameters for materials employed in the models and a parametric analysis was carried out considering different types of masonry, thickness of mortar layers and amount of composite mesh.
The global effects of the increment of shear strength and displacement capacity of the walls given by the strengthening were evidenced through the study of the behaviour of a whole masonry building.

3.5 Line A - Research Unit no. 5

The main objectives of the research have been:

- investigation of bond behaviour between brick masonry panels and FRP sheets bonded according to the EBR technique;
- investigation of the effect of surface preparation and of the type of brick on the bond strength;
- modelling of the FRP-masonry bond behaviour;
- definition of design criteria for masonry panels strengthened with FRP reinforcement.

More precisely, the research program was divided in two experimental phases: the first was devoted to the investigation of the FRP-masonry bond behaviour, in order to find out the main parameters affecting the problem; the second one was devoted to the structural behaviour of some masonry panels strengthened with FRP elements.

3.6 Line A - Research Unit no. 6

The research activities have been the following:

- elaboration and formulation of a unified overall, comprehensive treatment of the NT (No Tension) theory;
- re-formulation of the NT theory and development of structural analysis methods in presence of FRP refurbishment;
- formulation of NT models for vaulted masonry structures and development of closed-form solutions;
- analytical investigation of the scale effects on historical and monumental masonry constructions;
- analysis methods for vaulted masonry structures;
- analysis methods for vaulted FRP reinforced masonry structures;
- calculus procedures for vaulted masonry structures with or without FRP reinforcement;
- static and possibly dynamic analysis for the analytical-numerical and/or experimental investigation of arched or vaulted masonry structures.

3.7 Line A - Research Unit no. 7

The research unit has carried out the following activities:

- structural analyses of several types of vaults and arches as found in masonry buildings damaged by the earthquake of Aquila;
- proposal of reinforcing interventions by using traditional and non-traditional techniques, these last ones by using FRP materials;
- non-linear numerical analyses of reinforced vaults, assuming as an evolutive parameter the motion of the boundary walls, due to the seismic behaviour of the building where the vault is.

3.8 Line A - Research Unit no. 8

The main aim of the research activities was the definition of the effective structural reinforcements that ensure optimal interventions from the point of view of the increase in load-bearing capacity, and of the compatibility with the existing structure. Particular attention was devoted to the definition of minimally invasive, easy to make and low environmental impact techniques.

The research program included:

- mechanical tests on scale models of vaulted structures reinforced with fibre-reinforced composite materials;
- the development of computational models able to predict the behaviour of the examined structures.

3.9 Line A - Research Unit no. 9

The research activities of this Unit comprised several investigations dealing with:

- the mechanism of interaction between FRP reinforcements and masonry supports by means of experimental tests;
- the implementation of a numerical code for predicting the debonding phenomenon in reinforced masonry structures.

3.10 Line A - Research Unit no. 10

The main objective of this research project was to investigate and to clarify the methodological principles and the operative procedures for an accurate non-linear static and dynamic analysis of reinforced high strength concrete structures. Within this framework the purpose of the research was to develop finite element models and algorithms for the non-linear analysis of framed and shear-walled structures.
The research was subdivided into the following parts:

- development of the fibre-free approach;
- modelling of the non-linear behaviour of reinforced concrete shear walls and formulation of up-to-date guidelines for building code instructions.

3.11 Line A - Research Unit no. 11

The activity of this research unit has been focused on the development of suitable mechanical models of the unreinforced masonry. Such an activity has been also extended to FRP-reinforced masonry structures on employing topology optimization techniques for the optimal design of the FRP wrapping system. Furthermore, the activity has been focused on the prediction of the state of stress associated with real examples of FRP-reinforced masonry structures.

3.12 Line B - Research Unit no. 12

The activities involved experimental and theoretical studies to increase the database of experimental data already available in the scientific literature and to improve the knowledge on FRP strengthening of reinforced concrete elements. The theoretical activity aimed at providing analytical models to design FRP strengthening interventions, such models were also validated by means of comparison with experimental data. In particular the main activities were as follows:

- modelling the buckling restraint provided by FRP wrapping;
- modelling the bond improvement to lap slices provided by FRP wrapping;
- experimental tests on corner joints under constant axial load and transverse cyclic loading in the as-built and FRP strengthened configuration.

3.13 Line B - Research Unit no. 13

The research program dealing with pre-stressed FRPs was carried out, firstly, by proposing and realizing a new tensioning device suitable for FRP fabrics, whose effectiveness was tested in the lab. Then, a theory of RC beams strengthened with FRP was developed, which eventually led to some design equations that can be used to optimize a strengthening intervention through pre-stressed FRP. Finally, advantages of using pre-stressed FRP in strengthening RC beam were evaluated and analysed by using purposely developed easy-to-use spreadsheets.
Furthermore, the research program dealing with NSM elements included the following two activities:

- execution of parametric studies by means of the original and more complex predictive model;
- production of a simplified formula for practitioners.

3.14 Line C - Research Unit no. 14

The research program focused on:

- evaluating the experimental behaviour of MF-FRP connections subjected to direct shear tests (DSTs);
- calibrating, on experimental basis, the constitutive law of the connection and then validating such law through numerical simulations;
- modelling the flexural behaviour of RC beams strengthened with MF-FRP laminates.

3.15 Line C - Research Unit no. 15

A first research activity consisted of compression tests on masonry columns confined with the traditional FRP systems and with innovative systems in comparison to the first ones. Different strengthening schemes, types of reinforcement and types of matrix were analysed and tested.
In the second research activity, within the first one, a wide experimental program was performed through tests of diagonal shear on masonry walls with single or double hanging. Particularly the tests of diagonal compression are performed for studying the influence of new types of reinforcement (basalt and natural fibres), the geometrical configuration of the scheme of the reinforcement (diagonal strips or grid) and the eccentricity of the reinforcement (fibrous reinforcement applied on one or two sides of the wall panel).
The last research activity consisted in studying the bond behaviour between fibrous reinforcement and masonry support through bond tests with a new test apparatus for "single lap-shear test”. The following parameters were analysed: different types of masonry substrate (tile and natural stone), different fibres (glass, steel, carbon, basalt and natural fibres as flax and hemp).

3.16 Line C - Research Unit no. 16

The project comprised different stages: the production of resin-based composites and mortar-base composites, the mechanical characterization of the matrices and composites and tests on masonry microelements strengthened with natural fibres-based composites. Natural materials made of flax, hemp, jute and sisal were used in order to carry out the experimentation.
The experimental program was organized as follows:

- tensile tests on single yarns, on non-impregnated fabrics and on natural fibre based composites with epoxy and polyester resin and mortar;
- three point bending tests on unreinforced brick and reinforced brick with natural fibre based composites (flax and hemp) using epoxy based matrix;
- pull-out tests on unreinforced brick and reinforced brick with natural fibre based composites (flax and hemp) using epoxy based matrix;
- single-lap shear bond tests on reinforced brick with natural fibre based composites (flax and hemp) using epoxy based matrix and mortar based matrix;
- shear bond tests carried out on concrete specimens strengthen with PBO fibre meshes and cementitious mortar varying the bonding length and the thickness of fibres.

3.17 Line D - Research Unit no. 17

On the subject of bolted connections, the activities have been focused on the experimental and numerical evaluation of both the axial stiffness and strength of web-flange junctions of PFRP pultruded I-profiles.

On the subject of adhesive joints, the activities were focused on: i) the implementation of a numerical code for studying the behaviour of adhesive curved joints; ii) a numerical procedure to evaluate the optimal bonding length, and iii) the implementation of a numerical code to study the mechanical behaviour of FRP/masonry or concrete joints.

3.18 Line D - Research Unit no. 18

The research comprised the following activities:

- characterization of the constitutive behaviour of FRP composite materials, even functionally graded, and of structural elements made of FRP composite;
- characterization of the mechanical behaviour of bolted joints between structural elements made of composite materials;
- development of methods for analysis and design of full-composite structural modules for civil applications, accounting for aspects of vibration control and dynamic protection.

3.19 Line D - Research Unit no. 19

The activity developed by the Unit took into account the following topics:

- several non-linear analyses of pultruded elements in the presence of out-of-flatness, out-of-straightness and angular imperfections;
- evaluation of the full-section properties of the profiles using the results of static bending tests and the inverse equations of the Timoshenko beam model.

3.20 Line E - Research Unit no. 20

In the first year the topic of the mechanical characterization of new materials HPFRC (High Performance Fibre Reinforced Concrete, or high-performance fiber-reinforced concrete) and TRC (Textile Reinforced Concrete, concrete with Alkali-resistant glass fabrics) was carried out. In the second year the transmission of tangential forces between uncracked and pre-cracked weakly-reinforced concrete elements (with cracks of variable widths) and surface retrofitting layers made of the advanced innovative materials investigated in the first year, by using a new identification test (DEWS = Double Edge Wedge Splitting test), capable of applying a state of uniaxial traction on undamaged element or pre-cracked constituent element was investigated. Finally, in the third year the following three topics were developed: DEWS test reliability to identify the uniaxial post-cracking response, durability of TRC material when subjected to freezing and thawing cycles, effectiveness evaluation of the use of advanced materials retrofitting compared to the use of stainless steel wire mesh introduced in layers of traditional mortar. The last topic was investigated with reference to concrete weakly-reinforced coupling beams, typical of stabilizing cores like staircase group buildings to offer a limited, but ductile seismic resistance, when loaded by significant shear deformations.

3.21 Line E - Research Unit no. 21

The aim of the research was to develop new clean, cost and time effective techniques, based on the use of innovative materials, which allow to improve the in-plane shear resistance of Un-Reinforced Masonry (URM) structures. The strengthening technique employed an advanced nanocomposite steel fibre reinforced mortar not containing cement but only particles of calcium aluminates and corundum. The research aimed at proving the effectiveness of such a technique for improving the in-plane shear resistance of URM walls.

The Unit performed an experimental program which included tests on a series of clay brick masonry walls aiming at representing a portion of an existing load-bearing cantilever wall placed in a typical two-storey masonry structure built in the middle of the 20th century. The test walls were 3070 mm long, 230 mm (solid bricks) or 250 mm (hollow bricks) wide and had a total height of 1970 mm; the dimensions of the specimen were chosen to get an effective length-to-height aspect ratio (L/H) of about 1.5, which promotes a shear critical behaviour of the member. The specimens were constructed by using both 230 mm (length) x 50 mm (height) x 110 mm (width) solid-clay bricks and 300 mm (length) x 200 mm (height) x 250 mm (width) hollow clay bricks. The nominal head and bed joints thickness was 10 mm; the mortar used to fill the joints had a compressive strength of 4 MPa. All the tests were performed under a constant axial and variable in-plane horizontal reverse-cyclic load.

3.22 Line E - Research Unit no. 22

The first year of activity was devoted to the definition of the high performance fibre reinforced material suitable for seismic retrofitting. At this aim different concrete mix and fibre typologies were considered and combined, in order to obtain a material fulfilling the required performance. In particular it was requested a fibre reinforced concrete with hardening behaviour in tension and significant strength both in compression and tension.
In the second year, the effectiveness of the application of a layer, or a jacket of the defined fibre reinforced material, was analysed in a numerical and analytical way, with reference to simple R.C. elements, such as beams, columns or walls. To this aim a procedure based on the definition of an “equivalent section” was defined for the evaluation of the local and global behaviour of beam-column elements.
During the third year the research was extended to simple structures, in order to catch the effect of a local or global strengthening on the whole global behaviour of the structures. Numerical Push over and non-linear dynamic analyses were performed for the evaluation of the influence of the strengthening layer on the global strength and on the “behaviour factor”.

3.23 Line E - Research Unit no. 23

The first year of activities was devoted to perform experimental tests on retrofitted column-foundation and interior beam-column joints. Furthermore, experimental results were compared with analytical evaluations. A simplified numerical model able to describe the seismic behaviour of unretrofitted corner beam-column joints was proposed.
The second year of activity was devoted to perform cyclic experimental tests on full scale corner beam-column joints designed according to the Italian construction practice of the 70s. Unretrofitted and retrofitted joints were examined and comparisons between experimental and analytical results were developed. In particular, the numerical evaluations were performed by using the proposed model.
During the third year the experimental and numerical investigation was enlarged. A spreadsheet for the design of the retrofitting proposed solution was pointed out.

3.24 Line E - Research Unit no. 24

The activity of the unit was focused on safeguarding the glazing against seismic actions. Two possibilities were individuated:

- isolate the glazing, in such a way that the oscillation of the main structure does not load it;
- check that in case of breakage, glass does not have catastrophic failures and/or detachments that may endanger the safety of people.

The research was directed on one hand towards the definition of general criteria for the design of the isolation of glazing and, on the other hand, to define the mechanical behaviour of laminated safety glass, which offers a better performance in the case of glass breakage. A further increase in performance can also be achieved by reinforcing glass panels with FRP plates.

4 MAIN RESULTS AND DISCUSSION

4.1 Line A - Research Unit no. 1

The experimental research focused on different issues and different materials for the consolidation of masonry elements.

Connectors in fibre-reinforced materials

The aim of this activity was to test the strength of a new generation of connectors in fibre-reinforced materials, consisting of long unidirectional fibres in blind holes or with anchors bow. The experimental tests included mechanical characterization of single system components (two epoxy resins and connector in CFRP and GFRP) and pull-out tests of connectors on masonry elements. The results showed that the bricks and mortar characteristics and the techniques of impregnation of the connectors influence the failure mode of the system. A numerical analysis with FEM program and an analytical model were developed to simulate the relative displacements that occur between support and connector.

Characterization of fiber reinforced cementitious matrix (FRCM)

The study included the mechanical properties and durability of reinforced system that not involve the use of epoxy resin but of inorganic matrices (FRCM), and the definition of tests for the acceptance of these materials. The analysed materials were FRCM made with PBO (polyparaphenylene benzobisoxaole), glass fibres and carbon fibres.

The characterization of the mortars and the fibre textile was performed, and tensile tests (monotonic and with loading-unloading cycles) were carried out on FRCM specimens (Figure 1).

Push-pull double lap and pull-off tests were carried out to analyse the bond behaviour between FRCM and substrate and between mortar and fibres.

In order to study the durability of FRCM materials subjected to freeze-thaw cycles and saline and alkaline environment, experimental tests were performed at the University of Miami.

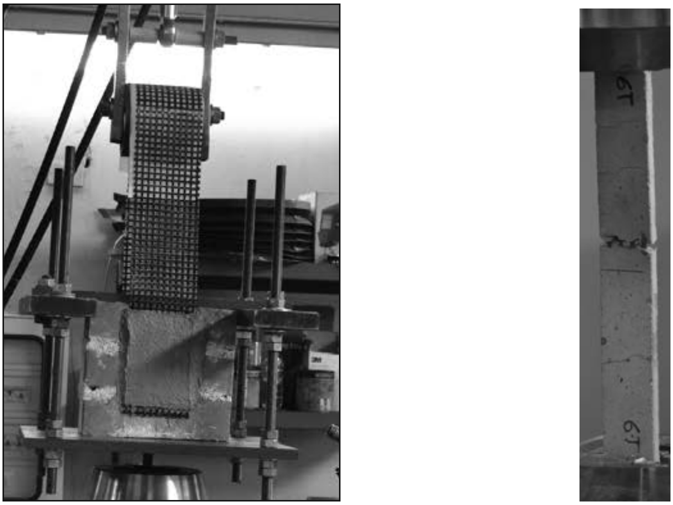
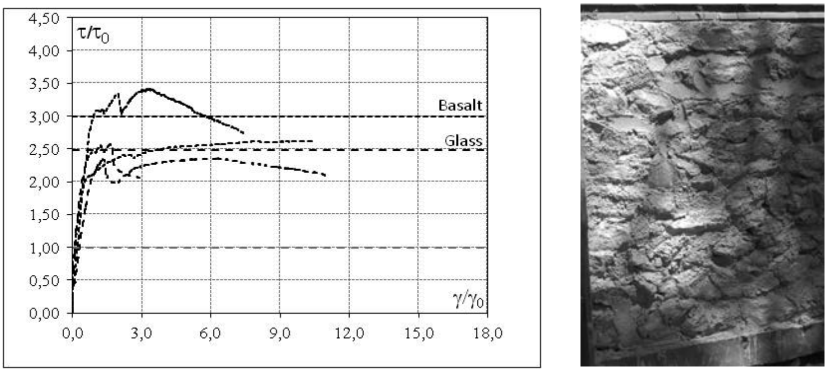


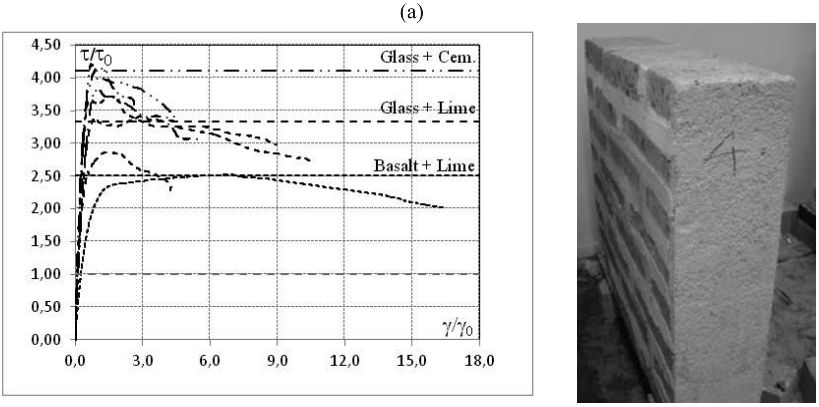
Figure 1. Characterization of FRCM.

4.2 Line A - Research Unit no.2

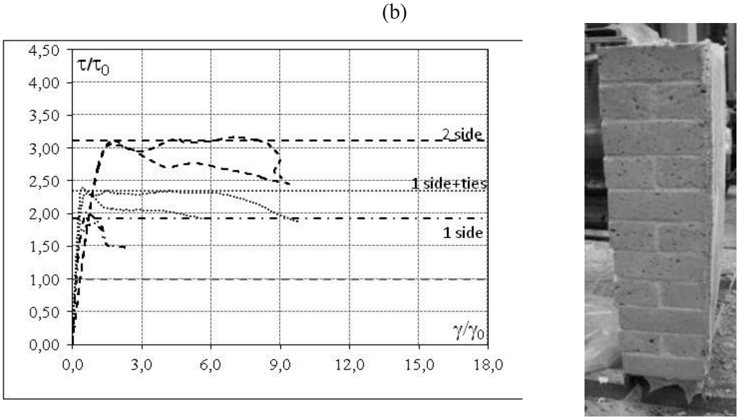
The experimental outcomes of 30 tests on masonry panels reinforced with FRCM strengthening technique have been analysed along with the results of 5 tests on a different masonry typology representative of existing masonry buildings of L'Aquila. The experimental programs clearly demonstrated that the use of FRCM strengthening solution may significantly improve the in plane mechanical behaviour of buildings made by yellow tuff or uncoursed stone masonry (see Figure 2). The experimental results validated the effectiveness of lime mortar as an alternative matrix system to the cementitious one; the lime mortar may be a very promising solution especially in cases for which the compatibility with physical and mechanical properties of existing masonry is a crucial aspect. The number of FRP plies slightly influences the shear strength and strain of reinforced masonry panels while a significant effect is ensured by the application of the IMG strengthening solution on both sides of masonry panels rather than one. The use of SFRP ties may be strongly necessary in cases of very poor original masonry (as the case of uncoursed stone masonry or masonry with very poor mortar mechanical properties) to delay the reinforcement debonding and to fully exploit the shear strength capacity increase provided by the IMG strengthening solution.



(a)



(b)



(c)

Figure 2. Stress - Strain Behaviour of the Samples Uncoursed Stone (a); Yellow Tuff One Head (b); Yellow Tuff Two Heads (c).

4.3 Line A – Research Unit no. 3

Several experimental investigations were performed (Figure 3). The main results are summarized below.

Experimental investigation on masonry columns: three series of uniaxial compression tests, with a total of 19 specimens, were conducted on masonry columns with these variables: cross-section geometry, amount and scheme of confining reinforcement. The proposed strengthening system has then been applied to the design process of the retrofitting intervention for a masonry column, belonging to a portico built inside The San Girolamo cloister in the city of Spello (Italy).

Experimental investigation on masonry walls: 13 diagonal-compression and shear-compression tests have been carried out on masonry specimens of three different types of masonry. Masonry walls have been reinforced with a mortar coating reinforced with a GFPR grid.

Experimental investigation on tile arches: eight prototypes of brickwork arches, strengthened by overlapping different layers of tiles and laminates, embedded within an hydraulic mortar, were tested under a monotonic vertical load applied at the keystone. The influence of the types of reinforcement, number of layers and properties of hydraulic mortar has been investigated.

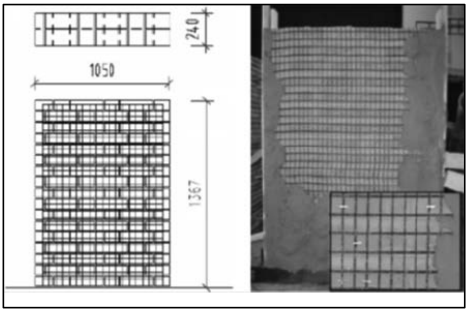
Experimental investigation on panels reinforced with artificial diatoni: both laboratory and in situ test (overturning) were carried out on masonry panels reinforced with diatoni of different types. Also, monitoring for the determination of the loss of load over time was carried out.

Figure 3. Experimental investigations on masonry columns, walls and arches.

4.4 Line A - Research Unit no. 4

The Unit performed a numerical investigation on masonry walls reinforced with FRPs under the application of vertical pre-stressing loading and cyclic horizontal loading applied on top of the walls. The masonry panels had overall dimensions of 240 mm in thickness, 1050 mm in width and 1367 mm in height, made by 17 courses of adobe bricks arranged according to typical Flemish bond pattern. One type of reinforcement was made by polymeric grids (polyester and polypropylene) according to symmetric configuration on both surfaces of the

walls. The position of the polymeric grid is illustrated in Figure 4. The walls were rendered with about 2 cm thick plaster made of an adobe-based mortar with a low percentage of sand.
It was observed that the application of the reinforced mortar layers onto the wall's surfaces allows the panels to reach a higher value of strength, with an increment of about 20% and 30% in the case of polyester and polypropylene grid, respectively, and produce a significant enhancement in terms of ductility (until 80%).



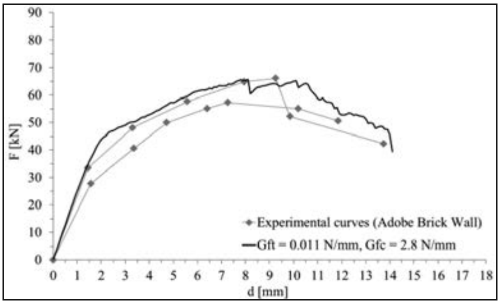


Figure 4. Experimental investigation under cyclic loads.

4.5 Line A - Research Unit no. 5

The activity of the Unit included both an experimental and a numerical investigation.

<u>*Experimental phase*</u>**:** The effect of different types of FRP reinforcements on the bond behaviour, the effective bond length and the role played by the mortar layers orientation were investigated: not only on the bond strength but also and particularly on the shear stress-slip law. The other fundamental aspect of this phase of tests is the adoption of Digital Image Correlation as a strain measuring technique, able to locally evaluate the bond behaviour. The series of bond tests carried out in the framework of a Round Robin Test program organized within the activity of RILEM Committee 223 MSC, the local coordinator is part of, were completed.
Further bond tests were developed on glass fibre grids bonded to single bricks by using cementitious mortars (FRCM). Results showed that bond capacity is usually higher than the tensile strength of the composite; this type of behaviour is in clear contrast with the recognised behaviour of classical FRP composite materials and specific design criteria will be required. In the same framework, several direct tensile tests on FRCM specimens were carried out in order to find out reliable evaluation criteria of the effectiveness of the composite.

Results clarify the fundamental role played by the cementitious matrix in governing the cracking phase and the corresponding possible damage localization.

Numerical phase: Numerical activities were also performed (Figure 5), starting form results and models obtained the second year, by analysing the possible strategies for the description of bond-slip laws and by verifying the most effective calibration techniques.

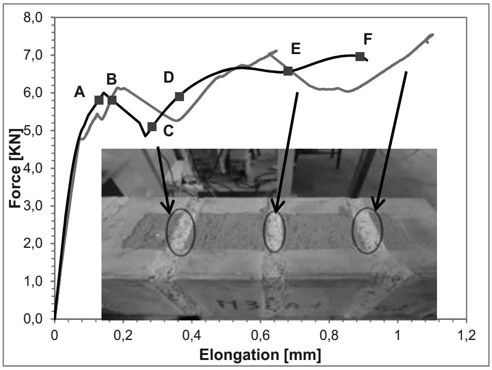


Figure 5. Description of bond-slip laws.

4.6 Line A – Research Unit no. 6

The research activity of this Unit allowed to obtain theoretical and numerical results relative to masonry structures.

Firstly, the Unit elaborated a mechanical model (Figure 6) of the masonry vaults based on the Pucher's approach, identifying some closed-form solutions for some vaults with given geometries. Moreover the Unit investigated and formulated new approaches for generating suitable sub-domains where one may search for equilibrated and (approximately) compatible solutions.

Other results concern the problem of static analysis of three-dimensional masonry buildings. The approach proposed by the Unit is based on the no tension (NT) assumption of its wall elements. The novelty of the method mainly consists of performing a full plastic-holonomic analysis of the structure, which is decomposed in its structural elements, starting from the treatment of the single masonry panel using the NT theory. The proposed approach allows a more exhaustive representation of its behaviour with respect to the classical POR approach, as well as the inclusion of architraves, tendons and other reinforcements in the walls.

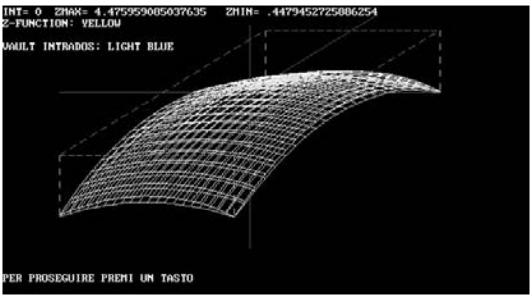


Figure 6. Mechanical model of masonry vaults.

4.7 Line A -Research Unit no. 7

The Unit gave interesting contributions on the strengthening of curved masonry structures (Figure 7). In particular, the Unit formulated several proposals of interventions by using both traditional and non-traditional techniques. The last ones were developed by using FRP materials. Numerical comparisons were also obtained.

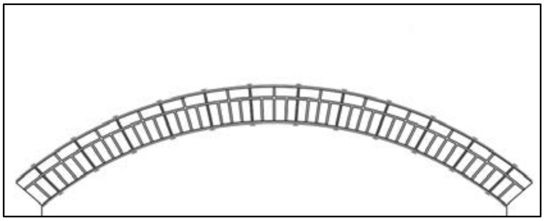


Figure 7. Strengthening of curved masonry structures.

4.8 Line A - Research Unit no. 8

Mechanical tests on a 1:3 scale model of a brick masonry cross vault reinforced with glass fibres reinforced concrete (GFRC) were performed (Figure 8). The mechanical response of the model was schematized using the computer code NOSA which provided results in good agreement with the experimental evidences.

A numerical model was developed for the determination of the collapse load and the collapse mechanism of masonry arches reinforced at the intrados with CFRP. The voussoirs and the reinforcement strips were schematized as rigid blocks. The internal forces acting at the interface were represented by the stress resultants. A feasible domain was defined according to the experimental evidence. The numerical model, developed using an associated plasticity framework, provided predictions in good agreement with the experimental results.

Experimental tests were also carried out on rammed earth arch scale models reinforced at the intrados and the extrados with a jute fabric. All the materials used were completely biocompatible. The arches were loaded asymmetrically at the extrados. The tests showed that the jute fabric reinforcement increases considerably both the collapse load and the ductility of the structure. Numerical simulations were performed using the "concrete model" implemented in the code ADINA. The mechanical parameters were determined through simple compression and three point bending tests carried out on the earth material. The predictions of the numerical model were in good agreement with the experimental data.

Figure 8. Strengthening of a brick masonry vault.

4.9 Line A - Research Unit no. 9

The Unit examined the problem of micromechanical modelling of an interface able to match the evolution of the damage (microcracking), the condition of non-penetration (Signorini condition) and the effect of friction (Coulomb's law). The theoretical analysis was preceded by an extensive experimental investigation (Figure 9). For what concerns the specific topic of the evolution of the damage, the Unit provided a numerical procedure based on the return-mapping algorithm and classical backward-Euler integration scheme. Several numerical investigations have been performed with reference to the case of monotonic and cyclic loads, which highlight the ability of the proposed approach to simulate the behaviour at the interface.

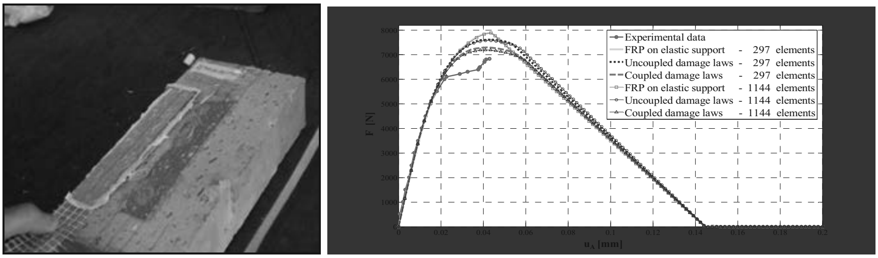


Figure 9. Theoretical and experimental investigation on the interface.

4.10 Line A - Research Unit no.10

During the first two years of research fibre-free techniques for the elasto-plastic analysis has been applied to beam cross sections, endowed with elasto-plastic constitutive law with softening, in order to evaluate analytically the relevant integrals appearing in the expressions of the resultant forces and of the stiffness matrix. In the third year this approach was applied to shear walled reinforced concrete structures.

In particular, a newly formulated shell element has been presented which allows for the definition of nonlinear material properties for modelling both concrete and reinforcements behaviours. Nonlinearity due to the axial-flexural behaviour of the wall is accounted for by adopting the fibre-free approach.

The proposed formulation has been validated by means of comparisons with both numerical and experimental benchmarks and it has been successfully applied to real scale structures as illustrated in the Figure 10.

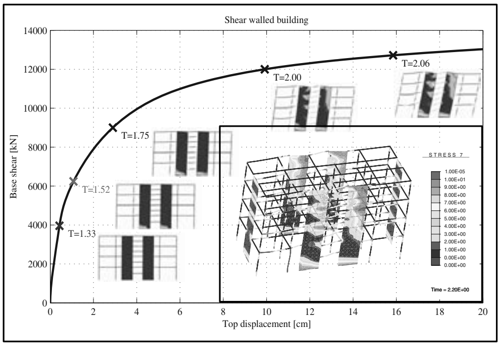


Figure 10. Comparisons with both numerical and experimental benchmarks.

4.11 Line A - Research Unit no. 11

The activities developed by this Unit have led to the prediction of the continuous stress field associated with arbitrary lumped stress-models of masonry structures. Such force networks are associated to polyhedral stress functions defined over arbitrary triangulations of 2D domains. Upon projecting the stress function corresponding to an arbitrary lumped stress network over a structured mesh (Figure 11), a secondary force network with ordered structure is obtained. The latter is employed to formulate a `microscopic' definition of the Cauchy stress of the system. Under suitable regularity assumptions, the convergence of the microscopic stress to its continuum limit has been proven, as the mesh size approaches zero.

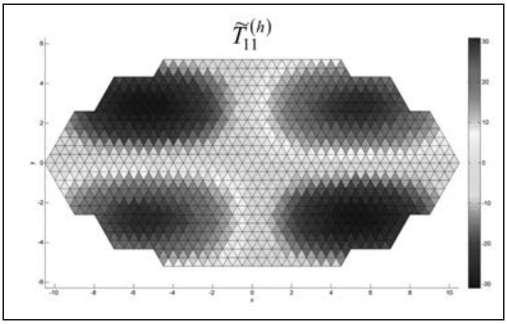


Figure 11. Illustrations of the membrane state of stress associated with a lumped stress model of an elliptical dome.

4.12 Line B - Research Unit no. 12

<u>*Modelling the buckling restraint provided by FRP wrapping.*</u>

The buckling phenomena can be prevented by applying further restraint to slender bars in between the restraints provided by the stirrups, especially if they present large spacing. The mechanical effect of FRP wrapping has been modelled as additional springs, thus increasing the critical load for the bars. A simple yet effective analytical model is provided to account for this beneficial effect.

Compressed longitudinal reinforcement bars were modelled as Euler beams restrained by elastic springs along their length. Springs are representative of FRP wrapping stiffness different in the case of circular and rectangular cross sections. To this aim a confinement model previously proposed is the basis for the evaluation of the stiffness of the confining device accounting for both circular and non-circular concrete cross sections.

The efficiency of FRP wrapping depends also on the position of the bars in the cross section. Two limit positions were considered in the case of rectangular cross sections, namely corner and central bars.

The main results have shown that the effect of FRP wrapping is negligible for L/D ratios less than about 6.5 for both circular and rectangular cross sections, while for higher ratios FRP could effectively enhance buckling load in the case of circular cross sections and of rectangular cross sections, but in this case only for corner bars. For rectangular cross sections the FRP is not able to prevent the central bars buckling, mainly because of reduced thickness of FRP wraps and to this aim both flexural and membrane regimes were analysed for FRP wrapping. The proposed model accounts also for inelastic buckling by means of reduced modulus theory. This model was also validated by means of experimental-theoretical comparisons.

Modelling the bond improvement to lap slices provided by FRP wrapping.

The optimal bond between the reinforcement bars, at the lap splice locations, is crucial for the seismic structural safety and the FRP wrapping is a common practice for the seismic retrofit of RC columns. The effect of FRP wrapping at the lap splice locations induces additional lateral stress, increasing the bond stress of the spliced bars and preventing slippage. These phenomena have been studied by means of refined confinement models in circular and noncircular cross sections. In particular, by means of parametric analyses, two main aspects have been investigated for circular, square and rectangular cross sections.
The first is the proper anchorage length to prevent the reinforcement bars slippage. The proper anchorage length was evaluated as a function of the confining stress due to FRP wrapping, in reinforced concrete cross sections, and it was provided for many bar diameters.
The second is the working stress in the steel reinforcement bars. It is increased by FRP wrapping, at the lap splice location, and it has been investigated in both the cases of uncracked and cracked concrete covers. In the case of cracked concrete cover the overall trends are the same but, as expected, a global worsening of the performances is noticed. Once the FRP stiffness is defined, the greater is the anchorage length, the greater the working stress becomes.
In the case of noncircular cross sections the aspect ratio has a clear influence on the effectiveness of the FRP wrapping. In general the higher is the aspect ratio, the worse the performance becomes. Many parameters influencing these phenomena has been considered, namely the anchorage length, the FRP strain, the bars diameters, and the FRP stiffness. Theoretical and parametric analyses have shown that the FRP wrapping could significantly enhance the steel performance at lap splice locations.

Experimental tests on corner joints under constant axial load and transverse cyclic loading in the as-built and FRP strengthened configuration

The experimental work involved tests on six non-conforming full scale RC beam-column joints in the as-built and FRP strengthened configuration. The experimental results allowed to assess the reliability of available capacity models on as-built poorly detailed joints and to quantify the benefits provided by different FRP strengthening layouts.

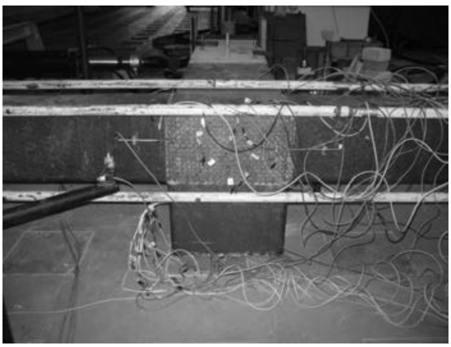


Figure 12. Experimental test on beam-column joints strengthened with FRP systems.

The experimental tests on FRP strengthened specimens showed the effectiveness of the proposed strengthening solutions for the seismic retrofit of poorly detailed RC beam-column joints. In particular, they showed that:

- the FRP strengthening layouts were able to significantly increase the subassembly strength capacity; the strength increase depended on substrate mechanical properties, number of FRP layers and anchoring solution;
- the use of 4.0$\mu\varepsilon$ as design maximum strain for FRP retrofit of beam-column joint seems to be too conservative according to the presented experimental results; the maximum strains recorded on FRP joint strengthening panels at specimens peak strength capacity were in each case larger than 4.0$\mu\varepsilon$, with a maximum value of 10.2 $\mu\varepsilon$;
- the amount of FRP joint panel reinforcement strongly affects the joint panel deformations that could significantly influence the subassembly mechanical behaviour; to reduce joint flexibility it could be necessary to limit the effective FRP strain to values lower than 10$\mu\varepsilon$;
- the use of a proper FRP joint panel anchorage solution made by a U-shaped uniaxial sheet also wrapped around the beam top side avoided the FRP end full debonding;
- a proper amount of joint panel FRP fibres combined with a stronger anchoring system reduced joint panel shear deformation leading to significant strength enhancement. This resulted in a subassembly seismic capacity improvement and energy dissipation increase associated to a more favourable ductile failure mode;
- the FRP strengthening provided no changes in the subassembly initial stiffness while a more gradually stiffness degradation was observed after joint panel first cracking.

The experimental program results pointed out the potential of FRP strengthening solution to increase the seismic performances of poorly detailed beam-column joints; the experimental validation of the recommended FRP solution may strongly encourage the use of FRP to considerably reduce the seismic vulnerability of existing RC buildings. The collection of further experimental data in terms of FRP joint panel effective strains is crucial to calibrate reliable and simple design expressions to determine the minimum reinforcement amount required to avoid the joint shear brittle failure mode in case of seismic actions.

4.13 Line B- Research Unit no. 13

For what concerns the analysis of pre-stressed FRPs, a device to effectively seize and prestress FRP fabrics was optimized in order to minimize the weight and the time of intervention. The main advantage of the seizing tensioning device is that it may also allow differential pre-stressing of the fabric along the beam axis, thus inhibiting the risk of debonding at the free ends of the fabric, if this is not mechanically restrained.

The device consists of a cylinder of appropriate diameter on which the fabric can be wrapped around without the need of resin and of seizing devices. This cylinder is mounted on a moving sledge that can slide into another fixed sledge mounted directly on the beam soffit. The cylinder can rotate on its axis, in order to wrap around the amount of fabric necessary to properly restrain it by simple friction and put it in a slight tensional state.

Furthermore, for what concerns the analysis of NSM elements (Figure 13), the theoretical aspects of the problem were examined and a formula for practitioners was proposed.

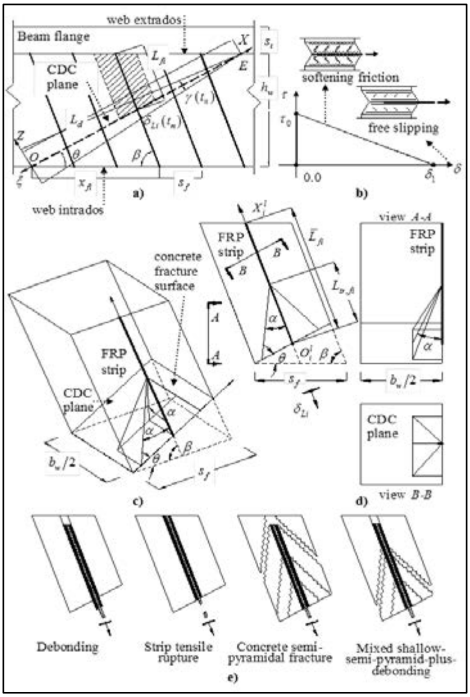


Figure 13. NSM elements.

4.14 Line C - Research Unit no. 14

The results obtained by this Unit refer to the following topics:

Analysis of the experimental results from direct shear tests and comparison with numerical simulations: The performed tests have highlighted that the use of a washer significantly improves the FRP-concrete interfacial behaviour.

A trilinear constitutive law of the connection was calibrated on experimental basis, specifically identified for fasteners equipped or not with washer. Then, a 1D finite element model has been formulated for simulating the experimental behaviour of the MF-FRP connections which allowed for the validation of the interfacial behaviour previously identified.

Development of a FE model for simulating the flexural behaviour of RC members strengthened with MF-FRP systems and comparison between the numerical simulations and the experimental results: The FE model (Figure 14) has been formulated by assembling three components, i.e.: a) an "Euler-Bernoulli" beam finite element for the RC member; a rod element for the FRP strip; c) two spring elements for connecting the relative displacements at concrete-FRP interface.

In order to validate the numerical model, a wide database of experimental tests on MF-FRP strengthened RC beams has been collected from the literature. The experimental-numerical comparisons have allowed both verifying the accuracy of the implemented numerical modelling and investigating the influence of some parameters on the flexural response of the simulated members, such as: the force-slip constitutive law adopted for simulating the FRP-concrete interface behaviour, and the tension-stiffening effect which is generally neglected in the numerical models.

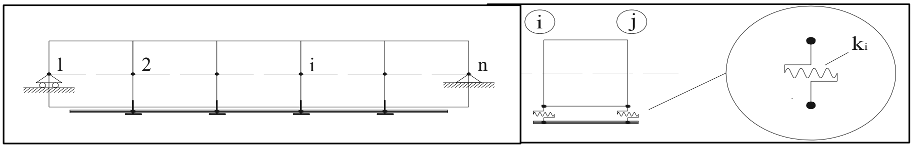


Figure 14. MF-FRP FEM results.

4.15 Line C - Research Unit no. 15

Twenty-four building columns made with calcareous stone have been built. The thickness of the mortar joints was 10 mm. The specimens have been confined with a layer of unidirectional FRP or two layers of basalt fibre grid; the composite have been glued to the masonry core by using two different epoxy systems, one traditional, the other with a different resin agent due to the thixotropic properties required for the activation of Shape Memory Alloys (SMA). The SMA has been employed as incorporated innovative device in the GFRP reinforcement to activate confinement before the mechanical forces of compression could produce the lateral expansion of the column. All the specimens have been tested applying a load of axial compression by a hydraulic jack from 150-ton inside a steel frame. In all the tests two LVDTs have been used for measuring the shortening of the columns under load. LVDT has also been set on the steel frame supporting beams to monitor possible settlements of the frame. The load has been recorded through a 200-ton load cell, six electric strain gauges have been applied on the GFRP and BFRP in the direction of the transversal fibres, in different positions: to 100, 300 and 500 mm beginning from the inferior base of the columns. The load, the deformations and the axial displacement have been all recorded by an electronic system of data acquisition. All the tests have been conducted under the same environment with the purpose to minimize the effects due to temperature and humidity values. Figure 15 illustrates the position of the strain gauges that were paced also along the overlapping region that was 200 mm long, equal to the 24% of the total length of the FRP confining sheet. All the specimens have been strengthened through the technique of the manual wet lay-up, through application of an epoxy primer prior to the fibre bonding.

Figure 15. Confinement of calcareous stone columns.

4.16 Line C - Research Unit no. 16

Several results of tensile tests were obtained. It was possible to note that flax is the material with higher mechanical properties, followed by jute and by sisal. Fabrics impregnated with epoxy resin (NFRP) presented greater tensile strength than polyester resin, so consequently it can be stated that the epoxy resin is most suitable from the mechanical point of view as a matrix of the natural fibre composite materials tested. Figure 16 shows the different levels of strength of the natural fibre composite examined with the different matrices used. Regarding the results obtained from three point bending tests, they demonstrated that the reinforced bricks are more resistant when compared to unreinforced bricks (f_f=800kPa), as expected. Indeed, it was possible to note that the reinforced bricks are characterized by an increment of flexural resistance of almost 38% with flax (f_f=1050kPa) and 32% with hemp (f_f=980kPa). About the results of pull-out tests, they indicated that the strength is practically independent of the fibre: in fact it can be observed the same value of tensile bond strength (f_t=1200kPa). Finally, from the tests of single-lap shear bond tests, it was possible to distinguish four types of fracture: the most widespread was that of cohesive kind in the mortar, with a percentage of 61%, taking into account that for each type of material were tested a number of 6 specimens each.

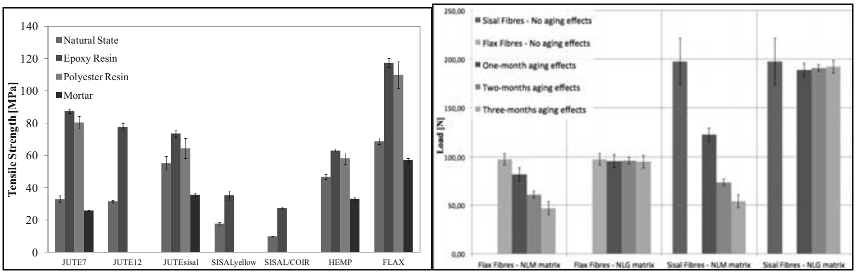


Figure 16. Different levels of strength of the natural fibre composite examined.

4.17 Line D – Research Unit no 17

The results obtained by this Unit concern the following topics:

Behaviour of the web-flange junctions of the majority of commercially-produced pultruded composite profiles.

A total of 28 pull-out tests have been performed on the web-flange junctions of two different sizes of PFRP pultruded I-profiles in order to evaluate both the axial stiffness and strength of such junctions (Figure 18).

Full-scale experimental results indicated that the junction strength depends on the location of the applied pull-out force. In particular, the values of the failure load obtained from the end-point pull out tests were about one-third smaller than those obtained from the mid-point tests for all the specimens evaluated in this phase. In addition, experimental results also showed that the load-displacement (P-δ) curves of Web-Flange Junctions (WFJ) are influenced by the size of the PFRP profiles; for the I_160_EP and I_160_MP specimens the load-displacement curves present a linear behaviour until the elastic limit after which a non-linear behaviour and a stiffness degradation is observed up to the failure load. Whereas, for the I_200_EP and I_200_MP specimens an increasing linear displacement was observed until ultimate failure.

Figure 17. Analysis of web-flange junctions.

Adhesive joints.

The experimental setup developed was composed of a tensile test machine designed and built at the Official Laboratory Testing Materials and Structures, Department of Civil Engineering, University of Salerno. The experimental results thus obtained are helpful for the comparisons with multiple numerical results already obtained. Among the latter the most recent ones are relative to single-lap joints between adherents with generic curved axis. Such numerical experiments, concerning adherents made of FRP, as the reinforcement, and concrete/masonry as the support, have highlighted that a finite curvature radius is generally beneficial, in the sense of a delay in the energy absorption as the curvature radius decreases. The above mentioned beneficial effect vanishes near collapse. Finally, the Unit has also investigated the influence of all the parameters on the effective stress transfer length. In particular, this length can be evaluated assuming two sub-lengths: the length of the cohesive zone, which is not-dependent on the fracture energy of the interface, and a second one, which represents the elastic attenuation length and can be predicted by using a well-known closed form formula.
Regarding the estimation of the length of the cohesion zone a project formula has been identified.

4.18 Line D - Research Unit no. 18

The results obtained by this Unit concern the following topics.

Modelling and numerical analysis of composite functionally graded materials.

The closed-form relationships obtained allow us to describe the constitutive behaviour of the equivalent material at the macroscale, enabling us to plan strategies for reducing shear stress concentrations at the fibre-matrix interface, by the identification both of appropriate constitutive laws of gradation and geometric features at the microscale (Figure 18).

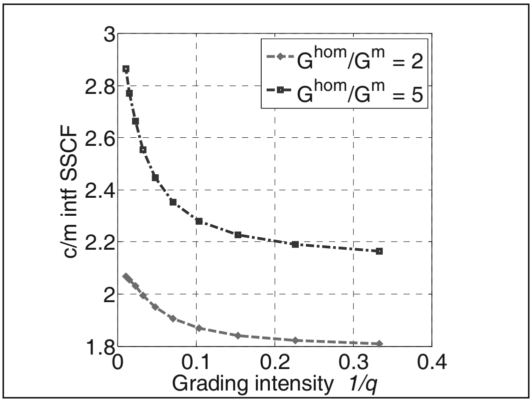


Figure 18. Composite functionally graded materials.

Modelling and analysis of bolted joints between structural elements in composite.

Continuing the work initiated in the first two years, generalized three-dimensional numerical models based on finite element formulations have been developed, that allow us to analyse the performance of the junction, taking into account the influence of both unilateral contact conditions (also with friction) in the bolt-plate coupling, and geometric and constitutive non-linearity. In this context, early indications, useful in design practice, on load transfer mechanisms (i.e., loading partition) have been obtained, considering as parameters the joint geometry and the constitutive properties of the joined elements.

Modelling and analysis of structural elements made of composite materials; conception, design and analysis of structural modules for lightweight and provisional structures.

Starting from a constrained variational approach, theories of beams and plates in composite materials with high anisotropic behaviour have been deduced, by considering different accuracy levels for describing shear deformability effects and with specific reference to thin-walled pultruded beams and laminated plates. Models that have been developed allow to account for unilateral effects related to the bimodular material behaviour as well as to Signorini type contact conditions. Moreover, by adopting a semi-analytical homogenization approach, based on a perturbation technique applied to the classical Hashin problem, closed-form relationships have been deduced, allowing us to characterize the equivalent elastic properties of fibre-reinforced composite materials with wavy fibres.

Modelling and analysis of the dynamic response of composite modular structures; analysis of advanced strategies and devices for vibration control.

With parallel and synergistic activities to those described above, the problem of the dynamic characterization of light structures, such as those based on a "all composite" conception, has been addressed. In particular, their optimal vibration damping has been investigated, considering both control systems based on tuned masses, and the use of smart devices involving shape memory alloys and/or of composite piezoelectric materials.

4.19 Line C - Research Unit no. 19

The analysis pointed out that, for wide-flange pultruded columns, the experimental imperfection amplitudes given in the literature yield to well documented failure mechanisms, such as the delamination of the web-flange junctions, and a small influence of the buckling-mode interaction. Moreover, for slenderness values typical of members of truss structures, a narrow-flange profile in compression may perform better than a wide-flange profile of equal cross-section area (Figure 19). Finally, for beams bent in the major-axis plane, the interaction between the flexural-torsional buckling and the local buckling of the compression flange may become significant (Figure 19).

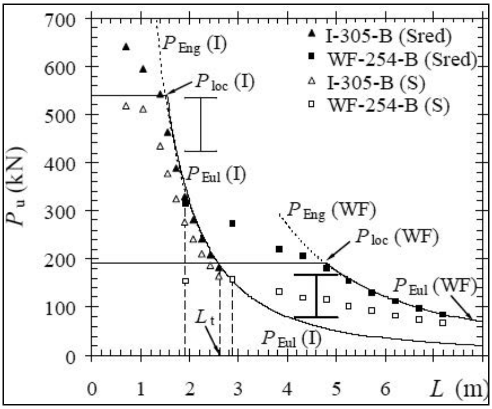


Figure 19. Members of truss structures.

Furthermore, a new 4-point bending test configuration, giving more reliable estimates of the shear modulus with respect to the traditional bending tests, was proposed.

4.20 Line D - Research Unit no. 20

Several results on High Performance Fibre Reinforced Concrete Double Edge Wedge Splitting tests (HPFRC DEWS) without damage (Figure 20), on Textile Reinforced Concrete (TRC) in serviceability conditions and subjected to exceptional loading conditions were obtained. They are summarized in the papers published by the Unit.

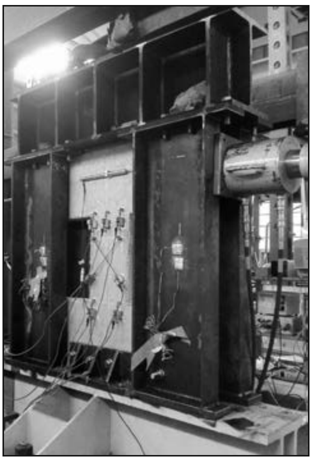


Figure 20. HPFRC DEWS tests without damage.

4.21 Line D - Research Unit no. 21

A total of four full scale solid brick walls (Fig. 21) were tested, i.e. one un-strengthened (MW1) wall, two strengthened walls (MW2-ST and MW3-ST) and a pre-damaged wall repaired with the same coating (MW1-R).

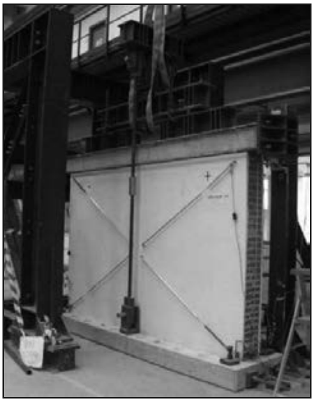


Figure 21. Full scale solid brick walls.

The experimental results obtained in this research yielded the following observations: all the three strengthened walls reached the maximum capacity and rocking appeared with an ultimate strength 23% - 35% higher than the one exhibited by the URM wall; compared to that, the strengthened specimens presented an initial load-deflection response characterized by a considerably higher lateral stiffness and a significantly postponed cracking stage; the lateral stiffness improvement shown by the repaired wall is actually comparable to the performance exhibited by the not pre-damaged specimens; the two tested connection systems have shown different performances in terms of shear stiffness provided to the masonry-to-coating interface.

Three push-pull tests were performed on URM specimens with SFRM overlays, i.e. one without connecting dowels, one with ϕ 6 self-tapping connection steel dowels with steel anchor plates (dim. 25 mm x 25 mm) and one with ϕ 6 self-tapping connection steel dowels with anchor steel plates (dim. 50 mm x 50 mm). The tests proved that, compared to the specimen without dowels, the use of a steel connection with an appropriate anchor plate allows a significant improvement of the masonry-to-coating interface shear capacity and leads to a more gradual detachment of the SFRM layers at the ultimate conditions.

Uniaxial compression cyclic tests were performed on small wallets made of the same solid brick URM used to build the full-scale walls tested in this research. The tests provided useful experimental data that were also used to validate the compression model implemented in the Disturbed Stress Field Model (DSFM).

The DSFM was improved by including the post-cracking softening behaviour of mortar joints. A routine was developed and implemented in the DSFM to simulate the compressive behaviour of URM subjected to reverse cyclic loading.

4.22 Line D – Research Unit no. 22

The activity has been devoted to the evaluation of the non-linear behaviour (in static and dynamic field) of framed tri-dimensional structures (Figure 22), reinforced with HPFRC jackets. The numerical analyses (push-over and non-linear dynamic) were performed by adopting the procedure of "equivalent section" developed during the second year of the research.

The final results can be summarised as follows:

- validation of the procedure of the "equivalent section";
- evaluation of the effectiveness of the technique for retrofitting in seismic zones;
- evaluation of the influence of the HPFRC strengthening on the global strength and on the "behaviour factor".

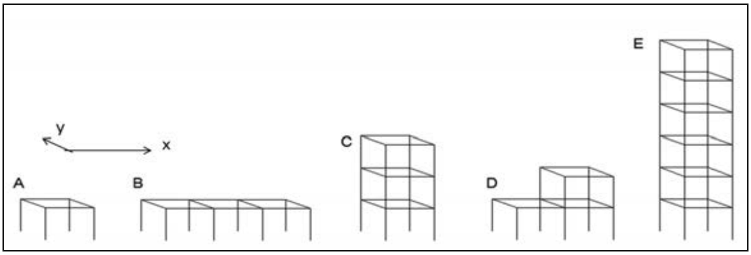


Figure 22. Framed tri-dimensional structures.

4.23 Line D - Research Unit 23

Tests on three full-scale beam-column corner joints specimens retrofitted with the metal insert were performed, in order to evaluate the effectiveness of a technique alternative and complementary to the HPFRC jacketing solution, with the aim to solve some problems of the previously studied technique which have been come to light during the tests of the second year and connected to detachment phenomena between the HPFRC and the base concrete.

The design of the retrofitting intervention has been performed on the base of the results of detailed numerical analyses, which have allowed to highlight some aspects, among which the most important one is probably the mistake that a designer makes if he assumes perfect bond between the reinforcement steel and the surrounding concrete in the case of smooth bars, assumption which may lead to a ductile flexural mechanism rather than a brittle shear failure, with a non-conservative assessment of the joint seismic response.

The experimental results have shown in all cases that the proposed retrofitting solution is effectively able to change the failure mechanism from a brittle shear failure in the joint panel to a ductile failure mode with formation of a flexural hinge in the beam.

The solution with metal insert and only cover-concrete restoration has been effective in moving the failure from the joint to the beam. However the increase in strength with respect to the unretrofitted solution, equal to 20% for moment closing the joint, has been relatively low in the case of moment opening the joint. Better results have been achieved in terms of ductility increment.

The best solution, in terms of both strength and ductility, is the one that couples the two retrofitting techniques: metal insert and HPFRC jacketing of the structural member converging in the joint. The increase in strength has been estimated at around 45% for moments closing the joint and 40% for moments opening it, with a ductility doubled with respect to the unretrofitted joints, a more stable behaviour also for high drifts value (6%) and the moving of the failure mechanism from the joint into the beam and no evident damage of the panel region.

4.24 Line E - Research Unit no. 24

The mechanical behaviour of laminated glass has been analysed (Figure 23), evaluating more precisely the flexural coupling of glass plies through the shear stiffness of the interlayer. The latter has a viscoelastic behaviour, which is usually treated as linear elastic but with equivalent elastic moduli that take into account duration of loads and temperature. This is the so-called "quasi-elastic" approximation, for which a new simplified method has been proposed, much more accurate than previous formulations. The method is versatile because it is applicable to laminated plates and multilayer packages. Tables for easy reference have been drafted to facilitate the design; they have been applied to various case studies, demonstrating their efficiency and practicality. The method has been compared with other formulations implemented in standards, some of which are deemed erroneous, demonstrating its superior reliability.
Regarding the post-breakage behaviour of the material, using the photoelastic properties of the polymer, a series of tests were performed to measure delamination of the interlayer from the glass support. The results will serve as a basis for extending the methods of analysis to the post-breakage phase of laminated panels, also considering possible mechanisms of crack shielding.

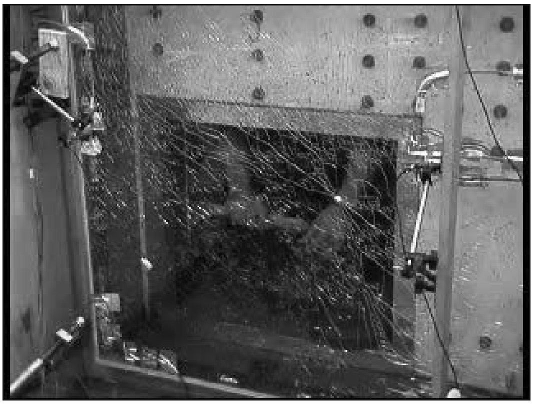


Figure 23. Laminated glass.

5 VISIONS AND DEVELOPMENTS

A theme so peculiar and absorbing, such as the seismic consolidation of existing buildings, is naturally associated with the use of innovative materials, in order to achieve the best results with non-invasive interventions.
This possibility is offered through the use of innovative materials and techniques, made available by modern engineering: from the material engineering to that which specifically deals with rehabilitation interventions and structural modelling.
The ReLUIS project 2010-13 has given priority and space to the topic of innovative materials used for seismic consolidation, providing answers to multiple requests coming from the construction industry, practitioners and contractors and in particular, the requests with specific interest for Civil Protection.

It is an unquestionable fact that the use of innovative materials has played a major role in reconstruction processes following the recent earthquakes which have affected the Italian territory, from North to South, recalling the earthquakes in Aquila and Emilia.
However, not only, the use of innovative materials in structural rehabilitation, and more generally in Civil Engineering, has been gradually growing in the last two decades. This is highlighted by the numerous International Conferences and Journals dedicated to such topics.
The results reached by Task 3.1 within the ReLUIS Project 2010-13 have allowed to validate the effectiveness of various structural innovative materials as well as to select the most appropriate interventions for each of them.
The studies carried out over the last years range from the problem of qualification, a primary requirement for any material to be used for structural purposes, as expressly required by the Italian Normative, to structural modelling, in order to provide design formulas and simulations of the mechanical behaviour of structures which have been retrofitted by using innovative materials.
It should therefore be noted that most of the results have allowed to develop and/or to update the national guidelines on the use of innovative materials and techniques.

Contribution to FRP materials

It is immediately worth considering the updated R1 version of the CNR DT 200, carried out in 2013, i.e. the Instructions on the use of FRP materials for the consolidation of reinforced concrete structures and masonry structures. On the basis of much of the research developed by the Task 3.1, it was possible to obtain more reliable values of correction coefficients present in the various design formulas. In fact, they were estimated on the basis of statistics developed on much larger experimental populations than those taken as a reference in the first edition of 2004.
Furthermore, thanks to Task 3.1, the experience gained in the field of FRP qualification provided a further boost in drawing up a *Guideline for the Qualification and Control of acceptance of fibre-reinforced polymer composites to be used for the structural consolidation of existing constructions*. The working group appointed by the Italian Ministry of Infrastructure and Transportation (MIT) consisted of several researchers involved in the ReLUIS project and many articles of the guideline were inspired by the experience gained in the project.
However, the ReLUIS project has not only helped to improve the structural knowledge of FRPs, important cognitive contributions have also been made to other innovative materials, such as FRCM (Fibre Reinforced Cementitious Matrix) and HPFRC (High Performance Fibre Reinforced Concrete).

Contribution to FRCM materials

It is well-known that the use of FRCM (*Fabric-Reinforced Cementitious Matrix*) is also gaining popularity. These composites are obtained by matching a fibre mesh with an inorganic matrix composed of cement or lime mortar. Generally, the mesh is composed of fibres in either a dry state or impregnated in polymer resins. The rapid diffusion of FRCM materials is due to their better properties compared to FRP materials: they have good mechanical performances, excellent resistance to high temperatures and fire, good vapour permeability and can be applied on wet surfaces. Strengthening existing buildings with FRCM has become increasingly frequent, especially for the repair of earthquake damaged buildings, such as the recent experiences in Abruzzi and Emilia regions have shown.

In spite of the increasing use, a completely shared qualification procedure of these composites has yet to be identified. At the same time, the mechanical modelling of structures reinforced with these materials requires further studies. The results available in current literature, and in particular those achieved in the ReLUIS project, still do not allow for a fully conscious use of these materials, a situation that contrasts with the extensive use that is being made of them.

Contributions to HPFRC materials

Another innovative material that has attracted the attention of researchers, practitioners and contractors for the strengthening and retrofitting of existing R.C. structures in seismic zone is high performance fibre reinforced concrete (HPFRC). By varying the concrete mix and fibre typologies it is possible to obtain a material fulfilling the required performance. In particular, the properties required of HPFRCs are a hardening behaviour in tension and a significant strength both in compression and tension.

In current literature, there are several studies devoted to the design of new structures realized with the use of HPFRC. Recently, the use of these materials for the strengthening of existing structures is becoming more and more widespread. The cases of Abruzzi and Emilia regions are a clear example.

Task 3.1 has given several contributions to these types of application. Nevertheless, this field, characterized by a highly attractive potential, needs further study, especially with full scale experimental tests. It is desirable that in the future, it will be possible to find adequate funding to provide answers to the unanswered questions.

Contributions to full composite structures

Recent reports show the increase in national sensitization to seismic events. These unexpected events can produce great discomfort for people as well as possible large structural and infrastructural damage. In order to reduce their effects, a fundamental task for the Civil Protection Service consists in the fast realization of provisional structures in order to restore primary services (e.g. viability of the main roads, accommodation, etc.), to manage relief operations in emergency and difficult situations, as well as related to the rehabilitation and restoration of existing structures. In these cases, light structures (for crossings, roofing, etc.), designed in accordance with modular concepts in order to allow easy moving (by land, air or sea) and easy assembly in situ, as well as flexible arrangement and utilization, but at the same time exhibiting suitable strength and durability, have to be necessarily employed.

The materials traditionally used in the field of the civil engineering, such as concrete and steel, due to the specific requirements in terms of weight and size, are not suitable for modular structures that have to be realized in a short time and under difficult conditions. On the other hand, due to their stiffness/weight and strength/weight high ratios, as well as their great resistance to oxidation/corrosion, fibre-reinforced composite materials (FRP) may be considered particularly attractive for these applications. In a civil engineering context, such an application field represents a natural extension of those already existing. In recent years, FRP elements have been used mainly for the structural rehabilitation and adjustment of frameworks made of concrete or mortar. On the contrary, the number of new civil constructions entirely composed of FRP is almost limited. Therefore, the use of these materials for realizing “all composite” modular structures can be undoubtedly considered as an innovative and unconventional development.

Task 3.1 has helped to clarify some aspects of the problem. More questions are still waiting to be answered and it is therefore desirable that the research in this area continues, especially in relation to the possible consequences to provisional structures.

Contributions to glass structures

Observation of the damage caused during recent earthquakes has highlighted critical elements that, while not participating to the gross structural stability, nevertheless may cause severe damage to people and property after collapse. These are generally “non-structural” elements, unfortunately not sufficiently well analysed with verification calculations, but at which the utmost attention must be paid. Among these, it is without doubt worth considering glazing systems, which are particularly critical since glass is a brittle material *par excellence* and, in case of an earthquake, it does not allow for any form of plastic adaptation as other building materials. In existing buildings, it is necessary to establish criteria for the replacement or adjustment of old glazing systems with more performing ones. In new constructions, large pieces of glass are used for which specific criteria need to be established for their fail-safe design against seismic action.

Many of the obtained results have been used in standardization. Among these, it is worth recalling the technical instructions on structural glass CNR-DT 210 and the guidelines for the preparation of the new Eurocode on glass.

It is desirable that in the near future, the research in this area can be further developed with reference to the different structural types in use, in order to offer a wide series of examples and, at the same time, diffuse the knowledge of the most suitable methods of structural analysis for the design and verification.

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DEVELOPMENT AND ANALYSIS OF NEW TECHNOLOGIES FOR SEISMIC UPGRADING

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1 INTRODUCTION

Seismic isolation and energy dissipation technologies, although today mature from the scientific point of view and of sure efficacy, require further deepening in the perspective of their possible applications on a large scale in order to reduce seismic risk of constructions in Italy, and of reduction of costs. All these aspects have been confirmed by the experience of the last Italian earthquakes. Particularly with reference to the design of seismic retrofit and improvement interventions on existing structures, it should be important to know when seismic isolation is more convenient in safety and economic perspectives, and which technological solution is more appropriate.

As regards energy dissipation systems, to which codes refer in a less clear way, the delivery of guidelines for the design of such systems is necessary, also on the base of the results of the research developed within Line 7 of previous DPC-ReLUIS 2005-2008 project.

Furthermore, although semi-active control systems cannot be considered for widespread applications, due to their complexity and to very low probability of events able to activate them during life of construction, they could be more useful in structures equipped with seismic monitoring and/or early warning systems.

Italy surely holds an important record in the development and manufacturing of structural bearings for bridges and other civil applications on a worldwide scale. Moreover, our Country is on the cutting of edge in the seismic isolation field exporting rubber-steel isolators all over the world. High costs and weight characterize all steel laminated rubber bearings. Taking into account that the most of seismic area are in developing countries, these aspects cannot be disregarded.

The recent realizations triggered the development of new technologies for structural isolators such as friction pendulum systems (FPS) with one, two or three sliding surfaces; in this framework emerged the needs for new design tools and procedures to allow the spreading of base isolation in the common practice. More recently, other research ideas have been developed, as the one of elastomeric bearings in which the elastomer layers are made of recycled rubber and the steel reinforcement is substituted by fiber layers: this innovative bearings are able to reduce both the high cost and weight characterizing the traditional steel-laminated rubber bearings.

The effectiveness of a base isolation system (BIS) depends on its filtering capacity over the range of frequencies where seismic energy is larger. However, filtering action of a BIS has, on occasions, to be applied to an unpredictable excitation having random dynamic characteristics: it is known that even when detailed data from previous seismic events are available, the hazard should be derived from more than one seismic source, and it is impossible to define a single earthquake scenario that is compatible with the results of probabilistic seismic hazard assessment. Therefore, it is clear that the first natural frequency can never shift out of the “possible” energy content frequency range for any type of seismic excitation. The new idea of a hybrid system, based on a combination of the tuned mass damper strategy and base isolation (TMD + BIS), came from the observation that the response of properly designed isolated systems is dominated by the first-modal contribution and that TMD is able to reduce solely that fundamental vibration mode. The objective is to protect the BIS from those excitation components, which are close to the natural vibration frequency, by controlling the amplitude of the fundamental modal contribution due to the TMD action installed on the base isolation layer.

2 BACKGROUND AND MOTIVATION

Today the base isolation and the energy dissipation strategies lead to very attractive possibilities to favorable control the seismic behavior of structures. However, how it is well known, both these innovative protection strategies could still have difficulties to guarantee the right effectiveness and robustness due to the lack of knowledge of input signals, for the case of base isolation, or due to the lack of design procedure to optimally allocate the damping resources in the case of framed structures, particularly for the case of irregularity. The main motivations of the research of Task 2.3.2 of DPC-ReLUIS 2010-2013 project are:

- The need for the implementation of clear and reliable design tools for base isolation systems applied to new and existing (retrofit) structures (both buildings and bridges).
- The still incomplete knowledge about ultimate limit states of the High Damping Rubber Bearing (HDRB) devices for base isolation: design codes only give general design criteria, not accounting for all possible failure modes and local effects that affect the collapse.
- The need to detect the perspectives and the limits of the new technologies, such as FPS devices, that, despite the great number of experimental tests performed, have not yet been tested during a real earthquake, i.e. there are still unknowns about their behavior in limit conditions (collapse, uplift, excessive heating of sliding surfaces, etc.).
- The necessity to study the properties of self-lubricating materials for the sliding surfaces: large values of friction provide high damping capacity, but also high lateral stiffness, and promote a huge increase in temperature at the sliding interface, which in turn affects the properties of the friction materials.
- The idea of reducing both the high cost and weight characterizing the traditional steel-laminated rubber bearings, by developing new elastomeric bearings in which the elastomer layers are made of recycled rubber and the steel reinforcement is substituted by fiber layers.
- The necessity of harmonizing the different procedures aimed at the design of seismic control systems based on the use of passive devices, in order to define simple and shared design criteria that can be suggested to professionals when approaching to such kind of technologies.
- The consideration that semi-active control systems, at the current state of knowledge, involve a relatively high complexity in their implementation and management: although this complexity seems to represent today a strong limitation to the practical use of such technology for civil construction, these systems may however be of interest for structures already equipped with seismic monitoring and/or early warning systems.

3 RESEARCH STRUCTURE

The main objectives of the research within Task 2.3.2 consist of outlining:

1) The study of the response of seismically isolated buildings featuring an inelastic behaviour of the superstructure, in order to assess the applicability and effectiveness of seismic isolation for the seismic improvement of existing buildings.
2) The evaluation of the effectiveness of devices for the protection of existing buildings and precast systems, from both structural and economic points of view.
3) The definition of suitable techniques for the insertion of devices on existing buildings and precast systems.
4) The study of robustness and effectiveness of the combined strategy TMD + BIS.
5) The delivery of design technical indications to retrofit or to improve the behaviour of existing non-ductile reinforced concrete (RC) buildings by means of dissipative systems such as buckling restrained braces and HDR-based devices: preliminary design criteria, reliability of simplified analysis methods (behaviour factors, equivalent damping) and feasibility problems (detailing and interaction with non-structural elements, positioning within the existing structure).
6) The development of simplified formulas (in terms of reduction factors for the earthquake forces) for the seismic design of structures which exploit the combined effects of viscous and hysteretic dissipation, as provided by dampers and by post-yielding behaviour of the structural members, respectively.
7) The creation of a database of properties of current sliding materials and the delineation of the relationship between their friction and physical/mechanical properties.
8) The development and characterization of novel frictional materials with optimal properties for use in seismic isolation systems.
9) The theoretical and experimental study of the combined use of three different technologies, having some shared aspects and tools involved: the semi-active control, the structural monitoring, and the seismic early warning. The aim is the accurate identification of sensors and electronics needed for semi-active control, monitoring and seismic early warning systems, the investigation on the possible use of components shared by two or all the above systems. A laboratory testing campaign will be designed in order to experimentally highlight the feasibility and effectiveness of the proposed solutions, also with a view for optimizing performance and costs, and to measure these benefits in terms of robustness with respect to the protection that can be offered by purely passive systems.

In order to better organize the work considering the different activities and objectives of the research, Task 2.3.2 has been organized in four Activities, and two coordinators have been nominated for each Activity (Activity 3 has been included in Activity 1 after the discussion). The organization of the Task in Activities allows to significantly improve the coordination among Research Units on specific topics and to more efficaciously develop joint experimental activities.

<u>Activity 1. Seismic retrofit and improvement of existing buildings through seismic isolation (includes bridge piers with seismic isolation).</u>

The activity plans the study of standardized operational methodologies for the insertion of devices at the base of existing buildings paying a special attention to masonry buildings. It also concerns the pointing out of the recurrent work typologies for inserting devices. For these typologies, the operational successions are being developed with a particular focus on the constructive aspects of each phase: access to the sub-structure, preliminary works, strengthening of the sub-structure, performing the super-structure detachment, creating the spaces for the devices, installation of devices, successive works, final solution of continuity, collateral problematics.

The activity also includes the development of simplified methodologies for defining the optimum isolation thresholds, as a function of the usable devices and of the collapse mechanisms of the super-structure. Evaluations of the economic compatibility of the seismic improvement through isolation are also carried out.

Activity 2. Development of new isolating devices, also low-cost.

The activity includes the study of cost-effective devices that can be inserted within the connection joints of the precast constructive systems: economic, constructive as well as performance aspects are being considered at the same time. The protection systems can be based on friction devices, sliding devices, plastic devices, and other dissipating devices. Their concept and assembly should be simple for containing the costs, for avoiding constructive complications, for allowing adaptation to the pre-existent productive lines, for being characterized by an economic compatibility of the whole project. The goals of the application of protection systems to the precast systems are: the increase of the seismic capacity of the earthquake-resistant system, to make compatible the use of non-optimal construction systems in earthquake prone areas, and the reduction of the construction cost of the elements for compensating the cost of the devices.

For all the proposed innovative devices, the program of the activity consists on the definition of the geometry and the best materials to develop the isolators, the planning of the testing activity on prototypes, the theoretical study of defined prototypes, and the experimental activity on prototypes.

The activity is also focused on testing and characterization of the sliding properties of current self-lubricating materials, development of a database of properties and formulation of "structure – property" models, development of frictional materials with optimal friction coefficient and improved load bearing capacity and wear endurance, performance of dynamic tests on real scale isolation systems employing the novel frictional materials.

Activity 3. Manual with guidelines for the design of passive energy dissipation systems.

This activity consists in: selecting some simplified method, among those available in technical literature and/or developed by the Research Units, for the preliminary design of structures retrofitted by elastoplastic and viscoelastic dampers; evaluating the effectiveness of the simplified design method by using a probabilistic methodology accounting for the uncertainties of the earthquake action and of properties of the bare and retrofitted structures; in selecting case studies to develop benchmark applications, and in developing optimal constructive details for the benchmark applications. An applicative manual represents the product of the activity.

The aim of the applicative manual is to guide the professional engineer from the choice of a target reduction in the seismic response of the structural system (with respect to the response of a structure without any additional damping devices), to the identification of the corresponding damping ratio and the mechanical characteristics of the commercially available dampers to be inserted in the building.

Besides, the coupling of viscous and hysteretic dissipation will be investigated. The central issue behind this idea lies in the determination of the correlation between the force reduction factor and the ductility demand, when viscous dampers are located inside the structure. Such correlation has broadly been studied in the past for structures not equipped with additional dampers, and represents the heart of the usual seismic design methodologies. According to the framework of the conventional design procedure (response spectrum analysis with force reduction factor), the purpose of the research work is to provide a proper value of the force reduction factor for structures characterized by high level of damping (e.g. damping ratio equal to 30%). This value should allow to reach the same safety level actually provided by the code for the same structure without dampers. In more detail, the relationship between the force reduction factor and the ductility demand will be investigated in order to understand how it is influenced (i) by the presence of additional viscous dampers and (ii) by high values of damping ratio.

Activity 4. Integration among semi-active control, monitoring and early warning systems.

In the framework of "*seismic isolation of bridges*", this activity consists in investigating the addition of semi-active devices to a seismic isolated bridge, and then defining a standard method to evaluate the amount of supplemental damping given by variable dampers installed between piles and deck of isolated bridges. As regards to the "*semi-active control combined with monitoring and EW*", the activity is focused on the:

1) Analysis of sensors and electronics needed for:

- Semi-active control activity: elements of input (sensors and conditioners), processing elements (hardware and software), control elements (devices, electricity and power), laboratory and on-site systems (operational possibilities and comparison).
- Monitoring: elements of input (sensors and conditioners), processing elements (hardware and software), laboratory and on-site systems: (operational possibilities and comparison).
- Seismic Early Warning: elements of input (sensors and conditioners), processing elements (hardware and software), laboratory and on-site systems (operational possibilities and comparison).
- Combined use of the three technologies: common elements and needed integrations.

2) Experimental activity (JET-PACS model derived from the previous ReLUIS Project):

- Semi-active control: designing the tests referring to the number and positioning of the devices, selecting the control algorithms, discussing the level of dissipation/force amount to be given.
- Semi-active control combined with monitoring and seismic EW: designing the tests considering different algorithms (corresponding to a smart passive or semi-active use of the variable devices), discussing the level of dissipation/force amount to be given.
- Comparison in terms of costs and structural performance in respect to passive systems.

4 MAIN RESULTS

4.1 ACTIVITY 1: Seismic retrofit and improvement of existing buildings through seismic isolation (includes bridge piers with seismic isolation)

4.1.1. Operational aspects regarding the installation of isolators at the base of existing masonry buildings (case studies) and estimation of costs

The following activities were performed:

- Completion of the classification of the critical aspects concerning the insertion and location of isolating devices below existing masonry buildings have been pointed out and the solution strategies have been outlined.
- Completing the outlining of the procedures, technologies and operational modalities for the cutting of existing masonry.
- Completion of the definition of the operational phases and the related works for placing the devices below masonry buildings.
- Completion of the outlining of solutions for maintaining the compatibility of the existing vertical communication systems within the isolated configuration.

- Arrangements of local/partial seismic isolation on historical buildings have been drawn to eliminate the vulnerability of particularly critical structural macro-elements.
- Suitable procedures of Performance Based Seismic Design (PBSD) for the evaluation of the economic compatibility of the base-isolated option for the seismic protection of an existing building have been outlined and applied to real study cases.
- The procedures, technologies and solutions outlined for the insertion of base isolation within existing masonry structures are being critically verified through the application to the design and the building of real study cases.
- Real applications of seismic isolation have been developed even on historical buildings within the ambit of cooperation projects established with structural designers.

4.1.2. Evaluation of the inelastic behaviour of existing RC frame structures retrofitted through base isolation

The parametric study on the inelastic behaviour of RC frame buildings with seismic isolation has been further developed by focusing the attention on the seismic effects of infilled masonry panels. Furthermore, the research has been focused on the application of seismic isolation for the seismic rehabilitation of a real RC frame building, accepting limited plastic deformations in the superstructure under strong earthquakes. This situation is representative of what would happen if seismic isolation was designed considering, for existing buildings, seismic rehabilitation objectives somehow inferior to those typically adopted for new buildings but still superior to the basic safety objectives that must be achieved in any rehabilitation project. The achievement of the target objectives of the design has been verified through nonlinear dynamic analyses.

The selected case study is represented by a 4-storey building with a rectangular plan of approximately 230 m^2, characterized by internal frames in only one direction and no openings in the infills of the weak direction. In the numerical model for nonlinear dynamic analyses, the plastic hinges of beams and columns have been modeled using link elements characterized by an hysteretic degrading cyclic behaviour (Takeda degrading-stiffness model).

The shear strength of the structural members has been suitably taken into account in the numerical model. Finally, the infilled masonry panels have been modeled as equivalent struts, considering an elastic-fragile cyclic behaviour.

Three different types of isolation system have been alternatively considered, i.e.: (i) HDRB, (ii) HDRB + flat surface sliding bearings, and (iii) curved surface sliding bearings (FPS). The nonlinear dynamic analyses of the selected case study are aimed at investigating also a series of critical aspects relevant to the application of seismic isolation for the rehabilitation of existing RC frame buildings, i.e.: (i) influence of the variability of the mechanical behavior of the isolation devices (variability of the friction coefficient for FPS, effects related to scragging, temperature and axial load for HDRB, etc.); (ii) differences related to the modeling technique (linear equivalent or non-linear) adopted for the isolation system; (iii) effects due to the use of commercial isolation devices instead of "ad hoc" isolation devices.

The results obtained so far, considering HDRB and HDRB + flat surface sliding bearings at standard temperature (20 °C), indicate that the global ductility demand to the superstructure is substantially in line with those obtained examining the 2-DOF models. The analyses conducted on the 3D model also permit to obtain detailed information on the local behaviour of the structure. In particular, the results of the analyses, for instance, show that values of global ductility demand of 2 are associated to plastic hinges rotations of beams and columns between the limit values corresponding to Immediate Occupancy (IO) and Life Safety (LS) Performance Levels, according to FEMA356. However, the results also indicate the critical role of the short columns intercepted by the knee beams of the stairs, which undergo plastic hinge rotations higher than the limit values corresponding to Life Safety Limit State, thus requiring local strengthening. The analyses have also pointed out the influence of the solutions "from catalog", for what concerns the choice of the isolation devices, on the design objectives of the seismic rehabilitation, with respect to “ad hoc” solutions optimized for the purpose.

A numerical study was carried out, aimed at evaluating the applicability and effectiveness of seismic isolation for the rehabilitation of RC frame buildings. The aforesaid study is based on nonlinear time-history (NTH) analyses carried out using 3D models. A set of 7 natural accelerograms has been selected using the software REXEL-DISP. These signals are spectrum-compatible with the target spectrum for L'Aquila (soil B and topographic category T1). Four multi-storey RC frame buildings, designed for gravity loads only, have been selected as building prototypes. The buildings feature a substantially symmetric structural layout in both horizontal directions, floor area of approximately 230 m^2 and number of storeys ranging from 2 to 8. Three different types of isolation systems (IS) have been examined: HDRB, HDRB+flat sliding bearings and FPS. The methodology of the research activity has been developed in two phases. First of all, the IS has been designed to provide an elastic behavior of the superstructure (SS) in case of severe motions. In this case, devices displacements of approximately 50 cm have been found. These values are certainly incompatible with the maximum displacements guaranteed by the seismic devices currently in use. In the second phase, using a simplified procedure based on a 2D model developed during the first year of the research project, the IS has been re-designed so as to contain the extent of the displacements while accepting the plasticization of the SS. The NTH analyses carried out during this phase showed (for each IS type) a significant reduction of the IS displacements as well as a substantial compatibility between 3D and 2D models in terms of ductility demand to the SS. In the 3D model, the structural behavior at the local level has been evaluated in terms of plastic hinges rotation of beams and columns considering also the brittle mechanisms due to shear stresses. The inelastic response of the structure, evaluated as described above, is consistent with the expected performances. The verifications in terms of shear indicate the critical role of the short columns intercepted by the knee beams of the stairs.

Basing on the results obtained during the third year, guidelines are proposed. The main points are summarized below: (i) the collapse limit state of seismically isolated structures should be based on the lateral capacity of the SS without significant reliance on its inherent hysteretic damping or ductility capacity; (ii) particular attention shall be paid in the selection of an acceptable value of the global ductility demand to the SS, in particular, the choice must be carried out considering the structural characteristics of the building and more importantly its inelastic mechanism, i.e.: μ_d=2 (q≈2.5) for buildings featuring a weak-beam/strong-column inelastic mechanism, being associated to plastic hinge rotations in beams (and columns) between the limit values corresponding to SLD and SLV, μ_d=1.5 (q≈1.5-2) for buildings featuring a soft-storey mechanism, being associated to plastic hinge rotations in columns consistent with the limit values corresponding to SLD; (iii) brittle failure should be avoided.

4.1.3. Theoretical study of the dynamic behaviour of isolated structures in phase space / Seismic response analysis of structures with pendulum isolators under near-fault events

The research group has continued research on innovative strategies for seismic protection which use isolation and energy dissipation. At the time, the attention is mainly faithful to the study of innovative design procedure to achieve robust and/or optimal strategy to control the

seismic behavior of framed structures. In the following the main results for base isolation strategy are represented.
The seismic behaviour of base isolated buildings has been analyzed by using an innovative mathematical formulation in the state space, that allows us to describe the dynamic response of structures in the case of non-classical damping. Particularly, the seismic response of base isolation with linear–viscous behaviour is herein investigated by studying the mode shapes, the frequencies and the modal participation factors, that are obtained by the proposed formulation varying the main design parameters. In such a manner, the effect of these parameters on the isolated structure behaviour as a whole is evident.
As examples, the following Figures 1 and 2 show the frequencies and the modules of the modal participation factors associated with the first and the second mode shapes of the isolation devices.

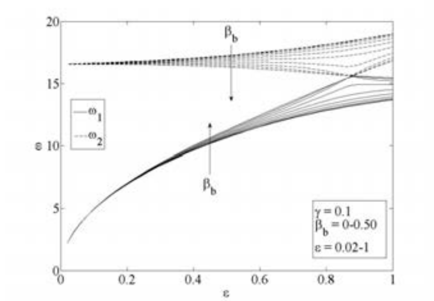


Figure 1. Frequencies associated with the first and second modal shapes evaluated for γ=0.1.

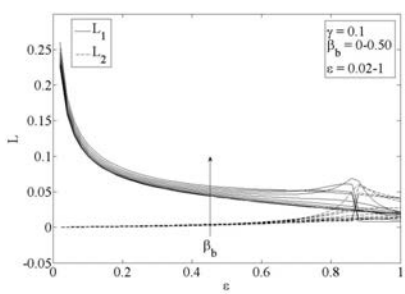


Figure 2. Modules of the modal participation factors associated with the two modal shapes evaluated for γ=0.1.

The obtained results lead to the following main conclusions:

- values of the damping factor of isolation devices higher than 0.35 cause a worsening of the superstructure response;
- low values of the mass ratio, less than 0.2, can lead to a worst behaviour of the superstructure. Moreover, high values of the degree of coupling can lead to an inversion of the mode shapes;
- values of degree of coupling near to 0.5 lead to inside resonance effects between base isolation and superstructure which maximize the effective damping on the first mode shape;
- high values of the mass ratio, greater than 0.9, can lead to overdamped behaviour in the second mode shape for greater values of the isolation factor damping and the degree of coupling. In particular, values of greater than 0.30 lead to a worst behaviour of the superstructure.

For the case of Friction Pendulum Bearings (FPB) the research aims to investigate cases where the vertical component of seismic motion is relevant like that of nearfault events. To investigate the potential performance that could be achieved by FPBs, the study considers an ideal isolated system characterized by a perfect rigid superstructure neglecting lateral-torsional effects. In particular, a nonlinear 3-DOF system has been described in SAP2000 to analyze the seismic response to recorded nearfault events like L'Aquila 2009 and Emilia Romagna 2012. The analysis of results shows that the vertical component of seismic events could significantly affect the behavior of the isolated structures in the case of FPBs.

4.1.4. JETBIS Project: set up of experimental physical model (one-story steel building, isolated at the base by different isolation devices analyzed within the Task), definition of experimental tests program on shaking table and characterization test of devices

All the steel elements (plates, special components, etc.) needed to complete the experimental set-up of the shaking table tests campaign on the base isolated structure (Figure 3 and 4), were designed and prepared.

The Research Unit of Naples Federico II, responsible of the shaking table, has developed the configuration of sensors to measure all the required experimental quantities, has defined the seismic inputs, and the type of tests to be performed, in agreement with the other RUs.

Shaking table experimental tests were performed on the isolated structure by means of low cost recycled rubber isolator reinforced with layers of high strength quadri-directional carbon fiber fabric (Figure 3, 4 and 5):

- dynamic characterization tests of the structure, to the aim of evaluating the first two frequencies and the corresponding damping factors;
- seismic tests through the application of seven properly selected natural earthquakes;
- fatigue tests by the application of a moderate earthquake to the base of the structure, for twenty times, to the aim of verifying the degradation of the isolators.

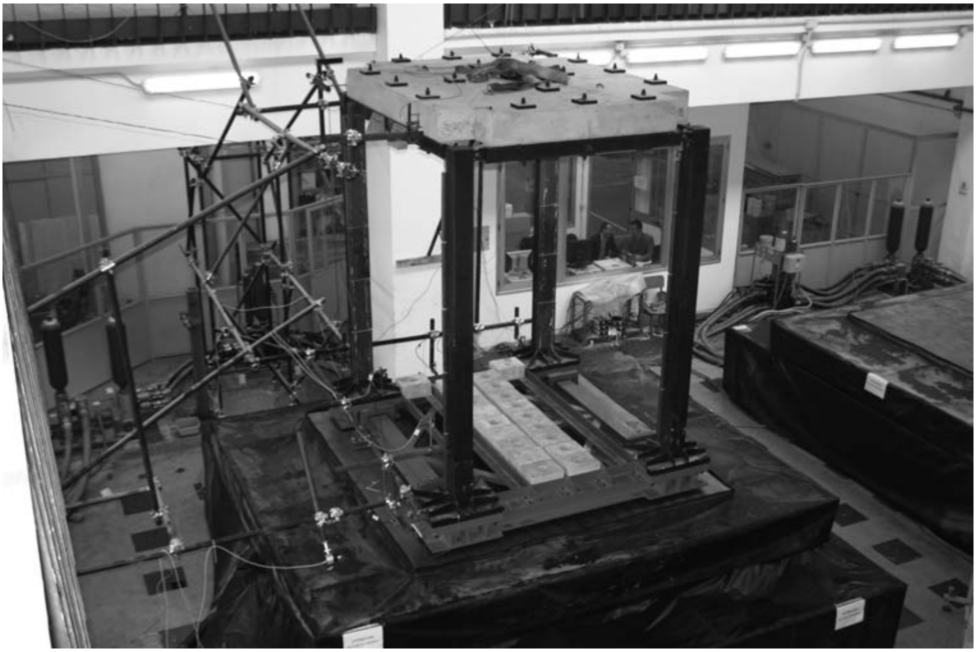


Figure 3. Experimental base isolated structure.

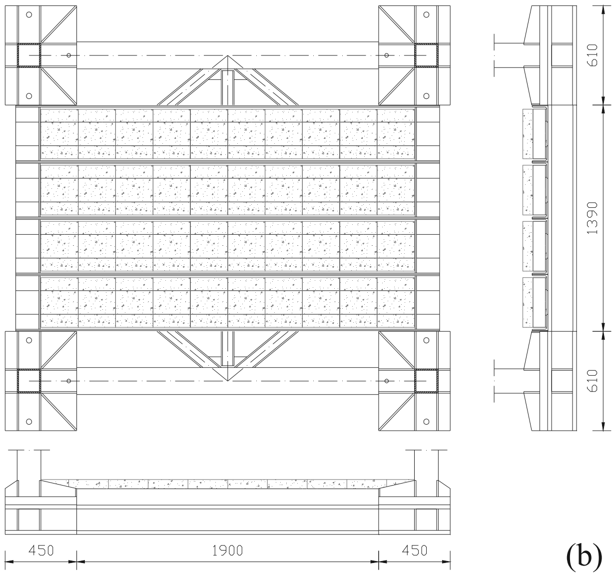


Figure 4. Experimental frame structure and additional base horizontal frame.

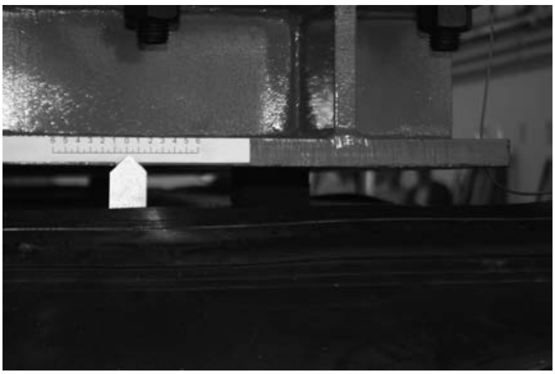
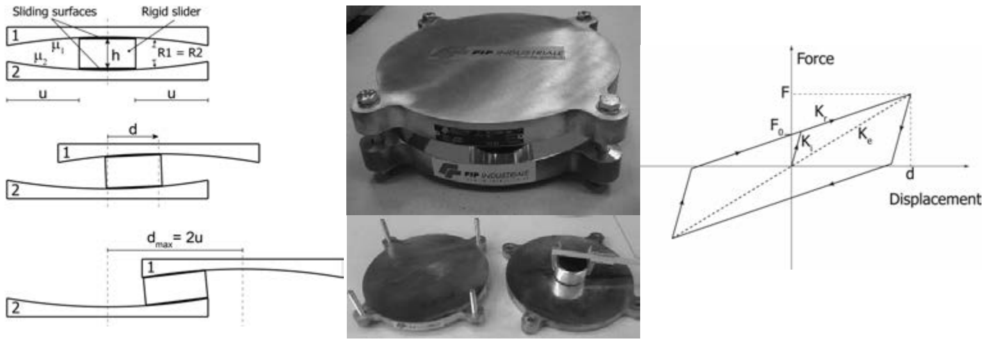


Figure 5. Low cost isolator mounted under the isolated structure.

Shaking table testing was carried out on 1/3 scaled structural steel model protected seismically using Double Concave Friction Pendulum base isolators (DCFP). These isolators were produced by FIP-Industriale (FIP-D isolator – Figure 6). Testing demonstrated the effectiveness of the isolation system across the various testing configurations (model with masses both symmetrical and eccentric, isolators with and without lubrication).

Before shaking table testing some characterization test on DCFP was carried out at Seismic Laboratory of the University of Basilicata. The bearings were tested under a N = 32 kN design vertical compressive load. A series of sinusoidal lateral displacement histories has been imposed in accordance with testing of Curved Surface Sliders prescribed by Eurocode (UNI-EN 15129, 2009). Figure 7 shows experimental force-displacement relationships of the devices carried out on different conditions of surfaces: i) Standard, ii) Lubricated (SL) and iii)

Not-lubricated (SNL). A silicone based (lithium soap) lubricant was applied to the top and bottom face ($\mu_1 = \mu_2$) of the rigid slider. Figure 7 shows also the numerical simulations results obtained by considering both constant and variable friction models.



a) b) c)

Figure 6. a) functioning scheme of the DCFP device; b) overview of the DCFP device; c) theoretical hysteresis cycle of DCFP bearing having equal radii of curvature and equal friction coefficients.

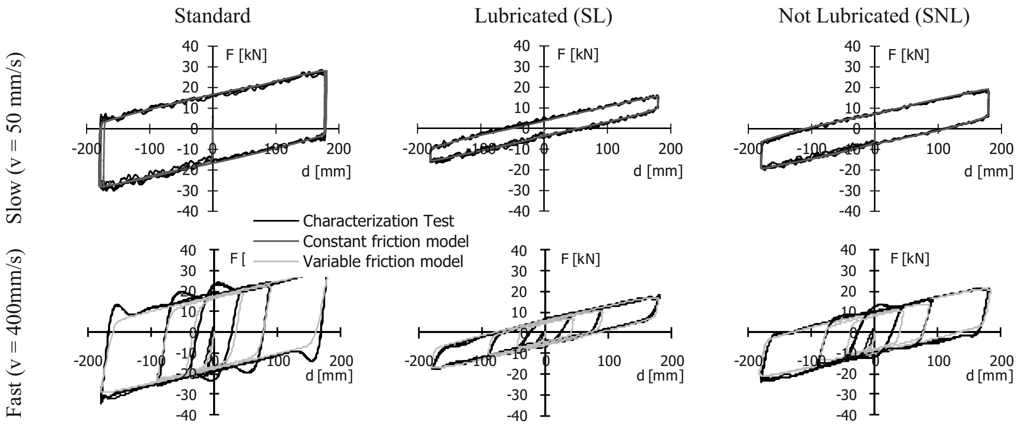


Figure 7. Experimental and numerical Force–displacement at peak velocities of 50 and 400 mm/s. Both tests were conducted on Standard, Lubricated (SL) and Non-lubricated (SNL) surfaces.

The shaking table tests demonstrated also the potential for the use of low-cost and low-quality elastomers for the production of fiber reinforced bearings.

Low-performance materials are suitable because the bearings are unbonded and reinforced with flexible fiber sheets. In this configuration, devices can deform freely without generating high tensile stress, which is common in transversal layers of conventional isolators.

The absence of tensile stresses prevents the vulcanization of rubber to the reinforcements. For instance, devices for the tests were manufactured by gluing layers of a recycled elastomer to layers of fiber reinforcements with a polyurethane adhesive, without the need for costly and high-energy demanding vulcanization processes. The proposed devices are low-cost and eco-friendly.

A series of 27 records was employed for the shaking table experiments. The selected ground motions were representative of moderate to high seismic regions in Italy. With such severe inputs, the bearings performed exceptionally well. They demonstrated robust behavior and re-centering capabilities in all tests; a visual inspection of the devices confirmed no damage. The tests gave a preliminary assessment of the viability of the concept. Future multi-directional experiments are required for a complete understanding of the nonlinear behavior of the proposed base isolation system and for its acceptance by the construction industry. The proposed technology could influence the retrofitting of historic buildings and unsafe public housing in seismic-prone regions of the world.

4.1.5. Observatory of Isolated structures in L'Aquila / Artificial "isolated" soil

The following activities were performed:

1. institution observatory on isolated structures built in l'Aquila;
2. seismic analysis of an artificial "isolated" ground.

Regarding the observatory on isolated structures in L'Aquila (point 1), data on isolated buildings were collected, build, in stages of construction or planning in the area of L'Aquila, in order to create a database (n. of elastomeric isolators and FPS, design displacement; period of isolation; maximum vertical load, etc.) useful for the design and control of seismic isolated buildings.

Concerning the artificial "isolated" ground (point 2), 2 seismic isolated plates of more than 26000 m^2, with overlying buildings with a number of storeys between 1 and 15 (Figure. 8), had been designed and analyzed. The isolation of the artificial ground and of all the above buildings was achieved by placing the plates on elastomeric isolators arranged head to the columns of the basement (parking). The performed numerical analyses showed that the "isolated" artificial ground can decouple the vibration modes of the structural system, despite the overall complexity of the analyzed structural system.

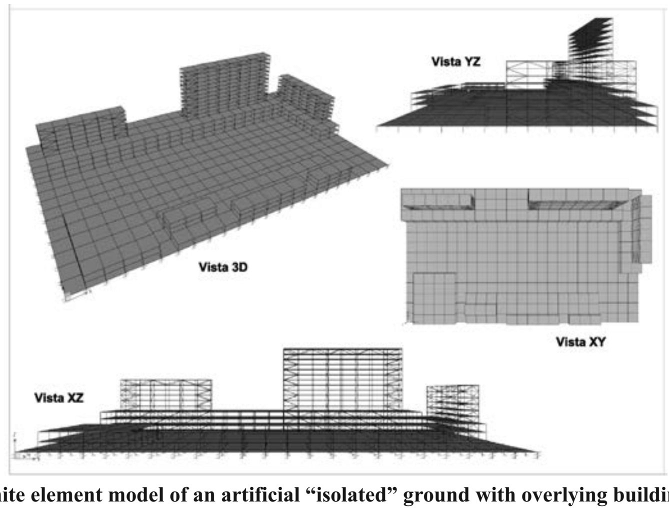


Figure 8. Finite element model of an artificial "isolated" ground with overlying buildings of 1 to 15 storeys.

4.2 ACTIVITY 2: Development of new isolation devices, also low-cost

4.2.1. Manufacturing of prototypes and execution of experimental tests on new low-cost isolation and dissipation systems

The low cost devices proposed are unbounded isolators made of a recycled rubber compound reinforced with carbon FRP layers. The geometrical and mechanical characteristics of the prototype isolators were designed by considering their strength and stability behaviour under the experimental structure. Their experimental behaviour has been studied in detail, by means of compression tests (at the laboratory of the Department of Structural Engineering of University of Naples Federico II – Figure 9), and shear tests (at the laboratory of the Department of Mechanical Engineering for Energetics of University of Naples Federico II – Figure 10). After the experimental tests, the vertical stiffness and the horizontal stiffness have been computed, as well as their variability, respectively, with the vertical loading and applied horizontal displacement. The low cost devices are unbounded isolators made of a recycled rubber compound called "polverino" (10% granules of SBR and 90% very small granules of SBR) reinforced with carbon FRP layers. The geometrical and mechanical characteristics of the prototype isolators have been designed by considering their strength and stability behaviour under the experimental structure.

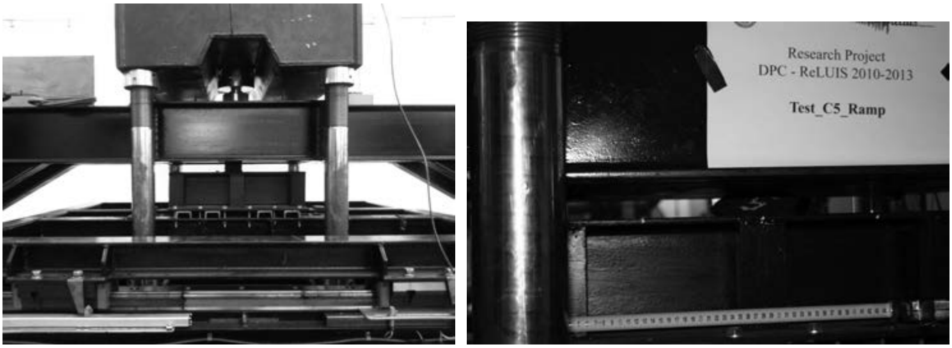


Figure 9. Shear tests on a prototype recycled rubber isolator.

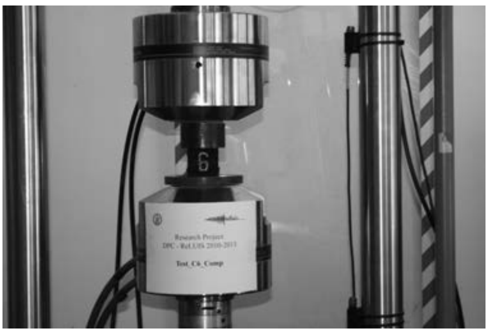


Figure 10. Compression tests on prototype recycled rubber isolators.

Dynamic testing have been performed at the structural laboratory of the University of Basilicata, on a 3-storey, 2/3 scaled post-tensioned timber structure (Figure. 11) in order to verify the effectiveness of an innovative protecting system based on the coupling of post-tensioning rods and L-shaped steel dissipating plate elements. The first element provides elastic recentering to the structure when subjected to horizontal seismic excitation while the second one provides additional strength and dissipative capacity. Testing have been performed both with and without the addition of dissipative steel angles.

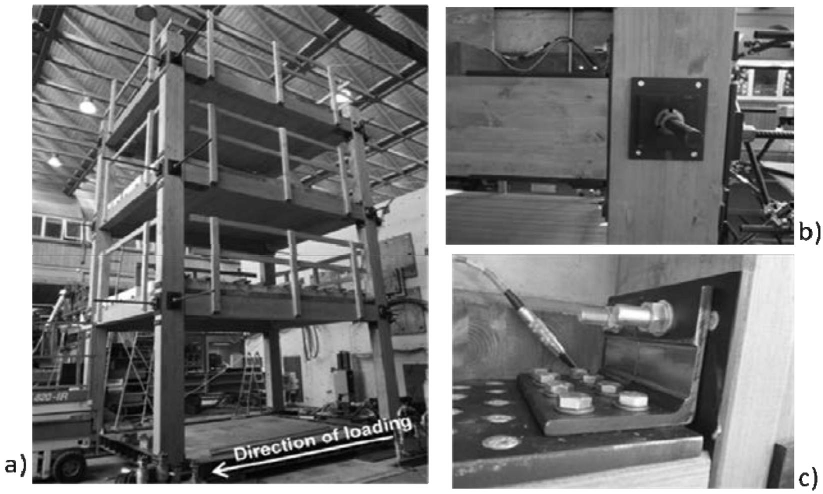


Figure 11. a) 2/3 Scale Post-Tensioned Glulam Test Frame; b) Post-Tensioned Glulam Frame Beam-Column Connection; c) Yielding Steel Angle dissipation devices.

4.2.2. Analytical forecast of the mechanical properties of Wire Rope devices, on the base of coils and wires' geometry

The wire-rope devices proposed by the research Unit Naples Federico II have been used for a long time for isolation from vibrations and protection from the bumping of equipment in the military, electronic and air space fields: they consist of cables in stainless steel wound in the shape of a coil or an arc on drilled bars in aluminum alloy; each cable consists of several plaited strands, while each strand in turn consists of several wires, the number of which varies according to the device in question. A peculiar characteristic of the wire-rope isolators is that of being deformable in both the two horizontal and in the vertical directions, and of possessing at the same time a significant dissipation capacity due to the hysteretic damping achieved thanks to the friction produced by the rubbing of the individual strand wires and between one strand and another. The possibility of incorporating the filtering with the dissipative function in a single element makes these devices particularly interesting also for seismic isolation, and suitable in particular for the protection of light but costly equipment, in view of their quite considerable deformability in the vertical direction

An analytical study on wire-rope devices was completed (Figure 12), in order to single out the mathematical relationships existing between the geometrical characteristics and the mechanical properties of these kind of devices: this may allow an effective help in the design of the dimensions of wire-rope devices to be manufactured for the isolation of the simple experimental structure.

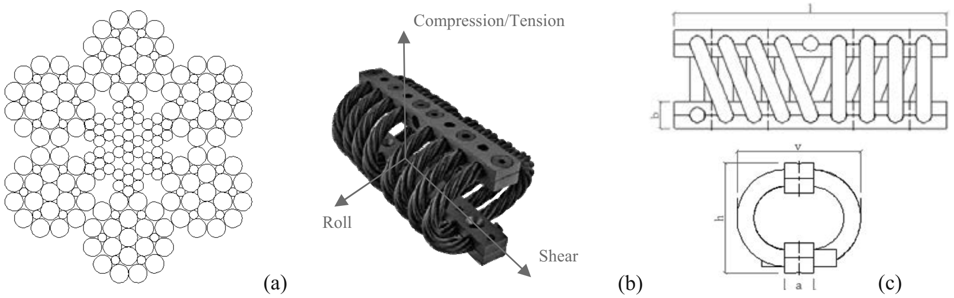


Figure 12. (a) Sketch of cable section; (b) loading directions of a wire-rope; (c) drawing of a wire-rope.

4.2.3. Study of mechanical behavior of laminated rubber bearings

The numerical study has been made by developing nonlinear models of equivalent beam for the analysis of buckling (and post-buckling) of fiber reinforced elastomeric isolators (SLFREI), subject to nonuniform shear warping (out of the plane displacements). The models are developed locally in terms of the classical linear analysis and subsequently set coherently in a nonlinear terms within the framework of the implicit corotational method using a quadratic asymptotic development of Biot deformation tensor. With reference to the stability analysis, the results obtained for several different models have been compared with those produced by equivalent beam models established in the literature.

The possible failure modes of circular elastomeric isolators (debonding; instability, yielding of reinforcing steel plates; cavitation of the rubber; roll out) have been identified and studied. Then, the domain of stability in Figure. 13 has been defined; it provides the limit medium pressure (P_m) which can be applied to the isolator in the presence of earthquake at different shear distortion (g_s). In Figure. 13, $P^{m,0}$ is the maximum pressure that can be applied to the isolator for g_s=0; S_2 is the secondary shape factor. In particular, from the figure it is observed that the domain is delimited at the top by the straight line of Eq. $P_m/P_{m,0}$=1-1.2·g_s/S_2, that is representative of collapse by stability or plasticity of the plates, the lower the curve of Eq. $P_m/P_{m,0}=g_s/S_2\cdot(G\cdot H/t_e)/(1-g_s/S_2)$, that is representative of collapse for roll out, and on the right from the vertical line of Eq. $g_s/S_2=2/S_2$ that corresponds to the limitation $g_s \leq 200\%$.

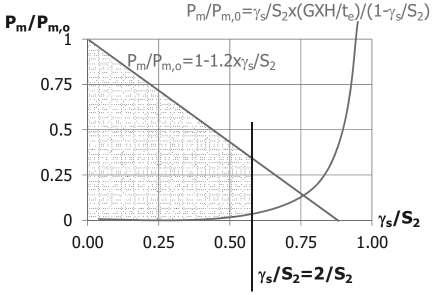


Figure. 13. Stability domain for circular HDRB devices for base isolation

4.2.4. Characterization and design of Rubber-Layer Rolling-Bearing (RLRB)

In the first year of research the theoretical model of RLRB has been planned in order to define the fundamental parameters. Activities of the second year have allowed to improve the numerical study of the device, its mechanical design and of its components, studying, also, interaction between steel and rubber in order to understand the consequent behavior of the device. Moreover, it has been developed the plan of the device used in last year of activity to make characterization tests.

In the third year of activity, after a prior examination of the device's peculiarities and links necessary to assembling the device on the test machine, characterization tests have been performed in order to check the operating principle of the device. It has been used an Instron 5869 electromechanical testing machine, with low frequency. Usually, Instron 5869 material-testing machines permits to test the tensile or compressive strengths of a sample material. In this case, by an accurate design of the link components rolling tests have been performed obtaining the hysteresis cycles.

Tests have been made changing the normal load on the device by using UPN steel elements used in order to distribute the load in a homogeneous way. The link between steel layers and the tests machine has been studied with a mechanical constrain which permits rotation around a Φ 24 steel shaft and, for the central steel layer, it has been chosen a constrain with a two link point.

Characterization tests have been made with 0.05 Hz frequency and realizing five cycles; it has been defined a displacement on the internal plate of +/-25.0 mm. Three different hypotheses about the normal load on the device have been assumed. The first load has been defined in relation to the shaking-table structure, 2.0 tons, and after other two loads of 1 ton and 3 tons have been used to check the behavior of the device. The characterization tests demonstrated the correct behavior of the device in a geometric and dimensional aspect and in performance aspect. The hysteresis cycle, in fact, has been useful to demonstrate the dissipation efficiency of the isolation system, because also the rubber layer has not presented damages because of the high load. Characterization tests, therefore, have not highlighted an unexpected behavior of the device; on the other hand they demonstrated, although in a preliminary way, the efficiency of the RLRB device.

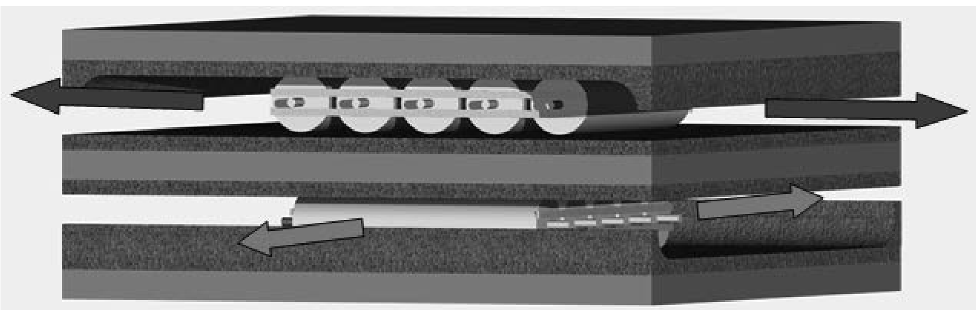


Figure 14. Render of the isolator Rubber-Layer Rolling-Bearing (RLRB).

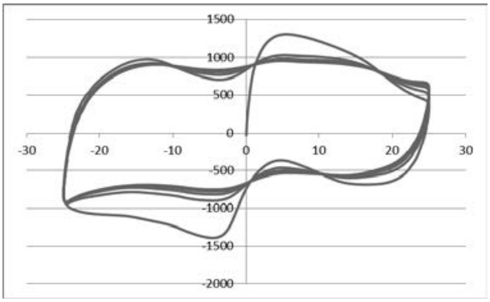


Figure 15. Characterization test on RLRB device with load 1 ton.

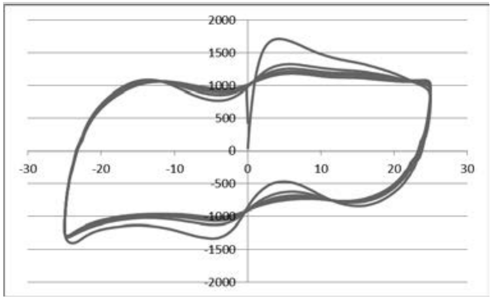


Figure 16. Characterization test on RLRB device with load 2 ton.

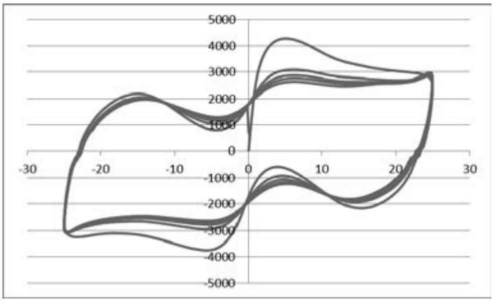


Figure 17. Characterization test on RLRB device with load 3 ton.

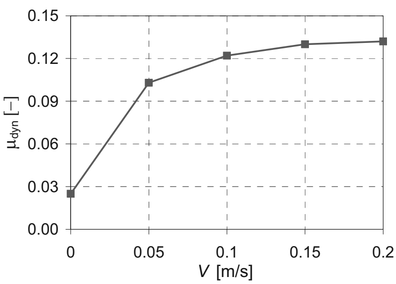
4.2.5. Development and experimental characterization of a new auto-lubricant material based on optimized PTFE for sliding isolators

The following activities were performed:

(1) Implementation of three-dimensional finite element models of sliding isolators.

(2) Assessment of the thermal-mechanical behavior of sliding isolators in numerical analyses; investigation of frictional heating and its effect on the dynamic response of the isolators under different loading histories.

(3) Design and production of sliding isolators for tests on shaking table (JetBis project).



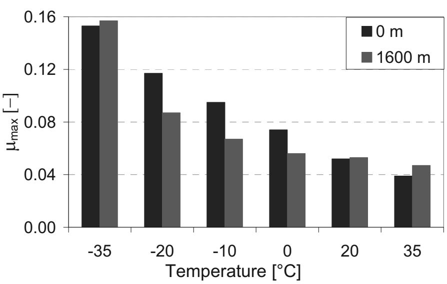


Figure 18. Dependence of the coefficient of friction of metal filled PTFE on sliding velocity (left) and on temperature and accumulated sliding path (right).

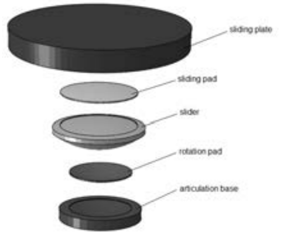


Figure 19. Three-dimensional model of the curved surface sliding isolator.

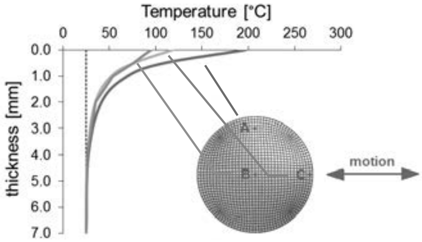


Figure 20. Temperature profiles through the thickness at different points on the surface of the friction pad.

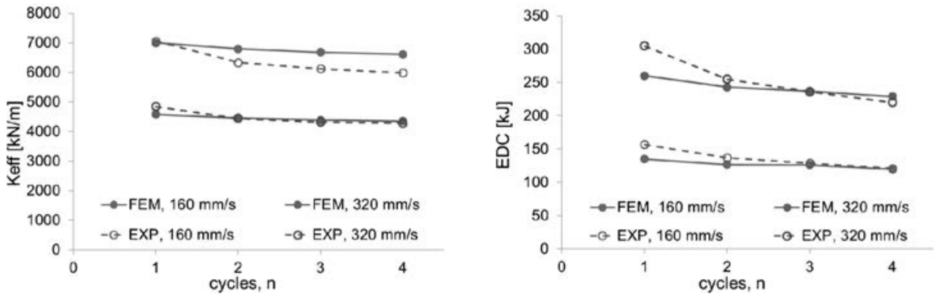
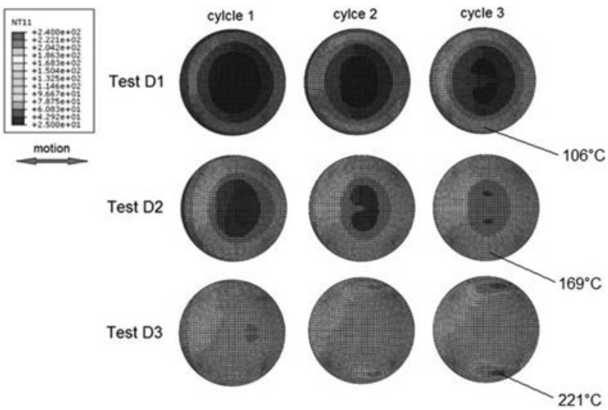


Figure 21. Dynamic properties of the isolation unit (lateral stiffness K_{eff} and Energy Dissipated per Cycle EDC) in shaking tests at different speeds: numerical calculations (FEM) and experimental validation (EXP)



test	velocity [mm/s]	Tmax [°C]	Tavg [°C]
D1	85	106	60
D2	170	169	80
D3	340	221	130

Figure 22. Temperature distribution of the surface of the friction pad predicted in analyses of unidirectional tests at different velocities.

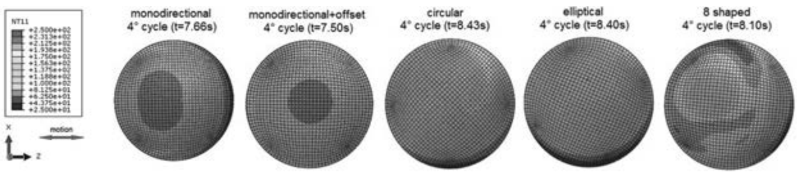


Figure 23. Influence of the loading path (uniaxial vs. biaxial orbits) on temperature distribution on the surface of the friction pad.

4.3 ACTIVITY 3: Manual with guidelines for the design of passive energy dissipation systems

4.3.1. Definition of design methodologies of dissipative braces for the retrofit of existing buildings

With reference to the viscous or viscoelastic dampers (Hwang et al. 2008), the possibility of achieving the seismic protection through the integration of the elastic lateral stiffness resources and the viscoelastic properties of a dissipative bracing-damper system has been investigated. The innovative aspect consists of considering the viscoelastic damping resources as a design variable in order to control the dynamic response. It has been thus proposed and developed an integrated design methodology to ensure a preassigned performance, within the displacement based design approach, which explicitly takes into account the dynamic behavior both of the structural and control systems (Ou et al. 2007).
The optimal design criterion has been defined by determining the combination of the variables which minimizes a total cost function evaluated by considering the relative cost between the elastic resources and viscoelastic dissipative ones (Ang and Lee 2001).
Then a validation of the integrated procedure has been performed by verifying that the dynamic response of an optimal single-degree-of-freedom integrated system achieves in average the expected performance displacement by considering a set of seven unscaled acceleration records compatible in average with the elastic spectrum relative to the life safety state, provided by the new Italian seismic code. A new strategy of extra-structural dissipation of energy has been studied by taking advantage of the innovative formulation in state space, which allows to describe the dynamic behavior of structures in the case of non-classical damping. The new base damping strategy view the concentration of the dissipation of energy in special devices located at the base of mixed wall systems in such way preserving the structural elements of the superstructure from damage.
The research has mainly focused on the development of the guide lines for the design of energy dissipation systems, with particular attention to case studies. Secondly, in order to furnish the design indications, some investigations about the behavior of existing continuous bridges protected by seismic isolators are also carried out. With reference to the first activity, during the second year, a probabilistic methodology that permits to evaluate the seismic vulnerability of the structural systems and the effectiveness of the retrofit based on the introduction of dissipative braces is developed. The methodology is based on the use of local engineering demand parameters (EDPs) for monitoring the seismic response and on the development of component and system fragility curves before and after the retrofit. During the third year the methodology has been extended for the seismic risk assessment, by convolution of the fragility curves built using local and global EDPs with hazard curves corresponding to various hazard scenarios. The methodology has been applied to a benchmark 2-dimensional RC frame retrofitted by introducing Buckling Restrained Braces (BRBs)

designed for different levels of base shear capacity. according to a design method already validated in the first year of the research. The obtained results have confirmed the efficiency of the retrofit and have shown that, in general, the use of global rather than local EDPs results in lower safety margins.
The last period of the third year has been dedicated to develop a benchmark application to introduce into the guide lines for the design of energy dissipation systems. To this purpose several study cases have been analyzed and, at the end, a real RC building composed by 6 levels and built in 1983 (Figure. 24a) was chosen.

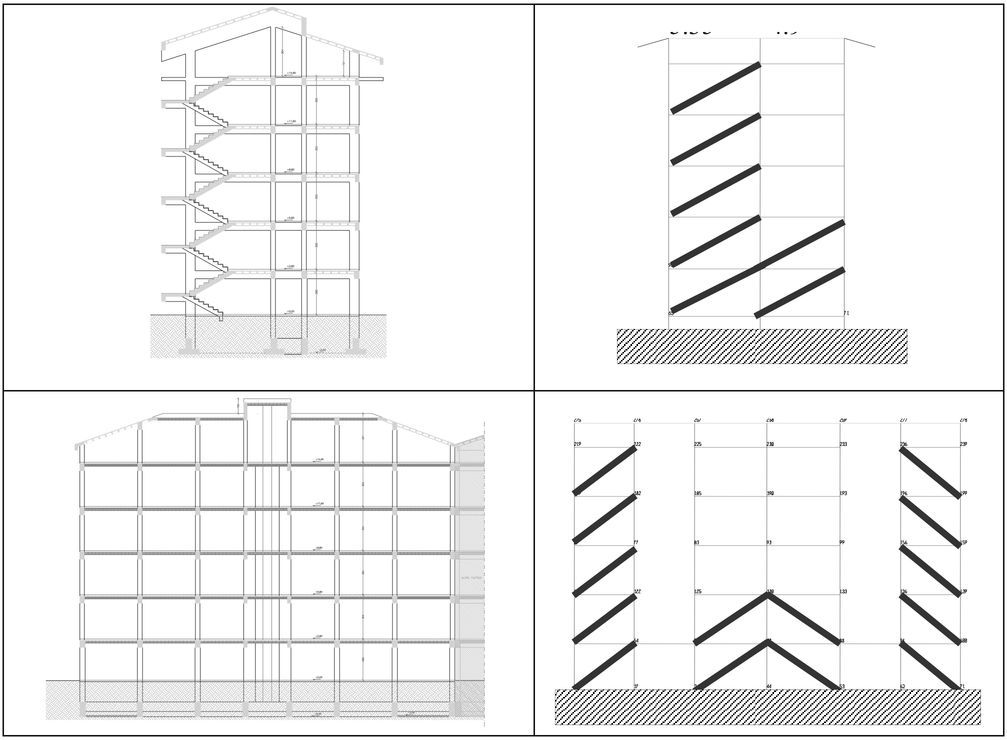


Figure 24. Representation of the chosen case study (a) and configuration of dissipative braces (b).

A 3D non-linear model of the frame has been produced by the code SAP 2000 in order to perform a PushOver analysis for evaluating the seismic vulnerability. The results have shown that in both the horizontal directions the obtained frame capacity is significantly lower that the demand. Both elasto-plastic and visco-elastic dissipative braces are considered for the retrofit. To define the characteristics of the dissipative braces the design method validated during the first and second year of the research is used. Different configurations have been developed (Figure. 24b) in order to find the optimal solution. With reference to the chosse solution both the braces components (devices and link braces) have been dimensioned and constructive details have been developed.
With reference to the second activity the dynamic properties and the seismic behaviour of isolated bridges with transverse constrains at the abutments has been investigated and simplified procedures for the preliminary design have been developed.

Part of activities 3 were coordinated with the specific task of structural health monitoring. For it have been studied and compared techniques and procedures relating to the parametric identification of structural models representative of frame structures (Antonacci *et al.,* 2012c) . The research group continued the study of the development of possible strategies for structural health monitoring using smart sensors connected to each other through wireless networks (Braga *et al.,* 2012). In this regard it should be noted the improvement of the structural health monitoring for the Basilica of Collemaggio through the installation of 11 exstensometer and 3 inclinometers (Figure. 25). This system, can be useful for a better understanding of the structural behavior, moreover this constitutes a case of study for testing the effectiveness of this new technology (Braga *et al.,* 2012; Braga *et al.*, 2013). Furthermore the RU has continued its studies for the assessment of seismic vulnerability of buildings damaged by the earthquake Aquilano (Cardone *et al.,* 2012; Castaldo *et al.*, 2013), which can provide useful information about the real situations and highlight the possibilities of applications of the technologies studied in this task.

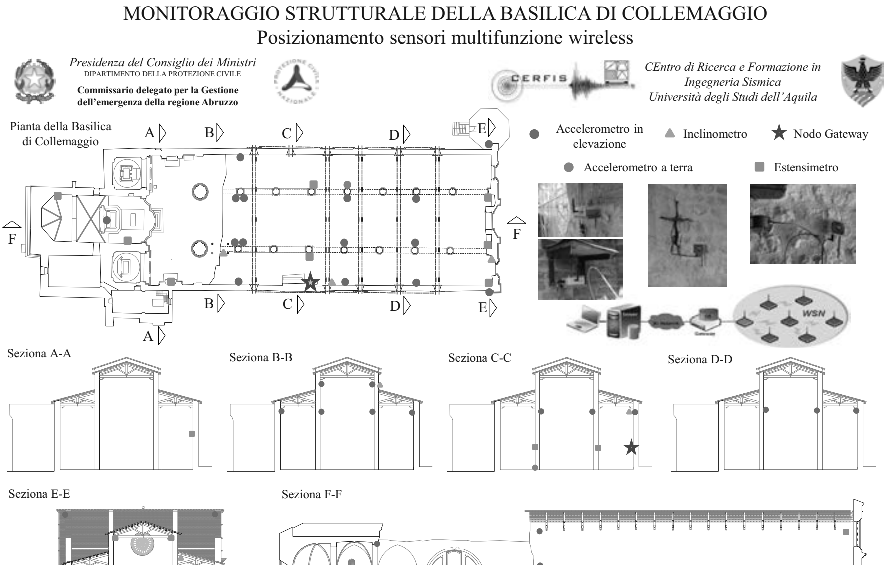


Figure. 25. Monitoring of the Basilica of Collemaggio: multifunction sensors placement.

4.3.2. Study and delivering of simplified design procedures of dissipation systems based on linear analyses with structure factor

The parametric study on the inelastic behaviour of existing RC frame buildings retrofitted with energy dissipation bracing systems was concluded by focusing the attention on the seismic effects on both direction (acting simultaneously) and on medium rise buildings.

In line with the studies conducted during previous years, 20 cases of study considering buildings designed for gravity load only, have considered in the parametric analysis. The beam and column dimensions and detailing were keep as typical of Italian construction of the 70's and 80's.

The design of the bracing systems was optimized and applied for each of the 20 basic structures considering both main directions (direction X and Y) and two diverse bracing arrangements (V inverted (V) and diagonal (X)). In each case the same design procedure did not consider any specific intervention to the structural elements (beam and columns) where the bracing was applied. Coherently to previous years, different design targets were used considering (i) 4 values of structural ductility (μ^* 1.0, 1.15, 1.3, 1.5) and (ii) 3 values of ductility of the equivalent bracing (μ_{DB} 4, 8, 12), a total of 960 cases simulated.
The analysis have confirmed the effectiveness of the analytical formula proposed for the calculation of the structural factor for braced buildings q_B uses a coefficient C to augment building initial value of q ($q_C = C$ *·q). The increase of the structural factor is ranging from 1 to 4 depending on the combinations of design parameters. The best correlation between the values of C evaluated using NLSA and that calculated C_{cal} considering different combinations of the proposed independent variables was obtained through a linear regression considering only three parameters with higher weight: i) structural ductility μ^*; ii) the ratio between the bilinear equivalent period of the braced structure and that of the original structure T_B^* / T^* ; iii) the ratio between the yield point of the bracing and the resistance of the original structure F_{DB} / F_y^*. From the analyses performed in order to avoid the overloading of the elements of the original structure it is recommended that: i) the yield force of the equivalent bracing F_{BD} is not too high in with respect to the yield force, F_y* of the original structure (i.e. $F_{BD} / F_y^* < 1.3$), and ii) the stiffness of the braced structure is not too high with respect to the original structure (i.e. $T_B^* / T^* > 0.2$).
During the third year the robustness of the technique was verified and a simplified design procedure, based on the linear analysis with q-factor, was proposed as alternative to non-linear methods. Moreover, the non-linear dynamic analysis (NLDA) and non-linear static analysis (NLSA), performed on a benchmark case by using two different finite element programs (CDS-Win and SAP2000), were completed and compared. The benchmark structure was a 4 storey rectangular plan (structure type 2) located in a high seismic zone. A good agreement between numerical results was observed, this demonstrates the effectiveness of commercial software in the simulation of the seismic behavior of buildings with such passive protection systems. The selected case study was selected as example for the application of the proposed design procedure in the *Guidelines for the design of passive energy dissipation systems*.
Regarding to the activities targeted to analysis of problems related to the implementation of the energy dissipation to existing framed buildings, starting from the monotonic moment-curvature analyses performed by varying different parameters (axial load ratio, volumetric transverse reinforcement ratio, longitudinal bars percentage, concrete strength and steel grade) a design equation linking the curvature ductility to transverse reinforcement amount to be provided to RC section has been derived. In the non-linear analyses the BGL model has been used for evaluating the confinement effects on section response.
As far as the nonlinear behavior of poorly detailed concrete buildings with smooth bars is concerned, the influence of anchorage loss of passing bars within joint panel has been investigated. The nonlinear analyses have been performed on an internal beam-column joint reproducing a connection of the concrete 3D frame, made up in scale 2:3 and designed only for vertical loads. The numerical investigations, taking also into account slippages effects, have been compared with the experimental results of the joint that has been subjected to two consecutive tests: without and with FRP wraps applied at the columns zones near the panel joint. The last intervention may simulate a local strengthening of columns critical regions when braced system is applied. Starting from this preliminary study, the loss of anchorage of passing bars within the joint panel may also be simulated into the entire model of the 3D

concrete frame. The obtained nonlinear fiber model will allow us to evaluate the interactions among failure mechanisms observed on the experimental tests of RC subassemblages reproducing beam-column joints of the entire structure.

An extensive study on damped SDOF systems has been developed aimed at deriving a relationship between the force reduction factor *R* and the ductility demand μ, under a specific criterion of equal structural safety level between the damped and the corresponding undamped system. The analyses have been carried out for values of damping ratio between 0.05 and 0.35 and showed that the force reduction factor *R* is basically not influenced by the damping ratio. Thus, for practical purposes, the force reduction factor for structures with added dampers can be assumed approximately equal to the force reduction factor for systems without added dampers (typically provided by codes). Furthermore, the analyses show that increasing viscous damping significantly decreases the coefficient of variation of *R*, thus providing a higher level of structural safety (in terms of robustness) with respect to the case of structures without additional viscous. Based on this result, a global reduction coefficient n_{tot} (accounting for both the ductile capacity of the structure and the amount of damping ratio provided by the added dampers) has been proposed to be applied to the elastic spectrum for structures equipped with added viscous dampers.

As far as the third objective is concerned, an innovative conceptual design approach is proposed aimed at obtaining an optimized seismic behaviour of the building structure. The approach leads to an "enhanced first-storey seismic isolation system", which is borrowed from the idea of soft-storey isolation, first proposed in the late '60 by Fintel and Khan and revised according to the Performance Based Seismic Design framework. Among all the possible solutions, the seismic story isolation can be obtained through the insertion of special hysteretic devices at the bottom level of the building only. These special elements (called "Crescent-Shaped Braces") are designed in order to satisfy the prefixed multiple seismic performance objectives, also accounting for P-D effects. The performances of the building under multiple earthquake design levels are finally verified through non-linear time-history analyses.

4.3.3. Development of simplified methods for designing dissipative coupling systems among adjacent structures

The activity was intended to continue the case studies concerning the structural control and the possibility of integration with monitoring systems.

In particular, it has been defined a strategy for the design of dissipation systems in adjacent structures (Figure. 26 a, b, c, d) which is based on the parametric analysis of eigenvalues varying the design parameters (stiffness, h, and viscous element, g, of the device, Figure. 26 e and f). The result of this activity was the creation of a design map for a dissipative element elastic-viscous (rheological model of Kelvin-Voight) and viscous fluid (rheological model of Maxwell), see Figure. 26 g, h. A more detailed description of the modeling, implementation and testing of the strategy is shown in (Antonacci *et al.,* 2012a). Moreover, this method has also been adopted in the activities concerning the seismic retrofitting of structures of the Faculty of Engineering (Antonacci *et al.,* 2012b). Finally, all the work was synthesized in the manual for the design of the devices in the chapter on the dissipative coupling.

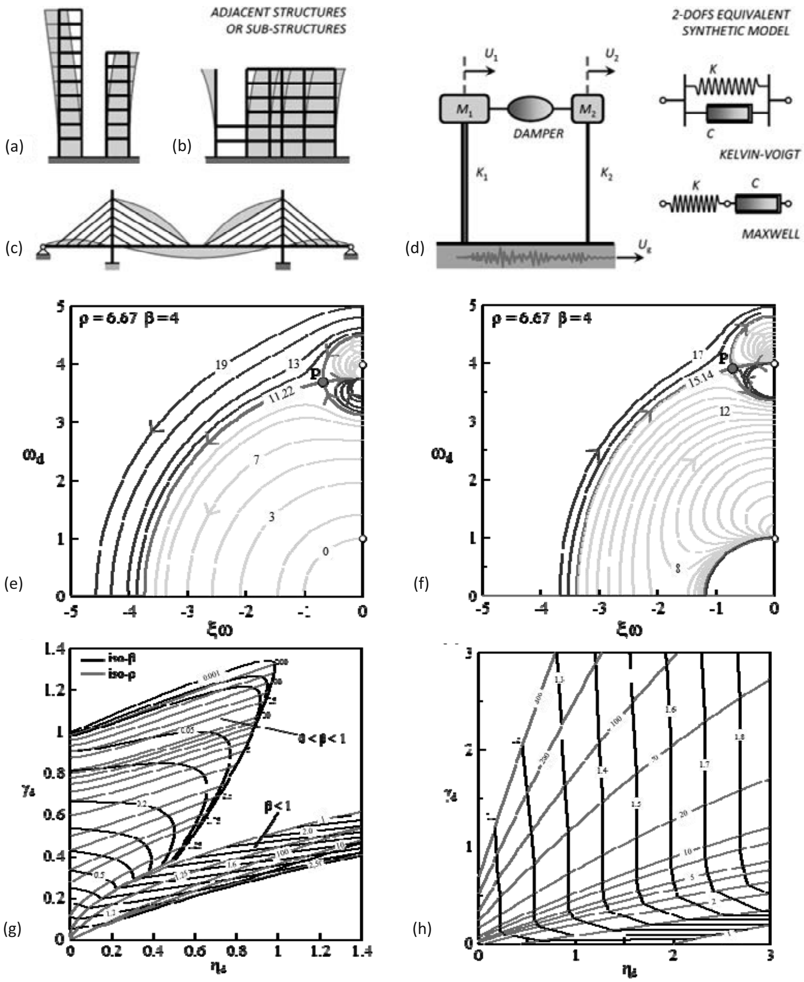


Figure 26. Adjacent structures: (a) tall parallel buildings, (b), (c) weakly coupled substructures in complex buildings or bridges, (d) synthetic equivalent model to 2gdl composed by 2 simple oscillators coupled through different models of the device. Loci of the eigenvalues varying γ with η constant, represented in the Argand plane: (e) Kelvin-Voigt model, (f) Maxwell model. Design maps for a dissipative connection of two simple oscillators: (g) Kelvin-Voigt model, (h) Maxwell model.

4.3.4. Drafting of the design manual

The final purpose within Activity 3 was to write a manual that could be useful to engineers who want to design a new/existing structure with dissipative systems. Dissipative braces and dissipative coupling systems are considered.

The manual is organized into two parts. The first part is an introduction whose index follows what was established during the plenary meeting of March 22, 2012, at Piccolo Auditorium Reiss Romoli in Coppito (AQ). The index is the following:

- Purpose of the manual;
- Protection Strategies;
- Current Codes;
- Protection Systems;
- State of art;
    - Rate-independent devices;
    - Rate-dipendent devices;
- Dissipative braces: configuration and arrangement;
- Other types of device.

The second part, instead, lists the design methods. Regarding this, several papers were studied. These were analyzed and summarized in flow-charts and then divided for kind of device and design method. In particular the following devices are considered:

- Viscous devices;
- Viscoelastic devices;
- Hysteretic devices (yield or friction devices).

The identified design methods, instead, can be summarized as follow:

- target damping method, where a surplus of damping is established to be assigned to the structure by dampers;
- performance point method, where the bare frame is subjected to push over analysis. Comparing the behaviour of the structure with the seismic request, the dampers characteristics are designed.

Several study cases are presented.

4.4 ACTIVITY 4: Integration among semi-active control, monitoring and early warning systems

As part of the JETBIS project, investigations have been performed regarding the use of semi-active seismic protection devices integrated with monitoring systems and early warning.

The research group has produced advances in research on the possible integrated use of the technologies mentioned above. In particular, concerning the seismic protection system proposed during the first year, consisting in the smart passive use of variable (magnetorheological) dampers based on a prior knowledge of parameters measuring the seismic intensity of upcoming events, provided an early warning system earthquake be present, the research activity was preliminary to:

- Measure the robustness of the proposed control system over the uncertainties of the information (intensity measurements) from the early warning system and related to the upcoming earthquake.
- Investigate the possibility of adopting such a integrated technique of seismic protection for bridges, but also for existing buildings.

- Investigate the possibility of defining "regional" control algorithms which, again based on prior information provided by the early warning system, are able to lead to the optimal calibration of the devices according to the local seismic hazard and local response expected.

The research also focused on specific aspects related to the mechanical characterization, the response promptness and the dissipative capability of magnetorheological dampers. Further investigations were conducted, in particular, on the numerical modeling of the hysteretic behavior of these devices, the formulation and calibration of models (built in the Simulink) of structures with controlled semi-active techniques. These models are calibrated on the basis of experimental results and can be helpful to:

- Assess the functioning and effectiveness of control algorithms from literature.
- Investigate possible methods of calibration of these algorithms.
- Simulate the effectiveness of semi-active control when driven by information coming from a seismic early warning system.

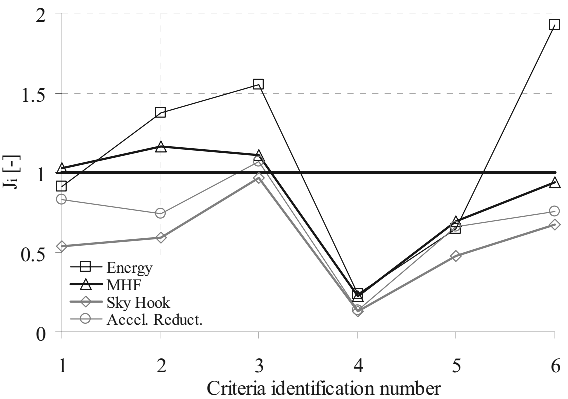


Figure 27. Final results of the comparative analyzes.

An analysis of the experimental data recorded during the campaign JetPacs carried out under the project ReLUIS I has been performed, with the purpose of comparing the effectiveness of the 4 control algorithms adopted for the seismic protection of a 2-storey steel frame equipped with 2 magnetorheological dampers, tested on a shaking table. The preceding figure 27 shows one of the final results of these comparative analyzes: it contains the measure of 6 performance indices Ji (ratio of the maximum values the analysis of the controlled and uncontrolled structure leaded to in terms of: 1. interstorey drift at the first level, 2. interstorey drift at the second level, 3. base shear, 4. control forces, 5. stroke of the devices, 6. sum of elastic and kinetic energies related to the use of each of the four control algorithms under the action of the same earthquake.

The research group during the third year produced further advances in research on the possible integrated use of technologies mentioned above. In particular, concerning the seismic protection system proposed and partially analyzed during the first two years, consisting in the smart passive use of variable (magnetorheological, MR) dampers based on prior knowledge of parameters measuring the seismic intensity of upcoming events, provided an early warning system earthquake, the research has been addressed to:

- assess the robustness of this control system against undesired, even possible, malfunctioning of the control chain (e.g. false alarms, uncorrect prediction of the seismic intensity, electrical black out and related problems of power supplying for the MR devices);
- generate “regional" control algorithms which, again based on prior information provided by the early warning system, are able to lead to the optimal calibration of the devices according to the local seismic hazard and local expected response.

Research has also focused on specific aspects related to the responsiveness of magnetorheological dampers, in particular by carrying out a statistical study of the delays in the response of two devices prototypes subjected to hundreds of tests and arriving, finally, to generate approximate formulas for the time delay prediction.
Still aimed at integrating technologies of a different nature, an experimental campaign was conducted by shaking table tests (at the laboratory of the RU of Naples Federico II) of a steel frame that, equipped with recycled rubber isolators, was "controlled" at the base with semi-active devices. These hybrid control tests were intended to show how the use of semi-active devices coupled to the base isolators can be useful to correct the modal shapes, in particular to the attainment of a first mode as tending to the rigid motion of the superstructure.

Figure 28. Experimental tests on shaking table.

5 DISCUSSION

The activities carried are fully in line with the timesheet reported in the initial proposal of Line 2.3 of Task 3 within RELUIS project. The planned objectives have been achieved and the results are satisfactory even though a further development on a larger sample of real structures is needed. The subdivision of the research project into activities and their subdivision in arguments and phases allows to check the development and the fulfillment of the proposed intermediate goals.
In the third year connection with other research units from this task and task AT3.1.3 have been reinforced, in particular relating to the JETBIS project (Joint Experimental Testing on Base Isolation Systems). Links have been established among UNINA, POLIBA, UNIBAS, POLIMI, UNIPARTH, UNICAL and UNIUD for the development of JETBIS project. The activity on the Observatory on isolated structures in L'Aquila has been conducted by means a cooperation between UNINA_DL and UNIVAQ.

6 VISIONS AND DEVELOPMENTS

The following activities have been planned on the seismic isolation:

1) Technical Reports tests first project JETBIS and reports summarizing the results of numerical simulations and comparison with experimental data. Taking up any new experimental activities supplementary to investigate the effects of bi-directional behavior of the devices under investigation and subsequent elaboration and updating of codes.
2) Technical report on the use of seismic isolation for the retrofitting of existing structures considering the hike in the plastic range of the superstructure.
3) Technical Reports of tests on seismic isolation devices based on innovative materials. Definition / upgrade of test protocols for the qualification / acceptance.
4) Design provisions and regulations for the control of unwanted movements of isolation systems due to residual displacements, differential motions and vertical displacements.
5) Technical Reports of experimental evidence relating to the applicability of seismic isolation precast structures.
6) Definition of design methods, procedures and software dedicated seismic isolation and proposed regulations developed under this project.

On the other hand, for the dissipation topic the following activities have been scheduled:

1) Guidelines with design methods for structures with energy dissipation systems based on: dissipative braces, coupled with seismic isolation systems and coupling of adjacent medium / high. Guidelines / additions regulations.
2) Technical Reports of tests on energy dissipation devices based on new materials and technologies. Definition / upgrade of test protocols for the qualification / acceptance.
3) Technical Reports of tests for systems of dissipation mechanisms associated with rocking for the seismic protection of prefabricated structures.
4) Review and update of the reference standards in relation to the definition of the properties of the system power dissipation, modeling and structural analysis and test procedures for the qualification / acceptance.
5) Definition of algorithms for integrated control systems, obtained by the combined use of semi-active devices for seismic early warning systems.
6) Definition of design methods, procedures, systems and software designed for energy dissipation and regulatory proposals developed under this project.

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DEVELOPING GUIDELINES FOR DISPLACEMENT-BASED SEISMIC ASSESSMENT

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1 INTRODUCTION

The need for accurate seismic assessment methods is particularly evident in Italy, a country that has historically suffered greatly from earthquakes. Recent seismic events such as the 2009 L'Aquila and the 2012 Emilia-Romagna earthquakes have increased public awareness of the risks posed by earthquakes, but there is concern that these events may be forgotten without changing attitudes and practices for seismic assessment and retrofit. Furthermore, these recent earthquakes are relatively small events for Italy if one considers the estimated 150000 people killed in the Val di Noto earthquake of 1693 or the 50000 people who lost their lives in the Calabria earthquake of 1783.

One might argue that modern engineering should certainly have reduced the risk of earthquakes with respect to the 17th and 18th centuries. However, in addition to historical buildings, a large proportion of the building stock in Italy was realized in the 1950’s and 1960’s during a period of rapid urbanization with few controls in place to ensure good quality construction and design solutions developed using building codes with very minimal seismic considerations.

Traditional seismic assessment methods have tended to rely on a simple comparison of estimated base shear capacity and base shear demand specified by a code (Priestley et al. 2007). The required code base shear is found by reducing the elastic base shear force corresponding to the elastic stiffness of the structure, by a code-specified force-reduction or behaviour factor. The actual assessed base shear strength is then estimated and compared with the demand to identify whether the structure has enough strength to survive the earthquake. As pointed out by Priestley et al. (2007), the problems with this approach are that: (i) no assessment is made of the actual displacement or ductility capacity, (ii) no capacity design check is included to determine undesirable failure modes, and (iii) no estimate is made of the risk of a structure which is deemed to fail the strength check.

In recognition of the limitations of force-based design and assessment methods, the Direct Displacement-Based Design (DBD) approach of Priestley et al. (2007) has been proposed. This is just one of many different displacement-based methodologies (see Sullivan et al. 2003) but Direct DBD is the most developed displacement-based procedure and recently a Model Code for the Displacement-Based Seismic Design of Structures (Calvi & Sullivan 2009, Sullivan et al. 2012) was published as part of the work by research line IV in the 2005-2008 RELUIS project. The research undertaken in the 2005-2008 RELUIS project highlighted that Direct DBD can be used to provide effective seismic design solutions through relatively simple calculations. In the 2010-2013 project, the focus shifted towards development of the assessment procedure and this paper reviews the principal developments made.

2 BACKGROUND AND MOTIVATION

The text by Priestley et al. (2007) presents the fundamentals for the Direct DBA approach, which is reviewed here with reference to Figure 1 (from Sullivan and Calvi, 2013). As other assessment procedures, the first step (Figure 1a) requires examination of the structure to identify material properties, member sizes and the general geometry. The Direct DBA approach utilises the substitute structure concept of Gulkan and Sozen (1974) and Shibata and Sozen (1976) to represent the structure as an equivalent Single-Degree-Of-Freedom (SDOF) system (Figure 1b), characterised by a secant stiffness, K_e, at the displacement capacity Δ_{cap} (or any other limit state of interest) as shown in Figure 1c. To do this, the designer must first assess (by comparing the relative strengths of members) the inelastic mechanism that is likely to develop. For example, in the case of a RC frame structure either a column-sway (such as that shown in Figure 1b) or a beam sway mechanism might develop, but the assessment could also need to identify how the effects of smooth reinforcement or undesirable lap-splice locations could affect the resistance or deformation capacity. Having identified the expected mechanism, the designer must estimate the displacement capacity associated with the mechanism. This requires consideration of both element deformation capacity and the system displacement shape. In addition, care may be required to properly account for higher mode effects on the likely mechanism and local deformation demands. The considerations required for the identification of the plastic mechanism, shear and displacement profiles for various structural systems have been an objective of the 2010-2013 project and developments made will be reviewed in Section 4.

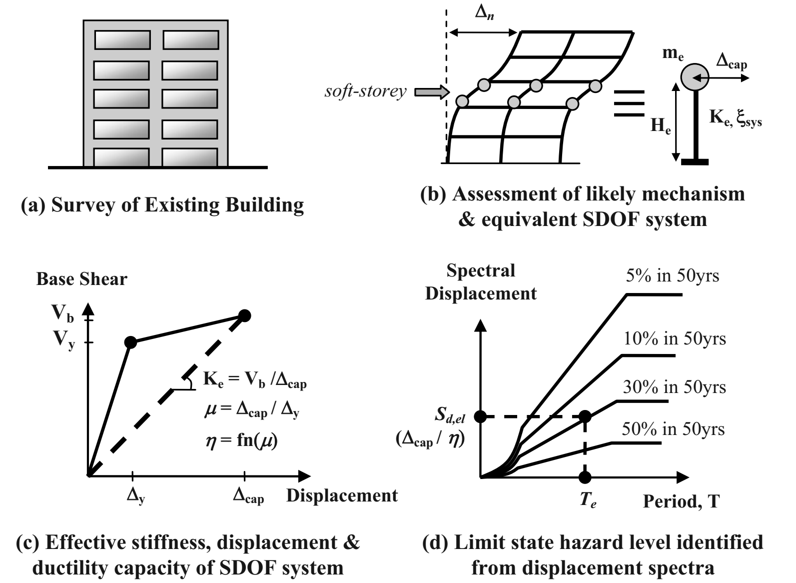


Figure 1. Overview of the Direct Displacement-Based assessment Approach (figure taken from Sullivan and Calvi, 2013).

Once the shear and displacement profiles at peak response have been established, the equivalent SDOF system characteristic displacement capacity, Δ_{cap}, effective stiffness, K_e, effective mass, m_e, and (if necessary) effective height, H_e, can be found using the following equations:

$$\Delta_{cap} = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \tag{1}$$

$$K_e = \frac{V_b}{\Delta_{cap}} \tag{2}$$

$$m_e = \frac{\left(\sum m_i \Delta_i\right)^2}{\sum m_i \Delta_i^2} \tag{3}$$

$$H_e = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \tag{4}$$

where Δ_i, m_i and h_i are, respectively, the displacement, seismic mass and height associated with level i of the structure, whereas V_b is the base shear resistance of the equivalent SDOF system at the system displacement value of Δ_{cap} (see Figure 1c).

The effects of energy dissipation and non-linear response are accounted for via the use of an empirical ductility-dependent scaling factor, η, which is divided into the equivalent SDOF system characteristic displacement capacity to give $S_{d,el}$, the equivalent elastic spectral displacement capacity for the same effective period, as shown:

$$S_{d,el} = \frac{\Delta_{cap}}{\eta} \tag{5}$$

As such, the η-factor represents the ratio of the inelastic displacement demand to the elastic spectral displacement demand at the effective period. Traditionally, this ratio is obtained as the combination of a ductility-dependent equivalent viscous damping ratio together with a damping-dependent spectral displacement scaling expression. However, as explained in Pennucci et al. (2011), the scaling factor (also known as a spectral displacement reduction factor) can alternatively be established directly as a function of the ductility demand. In either case, current expressions for the η-factor are empirical, calibrated to fit the results of numerous non-linear dynamic analyses on SDOF systems, and this has been a research activity during the current project, as explained in further detail in Section 4.

With the equivalent elastic spectral displacement capacity established for the assessed effective period, the earthquake intensity expected to cause the limit state to be exceeded is then established as shown in Figure 1d. The engineer, in consultation with the client and local code requirements, can then decide whether the seismic risk is acceptable or whether retrofit is required. Note that the assessed earthquake intensity is expressed in Figure 1d as a probability of exceedence in 50 years. As pointed out by Priestley et al. (2007), this form of the assessment procedure therefore provides information on the probability that a specific limit state is exceeded and for the case shown in Figure 1d it would have been deduced that the probability of exceeding the limit state during a 50 year period is around 29%. Clearly, an estimate of the probability of exceeding a certain limit state may be more useful than a simple pass-fail assessment approach that will fail to highlight the severity of any problems, if they exist. Nevertheless, as explained in Section 4, the probabilistic considerations made within the DBA procedure of Figure 1 can be improved.

A general benefit offered by the DBA procedure described above is that the engineer is required to consider the likely mechanism that will form and arrive at an estimation of the likely displacement capacity of the structure, considering both local and global deformation capacities. Such considerations are likely to improve the accuracy of seismic assessments. However, guidelines provided in Priestley et al. (2007) were relatively limited focusing on the general considerations required and without providing detailed guidelines for different structural systems. Such observations have motivated research into the DBA of a range of structural typologies, as will be explained in the next section.

3 RESEARCH STRUCTURE

The main objectives of research Line 2 of the RELUIS 2010-13 project have been to develop the general principles and detailed rules of the Displacement-Based seismic Assessment approach. In order to achieve these general objectives, the nine research areas listed in Table 1 have been identified and assigned to different Italian Universities who have strong competencies in the specific research areas. Each research area refers to a different structural typology, such that guidelines for DBA of the main structural typologies that exist in Italy can be developed.

Table 1. Displacement-Based Assessment Research Group for the 2010-2013 RELUIS project.

Research Area	Responsible University	Research Leader
1. General DBA aspects	Pavia	Calvi & Sullivan
2. RC Buildings	Bologna and Pavia	Benedetti and Sullivan
3. Pre-Cast RC Buildings	Bergamo	Riva
4. Masonry Buildings	Genova and Pavia	Lagomarsino and Magenes
5. Steel and Composite Structures	Naples Federico II & Pisa	Della Corte and Salvatore
6. Timber Structures	Trento	Zanon & Piazza
7. Bridges	Basilicata & Pol. of Milan	Cardone and Petrini
8. Retaining Structures	Perugia	Pane
9. Foundations & SSI	Polytechnic of Milan	Paolucci

The overall programme followed by the research line is shown in Figure 2. It is shown that in the first year the project aimed to select case study structures with subsequent consideration of typical mechanisms and deformation limits. The second year aimed to assess each structure using both traditional and displacement-based assessment procedures, with verification of performance via non-linear dynamic analyses. In the final year the research should include improvement of the displacement-based assessment guidelines and re-evaluation of their performance through the use of advanced non-linear analyses or experimental data. In addition, Direct DBA guidelines should be prepared together with a draft model code.

While Figure 2 does provide a good overview of the research programme for the research line, it should be noted that the state-of-the-art varies greatly from one structural typology to another and as such, while this overall research programme is representative of the activities programmed for each research area, the specific objectives of some research units varied slightly. This is particularly true for foundation systems and as such, the activities of the Polytechnic of Milan focused on developing improved means of estimating the stiffness and equivalent viscous damping of foundations, and then incorporating these developments within the DBA of bridge structures. In addition, the Polytechnic of Milan has assisted in providing a software for the selection of displacement-spectrum compatible accelerograms.

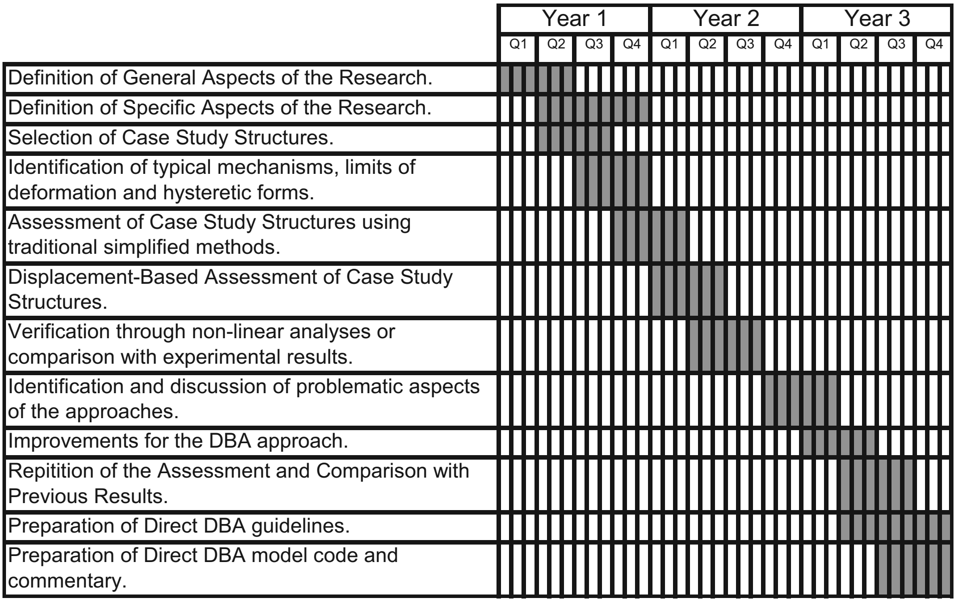


Figure 2. Overview of the research programme for the 2010-2013 RELUIS project.

4 MAIN RESULTS

The project has successfully made a number of developments to the DBA procedure which are presented in detail in the report edited by Sullivan and Calvi (2013). The following sub-sections first provide an overview of the general developments to the assessment approach, which is then followed by a description of the progress made for specific structural typologies.

4.1 Improved probabilistic considerations

The procedure explained in Section 2 (after Priestley et al. (2007)) provides an indication of the probability of exceeding the assessment limit state. However, the estimated probability appears to neglect the impact of uncertainties in the demand and capacity. As such, during the course of the 2010-2013 RELUIS project steps were taken to improve the probabilistic considerations being made in the DBA process.

There will be many uncertainties facing the seismic assessment of a structure. One of the most significant sources of uncertainty will clearly be the ground motion intensity but for existing buildings, characterisation of the structure might be considered equally uncertain. In probabilistic assessments, uncertainties tend to be classified as either aleatoric or epistemic and there are various means of dealing with them, as discussed in *fib* Bulletin 68 (fib, 2012) and elsewhere. Note that in the *fib* Bulletin a critical general discussion of probabilistic methods is provided. Reviewing the general DBA procedure proposed by Priestley et al. (2007) and described in the previous section, it is clear that the effects of uncertainty are not incorporated within the probability estimate obtained at the end of the process (as per Figure 1d). In this project it is proposed that a simple means of accounting for uncertainty within the DBA procedure (both for what regards the demand and the capacity) is to use the SAC-FEMA approach of Cornell et al. (2002) simplified in line with the recommendations of Fajfar and Dolsek (2010). The SAC-FEMA approach suggests that the probability, $P_{LS,x}$, of exceeding a given limit state can be established with an x-confidence level according to:

$$P_{LS,x} = \tilde{H}\left(S_{a,\tilde{C}}\right) C_H C_f C_x \tag{6}$$

where $\tilde{H}(S_{a,C})$ is the median value of the hazard function at the seismic intensity $S_{a,C}$, that causes a selected limit state to develop, C_x varies as a function of the confidence level desired, C_f accounts for the dispersion in demand and capacity and C_H transforms between mean and median hazard values. Fajfar and Dolsek (2010) point out that this calculation is considerably simplified if it is assumed that mean and median hazard values are approximately equal (such that C_H = 1.0) and that a 50% confidence level is sufficient (such that C_x = 1.0). Adopting these simplifications, the probability of the exceedence of a given limit state $P_{LS,x}$ with a 50% confidence level can be obtained as:

$$P_{LS,x} = \tilde{H}\left(S_{a,\tilde{C}}\right) C_f \tag{7}$$

where the symbols have been defined above. Note that in the context of DBA, the median value of the hazard function $\tilde{H}(S_{a,C})$ expected to cause a selected limit state to develop can be considered equivalent to the probability value that is being identified in the general DBA approach explained in Section 2. As such, improved consideration of uncertainties in the DBA process only requires evaluation of the dispersion factor, C_f.

Cornell et al. (2002) report that the C_f factor intended to account for dispersion (uncertainty) in demand and capacity can be computed, assuming log-normal distributions of demand and capacity, as:

$$C_f = \exp\left[\frac{k^2}{2b^2}\left(\beta_{DR}^2 + \beta_{CR}^2\right)\right] \tag{8}$$

where: b is a constant that relates the Engineering Demand Parameter (EDP) to the intensity measure and is typically taken as 1.0 (but in reality it should be updated as part of future research to account for different structural typologies); k is a constant (with values of around 2.0 or 3.0 typical in Italy) used in a power expression to relate the hazard with a probability of exceedence; and β_{DR} and β_{CR} are dispersion measures for randomness in demand and capacity respectively. Fajfar and Dolsek (2010) report that reliable data on dispersion is not yet available and they used a value of ($\beta_{DR}^2 + \beta_{CR}^2$) = 0.2025. The more recent ATC-58 (2011) document provides many different values of dispersion to account for different phenomena. Using k=2.0, b=1.0 and the dispersion values of Fajfar and Dolsek (2010), one finds from Eq. (7) that the estimated probability of exceeding the key limit state is 1.5 times that estimated without account for uncertainty. This gives an indication of the effect that accounting for uncertainty can have on the assessed probability and note that, formulated in this way, it always leads to an increase in the likelihood of exceeding a given limit state. The accuracy of the SAC-FEMA approach is limited (see Aslani and Miranda (2005)) but it does permit consideration of uncertainty and therefore its implementation within the DBA procedure is considered to be a useful development that could be implemented into national codes to help engineers make a transition into more probabilistic seismic assessment procedures.

4.2 Relationships between inelastic and elastic spectral displacement demands

Another general development made during the research project has been to further develop simplified expressions for the evaluation of residual and maximum displacements of structures subjected to seismic actions. These expressions can be of help to the seismic design of new structures and in particular the seismic verification of existing structures. In the last year the analysis database has been increased to include systems with the following features:

- hysteretic cycles (Takeda, Bilinear, Flag, SINA)
- 10 structural periods (0.1s, 0.2s, 0.3s, 0.4s, 0.5s, 0.6s, 0.8s, 1s, 1.5s, 2s, 2.5s, 3s, 3.5s, 4s)
- 14 levels of lateral resistance of the structure (values between 2.5% and 50% of the vertical load)
- 4812 real accelerograms.

The characteristics of the systems were chosen to represent a wide range of possibly existing structures. The high number of accelerograms was kindly provided by Dr. Stafford (Imperial College London). These records are of high quality and have already been suitably filtered and used with success in previous studies (Stafford et al. 2008). The huge amount of data generated for this study from about 2.7 million non-linear dynamic analysis was synthetically contained in a few hundreds of megabytes of data, where each maximum displacement and residual displacement is related to the system and accelerogram input that generated them. This database has been used to investigate the maximum displacement and the residual inelastic displacement.

An example of the value of the results obtained in this work is represented by the following figure, which shows best-fit lines obtained for the spectral displacement reduction factors (Figure 3a) and median values of the ratio of the residual to maximum displacement (Figure 3b) for different hysteretic models. A full summary of the results obtained are included in Sullivan et al. (2013).

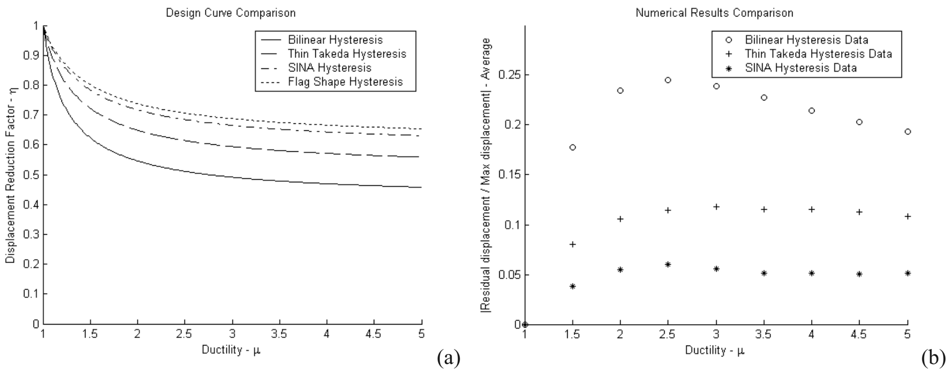


Figure 3. Summary of displacement reduction factors and residual displacement ratios (from Sullivan et al. 2013).

4.3 Extending the assessment procedure for the quantification of monetary losses

The research project has made some progress in extending the methodology to permit simplified estimation of direct monetary losses, simplifying and adapting the PEER framework. This started with the proposal by Sullivan and Calvi (2011) that a simplified displacement-based building-specific loss assessment could be undertaken using simplified loss models. This idea was extended by Welch et al. (2012) with the proposal of a four-point loss model, such as that shown in Figure 4a, and in the last year of research it was successfully applied to another case study building with results published in Welch et al. (2014).

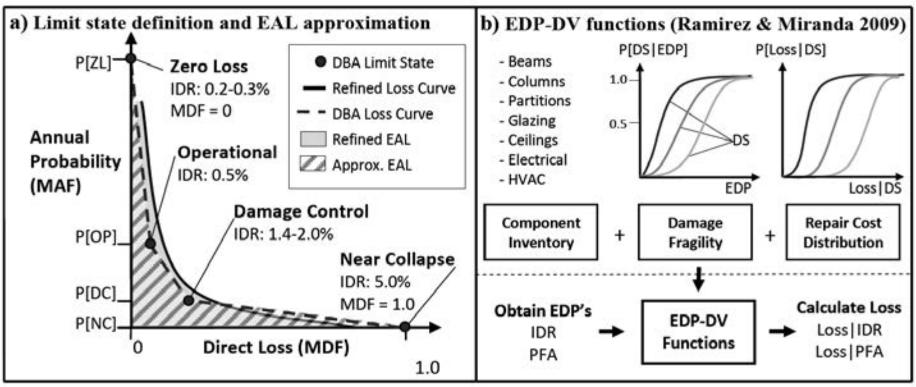


Figure 4. Key limit states used to define simplified loss model (a) and EDP-DV functions (b, after Ramirez and Miranda, 2009) for estimation of direct loss using a Direct DBA approach (Welch et al. 2014).

In the approach by Welch et al. (2014) the probability of exceeding four key limit states are assessed using the DBA procedure, with uncertainty in the demand and capacity accounted for using a simplified form of the SAC-FEMA approach (described in Section 4.1). Direct losses due to deformations (storey drifts) and accelerations at the selected limit state are computed using EDP-DV functions (see Figure 4b) proposed by Ramirez and Miranda

(2009). The total expected loss is then computed by integrating the loss model of Figure 4a. By limiting the seismic assessment to four points (limit states) the amount of work required of obtain a loss estimate is relatively limited and the results obtained for two case study buildings (see Welch et al. 2012), suggest that the procedure is promising. However, for assessment of existing structures in Italy, there remain a number of significant uncertainties, not least of which is the cost to be associated to different damage states. As such, it is considered that while the research project has permitted good conceptual development of a simplified building-specific loss assessment approach, further research is required in order to make the new tools applicable in Italy.

4.4 Selection of spectrum-compatible accelerograms

The Polytechnic of Milan also contributed to general developments via the improvement of the REXEL-DISP software, developed in cooperation with the University of Naples Federico II for the selection of displacement-spectrum compatible ground motions and available in the ReLUIS web site (www.reluis.it). A detailed description of the research done is provided in Paolucci et al. (2013a).

This activity lead to the publication of Smerzini et al. (2013) and saw a number of modifications and improvements to the current version of REXEL-DISP (v1.2), where a number of additional features were included, namely:

- Re-processing of records: raw acceleration time histories were re-processed relying on the procedure used to process records in the Italian strong-motion database ITACA, with special care to better define the filter bounds and to ensure compatibility of corrected records, in the sense that single and double integration of the corrected accelerograms produce velocity and displacement time series with zero initial conditions and without unphysical baseline trends.
- Enlarge the number of records: an important set of new records was added, with special care to near-field conditions, including the Emilia earthquake sequence and an updated set from the Christchurch earthquake. The SIMBAD database presently consists of 467 three component records from 130 earthquakes worldwide. Most records come from Japan (47%), Italy (18%), New Zealand (17%), and USA (9%), with minor contributions from Greece, Turkey, Iran and other European countries (9%).

The new tools available for record selection should help practicing engineers interested in undertaking non-linear dynamic analyses for the assessment of seismic performance.

4.5 Displacement-based assessment of RC structures

Research undertaken into RC structures has concentrated on different aspects: study and application of the general DB assessment procedure for bare frames with particular reference to the prediction of the collapse mechanism; nonlinear static and dynamic analyses of different types of infilled multi-storey frames for studying the collapse mechanisms and the displacement profiles; proposal of a new DB assessment procedure for infilled RC frames. The findings of this part of the research are reported in detail by Landi and Benedetti (2013).

The work included examination of a number of bare RC frame case study structures: a five storey three bay frame, a five storey five bay frame and a ten storey three bay frame. Both DBA and pushover analyses were conducted considering the formation of a global collapse mechanism. For all the analysed frames comparisons between the DBA procedure and nonlinear dynamic analysis results were made. Moreover an approach has been proposed

based on limit analysis for the rapid prediction of the collapse mechanism. The application of this approach has provided results in agreement with those of pushover analyses.
Furthermore the application of the DBA procedure to infilled RC frames has been studied. According to the Displacement Based procedure, the seismic assessment of existing buildings requires the definition of the damping of the examined structure. To this purpose in the second year an extensive campaign of nonlinear dynamic analyses had been carried out. From the results of these analyses it was possible to derive proper ductility-damping laws for infilled frames which could be used in the seismic assessment, without knowing in detail the real response of the infilled frames in terms of stiffness and strength. Moreover in the second year an equivalent strut model was calibrated on the basis of comparisons with available experimental results relative to monotonic and cyclic loading cases. This model was applied for analysing a five storey RC frame with and without the presence of masonry infills. Pushover analyses have been performed in order to obtain the response in terms of base shear-top displacement and to evaluate the configuration at collapse and the displacement profile. In the third year, for the same structures, nonlinear incremental dynamic analyses were also performed. Through the execution of incremental dynamic analyses it has been possible to evaluate the average peak ground acceleration at collapse: for the bare frame 0,544g and for the infilled frame 0,767 g. As seen in Figure 5, the predictions obtained via the IDA approach correlate very well.

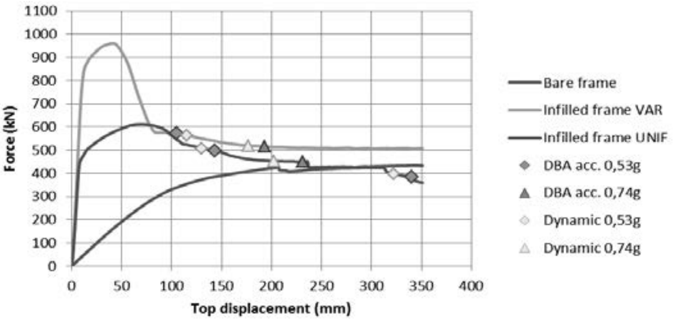


Figure 5. Comparison between the displacement demand obtained with the nonlinear dynamic analyses and the one derived with the proposed DBA for infilled frames.

Overall, the research undertaken through the course of this three year research project has lead to the formation of useful guidelines for the assessment of RC structures, and it has been shown that for existing RC structures, the DBA approach can be an effective and practical assessment tool.

4.6 Displacement-based assessment of pre-cast concrete structures

Guidelines for the displacement-based assessment of pre-cast concrete structures were developed by Torquati et al. (2013) and included examination of both single and multi-story precast (typically industrial) buildings with consideration of the peculiarities of these buildings, related to the influence of the connections between the structural elements and to the high deformability offered by the static scheme mainly identified by columns hinged to the beams.
As reported in Torquati et al. (2013) guidelines have been provided for the definition of the force-displacement and moment-rotation relationship of the joints between pre-cast beam-

column elements, utilizing formulations available in the literature. Furthermore, the formulation for the yielding curvature of reinforced concrete columns provided by Priestley (2003) has been recalibrated with a series of moment-curvature parametric analyses on different cross-section types, in order to provide more adequate expressions for the columns used in this type of construction. The high deformability which characterizes this type of structures can significantly intensify the second order effects, and for this reason, a study on P-Δ effects has also been performed as reported in Torquati et al. (2013).

Considering the general displacement-based assessment procedure for precast buildings, Torquati et al. (2013) developed two methods for the evaluation of the inelastic displacements profile of the structure: a Pushover Method (PM) and an Equivalent Column Simplified Method (ECSM). The effectiveness and applicability of the proposed DBA procedure for existing precast buildings has been evaluated considering two case studies and has been compared with other seismic assessment methods available. The case studies considered are representative of a three-story precast building, whose seismic vulnerability is evaluated with different analysis methods and then compared with each other in terms of the estimated PGA required to reach the assessment limit state. The assessment methods utilised in this work include the: Pushover (N2 Method), DBA – Pushover Method (DBA-PM), DBA – Equivalent Column Simplified Method (DBA-ECSM), and an Incremental Dynamic Analysis (IDA) approach. The beam-column connections are considered as dowel connections with neoprene cushions at the supports. The following figure shows one of the case study structures examined (case study A).

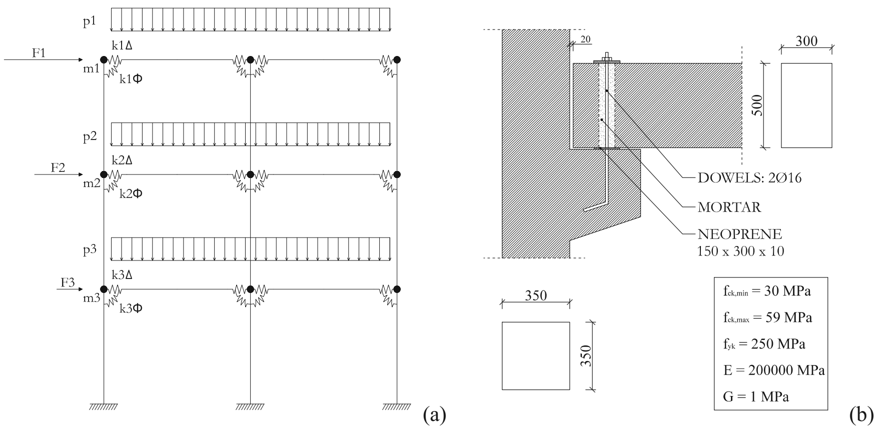


Figure 6. Elevation (a) and connection detail (b) of case study A pre-cast building (after Torquati et al. 2013).

In order to evaluate the influence of the stiffness of the structure during the calculation of the PGA associated with the ultimate limit state, a second case study was subsequently evaluated, assuming column cross-sections of 60x60cm and a stiffness of the beam-column connection four times greater at the same strength (case study B).

IDA were carried out to evaluate the effectiveness of the proposed procedure, and the limit state PGA has been obtained as the mean value of the results of 7 selected ground motions spectrum compatible in displacement with the target spectrum (Serra Pedace – Cosenza): a_g=0.276g, F_0=2.438, T_C*=0.374, S=1.296, C_c=1.453.

The analyses on the case studies provided the results shown in Table 2:

Table 2. Assessment results obtained by Torquati et al. (2013) for pre-cast concrete buildings.

	Case Study A – PGA (g)		**Case Study B – PGA (g)**	
	No P-delta	With P-delta	No P-delta	With P-delta
Pushover – N2	0.274	0.253	0.516*	0.516*
DBA - PM	0.319	0.316	0.507	0.484
DBA – ECSM	0.276	0.260	0.310	0.310
IDA	0.315±0.050	0.307±0.048	0.493±0.093	0.471±0.076

* PGA corresponding to the maximum return period of the site considered: higher PGA values not included in the available site data.

The ultimate limit states associated to the PGA of the previous table refer to the failure of a beam-column connection for case study A, and to the failure of one of the base columns associated with the ultimate rotation capacity for case study B.
Taking as a reference the results provided by IDA, the Pushover-N2 method shows unreliable results: the structural response in case A is underestimated (conservative solution), but it is overestimated in case B. This method does not take into account the effective damping of the system.
Regarding the assessment procedures, conservative results are obtained using the simplified approach DBA-ECSM, which underestimates the PGA of the dynamic analysis by about 15% for case A and about 50% for case B; the approach DBA-PM leads to a good PGA estimation, with about 3% error in both cases.

4.7 Displacement-based assessment of masonry structures

Developments for the displacement-based assessment of masonry structures have been made by Cattari and Lagomarsino (2013). After a general review (first year) of the issues to address for the DBA of masonry structures, with respect to both the global and out-of-plane seismic response, the research mainly focused on the global response. Moreover, the issue of seismic assessment has been faced also according to a probabilistic approach. In the second year, both specific modelling tools (multi-linear constitutive laws for masonry panels on phenomenological basis which have been implemented in the Tremuri software) and criteria for the definition of Limit States (a multi-scale approach based on combined checks at element, macroelements and global scales) have been developed. The main aim was to provide tools that could simulate more reliably some of the relevant features of existing masonry buildings, such as weak spandrels, flexible floors, irregularities: factors that greatly affect their seismic response more than in the case of the new buildings. Finally, in the third year, after the validation of such proposed tools, nonlinear parametric analyses on some prototype configurations were performed in order to establish correlation laws between Limit States and some of the entities necessary for the application of DBA (e.g. damping and deformed shapes).
The reliability of the multi-linear constitutive laws (phenomenological basis) for masonry panels implemented in the Tremuri software (that works according to the equivalent frame approach), and that of a multi-scale approach for the definition of Limit States on the capacity curve have been validated through the simulation of the seismic response of a real building located in San Felice sul Panaro and seriously damaged by the May 29, 2012 event. Both non-

linear static and dynamic analyses have been performed. Results highlighted a good agreement with the real response that occurred (see Figure 7); moreover, the damage level simulated (based on the comparison between the maximum displacement obtained from the nonlinear dynamic analysis and the SL thresholds assessed through the multi-scale approach) complies with that observed.

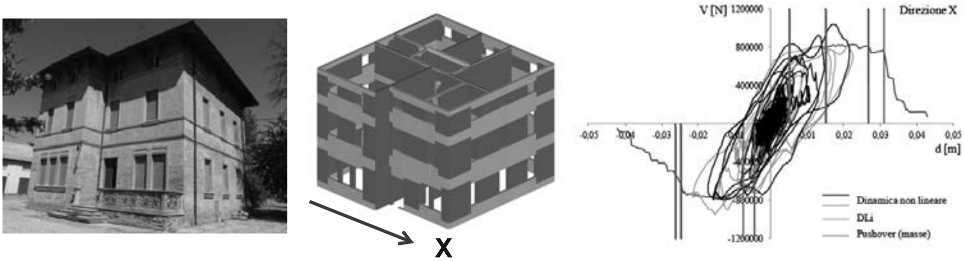


Figure 7. Numerical simulation (with Tremuri program) of a masonry case-study building located in San Felice sul Panaro (from Cattari and Lagomarsino 2013).

The multi-linear constitutive laws developed were found to be capable of simulating different types of hysteretic behaviour as a function of various prevailing damage modes (if flexural, shear or mixed) and two types of masonry panels (pier and spandrel). They constitute a very useful and versatile tool to describe some of the specific features of various recurring global seismic responses of existing masonry buildings (as testified by the damage survey) and to characterize them in terms of different hysteretic properties, ductility and collapse shapes, as seen in Figure 8.

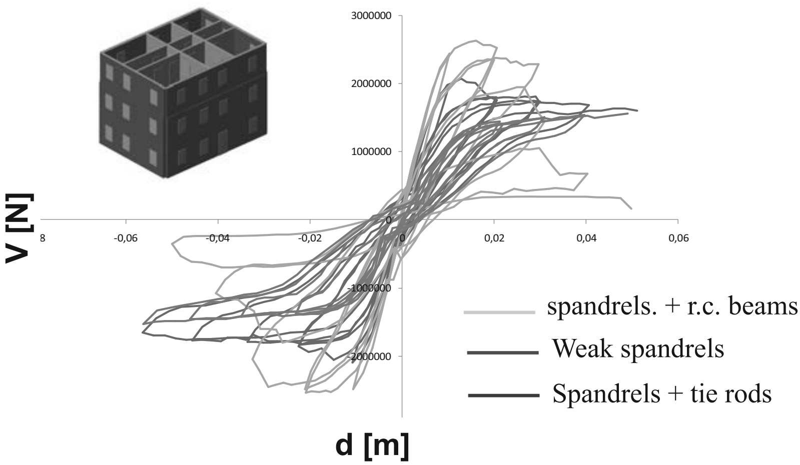
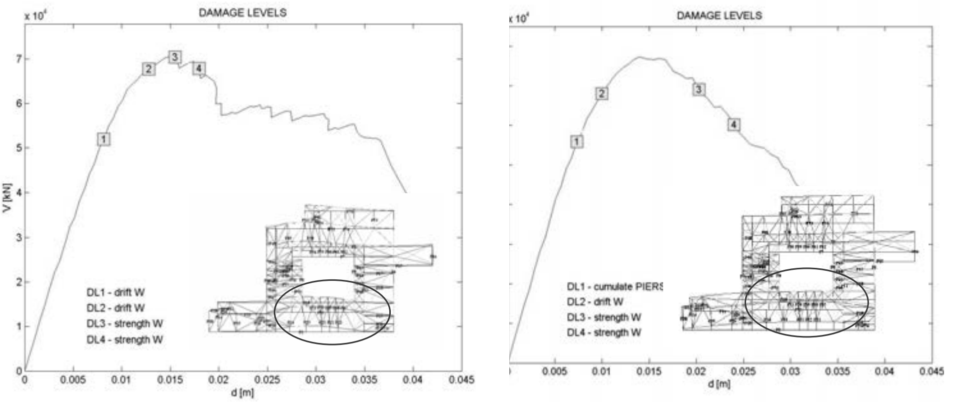


Figure 8. Capability of multi-linear constitutive laws to simulate (through nonlinear cyclic static analyses) some of the specific features of existing masonry buildings (Cattari and Lagomarsino 2013).

Once validated, the aforementioned tools have been adopted for the subsequent research activities that undertook nonlinear parametric analyses on some prototype configurations. The final aim of this activity was to define proper correlation laws - applicable to the case of existing masonry buildings - between Limit States and some of the entities useful for the application of the DBA (e.g. damping, story drift, deformed shapes). The analyses have been carried out as a function of: five prototype configurations; two classes of masonry type; flexible or rigid floors; with or without the presence of some recurring strengthening interventions (e.g. tie-rods). The results achieved allow to differentiate such laws as a function of factors related to the irregularity and floor stiffness. For example, results highlighted that, in the case of flexible floors, the definition of a global limit state tends to be affected more by the checks performed at element and macroelement (e.g. masonry walls) scales, that tend to move the limit state position further back on the capacity curve than in the case of rigid floors, as indicated by Figure 9.



(a) (b)

Figure 9. Example of the influence of the floor stiffness on the definition of limit state within the pushover curve: (a) flexible and (b) rigid floors (after Cattari and Lagomarsino 2013).

4.8 Displacement-based assessment of steel structures

Displacement-based seismic assessment guidelines for steel structures have been developed by Della Corte et al. (2013a, 2013b). The percentage of steel structures within the whole Italian building stock is fairly low. However, a significant portion of the industrial buildings in Italy are made of steel, and the risk of business interruption due to earthquakes may be significant in case of industrial buildings. This last statement is strengthened by consideration of the larger deleterious effects of damage to one or few buildings on the complete production chain in a given industrial sector.

As a result of the above, the research conducted by Della Corte et al. (2013a, 2013b) was focused mainly on issues related to modelling and response analysis of steel structures typical of industrial buildings. However, a more general study was carried out with reference to bolted end-plate connections which are frequently encountered in any type of steel structure.

The research into beam-column joints by Della Corte et al. (2013a) lead to the realisation of simple methods to characterize the mechanical behaviour of bolted end-plate beam-to-column joints. The method currently adopted by Eurocode 3 is well known and named the “component method”. It decomposes the joint response into the assemblage of response of simpler components. The stiffness and strength of each component is estimated analytically and all the components are then assembled together based on simple kinematic assumptions. This is an analytical and conceptually general method but requires time for implementation and does not provide general and quick information about trends of mechanical characteristics with key joint parameters. Through parametric analysis of typical beam-to-column joints and using the component method, the possibility to derive simple closed-form equations to evaluate the stiffness and strength of preselected joint configurations has emerged. Such closed-form equations could provide designers and analysts with simple and ready to use tools to evaluate the key joint mechanical characteristics. Differences between experimental results and theoretical predictions have been shown to be appreciably larger than differences between the component method and the simplified equation predictions. It is concluded that further research is justified, because the component method is laborious to use and requires many calculations whereas the alternative method being formulated allows one to make simpler (faster) calculations that should provide reasonable accuracy. Clearly there is a trade-off between simplicity, accuracy and generality, and future research should aim to strike the right balance between these factors.

The other important area of research for steel structures included the detailed examination of an industrial case-study building, with results published by Della Corte et al. (2013b). It was found that a careful evaluation of the column base connections was paramount for the behaviour of the whole system. Stiffness, strength and deformation capacity of such connections need assessment in order to evaluate the structural performance. Such an assessment of connection characteristics can be difficult because of the differences in geometry between existing connections and standard types covered by current Structural Codes. In addition, even in the case of connections conforming to standard types, no explicit analytical method is currently available to evaluate the deformation capacity. As far as strength and stiffness is concerned, extensions/adaptations of available methods have been proposed, based on simple mechanical principles. The validity of such methods needs evaluation through experimental tests. Another important observation made by Della Corte et al. (2013b) was that some mechanisms could only be reproduced in non-linear dynamics analyses when 3D models were used, suggesting that some guidelines for modelling requirements could be developed as part of future research.

4.9 Displacement-based assessment of composite steel-concrete structures

Developments for the displacement-based seismic assessment of composite steel-concrete structures have been made by Morelli and Salvatore (2013). The University of Pisa unit group developed a beam-to-column joint cyclic model starting from the component method proposed by Eurocode 3 (for steel structures) and Eurocode 4 (for composite structures). Each joint component has been modeled by a suitable force-displacement or moment-rotation relationship, while the concrete slab was schematized by fiber elements in order to model, as accurately as possible, the non-linear behavior and the crushing of the concrete (Figure 10). The characteristics of each joint component was calibrated against the results of experimental tests executed on substructures in order to evaluate the capacity of the model to represent the actual behavior of the joint.

Starting from what was presented in Braconi et al. (2007) a component model was developed to reproduce the observed response of the entire sub-assemblage beam-to-column test specimens, including beam and column flexural behaviour. In the model, the connections between the beam endplates and the column flanges were represented by equivalent T-stubs localized at top and bottom beam flanges. As shown in Figure 10 for the interior joint, the model accounted for the response of: the unconfined concrete in compression (1), the confined concrete in compression (2), the lower T-stub (3), the upper T-stub (4), the wire mesh (6), the reinforcing bar (7) and the concrete in tension (11). Two rigid elements were introduced to simulate the connection between the composite beam and the joint. A fiber representation was adopted for the concrete slab in compression and in tension to adequately capture the non-uniform stress distribution over the slab thickness.

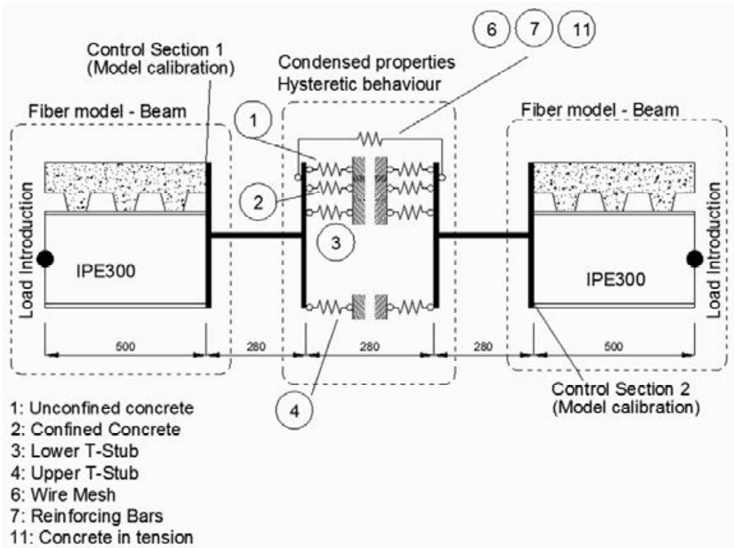


Figure 10. Illustration of the component model developed for composite beam-column joints (from Morelli and Salvatore 2013).

The model shown in Figure 10 was accurately calibrated to the results obtained from cyclic tests on joint sub-assemblages executed in Pisa. The development of new guidelines for the modelling and non-linear analysis of beam-column joints composite structures is considered to represent a valuable development for displacement-based assessment since it will permit improved understanding and evaluation of the force-displacement response of composite structures.

4.10 Displacement-based assessment of timber structures

Guidelines for the displacement-based seismic assessment of timber structures have been provided by Loss et al. (2013). The approach proposed by Loss et al. (2013) is based on the definition of simplified models for calculating the structural capacity, specific for the most likely failure mechanism and the reference limit state (ultimate and serviceability). These models allow evaluation of the displacement, force and energy dissipation capacity of shear wall elements, where the concept of “shear walls” extends to both framed and cross-laminated timber panels. Models have been developed both for the serviceability and ultimate limit states. The capacity of the individual wall elements is extended to a known structural system through simple analytical models which allow to estimate the limit displacement and deflection of the structure for a given structural mechanism. The deflected global shape can be estimated using simplified formulas similar to those proposed for displacement-based design of timber buildings. The most critical structural failure mechanism is identified with the aid of so-called mechanism indices, which suggest the most likely failure mechanism based on the mechanical-geometrical properties of walls and connections and on the load configuration. Figure 11 illustrates two different mechanisms that can be expected for timber framed wall structures.

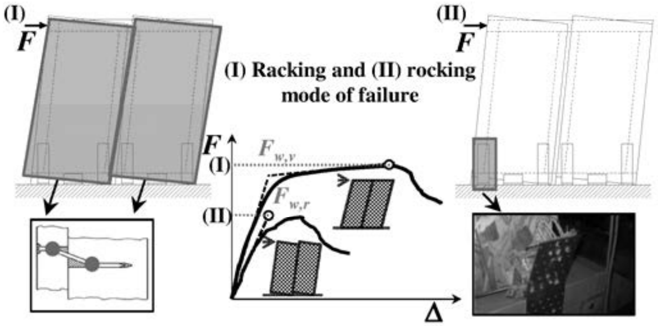


Figure 11. Racking and rocking mechanisms for framed timber wall structures (from Loss et al. 2013).

The proposed method has been validated, first, via numerical simulation and, then, through a comparison with the outcomes of laboratory results. In addition, the procedure has been validated on a real case study timber building. Figure 12 compares displacement profiles obtained from non-linear dynamics analyses with those predicted using DBA. The method in the present form is directly applicable to buildings where second order effects and torsional effects can be neglected. In addition, the effectiveness of the procedure is conditional to the possibility of identifying the most critical failure mechanism or at least the most likely.

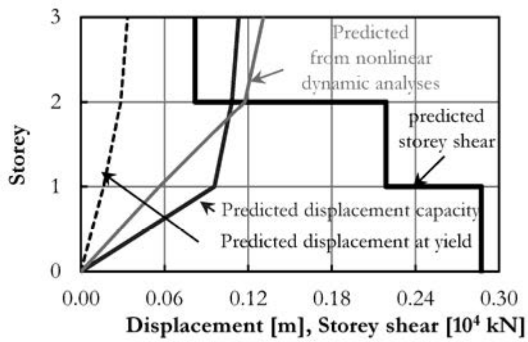


Figure 12. Assessment results for a three-storey case study building (from Loss et al. 2013).

In a strict sense, the assessment method proposed by Loss et al. (2013) has been conceived for timber buildings which are regular in plan and elevation. Nevertheless, the method could be applied to buildings which do not satisfy regularity criteria, provided that the engineer carefully evaluates the analysis results.

4.11 Displacement-based assessment of bridges

A comprehensive procedure for the Direct DBA of existing bridges has been developed by Cardone and Perrone (2013). The fundamental step of the proposed procedure is the definition of the so-called Performance Displacement Profile (PDP) of the bridge, corresponding to the inelastic bridge deformed shapes associated with the attainment of selected Damage States (DS's) in some critical elements of the bridge.

In the work by Cardone and Perrone (2013), the Displacement Limits associated with different DS's of the piers, abutments, joints, bearing devices and shear keys have been defined and comprehensively discussed. Moreover, a number of alternative approaches for the definition of the PDP have been examined, including: (i) Displacement Adaptive Pushover (DAP) analyses, (ii) Effective Modal Analysis (EMA), (iii) analysis of Individual Pier-deck Models (IPM), for bridges with simply supported independent adjacent decks and (iv) rational analysis, for continuous deck bridges. Finally, several aspects related to bridge modelling, including the selection of a suitable skeleton curve and effective damping ratio for each structural member, have been discussed.

Cardone and Perrone (2013) applied the proposed DDBA procedure to a set of eleven bridge configurations, differing in pier layout, deck type and bearing device characteristics. The predictions of the proposed DDBA procedure have been compared with the results of accurate NonLinear Response time-History Analysis (NLRHA), carried out on refined numerical models of the bridge, using two sets of seven accelerograms compatible (on average) with given reference response spectra scaled to the PGA values provided by the DDBA procedure for different Performance Levels (PL's) of the structure. Figure 13 illustrates the model developed to assess the non-linear dynamic response of the Kavala bridge, from the Greece Egnatia motorway.

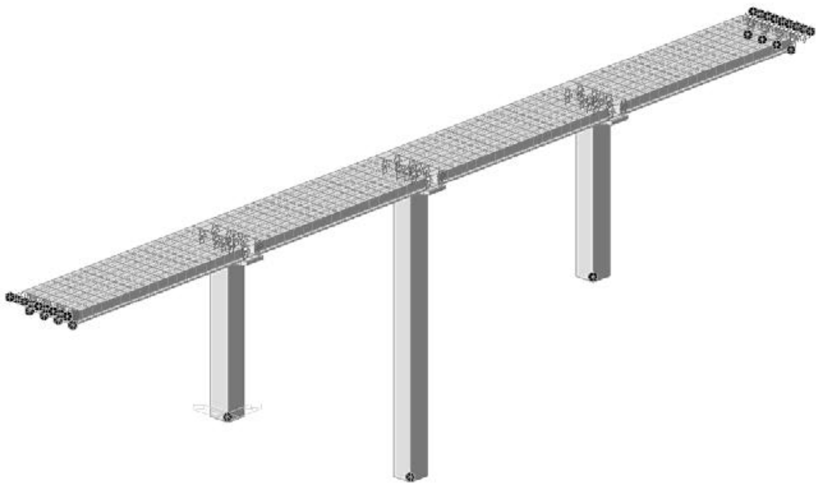


Figure 13. Analytical model of the Kavala bridge (from Cardone and Perrone 2013).

The comparisons between DDBA predictions and NLRHA results reported by Cardone and Perrone (2013) confirm the good accuracy of the proposed procedure in predicting the PGA values associated with slight-to-severe DS's of piers, bearing devices and abutments. In all the examples considered, indeed, the DDBA procedure was found to correctly identify the critical element of the bridge, where a first given DS is reached. Moreover, the PDP of the bridge assumed in the analysis was seen to be in good accordance with the maximum deformed shape of the bridge obtained from NLRHA. Figure 14 provides a sample of results obtained, comparing the peak displacements assessed by different methods for the Kavala bridge at the moderate (repairable) damage limit state.

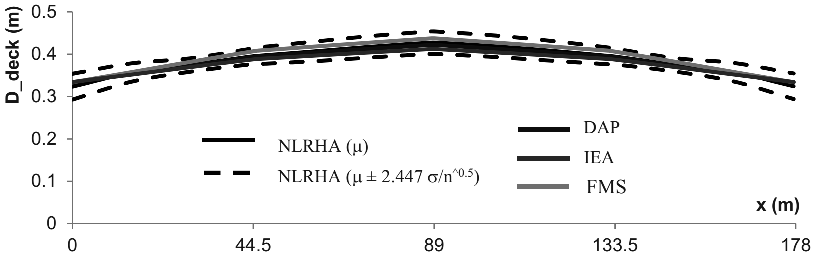


Figure 14. Comparison of peak displacements predicted for the Kavala bridge at the repairable damage limit state (from Cardone and Perrone 2013).

4.12 Allowing for soil-structure interaction

An iterative pseudo-static seismic assessment procedure for multi-span reinforced concrete bridges, based on the DDBA procedure coupled with Non-Linear Soil-Structure Interaction (DDBA + NLSSI) was developed by Paolucci et al. (2013b) during the project, which allows the introduction of non-linear dynamic soil-structure interaction effects using an equivalent linear visco-elastic approach. The procedure is iterative in order to arrive at a displacement profile (that includes effects of foundation deformations) and an internal force distribution that is consistent with the inertia force distribution that the displacement shape implies. The procedure has been developed for SDOF (single pier) and MDOF (whole bridge) systems subjected to longitudinal or transversal seismic action and has been applied to hypothetical SDOF/MDOF systems and to a real existing bridge; the Fiumarella Bridge.

For the validation of the DDBA+SSI procedure, IDA were used with SSI being taken into account using a macro-element model. A preliminary study aimed at validating the macro-element with experimental results and with numerical results obtained with another software named CHOPIN_F10 was performed.

During IDA, for each level of seismic intensity and corresponding scale factor considered, the maximum top displacement at each pier was recorded and compared with its displacement capacity obtained from the DDBA+SSI procedure. When one of the pier tops reached the displacement capacity, the corresponding scale factor was recorded as a Capacity/Demand ratio and the envelope of the pier/abutment displacements was collected as the "bridge displacement profile". The mean value of Capacity/Demand (C/D) ratios for the considered accelerograms was taken as the global C/D ratio for the structure being assessed.

The comparison between the C/D ratios obtained from DDBA+SSI procedure and from the IDA has given a quantitative indication of the procedure's accuracy. Moreover its accuracy has also been evaluated comparing the bridge displacement profile expected based on DDBA+SSI with the average of the displacement envelopes collected for the considered accelerograms.

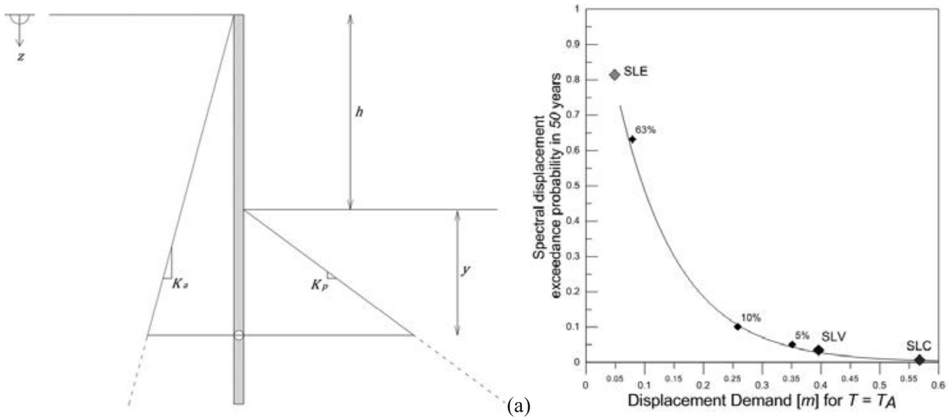
For the cases studied, the errors in terms of C/D ratio were found to be in general lower than 5% (only in 1 case it reached 7.2%). The errors in terms of displacement profile are always

lower than 20% for the piers but they can reach very high values (up to 300%) for the abutments indicating a limit in the procedure in describing the abutment behaviour.
To gain better insight into the likely effects of SSI for bridge pier response, the assessment of four bridge piers designed with a minimum consideration of horizontal loading (design against wind loading) were also considered for assessment of the seismic demand against earthquakes with different probability of occurrence. Both a simplified procedure and non-linear time history analyses were considered to evaluate the role of the non-linear behaviour of the foundation system on the overall seismic assessment of the structure. For this purpose three cases have been investigated (all of them with non-linear behaviour of the superstructure): 1) fixed-base; 2) flexible foundation with linear behaviour; 3) flexible foundation with non-linear behaviour. The parametric analyses highlighted that for low-rise piers, tentatively with height $H < 15$ m, the role of non-linear foundation response may indeed be critical in reducing the ductility demand and in changing assessment outcomes, with respect to piers with fixed (or linear-elastic) foundations.
In conclusion, the research into foundations and SSI has been focused on the case of bridge structures and has shown that SSI effects can be very significant. The work has also indicated that a DBA approach can provide good indications of the effects of SSI and should therefore be developed further as part of future research, developing tools to deal with other foundation typologies (e.g. piled foundations) and other structural systems (e.g. buildings).

4.13 Retaining structures

Proposals for the displacement-based seismic assessment of retaining structures have been developed in this project by Cecconi and Pane (2013). The research undertaken included the development of a normalization-procedure for simplified application of the displacement-based method to retaining structures. The proposed procedure provides non-dimensional charts and equations for the seismic thrust, thus allowing the designer to skip the mass discretization and the associated displacement profile, and to use simple (non-dimensional) reduction factors of the seismic thrust as a function of the design displacement. The procedure is aimed at analysing the comparison between the displacement capacity *vs.* the displacement demand and, at the same time, at evaluating the proper design solution.
Different strain fields were considered in order to improve and rationalize the definition of the equivalent damping ratio for the whole soil/retaining structure system. Cecconi and Pane (2013) developed a simple equation for the equivalent damping of a retaining wall system as a function of the peak displacement by assuming that the non-linear stress-strain behaviour of the soil under monotonic loading could be described by the Ramberg-Osgood model with the Masing criterion used to model the hysteretic behaviour upon cyclic loading. The resulting damping relationship is recommended by Cecconi and Pane (2013) for use with all cantilever retaining structures in non-plastic soils.
The research also investigated the general applicability of DDBA to cantilever retaining structures. In the initial stage, and for different limit states, the assessment displacement (occurring at the top of the wall) was defined based on available structural details. A plastic hinge is assumed to form at the RC wall section where the curvature attains its maximum value. The CUMBIA code (see Priestley et al. 2007) was used to build a simplified bi-linear moment–curvature relationship and then the force-displacement response was calculated. The displacement profile for the whole system was then tested by means of soil/structure interaction numerical analyses, by varying the relative stiffness of the components. The comparison between the capacity displacement and the demand displacement or, in other

words, the evaluation of the C/D ratio represents the vulnerability of the system for the considered limit state.
The assessment procedure was applied by Cecconi and Pane (2013) to different case-study retaining structures of the form indicated in Figure 15a, embedded in cohesion less medium-dense sands. The results of the assessment procedure for one of the case study walls are summarised in Figure 15b, which shows the assessed displacement demands and the exceedence probability in 50years for different limit states.



(a) (b)

Figure 15. Embedded retaining wall system (a) and results of displacement-based seismic assessment of a case-study retaining wall system (b) (from Cecconi and Pane 2013).

5 DISCUSSION

Reviewing the results described in Section 4, it is evident that the project has significantly improved the state of the art for displacement-based seismic assessment. A number of useful tools and guidelines have been developed that are expected to provide the practicing community in Italy valuable insight into the critical factors influencing the seismic vulnerability of existing structures. Developments for certain structural typologies (e.g. bridges) have been more significant than for others (e.g. retaining walls) but it is considered that this essentially reflects the different amounts of research done previously for certain structural typologies and not for others. Overall the project has been very successful in developing guidelines for the displacement-based seismic assessment of structures, that have been published in a detailed research report, available on-line (Sullivan and Calvi 2013). The project also saw the formation of a first draft of a model code for displacement-based assessment. However, it was not possible to develop this model code to a point that could be published, owing to the large number of areas that were identified as requiring further research. Nevertheless, the large number of valuable developments to the DBA method made during this project implies that the publication of a model code for displacement-based seismic assessment should be possible in the near future.

6 VISIONS AND DEVELOPMENTS

Tools for accurate but simple seismic assessment of structures are particularly important for Italy owing to the large stock of existing buildings and the reasonably high levels of seismicity that exist in different parts of the country. Such tools and guidelines would be all the more valuable if their application could instill in engineers a better understanding of the fundamental concepts of seismic assessment and the features of a structural typology that are likely to critically affect the seismic risk. These observations underline the real value in considering the adoption and further development of displacement-based design and assessment methods in Italy.

Considering various technical points raised in the literature, it becomes apparent that there are a number of inconsistencies between seismic design and assessment methods in current codes, so much so that it may even prove difficult to demonstrate that a newly realized structure designed using force-based methods actually meets current code assessment criteria. The application of displacement-based design and assessment procedures overcomes such issues and additionally provides engineers with a better sense of the role that structural proportions, material properties, member detailing, and capacity design concepts all play in the apparent seismic risk. This project has brought the possibility of displacement-based codes one step closer, successfully developing a detailed set of guidelines and tools for the displacement-based seismic assessment of buildings and bridges. Furthermore, this project has evidenced what appears to be an opportunity to extend the displacement-based assessment procedure for simplified assessment of the probability of exceeding key limit states and provide information on more tangible performance parameters, such as monetary losses. This also suggests that a vision for the future should include consideration of how displacement-based methods that provide building-specific loss assessment could guide the identification of effective retrofit strategies, that consider not only up-front construction costs but the whole life-cycle performance. Such observations underline the value gained from this ReLUIS project and suggest that displacement-based methods could play an important role in reducing seismic risk in the years to come.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 353-366, doi: 10.14599/r101309, © 2015 Doppiavoce, Napoli, Italy

TECHNOLOGIES FOR RISK MONITORING AND EMERGENCY MANAGEMENT – DEVELOPMENT OF TECHNOLOGIES FOR THE MONITORING AND SEISMIC RISK MANAGEMENT

TASK AT 3.1.1 – MOBILE SYSTEM FOR EARTHQUAKE EARLY WARNING

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1 INTRODUCTION

The concept of Earthquake Early Warning System (EEWS) today is becoming more and more popular in the seismological and engineering communities, especially in the most active seismic regions of the world, as one of the most effective strategies for the real-time mitigation of earthquake risk. EEW means the rapid detection of an ongoing earthquake and the prompt broadcasting of a warning message in a target area, before the arrival of the destructive waves. During the last three decades, EEWS have experienced a sudden improvement and a wide diffusion in many active seismic regions worldwide. They are actually operational in Japan [Nakamura 1984, Allen and Kanamori 2003, Odaka et al. 2003, Horiuchi et al. 2005], Taiwan [Wu and Teng 2002, Wu and Zhao 2006], and Mexico [Espinosa-Aranda et al. 2009]. Many other systems are under development and testing in other regions of the world such as in California [Allen and Kanamori 2003, Allen et al. 2009a, Allen et al. 2009b, Bose et al. 2007], Turkey [Alcik et al. 2009], Romania [Bose et al. 2009], and China [Peng et al. 2011]. In Italy, the early warning system PRESTo (PRobabilistic and Evolutionary early warning SysTem) [Satriano et al. 2010] is under testing in southern Apennines, since December 2009. It is currently used to monitor the Apenninic fault system and to detect small-to-moderate size events along the fault zone where the M_W 6.9, 1980 Irpinia earthquake has occurred [Zollo et al. 2009a, Zollo et al. 2009b, Iannaccone et al. 2010].
Most of existing EEWS essentially operate in two different configurations, the "regional" (or network-based) and the "on-site" (or standalone station-based), depending on the source-to-site distance and on the geometry of the considered network with respect to the source area. The regional configuration is generally adopted when the network is deployed in the source area, while the targets to be protected are far away from it. In this approach, the early portion of recorded signals is used to rapidly evaluate the source parameters (essentially, event location and magnitude) and to predict a ground-motion intensity measure (e.g., Peak Ground Velocity, PGV, and/or Peak Ground Acceleration, PGA) at distant sites, through empirical Ground Motion Prediction Equations (GMPE). As data are progressively acquired by the network, the initial estimations are updated, providing a continuously refined information on the earthquake parameters and providing the ground

shaking prediction at the target sites. Given the source-to-site distance, the "lead-time" (i.e., the time between the alert issue and the arrival of damaging waves at the target site) can be relatively long in a regional configuration, although the prediction of the shaking at distant sites may be affected by large uncertainties due to the use of empirical predictive relationships and errors in location and magnitude estimates.

The on-site approach, instead, is generally used when the sites to be protected are close to the source area or the source area is not accessible (e.g. off-shore). In this configuration the early portion of recorded P-wave signals is used to predict the ensuing peak ground-motion at the same site and to provide a local alert level, based on the combination of early warning observed quantities (such as P-wave peak displacement and/or predominant period). The main advantage of such an approach is that the alert for an impending earthquake at the target site is issued based on a local measurement of P-wave ground motion amplitude, avoiding the use of empirical predictive laws and bypassing the estimation of earthquake location and magnitude, which might be affected by large uncertainties in a real-time analysis.

The new idea for EEWS developed in this RELUIS II project is the integration both from the technological and methodological points of views, of the two approaches, which allows to get accurate estimations of earthquake parameters, reliable prediction of the expected ground motion and quite large lead times. The integrated approach, proposed by Zollo et al. 2010, is essentially based on three key-elements: i) the definition of a local alert level from the combination of the initial Peak Displacement (P_d) and the average dominant period (τ_c); ii) the use of the initial peak displacement as a proxy for the Peak Ground Velocity; iii) the real-time mapping of a Potential Damage Zone (PDZ). The integrated approach has been off-line tested for the 2009, M_W 6.3 L'Aquila (Central Italy) earthquake and ten Japanese large earthquakes [Colombelli et al. 2012]. Recently, the method has been also implemented in the PRESTo software platform, and is currently under testing in Southern Italy using data streaming of small-to-moderate events from the Irpinia Seismic Network (ISNet).

In the RELUIS II project, one additional objective was to design and develop a mobile early warning system to install and use during seismic crisis occurring before (foreshocks) and after a main event (aftershocks) to complement an existing permanent alert system. Such a mobile system has the aim to support the Department of Civil Protection for the real-time monitoring of ongoing seismic activity and provide with early warnings to be used for emergency management. We aimed at developing the technologies for a single node of the network (EW box) which included the PRESTo software platform implementation, focusing on the methodologies and algorithms for the real-time notification of the seismic alert and on-site threshold based alert strategy.

In addition to the aforementioned objectives, a new line of research, structural damage monitoring from the analysis of seismic noise and seismic interferometry techniques, was developed in collaboration with researchers of the Polytechnic of Turin. The goal was to use ambient noise and shot records from an hammer to retrieve the impulse response of a bridge from cross-correlation analysis. During the acquisition, the bridge was damaged to mimic the interaction between the basement of the bridge and the water flow in a river and the damage quantified as a percentage variation in average wave velocity in the structure. This method can complement an early warning system, because it monitors the structural behavior of a structure without invasive methods and may indicate when it approaches to severe damage.

2 BACKGROUND AND MOTIVATION

In the framework of RELUIS II project the threshold-based earthquake early warning methodology [Zollo et al. 2010] has been verified and applied to a set of Japanese earthquake (M>6) occurred during the last decade. This approach represents an integration of regional/on-site early warning method and can be used in the very first seconds after a moderate-to-large earthquake to map the most Probable Damaged Zones (PDZ).

The key element of the method is the real-time simultaneous measurement of initial peak displacement (P_d) and period parameter (τ_c) in a 3-second window after the first P-arrival time at accelerometer stations located at increasing distances from the epicenter. As for the on-site approach the recorded values of P_d and τ_c are compared to threshold values, which are set for a minimum magnitude M = 6 and instrumental intensity I_{MM} VII, according to empirical regression analysis of strong motion data from different seismic regions. At each recording site, an alert level is assigned on the basis of a decisional table with four entries defined by critical values of the parameters P_d and τ_c. The alert levels refer to the possible occurrence of a damaging earthquake nearby or far-away the recording site. A regional network of accelerometers provides the event location and transmits the information about the alert levels recorded at near-source stations to more distant sites, before the arrival of the most destructive S phase.

A set of off-line performance tests of this method has been carried out by using ten, M > 6 Japanese earthquakes showing that the methodology is very robust for mapping the PDZ in the first seconds after a moderate-to-large earthquake (Fig.1).

The results displayed a very good matching between the rapidly predicted and observed damage zone, the latter being mapped a few days after the event from detailed macro-seismic surveys.

A second aspect of earthquake early warning considered in the framework of the project concerned the development of a Mobile Early Warning Network (EW MobNet). The EW MobNet represents an innovative system able to integrate both regional and on-site approaches (represented in Fig. 2) based on the real-time measurement of peak displacement and predominant period on early P-wave signals. Using the threshold-based method [Zollo et al. 2010] the system can provide a rapid estimation of the PDZ, by-passing the magnitude estimation for ground motion prediction. Moreover it can be easily integrated with recently developed regional methods, e.g. PRESTo [Satriano et al. 2010].

The most critical issue for an EW MobNet is the choice of the data transmission system. It is well known that satellite transmission may be slow to be effective for early warning (due to several seconds of time-delays between transmitters and receivers), expensive and difficult to be implemented. On the other hand, radio links can be not fully suitable for streaming due to actual band limitations. Proprietary WI-FI and/or GPRS/UMTS are our preferred solutions which enable to reduce the time transmission and to increase the lead-time (i.e., the time between the alert notification and the arrival time of potentially destructive waves at a given target site), available for risk mitigation actions. The choice of the optimal sensor and Analog-to-Digital Converter (ADC) is also critical for the realization of an EW MobNet, in relation to the target of detecting and issuing an alert for moderate to strong intensity events (Instrumental intensity larger than V-VI). Our research group is developing a prototype node for an EW MobNet based on Micro-Electro-Mechanical Systems (MEMS) sensor technology. Three different axial accelerometers are being tested in combination with several ADC (12, 14, 16, 18, 24 bit) to define the optimum hardware configuration in terms of cost, energy consumption, portability and dynamic range.

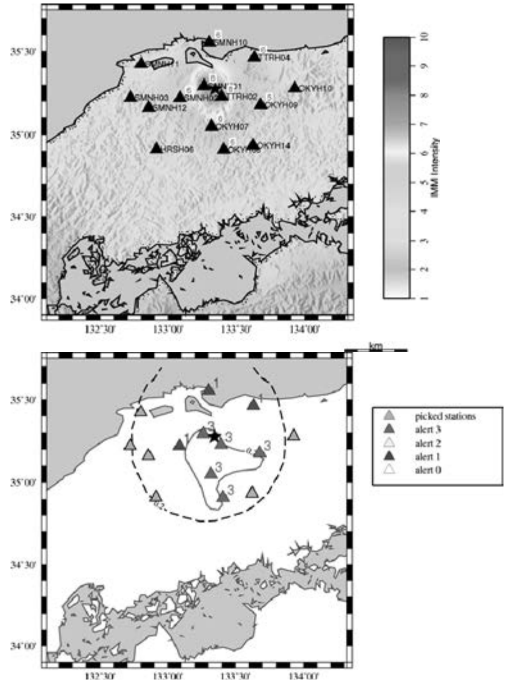

Figure 1. Example of output maps of a threshold-based EW system. The figure refers to the 2000, M = 7.3 Tottori earthquake and shows the results obtained about 12 seconds after the origin time. The real-time intensity map (top): the epicenter is identified by a black star; stations for which 3 seconds of data are available are represented by a black triangle. The color scale represents the predicted intensity value, while the real intensity value is identified by the red number close to each station. The "operative" alert level map (bottom): white triangles represent stations triggered by the earthquake, while red and blue triangles suggest the alert level value at each station, 3 and 1 respectively. The red line delimits the true PDZ, while the "maximum" PDZ is identified by a black dashed line.

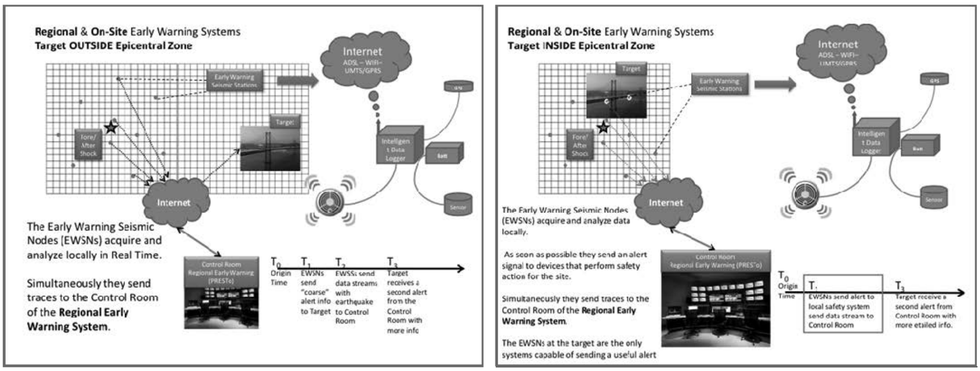

Figure 2. Example of Regional and On-Site Early Warning System. The scenario with the target outside the epicentral zone (left): during earthquake each node (red point) acquire and analyse data in real-time and send a "coarse" alert signal to the target. Simultaneously they send traces to the control room of the regional early warning system. The control room send a second alarm to the target with more information. The scenario with the target inside the epicentral zone (right): during earthquake each node (red point) acquire and analyse data in real-time and send an alert signal to the target. Simultaneously they send traces to the control room of the regional early warning system.

During the project, the structural damage of a structure was monitored using interferometric methods.

Several authors have theoretically and experimentally demonstrated that cross-correlation of a random isotropic wave-field computed between two recording points A and B results in a waveform that differs only by an amplitude factor from the Green function $G_{AB}(t)$ between the receivers [Sanchez-Sesma & Campillo, 2006; Shapiro et al., 2005]. In the last decade this relation has been widely used in seismology to obtain information about the Earth's crust, and generally about the wave propagation media, from the seismic ambient noise. Methods based on similar approaches are recently applied also for structural monitoring and for studies on structural material changes [Larose et al., 2006; Stähler et al. 2011].

In order to test these methods for seismic engineering applications, we used them to evaluate the variation of elastic properties of a 1:2 scaled model of a masonry arc bridge undergone to an increasing level of controlled damage. The bridge was constructed in the laboratory of the Department of Structural and Geotechnical engineering of the Polytechnic of Turin (Italy) to study the evolution of damage mechanisms related to the application of foundation movements. Different excitation sources were applied to the bridge model: ambient vibrations, impact hammer and a shaker; then acceleration signals where acquired by 18 mono-axial accelerometers, distributed in different points of the bridge and recording the signal at a sampling rate of 400 Hz. For our analysis we used 3 dataset composed by 3 minutes of signals acquired with the bridge excited by ambient noise vibration at three different level of damage. Following the process proposed to obtain reliable Green Functions from seismic ambient noise measurements [Bensen et al., 2007], we organized the single station data in 18 window of 10 s length that have been equalized in both time and frequency domains by whitening and one-bit normalizations. The records from couple of stations have been cross-correlated to build up a database of Green's Functions. Finally, in order to relate the variations of Green's Functions to the damage level of the bridge, the functions at the same receivers couple for different damage level are cross-correlated and a statistical analysis was performed computing the mean of correlation and the standard deviation. The final correlation values are significantly lower when they are computed between receivers couples in undamaged-damaged cases respect the values of undamaged–undamaged or damaged-damaged cases.

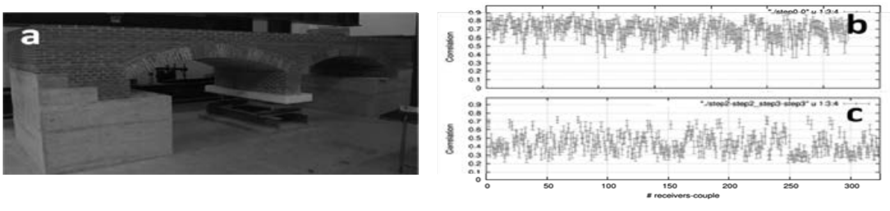

Figure 3. (a) the scaled madison bridge. Mean cross-correlations and standard deviation for each receivers couple. The correlations values and related standard deviations are computed by Green's Functions between undamaged-undamaged (b) and undamaged-damaged cases (c).

3 RESEARCH STRUCTURE

The work in the ReLUIS II projectwas organized in activities and sub-activities as follows:

1) PRESTo software platform

- Methods of signal analysis and algorithms for the real-time notification of the seismic alert
- PRESTo^{PLUS} software platform implementation with on-site threshold based alert strategy for Early Warning Box (EW-Box)

2) Early Warning Box as a node of EW MobNet
- Technical specifications of the system
- Prototype installation at a ISNET seismic station
- Development of a P-wave based threshold EW algorithm running on a single station

3) Structural damage from the analysis of seismic noise and seismic interferometry techniques
- Cross-correlation analysis aimed at detecting the structural damage of bridges
- Temporal variations of the elastic properties of a structure

4 MAIN RESULTS

4.1 Methods of signal analysis and algorithms for the real-time notification of the seismic alert. On-site Threshold based Alert Strategy: methodological development and testing

In the line with the threshold based approach best suited for dense regional and national seismic networks, we developed a P-wave threshold-based alert strategy for a stand-alone seismic station (onsite approach) to be finally implemented in the EW node of the EW Mobnet. The methodology is based on two observed correlations between EW parameters and ground motion quantities. The first one is the empirical correlation between the Peak Displacement (Pd) (measured in the first few seconds of P-wave signal) and the Peak Ground Velocity (PGV) (measured on the entire record) [Zollo et al. 2010]. The second one is the correlation between the PGV and the Instrumental Intensity (Mercalli-Modified Intensity, MMI) at each site. Given this empirical scaling, and assuming a threshold Intensity value, we defined a threshold value for the observed PGV. This value can be converted into a corresponding threshold value for the Early Warning parameter, Pd.
The strategy for the Onsite warning consists in the continuous measurement of the Peak Displacement, starting from the P-wave trigger time, and progressively expanding the observation time window, following the approach of Colombelli et al., 2012a. As soon as Pd overcomes the established threshold, an alert level is issued at the considered station.
The “threshold-based” alert strategy has been tested off-line using a database of moderate-to-strong Japanese earthquakes ($4<M<9$), including the last destructive Magnitude 9.0 2011, Tohoku-Oki earthquake. We performed a massive statistical analysis using a total number of about 12.300 accelerometer waveforms, corresponding to 68 earthquakes, in the distance range between 0 and 500 km.
Following the Alert table as defined by Colombelli et al., 2012b, for each available record, we compared the observed PGV value with the one predicted from the Pd vs. PGV equation. Based on the combination of the observed Pd and PGV at each site (Fig. 4a), we defined “successful”, “missed” and “false” alarms (see Table 1) and counted their total number.

Table 1. Criteria adopted to distinguish among successful, missed and false alarms in testing the performance of the PRESTo system in the regional configuration.

***Successful* alarm**	Pd ≤ Pd* and PGV ≤ PGV* Pd > Pd* and PGV > PGV*
***Missed* alarm**	Pd ≤ Pd* and PGV > PGV*
***False* alarm**	Pd > Pd* and PGV ≤ PGV*

The results of the cumulative statistics is shown, in the form of histograms, in Figure 4b, for the Intensity threshold value MMI = 6 and in Figure 4c, for the Intensity threshold value MMI = 7. In both cases we found an excellent percentage of successful alarms (> 90%), a small percentage of false alarms (< 6%) and a percentage of missed alarms smaller than 1%.

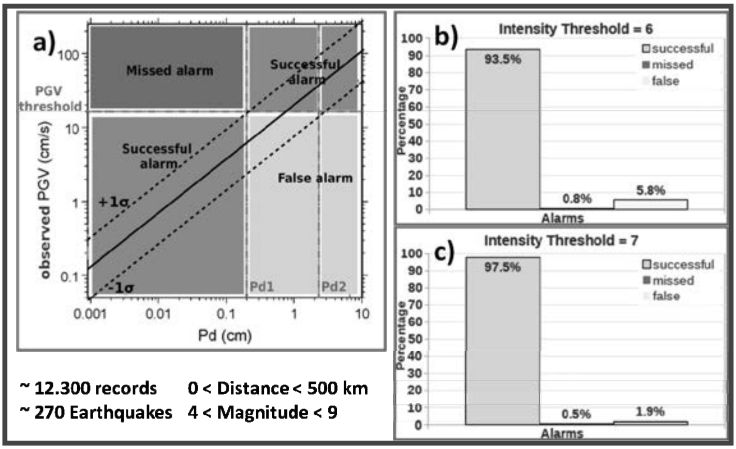

Figure 4. Panel a) Threshold-based scheme: definition of successful", "missed" and "false" alarms based on Pd and PGV correlation. Cumulative statistics on the entire database for an Intensity threshold of 6 (panel b) and 7 (panel c).

4.2 Defining the architecture and components of the system with the technical specifications of the hardware

A new prototype of low-cost sensor named ASTERISK was produced to represent the main component of the EW node of MobNet. The prototype is based on MEMS technology as regards the accelerometric sensors, and a processor ATMEL ARM9 @ 400Mhz for the part of the processing and management of information (Fig. 5).

The performances of the new sensor have been defined by the measurement of its electronic noise level (Fig. 6), as well as by the tilt test. The sensitivity of the MEMS sensor resulted equal to 0.061×10^{-3} g (6.1×10^{-3} gal), while the level of electronic noise was 13.8×10^{-3}g (1.38 gal).

Furthermore, keeping into consideration the nominal and effective sensor performances, that is to say including the sensor electronic noise except for the environmental seismic noise, the threshold levels for the recordable ground motion in terms of P- and S- waves have been defined considering different magnitudes and distances (Fig. 7). For example, Figure 7 shows that in principle ASTERISK should be able to trigger on the P-waves a magnitude 2 event at the epicentral distance of 10 km.
Finally, in order to perform a field assessment of the sensor performance, a prototype has been recently installed at the ISNET seismic station named ‘Rocca San Felice’ (AV), in proximity to high quality sensors (Fig. 8).

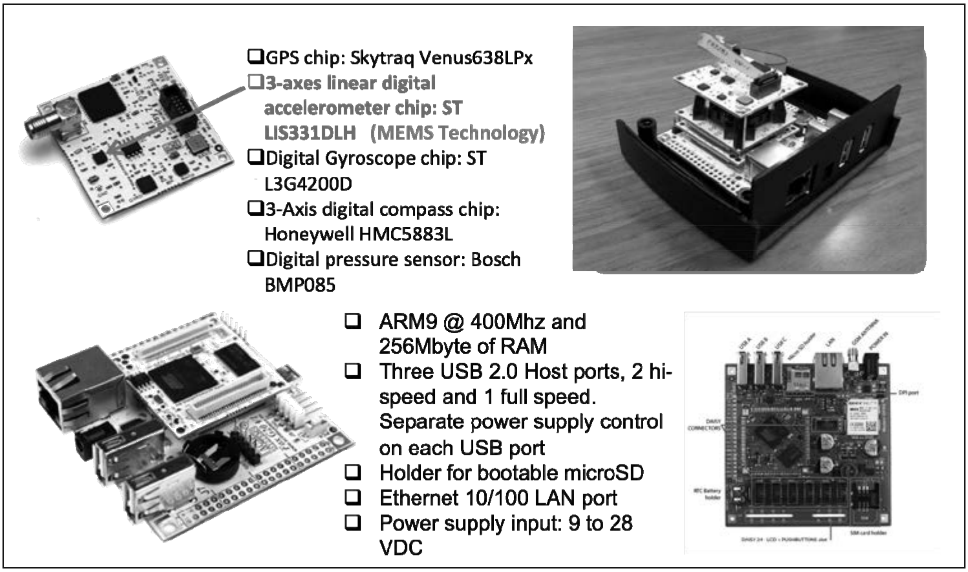

Figure 5. ASTERISK hardware components and main characteristics.

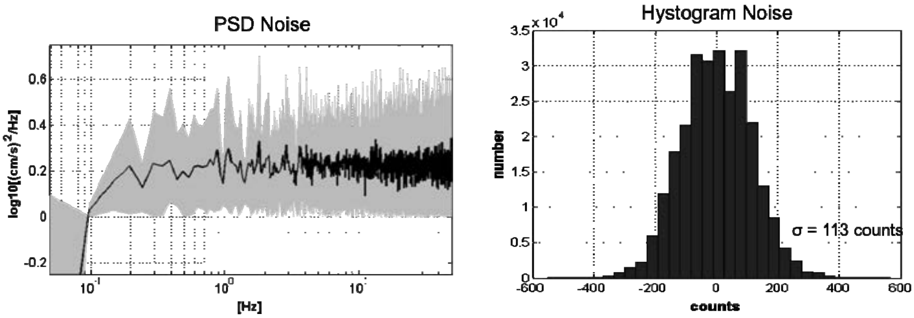

Figure 6. PSD spectra of the ASTERISK’s electronic noise (left) and its histogram (right).

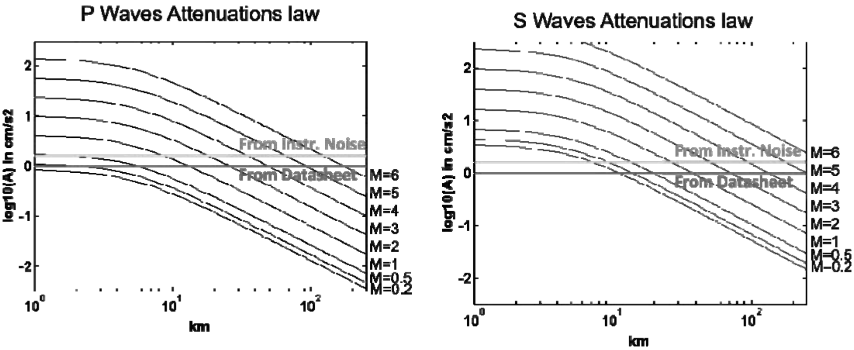

Figure 7. Nominal (red) and effective (green) ASTERISK's sensitivity with respect to the theoretical ground motion for different magnitude and distance ranges (black).

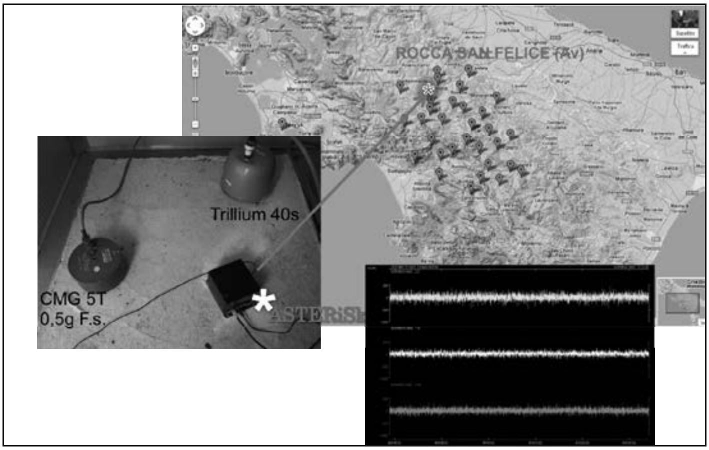

Figure 8. Example of ASTERISK installation at the ISNET station of Rocca San Felice (AV).

4.3 Structural damage from the analysis of seismic noise and seismic interferometry techniques

This activity is carried out in collaboration with the Department of Structural and Geotechnical Engineering of Polytechnic of Turin.

Several authors have shown that the cross-correlation of a wave field isotropically diffused inside a medium and recorded at two receivers approximates the Green function between these two points [Sanchez-Sesma & -Campillo, 2006; Shapiro et al., 2005]. This relationship allows to obtain information about the velocity model and its temporal variation from the analysis of the ambient seismic noise and the earthquakes codas. Recently, similar methods have been also applied for the monitoring of the structures [Larose et al., 2006; Stähler et al. 2011].

In this activity, we tested these methods to estimate the variation of the elastic properties of a masonry arch bridge, built in laboratory at a scale 1:2, subjected to an increasing level of controlled damage. The bridge was excited with different types of sources: ambient vibrations, hammer hits and shaking table. The signals were acquired by 18 mono-axial accelerometers, distributed in different points along the bridge and with a sampling frequency of 400 Hz. For this analysis we used three datasets of 3 minute-long time records of ambient noise for the three levels of damage. Analogously, we also analyzed the records in correspondence of the hammer hits. We arranged the noise records at each station in 18 time windows of 10s, equalized both in the time domain and the frequency domain through spectral whitening and one-bit normalization. Similarly we extracted hammer related events and selected their codas as the portion of the signal that does not contain the ballistic arrivals and which exponentially decays in time. The noise and the codas were separately cross-correlated for couples of stations to build a database of Green's functions. Finally, these were compared with each other for the three levels of damage. In the case of the ambient noise main differences were observed in the Green's functions during the three phases. This difference was quantified by the cross-correlation of the Green's functions. Anyhow, at this level we were not able to distinguish the effect of the variability of the propagation medium from the variability of noise sources, which may be different in the three phases of the experiment. Analyzing the codas of the signals, instead, we found that in a broad range of frequencies (30-70 Hz), the Green's functions for the damaged bridge were delayed as compared to the Green's functions of the intact bridge (Fig. 9) . On a single section, with a selection of the cross-correlations with a signal to noise ratio above 5, we measured the variation in the arrival of the phases ΔT and the ratio $\Delta T/T$ related to the relative variation of speed $\Delta V/V$. This was estimated to be of the order of 3%, uniformly in the analyzed frequency band. Assuming a S wave speed of 3km/s, the speed change is of the order of 100 m/s.

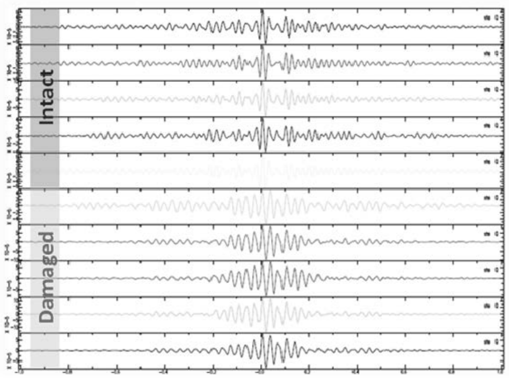

Figure 9. Cross correlated of event codas for the un-damaged and the heavily damaged bridge.

5 DISCUSSION

The research activities were fully compliant in the timing and objectives of the program originally proposed.

Our research unit closely collaborated on Early Warning developments with the Department of Structural Engineering (University of Naples Federico II). We have jointly participated to the project "*Early Warning Sismico: sviluppo e test di un sistema prototipo per l'allerta sismica preventiva in un edificio pubblico*" in the frame of the F.A.R.O. Programme (funding programme managed by the Polo delle Scienze e delle Tecnologie dell'Università di Napoli "Federico II").

As a follow-up of the scientific interaction with research groups working at the same research line, we established a new collaboration with the researchers from the Department of Structural and Geotechnical Engineering of Polytechnic of Turin on the topic of structural damage monitoring from the analysis of seismic noise and seismic interferometry techniques.
The results obtained in the project by our research unit allowed to make a step forward in the design and implementation of reliable early warning systems for engineering and societal applications. The regional, network-based method and on-site warning have been fully integrated and made operational in the software platform PRESTo, which is now delivered freely under GUI license through the site http://www.prestoews.org/. The software is actually running in testing mode on the seismic networks of South Korea, Romania and INOGS in Italy.
A prototype of the Early Warning Box has been designed and realized in laboratory. The technical characteristics of the node HW components (data logger, sensor, data transmission, local storage and computing capabilities) have been specifically designed to be integrated in a EW mobile network for aftershocks, although no network testing has been performed due to limited financial resources. A new prototype of low-cost sensor named ASTERISK was produced based on MEMS technology as regards the accelerometric sensors, and a processor ATMEL ARM9 @400Mhz for the part of the processing and management of information. The performance of the sensor has been tested by comparing the noise and signal records at the ISNET seismic station named 'Rocca San Felice' (AV), in proximity to high quality sensors (Fig. 8).
We found that the use of ambient noise for isolated experiments does not allow to distinguish the effect of the variability of the propagation medium from the variability of noise sources, which might have been different in the three phases of the experiment. Analyzing the codas of the signals, instead, we found that in a broad range of frequencies (30-70 Hz), the Green's functions for the damaged bridge were delayed as compared to the Green's functions of the intact bridge (Fig.9). The relative time delay is a proxy for the relative variation of the wave speed $\Delta V/V$. This was estimated to be of the order of 3%, with a speed change of the order of 100 m/s, one order of magnitude larger than the value reported for fault preparing to large earthquakes.

6 VISIONS AND DEVELOPMENTS

The RELUIS II project has supported and co-financed the research activity of our group during these years aimed at the development and implementation of advanced methodologies for early warning. The new generation of EWS that our group has developed integrates the concepts of regional and on-site approaches providing an evolutionary and probabilistic estimation of the source parameters (location and magnitude) but also a reliable estimation of the ground shaking amplitude at the site to protect, trying to maximize the lead-time and minimize the uncertainties on the estimated parameters.
The project RELUIS II allowed to develop the new version (PRESTo^{PLUS}) of the previously existing EW platform which now can operate both as a regional or onsite, threshold-based system, providing an alert level at each recording site, in addition to the estimation of earthquake location and magnitude, and the predicted peak ground motion through the GMPE. Furthermore, the measure of P-wave peak amplitude and characteristic period allows to map in real-time the Potential Damage Zone, e.g. the near source region where ground shaking is expected to produce significant damage during a destructive earthquake.

The software platform PRESTo^{PLUS} is now distributed and a number of researchers managing local and regional seismic networks are testing the performances in other seismic area of the world. An experimentation is ongoing also in Italy , in collaboration with INOGS of Trieste.
One of the main products of our research is the design of an early warning box whose prototype has been tested in laboratory and in the field. The hardware components of the node are specifically designed to operate as a stand-alone instruments providing a local alert or being integrated in a larger network by sharing the acquired information with other nodes of the network. The node has the software capability to run the algorithms that have been developed for the platform PRESTo and adapted to run in stand-alone mode. After the first exploratory phase and the realization of the prototype, done in this project, we are currently investigating the feasibility of a joint venture with a specialized company to produce a commercial version of the EW node.
Interferometric techniques are passive, non-invasive methods that allow to retrieve changes in the structure without modifying it. However, to constrain the velocity change in the structure a complete characterization of the noise in repeated experiments is required. Specifically, to avoid deconvolution for noise sources, ambient noise should be stationary. To deeply explore the changes, moreover, higher frequency records are required. Due to the relatively large speeds in the medium, anomalies may be detected if kHz records are available. The same results can be obtained by the use of shots (realized by hammers); again, shots require the same source time function to avoid source deconvolution.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 367-386, doi: 10.14599/r101310, © 2015 Doppiavoce, Napoli, Italy

TECHNOLOGIES FOR RISK MONITORING AND EMERGENCY MANAGEMENT – DEVELOPMENT OF TECHNOLOGIES FOR THE MONITORING AND SEISMIC RISK MANAGEMENT

TASK AT 3.1.3 - HEALTH MONITORING

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1 INTRODUCTION

Structural Health Monitoring (SHM) aims to increase the knowledge about linear and non-linear dynamic behaviour of structures in order to develop accurate strategies of analysis for the safety assessment. Particularly, structural monitoring, especially for structures located in seismic prone areas, assumed great importance for the possibility to provide rapid and useful information about the state of the health of strategic structures and infrastructures over time and to perform an objective quick estimate of the damage occurred on structures just after a seismic event.
These tools allow to better understanding the structural behaviour during earthquakes and related damage mechanisms. SHM permits to develop effective and at the same time cost-effective protection strategies. The international scientific literature provides very powerful new mathematical tools for non-stationary signal analysis and then to analyse stationary and non-stationary, linear and nonlinear, structural behaviour before, during and after seismic events. One of the main goal of the Task 3.1.3 has been to develop techniques for structural dynamic identification and for structural damage detection and localization on framed reinforced concrete structures. Some of the techniques developed during the project are based on the Stockwell Transform that is a representation on the time-frequency domain. Using linear and nonlinear numerical models and performing shaking table tests on a 1:15 scaled structure all developed techniques have been tested.
The research carried out is also focused on the development and optimization of SHM methodologies for strategic structures and infrastructures in seismically prone areas. Europe and, in particular, Italy are characterized by a large number of urbanized areas, where an high percentage of structures have been designed and erected according to outdated codes of practice. In addition, the age of the structures and actual performance of the construction materials significantly affect the overall behaviour of the existing buildings. This is also the case of many Italian urban areas, where many existing structures can be classified as functionally inadequate due to their age and structural deficiencies. Structural assessment and rehabilitation are therefore becoming critical issues in urban management and planning.
Serious damages and structural collapse, which have occurred all over the country in recent years, point out the need to evaluate and enhance the structural safety of existing structures against hazardous natural events, such as earthquakes. As clearly shown by recent

seismological classifications a large part of the Italian territory is exposed to medium/high seismic risk. This circumstance highlights the need for effective measures to be taken in order to protect constructions at risk and to mitigate injury and death due to seismic events. The first step in achieving this goal is to increase the knowledge about the structural behaviour of existing structures by providing guidelines that define measures to protect them and, at the same time, decrease the probability of structural damages. Since the continuous monitoring of relevant parameters and performance is certainly of interest, some research activities started in the context of the present programme aiming at the development of optimized systems and methodologies for structural health monitoring of relevant structures in seismic areas, including geotechnical systems.

2 BACKGROUND AND MOTIVATION

Studies on the Structural Health Monitoring (SHM) are mainly aimed at deepening understand on dynamic behaviour of structures subjected to seismic action, in order to develop strategies for accurate analysis and safety assessment. Particularly, Structural Health Monitoring, especially for structures located in seismic prone areas, has assumed a meaning of great importance, for the possibility to make a more effective and more rapid estimation of the damage occurred on buildings after a seismic event. These tools are also useful to analyse the dynamic behaviour of structures, taking into account their interaction with soil, and to study the damage mechanisms in order to develop operative strategies for seismic retrofit of existing buildings and to reduce their economic impact. In the last years, there have been significant advances both on the theoretical approach, concerning new techniques for the building dynamic identification and on development and application of new techniques based on time-frequency analyses. Particularly, during the last twenty years, significant efforts have been devoted to the field of Non-destructive Damage Evaluation (NDE) using the variation in time of the dynamic characteristics of a structure such as frequencies, mode shapes and global dissipative characteristics (equivalent viscous damping factor). In order to increase the performance level of damage detection and localization on monitored structures, it is necessary to support the theoretical criteria with numerical and experimental tests on both real and scaled structures using in laboratory and in situ tests. Within the Reluis II project, the Task 3.1.3 focused on the possibility to increase the level of knowledge and the related performance of techniques for structural dynamic identification, damage detection, model updating of structures, infrastructures and cultural heritage. Together with the research units of the Task 3.1.2 of the same project, one the main goal focused on the possibility to integrate techniques and technologies for SHM and Early-Warning systems.
Taking into account the advantage of the synergies derived from the heterogeneous group of researchers, the investigation carried out within the Task 3.1.3 mainly focuses on: i) development of monitoring and damage detection methodologies for strategic structures and infrastructures in seismic prone areas (including cultural heritage); ii) investigation of soil-structure interaction effects; iii) design and installation of structural health monitoring systems on sample structures; iv) development of data processing procedures and optimization of structural health monitoring systems for seismic emergency management. Particularly, both stationary and non-stationary approaches for the data analyses have been investigated.
The former can be considered for the i) development and evaluation of the robustness of identification linear dynamics methods in the frequency and time-frequency domain techniques for the identification and validation of dynamic and diagnostic methods based on

ambient vibration measurements; ii) calibration of finite element models of framed structures and of monumental masonry buildings and churches. This kind of approach can be used for both damaged and undamaged structures, for the definition and evaluation of the safety measures and for the validation of the structural rehabilitation interventions.

The latter, related to one of the main applications of structural health monitoring systems, can be adopted for the detection and characterization of damage, in order to evaluate structural integrity and to implement condition-based maintenance strategies. Particularly, dynamic identification protocols based on the Stockwell Transform have been developed to extract time-varying dynamic parameters of structures and to setup new methodologies for damage localization on structures. In order to increase the performance level of damage detection and localization on monitored structures, it is necessary to support the theoretical criteria with numerical and experimental tests on both real and scaled structures using in laboratory and in situ tests. In the last years, in order to localize and quantify the damage occurred on both single structural elements and structures, several authors proposed to use the mode curvature variation over time. Practically, comparing the geometric mode shape curvature displayed by the elements, and/or by the structure, over time it is possible to localize in an accurate way where the damage occurred. Most of these techniques are based on the variation of the mode shape curvature related to the fundamental mode of vibration of the structure. Pandey et al. (1991) proposed the mode shape curvature to be a sensitive parameter for damage localization. Sampaio et al. (1999) extended the idea of Pandey et al. (1991) by applying the curvature-based method to frequency response function instead of mode shape and demonstrated the potential of this approach by considering real data. Radzienski et al. (2011) performed an experimental campaign on an aluminium cantilever beam to identify the modal parameters. In these studies it was found that mode shape curvature is a useful parameter for damage detection and localization. Dilena et al. (2011) demonstrated that mode shape curvature could be a useful term for damage location on a reinforced concrete single span bridge. Roy and Ray-Chaundhuri (2013) provide a mathematical basis to show the correlation between a structural damage and a change in the fundamental mode shape and its derivatives. For a cantilever shear beam this approach demonstrates that the change in the fundamental mode shape due to any damage is an excellent indicator of damage localization. Further, also the change in higher derivatives of the fundamental mode shape could be used to increase the performance of damage localization techniques. With this aim, during the Reluis II project several shaking table tests have been performed on a 1:15 scaled structure and conducted at the Seismic Laboratory of the University of Basilicata (SISLAB). One of the main goal of the shaking table tests performed on the five floors scaled structures was to validate a new procedure for damage detection and localization on framed structure (Ditommaso et al., 2014). The proposed procedure is based on the evaluation of the fundamental mode shape curvature variation over time, when subjected to strong motion earthquake.

During the Reluis II project, the Task 3.1.3 focuses also on the definition of optimal strategies for data acquisition and transmission and to setup protocols for SHM of historical masonry structures. At this regards, some experimental campaigns have been conducted on damaged and undamaged buildings and churches. The research proposes a strategy approach of structural monitoring - of basilica-type churches - to verify the failure mechanisms, to control the safety measures and the structural rehabilitation. The research also proposes a methodology for monitoring and structural control of the basilica-type monuments through the behaviour of macro-elements considered as representative of the whole system. As example, the Basilica of Collemaggio was monitored using a continuous monitoring system based on smart sensors operating wirelessly.

3 RESEARCH STRUCTURE

The work related to the Task 3.1.3 was subdivided among six research units: University of Basilicata (UNIBAS), University of Molise (UNIMOL), Polytechnic of Torino (POLITO), University of Naples "Parthenope" (UNIPARTH), University of L'Aquila (UNIVAQ) and University of Venezia (IUAV). Aim of the research was to evaluate the best solutions in terms of data analyses techniques, technologies and geometric configuration for Structural Health Monitoring, damage detection and damage localization on structures, infrastructures and cultural heritage. With this aim several numerical and experimental campaign have been organized. The research activities developed at UNIBAS have been organized as follows:

- Development of hybrid techniques based on seismic interferometry and time-frequency analyses (S-Transform);
- Development of nonlinear numerical models to calibrate a fast method for structural damage detection on reinforced concrete framed structures;
- Automation of the fast method for damage detection;
- Development a II Level method for damage location on R.C. framed structures;
- Development of a theoretical model to analyse non-stationary – linear and non-stationary – nonlinear structural dynamic behaviour during seismic events;
- Design a 1:15 scaled model for shaking table tests;
- Shaking table tests on the designed 1:15 scaled model, analyses of experimental dataset and validation of the techniques developed during the project.

The research activities carried out at UNIMOL are organized as follows:

- Completion of the installation of the integrated structural-geotechnical system for the New Student House at the University of Molise in Campobasso;
- Development and implementation of appropriate data acquisition strategies for the above mentioned system, in particular in the case of an earthquake;
- Design of a structural health monitoring system for the main building of the School of Engineering at the University of Molise in Termoli;
- Installation of the structural health monitoring system for the main building of the School of Engineering at the University of Molise in Termoli;
- Analysis of the collected data and presentation of the results for the two mentioned case studies.

The research activities related to the POLITO research unit can be divided into four main phases, in accordance with the time schedule of the task:

- Experimental test program on a laboratory case study, a 1:2 scaled experimental model of a masonry arch bridge;
- Development of a time – frequency linear identification method;
- Optimization procedure to estimate instantaneous damping aimed at studying the amplitude dependence;
- Assessment of dynamic identification techniques and diagnostic methods based on structural monitoring.

According to the scheduled program, all tasks carried out dynamic identification techniques and diagnostic methods based on structural monitoring measurements. In detail, two different activities have been conducted:

- Application to the benchmarks monitoring systems supplied by DPC and IUAV;
- Application of dynamic monitoring in the evaluation of structural safety on bridge beams of Pesio viaduct (Torino-Savona freeway).

The research activities developed by the UNIVAQ research unit are mainly related to the SHM of bridges and churches. Particularly, dynamic identification of bridges conducted using ambient vibration measurements, dynamic characterization of churches, setup of continuous monitoring system operating in both static and dynamic conditions have been studied and implemented. The experimental data collected will also be of relevance for the identification of the mechanical parameters describing the masonry and for a correct evaluation of the actual geometric configuration. It may thus have the ability to adjust in a appropriate manner numerical model for prediction of the damaged structure.

The research activities performed by the IUAV research unit can be synthetized as follow:

- to know the state of degradation of structures damaged by the earthquake;
- to define and validate the design of safety measures and structural rehabilitation;
- to monitor the structure during the safety measures and structural rehabilitation;
- to monitor the effects of safety measures;
- to define and validate the analytical models of structural functioning.

Planning the phases of the second issue described above:

- study of the global behaviour, determining the characteristics of the structure;
- study of the local behaviour through the macro-elements and the evaluation of stress distribution on the macro-elements that define the mode of collapse;
- to determine the distribution of seismic actions on macro-elements;
- to determine the seismic capacity of macro-elements up to failure;
- calculation of a partition coefficient that allows to evaluate the distribution of the seismic action on the different macro-elements;
- analysis of the influence of the different building geometry on the response of the macro-elements;
- assessment of the monitoring methodology to be applied to macro-elements as representative of the global system.

4 MAIN RESULTS

4.1 Shaking table tests: validation of the new procedure for damage detection and localization

During the Reluis II project several new protocols and automatic algorithms to characterize the dynamic behaviour of structures under seismic excitation, to detect and to localize the damage occurred on the structure have been developed. The method has been described in the Reluis reports of the first two years of activities. It is based on the fast analysis of accelerometric recordings performed on bottom and top levels of a monitored structures subjected to earthquakes. In order to optimize the technique for damage detection, several parameters automatically extract by the recordings have been considered: maximum top acceleration (filtered using a narrow-band-pass filter centred on the fundamental frequency of the structure), elastic initial fundamental frequency (before earthquake), minimum fundamental frequency reached during the seismic motion, elastic final frequency (after earthquake), equivalent viscous damping factor variation, evaluated before and after earthquake. The fast method is based on the existing linkage among the variation of the described parameters and the maximum inter-story drift (used as damage index).
The solution for the automatic evaluation of both fundamental frequency variations and equivalent viscous damping factor variations has been described on the second year report. It

is based on the Short Time Impulse Response Function (STIRF), retrieved using interferometric seismic analysis and using bottom as reference station, and the Stockwell Transform (Ponzo et al., 2012).
A very important and, at the same time, very critical aspect is a correct evaluation of the fundamental frequency variations: in some cases, during earthquakes, structures could exhibit apparent frequencies variations due to non-stationary effects attributed to a particular combination of seismic input and structural response. For an effective evaluation of apparent fundamental frequency variations of simple and complex structures, numerical and experimental investigation have been done, using both scaled and real structures. A detailed description of mathematical and physical aspects are reported in Ditommaso et al. (2012b).
In addition to the fast method for damage detection, an algorithm for damaged localization on reinforced concrete framed structures has been implemented. The proposed method is based on the band-variable filter, described in previous reports, thanks to which it is possible to extract mode shapes related to the first mode of the monitored structure, before, during and after the earthquake. The main advantage of this kind of approach is the possibility to work directly on the acceleration recordings, removing errors due to the double integration (when it is necessary to work using displacement). Using the mode shapes retrieved before, during and after the seismic event it is possible to evaluate the related modal curvature and their variation. It is assumed that maximum curvature variation is strictly linked to the position of damage occurred on the structure.
In order to verify the effectiveness of the proposed methodologies an experimental campaign has been programmed and executed during the third year of activities. Particularly, a 1:15 scaled model has been realized using steel and aluminium tapered rods, calibrated in strength and stiffness. The scaled model has been designed to reproduce the seismic behaviour of framed reinforced concrete structures: i) designed using different codes; ii) different number of floors; iii) different collapse mechanism; iv) different in plan and in elevation regularity condition.
Design of scaled models has been supported by numerical analyses performed on nonlinear models where stress-strain cyclic behaviour were calibrated using experimental results of tests performed on single elements (aluminium rods heat treated). Both static and dynamic tests were performed on single beam-column joint considering different internal and external constrain conditions.

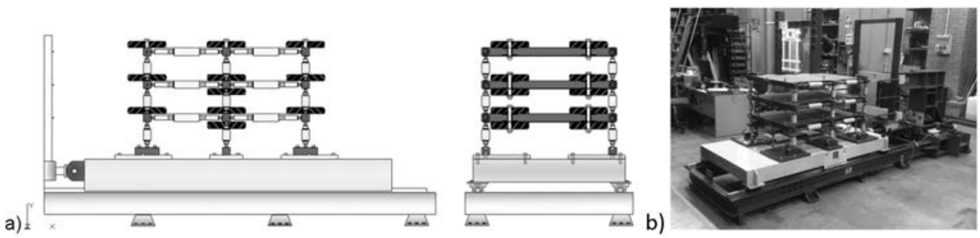

Figure 1. a) experimental model 3-M2 characterized by a scale factor equal to 1:15; b) experimental model on the shaking table of the University of Basilicata.

Preliminary experimental studies focused on aluminium tapered rods characterized by a diameter equal to 6, 8, 10, MA16 mm. In order to obtain the best performance from the shaking table available in the Structural Laboratory of the University of Basilicata, the scaled model was calibrated considering the dynamic characteristics of the shaking table system. On the analyses of experimental results, particular attention was paid to the effects of distortion associated with the scale of the model. In order to correctly interpret the results and the effects induced by the distortions on the dynamic response of the experimental model, an innovative approach has been used to analyse the data. The mentioned approach is based on the distortion of the seismic motion as a function of the desired dynamic behaviour of the scaled model.

Several shaking table tests have been performed in the Structural Laboratory of the University of Basilicata using the model depicted in Figure 1. All the considered models are representative of reinforced concrete structures designed for vertical loads only. Response spectra corresponding to the input motion used for the tests are depicted in Figure 2. The input motions are compatible with Italian Seismic Code (NTC 2008). A total of 24 tests were performed (8 for each input) varying the acceleration intensity from the 25% to 200% of the original input. All the dynamic parameters (acceleration, displacement, force) were directly acquired on both model and shaking table. Using the experimental campaign all the proposed techniques have been tested and very interesting results have been retrieved.

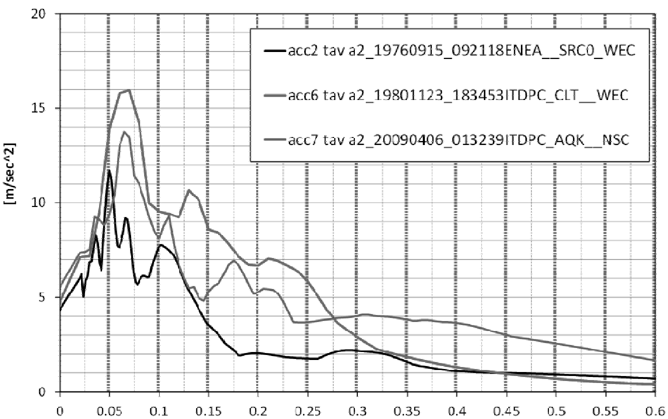

Figure 2. Acceleration Response Spectra used for the shaking table tests (Scale Factor 1:15).

The experimental structural eigen-frequencies are very close to those expected from the numerical linear and nonlinear analyses. A comparison between numerical and experimental results is reported in Table 1.

Table 1. Comparison between experimental and numerical results.

	3-M2 Numerical Model	3-M2 Experimental Model	Scaled Structure	R.C. Real Structure
	Scale Factor 1:15	*Scale Factor 1:15*	*Scale Factor 1:15*	*Scale Factor 1:1*
1 ° Mode_Dir X	0.137	0.157	0.134	0.519
2 ° Mode_Dir X	0.047	0.049	0.044	0.171

Even the non-linear structural behaviour evaluated through numerical simulations has been confirmed from the experimental campaign. In general, the nonlinear behaviour of the scaled model starts from a value of the inter-story drift equal to 0.5-0.6%. Value reported in Table 1 are referred to the tests where the maximum excitation was recorded on the columns located at the first level and the maximum inter-story drift reached a value equal to 2.5%. The comparison between numerical simulations and experimental responses in terms of maximum displacements appeared satisfactory. Similarly, the same comparison in terms of time histories shows that the two responses (numerical and experimental) are in satisfactory agreement also in terms of maximum deformation associated to the first mode of vibration. Results of the experimental campaign were analysed using all the techniques developed during the RELUIS project. This campaign was very useful to validate the proposed techniques in order: i) to analyse in quasi-real time the stationary and non-stationary structural behaviour before, during and after an earthquake and ii) to assess the structural damage occurred on the structure using the fast method based on the statistical estimation of the maximum inter-story drift.

4.2 Procedures based on the Operational Modal Analysis: results

In order to test the algorithms based on the Operational Modal Analysis (OMA) a monitoring system has been installed on a framed structure. Particularly, some of the researchers involved in the Task 3.1.3 developed and managed a structural monitoring system for the building of the University of Molise in Termoli. Strategies for data acquisition and storage able to fit the needs of both operational monitoring and seismic monitoring have been developed and implemented into a prototype system. Finally, specific attention has been focused on the study and validation of an innovative algorithm for the automated identification of the modal parameters under operational conditions. This algorithm works as modal information engine for modal based damage detection and structural health monitoring purposes. All the above mentioned activities have been completed within the third year of the present research project. The obtained results are as follows:

- The design of a modular and versatile monitoring system has been completed. Its architecture takes advantage of a MySQL relational database, which has been specifically designed and developed for dynamic data acquisition;
- Two parallel data storage procedures have been developed and implemented into a prototype system: the first refers to the continuous data acquisition under operational conditions, while the second is based on a threshold for the acquisition of the response data associated with seismic inputs; under operational conditions, the size of the database ensures that a preset amount of data is stored (for instance, 15 days at 100 Hz) and the oldest data are continuously deleted according to a FIFO approach; in the case of seismic inputs, a new table is added to the database and the structural response associated to the event is collected in the new table for permanent storage;
- A dynamic data acquisition system based on programmable hardware has been developed, integrated with the database and installed on the structure; sensor layout, cable path and other installation details have been defined by taking into account the results of the inspection of design drawings, an accurate geometric survey and the setting of a preliminary numerical model of the structure;
- The dynamic response of the structure under operational conditions has been analysed in order to get estimates of the dynamic properties of the fundamental modes;
- An innovative algorithm for the automated output-only modal identification for structural health monitoring and damage detection applications has been developed and validated against numerical and experimental datasets; the obtained results are very promising, since accurate and precise estimates of the modal parameters, including damping ratios, have been obtained.

4.3 Application of diagnostic methods to permanent monitored systems

Experimental modal analysis and FE modelling with updating techniques, developed by UR POLITO, have been applied to the data collected and shared by DPC and IUAV. By using both experimental data and a numerical model useful indication were obtained regarding both possible improvement to the installed structural monitoring system (i.e. sensor location) and the effectiveness of the seismic retrofitting intervention.

The building that have been chosen as benchmark by the DPC are:
a) City hall of S. Romano in Garfagnana
b) Hospital of Castelnuovo in Garfagnana
c) City hall of Casola in Lunigiana
d) Church of Collegnago (Fivizzano)

Table 2. The four building studied in the pilot study.

	Ed. (a)	Ed. (b)	Ed. (c)	Ed. (d)
Typology	Masonry	R.C. Frame	R.C. walls	Masonry
Year	1930	1972	1972	1359 (rebuilt 1761)
Vol. [m^3]	2990	1687	7726	1764
Height [m]	14.8	15.8	16.3	9.5 (bell tower 23)
Sensor N°	20	16	22	26
pga X [g] **pga Y [g]**	0.023 0.018	0.024 0.031	0.035 0.020	0.035 0.053

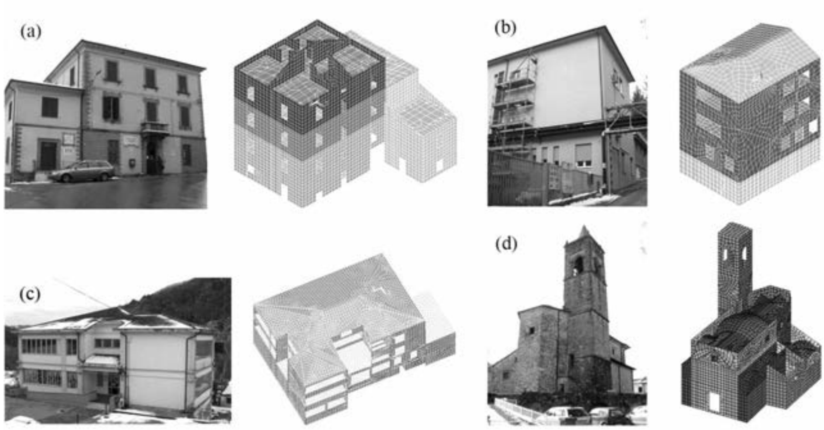

Figure 3. The four buildings analysed: (a) City hall of San Romano in Garfagnana; (b) Hospital of Castelnuovo in Garfagnana; (c) City hall of Casola in Lunigiana; (d) Church of S. Caterina in Collegnago.

The buildings selected as case studies by the DPC were all monitored by the "*Osservatorio Sismico delle Strutture*" (OSS) during the seismic event occurred in Lunigiana and Garfagnana the 27/01/2012. For each one of them, the structural response acquired during the seismic event has been utilised for carrying out the identification of the modal parameter and the calibration of a numerical model. This highlighted that it is possible to apply in an

effective way output-only modal identification techniques, which are developed for stationary input, also in presence of moderate intensity earthquakes.
The application of experimental data together with a numerical model allows to find useful indication to the improvement of the installed monitoring system (e.g. sensor location), in order to maximise the information on the dynamic behaviour of the structure. The numeric model, calibrated on the basis of the identified modal parameters, is an invaluable tool for evaluating the effectiveness of seismic retrofitting intervention or for the evaluation of the structural seismic towards seismic action.

Maria del Suffragio Church
Santa Maria del Suffragio church in L'Aquila, well known as Anime Sante, which was severely damaged by the earthquake of 2009 is a complex example of monitored structure, for the different variables interacting such as the intrinsic characteristics of historic masonry, widespread damage, partial collapse, safety measure and degradations of global and local stiffness.
The information deduced from the signals, in particular applying CVA identification method, have been used for the updating of finite element models, taking into account the condition of damage and degradation of stiffness with the influence of the safeguards of safety measures as changes introduced in the numerical system. This approach makes it possible to obtain significant results through the driven-model SHM analysis by comparing the two states of the system, initial and modified.

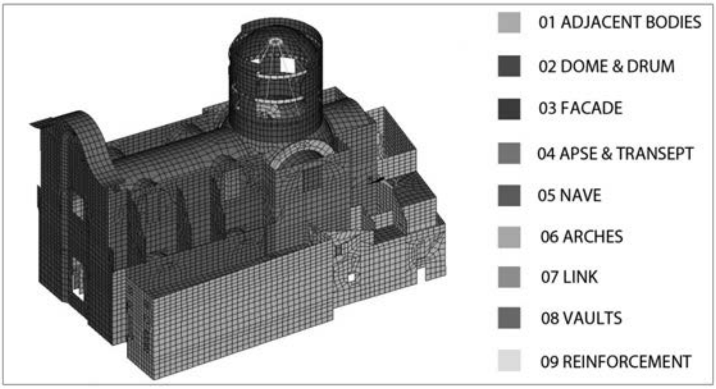

Figure 4. Numerical model of the church of S.Maria del Suffragio and seismic retrofitting interventions.

4.4 Assessment of structural reliability of bridge beams of Pesio viaduct (motorway TO_SV) based on monitored data

The experimental testing campaign was aimed at evaluating the residual strength a set of nine bridge beams, removed from viaduct Pesio during the modernization works of A6 Torino-Savona motorway, after fifty years of service life.
Each deck of the viaduct was formed by five 35 m long simply supported beams. The section of the beams had a trapezoidal shape with linearly varying height from 1.55 m at the supports to 2.95 m at the midspan. The beams were constituted by a bottom precast part completed by a 0.1 m thick, cast-in-place slab. The post-tensioning reinforcement consisted of 5+5 six-strand tendons, placed in the two sloping webs. The tested beams presented different deterioration levels, mainly related to the position occupied in the deck. The health state of

the beams was classified in three levels (good, intermediate and bad condition) on the basis of a visual inspection.
Both static and dynamic tests have been performed on the beams. A four-point static bending test was chosen to determine the ultimate load capacity. Different excitation sources have been employed for dynamic tests: ambient vibration, impulsive loads applied using a sledge hammer and forced test by means of electro mechanical vibrodyne. The signals acquired throughout both the free decay vibration tests and ambient noise were used to perform the modal identification of the beams. The analysis of tests highlights the connection among residual strength and dynamic characteristics, such as natural periods (Quattrone et al., 2012).
Each deck of the viaduct was built by five beams having the geometrical and mechanical characteristics nominally equal. Therefore, the identified modal parameters can be attributed to beams which, although have been in service for the same number of years, show different deterioration levels, expressed in terms of ultimate flexural strength. Under these assumptions, the identified modal periods can be used as symptoms to describe the evolution of the structural conditions of a beam, having the same characteristics of the investigated structures, affected to deterioration. Figure 6 shows the experimental bending moments at collapse, as a function of the first flexural periods of the tested beams, normalized by the corresponding values estimated for the undamaged conditions. Both the scaling factors have been estimated through a FE model, assuming the mean values of the material properties and the geometrical dimensions reported in the original design documents.
The residual resistance of the beams has been expressed as a function of measured symptoms and the evolution in time has been estimated. Nevertheless the reliability of the beams has been also estimated. Finally, the results obtained indicate that the knowledge coming from monitoring systems and the classical structural safety formulations can be usefully combined for improving the reliability assessment of existing structures.

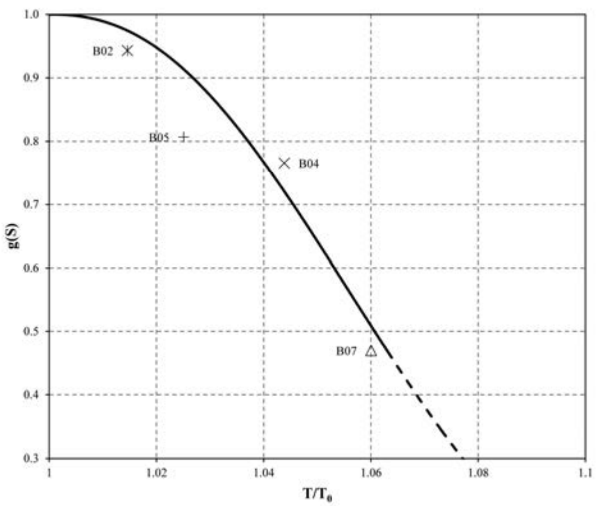

Figure 5. Normalized symptom-variant flexural resistance.

4.5 Analysis of the dynamic behaviour of the "Basilica of Collemaggio"

The system which has been developed and installed for monitoring the Basilica of Collemaggio is constituted by a accelerometric network constituted of 15 wireless nodes and a network of 11 static sensors (8 strain and 3 inclinometers). The nodes have been constructed by assembling the platform Imote2 (IPR2420), equipped with a central processing unit (CPU) and a radio card, with the sensor Board ISM400 developed from Illinois Structural Health Monitoring Project. On the ISM400 is mounted a three-axial capacitive accelerometer type MEMS ST Microelectronics (model LIS344ALH) with DC up to 1500 Hz, the characteristics of which are described in (Antonacci et al. 2011c). This combination allows to have synchronized data wirelessly. The board has the ability not only to measure the acceleration in the x, y and z but also the level of humidity and temperature. It is also present a infrared light sensor. It also highlights the possibility of inserting an analog input signal to 16-bit ADC. In addition, the greater potential concern is the ability to select a sampling frequency variable and anti-aliasing filters. It is also important the possibility to install an open source software that can perform useful commands for performing specific functions automatically such example to be able to record the accelerations 24 hours 24. A sensor so designed can be used for many purposes such as structural monitoring but also as a detector of seismic events or as a tool to perform vibration analysis of civil structures and mechanical properties. The entire system is also controlled remotely by saving the data to a dedicated server properly. The potential of remote management also concern the possibility of being able to execute remote commands such as the switching on and off of all or part of the electrical supply network of sensors. The recorded data, especially those of after-shocks of earthquakes or other nature (Aquila October 2012, Emilia May 2012), will also be useful to perform procedures for the identification of dynamic parameters needed to improve the finite element numerical models describing the seismic behaviour of interaction structure-safety of the Basilica of Collemaggio.

4.6 Installing monitoring systems on historical masonry buildings and bridges

The activities of some researchers of the University of L'Aquila can be divided into two main topics, the first oriented to the implementation of dynamic identification tests devoted to the validation of FE models for different purposes, and the other devoted to the structural health monitoring using repeated dynamic tests conducted on the basis of *ad hoc* programs.

Dynamic identification tests

Over the three years, different structures such as civil structures and infrastructure, historic buildings have been subjected to experimental testing.

The ultimate goal of these investigations is the validation of numerical models predictive of the response produced by environmental actions.

As an example, worth noticing the survey on the historical masonry building, where is located, in Udine, the CISM (International Center for Mechanical Sciences).

The investigation focused in particular on the whole structure as well as the experimental analysis of several sub-structures as wood floors and the masonry wall of the facade.

Among the infrastructure, we report investigation on the skew arch-stayed bridge on the river Versa in the Friuli Region. The complex structural characteristics of such a bridge realize a quite interesting case-study in which all the standard and non-standard problems connected with the modal identification are comprised.

Is also on going a campaign of investigation of some of the A24-motorway viaducts between Rome and L'Aquila, finalized to the seismic assessment of the structures.

Structural health monitoring

Structural Health Monitoring is the strategy for the detection and the characterization of the state of damage, defined as changes to the material or geometric properties of a structural

system, including variations of the boundary conditions which adversely affect the performance of the system, with particular reference to the ability of the structure to carry external actions. The process involves the observation of the structural system in time using dynamic measurements obtained periodically from an array of sensors, the extraction of characteristic measures sensitive to the state of damage from these measurements, and the statistical analysis of the measures to determine the state of health of the system.
The long-term SHM provides regularly updated information about the ability of the structure to correctly undergo the standard actions notwithstanding inevitable aging and degradation caused by the environment.
In this perspective, is on going from the year 2002 a campaign of structural health monitoring of a set of bridges managed by the Local Public Territorial Authority Provincia di Teramo on 50 bridges belonging to different structural classes and, among them, 6 twin-arched bridges.
They have been tested annually in the period 2002-2004 and, for the 6 bridges belonging to the mentioned sub-class, the tests have been repeated between 2010-2013.
After extreme events such as seismic events the SHM has been used for a screening providing, in near real time, reliable information regarding the integrity of the structure.
In this perspective some dynamical tests have been conducted on the bridge Belvedere in L'Aquila, after the 2009 earthquake to test the structural integrity.

4.7 Monitoring systems installed on Churches

Numerical analyses and experimental test have been performed in order to define exhaustively the state of conservation of monuments through reliable and expeditious assessment's methodologies to allow an in-depth survey on the level of vulnerability of the historical buildings. These procedures allow the introduction of a partial confidence's factor not only of material but also of whole structure (WS). The two figures below summarize the analysis carried out on various examples. In particular, figure 6 shows the analysis and dynamic identification through the macro-elements (ME).

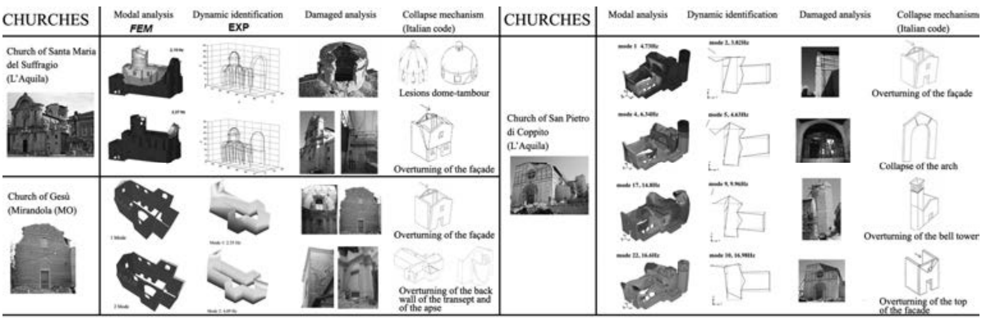

Figure 6. Analysis and dynamic identification through the macro-elements.

Figure 7 shows schematically the approach adopted for the analysis of the outbuilding's effects and the direction of the seismic component on the vulnerability assessment. The table is a first reference document, which tries to define the frequency range both for the WS and the ME, damaged and undamaged, classified for the different types. The diagram shows a first test of reliability of the monitoring of the structure through the macro-elements highlighting a good matching.

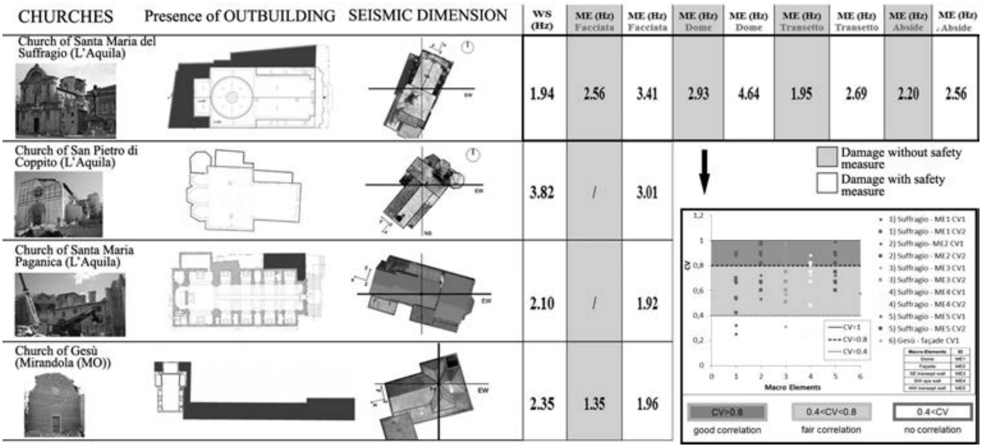

Figure 7. Characteristics of Churches.

The methodology described above was applied for the Church of Jesus of Mirandola (MO) that has been hardly hit by the earthquake of May 2012. Through the FE seismic analysis were analysed the different macro-elements taking into account two directions of the seismic action and the outbuilding's effects. Figure 8 shows the distribution of the vertical stress in the global model and in the macro-elements that were used.

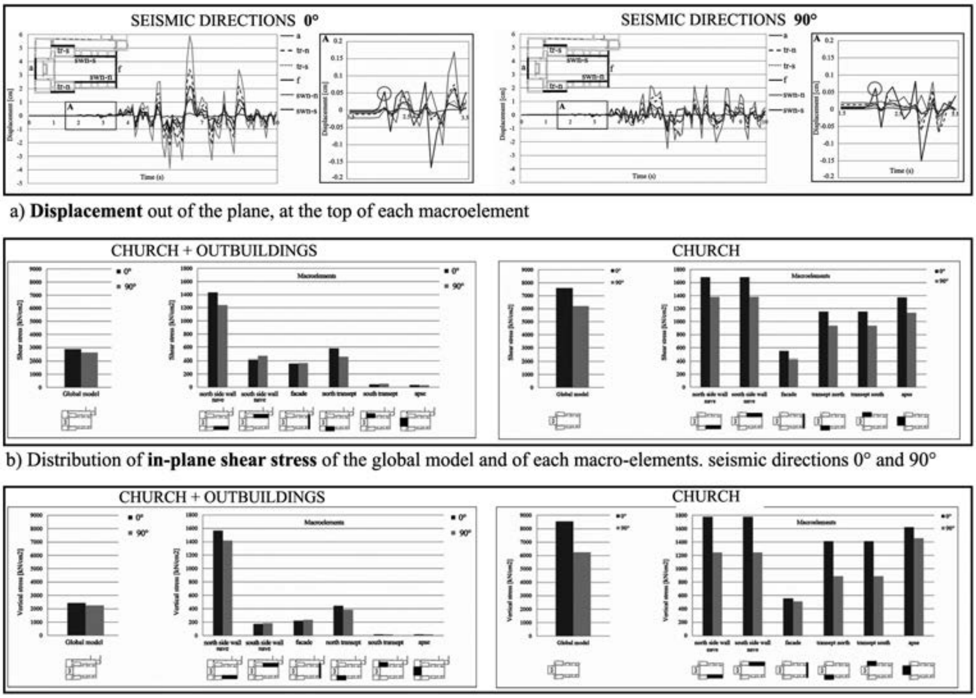

Figure 8. Distribution of vertical stress of the global model and of each macro-elements – seismic directions 0° and 90°.

Regarding the relationship macroelement-structure, we can conclude that:

- the first vibration modes of the global model are in longitudinal and transverse direction, while for the macro-elements the modes are orthogonal for the main overturning mechanism of masonry wall;
- the irregularity and the loss of the box-like behaviour affect the possibility to carry out the dynamic identification through the ME;
- thanks to the monitoring through the sub-parts (via multi-run procedure) and the subsequent correlation of these it is possible to identify the local modes highlighting the ME behaviour.

In reference to the outbuilding's effect: the model of WS without outbuilding presents an increment of stresses by 60% with a negligible difference between the macro-elements. In the model with outbuildings the results between the macro-elements differ greatly with a decrement of the stresses of about 70% for the constrained macro-elements. The damage may have occurred both to the intrinsic vulnerability of the church, the presence of the outbuilding and the direction of the seismic action.
The results show that the real seismic action, parallel to the nave, has resulted an increment of stresses of 10% compared to the simulation with the orthogonal seismic action.

5 DISCUSSION

The activities carried out are fully consistent with the timesheet reported in the initial proposal of the Reluis II project and with those provided for the Task 3.1.3. The planned objectives have been achieved and the results are satisfactory even though a further development on larger real structures, infrastructures and for cultural heritage shall be performed. The subdivision of the research project into activities and their subdivision in arguments and phases allows checking the development and the fulfilment of the proposed intermediate goals. New monitoring protocols have been developed both for ambient vibration tests on structures and infrastructures and to analyse the time-varying behaviour of structures during earthquakes. In addition, the algorithms for the automatic damage detection of strategic structures have been enhanced. The proposed procedures have been tested and validated using in laboratory and in situ tests. Especially during the last year of activity, the research has been characterized by new synergy and by a strong interaction among the different research units.

6 VISIONS AND DEVELOPMENTS

Thanks to the Reluis II project and the researchers involved in the Task 3.1.3 new algorithms, new protocols and new automatic techniques for Structural Health Monitoring and Damage Detection of civil structures and infrastructures, and for cultural heritage, particularly for churches, have been developed and validated using numerical and experimental experiences. New codes and guidelines to rule the use of dynamic identification techniques for Structural Health Monitoring purposes have to be implemented. It is important to provide rules to design and to implement monitoring systems on structures, cultural heritage, infrastructures and soil.
From the research point of view, in order to reduce the uncertainty on the structural parameters estimation, it is now important to integrate techniques able to detect the stationary behaviour of a generic structure with techniques able to analyse the time-varying behaviour of structures subjected to exceptional loads like earthquakes. As well as, a key topic is related to discriminate the contribution of structural and non-structural elements, and the contribution of

the soil-structure interaction, on the dynamic behaviour of a structure under ambient vibration, weak- and strong-motion earthquakes. Using this kind of approach it will be possible to use easily the dynamic parameter retrieved from an existing structure to calibrate a numerical model both for vulnerability assessment and for retrofit interventions. At the same time, considering the fast evolution of the technologies that could be used to improve the performance of a monitoring network (including also the seismic stations), it will be also necessary to better understand the real performance level of low cost technology compared with the standard technology used for dynamic identification of soil and structures, and the related technology for data transfer and data storage.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 387-407, doi: 10.14599/r101311, © 2015 Doppiavoce, Napoli, Italy

PREPAREDNESS AND RAPID RESPONSE

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1 INTRODUCTION

This manuscript describes activities carried out in the framework of the Research Line 3.2 – *Services for emergency management and rapid response-* following three tasks, specifically: 3.2.1 - *Organization and management of technical support to Department of Civil Protection,* 3.2.2 - *Organization and management of technical support to the professional community* and 3.2.3 *Management of database and experimental tests by RELUIS.*
The manuscript is organized considering two main topics, respectively, aimed at explaining the activities carried out in the framework of the organization and management of technical support to the Department of Civil Protection and to the professional community and those related to the management of the databases of experimental tests provided by the ReLUIS's researches.
Specifically, the main goal of Task 3.2.1 was to clearly mark out, to refine and improve procedures for the post-earthquake emergency management through a series of activities aimed at sharing, at the national level, an unique approach for the technical management of a seismic emergency. On the same way, Task 3.2.2 aims at improving the cooperation between ReLUIS and the Italian professional communities through a suitable exchange of scientific and technical information.
With regard to Task 3.2.3, although significant steps have been made individually in the network of laboratories RELUIS, a common strategy for the archive of experimental data only partially exists. As shown in the activities of US NEES (Pauschke and Mujumdar, 2004, Reinhorn et al., 2004), an important improvement of the experimental ability of a network of laboratories and in the use of experimental results can be achieved by using a common database, by the use of a common formatting of data and the predisposition of a prototype of teleparticipation and telepresence among different laboratories. Moreover, given the potential of Internet, and in particular the possibility of viewing the results of experimental tests by operators and professionals via web, results definitely to be an asset for the usability of the network of RELUIS laboratories.
In general terms, this activity aims at improving the dissemination and use of experimental results, the impact of promoting technical research in the field of earthquake engineering, encouraging innovation and the reduction of seismic risk and providing a broad and solid foundation for the calibration of numerical models. More in particular, this requires the harmonization and unification of database activities and the ability to access data through a single portal.

The combination of the aforementioned aspects (Pegon, 2007; Fardis, 2009) allows for: i) making more visible experimental findings both in the academic experimental and in the professional field; ii) permitting a greater interaction between the various laboratories and providing a better quality of results thanks to cumulative experience of those involved; iii) better documenting test results and allowing for a better interpretation of data obtained; iv) improving the accessibility and recoverability of experimental results also carried out in the past. Those are some of the goal that research partners have explored. In greater detail, the following objectives have been pursued: i) development and implementation of a platform called the Distributed Database (DDB) to enable the sharing of data and knowledge amongst the Italian University laboratories; ii) enrichment of the database with Laboratory data.

2 BACKGROUND AND MOTIVATION

2.1 Organization and management of technical support to Department of Civil Protection

During the emergency following the 2009 L'Aquila, but also after the 2012 Emilia, 2012 Pollino, 2013 Lunigiana-Carfagnana earthquakes, the importance of the cooperation between the professional and scientific communities was further demonstrated. In fact, the technical and scientific support to be given to professionals, particularly concerning the complex and demanding emergency management phases, became very important. After these earthquakes, several technical guidelines, training courses, seminars and public meetings were purposely performed by the ReLUIS team aimed at clarifying partial or misleading evaluations of what actually happened and, particularly, aimed at facilitating the practical application of methods, techniques and technologies to intervene on existing buildings, also taking into account economic and administrative issues. This is a crucial factor on which the technical support to the professional community should be based, especially after the enactment of the rules imposing the postgraduate training for all technical professionals.

The emergency following the 2009 L'Aquila, 2012 Emilia and 2013 Lunigiana-Carfagnana earthquakes has also shown the importance of the post-earthquake inspections aimed at assessing the usability and seismic safety of buildings. The ReLUIS consortium has been extensively involved in the first stage of the post earthquake emergency in Abruzzo and Emilia. It has supported the usability surveys of buildings, with particular attention to school buildings, strategic industries and sites with historical and monumental interest. Researchers from different Italian universities have contributed to surveys in the L'Aquila aftermath (more than 600 technical survey teams) and in the Emilia aftermath (about 250 researchers).

Several criticisms have been raised during the in situ inspections mainly related to the emergency-context and to the articulated framework in which such activities are carried out. These activities are crucial in order to quickly identify the buildings and the areas with a potential risk for the population and those without use restrictions. A primary goal in the post earthquake emergency is to reduce inconveniences to citizen as fast as possible. Therefore, a priority in the first stage of the post earthquake emergency is the accurate filling of the AeDES form. The AeDES form aims at evaluating the safety conditions of buildings in order to enable people to return to their houses and social and economical activities to start again. The form can be filled based on the visual in situ inspection of the entire building which represents the minimum structural unit with a significant impact on the people safety.

This form has been adopted after the Umbria-Marche earthquake in the 1997 and in all the subsequent major Italian seismic events. However, this form as well as the instructions to correctly fill the form have been formally approved in a proper Decree of the President of the Council of Ministers issued on 5 May, 2011. Such a Decree states that all government departments, regions, autonomous provinces of Trento and Bolzano and local authorities provides the AeDES form and the instruction manual to their operative structures. Furthermore, the Article 1 decreed that "*to support the post- earthquake inspection programs, the administrations of the State, of the regions and of the autonomous provinces of Trento and Bolzano may create lists of technicians who have received an appropriate training with a final verification and periodic updates, planned with the Department of Civil Protection. Registration lists must be confirmed every five years, after an upgrade training carried out also through electronic procedures. They shall be transmitted annually to the Department of Civil Protection of the Presidency of Council of Ministers by 31 December*".

In this framework the activity of the consortium ReLUIS was aimed, with the support of the Department of Civil Protection, at developing specific meetings to deeply analyze the main aspects related to the post earthquake emergency and to create a proper list of technicians to be mobilized in accordance with emergency procedures in case of earthquake.

According to Article 1 of the Decree of the President of the Council of Ministers, 5 May 2011, the Department of Civil Protection in cooperation with the administrations of the Regions and of the Autonomous Provinces, has been involved in a series of actions to improve the management system for the technical operations of damage and usability survey in the post-seismic emergency. The goal is to create several lists of technicians trained for these activities to be mobilized in accordance with emergency procedures. The ReLUIS consortium activity focused on a creation of an own reference list of experts. This to create a group of expert technicians to be mobilized for the aftermath damage and usability surveys. To this purpose, specific tools and effective education programs have to be developed and carried out in order to ensure:

- the application of technical standards for earthquake design, as effective and uniform as possible, throughout the whole country;
- the realization of a task force of professionals, suitably widespread throughout the country, to be trained in "peace-time" thus having expertise to effectively cooperate during the post-event activities. This ensures a prompt availability of technical experts able to perform demanding survey activities for the usability judgement in the immediate aftermath of a seismic event, thus providing a fundamental support to the National (e.g. Department of Civil Protection) and local structures involved in the seismic emergency management.

2.2 Organization and management of technical support to professional community

Following the same way, in the framework of the general activities aimed at increasing the capacity in the mitigation of seismic risk, also the goal of Task 3.2.2 - *Organization and management of technical support to the professional community* - is contributing to spreading and enhancing the culture of seismic design and of emergency planning and management within the professional community. In fact, the Italian professional community is made up of hundreds of thousands of technicians. They work as individual professionals and as employees of private companies or within government offices. With regard to the engineering community, the last available statistical report "Employment and remuneration of engineers in Italy - Year 2011", provided by the Italian National Council of Engineers (CNI), points out as the employment rate of the engineers, though reduced with respect to 2008 when it reached 78.4%, was around 75% in 2010, confirming a large advantage with respect to the whole Italian working population. Interestingly, the number of graduates in engineering is increased from 393,000 units in the 2004 to 572,000 units in the 2010, showing that, especially in the current time of economic uncertainty, great attention is placed in the courses of engineering study. Also data about employment are encouraging. They show that, despite the number of employees "under 35" declined to 63.2% in 2010, i.e. 8% less than in 2009 (71.5%), engineering graduates continue to have more chances to work than most of the other graduates. Besides, a growing number of professionals, who in 2010 reached 27.8% of the total compared to 26.8% of the previous year, is shown. Among individual professionals, the majority of them (about 90%) currently work in the building industry. Most of them are, and even more they will be in the coming years, directly involved in the important changes introduced by the design code framework set up in Italy after the 2002 Molise (OPCM 3274) and 2009 L'Aquila earthquakes, nowadays fully in effect (NTC 2008) and being further update. In fact, among the main consequences due to the new rules for the building industry, particularly in the seismic zones, there are:

- the need of continuing education and the resulting strong demand from the professional community;
- a more continuous and fruitful cooperation between professional and scientific communities mainly aimed at enhancing demand, quantity and quality of lifelong learning.

Such issues were already taken into account effectively by the OPCM 3274 law, where it was clearly stated that the Italian National Department of Civil Protection had to promote and implement education programs on seismic risk, also involving Regional institutions and professional communities (e.g. for the engineers through the CNI). The ReLUIS consortium was already identified as scientific reference for education activities to promote a wide, uniform and effective application of the new technical standards throughout the country. In this context, it is worth citing also the collaboration agreement signed between the Department of Civil Protection and the CNI in 2003 mainly aimed at:

- selecting and training a group of engineers for subsequent education activities in specific courses mainly directed to members of the 106 provincial Engineer Councils in Italy
- disseminating the knowledge on the new earthquake engineering design standards using standardized training programs.

Following this agreement, as well as those signed with other professional categories, many of training courses devoted both to graduated technicians (engineers, architects and geologists) and technicians holding a diploma (surveyors) have been carried out in the last years, mostly after the 2003. At the same time, technical handbooks on various aspects of seismic design, published by ReLUIS and Eucentre (European Centre for Training and Research in Earthquake Engineering) in collaboration with the Department of Civil Protection were prepared in order to further develop and enhance the relationships between scientific and professional communities.

2.3 Management of database and experimental tests by RELUIS

Due to lack of data integration among seismic laboratories belonging to the RELUIS network, there is an urgent need of creating a unique platform for Italian Universities Laboratories capable of sharing seismic experimental data and knowledge. Therefore, a central database with centralized access to database nodes that are distributed over the network is needed. This database will be able to dialog with a central portal in a uniform manner. According to the same perspective, and in order to foster a sustainable culture of co-operation among Italian research infrastructures and teams that are active in seismic experimental activities, the implementation of a distributed hybrid simulation framework was set. Its validation on a simple split-mass Single-Degree-of-Freedom system across the laboratories of UNINA and UNITN is presented.

3 RESEARCH STRUCTURE

The research program of Task 3.2.1 involved a series of activities aimed at sharing, at the national level, a unique approach for the technical management of a seismic emergency. To this purpose, specific meetings have been foreseen and carried out in order to ensure the promotion of standards, procedures, terminologies and operating methods to be properly adopted in case of damage and usability surveys. The program involved several meetings organized by the Department of Civil Protection in collaboration with ReLUIS. These training sessions were addressed to university professors and researchers of the Center of Competence ReLUIS, but also to post-docs, PhD students and employees. The standardization of procedures as well as the criteria adopted for a quick damage, vulnerability and post- seismic usability assessment of a building are key conditions for an homogeneous approach at the national level.

Regarding the professional community, the research program regarding the Task 3.2.2 was divided in the following activities:

a) *<u>Cooperation between ReLUIS and Italian professional communities</u>*
The main goal of this activity is defining actions aimed at improving the cooperation with the various professional categories, mainly through their representatives at national level (National Councils), specifically with the National Council of Engineers. This cooperation should encourage the realization of a national network among the various professional councils and the technical offices operating within the public administrations (e.g. National Civil Protection, Firefighters' Corp). The demands from the professional community are taken into account in the implementation of the activities b), c) and d) described below.

b) *<u>Definition of education standards</u>*
The main goal of this activity is setting up education programs according to the general objectives and targets of the Task. The education programs will be based on appropriate contents and bearing in mind the objective of ensuring uniform standards throughout the country. The main steps carried out are:
   - setting up a database containing a number of technical courses on the seismic risk already carried out in Italy;
   - defining a suite of education programs with format (contents, duration, etc.) specifically designed for the different professional categories taking also into account the work position, that is individual professionals, employees of private companies or within government offices;
   - setting up questionnaires for the *ex-post* evaluation of the education activities;
   - developing pilot courses for each professional category;
   - defining alternative solutions to the traditional *vis-à-vis* lectures to make teaching and learning activities less expensive but still effective; all forms of electronically supported learning and teaching (e-learning) are considered, e.g. providing video lessons prepared by a selected group of experts.

c) *<u>Dissemination of Guidelines and Manuals</u>*
Preparing Guidelines, Codes of Practice, and Manuals is not a goal of Task 3.2.2. On the contrary, its objective is promoting the widest diffusion of the main technical documents (e.g. Handbooks, Guidelines) already available, or to encourage the preparation of documents able to support the professional community within the other research Lines of the current DPC-RELUIS Project.

d) *<u>Exchange of scientific - technical publications and other information</u>*
Pursuing more fruitful and wide relations with the professional community requires that the interaction is bi- and not uni-directional, as frequently happened in the past. This could

be achieved, for example, by involving technical experts, both working as professionals and as employees in public offices, in preparing and carrying out research projects, preparation of seminars and conferences, degree or master thesis, etc. developed in the fields of earthquake engineering and/or emergency management.

The aim of the Task 3.2.3 is to achieve the following three goals:

- Development and implementation of a platform called the Distributed Database (DDB) to enable the sharing of data and knowledge amongst the Italian University laboratories;
- Enrichment of the database with Laboratory data;
- Development and execution of a distributed test between UNINA and UNITN.

4 MAIN RESULTS

4.1 Organization and management of technical support to Department of Civil Protection

Regarding Task 3.2.1, some meetings with university full professors, assistant an associate professors, post-docs, PhD candidates and employees have been carried out. In particular, the meetings consisted in three days: two days dedicated to seminars and a third one to a critical discussion on several case studies.

As concern as the seminars, the topics were: i) Introduction to the Italian National Civil Protection; ii) Civil Protection approach for emergency management; iii) Emergency technical activities: the Function damage census and usability assessment; iv) Usability assessment of ordinary buildings: the AeDES form ; v) The role of ReLUIS consortium in the post earthquake emergency management; iv) The *in-situ* surveys on public buildings: the case studies of L'Aquila school buildings; v); Usability assessment of precast structures.

These seminars were obligatory for post-docs, PhD candidates and employees of ReLUIS consortium.

They were carried out in two times: 8-9 April 2013 and 15-16 April 2013 at the Department of Civil Protection offices in Rome; on both cycles, the seminars were introduced by a short intervention of Franco Gabrielli (Head of Civil Protection Department), Gaetano Manfredi (President of ReLUIS consortium), and Mauro Dolce and Mauro Rosi (Civil Protection seismic and volcanic risks office).

The participants at these seminars were **125** (8-9 April) and **140** (15-16 April); they came from several Italian Universities (Calabria, Chieti-Pescara, Firenze, Genova, L'Aquila, Napoli Federico II, Napoli Parthenope, Padova, Palermo, Perugia, Roma Tor Vergata, Salerno, Sannio, Trieste, Udine, Basilicata, Bologna, Camerino, Cassino, Chieti-Pescara, Firenze, Messina, Milano Politecnico, Napoli Federico II, Pisa, Roma La Sapienza, Roma Tre, Salento, Trento).

Figure 1. Seminar on technical management of post earthquake emergency, 8 April 2013 - Department of Civil Protection, Rome.

As concern as the case studies, one day meeting was organised in several Italian cities. The one day meeting consisted of a training on usability surveys followed by a comparative analysis of case studies. This in order to share standards, procedures and terminologies. The trainings have been performed by a working group that illustrated the case studies and moderated the debate and comparisons. These trainings were obligatory also for Assistant Professors, Associate Professors and Full Professors of the ReLUIS consortium. This meeting concluded the project working meetings on the topic "Technical management of the emergency, damage and usability survey ". The one day meeting has been carried out in seven different days in Italy in order to easily allow the participation of researchers from all Italian Universities.

Figure 2. One day meeting on usability surveys.

In particular, they were carried out according to the following schedule:

- Naples (building of via Forno Vecchio), 10 May 2013: **68** participants (from Naples University of Federico II, University of Basilicata, University of Calabria, University of Salento, University of Salerno, University of Sannio)
- Napoli, (building of via Claudio 21), 13 May 2013: **56** participants (from Naples University of Federico II, Naples University of Parthenope, University of Enna)

- Rome, 21 May 2013: **65** participants (University of Sapienza, University of Tor Vergata, University of Roma Tre, University of Messina, University of Palermo)
- Milan, 31 May 2013: **53** participants (Polytechnic of Milan, University of Genova)
- Bologna, 11 June 2013: **69** participants (from University of Bologna, University of Camerino, University of Firenze, University of Perugia)
- L'Aquila, 14 June 2013: **47** participants (from University of L'Aquila, University of Chieti-Pescara, University of Cassino)
- Padova, 19 June: **59** participants (from University of Padova, University of Trieste, University of Trento, University of Udine)

Once the seminars were carried out a proper section has been inserted on the ReLUIS website containing some tests and videos of survey case studies. The activities allowed to define a list made of more than 400 researchers and technicians, involved in the ReLUIS consortium, trained for the aftermath damage and usability surveys. They can be mobilized in accordance with emergency procedures in case of seismic event.

4.2 Organization and management of technical support to professional community

Regarding Task 3.2.2, with respect to the actions reported in the previous section, some meetings with representatives of the national technical professional categories, particularly with the National Council of Engineers, have been held during the 2010-2013 ReLUIS Project. These representatives pointed out the peculiar interests of the professional community and provided requirements for a fruitful cooperation within the main objectives proposed by the Project, particularly concerning education programs.

In order to define and analyse activities carried out in the past years, a database (DB) of the most remarkable education courses on seismic and structural engineering themes was prepared collecting information mainly from: i) the meetings with the professional community, ii) the internet web-sites of the national and local councils of the involved professional categories, iii) the agreements and education activities carried out in the past years (particularly after the OPCM 3274/2003 and NTC 2008).

Specifically, the main aspects considered and analysed in the DB include:

- who teach ? (e.g. Universities, private structures, etc.);
- who learn ? (e.g. engineers or architects, individual professionals or employees in public offices, etc.) and
- which are the typical characteristics of the courses ? (e.g. content, duration, etc.).

Although not exhaustive, the DB is sufficiently large to understand the main characteristics of the education demand. Summarizing, the professionals involved in the courses were mostly engineers, and most of courses were devoted to structural design and technical codes, with a lower percentage devoted to emergency management.

Starting from the preliminary results, a permanent and deepened cooperation has been defined with the Italian National Council of Engineers (CNI). As a first activity of the cooperation, in order to collect demands and suggestions from a selected sample of technicians (e.g. the chairmen of the local Councils), a questionnaire named *"Questionario informativo/conoscitivo RELUIS – CNI"* has been prepared and has been sent to all the 106 local councils in Italy. The first part of the questionnaire shortly presents the RELUIS consortium, also describing objectives, activities and main results of the past DPC-RELUIS 2005-2008, as well as objectives of the current DPC-RELUIS project mainly focusing on Task 3.2.2. The second part contains several questions (either multiple pre-defined or free response) to be answered by the representatives of each local Council specifically concerning the education programs carried out by each council in the past five years. Besides, suggestions on future courses and possible cooperation activities are asked.

Although only a limited share of questionnaires has been presently compiled and returned (about 20%), processing the collected data some remarks can be done. As for the knowledge of ReLUIS network and past activities, the main results show that:

- about 40% of councils did not know ReLUIS before reading the questionnaire;
- about 85% did not have previous cooperation with ReLUIS;
- practically in all the councils, educational activities concerning earthquake engineering have been organized during the past five years.

As for the typical characteristics of the courses carried out in the past five years, they had both general and thematic content. General courses were mainly devoted to describe content and update of structural codes, while the courses on special structures (e.g. bridges) or emergency management had typically thematic content. Topics such as earthquake design of new buildings, and seismic assessment and retrofitting of existing buildings were offered during courses either general or thematic. Finally, the topics mainly suggested for future courses are assessment and retrofitting of existing structures, structural codes and guidelines with particular emphasis on application examples, innovative retrofitting methods and techniques (e.g. base isolation).

Based on the analysis of the data collected through the questionnaires some actions are in progress to promote and enhance the cooperation with the community of engineers. Specifically, some criteria for continuing education and refresher courses have been identified considering both general and thematic courses.

Moreover, some conferences, meetings and workshops have been carried out aimed at presenting and discussing specific topics where the role of scientific, technical, economic and social aspects has been considered and discussed. Particularly, in this context it is useful to make an analysis of the experiences and lessons learned in order to examine the possible role of the engineers with specific connection to the increasingly broad and fruitful cooperation with the scientific community.

These issues were carefully discussed during an *ad hoc* conference named "*The challenge of seismic safety in new and existing constructions: technical, economic and social perspectives*" held in Potenza (Southern Italy) on March 9th, 2013. The initiative was sponsored by the National Council of Engineers and the Regional Federation of Engineers of the Basilicata region with the cooperation of the City of Potenza and the University of Basilicata (UR-UNIBAS). Representatives of research centres committed to research activities on seismic risk (ReLUIS, EUCENTRE), National Council of Engineers and some researchers working at the Universities of Naples, Bologna and Basilicata participated at the conference.

Based on these activities a cooperation agreement between the National Council of Engineers and ReLUIS is currently under discussion. This agreement aims at regulating the collaboration for joint future activities in the field of seismic risk mitigation with special regard to training activities, and the definition and application of technical regulations and guidelines for construction industry.

In companion with the draft of agreement under discussion, an agreement between ReLUIS and the National Institute of Oceanography and Experimental Geophysics (OGS, Trieste) has been signed on November 23rd, 2012. This agreement aims at regulating the collaboration on joint activities in the field of seismic risk mitigation with particular regard to seismic hazard, local seismic response and monitoring of soil-structure interaction.

With respect to activities concerning emergency management and particularly those provided in cooperation with the Task 3.2.1, an effective collaboration has been set up with the world of volunteers of civil protection in order to promote a more active role of citizens in seismic risk reduction. To this purpose, an agreement with the National Association of Public Assistance (ANPAS) has been signed on September 29th, 2012 with reference to the following activities:

- Involvement of volunteers in the national system of civil protection;
- Development of studies and research carried out by ReLUIS through a more extensive transfer of knowledge to population, local administrators and technicians;
- Design and execution of training courses for volunteers, both technical and non-technical;
- Preparation and execution of outreach programs and seismic risk information to common people;
- Design, execution and analysis (debriefing) for civil protection exercises for the simulation of seismic emergencies;
- Preparation and implementation of joint research projects on seismic safety in the context of national and international research programs.

Future activities will be carried out on the basis of specific acts opportunely inserted within the activity programs of the National Department of Civil Protection.

Still with respect to the contribution to the activities concerning emergency management (Task 3.2.1), during the project a strong collaboration between DPC, ReLUIS, INGV (Italian Institute of Geophysics and Volcanology) and OGS (National Institute of Oceanography and Geophysics) has been established in order to transfer information to local technicians and administrators, as well as to civil protection volunteers and common people. These activities have been carried out prevailingly during the seismic swarm that is affecting the Pollino mountain range (Basilicata and Calabria regions, Southern Italy) since October 2010. Being not possible to predict the evolution of the sequence, currently ongoing, the civil protection offices, at national and regional level, took several initiatives to help people to cope with the swarm and to prepare to possible future large events. The National Department of Civil Protection and the Civil Protection Office of Basilicata Region decided to put in action some measures aimed at verifying and enhancing emergency preparedness. These actions have been carried out with a constant and fruitful collaboration among the main stakeholders involved (scientific community, local and national governmental agencies, civil protection volunteers, etc.). Particular attention has been devoted to the implementation at local scale of a national prominent initiative for seismic risk reduction, that is the national campaign *"Terremoto, Io non rischio - Earthquake, I do not risk"* (TINR, www.iononrischio.it) promoted by DPC and ANPAS in collaboration with INGV and ReLUIS (Task 3.2.1).

Figure 3 reports training and communication activities of civil protection volunteers (a), students (b) and population (c) carried out during the seismic swarm (More information are available in Masi et al., 2013).

The first edition of TINR campaign was held on 22 and 23 October 2011 in the squares of nine Italian towns located in high seismicity zone. A selected group of volunteers was firstly trained by experts from DPC, ANPAS, INGV and ReLUIS on basic concepts concerning hazard, vulnerability, seismic risk and communication procedures. Then, in turn, they trained other volunteers widening the number of actors to be committed in the process of knowledge diffusion to the population. During the first campaign 120 volunteers distributed information, illustrative material and provided answers to the citizens' questions on possible individual actions to carry out in order to mitigate seismic risk.

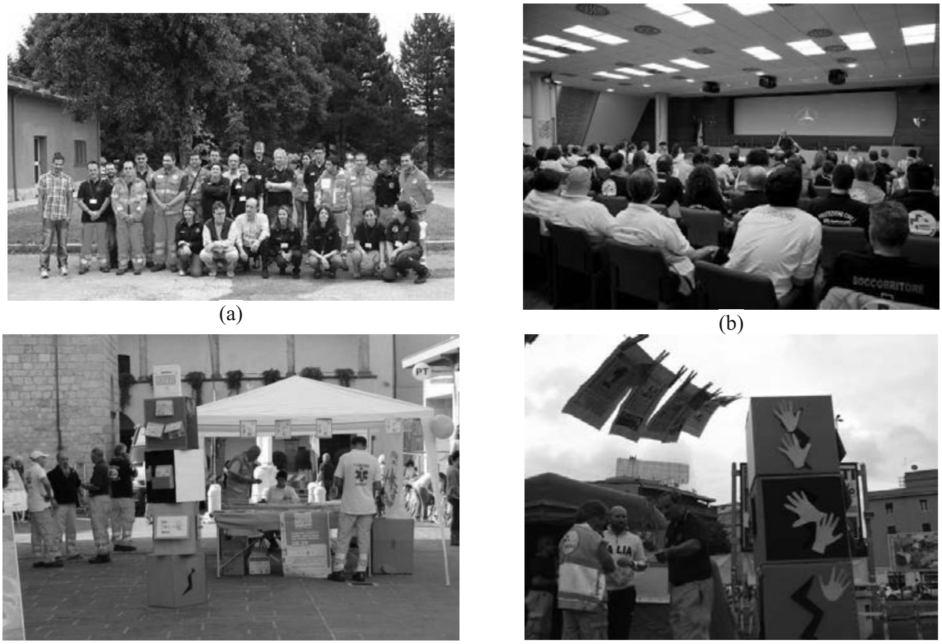
(a)

(b) (c)

Figure 3. Training and communication activities of civil protection volunteers (a), students (b) and population (c).

As a consequence of the positive feedback and results of the 2011 campaign, a second edition was planned on October 2012 extended to about 100 squares throughout the country (two of the involved locations are in the area affected by the Pollino seismic swarm, that is the villages Lagonegro and Rotonda). In the second edition of TINR campaign over 1.500 trained volunteers from 12 different national associations working on civil protection have been involved. The third edition of the campaign was planned on September 28 and 29, 2013, in more than 200 Italian municipalities (mainly located in areas classified as high seismicity zones). In the 2013 edition the trained volunteers involved in the campaign will be more than 3.000 units. In Figure 4 some pictures of the campaign have been shown.

Whereas the TINR campaign is designed and performed in "peace-time" (i.e. without emergency conditions) an analogous activity thought for the emergency phase following a damaging seismic event, has been performed after the May 2012 Emilia earthquake through the initiative named "*Terremoto - parliamone insieme / Earthquake: let's talk together*". It is mainly aimed at transferring information to local technicians, administrators and common people concerning the seismic sequence characteristics, criteria and results of the usability surveys, activities to be carried out to repair damage and strengthen the buildings in the affected area, etc. This initiative was promoted by DPC, INGV and local administrations, with the contribution of ReLUIS mainly involving local experts being variously part of the ReLUIS network. The initiative "*Terremoto - parliamone insieme*" has been also carried out recently after the M5.2 earthquake occurred on June 20, 2013 in the area of Lunigiana-Garfagnana, Tuscany.



(a) (b) (c) (d)

Figure 4. Some pictures of TINR Campaign activities.

4.3 Management of database and experimental tests by RELUIS

At present, different laboratories of Italian Universities store and manage experimental data in various fashions. Each laboratory deals with data with a unique local data model and user interface, language and scheme. Therefore, the dissemination and use of these experimental results outside the laboratory where they are produced can be problematic (Taucer, 2011). To overcome this, UNITN is adding a layer on top of the existing local databases that is accessible through a unique Data Access Portal (Bosi et al., 2013; Hasan et al., 2013, 2015).

With the increasing presence of high speed and more reliable networks the earthquake engineering community has developed over the years, different communication protocols in order to perform tests that are not limited to their own facility and therefore, take advantage of equipment that would have been otherwise impossible to have the hands on.

The ability to perform distributed tests is not a complete solution by itself since it provides means to connect laboratory equipment, not to connect researchers and to share results. Further work has been made in order to provide a full set of tools to perform distributed hybrid testing, share results and communicate efficiently.

In order to foster a sustainable culture of co-operation among all of the research infrastructures and teams that are active in seismic experimental activities in Italy, a distributed hybrid simulation framework was implemented. It was based on the well-known OpenFRESCO (Open-source Framework for Experimental Setup and Control) software (Schellenberg et al., 2008) and allowed for the simulation of a S-DoF split-mass system across the UNITN and the UNINA remote laboratories.

An ontology named as Earthquake Engineering Research Projects and Experiments (EERPE) has been developed; it uses a faceted approach that gives emphasis on research project management and experiments. With the invention of the Semantic Web, computing paradigm is experiencing a shift from databases to Knowledge Bases (KB), where ontologies play a major role in enabling inferencing that can make hidden facts unconcealed to produce better results for users. Moreover, KB-based systems provide mechanism to manage information and semantics thereof that can make systems semantically interoperable and as such can exchange and share data between them. To overcome the interoperability issues and to exploit the benefits offered by the state of the art technologies, the development moved to the KB-based system.

Figure 5. Some pictures of "Terremoto Parliamone Insieme" provided after the 2012 Emilia earthquake.

In this framework the University of Trento took an initiative to develop a prototype database and interface for the National Italian laboratory. In detail, user interface is a set of webpages. This is actually the view of the system. Html files with embedded java code for data presentation are used along with JavaScript and Ajax to make it more interactive; see Figure 6, in this respect.

In cooperation with UNINA and from a conceptual point of view, the Data Access Portal has been designed to act as an information space. Organizing functionality and content into a structure that users are able to navigate intuitively is not a trivial task. Researching the suitable Information Architecture of the Data Access Portal environment is of great importance. Effective information architecture enables users to step logically through a system aiming at supporting them getting closer to the information they require. Lacking a suitable Information flow increases the risk of creating great content and functionality that cannot be found. The content will be mostly fed into the system by the distributed databases that are maintained on the laboratories sides. However, the distributive character of the database makes the decision of the suitable information containers much more difficult.

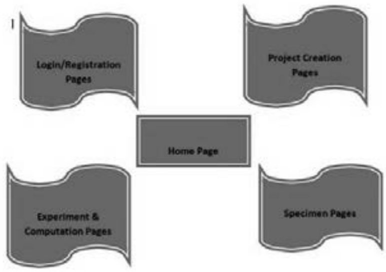


Figure 6. The organization of the views of RELUIS application.

The main idea in structuring the database was to store the basic data, provided by the researchers (in papers, reports, etc.), but also to be able to provide the derived data, which may assist researchers in their analyses (i.e. developing seismic performance models for different RC load bearing elements). Extracted and post-processed data may be used for various statistical studies in a user-friendly way, and for developing databases for using in a research and for developing performance/capacity models of structural elements.
The interface was designed to enable:

- Login/registration pages (to collect pages related to user authentication and registration);
- Database access (functionalities to interact with the whole database internal structures in a user-friendly way)
- Management of local users (Web interface allows different local users to access the database using different functionalities within the interface. The list of users connected in the web interface is shown in Figure 7)

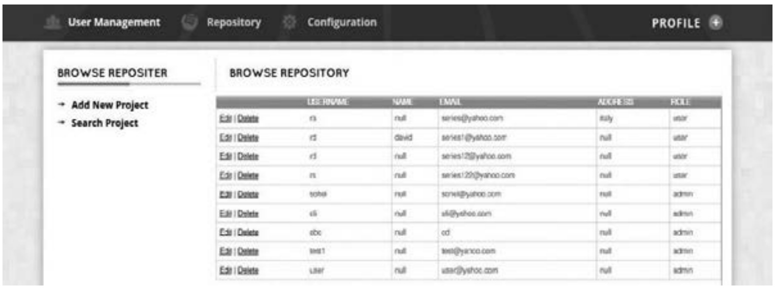


Figure 7. List of users.

Within the RELUIS research project, a distributed test between two nodes, i.e. UNITN and UNINA. Both UNITN and UNINA have developed telepresence capabilities as outlined in previous reports of RELUIS. Their implementation regards the possibility for a remote user to participate to a test. Despite not being physically present in the Lab during testing, the user is provided with sufficient key information allowing him/her to actively participate. Thus, more end-users are involved, increased visibility is achieved and time and cost expenditure are reduced. They also worked synergically to support the operation and interconnection of resources of (distributed) facilities and especially to define the experiment workflow engine, agreements between participants, the transmission of data and the preparation of results for their storage. Specific test capabilities require different approaches in the implementation. For instance during shaking table tests it is required to show what does occur in the laboratory; synchronized laboratory results may not be always available in real-time, and slow motion and various Sources for data & videos can be provided off-line. For instance during pseudo-dynamic tests, it is easier to display synchronized laboratory results, on-line, real-time (or accelerated motion in play-back). In fact typical Telepresence issues are bandwidth, firewalls and time synchronization.

The simple split mass S-DoF system was intended to investigate both the feasibility and the critical parameters that affect such testing procedure. In detail, the time integration algorithm and the Numerical Substructure (NS) ran at UNINA, whilst the Physical Substructure (PS) was handled at UNITN by means of a MOOG actuator. To this end, the XPC target - Scramnet - MTS Flex Test 90 architecture, based on the well-known OpenFRESCO software, was implemented. In detail, OpenFRESCO is a TPC/IP network interface between simulation nodes: UNINA and UNITN in the present case.

The block diagram of Figure 8 shows the architecture of the implementation of the distributed test (Abbiati et al., 2012). Light blue boxes belong the UNINA side, whilst red boxes to the UNITN side. In detail, both the time integration algorithm and the NS emulated by the Experimental SimDomain of OpenFRESCO were implemented on a Windows based system at UNINA. Another Windows based machine was installed at UNITN and connected to the MTS FlexTest 60 controller through a SCRAMNET\xPC-Target based interface. Both the machines were provided with public IPs.

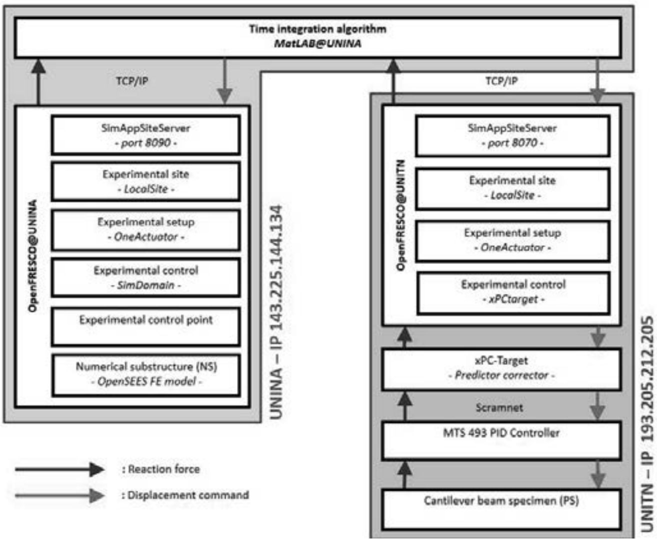


Figure 8. Architecture of the implementation of the distributed hybrid simulation.

In order to estimate the average transmission time between sending a target displacement and receiving the respective restoring force, a preliminary numerical simulation of the distributed test of an SDoF split-mass system was carried out. The OpenSEES software simulated the PS and NS at UNITN and UNINA, respectively. The minimum suitable simulation time step was selected according to transmission times logged by the Wireshark tool.

With reference to the entire ingoing and outgoing data flow, Figure 9 and 10 depict relative and cumulative distributions of transmission times.

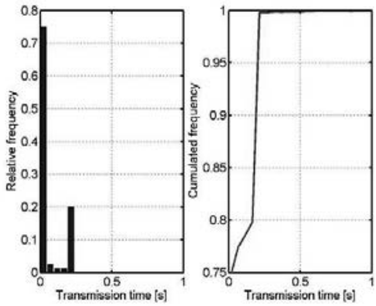


Figure 9. Relative and cumulative frequency distributions of the transmission time from UNITN (source) to UNINA (destination).

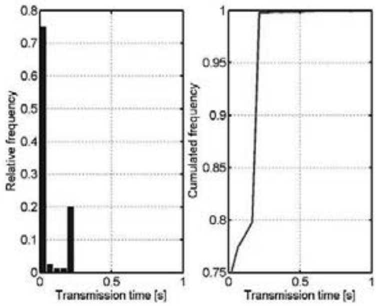


Figure 10. Relative and cumulative frequency distributions of the transmission time from UNINA (source) to UNITN (destination).

According to cumulative distributions, the maximum transmission time experienced by packages in both the directions can be assumed equal to 0.25s.
The structure selected for the distributed test was a simple portal frame with two masses concentrated at column-top nodes. The resulting split-mass SDoF system examined via distributed hybrid simulation is depicted in Figure 11.

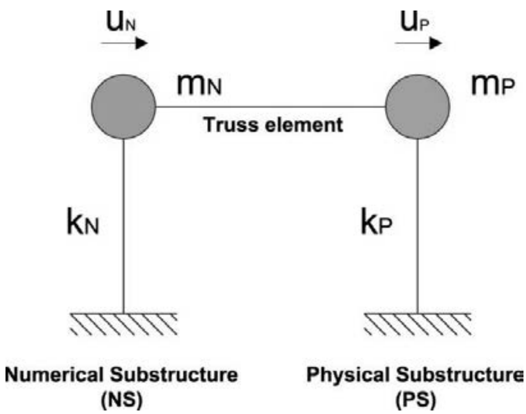


Figure 11. Split-mass Single-DoF System.

The comparison among experimental results and a linear numerical simulation of the tested system is shown Figure 12 and 13, respectively.

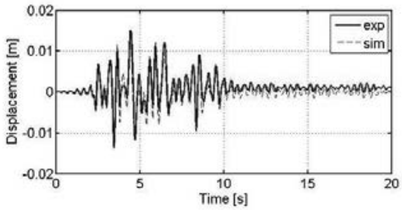


Figure 12. Comparison of experimental –exp- and simulated –sim- displacement responses.

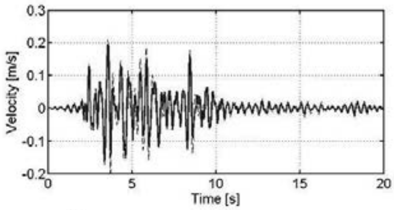


Figure 13. Comparison of experimental –exp- and simulated –sim- velocity responses.

Finally, a view of the terminal performing the numerical integration of the system and the element numerical substructure at UNINA is shown in Figure 14, that shows how the IP camera installed at the Laboratory of Structures and Material at UNITN provided a video streaming of the test to UNINA.

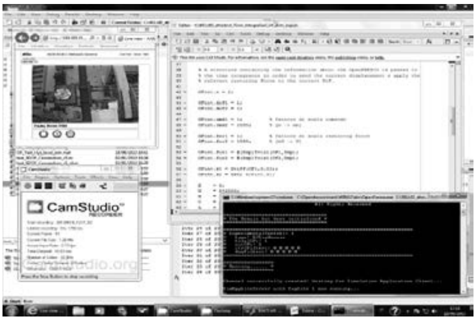


Figure 14. UNINA terminal performing the time integration and telepresence view of UNITN node.

5 DISCUSSION

A set of preventive actions can be carried out to mitigate natural risks working both to reduce the territorial vulnerability (technical actions) with respect to the specific natural hazard at hand, and to enhance the social capacity (cultural actions) of the involved community (people, authorities, professionals, researchers, etc). Even though the vulnerability reduction remains the main objective to be pursued in an effective policy of risk mitigation, building social capacity helps to increase the risk perception and awareness of people, thus their capacity to adapt to and cope with natural hazards. Seismic risk perception, governance and communication are worldwide a highly topical issue and even more they will be in the future also as a consequence of what happened in the pre- and post-event emergency management phases of L'Aquila (Central Italy) 2009 earthquake.

About these issues, in recent years, many public events, promoted by the DPC in collaboration with other components of the National System of Civil Protection (ReLUIS, voluntary associations, INGV) have been carried out. The two most significant initiatives have been "Terremoto, io non rischio" and "Terremoto, parliamone insieme", both designed to allow the communication between scientific community and citizens in order to transfer, in a simple but rigorous form, useful concepts and prevention behaviours , focusing in particular on the actions of self-protection.

The most important objective of these campaigns was to help people to overcome the fatalistic attitude which is often taken against earthquakes, explaining to citizens that, on the contrary, prevention is possible, and methods, tools and techniques to effectively reduce the effects of earthquakes are currently available.

In terms of sharing of data among research laboratories, an effective interface of the RELUIS Distributed Database was developed and was based on the PHP\MySQL framework. In detail, UNITN developed a visually appealing interface for creating, editing and deleting elements in the database regardless the database structure. Moreover, this interface allows different concurrent local users to access the database. Experimental measurements relevant to a real-time hybrid simulation of a full-scale piping network conducted in Trento were uploaded into the database to validate the interface software.

In order to foster a sustainable culture of co-operation among all of the research infrastructures and teams that are active in seismic experimental activities in Italy, a distributed hybrid simulation framework was implemented. It was based on the well-known OpenFRESCO software. A Single-Degree-of-Freedom (SDoF) split-mass system across the UNITN and the UNINA remote laboratories was considered. A steel cantilever beam acted as Physical Substructure (PS) in Trento, whilst an SDoF system was implemented in the OpenFRESCO environment as Numerical Substructure (NS) in Naples. The 2012 Emilia earthquake recorded in Mirandola was applied as seismic input and the distributed hybrid simulation was successfully conducted. In order to investigate the limitations of the method as well as the effect of the selected platform and hardware employed, future trials will be based on another platform, i.e. the UI-SimCor software (Kwon et al., 2005). Due to the Matlab based open source code, it is used to implement and verify new integration algorithms by researchers.

6 VISIONS AND DEVELOPMENTS

The positive results obtained by the information activities lead to attain in the following years an increasing number of citizens. At the same time, it is important to carefully examine the influence that some social factors have on individual behaviour and on the attitudes of the community respect to seismic risk prevention, in order to better understand the effects that the information activities have on the increase of social resilience respect to natural risks natural.

The international literature (e.g., Solberg et al., 2010) highlights the main factors that affect the population respect to the mitigation of seismic risk (seismic adjustment measures), particularly: (i) the perception of risk, (ii) the community customs, (iii) the social responsibility, (iv) the propensity to fatalism, and (v) the socio-economic development level.

Then, the availability of national investigation study focused on the influence of social, cultural and economic factors on decisions and behaviours of population with respect to seismic risk, and generally to natural hazards, could help to define models of analysis in which, beside factors directly related to the structural prevention (hazard, vulnerability, exposure), the "non-structural" prevention enters within the definition of policies for the seismic risk management.

In this framework, training activities to professional community have also a very important role. This issue was already addressed in the project DPC-ReLUIS 2010-2013 recognizing that most of the technicians working in the construction industry and therefore is, and will be even more in the coming years, directly involved in the changes of Italian (NTC Guidelines, etc..) and international (Eurocodes) regulatory framework.

The theme of training will become even more important in the coming years as consequence of the rules related to the required updating of professional competence.
This is a great opportunity to provide rigorous and uniform training throughout the national territory, particularly regarding the seismic risk assessment and management, as well as to identify more clearly the roles of the various professionals within the work carried out in the communication initiatives.
The recent earthquakes that have struck the Abruzzo region in April 2009 and the Po Emilia Valley in May 2012 have clearly demonstrated the centrality of the activities aimed at the analysis of the damage and to verify usability of buildings within the management activities of post-seismic emergency.
Then, in order to optimize the procedures for seismic emergency management in the future it is necessary to carry out activities aimed at provide a) a list of ReLUIS technicians to include in the Territorial Centres of Competence and within the National Technical Centre; ii) ongoing training of technicians on the technical management of the emergency.
The management of databases and experimental tests could be improved by a harmonization and unification of database activities and the ability to access data expanding the network from RELUIS laboratories to European network SERIES and US NEES network. The data integration among seismic laboratories belonging to different networks requires not only a uniform integration of informatics platforms, but first of all the agreement between researchers and general will and openness to cooperation at continental and world scale level.
Similarly to improve the sharing of knowledge and to make more fruitful the fruition of data stored in Distributed Databases by researchers, as well as local technicians and administrators, as well as by civil protection volunteers and common people, it is required the development and improvement of the seismic ontology. Databases have been serving quite effectively for dealing with large volumes of experimental data; however, they were not originally designed for managing Resource Description Framework allowing structured and semi-structured data to be mixed, exposed, and shared across different applications. This features opened up the opportunity for researchers, experimentalists, data scientists, data practitioners and many others from government, public and private sectors for unlimited share, use and reuse of datasets. In this way, a system provides intelligent query answering features, for instance a search starts from a user query and returns a set of documents that are ranked by relevancy with documents most similar to the query having the highest score.

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8 RELUIS REFERENCES

Differently respect to the other research lines, activities carried out within Line 3.2 - *Services for emergency management and rapid response* - are service and not research activities. As a consequence, generally their results are not provided in scientific publications but in technical and scientific cooperation agreements. However, some activities and results have been published on national and international conferences and journals, as follows:

Ludovico M. D., Prota A., Masi A., Manfredi G. (2013). “Sisma in Emilia Romagna: l’attività di ReLUIS”. *Progettazione Sismica*, N.01/2013.

Masi A., Mucciarelli M., Chiauzzi L., Costanzo G. D, Loperte G (2012). “Emergency preparedness activities during an ongoing seismic swarm: the experience of the 2011-2012 Pollino (Southern Italy) sequence”, *General Assembly of European Geosciences Union (EGU)*, Vienna, Austria, 22-27 April.

Masi A., Mucciarelli M., Chiauzzi L., Camassi R., Loperte G., Santarsiero G. (2013). “Emergency preparedness activities performed during an evolving seismic swarm: the experience of the Pollino (Southern Italy) sequence”, *Bollettino di Geofisica Teorica e Applicata (BGTA)*, DOI: 10.4430/bgta0115.

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RECENT ADVANCES ON SOME ASPECTS OF EARTHQUAKE GEOTECHNICAL ENGINEERING

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1 INTRODUCTION

The Geotechnical Line has contributed to the current project with three Macro-themes: Seismic response analyses and lifelines (MT1), Shallow and deep foundations (MT2), Retaining structures (MT3). For each Macro-theme, background and motivation, research strategy, main results and future developments are described in detail.

2 SEISMIC RESPONSE ANALYSES AND LIFELINES

2.1 Background and motivation

Since the well-known Mexico earthquake in September 1985, it is widely recognized that seismic site response significantly affects the seismic behaviour of buildings, infrastructures and lifelines. The damage of the latter systems, as shown by increasing examples in recent strong-motion earthquakes, can also aggravate the social loss induced by an earthquake by paralysing emergency activities and normal life for several weeks, not mentioning the costs of repair. However, both seismic response analyses and seismic design and protection of lifelines are currently not enough widespread in the technical practice, and often their importance is underestimated in the seismic codes.
The macrotheme MT1 was therefore addressed to a two-fold direction:

1. to improve the existing knowledge in Italy on the methods of seismic response analyses, providing guidelines addressed to engineers for the optimal use of existing methods and introducing innovative computational approaches;
2. to reciprocally calibrate simplified to advanced methods for engineering predictions of dynamic soil-lifeline interaction.

For both topics, the final goal is an update of the National Technical Code to make it more friendly to the end user, i.e. the designer.

2.2 Research structure

2.2.1 Task MT1.1: Updated criteria for simplified estimates of site amplification

The main goal of the Task was to trespass the current limitations of NTC 2008 in the subsoil classification system, i.e. the most simplified way to account for seismic site response for the prediction of surface ground motion. Criteria alternative to those currently adopted by NTC (2008), based on the equivalent shear wave velocity in the first 30m, $V_{S,30}$, or on simplified strength parameters (i.e. undrained shear strength or N_{SPT}), are under study and calibration worldwide. Also, specific code prescriptions for near-fault areas should be soon or later introduced into the codes (see Task MT1.3).
Innovative criteria for amplification coefficients and spectral shapes have been developed in the Project, by calibrating them on the basis of both experimental (seismic records on well-characterized sites, § 2.3.1.1) and numerical (site response analyses, § 2.3.1.2, 2.3.1.3) data. The activity were planned and performed in close co-operation with seismology research groups.

2.2.2 Task MT1.2: Developments of numerical methods for site response analysis

This task was first of all addressed to define the limitations, as well as to optimize the performance, of current numerical instruments, including commercial codes, used for numerical predictions of free-field site response of continuous layered subsoils.
Several uncertainties still exist in the scientific community on the optimized definition and selection of the different ingredients of a reliable dynamic site response analysis, as for instance:

1. the direction, location and most appropriate kinematic variables of the reference input motion;
2. the appropriate geometrical boundary conditions and discretization for advanced seismic response analyses;
3. the calibration of non-linear stiffness and damping parameters in different constitutive models;
4. the most significant parameters representative of surface and in-depth amplification.

On the other hand, many case studies and design issues increased the demand of updated numerical procedures to account for different sources of uncertainties, i.e.:

a) the selection of the reference input motion from recorded data;
b) the subsoil heterogeneity and random variability of properties;
c) the presence of a pre-existing or an induced shear failure in the soil itself, or at the interface between it and an overlying structure, or even inside this latter.

Therefore, innovative numerical procedures were progressively developed and calibrated by several research units. The relevance of such uncertainties was explored by means of analytical (§ 2.3.2.1) or numerical parametric studies (§ 2.3.2.2), development of original computational procedures (§ 2.3.2.3), and finally cross-comparisons between different codes addressed to their reciprocal validation (§ 2.3.2.4).

2.2.3 Task MT1.3: Effects of subsoil discontinuities on seismic site response

In this task it was expected to set up innovative engineering methods of analyses and pre-norm indications on how to account for the presence of anomalies due to subsoil discontinuities in conventional continuum predictions of seismic ground response. After

L'Aquila earthquake in 2009, two of such discontinuities appeared of outstanding interest, i.e. the proximity of a seismic source to the site and the existence of underground anthropic cavities in the subsoil.
Numerical and experimental activities were also addressed to investigate on the effectiveness of artificial discontinuities made up with soft mixtures on the mitigation of seismic actions on surface structures (§ 2.3.3.1). Parametric studies in plane strain conditions (§ 2.3.3.2) and 3D non-linear analyses (§ 2.3.3.3) were referred to well-documented case histories of subsoils including anthropic cavities.

2.2.4 Task MT1.4: Soil-structure-fluid interaction analysis of lifelines

It was aimed at developing and calibrating methods for the prediction of soil-structure-fluid interaction of buried tanks and of permanent deformation and damage of pipelines. First, due to the lack of well-documented case histories on this latter topic, empirical approaches (fragility curves) were compared with damage observations. Second, results of numerical predictions with advanced dynamic analyses were compared with those of previous model tests in the seismic centrifuge (§ 2.3.4.1).

2.3 Main results

2.3.1 Task MT1.1: Simplified estimates of site amplification

2.3.1.1 Proposals for updated site spectral shapes based on extended ITACA database

The activity was focused on proposals for the simplified classification of sites based on an empirical analysis of aggregation. With such analyses, it was shown that:
1) there are characteristics amplification functions and spectral shapes that cluster naturally into distinct classes. The amplification functions were calculated for each station as the mean ratios between observed spectra and spectra predicted on rock by the empirical attenuation law GMPE ITA10 (Bindi et al., 2011) calibrated for the national territory. The spectral shapes were defined, for each station, averaging the acceleration spectra observed for M>5 and epicentral distances R < 200 km normalized with respect to the peak ground acceleration. The cluster analysis identified 7 classes of normalized spectra on the basis of the resonance frequencies and the amplification amplitudes (Fig. 1):

- F (flat)
- B1 (broad band with low amplification)
- B2 (broad band with moderate amplification)
- L1 (low frequency band with moderate amplification)
- L2 (low frequency band with large amplification)
- H1 (high frequency band with low amplification)
- H2 (high frequency band with moderate amplification)

2) on the basis of available data, two proxies in terms of $V_{S,30}$ and the fundamental frequency of the site, to be associated to each class of amplification, have been identified.
Although the strategy of aggregating a-priori the amplification functions and then identifying the proxies looks promising for the purposes of simplified seismic classification, it is currently not possible to validate this procedure by the statistical point of view, due to the lack of geological and geotechnical information at most recording stations of the RAN. For each station, the normalized spectral shapes show a very low variability. However, the stations falling in each single class of aggregation do not appear homogeneous from the geological and geotechnical viewpoints.

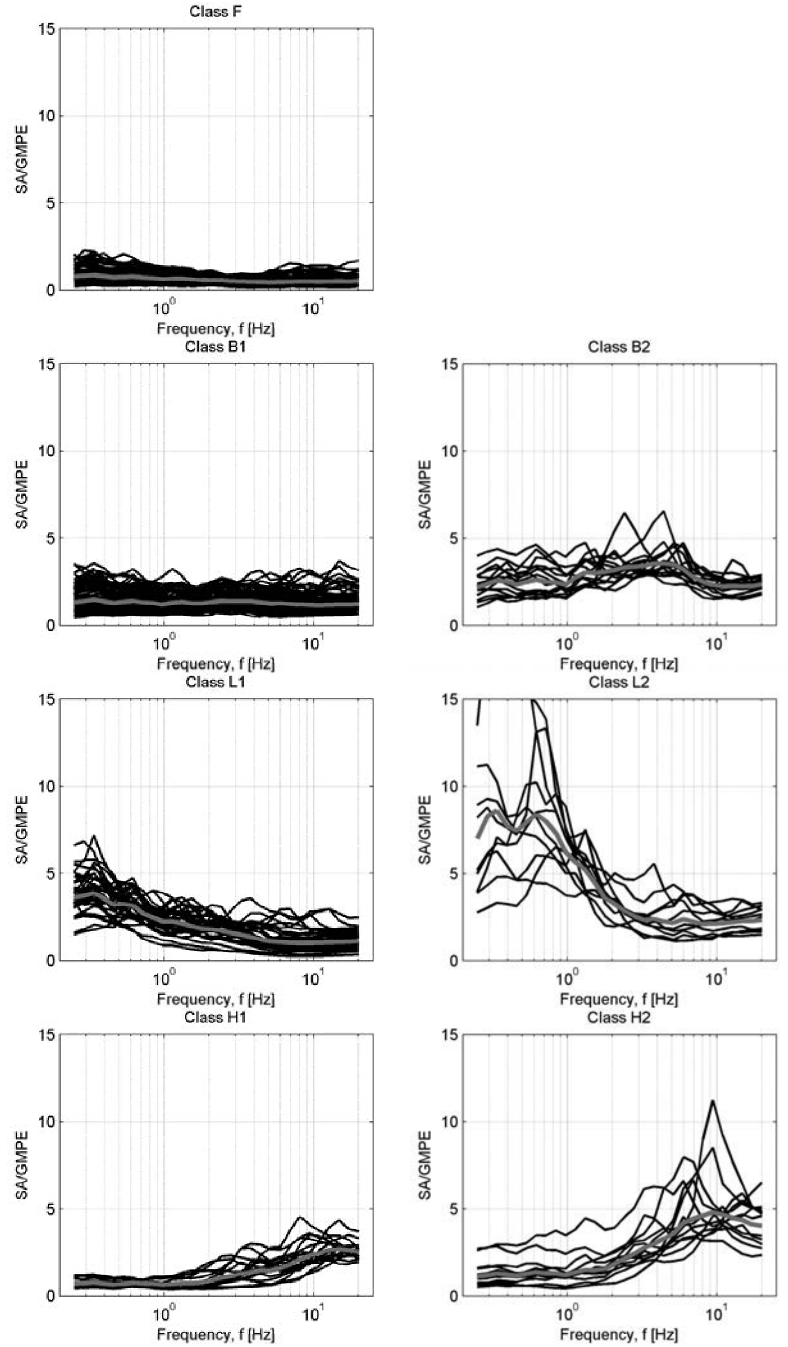


Figure 1. Mean empirical amplification function (red curve) and single station mean amplification function (black lines) relative to the 7 soil classes identified by the cluster analysis of normalised spectra.

2.3.1.2 Definition of new soil factors S_S and C_C for the Italian Building Code NTC08 from stochastic ground response analyses.

In order to define new soil factors S_S and C_C through analytical approach, a fully stochastic procedure was developed generating a database of 80.000 soil profiles (20.000 for each class B, C, D and E), varying the geotechnical parameters on the basis of Monte Carlo simulations. The procedure originates V_S profiles increasing with depth using two different laws of variation, hyperbolic and parabolic. The materials degradation curves (G/G_0 and D vs. γ) were assigned from the parameters of each layer in each soil model using the empirical approach of Darendeli (2001). Four sites, representative of the Italian seismicity and thus each one characterized by different hazard parameters (a_g, F_o and T_C^*), have been selected with the aim to evaluate the non-linear behavior within the different soil classes and therefore in order to define a new relationship between seismic hazard and soil factors S_S and C_C. The procedure has been developed to use, as seismic input, 7 real time histories for each site (7x4 in this study), recorded on stiff ground and spectrum-compatible, on average, with the reference spectrum for soil class A. Based on the results of the 1D, linear-equivalent stochastic ground response analysis, conducted for the 4 soil classes (B, C, D and E) in 4 different hazards conditions, the new parameters of soil factors S_S and C_C have been determined mainly adapting, through non-linear regression, the four branches code equation of the horizontal elastic response spectrum to the 84th percentile free surface acceleration response spectrum (mean + 1 standard deviation).

Comparing the soil factors (S_S and C_C) proposed by this study with those specified by the Italian Building Code (Table 1) it is noted, for the latter, a marked underestimate of the amplification coefficient S_S for the soil classes B and E, for any seismic hazard condition. For class C, instead, the code tends to underestimate the value of S_S in the two conditions of low and high seismicity, while between them the values seem rather aligned. Contrarily to other soil classes, the values of S_S for the class D proposed by NTC08 seem to a certain extent overestimated. In Fig. 2 the elastic spectrum obtained from the new relations of S_S and C_C for the soil class B is compared with the 84th percentile spectrum of the stochastic analysis, those specified by NTC08 and EC8 (type 1, i.e. medium-high seismicity M_s> 5.5) and finally that proposed by Pitilakis et al. (2013), for an average value of S for soil classes B2 and C1 compatible with class B in the NTC08.

Table 1. Site coefficients proposed in this study vs. those specified by NTC08.

Subsoil class	***This study***		***NTC08***	
	S_S	C_C	S_S	C_C
A	1	1	1	1
B	$1.37 \leq 1.75\text{-}0.39 \cdot F_o \cdot a_g/g \leq 1.60$	$1.07 \cdot (T_C^*)^{-0.18}$	$1.00 \leq 1.40\text{-}0.40 \cdot F_o \cdot a_g/g \leq 1.20$	$1.10 \cdot (T_C^*)^{-0.20}$
C	$1.24 \leq 1.81\text{-}0.70 \cdot F_o \cdot a_g/g \leq 1.65$	$1.04 \cdot (T_C^*)^{-0.33}$	$1.00 \leq 1.70\text{-}0.60 \cdot F_o \cdot a_g/g \leq 1.50$	$1.05 \cdot (T_C^*)^{-0.33}$
D	$1.17 \leq 2.04\text{-}1.33 \cdot F_o \cdot a_g/g \leq 1.80$	$1.00 \cdot (T_C^*)^{-0.31}$	$0.90 \leq 2.40\text{-}1.50 \cdot F_o \cdot a_g/g \leq 1.80$	$1.25 \cdot (T_C^*)^{-0.50}$
E	$1.38 \leq 2.30\text{-}1.00 \cdot F_o \cdot a_g/g \leq 2.10$	$1.02 \cdot (T_C^*)^{-0.31}$	$1.00 \leq 2.00\text{-}1.10 \cdot F_o \cdot a_g/g \leq 1.60$	$1.15 \cdot (T_C^*)^{-0.40}$

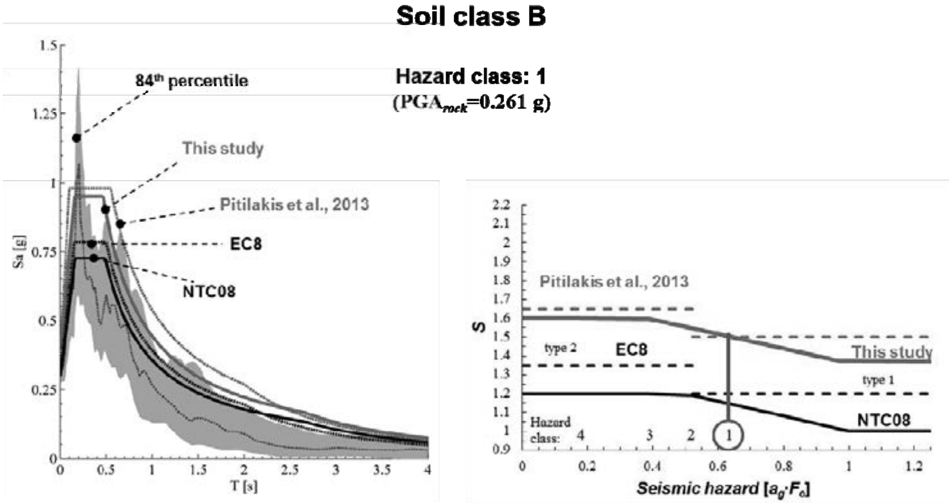


Figure 2. Comparison between elastic response spectra for the class of soil B in the same seismic hazard conditions.

2.3.1.3 Analysis of site amplification with stochastic models

A comprehensive stochastic site response study using numerical analyses has been performed: a great number of V_S profiles have been generated according to rules defined for the general database created during the second year of activity (i.e. a random selection of the main parameters according to some previously defined probability distributions). In detail 16 different seismic events have been selected among outcrop records reported in the acceleration time histories database ASCONA (Corigliano et al., 2012), which collects strong motion records from international databases. 320.000 deterministic predictions of site response for each seismic input have been carried out in order to have statistically meaningful results over a wide range of the characteristic parameters of V_S profiles.

The aim of this work has been the definition, through a multi-variable cluster analysis, of a convenient soil classification, which takes properly into account those soil parameters which mainly influence the amplification of the ground motion. A new subsoil classification scheme is then proposed, based on two parameters: the equivalent shear wave velocity computed down to the seismic bedrock ($V_{S,H}$) and the depth of the seismic bedrock itself (H_{bed}). Limit values of these two parameters have been defined in order to define classes that show amplification factors with a very small variance. In particular, the boundaries of each class have been fixed with reference to two values of acceleration input motion (0.10g and 0.21g). A pair of different factors has been adopted to express the amplification related to each class: S_S factor, that is the ratio between the peak surface acceleration and the maximum input acceleration (a_{max}/a_g) and S_A factor, ratio between the spectrum intensity of the surface acceleration time history and the input spectrum intensity, defined according to the formulation reported by Rey et al. (2002):

$$S_A = \frac{I_{soil}}{I_{rock}} \tag{1}$$

where

$$I_{soil,rock} = \int_{0.05}^{2.5} S_A(T)dT \tag{2}$$

The proposed subsoil classification is reported in Table 2: twelve different classes have been defined in function of $V_{S,H}$ and H_{bed}.

Table 2. Subsoil classes identified.

	H_{bed} ≤ 15 m	**15 m < H_{bed} < 30 m**	**30 m < H_{bed} < 45 m**	**H_{bed} > 45 m**
$V_{s,H}$ ≤ 180 m/s	Class 1a	Class 1b	Class 1c	Class 1d
180m/s ≤ $V_{s,H}$ ≤ 400 m/s	Class 2a	Class 2b	Class 2c	Class 2d
$V_{s,H}$ > 400 m/s	Class 3a	Class 3b	Class 3c	Class 3d

Fig. 3 shows the comparison, in terms of the variance of S_A factor, between the proposed classification and those reported in current codes (EC8 and NTC08) and in literature (i.e. Pitilakis et al., 2004).

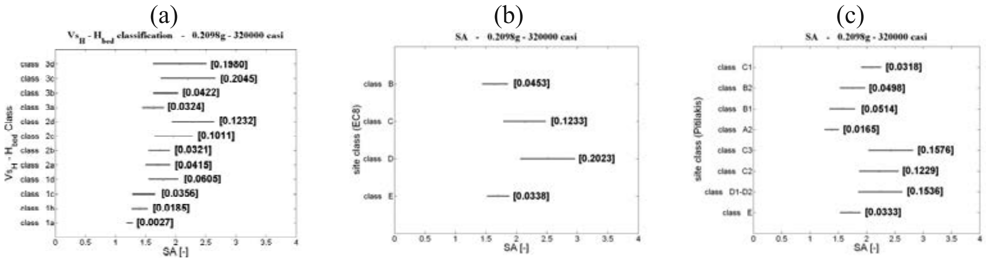


Figure 3. Comparison, in terms of the variance of the S_A factor, of proposed soil classification function of $V_{S,H}$ and H_{bed} (a) with those specified by current codes (b) and reported by Pitilakis et al. (2004) (c).

The database of numerical simulations has been used also to investigate the influence of inversion in the velocity profile (i.e. profiles in which deeper soil layers present V_S values lower than upper layers). The inversion in the velocity profile is excluded in subsoil classifications reported in current building codes, but the obtained results show that it doesn't appear crucial for the seismic site response. It is indeed associated to a slightly reduction in the mean values of amplification for all the subsoil classes.

The results obtained show a low influence of the seismic bedrock depth on the amplification factors. Therefore the changes in soil classifications recently proposed for the renewed version of NTC (not still available) seem to be rational: the extension of soil class E up to 30 m of depth (with respect to the 20 m currently defined) and the inclusion in soil class B also of soil with thickness lower than 30 m do not produce significant variations, in terms of mean and standard deviation, of the amplification factors.

Plasticity index seems to have greater influence on the amplification. Considering a soil with a zero PI or with PI equal to 100% over the entire thickness, important variations of the amplification factors have been obtained, with the same order of variation related to transition between different classes of soil. Therefore it is suggested to take into account of the effect of plasticity index in seismic site response introducing a suitable correction factor according to the following formulation:

$$S_S = f\left(V_{S,30}\right) \cdot f\left(PI\right) \tag{3}$$

2.3.2 Task MT1.2: Developments of methods for site response analysis

2.3.2.1 Analytical procedures for the determination of the reference input motion and the seismic response

In the site response analyses (SRA), the seismic motion recorded at an outcropping rock is first deconvolved at the depth of the bedrock of the soil deposit for which the SRA is required. Then, the resulting motion is propagated vertically through this soil deposit as S- and P-waves. Owing to this assumption, the horizontal component of motion is ascribed to S-waves, whereas the vertical component caused by P-waves. However, the propagation direction of the incoming waves generally deviates from the vertical, especially for earthquakes with relatively shallow source. As a consequence, the ground motion is generated by both S- and P-waves (in addition to R-waves) which are coupled with each other when they interact with the ground surface.

The main activity concerned the development of a procedure to define the characteristics of the input motion and to perform SRA using directly the recordings at an outcropping rock. This procedure consists of the following steps:

a. determination of the polarization directions of the shear waves to define the radial and transverse components of the motion;
b. evaluation of the incidence angle of the P- and SV-waves using some analytical expressions for harmonic motion;
c. deconvolution of the radial and vertical components of the motion from the surface to the depth of the bedrock of the soil deposit under consideration;
d. evaluation of the site response of the soil deposit using the output calculated at the previous step.

To solve the last two issues, a computer code is set up. Some results concerning a homogeneous layer on a deformable bedrock are shown in Fig. 4, as an example. Three components of the seismic motion recorded at the station AQG in L'Aquila during the 2009 earthquake, are imposed at the surface of this layer. After performing steps a), b) and c), the deconvolved motion is applied at the base of a second layer with the same geometrical and geotechnical characteristics of the former layer. Fig. 4 shows a comparison between the spectral amplitudes of the vertical recording and those calculated using the present code. As a confirmation of the reliability of this code, the spectral amplitudes calculated at the surface of the latter layer essentially coincide with those imposed at the surface of the former layer.

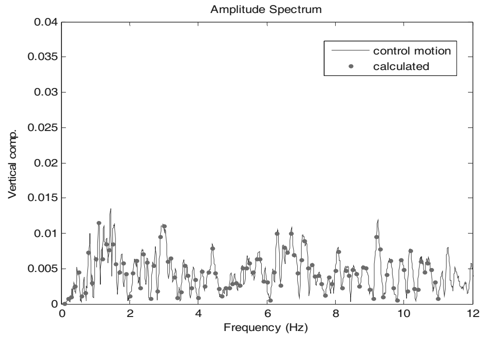


Figure 4. Spectral amplitudes of the vertical motion recorded at the AQG station.

2.3.2.2 Analysis of 2D effects due to deep morphology

The main goals of the research activity was to evaluate the influence of the subsurface irregularities on ground shaking and to develop simplified criteria for quantifying the effects related to the basin geometry.

During the third year of the project, a number of actual cases were analyzed with the aim to investigate the influence of some factors (buried morphology; stratigraphic and geotechnical characteristics of the deposit; impedance ratio; seismic input) on the main parameters representative of the site amplification effects. 2D numerical analyses of seismic response were carried out on several cross sections representative for an area of about 24 km^2, located on one of the most seismic areas of Tuscany, including the small town of Barberino di Mugello (FI) and its surroundings. The study was organized into four fundamental steps:

1. collection and processing of the available data from in situ (seismic refraction, boreholes, Down Hole tests) and laboratory tests (both in static and dynamic/cyclic conditions) aimed at the definition of the subsoil geological model (with identification of the main structures) and at the geotechnical characterization of the identified lithologic units;
2. identification and characterization of several significant cross sections to analyze the effects related to the basin geometry;
3. definition of a representative set of reference accelerometric signals to be used as input in the numerical analyses;
4. implementation of 2D local seismic response numerical analyses of each section and assessment of the amplification factors at low and high periods.

The analyses were performed by means of QUAD4M computer program. The results obtained on the different sections were then processed to assess the following amplification factors:

$$F_{PGA} = \frac{PGA_{out}}{PGA_{inp}} \tag{4}$$

$$FH_{(T_1-T_2)} = \frac{\int_{T_1}^{T_2} PSA_{out}\, dT}{\int_{T_1}^{T_2} PSA_{inp}\, dT} \tag{5}$$

where:

- PGA_{inp} and PGA_{out} are the peak acceleration of input and output motion;
- PSA_{inp} and PSA_{out} are the spectral accelerations (5% of critical damping) of input and output motion;
- T_1 and T_2 are the minimum and maximum values of period in the ranges assumed as representative of low (0.1÷0.5s) and high (0.5÷1.5s) periods respectively.

Thirteen sections, characterized by different stratigraphy, bedrock geometry, shear wave velocity profiles and materials with different experimental curves $G(\gamma)/G_0$ and $D(\gamma)$, were analyzed. By way of example, the results for a section are shown in Fig. 5.

A preliminary comparison has shown that the profiles of the amplification factors on some sections obtained from numerical analyses are in good agreement with the results predicted on similar simplified models considered during the research activity of the 2nd year of the project.

Unit	γ_n [kN/m^3]	V_S [m/s]	V_P [m/s]
SIV: Recent fluvial deposits (pebbles, sand and clay)	20.5	374	1200
MGO1a1: Fluvio-lacustrine deposits (clay and silt) (0-30m)	19.0	237	1600
MGO1a1: Fluvio-lacustrine deposits (clay and silt) (>30m)	19.0	333	1600
MGO1a1_v: Basal fluvio-lacustrine deposits (clay and silt)	20.0	587	1900
AQR: Bedrock (sandstone, marl)	23.0	1090	2600

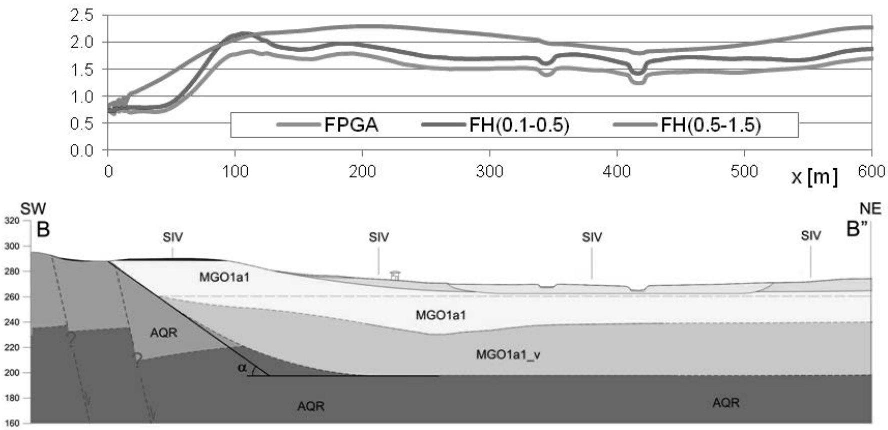


Figure 5. Geometric model and results in terms of amplification factors.

2.3.2.3 An innovative computer code for 1D seismic response analysis including failure shear mechanisms

This activity was aimed at introducing controls, procedures and utilities that enable the use of the computer code ACST originally developed by Ausilio et al. (2008).

The code performs the analysis of the 1D seismic soil response and the computation of permanent displacements with a coupled approach based on a *stick-slip* model, in the formulation proposed by Rathje & Bray (2000). ACST uses a lumped masses system, connected by viscous dampers and springs with hysteretic non-linear elastic behaviour (Fig.6).

The development of the sliding surfaces is modelled through plastic dampers that are activated when the limit shear strength defined by the Mohr-Coulomb criterion is reached. The formulation of the lumped mass system allows the possibility of introducing a simplified framed structure with shallow foundations.

The solution of the motion equations is carried out in incremental terms using the Newmark-β method.

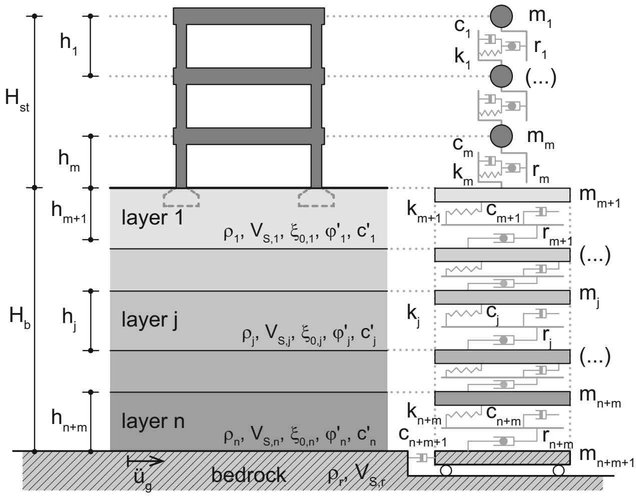


Figure 6. The ACST lumped mass system, including a simplified framed structure model.

In the ACST code, the viscous damping matrix of the dynamic system was defined according with the well-known Rayleigh damping formulation (Hashash & Park, 2002). The non-linear hysteretic response of the soil was modelled with the modified Kondner-Zelasko hyperbolic model (MKZ) together with the application of Masing rules. The stress-strain relationship in unloading-reloading conditions was defined according to a recent formulation proposed by Phillips & Hashash (2009), that modifies the Masing rules and introduces a strain-dependent reduction factor, which provides a better agreement between the analytical damping-strain function and the measured data. This factor can be assumed as a cyclic degradation parameter, since also brings a sensible decrease of the initial shear stiffness, G_0 (Tropeano et al., 2011).

The parameters of the model MKZ and of that proposed by Phillips & Hashash (2009), are archived in a database that includes many literature relationships, which are called during the input parameters of the dynamic system through an appropriate dialog box. The introduction of a new model requires, therefore, the calculation of these parameters, through nonlinear multi-regression of experimental data, and their recording in the database. These procedures had to be carried out separately from the code.

The main innovations have concerned:

1) introduction of controls during the input of parameters of the dynamic system and the management of general parameters of the numerical code;
2) introduction of a window for the computation of parameters of the models MKZ and Phillips & Hashash (2009). The interpolation of the objective functions is carried out through the Powell algorithm, calculating the relative minimum value of multivariate functions, modified with an optimisation procedure for searching of the bracketing interval (method of power progression attempts);
3) introduction of an output windows that displays the computed results in terms of time histories of acceleration, velocity, displacement, stress and strain for all layers in which the soil is discretized; maximum acceleration, strain and shear stress profiles; cumulative displacements and acceleration of the unstable mass.

2.3.2.4 Assessment of advanced models for seismic response analyses

The research activity was aimed at studying the seismic ground response by non-linear finite element analyses, executed adopting the hysteretic elasto-plastic constitutive model SSHSM (Small Strain Hardening Soil Model), implemented into the commercial FE code (PLAXIS® 3D).

FE analyses were performed with reference to a loose sand, characterized by the following mechanical properties: γ = 18 kN/m^3, e_0 = 0.83, c' = 0, φ' = 30°, K_0 = 0.5, G_0 variable with depth, according to the relationship proposed by Hardin & Black (1978) (S = 300, m = 0, n = 0.5); the curves of modulus reduction and variation of damping with shear strain are representative of a material with IP=0%, according to Vucetic & Dobry (1991). The SSHSM model has been validated by means of a set of simulations of cyclic shear tests at controlled displacements, performed on a weightless volume element, subjected to two different initial load conditions: NDP = isotropic loading-unloading; WDP = isotropic loading-unloading followed by a deviatoric loading-unloading. Fig. 7 depicts the variation of shear stiffness modulus and damping ratio with deformation level, estimated for both NDP and WDP pre-shear compression stages: the stabilized stiffness and damping appear not to be significantly influenced by the above initial stages.

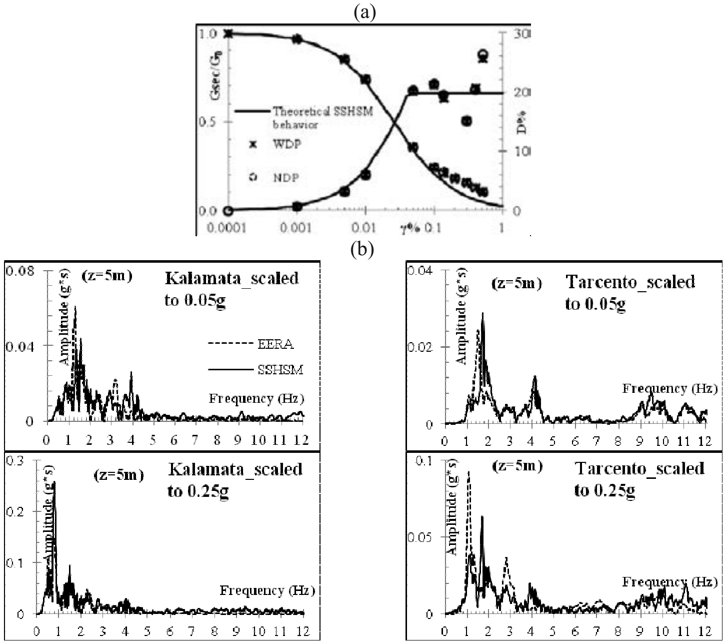


Figure 7. (a) Variation of shear stiffness and damping ratio with shear strain predicted by the analytical formulation of the model and numerical FE simulation on a soil element; (b) acceleration Fourier spectra predicted at 5m depth, for Kalamata and Tarcento records scaled to 0.05*g* and 0.25*g*.

The seismic ground response analyses were carried out with reference to a dry deposit of the same sand, 40 m thick. The extension of geometric model aimed at reproducing a plane strain condition. In the dynamic stage, a prescribed displacement distribution was applied at the bottom of the mesh and absorbing boundaries characterized the lateral sides of the geometric model, while no displacements were allowed in the out of plane direction. Wave propagation was studied with reference to four acceleration time histories; for the sake of brevity, only the results referred to Kalamata and Tarcento records are discussed here. Kalamata's duration is 29 s and Tarcento's acceleration time history is 15 s long; both signals are scaled to 0.05*g* and 0.25*g*.

As a term of reference, equivalent linear analysis results (EERA) were considered in terms of Fourier spectra, at a reference depth of 5 m from surface ground level. A satisfying agreement is noticeable between the spectra predicted by FE and those by EERA.

It can be concluded that the SSHSM model represents a good compromise between complexity of the problem and effectiveness of its solution, leading to results that prove to be comparable to those obtained by well-established methods.

2.3.3 *Task MT1.3: Effects of subsoil discontinuities on seismic site response*

2.3.3.1 *Seismic response analysis with mitigation countermeasures consisting of artificial discontinuities*

The Italian building heritage is partially made up of buildings designed without adequate anti-seismic criteria; the different techniques traditionally conceived to mitigate their seismic risk are generally based either on the decrease of the structural vulnerability by acting on stiffness and strength of the elements above surface, or on base isolation systems. An alternative approach would be to reduce the site-specific seismic hazard, acting on the foundation soil by locally changing the layering with injections of suitable materials, in order to produce a kind of artificial discontinuities. Such materials, whether characterized by a low dynamic impedance η (= $\rho \cdot V_S$, where ρ is the density and V_S the shear wave velocity), can reduce the inertia forces although amplifying the surface displacements; by one-dimensional and two-dimensional seismic response analyses, it has been observed that this treatment is able, in the generality of cases analyzed so far, to break down the inertia force transmitted to the area protected by the intervention.

The research activity was addressed to two directions: on one hand, laboratory characterization by cyclic and dynamic tests of the stiffness and damping properties of a polyurethane foam and of a Super-Absorbing Polymer (SAP); on the other hand 1D and 2D seismic response analyses in order to define optimized installation schemes compatible with the mechanical and physical properties of the materials considered. In Fig. 8, the results of different one-dimensional (circles) and two-dimensional (square) analyses of the propagation of Ricker wavelets and accelerograms with different dominant frequencies are plotted in terms of ratio between the maximum acceleration observed with and without the intervention, with reference to the impedance ratio α between the natural soil and the replacing material. With few exceptions for the cases of resonance between input motion and soil deposit, it appears that α values larger than 20-30 might be capable to significantly break down the inertia forces transmitted to ground level. For full validation of the selected strategy, further laboratory tests and numerical analyses are in progress, in order to predict the static behaviour, in terms of displacements and distortion, of such interventions on the structures to be protected.

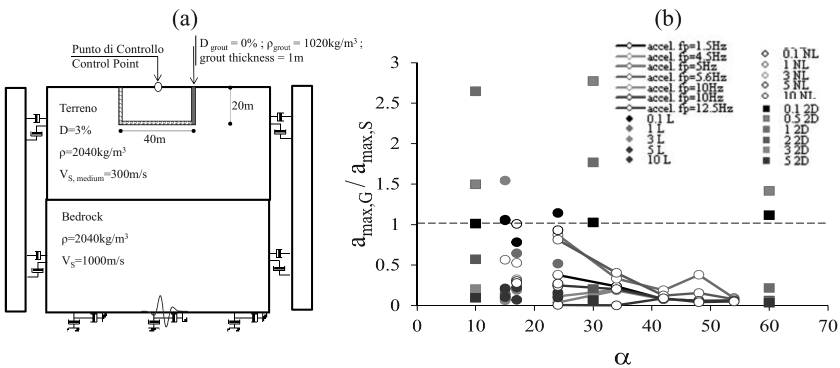


Figure 8. (a) Geometrical, mechanical and physical properties values for a possible soft grout injection with layers arranged to form a "isolating box", (b) results in terms of acceleration ratio with and without treatment plotted against impedance ratio α, by varying the input frequency, for one-dimensional (circles) and two-dimensional geometries (squares).

2.3.3.2 2D seismic response analysis in presence of underground cavities

The effects on the surface seismic response produced by the presence of cavities have been investigated numerically by means of a finite difference code (FLAC, Itasca, 2011).
The numerical routine, developed to take into account the inclination of the wave fronts in the incidence to the boundaries of the model, has been further developed to better simulate the free-field conditions near the surface of the half-space. If the dynamic response near the surface is of interest, the motion at the boundaries of the model is affected not only by the primary wave front, but also by those generated by the reflection of the primary front on the surface. This refinement allows obtaining consistent results, confirmed by the simulation of the motion produced on the surface by the oblique incidence of the wave fronts in an elastic half-space (for which a theoretical solution is available).
Further parametric analyses have been conducted for the study of the dynamic response at the surface in presence of cylindrical cavities with circular section. They led to some additional considerations with respect to those reported in the last year's report. The amplification of seismic motion at the surface, calculated as the ratio between the horizontal components of motion with cavities and without cavities, is plotted in Fig. 9. It has been found that accounting for a not negligible damping ratio D (modelled by a Rayleigh damping approach) yields to results poorly different from those obtained with a simple linear elastic constitutive model. This observation can be justified considering that the modification of the surface seismic motion is produced by the superposition of the wave field incident on the surface in free-field conditions to that generated by the reflection on the surface of the cavity. The latter travel along very short trajectories before reaching the surface, and therefore is moderately influenced by the presence of a damping.
Some analyses were also conducted to examine the geometric configuration of a set of surface cavities located side by side. This condition can be frequently found in urban contexts of ancient origin, grown on a quite soft material, but strong enough to obtain self-sustaining excavations for different purposes (mining of construction materials, warehouses, cellars, cemeteries). In presence of sets of cavities, the seismic motion at the surface is subjected to modifications concentrated in the same ranges of normalized frequency $\eta = D/\lambda$ (D= cavity diameter and λ = wavelength) obtained in the analyses with a single cavity, that is for a ratio between the harmonic wavelength and the diameter of the cavity lower than 4-5. Constructive interference of the diffracted fields can only be obtained, exclusively for the horizontal component, by approaching the cavity to distances less than about 3 times the diameter of the cavity.
Overall, the numerical analyses so far conducted allow some considerations:

- the influence of the cavities is manifested in the attenuation of most of the high frequencies, with amplifications limited to rather narrow ranges of frequencies and in very limited zones;
- the harmonics subjected to amplifications correspond to wavelengths lower than about 4 to 5 times the diameter of the cavity. By taking into account the smallest stiffness values of the materials in which it is possible to find unsupported cavities, the above mentioned values correspond to frequencies about at the upper end of the seismically relevant range.

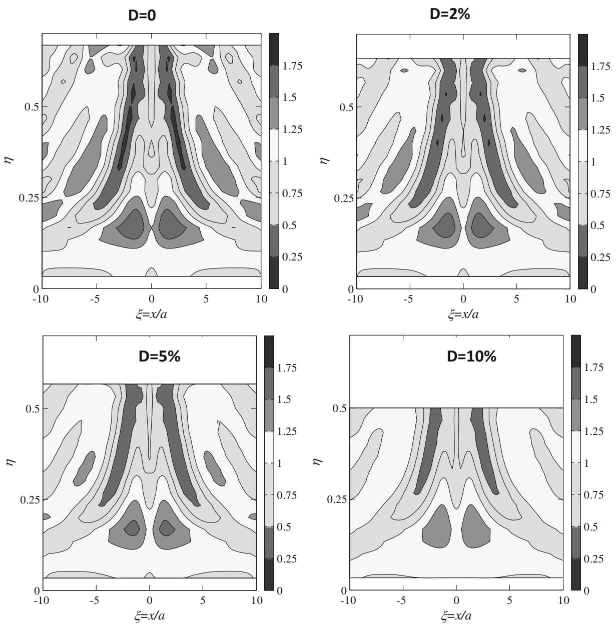


Figure 9. Comparison of the results of four numerical analyses with different values of damping ratio D, expressed in terms of amplification ratio of the horizontal component at the surface, Hx, as a function of the normalized distance from the axis, ξ, and of the normalized frequency, η.

2.3.3.3 3D seismic response analysis of a hill in presence of underground cavities

The activity was focused on the seismic response of the hill of Castelnuovo (AQ). The aftershocks records and the survey of the damages induced by the 2009 earthquake on buildings (Borghini et al., 2011) suggested the incidence on the damage distribution of both topographic amplification and the underground cavities, underlying most of the buildings along the southern side of the hill. Thus, numerical simulations have been performed on a three-dimensional model of the hill, generated from the geo-morphological studies carried out for the microzonation, and progressively improved in terms of geotechnical characterization of the peculiar white silt formation (Marcon, 2012).

Consistently with the previous 2D analyses (Lanzo et al., 2011), the results were reported in terms of surface distribution of the amplification factor of peak ground acceleration (AF_{PGA}) and Housner intensity ($AF_{HI0.1\text{-}0.5s}$ and $AF_{HI0.7\text{-}1.3s}$). Observing the trends predicted assuming a non-linear visco-elastic soil behaviour (Fig. 10), the topographic amplification effect appears significant throughout the whole frequency range expressed by the input motion, selected to represent the actual 2009 seismic event (Landolfi, 2013). All the amplification factors assume maximum values at the top of the hill; recalling the topography of the site, they decrease downstream, assuming values consistent with those calculated in the previous 2D numerical simulations (Lanzo et al., 2011) and similar to those from 1D analyses.

In the frequency range including the fundamental frequency of the hill (1Hz, according to the above mentioned instrumental records), the highest incidence of the topographic amplification is observed, with an increase at the hilltop by a factor of three with respect to the stratigraphic amplification (corresponding to 1D analyses).

The results of the analysis also show that the underground cavities do not induce any appreciable effect on the distribution of the ground motion induced at surface. This could be expected, according to the results of the 2D parametric study previously carried out (Chiaradonna et al., 2012). In fact, the small average size of the cross-section of the cavities (D = 5m), does not significantly affect the propagation of the wavelengths corresponding to the dominant frequencies of the signal.

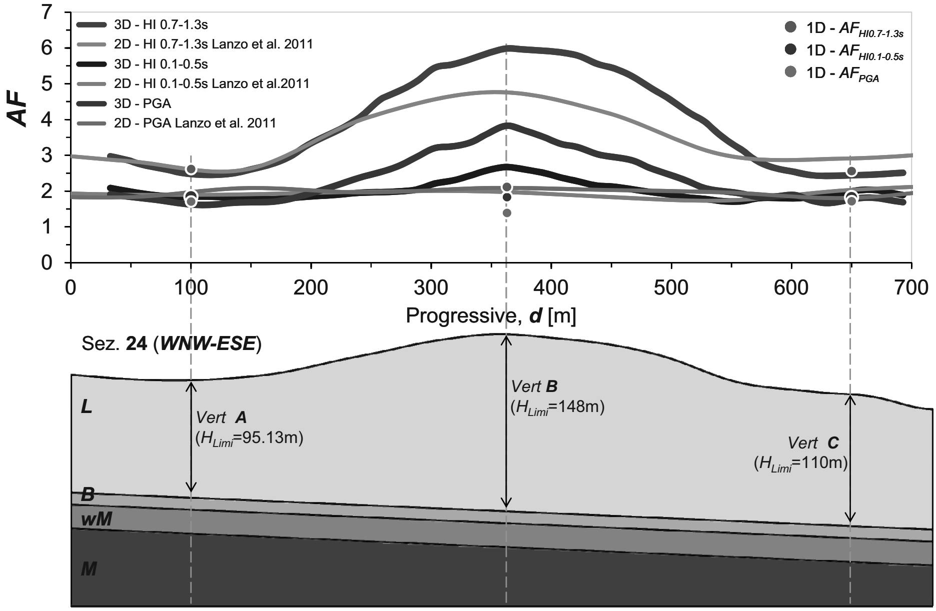


Figure 10. Comparison of the results obtained in the current study with a 3D model, those obtained by a 2D model (Lanzo et al., 2011) and those predicted by 1D analyses in selected verticals.

2.3.4 Task MT1.4: Soil-structure interaction analysis of lifelines

2.3.4.1 Numerical prediction of lifeline and tunnel performance during centrifuge dynamic tests

Centrifuge tests represent a unique source of experimental data to understand the behaviour of geotechnical structures under dynamic conditions, such as lifelines and tunnels, for which in situ measurements are rarely available. In addition, they allow to validate the existing design approaches and analysis methods, providing a quantitative insight into such a complex problem. The research activity referred to the centrifuge tests carried out at the University of Cambridge (Lanzano et al., 2012) to investigate the behaviour of lifelines and tunnels, modelled as long and uninterrupted pipes, during earthquakes. In particular, attention was devoted to the tests T3 and T4, characterised by an excavation cover of 150 mm. The two tests adopted the fraction E of the dry Leighton Buzzard sand, and differ for the value of soil relative density, which resulted to be equal to 75.9% in test T3 and 40.8% in test T4. The pipe was modelled using an aluminium-copper alloy tube. A centrifuge spin of N = 80 was fired in

the considered portion of the test before the shaking was applied at the base of the model. This consists of four separated pseudo-harmonic acceleration signals of increasing amplitude.
The numerical analyses were performed with the finite element code Plaxis 2D (2009) at the model scale. Soil behaviour was modelled using the *Hardening Soil model with Small-Strain Stiffness*, recently implemented in the material model library of the code. Model calibration was performed with reference to the laboratory experimental tests carried out on the same soil employed in the centrifuge tests (Visone & Santucci de Magistris, 2009). The elements adopted to simulate the pipe behaviour were weightless linear elastic plates, assuming an almost null values of the shear strength at the soil-lining interface (i.e. full-slip conditions). No water was considered in the model.
Fig. 11a illustrates the comparison between the acceleration history observed during the T3 centrifuge test and the corresponding one computed by the numerical model. The signal refers to accelerometer 9 located sufficiently far from the tube to be considered as representative of free-field conditions. A good agreement is observed between the experimental and numerical data, in terms of both peak values and zero crossings. The analyses revealed that the presence of the tube has some limited but not negligible effect on the travelling seismic signals.

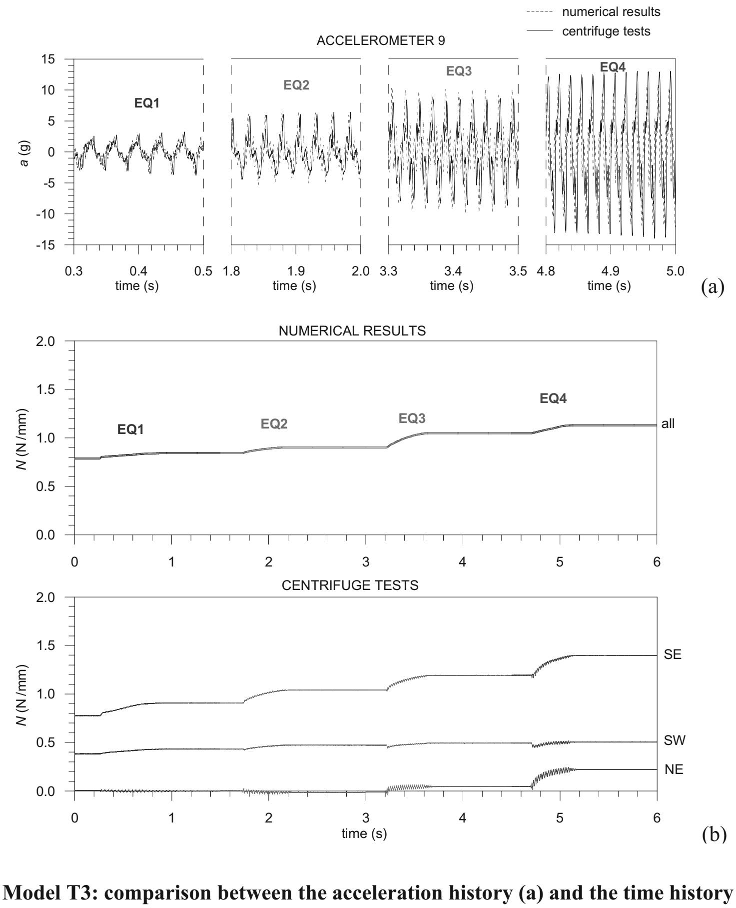


Figure 11. Model T3: comparison between the acceleration history (a) and the time history of normal forces in the lining (b) measured during the centrifuge tests and computed by the numerical model.

Concerning bending moments and axial forces acting in the pipe during the earthquakes, it is of great interest to observe that these quantities not only show a reversible oscillation but also tend to irreversibly cumulate, leading to a post-earthquake stress regime permanently acting in the lining that can differ significantly from the initial static one. These aspects were qualitatively reproduced by the numerical analyses (Fig. 11b), but the results differ from the experimental observations in quantitative terms, especially for the bending moments.

2.4 Discussion

For Task MT1.1, the gathering of different experimental and analytical contributions provided data, updated relationships and coefficients for the simplified estimates of soil amplification adopted by the codes.

The activities of Task MT1.2 clarified different aspects of seismic response analysis, such as the choice of the most suitable boundary conditions in terms of input motion and domain geometry; also, both original and existing software for non-linear 1D and 2D-3D analyses was either developed or critically validated.

In Task MT1.3, useful suggestions were given by the two parallel parametric studies on the effect of cavities on seismic site response; one of the studies demonstrated the poor influence of such discontinuities at the urban scale, with reference to a hill with complex morphology. Recent studies on the effectiveness of 'artificial discontinuities' inserted in the subsoil to protect existing buildings appear very promising.

In Task MT1.4 significant results have been obtained, comparing centrifuge models observations with numerical predictions of buried pipelines.

2.5 Visions and developments

The search for updated and improved site classification criteria is well supported by conceptual approaches developed and shared by the research units involved in the framework of Task MT1.1. However, the results collected insofar suggest further developments which should include the validation of the above mentioned criteria on well-documented test sites which are not currently available in the Italian seismic database.

Such test sites might be also useful to calibrate the predictions of original as well as well-consolidated procedures and codes for seismic response analyses, such as those developed in Task MT1.2. A reference experience might be that currently in progress in the International 'Prenolin' Project, joined among the others by several Italian researchers involved in ReLUIS.

The innovative results obtained in the pilot numerical studies on the effects of cavities and soft grouts on the deep and shallow seismic response (Task MT1.3) would deserve an experimental assessment by means of physical model testing, e.g. in seismic centrifuge devices.

Finally, more effort should be devoted in developing and assessing methods of simplified dynamic analysis in order to bridge the gap still existing between the use of empirical fragility curves and advanced numerical and physical models to predict the seismic behaviour of underground lifelines (Task MT1.4).

3 SHALLOW AND DEEP FOUNDATIONS

3.1 Background and motivation

Conventional dynamic analyses of structures are fixed-base, the foundation being considered to be constrained against swaying, vertical and rocking mode of oscillation. By contrast, published literature on coupled soil-structure system has shown that accounting for soil deformability and damping, including the radiation component, may lead to significant changes in the seismic response of structures.

When dealing with shallow foundations, the so called macro-element theory allows reproducing the response of the soil-foundation system under eccentric/inclined loads. In addition, this aspect has been complemented with identification analyses of a real structure (Ghirlandina tower in Modena).

Further relevant aspects to be clarified arise when considering the seismic behaviour of piled foundations. These include the relative role the kinematic and inertial interaction and the behaviour of inclined piles.

Furthermore, the inertial interaction analysis of a structure founded on piles is usually performed by assuming that the support motion at the foundation level is merely that of the free-field, thus neglecting the filtering action exerted by the piles, despite the evidences of frequency filtering effects. Therefore, a research program was also undertaken to clarify this aspect.

Scope of research project was to develop models which can be used in earthquake response of structures without neglecting soil-foundation structure interactions.

3.2 Research structure

The research program has been carried out according to the following points:

1. Soil-structure interaction model for shallow footings, including bearing capacity of shallow footings;
2. Pile foundations under lateral loads;
3. Pile foundations: kinematic versus inertial effects;
4. Inertial interaction taking into account seismic input filtered by piles.

3.3 Main results

3.3.1 Shallow footings: the macroelement approach, identification analyses and bearing capacity

A macroelement model (ME) capable of simulating the mechanical response of the soil/foundation system under inclined and/or eccentric loads has been developed. The fundamental goals were: 1) formulation of a macro-element model capable of reproducing the footing response under eccentric/inclined loads; 2) application of the macro-element theory to the case of inhomogeneous subgrades through finite element analysis.

The Ghirlandina tower (Fig. 12a) case study has been considered to develop a macro-element model for shallow foundations under markedly eccentric/inclined loads. The choice of the ancient bell tower in Modena as a reference case was motivated not only by its remarkable historical/artistic relevance, but also by the availability of an accurate site characterization, during recent restoration works.

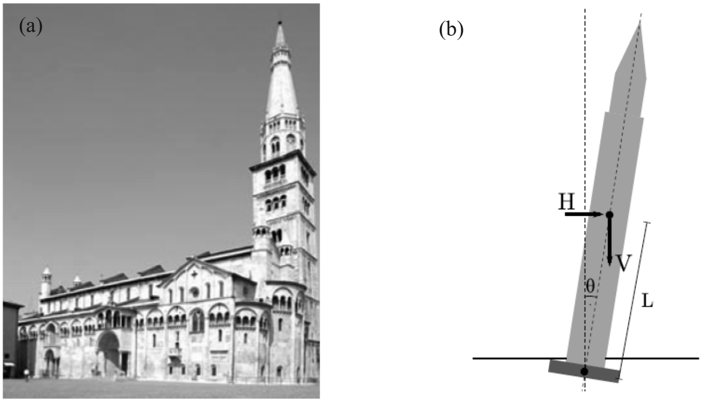


Figure 12. (a) The Ghirlandina tower; (b) Tower tilting induced by the lateral seismic load.

In Fig. 12b a tower-soil system is sketched along with the main applied loads, namely the vertical and the horizontal forces - V and H - given by the tower weight and the lateral seismic acceleration (vertical seismic actions are neglected).

The model is formulated in the framework of isotropic strain hardening elasto-plasticity, by adopting the following work-conjugate static and kinematic variables:

$$h = \frac{H}{V_c^{lim} - V_t^{lim}}; \quad m = \frac{M}{B\left(V_c^{lim} - V_t^{lim}\right)} \tag{6}$$

$$v = \left(V_c^{lim} - V_t^{lim}\right)u; \quad \omega = B\left(V_c^{lim} - V_t^{lim}\right)\theta \tag{7}$$

where B is the foundation width, $V_c^{\lim}$ and $V_t^{\lim}$ the limit loads under pure compression and traction, H and M the horizontal and moment load components, u and θ the associated horizontal and rotational displacements.

A HM ME incremental relationship can be posed:

$$\begin{Bmatrix} dv \\ d\omega \end{Bmatrix} = \begin{bmatrix} C_{vv} & C_{v\omega} \\ C_{\omega v} & C_{\omega\omega} \end{bmatrix} \begin{Bmatrix} dh \\ dm \end{Bmatrix} \tag{8}$$

The preliminary calibration of constitutive parameters has been carried out. The accuracy of the above ME model has been verified by comparison with finite element (FE) results concerning the simulation of displacement-controlled loading tests on the (rigid) foundation. In Fig. 13, the resulting ME and FE loading HM paths satisfactorily match for practically all the simulations performed. However, it is worth noting that the best quality in ME simulations is obtained in correspondence of the best match between analytical failure locus (dashed line) and FE failure points (triangular markers)

Overall, the HM ME model is capable of reproducing the footing performance arising from displacement controlled FE simulations, this confirming the soundness of the simplifying assumptions introduced. While requiring a few FE results for calibration, the HM ME model can be easily introduced into pushover procedures for seismic lateral stability analyses, allowing for even extensive parametric studies at low computational costs. Although a 2D FE model has been set up for Ghirlandina foundation, the elasto-plastic structure of the HM ME model offers the prospect of capturing 3D FE responses as well, with an even larger computational convenience.

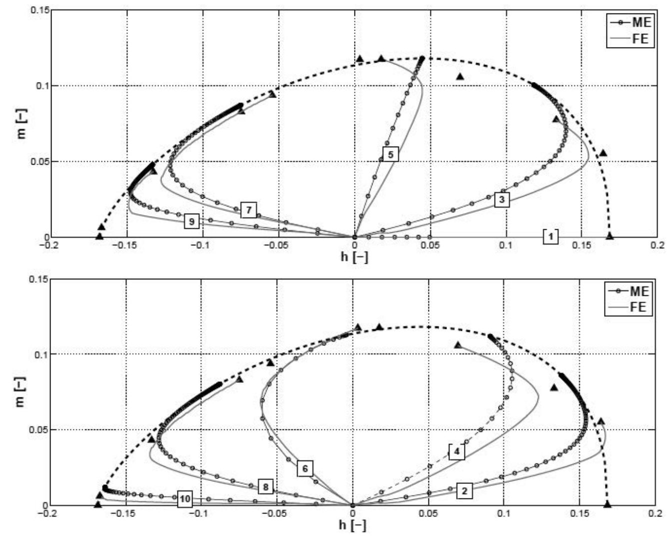


Figure 13. Comparison between the ME and FE simulation of displacement-controlled loading paths.

When considering the seismic vulnerability of a structure it is relevant at a first stage to identify all structural features of the structure as a whole, throughout an identification analysis in order to capture vibration modes, modal shapes and dissipation phenomena. To reach this goal an experimental study has been carried out, in order to identify the tower response as subjected to different sources: ambient vibrations, vibration produced by an heavy track passage, vibrations produced by bells.

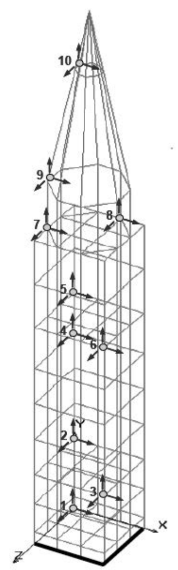
Time history of acceleration has been monitored on several points, as shown in Fig. 14, previously identified as relevant points by means of a FEM model. An dynamic signal recorder (LMS SCADAS III) with 24 channels, a laptop and 24 capacity type accelerometers were used. The identification analysis was performed in time domain by using several techniques (AutoRegressive Moving Average, Stochastic Subspace Identification, Eigensystem Realization Algorithm).

Figs. 15a,b show the bending mode shapes and the axial mode shapes, respectively. The results show that the soil structure interaction cannot be neglected. In fact, the first bending mode shape and the mode shapes related to vibrations along the tower axis (modes 8 and 9 in Fig. 15b) show that the rotation and the displacement pattern at the tower basis is due to soil deformability.

The identification analysis allowed to estimate the soil-foundation stiffness which were used in vulnerability analysis.

In order to study the role of soil-structure interaction in the long term performance and to assess the seismic vulnerability of the tower, a dynamic monitoring was started. As indicated in detail in Fig. 14, the measuring system is composed of 12 accelerometers installed at 6 locations along the vertical wall of the tower. Two earthquake events have been recorded: the earthquake of October 03, 2012 and the one of January 25, 2013.

Identification analysis



Positions and measurement directions

Identification analysis

Position	X (m)	Y(m)	Z(m)
1	0	0	0
2	0	12	0
3	10	12	10
4	0	30	0
5	0	37	0
6	10	37	10
7	0	51	7
8	7	52	0
9	0	60	7
10	4	79	4

Monitoring

Position	X (m)	Y(m)	Z(m)
1	0	0	0
2	0	10	0
3	0	35	0
4	10	35	10
5	0	50	3
6	4	77	4

Monintoring

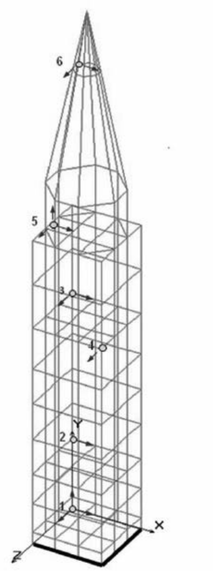


Figure 14. Identification analysis of Ghirlandina Tower.

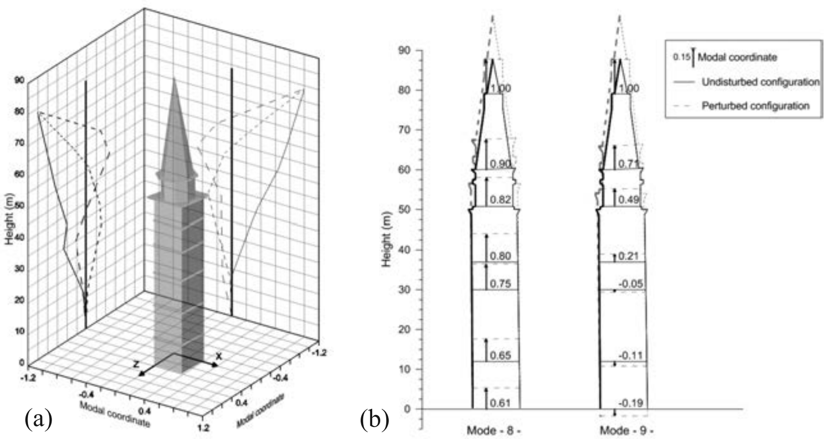


Figure 15. (a) Bending mode shapes; (b) Axial mode shapes.

Numerical and simplified solutions for the evaluation of the ultimate load of shallow foundations subjected to seismic loading have also been performed. An "*upper bound* " solution (Castelli and Motta, 2012) of the limit load has been developed to evaluate the corrective factors to be employed in the seismic conditions, to estimate the ultimate load of the soil-foundation system.

For the correct evaluation of the ultimate load it is necessary to take into account the soil mass interested by the failure surface, depending on the embedment depth D of the footing. As example, for a shallow foundation resting on a cohesionless soil, with horizontal ground surface and in absence of surcharge, the limit load q_{lim} can be expressed by:

$$q_{\lim} = \frac{1}{2} B\gamma N_{\gamma} i_{\gamma i} i_{\gamma k} d_{\gamma} \tag{9}$$

where: B is the width of the footing; γ is the unit weight of soil; N_γ is the bearing capacity factor; $i_{\gamma i}$ is the load inclination factor due to the inertia of the structure; $i_{\gamma k}$ is the reduction factor due to the inertia of the soil mass (*kinematic interaction factor*) and d_γ is hte depth factor.

The load inclination factor related to the inertia of the structure ($i_{\gamma i}$) has been discussed by some Authors, while less information are available on the depth factor (d_γ) and on the reduction factor due to the inertia of the soil mass ($i_{\gamma k}$).

Conventionally, the depth factor (d_γ) is assumed equal to unit (Brinch Hansen, 1970). Nevertheless, in an analysis in which the effects due to the inertia of the soil mass are taken into consideration, it is also necessary to take into account the inertia of the soil mass corresponding to the embedment depth D of the footing. In fact, in static conditions, by simple equilibrium considerations, it is possible to derive that the expressions of the bearing capacity factors N_c, N_q e N_γ are depending on the embedment depth D of the footing (Castelli and Motta, 2011).

The factor $i_{\gamma k}$ is defined in this study as the ratio between the bearing capacity factor N_γ^* derived for a given value of the horizontal seismic coefficient k_{h2} and the conventional bearing capacity factor N_γ (Fig. 16).

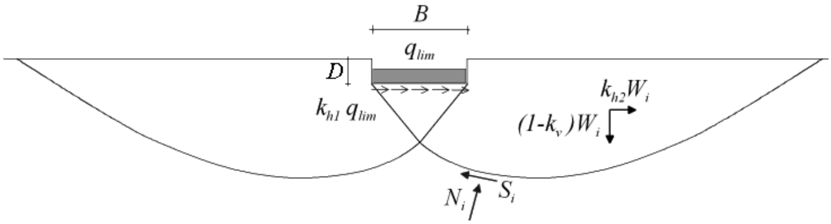


Figure 16. Problem definition and applied forces adopted in the analysis.

In Fig. 17 are reported the values of the kinematic interaction factor $i_{\gamma k}$ obtained for the friction angles of soil taken into consideration. Curves shown approximately a linear trend, thus it is possible to express the kinematic interaction factor $i_{\gamma k}$ as a linear function of k_{h2} by the equation:

$$i_{\gamma k} = k_{h2}\ cotg\ \phi' \tag{10}$$

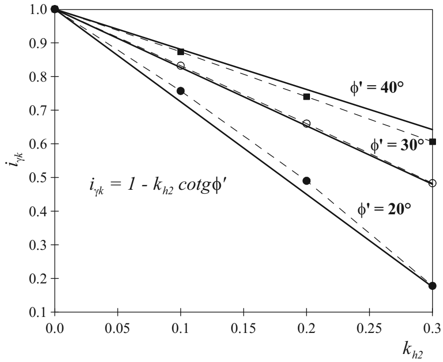


Figure 17. Values of the factor $i_{\gamma k}$.

3.3.2 Inertial interaction taking into account input filtered by piles

The inertial interaction analysis of a structure founded on piles is usually performed by taking that the support motion at the foundation level is merely that of the free-field, thus neglecting the filtering action exerted by the piles. By contrast, the existence of frequency filtering is confirmed through work referring to theoretical and experimental studies, even if this effect has not been so far taken into consideration in design practice.

The inertial interaction analysis of any structure resting on piles can be conveniently performed through the following three consecutive steps: (i) calculate the motion of the foundation in the absence of the superstructure, i.e. the so-called foundation input motion; (ii) determine the dynamic impedance functions associated to swaying, vertical, rocking and cross swaying-rocking oscillation of the foundation; (iii) evaluate the response of the superstructure supported on the springs and dashpots and subjected to the motion of the foundation determined at the first step. This method is truly convenient as an alternative to fully 3D analyses of the complete system and is commonly referred to in literature as the substructure or the kinematic-inertial decomposition method (Gazetas 1984; Makris et al. 1996; Mylonakis et al. 1997). The most common application of the substructure method is to assume that the support motion equals the free-field seismic motion. By contrast, the free-field motion is filtered out by the piles, especially in the case of soft soils, where piles are recurrently required to increase the bearing capacity of the foundation and/or to reduce settlements (de Sanctis and Russo 2008; Viggiani et al. 2011). The importance of the filtering effect has been reported in theoretical and experimental studies, even if this effect has not been so far taken into consideration in engineering practice.

The goal of this research is threefold: (i) to offer an insight into the pile-filtering mechanism; (ii) to propose a correction to design spectra; (iii) to quantify the beneficial effect coming from the filtering effect exerted by the piles on the seismic response of structures. With respect to this last point, the study has focused on the response of reinforced concrete frame buildings excited by either the filtered input motion or the free-field input motion, to quantify the beneficial effect coming from the piles on the seismic performance of the superstructure. Inertial interaction analyses have been performed for Linear-elastic (LE), non-dissipative structural models and for Non Linear (NL), dissipative structures with lumped plasticity.

Di Laora and de Sanctis (2013) demonstrated that in the case of a pile embedded in multilayer soil, provided that first layer is thick (i.e. whose thickness is larger than pile active length

(Randolph, 1981), pile-soil acceleration ratio, I_u, may be expressed through the following expression:

$$I_u = \left[1 + \frac{1}{20}\left(\frac{\omega \lambda_p}{V_s}\right)^4\right]^{-1} \tag{11}$$

where

$$\lambda_p = d\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} \tag{12}$$

is a characteristic pile wavelength which encompasses both diameter d and pile-soil stiffness ratio, ω is the circular frequency of excitation and V_s the shear wave velocity in the soil (first layer in the case of multilayer deposit).
The above expression clearly highlights the parameters governing filtering effect and offers insight into the physical interpretation of the interaction phenomenon. The new dimensionless parameter $\omega\lambda_p/V_s$ indeed represents the ratio of a pile characteristic length λ_p and the wavelength in the soil (proportional to the ratio V_s/ω by a factor of 2π). When this ratio increases, pile is progressively unable to follow soil deformation due to its larger stiffness, leading to smaller values of acceleration ratio I_u.
Eq. (11) is plotted in Fig. 18 against the new frequency parameter $\omega\lambda_p/V_s$. For comparison, literature results from Fan et al. (1991) and Di Laora and de Sanctis (2013) are reported in the graph.

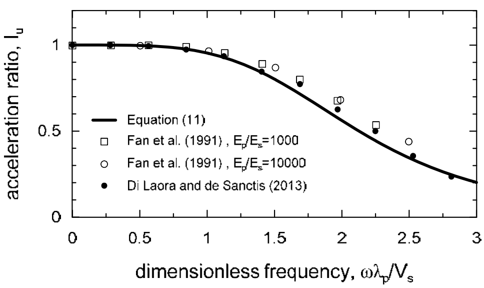


Figure 18. Pile-soil acceleration ratio as function of dimensionless frequency parameter $\omega\lambda_p/V_s$.

The accumulated experimental, analytical and numerical evidence about the filtering action exerted by piles has been recently proposed to be employed in routine design by Di Laora and de Sanctis (2013).

The authors performed transient dynamic Finite Element analyses of kinematically-stressed piles in a two-layer soil (Fig. 19), thus analyzing the ratio of the acceleration spectra of pile over that of soil motion. Some results of this study are reported in Fig. 20, where pile-soil spectral ratio ξ is plotted against the structural period T, by varying subsoil conditions and bedrock signal. It can be seen that all the functions have a square-root shape for which it is possible to recognize three critical points: (i) the spectral acceleration ratio at $T = 0$, ξ_0, which is a purely kinematic interaction factor, being the ratio of the maximum pile and soil acceleration, respectively; (ii) the point corresponding to the minimum value of the spectral acceleration (T_{min}, ξ_{min}); (iii) the structural period after which the filtering effect becomes negligible (T_{crit}, ξ_{crit}). For any subsoil the structural periods T_{crit} and T_{min} are practically unaffected by the earthquake event; such outcome may be also obtained by the dimensional analysis approach, as detailed in the original work.

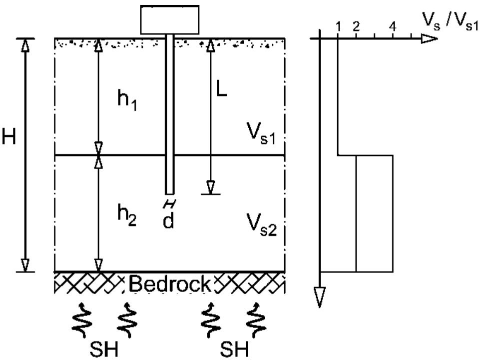


Figure 19. Problem considered in the parametric analysis by Di Laora and de Sanctis (2013).

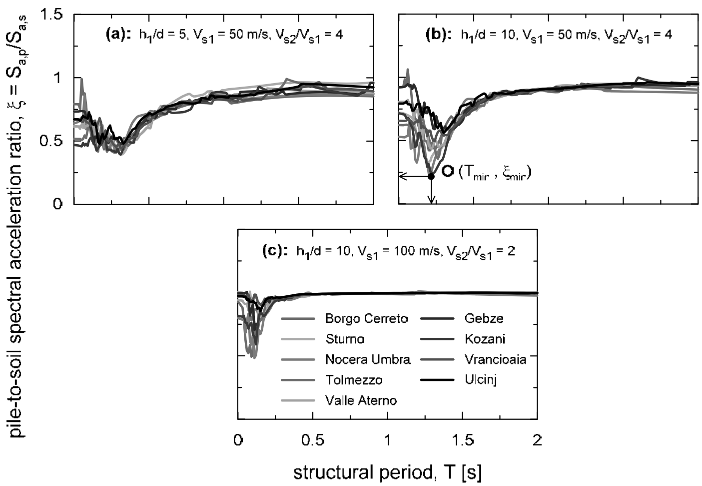


Figure 20. Pile-head spectral acceleration over that of the free-field.

The results of the parametric analysis have been conveniently condensed in the form of mean spectral acceleration ratios by averaging among the 9 different earthquakes employed, obtaining the frequency-independent spectral ratios. Upper bounds of such curves, that can be conservatively applied in design, are represented by the homogeneous soil case, as it is shown in Fig. 21.

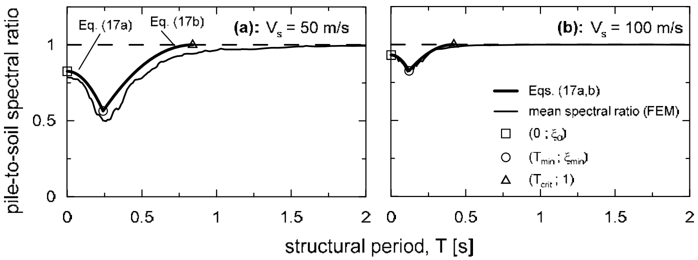


Figure 21. Mean spectral ratios for homogeneous soil conditions and corresponding proposed spectral reduction.

Approximate expressions of the curves corresponding to the homogeneous subsoil are given in the form:

$$T_{min} = 12\frac{d}{V_s} \tag{13}$$

$$T_{crit} = 3.5T_{min} \tag{14}$$

$$\xi_0 = \left(1 + 0.15\frac{\lambda_p}{V_{s1}} \cdot 10rad/s\right)^{-1} \tag{15}$$

$$\xi_{min} = 2.5\xi_0 - 1.5 \tag{16}$$

$$\begin{cases} \xi(T) = \xi_0 - \dfrac{(\xi_0 - \xi_{min})}{T_{min}^2}T^2 = \xi_0 - (\xi_0 - \xi_{min})\left(\dfrac{T}{T_{min}}\right)^2; & T \le T_{min} \\ \xi(T) = 1 - (1 - \xi_{min})\left(\dfrac{T_{crit} - T}{T_{crit} - T_{min}}\right)^2; & T_{min} \le T \le T_{crit} \end{cases} \tag{17a,b}$$

Multi Degree Of Freedom (MDOF) systems can behave quite differently from Single Degree Of Freedom (SDOF) systems. To address this point, the study has been focused on the response of linear and non linear frame buildings excited by either the filtered input motion or the free-field input motion, to quantify the beneficial effect coming from the piles on the seismic performance of non-dissipative and dissipative structures.
The parametric analysis has been carried out for five planar reinforced concrete (RC) frames (Fig. 22).
According to Eurocode 8 (2003) provisions, when a non linear analysis is being performed seismic effects can be computed by averaging the results coming from at least 7 acceleration time-histories, provided that the following rules be observed: a) the mean of the zero period spectral response acceleration values calculated from the individual time histories should not be smaller than the value of $a_g \cdot S$ the latter being the corresponding first value (T = 0) of the code based spectrum for the specific site; b) for T ranging between $0.2T_1$ and $2T_1$, with T_1 being the fundamental period of vibration for the superstructure, the mean 5% damping elastic spectrum, calculated from all time histories, is not smaller than 90% of the corresponding value of the 5% damping elastic response code spectrum. In order to match these

requirements the procedure suggested by Iervolino et al. (2008) has been adopted. Natural records have been selected from the Italian database SISMA (Scasserra et al. 2008) and the European Strong Motion database, ESD (Ambraseys et al., 2002).
Once performed the design, the analysis of the dynamic response of such structures under the action of 7 natural earthquakes has been conducted via time-history analyses, to assess the amount of the beneficial effect coming from the filtered effect exerted by the piles. Three subsoil conditions have been taken into account, and specifically: (a) h_1/d = 5, V_{s1} = 50 m/s, V_{s2}/V_{s1} = 4; (b) h_1/d = 10, V_{s1} = 50 m/s, V_{s2}/V_{s1} = 4 and (c) h_1/d = 10, V_{s1} = 100 m/s, V_{s2}/V_{s1} = 2. This choice is fully consistent with the assumption of ground type C. Dynamic time–histories analysis have been carried out for both linear and non linear structural models, so as a total number of 42 x 2 = 84 analyses for each building has been executed.

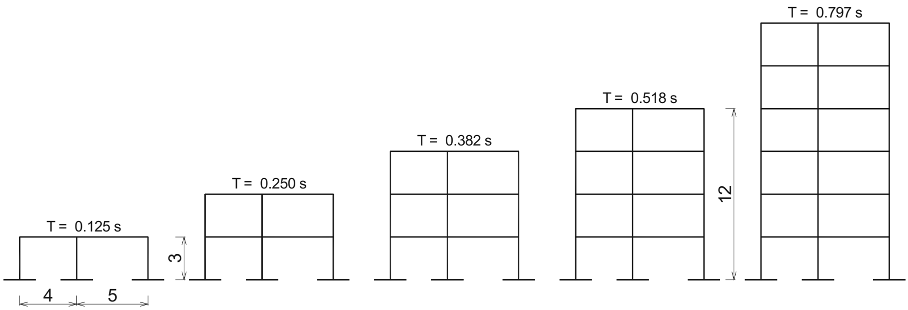


Figure 22. RC frame structures considered for inertial interaction analyses.

Linear time-history analyses have been performed with SAP2000 release 15.0 (2011) for each of the above frame structures under the action of the 7 natural earthquakes previously discussed. Roof displacement and base shear time-histories have been recorded for each analysis as output results. Then the structural demand reduction due to the pile filtering effect has been evaluated assessing: (i) the absolute value of the ratio between maximum roof displacements corresponding to pile filtered and free-field conditions respectively; (ii) the same ratio expressed in terms of base shear.
The mean value of the seismic demand can be adopted as the overall performance indicator of the structural model at hand, rather than the maximum seismic demand, also from the standpoint of current provisions, because acceleration time histories selected in this study satisfy the compatibility requirements prescribed by the Eurocode 8 (CEN, 2004). Fig. 23 refers separately to the three subsoil cases, namely (a), (b) and (c). Circles correspond to mean ratios of maximum base shear whereas rhombuses to mean ratios of maximum top displacements. The mean spectral ratios for the examined subsoil and the proposed spectral reduction for homogeneous soil conditions have been also added for comparison. It is worthy to notice that: (i) the reduction in seismic demand for base shear is the same as for top displacement; (ii) all the response reduction indicators result to be almost aligned on the thin lines representing mean spectral ratios; this indicates that the filtering effect due to the presence of the piles measured on SDOF does not change dramatically when referred to MDOF systems.
Non-linear, dissipative structures have been idealized as lumped plasticity models. The two ends of each element are modeled by a non-linear moment-curvature relationship, depending upon the current axial load and the amount of steel reinforcement. In addition to this, the

cyclic behaviour of such non-linear structural members has been described by the Takeda et al. (1970) constitutive model, which is thought of as the most common choice for RC elements.
The results coming from non-linear time-history analyses are synthesized in Fig. 24, where the results already plotted in Fig. 23 have been added for comparison. Due to non linear effects, the reduction of the maximum shear base is often negligible, so as the attention has been focused on the reduction of the average value of the maximum roof displacement, which can be considered as the seismic performance indicator of the structure when non-linearity of the behaviour is explicitly modelled.

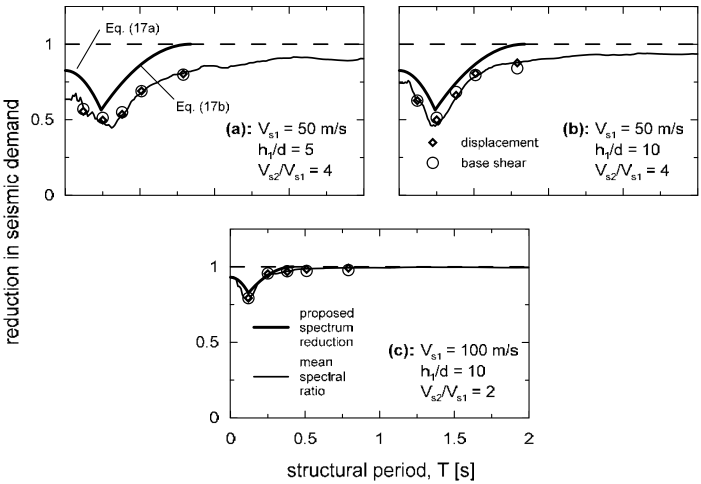


Figure 23. Reduction in seismic demand for MDOF systems.

With regard to V_{s1} = 50 m/s (Figs. 24a,b), the reduction of the maximum roof displacement associated to filtering effects is markedly affected by structural non linearity. At the lower value of the structural period (T = 0.125 s), the non linear (NL) model is much more sensible to pile filtering than the corresponding linear elastic (LE) model. For longer structural periods the situation is reversed, in the sense that the reduction in seismic demand for NL models is less pronounced than that pertaining to LE models.
With regard to V_{s1} = 100 m/s, the filtering action practically exerts the same effect regardless of structural non linearity.

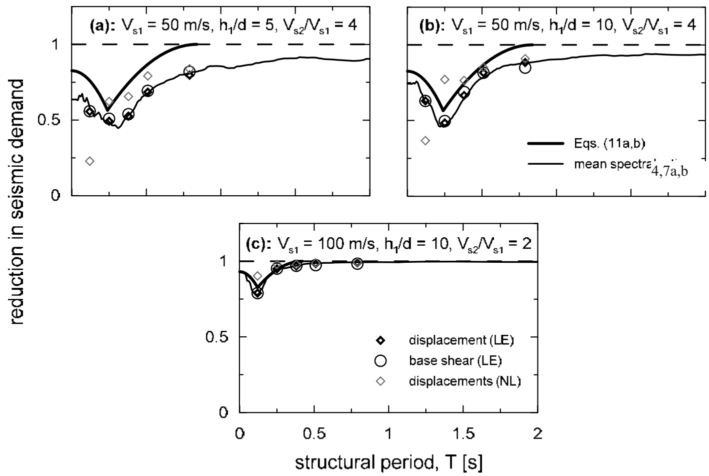


Figure 24. Non linear response of MDOFs in terms of filtering effect.

3.3.3 Pile-soil kinematic and inertial interaction from shaking table tests

The experimental data obtained within the project "Experimental Investigation of Soil-Pile-Structure Seismic Interaction", financed by the Transnational Access Programme of Seismic Engineering Research Infrastructures for European Synergies (SERIES), under the auspices of the 7th Framework of the European Commission, have been used as a benchmark to verify simplified formulae for bending moments induced by pile-soil kinematic interaction. Such data were obtained from the experimental activity was carried out at the BLADE laboratory of the University of Bristol The subsequent activities have been carried out thanks to the further financial support given by ReLUIS 2010-2013 project.

Laboratory investigation on model piles alongside with a few lessons from case histories, are essential tools to understand the actual response of single piles and pile groups. The model analysed on the shaking table at the BLADE of the University of Bristol is shown in Fig. 25a. It is made by five aluminum piles embedded in a bi-layer deposit (the ratio of V_s between the bottom and top layers is about 2). The tested configurations are schematically represented in Fig. 25b. The analysed set of data regards: the seismic wave propagation in the two-layer deposit contained in the shear stack (free-field condition), the kinematic and inertial interaction of single piles and pile groups. Tests were conducted with or without the superstructure.

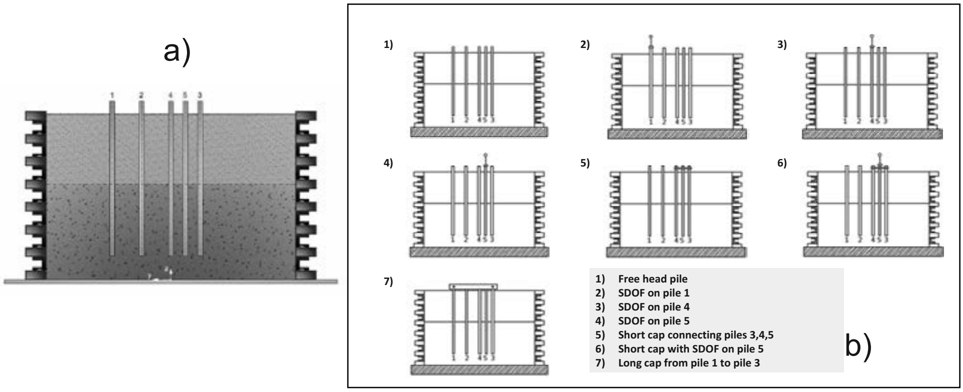


Figure 25. Model setup: (a) subsoil configuration, (b) configurations.

Fig. 26. shows typical results obtained applying sinedwell input motion on the shaking table, with different peak ground accelerations (ranging from 0.008*g* to 0.069*g*), and a fixed excitation frequency equal to 30 Hz.

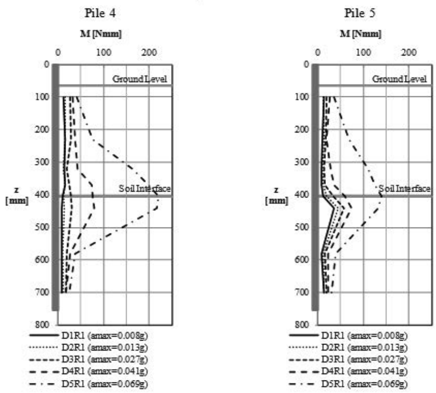


Figure 26. Envelope of bending moments vs. depth for sinedwell input motion with different peak ground accelerations and an excitation frequency of 30 Hz.

As expected, the free-field response within the deposit increases with the applied input acceleration. At the same time, kinematic interaction becomes more and more significant, especially close to the interface between the two soil layers. This aspect is more evident on the instrumented piles 4 and 5.

One of the aims of the experimental tests is to assess the dynamic structural response of simple one-degree-of-freedom structures (SDOF) accounting for the soil flexibility. The shaking table tests were conducted on different system configurations (see the schemes in Fig. 25) to investigate experimentally the soil-structure-interaction (SSI) effects. As it is well known, SSI gives rise to the following effects:

1. period elongation of the super-structure;
2. increase of the equivalent viscous damping of the super-structure.

The aforementioned combined effects are of paramount importance for the seismic design of structural systems, especially buildings and bridges; the total seismic base shear is indeed dependent upon the fundamental period of vibration (elongated period) and the compliant viscous damping (accounting for SSI).

Figs. 27 and 28 displays the transfer functions of the free head piles with SDOF acting on piles 4 and 5, respectively; the input acting on the shaking tables is a white noise with an amplitude acceleration of 0.02*g*. Different values of the structural masses are considered: 97*g*, 150*g*, 200*g*, 336*g* e 665*g*.

The dynamic response observed for piles 4 and 5 is similar. As the mass of the SDOF decreases (from the value of 665*g* to 97*g*) the frequency increases, as expected, since all the other mechanical parameters remain constant (in particular the lateral stiffness of the soil, the piles and SDOF). It is instructive to note that the decrease of the mass leads to the higher amplitudes of the transfer function, thus showing typical resonance in the structural response. The fundamental period of the soil is about 30 Hz.

The SDOF on pile 5 shows lower amplitude amplification with respect to the SDOF on pile 4, as the mass varies. The latter lower amplifications may be attributed to the different level of lateral confinement of pile 5 with respect to the case of SDOF on pile 4.

The short cap is a lateral restrain applied on the top of piles 4 and 5. Such restrain does not seem to vary remarkably the dynamic lateral response of the sample systems. The computed transfer functions in Figs. 29 and 30 provide the results of with the SDOF on pile 5 with the short cap. The results were determined for 0.02*g* white noise input on the shaking table. The plots display the amplifications at the top of the SDOF with respect to the input on the table. An additional mass of 1345 g was also considered for the SDOF. The system exhibits the resonance for lower values of masses. The response assessed for the case of piles with short cap shows also the onset of local resonances at higher frequencies.

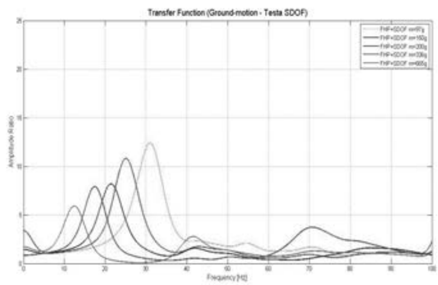


Figure 27. Transfer function relative to the free head pile with SDOF on pile n.4: SDOF versus shaking table input motion.

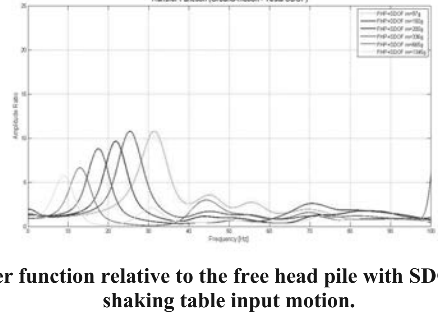


Figure 28. Transfer function relative to the free head pile with SDOF on pile n.5: SDOF versus shaking table input motion.

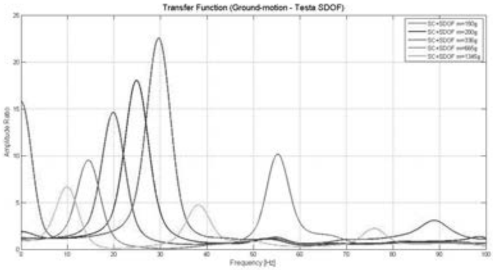


Figure 29. Transfer function relative to the short cap with SDOF on pile n.5: SDOF versus shaking table motion.

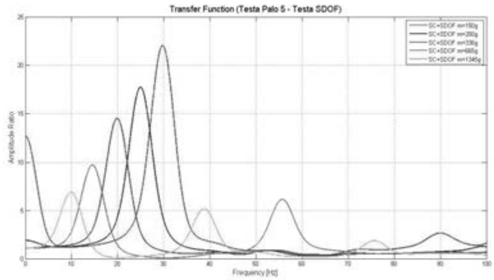


Figure 30. Transfer function relative to the short cap with SDOF on pile n.5: SDOF versus pile head.

The pile model layout is showed in Fig. 31. Sinedwell (harmonic) excitations with frequencies varying between 5 and 35 Hz and amplitudes ranging between 0.015g and 0.045g were applied.

A lumped-mass parameter model was implemented to perform numerical analyses in the time domain accounting for the nonlinear behaviour of the soil (Fig. 32). Free-field response is first evaluated and the interaction between the pile and the soil solved through the Winkler approach.

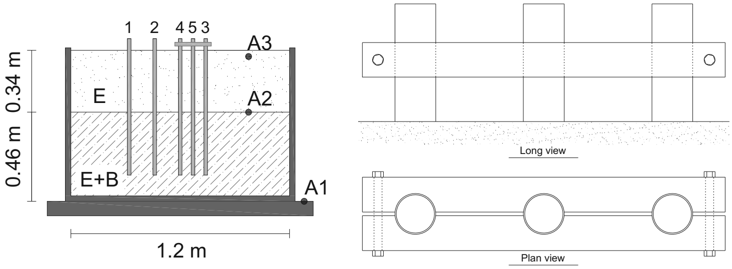


Figure 31. Model layout and boundary conditions at the head of piles 3, 4, 5.

Experimental results were back-analyzed in order to estimate the soil mechanical properties. White noise tests (inspector tests) were performed and accelerometers measures on the soil surface, on the shaking table floor and at the layers interface were used to obtain experimental transfer functions for the upper layer, $F_1(\omega)$, and the whole deposit, $F(\omega)$. A best fitting procedure were used to adapt the analytical transfer functions respect to the experimental results (Fig. 33), in order to evaluate the shear wave velocity, V_{s1} and V_{s2}, and internal damping, ξ_1 and ξ_2, of the two soil layers (Tab. 3).

Table 3. Properties of the soils.

Layer	Thickness (mm)	Soil type	Mass density (Mg/m^3)	Shear wave velocity (m/s)	Damping ratio (%)
Upper	340	Fraction E of LB Sand	1.39	54	10
Lower	460	Mixed LBS and Fraction E of LBS	1.78	89	7

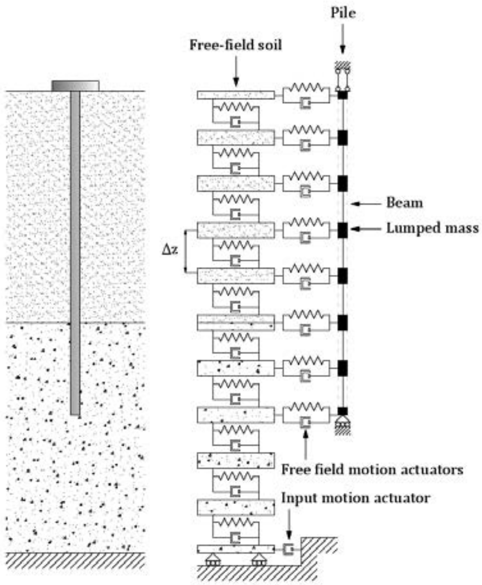


Figure 32. Lumped-mass parameter models.

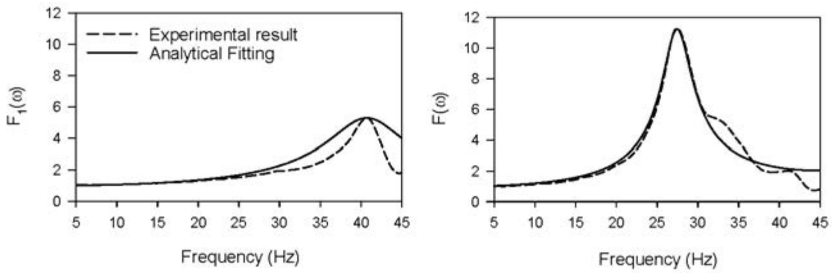


Figure 33. Experimental transfer functions best fitting.

The behaviour of inclined piles has also been investigated carrying out full scale field tests on three micropiles subjected to lateral loading. The first pile is vertical, the others are inclined respectively of 20° towards (+20°) and outwards (-20°) the direction of the horizontal force.

The piles have length 12 m, outer diameter 140 mm and thickness 12.5 mm. The diameter of the hole is 25 cm. Each pile is equipped with a cap to facilitate the action of the load, by placing a hydraulic jack horizontally between the cap and the toe of the concrete wall adjacent to the piles. The piles were instrumented with inclinometers tubes, placed inside for the whole length. Pairs of strain gauges are located on the lateral surface of the piles so as to be one on the opposite side of the other on the plane of bending. In the first 6 m of the length of the piles, the strain gauges are distributed in such position where the internal forces in the piles are expected to be large. They were also covered with a robust angle bar welded to the piles. Four dial indicators reading to 0.01 mm were positioned on the loaded surface of the cap: a couple on the upper part and a couple on the lower one. In this way, the measured displacements permit to evaluate the rocking and twisting mode of vibration of the cap. Cyclic loadings were applied to the pile heads in general agreement with the ASTM D3966 provisions. The soils, consisting of silty sand and sandy silt, were investigated with DMT and CPT tests, particularly. The maximum loads permitted by the equipment and instrumentations were 137.2 kN for the vertical pile, 156.8 kN for the pile inclined of -20° and 102.9 kN for the other one.
The inclinometer profiles for different levels of the loading are shown in Fig. 34. The depth displayed is computed from the head of the inclinometer tubes, which is placed 70 cm above the cap. All the piles exhibit the maximum bending at a depth of about 3 m, that corresponds to a distance of 1.3 m from the pile heads.

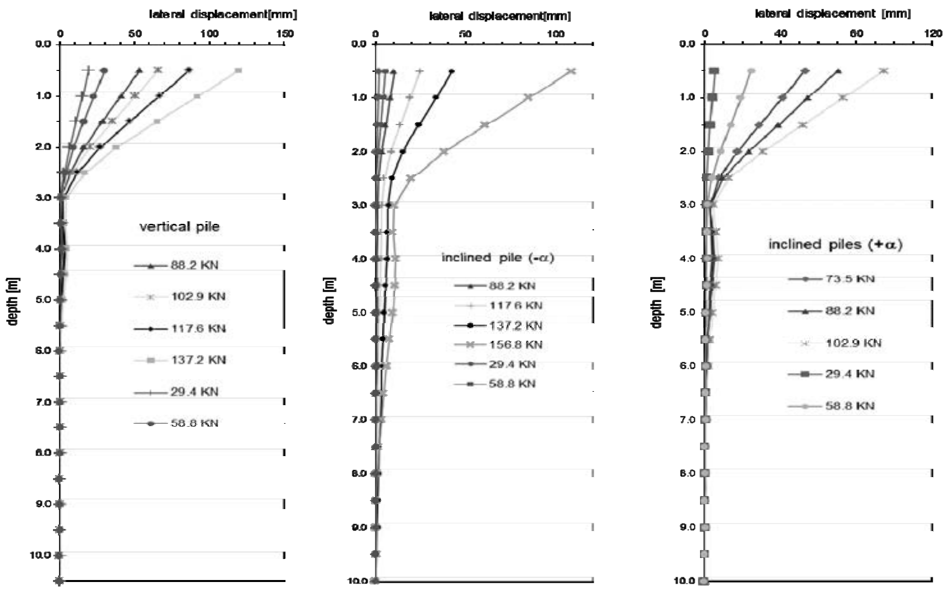


Figure 34. Deflection profiles from inclinometer readings.

The pile inclined in the opposite direction of the force always shows a minor displacement with respect to the others. Also during the unloading phase, the pile inclined of -20° undergoes the minor displacement, in the case of a force that decreases from 103 kN to 0 kN. The pile inclined of +20° shows smaller displacements than the vertical in correspondence of their heads, while increasing the depth the results tend to reverse and the vertical pile bends much more.

As a consequence, the better behaviour of piles inclined in the opposite direction of the applied lateral load can be confirmed by the obtained results. Therefore, under static forces which have the same direction is convenient to design the piles with negative inclination. On the contrary, if the forces have different directions, the behaviour of racked piles will differ considerably.

With respect to the top of the cap, the head of the piles are situated in the center of the cap. As a consequence, there is an eccentricity of the inclined pile respect to the base of the cap that is in contact with the soil. This particular geometry affects the response of the pile. When the lateral force acts near the smaller part of the base of the cap which is in contact with the soil, low amplitudes of the load are sufficient to produce the lifting of the cap. On the contrary, when the lateral force acts near the larger part of the base, a lower rotation can be observed as can be deduced from the inclinometer profiles and the dial indicators measurements.

3.4 Discussion

Many objectives of practical interest have been reached, that can be highlighted as follows.

The approach based on the so-called "macro-element theory" provided important results that have been validated through an identification analysis of a real case study. In more detail, a formulation of a macro-element model capable of reproducing the footing response under cyclic eccentric/inclined loads was developed and applied to the case of inhomogeneous subsoil.

Another aspect of the research concerned the bearing capacity reduction due to seismic actions. Numerical and simplified solutions for the evaluation of the ultimate load of shallow foundations subjected to seismic loading were used to evaluate the corrective factors which have be applied to the classic bearing capacity formula.

When dealing with piled foundations, inertial interaction analysis of the superstructure is usually performed by imposing that the foundation input motion is merely that of the free field. By contrast, the free-field signal is filtered out by the piles, yet this potential in reducing seismic demand is generally not exploited in engineering practice.

The results illustrated in this work, aimed at investigating pile-induced filtering effect, have shown that:

1. The mechanism of filtering effect is primarily governed by soil stiffness layer, pile diameter and excitation frequency; a unique parameter, encompassing simultaneously for all these variables, has been found to govern the phenomenon in harmonic oscillations, and is represented by the ratio of a characteristic pile wavelength λ_p over the soil wavelength V_s/ω;
2. With regard to transient response, the ratio of the spectral accelerations has a 'square root' shape and is characterized by a critical point, at which the filtering effect becomes negligible;
3. A reduction factor for acceleration design spectra has been suggested to be adopted in the presence of piled foundations.

3.5 Visions and developments

Further developments should be focused on the following main points.

a. Comparison of results based on macroelement approach and lumped parameter based approaches in order to define a rational guideline to assess the soil stiffness and damping by accounting for the strain level reached during a seismic event.
b. Further insights developments to clarify the relative role of kinematic and inertial interaction effect on piled foundations.
c. More insights studies on the development of plastic hinges in piled foundations.

4 EARTH RETAINING STRUCTURES

4.1 Background and motivation

Performance-based design is the goal currently pursued by most of the advanced seismic design codes worldwide. For gravity earth-retaining walls, the most significant performance indicator is the post-seismic, permanent displacement of the structure that can be related to the damage. In fact, as for these systems structural failure is not an issue, the collapse mechanisms may involve rigid body movement or bearing capacity failure. In principle, this kind of structure can be designed in such a way that overturning and loss of bearing capacity are prevented. The wall will then fail by sliding, that is an inherently ductile failure mechanism. This is an application to geotechnical systems of the capacity design concept widely used in structural design. Thus, the application of the performance-based design requires the definition of criteria for ensuring that gravity earth-retaining structures activate the more ductile failure mechanism which is sliding. This principle should be applied both at the design stage of new structures and in the development of methodologies for retrofitting existing ones. It is then necessary to develop simplified methods for calculating the residual displacement, beyond the limitations associated with currently used methods.
The performance of flexible retaining structures subjected to seismic actions can be evaluated with several methods at increasing levels of complexity from pseudo-static methods, or simplified dynamic methods, to fully coupled effective stress numerical analyses under dynamic loading.
In principle, the constitutive model adopted for the soil should be able to reproduce the main features of its mechanical behaviour under cyclic loading, such as the development of irreversible deformations, incremental non-linearity, hysteretic dissipation of energy and memory of previous stress history. This can only be achieved adopting advanced constitutive models developed within the framework of bounding surface plasticity, kinematic hardening plasticity and hypo-plasticity, generally not included in the libraries of commercial codes and requiring input parameters not routinely measured in field or laboratory tests.
Modern criteria for seismic design are based on performance-based approaches, in which the required performance depends on the return period of the earthquake.
The application of these approaches is based on the evaluation of the permanent displacements resulting from the activation of plastic mechanisms that can be achieved by means of advanced numerical analyses. The performance-based approach has been recently applied to the simple scheme of cantilevered wall by Callisto and Soccodato (2010), while Callisto and Aversa (2008) used some elements of the study for the seismic design of gravity walls and for pairs of flexible retaining structures connected by one level of support near the top. For multi-propped diaphragm walls, the application of performance based design approaches is less obvious as the system can fail with different plastic mechanisms. In fact, different construction phases may produce different seismic responses. It follows that the

main factors affecting the seismic behaviour of multi-propped diaphragm walls need still to be investigated.

4.2 Research structure

The scope of the research is the implementation of a new methodology to be used for the seismic design of new gravity earth-retaining structure and the retrofitting of existing ones in order to improve their expected performance in case of an earthquake. The new method includes the sliding block procedure originally proposed by Newmark for the seismic analysis of dams and embankments. Nevertheless, several differences exist between a soil block sliding along a slope and a wall that, in addition to the base excitation is also subjected to lateral loading induced by the backfill. The novel procedure improves the applicability of the Newmark method in computing the permanent displacement of gravity earth-retaining structures induced by earthquake loading.

The possibility of using the shaking tables and the laminar shear boxes of the laboratories of Eucentre and of the University of Bristol has been pointed out in the development of the project proposal. Validation through experimental verification by means of shaking table or seismic centrifuge testing was essential to endorse the outcome of numerical simulations.

The research activities planned for studying the seismic behaviour of diaphragm walls consisted of different activities in the following areas:

1. preliminary assessment of existing data obtained in previous research projects (ReLUIS 2005-2008, PRIN 2009) from small scale centrifuge models of pairs of diaphragm walls (cantilevered and with one or two levels of support) in dry sand;
2. physical modelling: four additional centrifuge tests on reduced scale models were performed to study the seismic behaviour of couples of diaphragm walls propped at the top and at dredge level, in saturated sand. The tests were carried out at the geotechnical centrifuge of the Schofield Centre, at the Cambridge University Engineering Department (CUED);
3. FEM and FDM back-analyses of centrifuge model tests using soil models capable to describe the cyclic behaviour of the soil in terms of effective stresses;
4. numerical simulations of more complex boundary value problems to evaluate the performance of multi-propped deep excavations.
5. simplified dynamic analyses: the results obtained by means of advanced numerical modelling and centrifuge tests were compared with those from simplified dynamic analyses.
6. definition of capacity design procedures for such structures, based on simplified hypotheses.

4.3 Main results

4.3.1 Rigid and earth-reinforced retaining walls

The experimental data obtained from the shaking table tests carried out on small-scale models of cantilever retaining walls in previous projects were compared with the results of numerical simulations of the tests. The comparison was carried out in terms of accelerations acting in the soil (base deposit and backfill) and on the wall and in terms of displacements and stresses acting in the structural elements. The numerical analyses were carried out using the finite difference code FLAC 2D (Itasca version 7). Fig. 35 shows the numerical model for configuration 1 for which a square grid of side length of 0.03 m was used. The soil was modelled as an elastic-plastic medium with failure criterion of Mohr-Coulomb (cohesion $c' = 0$; friction angle $\varphi' = 38°$, dilatancy $\psi = 0 \div \varphi$). The wall was modelled using elastic elements with a unit weight $\gamma_{rc} = 28$ kN/m^3, and elastic parameters $E = 70$ GPa and $\nu = 0.33$. Contacts between the different materials were modelled using layers of small thickness with suitable properties. The contact between the wall base and the foundation soil was described by a layer of elements having the same stiffness of Leighton Buzzard sand, and a friction angle of $\varphi_{int} = 23.5°$ and $\varphi_{int} = 28°$ evaluated experimentally for the smooth and the rough walls, respectively.

In the analyses the geostatic state of stress was first computed for the foundation soil, the wall elements were then activated in a single step, while the backfill elements were activated progressively, using layers of equal thickness (20 cm). For each step convergence was achieved. In the dynamic analyses, carried out at the end of the static stage, the free field boundary conditions were activated along the vertical edges of the analysis domain, together with viscous Rayleigh damping. As a first approximation the Rayleigh damping can be evaluated as the average frequency (f_{min}) between the fundamental frequency of the system and the predominant frequency of the input signal. In the present case, the fundamental frequency of the system is equal to about 46 Hz, while the input is an harmonic function at 7 Hz; then a frequency equal to 26 Hz was adopted with a damping of 5%. The acceleration time history recorded by channel 1, installed on the shaking table, (TA1_110624_D19R1) was used as input to the base of the model; however the input was applied in terms of velocity time history. The acceleration and the displacement time histories were computed at the same points for which measurements were provided by the shaking table tests (Fig. 1.1). Then, a comparison was possible between measured and computed values of the above quantities. Moreover the earth pressure and the plastic mechanism developed during the seismic loading were compared with those assumed in the analyses.

In the following, the results obtained using the results of run TA1_110624_D19R1 are shown. This input is characterised by a maximum acceleration $a_{max} = 0.19g$ that is higher than the one for which sliding was computed ($0.18g$ for configuration 1).

In Fig. 36 the acceleration time histories (black dashed line) computed along the vertical section S (see Fig. 35) are in a good agreement with the measured ones. Numerical analyses were also capable to reproduce the amplification of the acceleration time histories observed when moving from the base of the model to the top of the wall. When comparing the acceleration profiles, both measured and computed, along the three instrumented vertical sections it is seen that the amplification effects are mainly observed in the backfill, as the distance from the wall increases. Moreover for section FF, free field 1D condition are closely attained.

Sliding of the wall is seen to develop when an acceleration of $0.18g$ is achieved. This is in agreement, with the assumptions adopted for the design of the physical model. Computed and measured displacements of the wall are in a good agreement (Fig. 37), although computed displacements show lower oscillations.

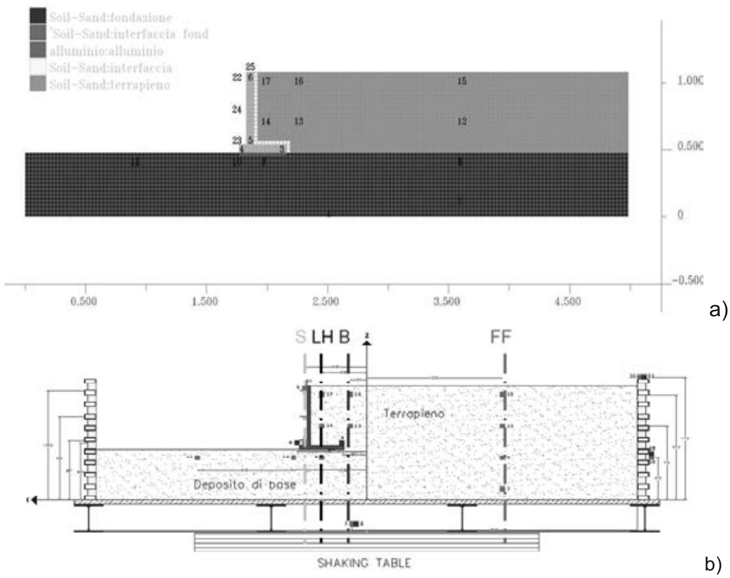


Figure 35. Flac Model a); Instrumented array: S: stem; LH long heel; B: Backfill; FF: Free Field b).

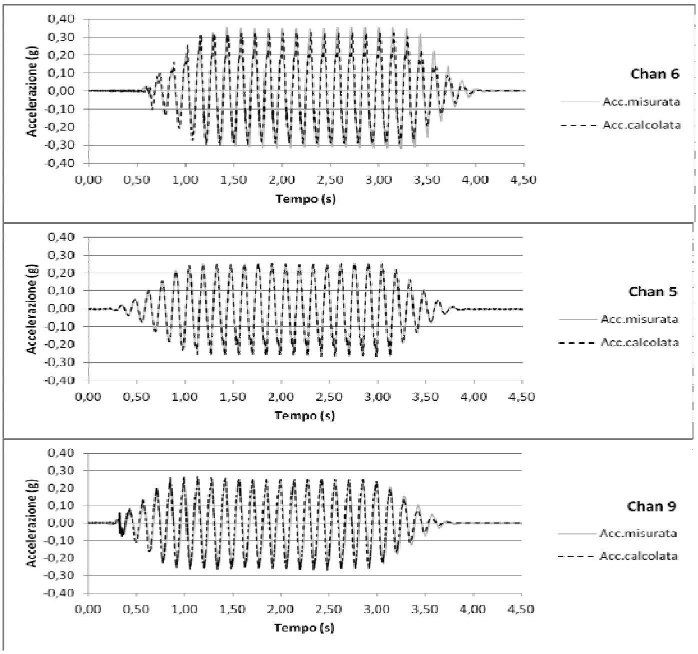


Figure 36. Comparison between computed and measured acceleration at section S.

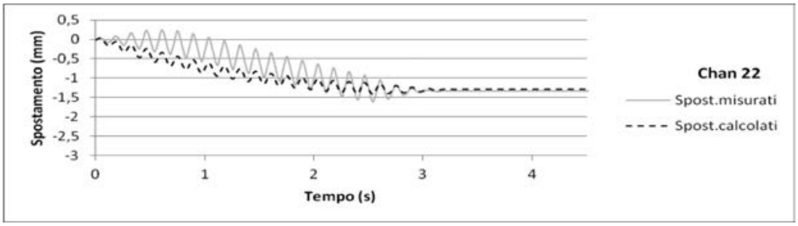


Figure 37. Comparison between computed and measured horizontal displacements.

The reduction and interpretation of the results from the shaking table tests on small-scale models of retaining walls on the 6-DOF seismic simulator of the University of Bristol have been also completed. Specifically, the attention has been focused on the analysis of wall displacements and bending stresses induced in the wall. Fig. 38 shows typical results obtained for the configurations 1 and 3 under harmonic excitations. Similar results were obtained by exciting the system with acceleration time histories representative of seismic events.

The measured displacements have been compared with those obtained by displacement analyses performed by the simplified method proposed by Zarrabi (1979). For wall with configuration 1, where the sliding mechanism has been clearly predominant, the analysis results are in a good agreement with the experimental data (Fig. 39a). On the contrary, for walls with configurations 2 and 3, where significant rotations have been observed, the kinematic of the wall cannot be interpreted by the simplified displacement method (Fig. 39b), as expected.

On the basis of the experimental evidence a procedure can be proposed, that may include the principle of the "capacity design" for the retaining walls.

The suggested procedure could be considered as a simplified dynamic approach, since it should start with the evaluation of sliding displacements induced by a proper input motion data set, and it should continue with a pseudo-static analysis to verify the different collapse mechanisms.

In detail, the steps of the procedure are given in the following:

1. individuation of the allowable displacement of the wall d_y (as a function of the restrain conditions and the tolerable displacements for the whole system);
2. analysis of the displacements induced by the selected input motions, and individuation of the threshold acceleration $a_t = N_{SLI} \cdot g$ of the wall corresponding to the allowable d_y;
3. among the group of walls having $N = N_{SLI}$, selection of those walls which verify the following conditions for pseudo-static actions corresponding to the seismic coefficient: $k_h = N_{SLI}$:
   - bearing capacity global safety factor PSF > 1 (say 1.1) (16)
   - overturning global safety factor PSF > 1 (say 1.1) (17)

Eqs. 16-17 guarantee that the potential kinematic of the selected walls, under seismic actions higher than the threshold acceleration a_t, will be the sliding one. In fact, as the maximum acceleration attains the value a_t, the wall will start to slide, and the thrust will not exceed the corresponding pseudostatic action, for which the bearing capacity and overturning mechanism have been verified .

With the suggested procedure, the capacity design principle is effectively implemented, since the sliding mechanism, which is ductile, is the preferred plastic mechanism of the wall, and the earthquake-induced wall displacements will not exceed the maximum tolerable displacement d_y.

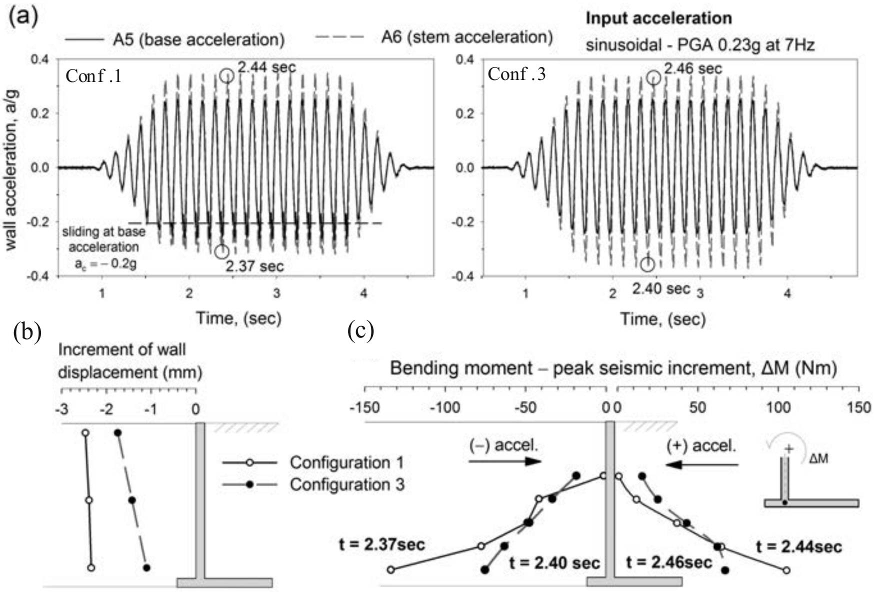


Figure 38. Comparison of typical experimental results for configurations 1 and 3 under harmonic-sinusoidal excitation: (a) measured wall accelerations, (b) increments of wall displacement (LVDTs D1-D2-D3: top, middle, bottom of wall); (c) peak seismic increment of bending moment.

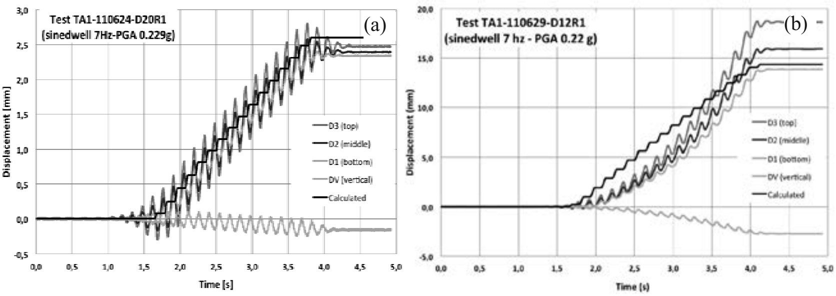


Figure 39. Comparison between typical experimental results of wall displacements and analytical sliding displacements obtained by Zarrabi method (1979): (a) configuration 1 and (b) configuration 2.

Starting from the work of Biondi et al. (2013), a procedure to evaluate the permanent displacements of cantilever L-shaped retaining walls undergoing sliding during earthquake loading has been developed. The displacement of the wall and that of the soil-wedge involved in the plastic mechanism is evaluated as the product of the displacement of the block that slides on a horizontal plane with the coefficients C_w and C_s respectively, depending on the geometrical and mechanical characteristics of the soil-wall system. The sliding block is characterized by the same critical acceleration coefficient of the actual soil-wall system.

Empirical relationships were also proposed to estimate the block displacement as a function of the soil-wall system critical acceleration and some significant parameters of the input motion (Fig. 39). The relationships between the sliding-block displacements and the seismic parameters were calibrated using the database of acceleration records ITACA (Italian Accelerometric Archive) and were compared with existing relationships developed using similar database of earthquake records.

Fig. 40 shows that for the parameters adopted in the analyses, coefficients C_w and C_s are nearly constant with the wall height H. Specifically, C_w is always lower than unity and, therefore, the displacements of the wall are lower than those of the sliding block. Conversely, coefficient C_s of the soil-wedge is always larger than C_w and is greater than unity (Fig. 39b). The proposed solution highlight that, due to the kinematic compatibility condition between wall and soil-wedge displacements, the vertical displacements of the retained soil-wedge are greater than the horizontal wall displacements, therefore representing the most critical design condition.

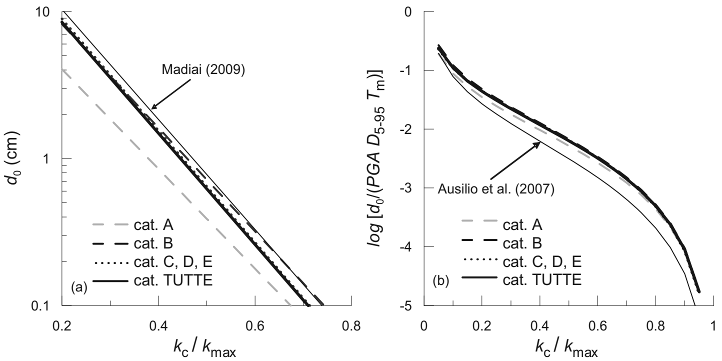


Figure 40. Empirical relationships to evaluate the sliding-block displacement induced by earthquake loading.

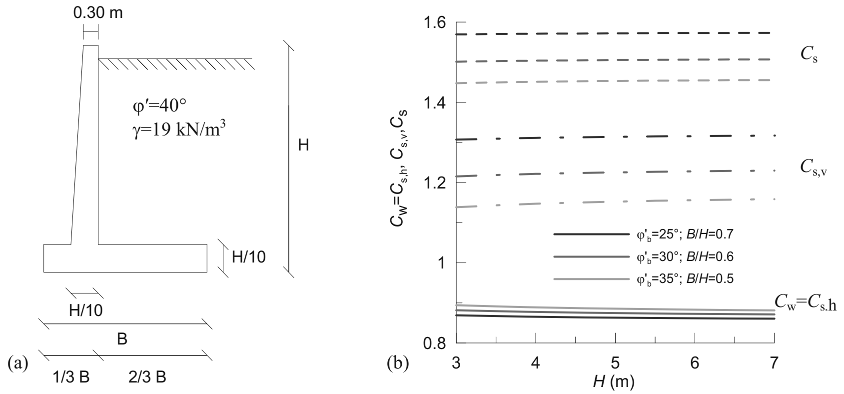


Figure 41. Coefficients C_w, $C_{s,v}$ e C_s versus wall height *H*.

A theoretical investigation on the dynamic behaviour of gravity retaining walls was also carried out to verify the soundness of simplified methods currently adopted for the seismic design of these structures. Some conceptual limitations of direct application of Newmark's sliding block method to the case of retaining walls have been discussed with reference to a simple scheme of two rigid frictional blocks resting on an inclined plane and interacting with one another. In particular, it has been shown that both the internal force between the blocks and their absolute acceleration are not constant during sliding, and must be computed by direct consideration of the dynamic equilibrium and kinematic constraints for the whole system. Based on these observations, a new method was proposed, yielding an extremely simple procedure to compute the relative displacements of the wall, by introducing a corrective factor in the Newmark's equation for displacements, related to the mechanical and geometrical properties of the soil-wall system. The comparison with the results of numerical analyses, in which the soil-wall system was subjected to real earthquakes and simplified input motions (wavelet), demonstrated that the proposed method is capable of describing fully the kinematics of the whole system under dynamic loading. Finally, the values of the critical seismic coefficient connected to overturning, sliding and bearing capacity collapse of the wall were obtained. The results show that, contrary to what is commonly accepted, for realistic geometries of the wall and of the strength of the foundation soil, the critical collapse mechanism is that corresponding to the bearing capacity of the foundation.

Evaluation of permanent wall displacements induced by earthquake loading can be complicated by two interacting phenomena: the coupled sliding and tilting motion of the wall and the generation of sliding surfaces in the backfill. A large number of non-linear, time-history analyses of gravity retaining walls have been performed using advanced numerical modelling (FLAC, Itasca, 2011). One type of foundation soil (stiff and dense sand) and two types of backfill (loose (B1) and dense (B2) sand) have been taken into consideration. Each soil has been modelled with Mohr–Coulomb (M-C) failure criteria combined with an elastic modulus-degradation technique calibrated on the experimental data proposed by Darendeli (2001). Two sets of different wall geometries, each one for a specific type of backfill (B1 and B2), have been previously designed for static loads and checked for overturning, sliding and bearing capacity. Ten real acceleration time histories recorded at outcropping rock sites have been selected in order to satisfy the criterion of spectrum-compatibility on a target elastic acceleration response spectrum (Type 1) on rock for 5% damping (recommended by EN1998-1, 2004). A view of the numerical model and horizontal displacement after a seismic event is shown in Fig. 42.

The response of the gravity retaining structures is significantly influenced by the soil compliance at the base. For this reason, two types of failure mechanisms have been considered in this study: loss of bearing capacity under the toe of the wall and residual horizontal displacements, set as 10% of the height of the wall.

The main outcome of the study is the development of a simplified method for fast preliminary assessment of the seismic permanent displacements of gravity retaining walls. The influence of different ground motion parameters has been discussed and the results have been compared with the most common simplified design procedures including Newmark sliding block method and the recommendations of Eurocode 8 (CEN, 2003).

The results showed that if the wall with backfill of dense sand is designed with an over-design factor (ODF) for sliding greater than 1.35 and for the backfill of loose sand with ODF > 1.2 (partial factors considered according to EN1997-1-2004), the wall does not collapse. However, a quantitative relationship between the ODF and the expected horizontal displacement could not be established. If it is assumed that the permanent horizontal displacements (according to EN 1998-5:2004) can be calculated as the product of the allowable displacement d_r and the ratio E/R (design horizontal forces and the resisting forces against sliding), this procedure tends to significantly underestimate the permanent displacements.

The permanent displacements calculated through some of the Newmark methods could be significantly underestimated for a_y/PGA greater than 50%, where a_y is the yielding

acceleration intended used in the sliding-block analysis. For lower values of this ratio the predictions become more reliable. The permanent horizontal displacements from a set of 10 spectrum compatible records vary significantly (from 5 cm to 55 cm), which means that the PGA is not the controlling ground motion parameter and a mean value of a horizontal displacement within a set of records is not reliable design parameter (Fig. 43). This is consistent with the results of similar studies available in the literature.
A linear regression has been calculated through the envelope of the permanent horizontal displacement as a function of the wall height, wall base width, Arias Intensity and type of backfill. The derived equation is written below:

$$d[m] = -m^3 \cdot \frac{BI_a}{H^{2m}} + c_1 \cdot I_a - c_2 \cdot \left(\frac{B^m}{H} - 1 \right) \tag{18}$$

where d is the horizontal displacement, H and B respectively the height [m] and the width of the gravity wall, I_a the Arias Intensity [m/s], and the coefficients c_1, c_2 and m are parameters that have to be determined. For this study, c_1 and c_2 values are respectively 0.25 and 0.3 for each type of backfill whereas m = 1 for dense sand (B2) and m = 0.7 for loose sand. The results from eq. (18) are compared with the results from the time-history analyses in Fig. 44.

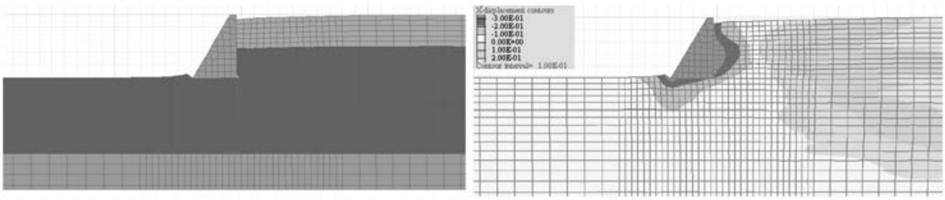


Figure 42. Left: General view of the soil deformation in the numerical model (FLAC2D) after an event; right: residual horizontal displacements in the numerical model after a seismic event.

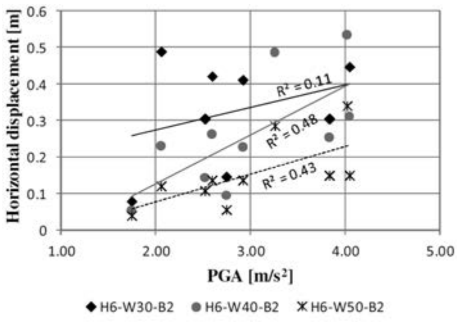


Figure 43. Relation between residual horizontal displacements of walls (with height 6m and backfill type B2, varying the base width: 3m, 4m and 5m) and PGA.

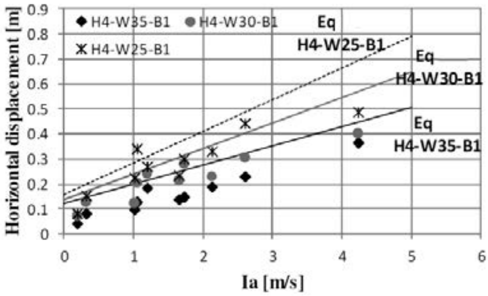


Figure 44. Comparison of the results from numerical models and Eq.1 (backfill B1, wall height 4m and base: 2.5m, 3m and 3.5m).

Part of the research was finally devoted to the earth-reinforced retaining walls that generally show a good performance under severe seismic events. However their behaviour is strongly influenced by the construction process and by the interaction between the soil and the reinforcements.

Some numerical studies were carried out on the seismic behaviour of a geogrid-reinforced earth wall. Specifically, a reinforced earth wall has been designed to activate local plastic mechanisms during strong ground motion, in which the resistance of the reinforcements is attained. Solutions based on limit analysis have been used to investigate plastic mechanisms both under static and pseudo-static conditions. Numerical analyses have been carried out to compare the plastic mechanisms formed under critical conditions with those assumed in the limit analysis-based solutions. The results show that the prevailing plastic mechanisms are different for static and pseudo-static conditions and that differences arise when comparing the results of numerical and analytical analyses under pseudo-static conditions. The seismic performance of the earth wall has been finally evaluated through dynamic analyses in which an acceleration time-history was applied at the bottom of the computation grid, using the same numerical model used for the static analyses. The analyses have been carried out assuming the reinforcements to behave as an elastic – perfectly plastic material characterised by either an infinite ductility or to deform up to a limit value, then loosing its strength. The results of the dynamic analyses showed that, besides the prevailing plastic mechanism, also observed in the pseudo-static condition, reinforcements of infinite ductility are able to contribute to energy dissipation during strong motion, due the mobilisation of the available strength in different portions of the soil-reinforcement system. Then, soil-reinforcements interaction strongly affects the development of plastic mechanisms and a good seismic performance of geo-grid reinforced earth retaining structures can only been achieved if ductile reinforcements are used.

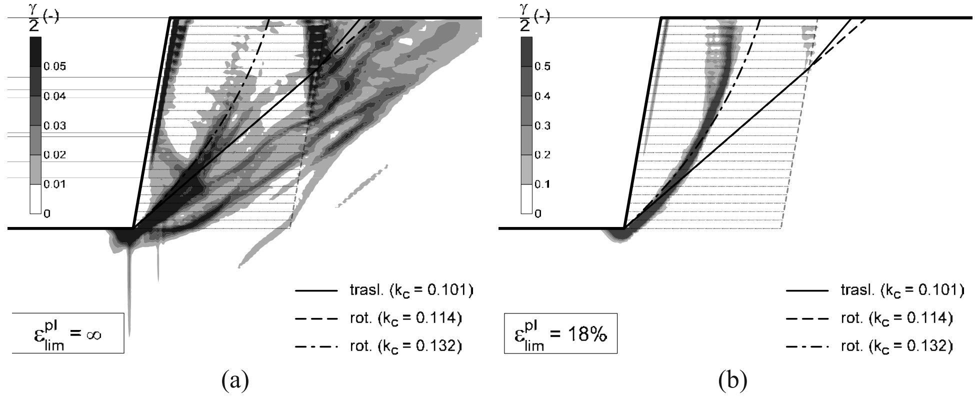


Figure 45. Dynamic numerical analyses, reinforcements of infinite ductility ($\varepsilon_{pl}^{lim} = \infty$) (a) contours of deviatoric strain; reinforcements of finite ductility ($\varepsilon_{pl}^{lim} = 18\%$), (b) contours of deviatoric strain.

4.3.2 Diaphragm walls

The activities have focused on numerical studies and experimental data from centrifuge tests. Numerical studies have concentrated on plastic mechanisms of cantilevered and single-propped embedded retaining walls subjected to steadily increasing static horizontal forces. Pseudo-static analyses produced, on the verge of collapse, the critical values of the horizontal acceleration, the soil-structure contact forces under critical conditions, and the corresponding internal forces. The above results were used in two distinct manners. First, they were tentatively reproduced employing the standard limit equilibrium design methods. While for the cantilevered walls only slight adjustments were necessary, for the propped wall it was necessary to develop an entirely new limit equilibrium analysis, in which the forces transmitted to the walls are a result of the interaction of two rigid blocks siding along planar surfaces, under active and passive limit conditions, respectively. A second stage consisted of a series of full dynamic analyses, in which the numerical models were subjected to acceleration time-histories applied to the bottom of the calculation grids, with sufficient amplitude to trigger the plastic mechanism previously analysed. Fig. 46 shows, for the two schemes under consideration, a synthesis of the maximum internal forces and the permanent post-seismic displacements obtained changing the embedment length, the angle of shearing resistance and the acceleration amplitudes.

The results of the dynamic analyses confirm the interpretation provided by the simplified approaches, showing that the critical seismic coefficient k_c is directly related to both the maximum internal forces and the maximum displacements: as k_c increases, the displacements decrease but the internal forces become larger. For the cantilevered walls, it was shown that the maximum internal forces can be readily evaluated using the results of simplified pseudo-static analyses of rotational plastic mechanisms. For the propped wall, the picture is complicated by the fact that a fixed constraint at the top of the wall may not be realistic for a real structure subjected to an earthquake, but it is a necessary hypothesis of any pseudo-static analysis. Additional dynamic analyses carried out with different constraints at the wall top showed that indeed this detail can influence substantially the internal forces in the wall. Therefore, the bending moments and prop forces evaluated from the proposed pseudo-static analyses of a rotational plastic mechanism should be regarded as reference values, that may be larger than the actual ones for a prop that moves during the earthquake, for instance because it is connected to an opposite wall; conversely, the actual dynamic internal forces may exceed those evaluates with the proposed procedure in the rather extreme case that the top of the wall is rigidly connected to a fixed point.

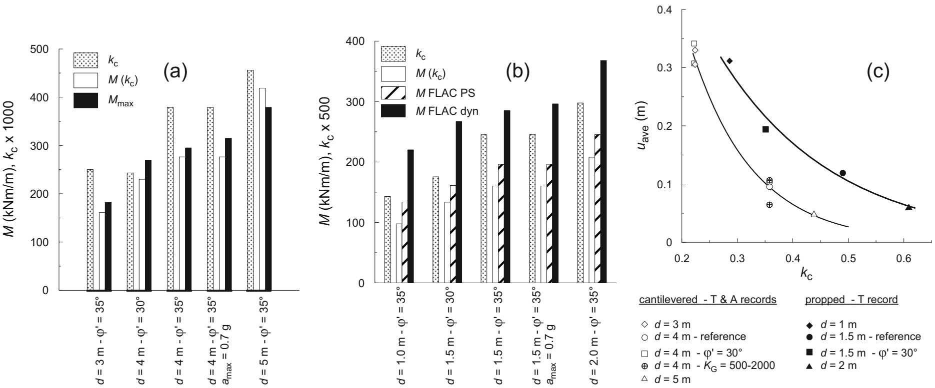


Figure 46. Maximum internal forces in cantilevered (a) and singly propped (b) retaining walls; permanent displacements plotted as a function of the critical seismic coefficient (c).

For diaphragm walls propped at the top and at the dredge level a parametric study was carried out in which, after a static stage of cantilevered excavation, the two opposite diaphragm walls (L = 8 m, H = 4 m, B = 16 m) were connected by two prop levels at the top and the bottom of excavation. The seismic input consisted of two natural acceleration time histories (A-TMZ, A-ASS), and two analytical waveforms corresponding to the peak impulse of the natural records (W-TMZ, W-ASS). All seismic inputs were scaled to three maximum acceleration levels (S01, S02 e S03).

The computed ground motion appeared to be strongly affected by both 1D wave propagation effects, due to soil stiffness changes and non-linear soil behaviour, and 2D effects, related to system geometry and soil-structure interaction phenomena.

Considering first 1D effects, Figs. 47a,b shows that the frequency content of input TMZ excites the system according to the first vibration mode, while the second mode is relevant for input ASS. Figs. 47c,e show that, for a given model, the peak acceleration a_{max} is not sufficient for describing the relationship between the seismic input and the maximum forces in the structure elements (M_{max} and N_{max}, on the lower prop); these appear to be mainly influenced by the coupling between the fundamental frequencies of the soil and the ones of the input signal, and by the time history of the seismic input before the attainment of the maximum acceleration. Post seismic actions seem to depend, keeping all other factors fixed, on the Arias Intensity, I_a, (Figs. 47d,f). However, non-linear effects are more pronounced with increasing I_A, especially for input A-TMZ.

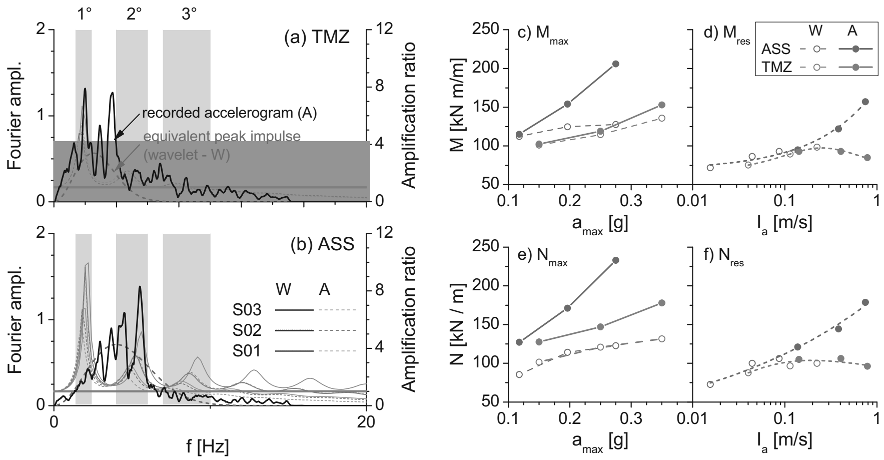


Figure 47. Comparison between Fourier amplitude of input accelerograms and non-linear 1D amplification function computed in free field conditions for ASS (a) and TMZ (b) waveforms. Maximum wall bending moment, M_{max} (c) and prop axial load, N_{max} (e), versus peak input acceleration, amax; post-seismic bending moment M_{res} (d) and prop axial load, N_{res} (f) versus Arias Intensity, I_a.

An experimental program of centrifuge tests has been also carried out at the Schofield Centre of the University of Cambridge (Aversa et al. 2015). The set of experimental program included four models of pairs of diaphragm walls embedded in saturated sand, at a centrifuge acceleration of 40*g*. Two tests (CWU1 and CWU2) were performed on cantilevered diaphragm walls and two (PWU1 and PWU2) on diaphragm walls propped at the top. Models were prepared using loose (D_r = 40%) and dense (D_r = 80%) Leighton Buzzard sand and were contained in a laminar box. The steady state hydraulic condition was hydrostatic at the dredge level. The pore fluid was Methyl Cellulose (HPMC), having a viscosity 40 times the viscosity of water, thus making the time scaling factor for inertial effect and pore pressure dissipation the same.

Each model was subjected to two or three earthquakes with the same frequency and increasing peak acceleration. Instrumentation was used to measure horizontal displacements of the walls, settlements of the model surface, accelerations and pore pressures at various locations within the model, bending moments in the retaining walls and the axial forces in the props, both during the seismic stages and in the subsequent post-seismic stages, in which excess pore pressure generated during the earthquakes dissipated (Fig. 48).

The experimental results indicate a significant increase in pore fluid pressure during the earthquakes, both in dense and loose sand (Fig. 49), changes in the bending moments distribution during the earthquakes (Fig. 50) and important attenuation of acceleration within the soil for the test in loose sand (Fig. 51).

During the third year two additional tests on saturated sand have been planned and are still under way. The new tests, one on pairs of cantilevered walls (CWU3) and one on pairs of single propped walls (PWU3) have similar geometrical and input features as the previous ones. In additions to the instruments already installed in the previous tests, flexible tactile pressure sensors (TEKSCAN, 1992) are installed on the walls in order to measure pressure distribution on the interface between the diaphragm walls and the soil.

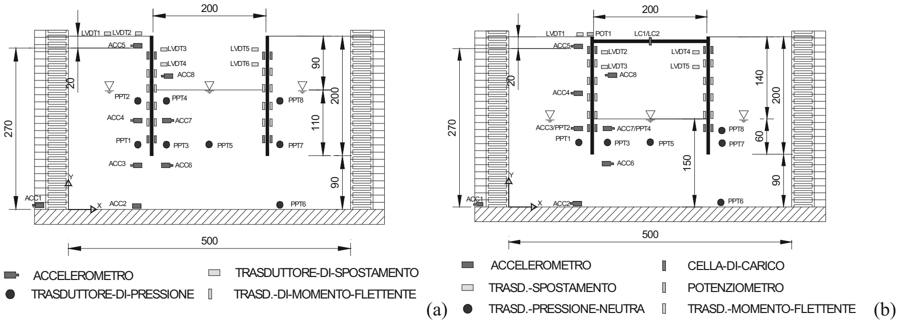


Figure 48. Layout instrumentation at model scale for CWU1 (a) and PWU1 (b).

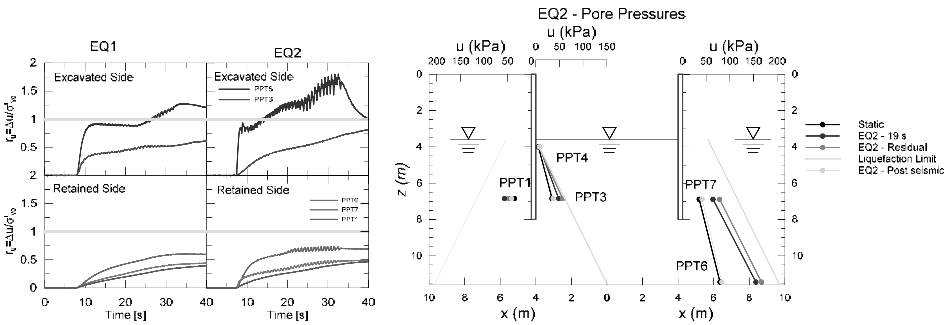


Figure 49. Pore pressure measurements during earthquakes at different location for CWU1 test and comparisons with a liquefaction indicator $r_u = \Delta u/\sigma'_v$

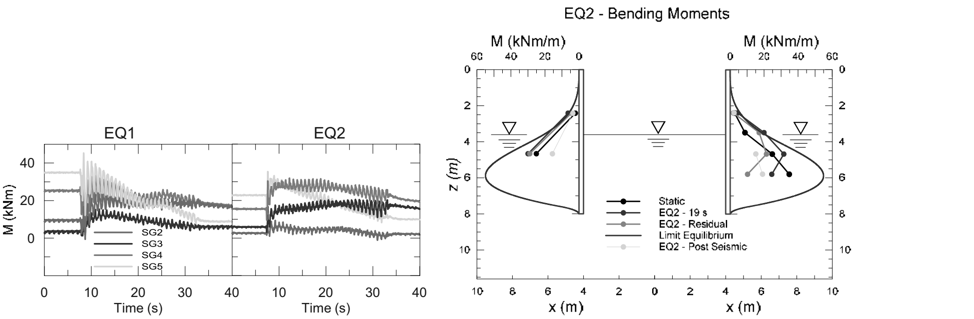


Figure 50. Bending moments during earthquakes for CWU1 test and comparisons with bending moments profile evaluated by limit equilibrium under static conditions.

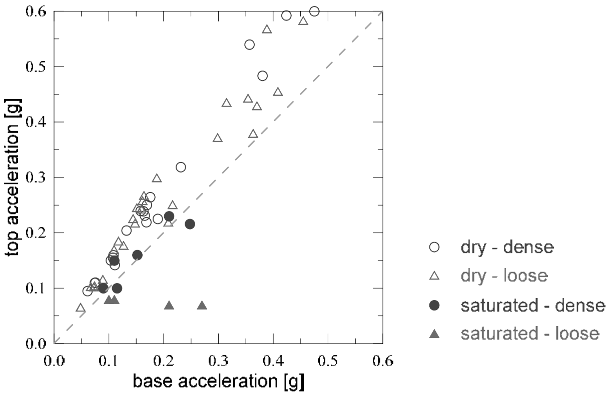


Figure 51. Amplification of accelerations: comparison between maximum accelerations measured in dry sand models and those recorded in the present tests.

A theoretical study was finally carried out addressing the problem of using velocity measurements to compute the shear strain induced by vertically propagating shear waves in a uniform soil layer. The problem has relevant impact in many areas of geotechnical earthquake engineering, such as in-situ measurements, centrifuge modelling and seismic design of pipelines and tunnels. It has been shown that the expression derived from the theory of elastic waves propagating in unbounded media, and used customarily in the literature, cannot be applied directly. By assuming isotropic visco-elastic behaviour for the soil, the exact point wise relation between particle velocity and shear strain was derived through the definition of a suitable transfer function

4.4 Discussion

4.4.1 Rigid and earth-reinforced retaining walls

A limit state is assumed to be reached when a structure is no longer able to guarantee the required performance in terms of safety and serviceability. The modern criteria for the design of retaining structure are based on performance-based approaches, in which the seismic performance is estimated allowing the strength to be fully mobilized over a finite time interval, thus resulting in the activation of plastic mechanisms and in the development of permanent displacements. Therefore, a key issue is to identify the most ductile plastic mechanism which allows the structure to develop permanent displacements without collapsing.

The experimental results obtained from the shaking table tests carried out on small-scale models of cantilever retaining walls provided a fundamental basis that should be used to calibrate the simplified methods proposed in the three year research project and to evaluate their capability to describe the behaviour of earth retaining walls under seismic conditions, both in terms of permanent cumulated displacements and stresses, and of the plastic mechanisms activated by earthquake loading.

4.4.2 Diaphragm walls

Numerical analyses carried during the three year project (e.g. Callisto and Soccodato 2010; Callisto 2014) show that the study of the soil-structure interaction for an earth retaining structure is especially complex under seismic conditions, because the inertial forces reach a portion of soil that is already mobilising part of its strength to sustain the excavation: a suitable soil model would need to include, in addition to non-linearity and damping, the progressive mobilisation of the soil strength. However, for design purposes it may be unnecessary to predict the detailed dynamic behaviour of the system: it could be sufficient to endow the system with features that will ensure a desirable behaviour under a severe seismic event. A desirable behaviour for retaining walls subjected to a severe earthquake is to mobilise a plastic mechanism deriving from the attainment of the strength in the volume of soil that directly interacts with the wall, while preserving the integrity of the structural members. Alternatively, the designer could select a plastic mechanism including the mobilisation of the wall capacity. Following this line of thought, it is shown that it is possible to use relatively simple pseudo-static tools, essentially based on the strength properties of the soil, to study the plastic mechanism associated with the desired behaviour; for a non-dissipating wall this procedure provides also the internal forces that the structural element is called to resist in order to ensure that the plastic mechanism will be maintained at its full strength during the seismic event. The adequacy of the proposed method is supported by the results of several dynamic analyses; these show on the one hand that the maximum internal forces are very close to the ones predicted by the iterative limit equilibrium method, and on the other hand that the seismic performance of the structure is uniquely related to the critical seismic coefficient, and is about independent from the plastic mechanism selected in the design

The main results obtained from centrifuge tests have been presented in terms of accelerations and pore pressures into the soil, and internal forces and displacements of the walls. Indirect measurements and visual inspection of the models revealed that, while excess pore pressures developed in both dense and loose models, liquefaction phenomena occurred only in the latter case, leading temporarily to a dramatic reduction of soil resistance. This fact had three major effects, which make the overall behaviour of saturated loose models substantially different from that observed in dry and saturated dense models: (1) the input signals were systematically de-amplified while propagating through the loose sand layers, with a sort of cut-off observed in the accelerations recorded close to the soil surface; (2) a significant reduction of internal forces was observed in the loose sand models, where even negative values of bending moments were reached during the transient stages; (3) retaining walls experienced significant permanent displacements, following mechanisms not always clear.

4.5 Visions and developments

4.5.1 Rigid and earth-reinforced retaining walls

For rigid earth retaining walls, the available experimental data obtained from shaking table test on small scale models of cantilever retaining walls, should be used to calibrate the proposed simplified procedures in order to evaluate their capability to describe the real seismic performance of rigid retaining walls. For earth-reinforced retaining walls, the possibility of carrying out dynamic centrifuge tests should be investigated, in order to provide an experimental base against which to calibrate the proposed simplified procedures.

4.5.2 Diaphragm walls

Numerical studies on the seismic performance of cantilevered and single-propped embedded retaining walls appeared to be able to reproduce the seismic behaviour observed in centrifuge tests carried out on granular soils. Results of parametric studies can then be used to isolate

and evaluate the main factors that affect the seismic behaviour of such structures thus providing important elements for effective design procedures.
However, experimental centrifuge tests on fine-grained soils should also be carried out to complete the experimental basis against which to calibrate results of numerical analyses.
Further research is also required to better investigate, both experimentally and numerically, the seismic behaviour of multipropped diaphragm walls that are of great interest for structures such as underground car parking and metro stations.

Results undertaken by the centrifuge tests have shown that the relative density affects significantly the seismic response of retaining walls embedded in saturated sand, where the excess pore pressures may lead to a substantial reduction of the soil resistance and, consequently, of the inertia forces into the soil. As a result, no increase of bending moments in the walls can be expected, as they strictly depend on both the inertia acting on the retaining side and on the soil passive resistance available below dredge level. From this perspective, the problem of computing the dynamic internal forces in the structural members become of little relevance, while the ability of predicting the permanent displacement displacements experienced by the walls play a major role

ACKNOWLEDGMENTS

The authors would like to acknowledge all the Researchers involved in the Geotechnical line of the ReLUIS project for the high level researches performed within this project and, among them, Dr. Luca de Sanctis also for his fundamental help in preparing this report.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 473-499, doi: 10.14599/r101313, © 2015 Doppiavoce, Napoli, Italy

TERRITORIAL SEISMIC RISK ASSESSMENT

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1 INTRODUCTION

Recent catastrophic seismic events (e.g. Umbria-Marche, 1997; San Giuliano, 2002; L'Aquila, 2009; Emilia, 2012), having caused severe damages on structures and unexpected high casualties with respect to the seismic intensity, have foreseen the attention of the authorities on the mitigation of the seismic risk in Italy. Considering the local seismic hazard and the vulnerability of existing built environment as well as the relative exposure, the research here described pursues realistic assessment of seismic risk, as a supporting tool for programming effective mitigation measures.

In this framework, the risk analysis (as assessment of possible consequences of seismic events on affected areas) have a dual purpose: to optimize planning and emergency management; and to develop strategies and policies to prevent and/ or mitigate the risk.

According to the scientific literature, *risk* is defined as the convolution of three probability functions, respectively attributed to the realization of three uncertain parameters, which are: hazard, exposure and vulnerability.

Hazard is the probability that, in a specific area, an event of given intensity occurs within an assigned time period.

Exposure is the extension, the quantity and the quality of different elements which characterize the examined area (people, buildings, infrastructures, etc.), whose conditions and/ or functionality can be altered or destroyed by the event. *Vulnerability* is the probability that these elements at risk can be damaged at the specific level since one or more events of given intensity occur.

Depending on its objectives, risk analyses can be developed at national or territorial (regional or sub-regional) scale. Based on the different level of application, it is necessary to choose an appropriate geographical Minimum Reference Unit (MRU), which must coincide with the minimum unit of analysis of hazard, exposure and vulnerability. Generally, for evaluations at national scale, MRU is the Municipality, while at regional scale, where greater detail is required, it can be taken as a sub - municipal area, constituted, for example, from a cell of the order of 500x500 m or even smaller up to 250x250 m according to the reliability of the data available.

The research activities here reported concern improvements on *vulnerability* and *exposure* for ordinary buildings, in the perspective of territorial seismic risk assessment.

2 BACKGROUND AND MOTIVATION

In the risk analysis, exposure and vulnerability are strictly connected. For each category of elements at risk under examination (people, buildings, infrastructures, etc.), seismic vulnerability assessment must be associated to "qualitative and quantitative analysis of the elements exposed" (exposure), in order to grouping the elements, with similar seismic behaviour, in categories called 'vulnerability classes'.
Theoretical evaluation of the seismic vulnerability can be conducted in different ways: expert judgment, examination of the damage caused by past events, analytical processing and / or experimental tests.
The seismic vulnerability of a building, identified with a single structural units, is the probability that the system (entire building), the subsystems (walls, frames, roofs, etc.), or the system components (cladding panels, windows, doors, etc.) are damaged by hazards of given magnitude.
The vulnerability definition requires to define the level of 'damageability'. In Table 1, a possible damage scale for buildings is proposed.
Studies on seismic vulnerability suggest, among others, to express the vulnerability through two main different tools: Damage Probability Matrices (DPM) and Vulnerability Curves.
DPM, introduced by Whitman in 1973, express vulnerability through damage distribution for discrete values of the hazard parameter adopted, generally the macro-seismic intensity. In Table 2, a DPM developed by the author for Italian buildings are reported. They have been developed through analyses of building damage caused by major earthquakes occurred in Italy from 1980 to 2008.
The vulnerability curves express the probability that a given "vulnerability class" exceeds a certain level of damage, given a level of seismic intensity measure , which may be the seismic acceleration peak, the spectral intensity, the macro-seismic intensity, the wave magnitude, etc. (Figure 1).
DPM and/ or vulnerability curves can be obtained through three different approaches: observational methods, mechanical methods and hybrid methods. *Observational methods* evaluate DPM through statistical analyses of buildings damage caused by past events. *Mechanical methods* evaluate vulnerability curves through statistical processing of the results obtained by non-linear mechanical analysis (for example, generated by Monte Carlo simulation) conducted on random samples of models representing the buildings in the geographical area of interest, subjected to representative sets of hazards. *Hybrid methods* evaluate DPM and / or vulnerability curves combining mechanical analysis and observation of damage caused by past seismic events.
In this framework, the goals of the research are two. The former is the quantitative evaluation , of the correlations between some specific typological-structural characteristics that we call 'vulnerability factors' (VF) and the main failure mechanisms potentially triggered by the seismic action for masonry building. The latter is the assessment of the vulnerability curves for each buildings typology that relate the ground shaking, expressed in PGA, with the damage level expected (Zuccaro and Cacace, 2012).
The assessment of buildings exposed to seismic hazard can be developed by statistical analyses of the typological characteristics of the buildings in the area under examination with the aim to evaluate the distribution of the different vulnerability classes.
In the field of seismic risk assessment at regional scale, one of the greatest difficulty is the absence of a buildings inventory containing sufficient information to formulate an estimation of the seismic behaviour. With the aim to overcome this obstacle, the author has developed a new procedure able to provide a seismic exposure assessment on the basis of ''poor''

information collected by the Italian Census Database on buildings produced by Italian Central Statistics Institute (ISTAT, Italian acronym).

Table 1. Damage scale for buildings.

Damage Level		Description
D0	No damage	
D1	Light damage	Negligible damage to structural elements
		Negligible damage to infill panels
D2	Moderate damage	Moderate damage to structural elements
		Moderate damage to infill panels
D3	Heavy damage	Heavy damage to structural elements
		Severe damage to weak infill panels. In a few cases, total collapse of infill panels
D4	Local collapse	Local collapse of structural elements
		Collapse of strong infill panels
D5	Total collapse	Total collapse

Table 2. Damage Probability Matrix developed through analyses of building damage caused by major earthquakes occurred in Italy from 1980 to 2008.

Vulnerability class (EMS '98)	Macro-seismic Intensity	Level of damage					
		D0	D1	D2	D3	D4	D5
A	V	0,3487	0,4089	0,1919	0,0450	0,0053	0,0002
B		0,5277	0,3598	0,0981	0,0134	0,0009	0,0000
C		0,6591	0,2866	0,0498	0,0043	0,0002	0,0000
D		0,8587	0,1328	0,0082	0,0003	0,0000	0,0000
A	VI	0,2887	0,4072	0,2297	0,0648	0,0091	0,0005
B		0,4437	0,3915	0,1382	0,0244	0,0022	0,0001
C		0,5905	0,3281	0,0729	0,0081	0,0005	0,0000
D		0,7738	0,2036	0,0214	0,0011	0,0000	0,0000
A	VII	0,1935	0,3762	0,2926	0,1138	0,0221	0,0017
B		0,3487	0,4089	0,1919	0,0450	0,0053	0,0002
C		0,5277	0,3598	0,0981	0,0134	0,0009	0,0000
D		0,6591	0,2866	0,0498	0,0043	0,0002	0,0000
A	VIII	0,0656	0,2376	0,3442	0,2492	0,0902	0,0131
B		0,2219	0,3898	0,2739	0,0962	0,0169	0,0012
C		0,4182	0,3983	0,1517	0,0289	0,0028	0,0001
D		0,5584	0,3451	0,0853	0,0105	0,0007	0,0000
A	IX	0,0102	0,0768	0,2304	0,3456	0,2592	0,0778
B		0,1074	0,3020	0,3397	0,1911	0,0537	0,0060
C		0,3077	0,4090	0,2174	0,0578	0,0077	0,0004
D		0,4437	0,3915	0,1382	0,0244	0,0022	0,0001
A	X	0,0017	0,0221	0,1138	0,2926	0,3762	0,1935
B		0,0313	0,1563	0,3125	0,3125	0,1563	0,0313
C		0,2219	0,3898	0,2739	0,0962	0,0169	0,0012
D		0,2887	0,4072	0,2297	0,0648	0,0091	0,0005
A	XI	0,0002	0,0043	0,0392	0,1786	0,4069	0,3707
B		0,0024	0,0284	0,1323	0,3087	0,3602	0,1681
C		0,0380	0,1755	0,3240	0,2990	0,1380	0,0255
D		0,0459	0,1956	0,3332	0,2838	0,1209	0,0206
A	XII	0,0000	0,0000	0,0000	0,0010	0,0480	0,9510
B		0,0000	0,0000	0,0006	0,0142	0,1699	0,8154
C		0,0000	0,0001	0,0019	0,0299	0,2342	0,7339
D		0,0000	0,0002	0,0043	0,0498	0,2866	0,6591

In particular, statistical relationships between building types and vulnerability classes were determined (Zuccaro et al. 2012). This has been possible thanks to the examination of 'specific' information on structural typologies on a wide sample of buildings spread out in all the Italian territory, investigated by a quick building survey promoted by LUPT- PLINIVS Study Centre[1] (University of Naples Federico II) and collected in a unique database (PSV-DB).

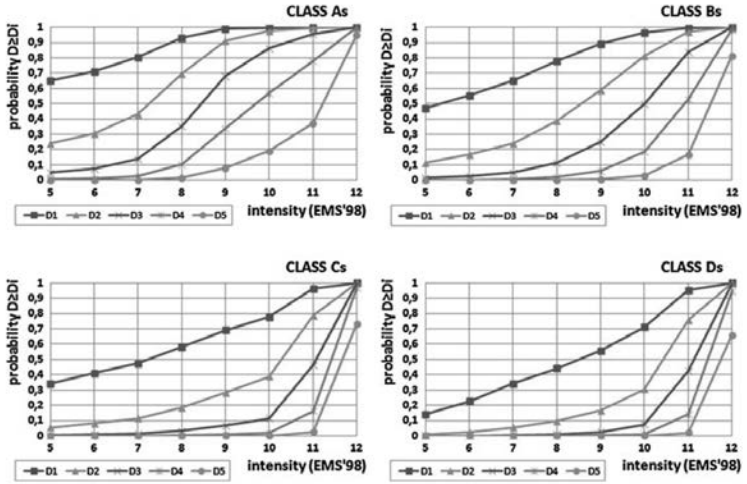


Figure 1. Seismic vulnerability curves: macro-seismic intensity (EMS'98) versus probability that the damage D≥Di (Zuccaro, 2004; Zuccaro et al., 2008a; Zuccaro and De Gregorio, 2013).

3 RESEARCH STRUCTURE

3.1 Building inventory at national scale based on census data analysisand seismic vulnerability according to EMS classes

<u>*Inventory and Census data*</u>

Development of risk maps at national scale requires the assessment of seismic vulnerability classes distribution (inventory). The proposed procedure allows to get it by using only the National Census data base provided by the Italian National Institute of Statistics (ISTAT-DB). It has collected during the 14th Italian General Census (21 October 2001). This contains either set of data on buildings aggregated in categories (i.e. number of buildings having vertical structure in masonry or reinforced concrete (R.C.); number of buildings with number of storey in the range 1-2, 3-5 etc.) or set of data disaggregated (i.e. number of masonry buildings having number of storey in the range 1-2, 3-5 etc..).

The approach here illustrated is based on statistical correlations between 2001 Census data and a database, containing structural- typological information of about 260,000 buildings in about 700 Municipalities collected by LUPT - PLINIVS (PLINIVS DB on typology) during about more than twenty years of field missions (in 'peacetime' or after earthquake).

[1] PLINIVS Study Centre for Hydrogeological, Volcanic and Seismic Engineering. Operating Structure of Interdepartmental Centre of Research Laboratory of Town and Territorial Planning *Raffaele d'Ambrosio* (LUPT), University of Naples Federico II, ITALY. Competence Centre for Italian Civil Protection.

The LUPT - PLINIVS study center has the whole aggregated ISTAT - DB (A – ISTAT - DB) and a selection of the disaggregated ISTAT - DB (D - ISTAT - DB) for 188 Italian municipalities, thanks to Italian Civil Protection.

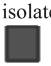
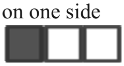
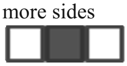
From the ISTAT DB, six parameters (see Table 3) have been selected, these are also included in the PLINVS - DB on typologies.

The parameters considered are:

1. Position of the building in the aggregate (3 classes: isolated, on one side, on two or more sides);
2. Material of vertical structures (4 classes: masonry, reinforced concrete, RC with pilotis at ground level, other);
3. Age of building (7 classes: before 1919, 1919- 1945, 1946- 1961, 1962- 1971, 1972-1981, 1982- 1991, after 1991);
4. Number of floors above ground (4 classes: 1-2, 3-4, 5-6, 7-8);
5. Altimetry of the municipality where the building is located (3 classes: plain, hill, mountain);
6. Demographic class of the municipality where the building is located (7 classes: <500, 5001,999, 2,000-4,999, 5,000-9999, 10,00049,999, 50,000249,000, >250,000 inhabitants).

The first four parameters (1-4) describe specific characteristics of buildings which directly influence the seismic vulnerability of structures. Whereas, the parameters 5 and 6 are indirectly connected with the seismic behaviour, since they identify two particular aspects of the municipalities, altitude (5) and demographic class (6), which are often linked with specific buildings typologies and quality.

Table 3. Parameters of buildings.

	PARAMETERS					
CLASSES	P_1	P_2	P_3	P_4	P_5	P_6
	Building position in the block	Material of vertical structures	Age of building	Number of floors above ground	Altimetry [m above sea level]	Demographic class of the municipality [inhabitants]
I	isolated 	Masonry	before 1919	1-2	plain (0-300)	< 500
II	on one side 	reinforced concrete (rc)	1919- 1945	3-4	hill (300- 600)	500-1,999
III	on two or more sides 	rc with pilotis at ground level	1946- 1961	5-6	mountain (> 600)	2,000-4,999
IV		other (mixed structures, steel, timber, etc.)	1962- 1971	7-8		5,000 –9,999
V			1972- 1981			10,000-49,999
VI			1982- 1991			500,00-249,999
VII			after 1992			>250,000

Description of the Procedure

The goal of the inventory analysis is to assess, for all Italian municipalities (8,101), the number of buildings and their distribution in vulnerability classes according to the European Macro-seismic Scale (Gruntal G., 1998).

The procedure is summarized below step by step.

1. Assignment of the vulnerability class to the 260,000 buildings of the PLINIVS - DB on the basis of the 'SAVE' methodology.

In the SAVE project (2000-2002) was developed a procedure to assess the Vulnerability class of buildings starting from the EMS' 98 classification mainly based on the vertical structure. The SAVE procedure was able to assess the influence on the vulnerability of other crucial typological features called 'vulnerability factors' (i.e. presence of ring beam, ties, pitched roof etc.). This influence was calibrated by statistical analyses of the damage database, available at PLINIVS Study Center, collecting the damage observed in the past events (see Zuccaro et al., 2008b; Zuccaro and Cacace, 2014 for details), taking into account the structural- typological characteristics of constructions (vertical and horizontal structures, roofs, number of floors, age, etc.), the vulnerability class to all the about 260,000 buildings in the PLINIVS - DB is assigned.

2. Correlation between Census data and vulnerability.

The buildings of the PLINIVS - DB are then grouped on the basis of the six parameters indicated in Table 3; the vulnerability classes distribution for each group is analyzed (e.g. Vulnerability classes distribution VS Age). The correlations between the six parameters and the buildings vulnerability classes are then found.

3. Vulnerability distribution assessment in each Municipality.

The correlations assessed in step 2, applied to the data in SI-DB, allow to assess the vulnerability distributions for each Census Section of the municipalities considered (Figure 2).

4. Validation for single parameter.

From the PSV - DB, a set of 188 Municipalities is extracted. They are the same municipalities included in the SI-DB. The vulnerability classes distributions in PSV-DB (step 1) are compared with the vulnerability classes distributions in SI-DB (step 3) with the aim to identify the most significant typological parameters P_i.

The error has been calculated by the following *comparative index* $K_{c,Pi}$:

$$\mathrm{Kc,p_i}=\frac{\sum_{Z=1}^{4}\left|C_{VC_{Z},P_i}-C_{VC_{Z},S}\right|}{4} \qquad (1)$$

where: $C_{VC_z,S}$ is the percentage of buildings of vulnerability class VC_z (A, B, C, D), surveyed in the PSV-DB (step 1), C_{VC_z,P_i} is the percentage of buildings of vulnerability class VC_z(A, B, C, D), assessed from SI-DB (step 3) applying the correlations found for the parameter P_i.

The result of this comparison is synthesized in Figure 3, where the value of the *comparative* index Kc,p_i is shown for each of the parameters P_i. In Figure 3, the value of $Kc,\overline{p_i}$ is also reported. It is obtained by averaging all the correlations corresponding to the six parameters, obtained by comparing the surveyed share $C_{VC_z,S}$ with the average, for each vulnerability class z, of assessed shares C_{VC_z,P_i}, according to the following relations.

$$\overline{Cvc_{z1}} = \frac{\sum_{i=1}^{6} Cvc_{z,p_i}}{6} \quad (2)$$

$$Kc,\overline{p_i} = \frac{\sum_{z=1}^{4} \left|\overline{Cvc_{z1}} - Cvc_{z,S}\right|}{4} \quad (3)$$

By examining the obtained results, it is clear that the best correlated parameter is the 'age,' the worst correlated are the 'position in the block' and the 'number of floors'. Nevertheless, the good result obtained considering the average correlation suggests not to neglect other parameters.

Summing up data from PLINIVS survey DB				
age class	A	B	C	D
"A" (before 1919)	61,7%	34,8%	13,1%	1,4%
"B" (1920 -> 1945)	17,9%	21,3%	13,1%	2,6%
"C" (1945-1961)	10,8%	29,7%	34,4%	18,0%
"D" (1962-1971)	2,1%	8,7%	23,5%	35,1%
"E" (1972-1981)	7,4%	4,7%	13,6%	19,3%
"F" (1982 ->)	0,1%	0,9%	2,3%	23,7%

ISTAT Census data for Census Section "XX"					
age class	buildings	A	B	C	D
"A" (before 1919)	15	= 15 x 61,7 %	= 15 x 34,8 %	= 15 x 13,3 %	= 15 x 1,4 %
"B" (1920 -> 1945)	25	= 25 x 17,9%	= 25 x 21,3%	= 25 x 13,1%	= 25 x 2,6%
"C" (1945-1961)	7	....	....	....	....
"D" (1962-1971)	35	....	....	....	....
"E" (1972-1981)	5	....	....	....	....
"F" (1982 ->)	0	....	....	....	....
	87	ed. A	ed. B	ed. C	ed. D

Figure 2. Example of the assignment of vulnerability classes distributions for Census zone, with reference to the single parameter 'age of buildings' (P_3).

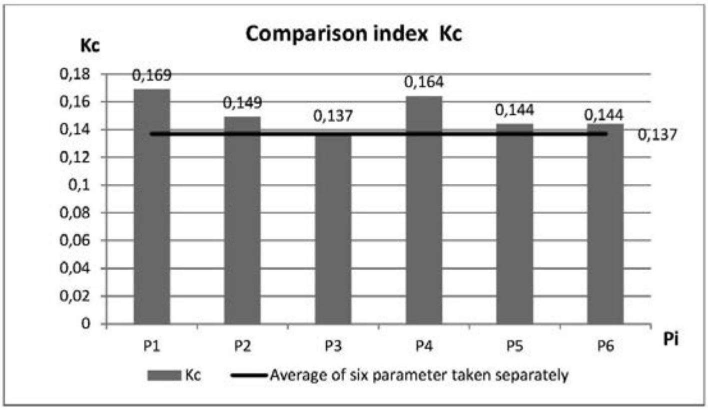


Figure 3. *Comparative* indexes Kc,p_i and $Kc,\overline{p_i}$.

5. Validation for combination of two and three parameters.

In this phase, buildings characterized by combinations of the two of six parameters in Table 3 have been considered. The number of possible combinations is 15 (Table 4).

For each of the identified combinations from the mentioned DB, the surveyed buildings with corresponding characteristics were extracted and the statistics of the vulnerability distributions were performed.
With reference to the set of 188 Municipalities, the vulnerability classes distributions in PSV - DB (step 1) are compared with the vulnerability classes distributions in SI - DB (step 3) with the aim to identify the most significant typological combination of parameters.
The error has been calculated by the following *comparative index* $K_{c,PiPj}$:

$$\mathrm{Kc}, p_i, p_j = \frac{\sum_{j=1}^{4} \left| C_{VC_z, p_i, p_j} - C_{VC_z, S} \right|}{4} \quad (4)$$

where: $C_{VC_z,S}$ is the percentage of buildings of vulnerability class VC_z (A, B, C, D), surveyed in the PSV-DB (step 1), C_{VC_z,P_iP_j} is the percentage of buildings of vulnerability class VC_z (A, B, C, D), assessed from SI-DB (step 3) applying the correlations found for the combination of P_i and P_j.
The result of this comparison is synthesized in FIGURE 4, where the value of the *comparative* index K_{c,P_iP_j} is shown for each combination of parameters P_i and P_j. In Figure 4, the value of $Kc, \overline{p_i p_j}$ is also reported. It is obtained by averaging all the correlations corresponding to the fifteen couple of parameters, obtained by comparing the surveyed share $C_{VC_z}S$ with the average, for each vulnerability class z, of assessed shares $C_{VC_z p_i p_j}$, according to the following relations.

$$\mathrm{Kc}, \overline{p_i, p_j} = \frac{\sum_{z=1}^{4} \left| \overline{C_{VC_{z2}}} - C_{VC_z, S} \right|}{4} \quad (5)$$

$$\overline{C_{VC_{z2}}} = \frac{\sum_{i=1}^{15} C_{VC_z, p_i, p_j}}{15} \quad (6)$$

By examining the obtained results, it is clear that the best correlated combination is ‘A’ (age + demographic class).
Anyway the results obtained using a combination of two parameters is better than the previous result reached with a single parameter.
The approach here illustrated has been repeated for combinations of three parameters (Table 5). The total number of the combination is 20.
The results, reported in Figure 6 , show a further slight improvement with respect to the combinations of two parameters. In particular, the best performance is still given by the average of the combinations ($Kc, \overline{p_i, p_j, p_k}$ = 0.1291). This result can be adopted as the final result, since the improvement observed in the transition from two to three parameters is not large, so it is conceivable that the implementation of combinations of four parameters cannot further improve the precision, moreover the “fragmentation” of the data-base on four parameters would give rise to place a large number of classes without buildings or with a number of buildings not statistically significant.

The performance rating of the Municipalities has been examined, finding three categories in function of the *comparative* index (Figure 8). As may be observed, two-thirds of the towns show an average error in the estimation of the vulnerability class less or equal to 15 %, almost one-third shows an average error between 15% and 25% and only few show an error of over 25%. A good compliance between the observed data and assessed data is evident, which supports the reliability of the method.

Table 4. Combinations two parameters (out of six) from ISTAT database.

COMBINATION CLASSES	FIRST PARAMETER P_i	SECOND PARAMETER P_k
A = P3+P6	Age (P3)	Demographic class (P6)
B = P3+P5	Age (P3)	Altimetry (P5)
C = P3+P2	Age (P3)	Vertical structure (P2)
D = P3+P4	Age (P3)	Number of floors (P4)
E = P3+P1	Age (P3)	Position in the block (P1)
F = P6+P5	Demographic class (P6)	Altimetry (P5)
G = P6+P2	Demographic class (P6)	Vertical structure (P2)
H = P6+P4	Demographic class (P6)	Number of floors (P4)
I = P6+P1	Demographic class (P6)	Position in the block (P1)
L = P5+P5	Altimetry (P5)	Vertical structure (P2)
M = P5+P4	Altimetry (P5)	Number of floors (P4)
N = P5+P1	Altimetry (P5)	Position in the block (P1)
O = P2+P4	Vertical structure (P2)	Number of floors (P4)
P = P2+P1	Vertical structure (P2)	Position in the block (P1)
Q = P4+P1	Number of floors (P4)	Position in the block (P1)

Summing up data from PLINIVS survey DB

age class	Nr. of floors	A	B	C	D	tot	perc. A	perc. B	perc. C	perc. D
-> 1919	1-2	35846	12751	3038	337	51972	69,0%	24,5%	5,8%	0,6%
1919-1945	1-2	12472	10035	3991	647	27145	45,9%	37,0%	14,7%	2,4%
1945-1961	1-2	7824	15078	11121	5047	39070	20,0%	38,6%	28,5%	12,9%
1962-1971	1-2	1320	4375	8690	8605	22990	5,7%	19,0%	37,8%	37,4%
1972-1981	1-2	5319	2196	4382	3492	15389	34,6%	14,3%	28,5%	22,7%
1982 ->	1-2	44	366	858	4471	5739	0,8%	6,4%	15,0%	77,9%
-> 1919	3-4	18171	9465	3910	378	31924	56,9%	29,6%	12,2%	1,2%
1919-1945	3-4	3209	3731	2903	616	10459	30,7%	35,7%	27,8%	5,9%
1945-1961	3-4	1699	4192	6177	3391	15459	11,0%	27,1%	40,0%	21,9%
......	......	......	......	......	......	......	......	......	......	.....
......	......	......	......	......	......	......	......	......	......	.....

age class	Nr. of floors	ISTAT Census data for Census Section "XX"				
		buildings	A	B	C	D
-> 1919	1-2	75	= 75 x 69 %	= 75 x 24,5%	= 15 x 5,8 %	= 15 x 0,4 %
1919-1945	1-2	97	= 97x 45,9%	= 97 x 37%	= 97 x 14,7%	= 97 x 2,4%
1945-1961	1-2	23	= 23 x 20%	= 23 x 38,6 %	= 23 x 28,5%	= 23 x 12,9%
1962-1971	1-2	84	= 84 x 5,7%	= 84 x 19 %	= 84 x37,8%	= 84 x 37,4%
1972-1981	1-2	5	....	....	....	....
1982 ->	1-2	0	....	....	....	....
1919-1945	3-4	45	....	....	....	....
1945-1961	3-4	76	....	....	....	....
......	......	......	....	....	....	....
		Σ	Σ	Σ	Σ	Σ

Figure 4. Example of the assignment of vulnerability classes distributions for Census zone, with reference to the combination of the parameters 'age of buildings' (P_3) and 'number of floors' (P_4).

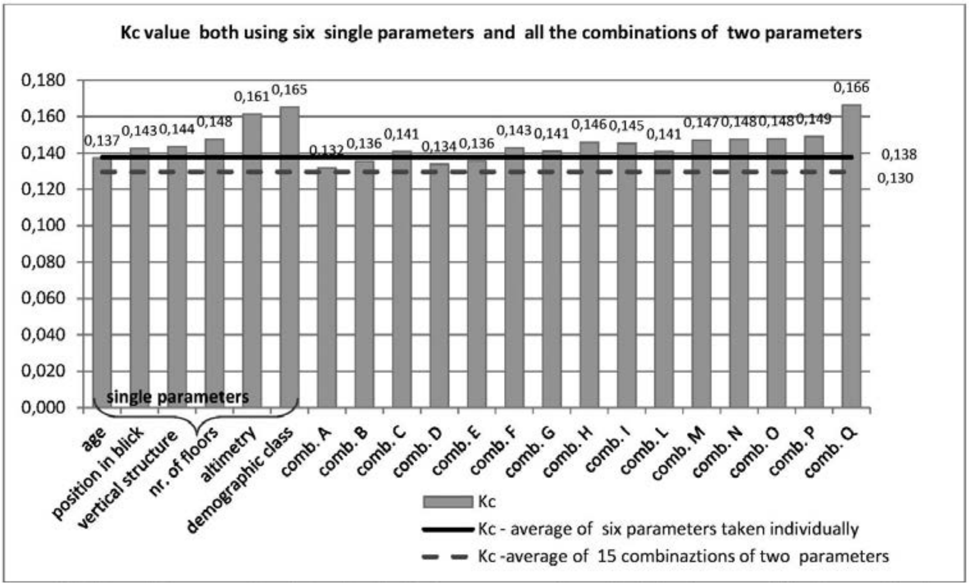


Figure 5. Values of *comparative* indexes for single parameter and combination of two parameters (out of six) from ISTAT database.

Table 5. Combinations of three among six parameters in the ISTAT database.

CLASS	I PARAMETER	II PARAMETER	III PARAMETER
3A	Age (P3)	Demographic class (P6)	Altimetry (P5)
3B	Age (P3)	Demographic class (P6)	Vertical Structure (P2)
3C	Age (P3)	Demographic class (P6)	Number of floors (P4)
3D	Age (P3)	Demographic class (P6)	Position in the block (P1)
3E	Age (P3)	Altimetry (P5)	Vertical Structure (P2)
3F	Age (P3)	Altimetry (P5)	Number of floors (P4)
3G	Age (P3)	Altimetry (P5)	Position in the block (P1)
3H	Age (P3)	Vertical Structure (P2)	Number of floors (P4)
3I	Age (P3)	Vertical Structure (P2)	Position in the block (P1)
3L	Age (P3)	Number of floors (P4)	Position in the block (P1)
3M	Demographic class (P6)	Altimetry (P5)	Vertical Structure (P2)
3N	Demographic class (P6)	Altimetry (P5)	Number of floors (P4)
3O	Demographic class (P6)	Altimetry (P5)	Position in the block (P1)
3P	Demographic class (P6)	Vertical Structure (P2)	Number of floors (P4)
3Q	Demographic class (P6)	Vertical Structure (P2)	Position in the block (P1)
3R	Demographic class (P6)	Number of floors (P4)	Position in the block (P1)
3S	Altimetry (P5)	Vertical Structure (P2)	Number of floors (P4)
3T	Altimetry (P5)	Vertical Structure (P2)	Position in the block (P1)
3U	Altimetry (P5)	Number of floors (P4)	Position in the block (P1)
3V	Vertical Structure (P2)	Number of floors (P4)	Position in the block (P1)

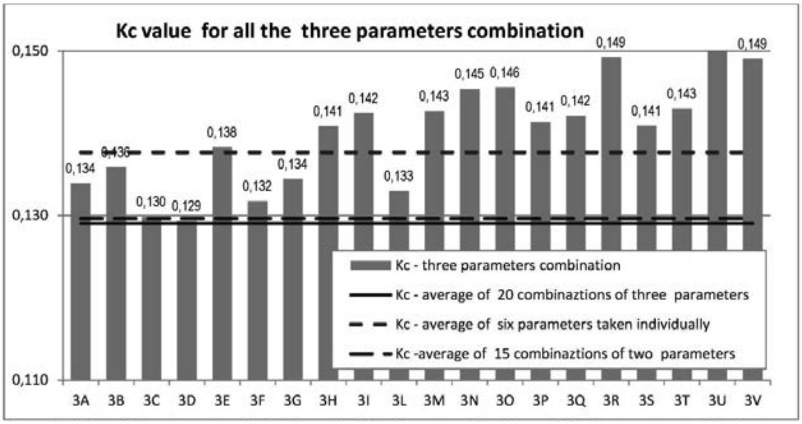


Figure 6. Values of *comparative* indexes for combination of three parameters.

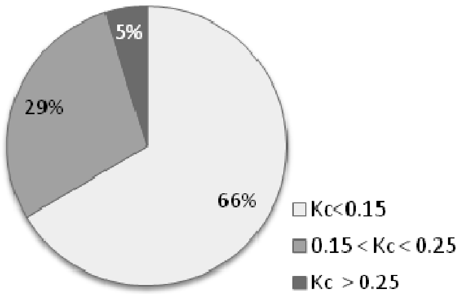


Figure 8. Classification of Municipalities as a function of Kc for the average combination.

Results

The method presented provides the buildings inventory and vulnerability classes distribution to develop quick seismic risk analysis or damage impact scenarios, with a level of reliability when detailed exposure and reasonable vulnerability data are not available. The vulnerability distribution is assessed by linking it with a little number of typological features available inISTAT Database. The search of the best correlation is made by comparing the assessed vulnerability with a large sample of buildings whose vulnerability is known from previous surveys.

The final result provides an easy-to-use assessment method, easily applicable to large regions. On the other side, the method is sufficiently reliable for the purposes related to territorial Risk and Scenario analysis. A further verification of the method may be made by using the detailed shaking maps and the damage data observed on occasion of the recent L'Aquila earthquake. This method has been applied in a first phase only for a restricted number of municipalities for which the disaggregated ISTAT data were available, just in order to calibrate the correlation parameters. Later on, it has been possible to apply the method widespread, over the entire Italian national territory, by means of an elaboration carried out at the Department of Civil Protection where the complete disaggregated DB is available. The application of the procedure allowed to draw up the vulnerability and risk maps, updated with the ISTAT 2001 data, (two of these maps are shown in Figure 9), which may represent the most advanced tool for the planning and mitigation activities of Seismic Risk at National Scale.

The identified methodologies are unavoidably affected by uncertainties. However, it still represents today a reasonable compromise between the necessity to cut down on-site surveys to a limited number of buildings and that of obtaining a final result sufficiently reliable for planning purposes, prevention, etc. The obtained results can be used, in combination with hazard maps, to easily develop risk or Scenario maps at Regional or National scale.

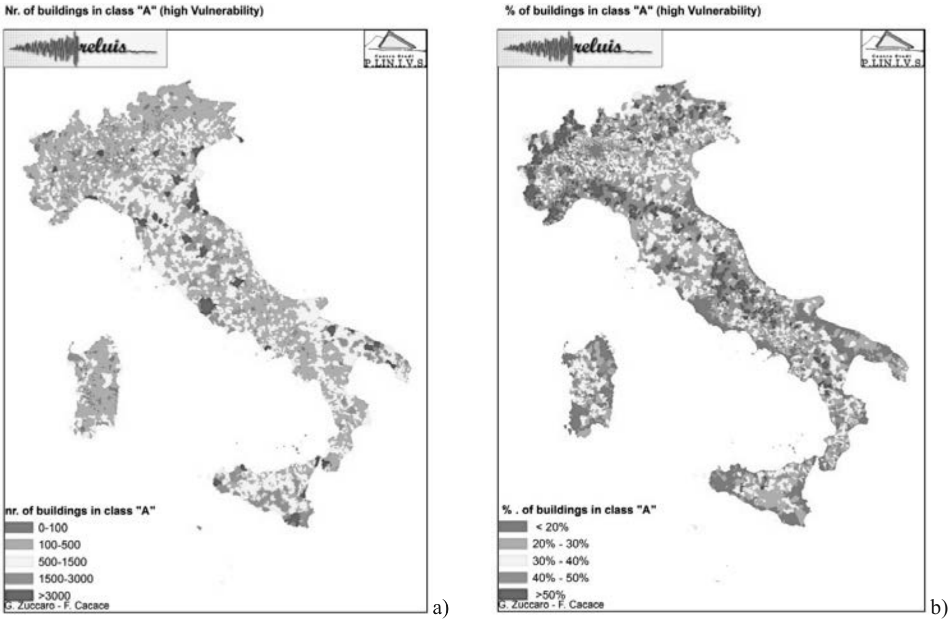


Figure 9. a) Number of buildings with high vulnerability (class A) by municipality. b) Percentage of buildings with high vulnerability (class A) by municipality.

3.2 Seismic vulnerability assessment for masonry buildings based on occurrence probabilities of main collapse mechanisms

General issues

The aim of this part of the research is to evaluate, in quantitative terms, the correlations between the typological-structural characteristics and the main failure mechanisms potentially triggered, by a seismic action on masonry buildings.

By using a Monte-Carlo simulation model, a large number of virtual buildings is generated, varying the relevant typological features deduced from PLINIVS - DB. For each of them, the main failure mechanisms are analyzed, evaluating the seismic response and its variation for different type of buildings.

By grouping the buildings according to the typological vulnerability classes, it is possible to assess the mean seismic response for each class. The result allows to build the vulnerability curves for typological classes as a function of the triggering probabilities of the collapse mechanisms.

The following paragraphs describe in detail the assumptions and methodological choices and the calculation procedures adopted in the construction of the simulation model, by considering separately the two aspects that characterize the problem: the automatic generation of building models and the evaluation of the seismic vulnerability.

<u>*Statistical analysis of most frequent collapse mechanisms observed in masonry buildings*</u>

Seismic damage of masonry buildings can be studied identifying the collapse mechanisms triggered on buildings. This analysis provides useful information in order to define the criteria for vulnerability assessment. The activation of each Collapse Mechanisms is strictly related to one or more structural weakness, defined 'typological vulnerability factors', which can be associated with the triggering probabilities of main collapse mechanisms. In this work, the correlations between typological characteristics and collapse mechanisms suggested in MEDEA methodology (Zuccaro and Papa, 2004-2007) can be adopted.

For masonry buildings, mechanisms are grouped into three categories: in-plan, out-of -plane and local mechanisms.

'Local' mechanisms are characterized by effect localized in a small part of the building, such as the breaking of chimneys or lintels. Overturning of walls, although limited to a single level of the building, involves the stability of the floors and affect the behaviour of a large part of the building, so the authors don't consider it as 'local'.

Since the contribution of local mechanisms (as just defined) to global building damage is generally lower, this study only considers the global mechanisms (in-plane and out-of-plane), that involve the structures as a whole.

The two behaviors can be described briefly as follows.

- *In-plane mechanisms* occur when the walls are affected by forces, in both directions, acting in their plane. The walls show the classical "X" damage, due to the diagonal tensile- compressive stress.

  These mechanisms are due to low tensile strength of masonry, but generally it demonstrates a good box-like behaviour of the building, under which the horizontal forces are correctly distributed on the walls placed along the direction of seismic action.
- *Out-of-plane mechanisms* occur when a rotational mechanism outside the plane of one or more walls of the building that loses its original configuration. The occurrence of such a mechanism is often caused by ineffective connection between the walls and between walls and floors, and by the push-action of floors and roofs. The mechanisms out-of-plane is usually related with high levels of overall damage, and even in case of slight damage, the incipient mechanism affects the safeness of the building in absence of temporary structures.

The two types of mechanism express two different behaviours, nevertheless under the action of the earthquake often they are both found on the same building. This assertion is confirmed through the analysis of the seismic damage data-base collected by the PLINIVS Research Center of the University of Naples Federico II. The data-base includes about 520 buildings surveyed using 'MEDEA' form.

Figure 10 shows the occurrence of collapse mechanism on buildings grouped by damage level. Figure 11 shows the combination of coexistence of the two type of collapse mechanism in buildings grouped by damage level and masonry quality.

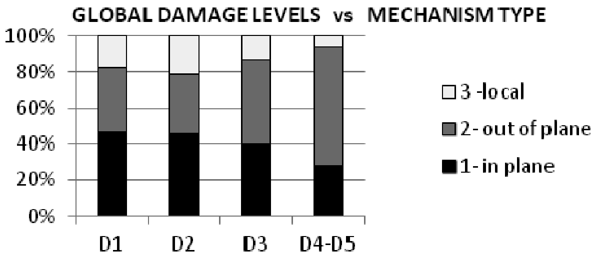


Figure 10. Percentage of occurrence of collapse mechanism type.

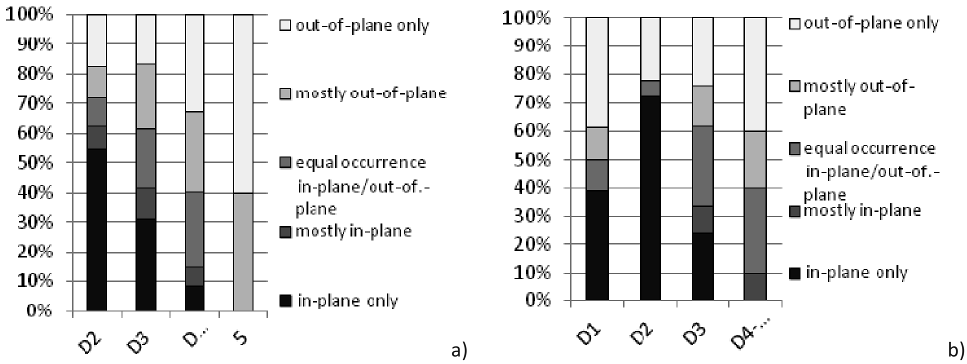


Figure 11. Prevalent mechanism type VS damage level for different quality of masonry: a) low; b) high.

Disregarding the buildings in which only in-plane or only out-of-plane mechanisms are present, and carefully examining merely the buildings presenting both typologies of mechanism, one may theorize that the in-plane mechanism occurs first, and the out-of-plane occurs only later when, as a result of the first mechanism, the effective connections between the wall panels become ineffective. This is confirmed observing that in many buildings the out-of-plane rotation occurs along the diagonal line of the crack caused by in-plane mechanism, which becomes cylindrical hinge for the overturning mechanism. Two examples are shown in Figure 12.

Figure 12. Tilting of the wall along shear-lesions diagonal direction.

Virtual buildings iterative generation

The Monte-Carlo computational procedure provides to generate virtual buildings. The geometrical, typological and mechanical parameters characterizing each virtual building randomly 'extracted' are the following.

a. Vertical structure typology.

Five typologies of masonry were considered:

- rubble stone masonry,
- squared stone masonry(dimension stone),
- tuff stone masonry,
- solid brick masonry,
- perforated brick masonry.

The parameters characterizing the type of masonry are the specific weight and the strength characteristics to compression and shear.

b. Horizontal structure typology.

Four typologies of slabs were considered:

- wooden beams slab,
- steel beams slab,
- R.C. with perforated bricks slab,
- vault.

c. Roof typology

The considered roof typologies are the same 4 typologies of the intermediate slabs, although the possibility that the top slab (roof) be "random extracted" differently from the intermediate ones is contemplated.

d. Roof geometry (pitched roof)

The eventuality that the building be topped by a pitched roof is considered as an on / off variable.

e. Number of floors

Buildings with a maximum number of 5 floors are considered. Basement floors are not considered.

f. Inter-storey height

The height of the inter-storey is considered to be ranging between 3 and 5 m.

g. Horizontal linkages

The presence of ties or others devices anchoring the panels to the orthogonal walls is considered as an on/off variable.

h. Wall thickness

The thickness of the ground-floor panel is a discrete random to be ranging, between 30 and 70 cm. If the horizontal structures are vaults, the thickness is increased by 10 cm. No matter what is the thickness "extracted" at the ground floor, in the building with more than two floors, two tapers are foreseen, in correspondence to the third and fifth levels.

i. Wall length
The entire length of the panel (which is intended as the distance between the walls of orthogonal thrust) varies between 3 and 7 m.

j. Openings percentage
A random variable is considered, which determines the entire percentage of the openings. Once extracted this value, the program automatically computes the number of openings in function of the entire surface of the panel. The resistance of the part of the wall under the windows is disregarded since it is usually thinner than the wall. The openings are considered aligned on all levels. The dimensions of the wall panels are generated randomly within given limits of proportion with the openings and the entire dimension of the panel.

k. Effectiveness of links between the walls
A random variable is introduced, which expresses the effectiveness of the wall joints.

l. Direction of single-way slab beams
For each of level, the possibility that the slab beams be structured in parallel or orthogonally respect to the panel is considered. This circumstance has influence on the vertical load transmitted to the wall panel and on the effectiveness of the wall/slab connection.

The models of ‘virtual’ buildings are generated following specific criteria. In fact, it is necessary that the simulated conditions be as representative as possible of real buildings. Therefore the probability distributions of random variables that parameterize the typological characteristics of randomly generated buildings must be accurately calibrated in order to reproduce combinations of coherent parameters having features recurrent in the building stock of the towns. In order to obtain this coherence, a wide statistical analysis of the typologies of existent masonry buildings has been conducted, using the PLINIVS Study Centre database (about 130,000 buildings, belonging to about 300 communities located in different regions of Italy). The data derive from survey campaigns effected in different periods with diverse techniques, therefore the modality of said surveys and the completeness of the available information are not homogeneous; however, this database represents a precious source of information on the typological characteristics of current buildings, and analyzing the contained data, it has been possible to define, not only the probability distributions of the random variables which determine the construction of virtual models, but also the joint probabilities of combinations of characteristics.
For example, once the vertical structure typology is extracted, main random variable, (sampled with an uniform distributed probability), the probability distribution of other random variables, i.e. the ‘number of floors’, results to be dependent from the wall typology, considered case by case. In the same way, almost all the secondary random variables are sampled considering distributions dependent on the combination of the principal ones, according to a specific priority order, as schematized in Figure 3.4. Frame direction and wall length are extracted as independent variables, as no robust correlation with other variables has been deducted from the available data. The statistical dependencies have been deduced from the analysis of the previously mentioned PLINIVS Study Center database. The ‘virtual’ buildings thus generated result being representative of real situations, with the further advantage of being classifiable according to the criteria foreseen by the ‘SAVE’ procedure for the assessment of the vulnerability classes. This aspect is particularly important for the purposes of this work.

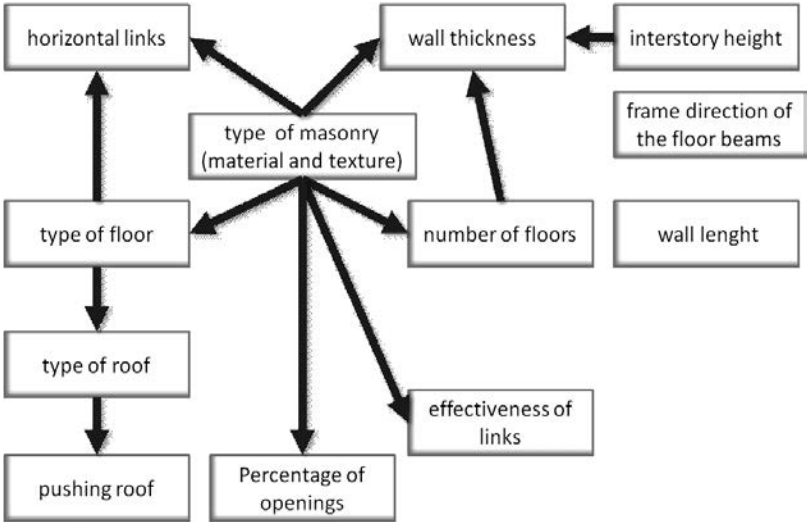


Figure 13. Hierarchical relationships of statistical dependency between the variables of the model.

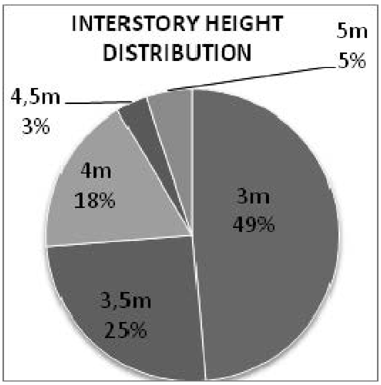


Figure 14. Detected rate of buildings with given inter-storey height.

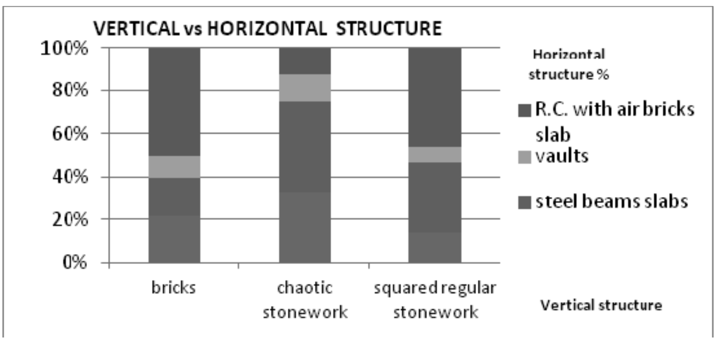


Figure 15. Detected correlation between vertical and horizontal structure.

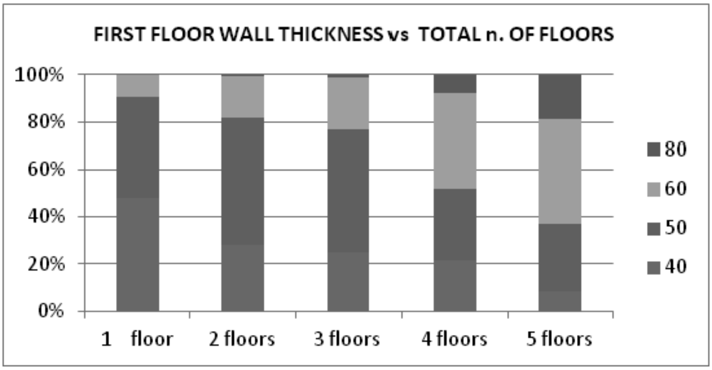


Figure 16. Detected correlation between number of floors and wall thickness at first floor.

Actually, it must be clarified that only the facade wall object of calculation and the orthogonal walls connected to it are generated, not whole building. On the other hand the automatic procedure generates panels of variable geometry which cover all the most frequent building typology cases. Furthermore, it must be considered that the accomplished analysis is comparative, and the significance of the results lies in the interpretation of the differences in performance among the building classes. In this view, the only parameter which has not been considered, is the increment of stress due to plan irregularities, which could in any case be introduced "artificially" as amplifying coefficient of the load randomly generated. However, this option should be supported by a deep analysis, both statistical and numerical, which may be taken into account in future improvements of the procedure.

Computation

Even from the computational point of view the two types of mechanism require two different kinds of structural analysis. In fact the in-plane mechanisms require an elastic-plastic collapse analysis, while the out-of-plane mechanisms are usually calculated with a simple kinematic analysis.

Two computation methods used are reported below, with the algorithms implemented in the model.

In-plane mechanism

The in-plane mechanisms are calculated adopting a macro-elements methodology to evaluate the horizontal collapse load for the entire wall (Giuffrè, 1991; Augenti, 2004)

Three potential mechanisms were considered:

- failure by bending moment in-plane of the wall (Figure 17a);
- failure by sliding due to shear in-plane of the wall (Figure 17b);
- failure by tensile stress due to shear in-plane of the wall (Figure 17c).

For example, the collapse load calculation method is described below for the first of these.

Normal stress distribution is a constant function, its value is equal to the ultimate compressive strength ($\sigma_{max} = \sigma_k$). Normal stress does not apply to the whole cross-section as the neutral axis is internal (Figure 18); since the material has reached the yield limit, a compressed band (not reversible strut) is generated inside the panel, and the wall cannot withstand any further load.

The ultimate bending failure load for walls unconstrained at the top (Figure 18) is:

$$T_u = \frac{N_{\max}}{H}\left[\frac{1}{2}\left(B - \frac{N_{\max}}{\sigma_k \cdot S}\right)\right] \tag{7}$$

The ultimate bending failure load for walls constrained at the top is:

$$T_u = \frac{N_{\max}}{2 \cdot H}\left[\frac{1}{2}\left(B - \frac{N_{\max}}{\sigma_k \cdot S}\right)\right] \tag{8}$$

These expressions define the elastic- plastic limit domain, and they allow to calculate the PGA level corresponding to heavy damage. After that *Tu* is determined, the spectral acceleration a^* can be derived as follows:

$$a^* = (T_u/W) \cdot g \tag{9}$$

where W is the total masses involved.
The horizontal seismic force assigned to each masonry pier is proportional to its stiffness. The collapse acceleration for to the entire wall is the value that causes the collapse in the weakest pier of the wall.
It should be noted that this value does not correspond to the total collapse of the wall. The collapse stress is computed in a simplified way applying the ductility factor as suggested by Italian Building Code (2008) for linear static analysis.

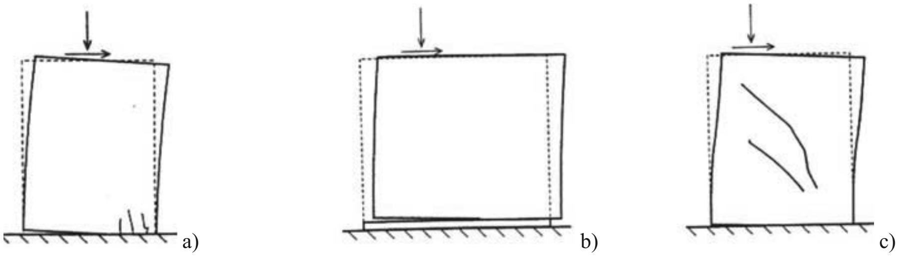


Figure 17. In-plane mechanism considered: bending (a), sliding shear (b) and tensile shear (c).

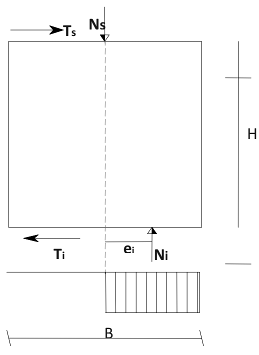


Figure 18. Ultimate limit state.

Simple Overturning Mechanism

Out-of-plane mechanisms calculation is made using kinematic analysis according to the explicative document of the National Italian Code (Circolare C.S.LL.PP n. 617/2009).

The wall is schematized as a mechanical system of rigid bodies that acquires a degree of freedom due to the activation of the mechanism. Under a given virtual rotation θ_k to the generic block *k*, it is possible to determine all the virtual displacements of the system as a function of θ_k and according to the geometry of the structure.

The actions applied to the elements of system are:

- weight of blocks, applied in their centre of gravity;
- vertical loads (dead and live loads of the floors and roof, other masonry elements not considered in the structural model);
- any internal forces (e.g. actions related to the connection between the parts of the wall);
- any external forces (such as those transmitted by ties);
- a system of horizontal forces proportional to the masses of the system by a factor α, that is called "*load multiplier*".

The aim of kinematic analysis is the calculation of α_0, i.e. the value of the multiplier can cause the triggering of the mechanism. This value is defined "collapse multiplier", and is obtained by applying the Virtual Work principle in terms of displacements, equalizing the total work performed by the external and internal forces applied to the kinematic chain. The Italian Building Code suggests the formula:

$$\alpha_0\left[\sum_{i=1}^{n} P_i\delta_{i,x} + \sum_{j=1}^{n+m} P_j\delta_{j,x}\right] - \sum_{i=1}^{n} P_i\delta_{i,y} - \sum_{h=1}^{o} F_h\delta_h = L_{fi} \tag{10}$$

where:

n	number of all the gravitation loads applied to the system;
m	number of all the masses not directly applied on the system but participating in generating of horizontal forces during earthquake;
o	number of forces, not associated to masses, applied to the system;
P_i	generic weight load applied to the bodies of the system;
P_j	generic load, not directly applied on the system but participating in generating of horizontal forces during earthquake;
$\delta_{x,i}$	horizontal virtual displacement of the application point of the i-th load P_i;
$\delta_{x,j}$	horizontal virtual displacement of the application point of the j-th load P_j;
$\delta_{y,i}$	vertical virtual displacement of the application point of the i-th load P_i;
F_h	generic force applied to the system;
δ_h	virtual displacement of the application point of F_h;
L_{fi}	virtual work of internal forces (it is assumed: $L_{fi} = 0$).

The mass M * (mass involved in the mechanism) can be evaluated by considering the virtual displacements application points of forces associated to the mechanism, as a modal shape of vibration:

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{x,i}^2} \tag{11}$$

The spectral acceleration a* is obtained by multiplying α_0 for the gravity acceleration and dividing by the participating mass fraction in the mechanism. The triggering spectral acceleration for the mechanism is:

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^*} = \frac{\alpha_0 g}{e^*} \tag{12}$$

where: *g* is the gravity acceleration; *e** is the participating mass fraction.
The resistance and the displacement capacity related to the damage limit state corresponds to the spectral acceleration which causes the triggering of the mechanism.
The resistance and the displacement capacity related to the collapse limit state are evaluated using the simplified structure factor *q*:

$$\alpha_u = \alpha_0^* \cdot q \tag{13}$$

It is assumed $q = 2$.

The adopted calculation model for the mechanism consists of a rigid-body rotation around a cylindrical hinge placed at the base of the portion of the involved wall. This is activated by seismic actions out of plane and is favored by the absence of connections with the orthogonal panels and by the lack of links at the top of the kinematic chain such as curbs or ties (Figure 19).

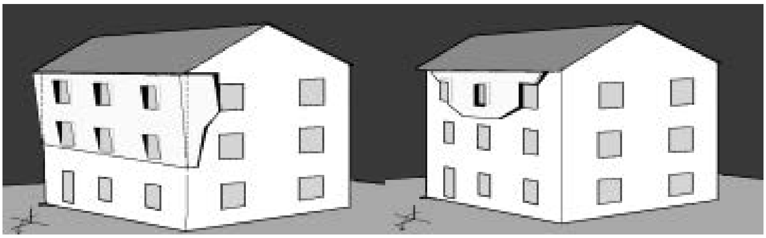


Figure 19. Overturning Mechanisms.

The overturning of the wall may involve one or more floors, according to the point in which, most probably, the cylindrical hinge is formed and involves more or less extended zones, depending on the presence of openings and on the distance of the orthogonal panels from the overturning one. The fundamental aspect for the analytical evaluation of the collapse multiplier is the individuation of the geometry of the overturning wall, from which it is possible to define a calculation scheme - possibly the most realistic model. If the building has already endured a seismic action, the overturning mechanism may be noted by the presence of vertical cracks at the intersection between the overturning wall and the orthogonal panels and by the unfastening of the slab or roof beams from the wall.

The simplest case is a monolithic panel (one floor building) that, due to the seismic action, could undergo an overturning, eventually favored by the presence of pitched roofs.

The calculation scheme chosen is derived from a widely adopted model (Sisma Marche, 1997). Nevertheless this model was modified by adding a stabilizing factor R_i which represents the resistance due to the connection between the walls (even if they are weak), which opposes the detachment of the panel from the orthogonal walls.

After that the geometry of the macro-element involved in the overturning mechanism is defined, all the loads acting on the panel are determined, along with its constrain condition. In this particular case, the system is considered to be hinged at the base, that is the point around which it rotates and presenting at the top a force which works against the overturning and represents the action induced on the panel by slabs or orthogonal ties. The loads acting on the panel are represented by the weights transmitted by the structures and superstructures acting on it, by static push and by horizontal forces due to the seismic event and calculated as the product of the value of the vertical action and the collapse multiplier α_0. After the restraining conditions and the loads acting on the system are set, it is possible to proceed to determine the moment of the forces which activate the overturning of the wall panel around the cylindrical hinge (*overturning moment*) and the moment of the forces that resist to said overturning (*stabilizing moment*). With reference to Figure 20, the rotational equilibrium equation around the cylindrical hinge situated in point A can be written, which allows to determine the collapse multiplier α_0. Therefore, the stabilizing moment and the overturning moment result to be respectively equal to:

$$M_S = W_1 \frac{S_1}{2} + F_{V1} x_{a1} + P_1 x_{p1} + N \frac{S_1}{2} + T_1 H_1 + R_i Y_{Ri} \tag{14}$$

$$M_R = \alpha_0 \left[W_1 y_{G1} + F_{V1} y_{a1} + P_1 H_1 + N H_1 \right] + F_{o1} y_{a1} + S_p H_1 \tag{15}$$

Equalizing the two terms, the collapse multiplier α_0 is obtained:

$$\begin{aligned} W_1 \frac{S_1}{2} + F_{V1} x_{a1} + P_1 x_{p1} + N \frac{S_1}{2} + T_1 H_1 + R_i Y_{Ri} = \alpha_0 \left[W_1 y_{G1} + F_{V1} y_{a1} + P_1 H_1 + N H_1 \right] + \\ + F_{o1} y_{a1} + S_p H_1 \end{aligned} \tag{16}$$

$$\alpha_0 = \frac{W_1 \frac{S_1}{2} + F_{V1} x_{a1} + P_1 x_{p1} + N \frac{S_1}{2} + T_1 H_1 + R_i Y_{Ri} - F_{o1} y_{a1} - S_p H_1}{W_1 y_{G1} + F_{V1} y_{a1} + P_1 H_1 + N H_1} \tag{17}$$

where:

W_i	specific weight of the panel at floor *i*;
F_{vi}	vertical component of the thrust of arches or vaults on the panel at floor *i*;
F_{oj}	horizontal component of the thrust of arches or vaults on the panel at floor *i*;
P_i	weight of the ceiling acting on the panel at floor *i*-th, calculated on the basis of involved area;
S_p	static thrust conveyed by the roof covering;
T_i	maximum value of the action of a possible tie beam present at the top of the panel at floor *i*;

N	general vertical loads acting from the top, presumed to be centred on the masonry;
S_i	thickness of the panel at floor *i*;
H_i	height of the panel at floor *i* respect to the pole;
y_{ai}	height of point of appliance of the thrust of the arches or vaults at floor *i* respect to the pole;
x_{ai}	horizontal distance of the point of appliance of the thrust of arches or vaults at floor *i* respect to the pole;
x_{pi}	horizontal distance of the point of appliance of the load of the ceiling on the panel at floor *i* respect to the pole;
y_{Gi}	height of the barycentre of the panel at floor *i* respect to the pole;
α_0	multiplier of the horizontal forces;
R_i	resistance offered by the wall connection with the orthogonal panels at floor *i*;
y_{Ri}	height of the point of appliance of the resistance R_i respect to the pole.

The model also calculates other possible mechanisms, such as overturning of multi-storey building facade and the vertical bending of the wall, the calculation schemes are similar each other, so they have not been reported for the sake of brevity.

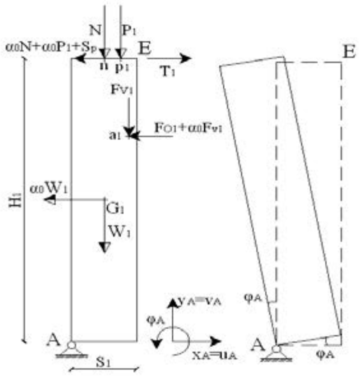


Figure 20. Simple overturning Mechanism of a monolithic panel.

<u>*Results*</u>

By using the iterative model described above a set of 100,000 buildings has been generated, whose typological and constructive characteristics are randomly defined at each iteration.

Each of the simulated buildings is assigned to a vulnerability class according to the criteria defined by the 'SAVE' first-level procedure.

The result of each iteration is:

1. the failure value a_d of the acceleration that leads to the damage limit state threshold for each of the implemented mechanism;
2. the failure value a_u of the acceleration that leads to the ultimate collapse limit state threshold for each of the implemented mechanism.

In this regard, it should be noted that for the purposes of the present work the results of single-pass calculation interested mainly in qualitative terms, and are statistically significant only if compared with all other iterations of the process.

By observing the calculation results, it is possible to determine the mechanism responsible for the failure at both limit states. Collecting the obtained results by typological vulnerability class, the diagrams in Figure 21 are obtained. Diagrams clearly show that, although the in-plane mechanisms are more frequent, the percentage of activation of out-of-plane mechanisms increases for higher vulnerability. This phenomenon is just less evident for the damage limit state. To investigate the behaviour of various typological vulnerability classes, for every class has been counted the number of activation of each mechanism, by varying the acceleration from 0.1g to 2.0 g. The analysis has been carried out taking into account both the mechanisms separately and grouped by type (in-plane and out-of-plane). The results of the analysis by type of mechanism are shown in Figure 22.

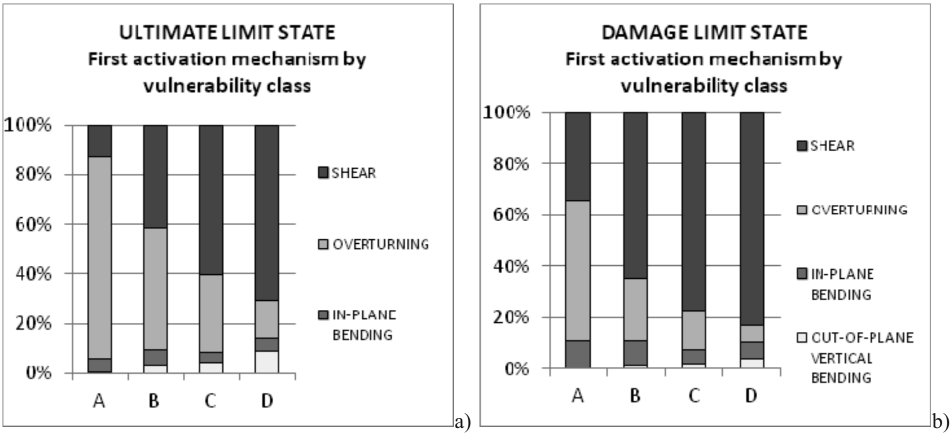


Figure 21. First mechanism which occurs for each typological class: a) Ultimate Limit State, b) Damage Limit State.

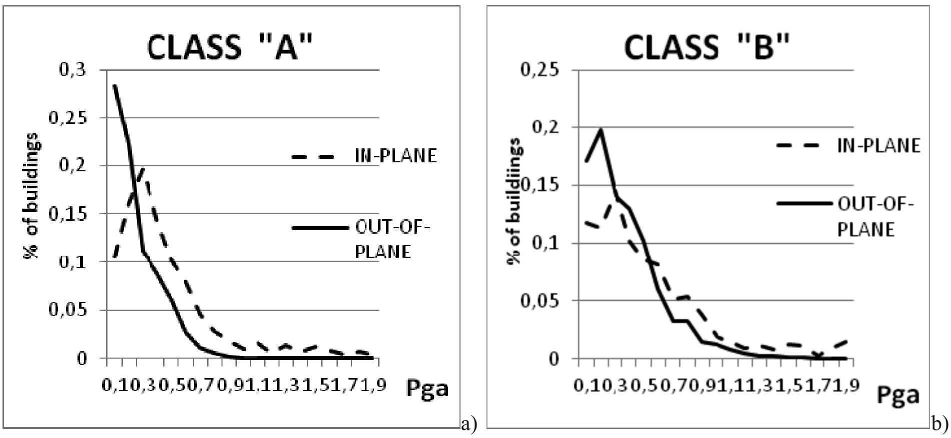


Figure 22. Percent of buildings in the data set by type of first activation mechanism varying seismic activity: typological classes 'A' (a) and 'B' (b).

On the other hand, if a building reaches the ultimate limit state for in-plane mechanism, it is reasonable to assume that its capacity to resist the occurrence of out-of-plane mechanism is seriously compromised, as it reduces the effectiveness of the connection of the façade with orthogonal walls and the floor.

For this reason the analysis was performed again for all the buildings that had reached the limit state by in-plane mechanism, but considering in the calculation the lack of resistance to out-of-plane actions due to the in-plane damage. This has been obtained by assuming the effectiveness of the links between the walls and the (R_i = 0) and the effectiveness of the connection with the vertical structure (T_i = 0) equal to zero.

With this assumption, the results was that the number of out-of-plane mechanisms is much higher also for medium or low values of the seismic intensity measure.

These results encourage to define a function of the damage expectation for given typological class function of the acceleration. In fact the global damage associated with the in-plane mechanism is generally low (also at the ultimate limit state of the mechanism), as confirmed by a large series of post-earthquake damage surveys, on the other hand, the mechanism out-of-plane is usually associated with a higher level of damage. By these observations, and taking the numerical results of the simulation, the vulnerability curves for different typological classes have been developed.

The vulnerability curves for the typological classes ‘A’ and ‘B’ are indicated in Figure 23. The work is still in progress and the results have to be considered as temporary, in fact some damage mechanisms are not yet considered and at the moment are being implemented in the model. Furthermore a calibration of the curves obtained is in progress, based on a comparison with post-earthquake observed damage.

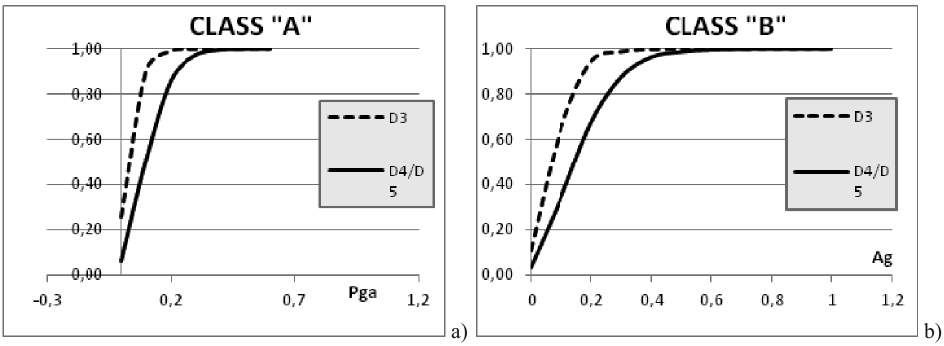


Figure 23. Vulnerability functions: classes ‘A’ (a) and ‘B’ (b).

It should be also considered that within each class there is a great variety of behavior depending also by factors which are not considered in ‘macro-seismic’ assessment such as the percentage of openings in the wall, the thickness, the inter-storey height.

As an example, Figure 24 shows the variation in performance of buildings in class ‘A’ and class ‘B’ as a function of the percentage of openings.

The results obtained confirm the underlying assumptions and are not discordant from the damage data actually occurred during recent earthquakes. Moreover, it is to possible compare the behaviour of the virtual buildings with the standard performance of real buildings vulnerability classes, defined by first level method.

The procedure proposed is an attempt to draw a new working methodology for vulnerability and risk assessment at regional scale. The methodology is still in progress, however the results obtained since now encourage pursuing this path.

Next improvement will be the implementation of further mechanisms, such as the bending out-of-plane or the vertical deflection involving more levels. The damage progression with a non-linear analysis should also be improved by setting a larger number of control points.

The procedure to generate building sample can also be improved by inserting additional virtual variables, mainly, for instance, to take into account the age or the regularity in-plane and elevation of the buildings.

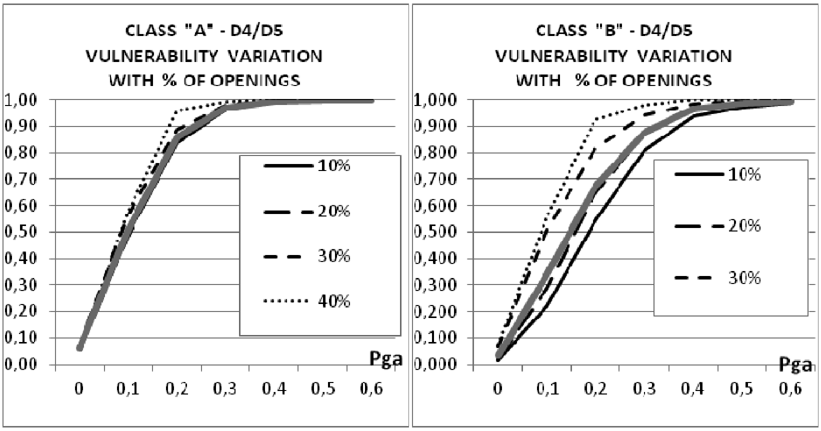


Figure 24. Variation of the vulnerability with the percentage of openings.

4 VISIONS AND DEVELOPMENTS

The output of this research have shown the feasibility of the goals listed above and fixed at the beginning of the project, reporting an important advancement on many of the issues under study.
The project has shown that:

- The seismic risk / impact evaluation at regional scale is at moment feasible using: statistical calibration of Census data with survey sample for exposure; and observational Damage Probability Matrix calibrated on damage surveys of the past events.
- It is possible to develop vulnerability functions of masonry buildings through simplified mechanical models, which may be calibrated by means of observed damage data and the first encouraging results have been presented in the project.
- Further research developments may address the use of vulnerability curves that relate the ground shaking expressed in terms of PGA with the expected damage level .
  This will also lead to a better relationship between the hazard assessment at regional scale and the vulnerability curves.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 501-526, doi: 10.14599/r101314, © 2015 Doppiavoce, Napoli, Italy

RECORD SELECTION AND SEISMIC INPUT DEFINITION FOR STRUCTURAL ANALYSIS

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1 INTRODUCTION

Since several years, seismic input definition is one of the hot topics of earthquake engineering because of its influence on simulations for estimating seismic structural performance. Herein, the efforts of the ReLUIS 2010-2013 project toward the development of practice-ready tools for hazard consistent seismic input definition aimed at seismic structural analysis is shown.

Determination of design seismic actions in seismic codes mostly relies on a target spectrum, which is, therefore, also the basis for record selection in seismic input definition when performing nonlinear structural analysis. Since a rational performance target should account for the seismic hazard at the site of interest, the uniform hazard spectrum (UHS), or an approximation of it, is often used as the design spectrum.[2] The UHS is built entering the elastic spectral acceleration, $S_a(T)$, hazard curves for several T values at a specified probability of exceedance of (e.g., 10% in 50 years or, equivalently, 475 years return period, T_r), and plotting the corresponding ordinates versus T.

Generally, the signals that can be used for structural simulation are of three types: (1) artificial waveforms; (2) simulated accelerograms; and (3) natural records. Signals of type (1) are often obtained via random vibration theory. Simulation records (2) are obtained via modelling of the seismological source and may account for path and site effects. Finally of type (3) are ground-motion records from real events (Bommer and Acevedo, 2004).

As far as it regards real records, given the UHS for the structural limit-state of interest (i.e., the UHS corresponding to a Tr), current or advanced (depending on the context where it is applied) practice today, which may require aid by a seismologist, would select a set of records reflecting the *likely* magnitudes (M), source-to-site distances (R), and other earthquake parameters thought to drive the probabilistic seismic hazard analysis (PSHA) for the site (McGuire, 2004), and which are believed to matter with respect to structural response (This information comes from a procedure called *disaggregation* of PSHA). Finally, the records are usually manipulated to match the UHS, individually or in average sense, at the period of the first mode of the structure (T^*), Figure 1, or in an interval around it (e.g., Iervolino and Cornell, 2005).

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[1] Many researchers contributed to the work described in this paper, all the authors of the paper in the ReLUIS reference list should be considered a co-authoring also this chapter.

[2] Although the use of UHS was only recently acknowledged by engineering practice and/or codes for design and assessment purposes (e.g., in Italy), some studies have already investigated the shortcomings of this kind of representation of ground motion and propose more sound alternative (see, e.g., Baker, 2011).

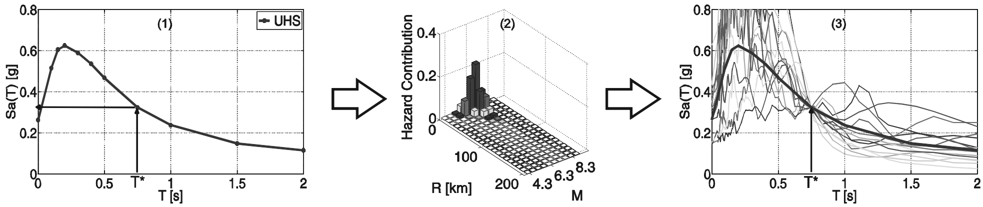

Figure 1. Steps to define seismic action according to the hazard at the site, from left to right: (1) UHS for the site and limit-state of interest; (2) hazard disaggregation for the spectral ordinate of interest (T*); (3) selection of a set of records compatible to disaggregation and matching the target spectrum at T*.

It has been discussed (e.g., Iervolino et al., 2008 and 2009) that international codes, at least in principle, may be seen as not very far from that approach. In fact, once the target spectrum has been defined, the main criterion is that the records have to match, or exceed it, in a range of periods. Codes often also require the selected records to reflect some characteristic of the relevant seismic sources (e.g., magnitude and distance) jointly with the design spectrum; this for example applies to ASCE Standard ASCE/SEI 7-10 (ASCE, 2010) and Eurocode 8 or EC8 (CEN, 2003). Therefore, the mainstream of the practice-related research on the topic deals, on seismological side, with linking the code approach to record selection to the probabilistic seismic hazard for the construction site and, on the structural side, with the characterization of the sample to capture the seismic response. In other words, to select samples, of different possible size, of real records matching arbitrary design spectra is the basic alternative which can be improved coupling spectrum compatibility with *disaggregation* (respecting distance-magnitude design range in record selection) or vector valued intensity measures (e.g., adding relation to ground motion duration besides the acceleration spectrum).
This motivated the part of the ReLUIS research dealing with real records, which lead to the inclusion in the last version (3.5) of REXEL (Iervolino et al., 2010a), the ReLUIS record selection software, of several additional features with respect to previous versions (see section 3). Moreover, as the databases of records are continuously improving with new events and improved processing, it may be worthwhile to have a REXEL-like code-related waveform selection tool operating directly on an online-repository of real waveforms. This sparked the development of REXELite (Iervolino et al., 2010g and 2011a) which has the REXEL search engine, but it operates on the portal of the Italian Accelerometric Archive or ITACA, the database of Italian seismic records of the *Istituto Nazionale di Geofisica e Vulcanologia* (INGV), which is also one of the main results of the project, as described in the following. One of the key features added to REXEL is the database of design earthquakes from disaggregation of probabilistic seismic hazard in terms of spectral ordinates for the whole Italy (Iervolino et al., 2011b), which allows a more hazard-constrained code-based record selection. The disaggregation results, are also available online, in form of a webgis, as an input for REXELite. Finally, to match the needs of recently proposed new paradigms of earthquake engineering such as the displacement-based seismic design and assessement (see the work by Calvi and Sullivan in this same book), one of the results of the project is also REXEL-DISP (Smerzini et al., 2014), which is the equivalent of REXEL, yet selection records based on the matching of a hazard-derived displacement elastic spectrum rather than an acceleration one.

As it regards the other options available to practitioners to define the seismic input to seismic structural analysis, that is artificial and simulated records, different issues motivated the research within the RELUS (2010-2013) project. The basic issue to be addressed is that artificial and simulated ground motions have to be proven equivalent to real records prior to be used as an alternative to them. In a context related to code-based practice, such an equivalence has to be measured in terms of structural response, and it should be evaluated provided that the artificial records at least match the same conditions (i.e., design spectrum) of an equivalent ideal ground motion set made of real records. This was the basis of the work in Iervolino et al. (2010b), where different types of artificial and real records matching the same design scenario, where compared in terms of structural response they induce.
As mentioned, simulated records also need validation, but in this case it can be carried out comparing the structural response of simulated records when the latter try to replicate historical seismic events (Galasso et al., 2012a-c and 2013). In this case the real-records benchmark is provided by the real ground motions from the recorded earthquake. Another possible form of validation of simulated records is to compare the non-linear response on simple single degree of freedom (SDOF) systems to what expected from a ground motion prediction equation (GMPE) developed in terms of nonlinear response as well, but based on real records (De Luca et al., 2011 and 2014). Finally, it is clear that, once validated, in a repository of simulated records is required for engineering practice in the long run. A prototype of such a database, perfectly analogous to those of real records, was developed within the project; indeed, it will be illustrated how the Synthesis database is intended to become a proof of concept for selection of scenario-based simulated records.

2 RESEARCH STRUCTURE

The research deployed on different lines, partly in continuation of the previous ReLUIS 2008-2010 project. Three main lines of action (tasks) were identified and pursued:

(1) practice-ready record selection software for seismic structural analysis;

(2) practice-ready tools for hazard-informed record selection;

(3) validation of simulated/artificial ground motion for seismic structural analysis;

(1) led to the further refinement of the already existing ReLUIS product REXEL as well as the extension of the REXEL family via the development of other record selection softwares with specific features (to follow). All the developed REXEL-based tools are currently available for practitioners to use. (2) basically concerned the identification, for Italy, of *design earthquakes* from probabilistic hazard disaggregation, that is the mapping of reference magnitude and source-to-site distance pairs, which control the seismic hazard at the return periods of interest to the construction code (CS.LL.PP., 2008) at any site in the country. Finally, (3) was related with a large effort, also developed in collaboration with international projects on the same topic, regarding engineering validation of artificial and simulated ground motions. As it regards artificial records, the validation was based on spectrum compatibility and in comparison to the nonlinear SDOF response to real records. As it regards simulated records, the result of this activity was twofold: (i) a series of studies of different kind to

assess, using as a benchmark nonlinear SDOF response GMPEs or MDOF response to some historical earthquakes, the possible bias induced by some ground motion simulation/generation techniques; (ii) the development of a prototypal online repository of simulated ground motions for engineering use.

The research structure reflects a coordinated and multi-disciplinary effort to get the deliverables. In fact, three research units mainly contributed to it. One belonging to the University of Naples Federico II (UNINA) and coordinated by the author of this article, one belonging to the Polytechnic of Milan (POLIMI) and coordinated by professor Roberto Paolucci, and finally one of the INGV (Milan section) coordinated by Dr. Francesca Pacor. These groups strongly collaborated and complemented each other. Indeed, UNINA took care of the REXEL and REXEL-related developments as well as the disaggregation study, POLIMI provided the database for the REXEL-DISP software as well as the displacement design spectra derived from long-period hazard assessment for Italy. The INGV contributed to the integrations of the developed tools (i.e., REXELite) in ITACA, as well as the validation of simulated ground motions and development of the Synthesis portal. Finally, the project benefitted of a fruitful collaboration with the *University of California at Irvine* and the *Southern California Earthquake Center* who had similar interests on the engineering validation of simulated ground motions.

3 MAIN RESULTS

3.1 REXEL 3.5

In a series of investigations, developed between 2005 and 2006, to assess the practicability of code provisions (with particular focus on the Eurocode 8), an algorithm was developed to analyze all possible combination of seven elastic spectra within a list, to find those having the average compatible with a target spectrum in a range of periods and with some upper- and lower-bound tolerance. That algorithm was employed to find sets compatible to EC8 spectral shapes, and to draw the conclusions depicted in the papers by Iervolino et al. (2008) and (2009). Subsequently, with the introduction in Italy of a new Building Code or NTC08 (CS.LL.PP., 2008), the algorithm at its second generation of development, was given of a graphic user interface (GUI), named REXEL 2.0 (beta), and released publicly at the RELUIS website.

The original procedure implemented for record selection deploys in four basic steps:

a) definition of the target horizontal and/or vertical spectra the set of records has to match on average; the spectra can be built based on some code provisions or may be arbitrary;

b) list and plot of the records contained in the database and embedded in REXEL which fall into the magnitude and distance bins specified by the user for a specific site class;

c) assigning the period range where the average spectrum of the set has to be compatible with the reference spectrum, and specification of tolerances in compatibility;

d) running the search for combinations of seven records which include one, two of all three components of motion and that, on average, match the design spectrum with parameters specified in step c; the records may be original (unscaled) or linearly scaled in amplitude.

One of the most important improvements of REXEL v 2+, Iervolino et al. (2009, 2010a, 2010d, 2010f), with respect to the first generation, was that the search algorithm was optimized to return, as fast as possible (i.e., within seconds), the combination with the smallest record-to-record variability with respect to the target spectrum. As it is well known, that large variability, which may result from such kind of search (e.g., Iervolino et al., 2008 and 2009), may affect significantly the confidence in the estimation of structural response if only seven records are used as an input for structural analysis. At that stage, the software enabled basically step 1 and 3 depicted in Figure 1, missing the link with the design earthquakes from disaggregation of hazard the design spectrum derives from.
This has been possible in the third generation of the software (v 3+). In fact, a comprehensive disaggregation study (for Italy so far; Iervolino et al., 2011b and 2012), now suggests M and R ranges consistent with the hazard for the design spectral ordinates of interest (i.e., step 2 of Figure); see also section 3.4. The same design earthquakes embedded may also be used, via the conditional hazard approach (Iervolino et al., 2010c and 2010e) also implemented, to select records matching the design spectrum and at the same time reflecting likely (in a probabilistic sense) other ground motions intensity measures; e.g., cyclic content of ground motion for duration sensitive structures.
REXEL v 3+ defines target spectra according to several international codes – (1) NTC08; (2) EC8 (Type 1 and Type 2 spectra); (3) ASCE/SEI 7-10 and (4) user-defined spectral shape – and the sets may be searched among three records databases embedded. The spectrum matching waveforms may be preliminarily selected, alternatively to M, R and *epsilon* (another, well known disaggregation result; Iervolino et al., 2011b) by bins of peak and integral ground motion intensity measures (IMs); i.e., step (c) above may be now also according to ground motion IMs and not only magnitude and distance. Finally, other options as displacement spectrum compatibility check, and repeating record sets' search excluding undesired records, further improve the selection.
In this section, a brief report of how the new features and enhancements of the last REXEL release (v 3.5) (Figure 2) allows an informed and practice-ready record selection, is given.

3.1.2. Embedded databases

REXEL had built in the records belonging to the European Strong Motion Database or ESD (last accessed July 2007 at http://www.isesd.hi.is/). Recently, Italian free-field records of earthquakes with M larger than 4 from ITACA (http://itaca.mi.ingv.it, updated to April 2011) and worldwide free-field records of earthquakes with M larger than 5 from the Selected Input Motions for displacement-Based Assessment and Design database, or SIMBAD, were also embedded. SIMBAD was developed in the framework of ReLUIS 2010-2013 Project, task *Displacement Based Approaches for Seismic Assessment of Structures*, as a strong ground motion database suitable for displacement-based design and assessment (Smerzini et al., 2012 and 2014). It contains at more than 400 three-component accelerograms from earthquakes worldwide (Figure 3).

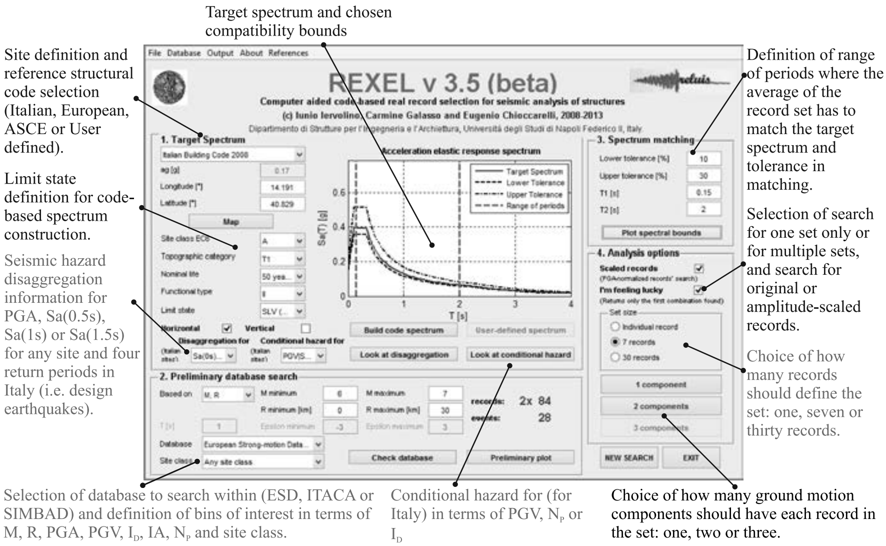

Figure 2. REXEL 3.5 user interface and main functions: new features and enhancements are reported in red.

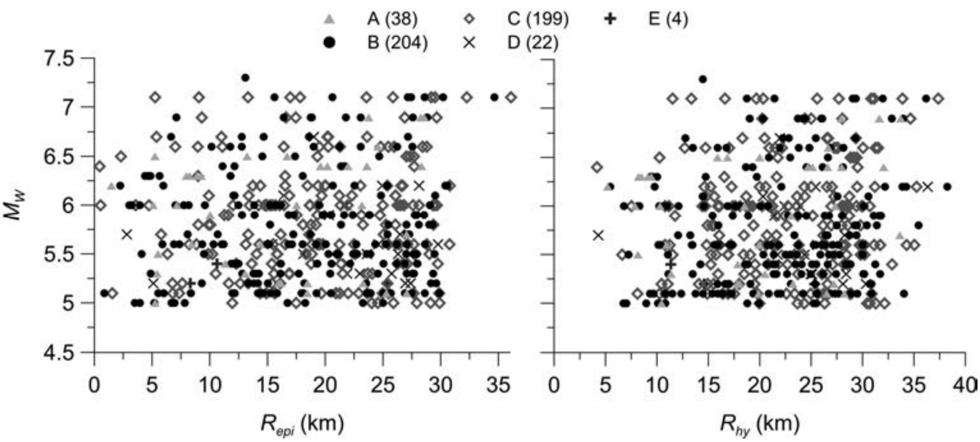

Figure 3. Distribution of Magnitude (M_W), epicentral (R_{epi}) or hypocentral (R_{hy}) distance and EC8 site class of the records included in the SIMBAD database.

3.1.3. Preliminary search parameters

To make sure the selected set of spectrum matching records has the desirable characteristic, the user can limit the search to waveforms falling in specific M and R bins or M, R and *epsilon* bins. If the spectrum is hazard-based, as it happens in Italy where the code assigns design spectra which are basically UHS, the selection of such bins can follow disaggregation of seismic hazard for the region of the target spectrum of interest (to follow). Alternatively, REXEL 3.5 introduces pre-selection in bins from a specific range of a selected IM. More specifically, the user may select the records in the databases corresponding to a given range horizontal peak ground acceleration (PGA), peak ground velocity (PGV), the *Cosenza and*

Manfredi index (I_D) (selection according to this parameter may be guided by conditional hazard, see section 3.1.5), *Arias Intensity* (I_A), and, finally, *Np* (Bojorquez and Iervolino, 2011).

A parameter that is desirable to include in record selection is the site classification. However, specifying a close match for this parameter in record selection may not always be feasible, because for some soft soils only a few records may be available. Moreover, if the spectral shape is assigned by the code, the site class of real records may be of secondary importance (e.g., Galasso and Iervolino, 2011). In light of these considerations, there may be cases in which it may be useful to relax the matching criteria for site classification. Therefore, in REXEL 3.5 beta it is now possible to select records from *same as target spectrum* soil or from *any site class*. The latter option, as shown in the following, could help to find spectrum matching sets when insufficient records are available for a specific site condition.

After M-R (or IM) bins and site classification are defined, the software returns the number of records, and the corresponding number of originating earthquake events, available in the intervals. This list constitutes the inventory of records in which to search for sets compatible, via their average, with the target spectrum. Spectra of records from preliminary database search may also be plotted along with the target spectrum (i.e., the *preliminary plot* option) to have a picture of the spectra REXEL will search among. This, in most of the cases, enables to immediately understand if the search for spectrum matching sets will be successful.

3.1.4. Design earthquakes from disaggregation

When the design spectrum is derived from PSHA (e.g., a UHS as it happens in the Italian code), the *disaggregation* procedure allows to identify, from a probabilistic point of view, the contribution to the hazard of each source (in terms, for example, of M, R and *epsilon*). In particular, *epsilon* (ε) is defined as the number of standard deviation by which the logarithmic ground motion (in terms of spectral ordinates) departs from the median predicted by the chosen attenuation relationship; Ambraseys et al. (1996) for this specific study. Such an information can address the identification of scenarios relevant for design; i.e., the *design earthquakes* which are dependent on the spectral ordinate of interest and on return period the spectrum refers to. In fact, as briefly reviewed in the introduction, most of the codes requires to account in record selection for the features of the seismic sources of interest for hazard at the site, and this may be rationally referred to disaggregation.

To address this issue, a comprehensive disaggregation analysis for the pseudo-accelerations hazard (in terms of four return periods (Tr): 50, 475, 975, and 2475 yr) has been carried out for more than 10000 Italian sites considering and four spectral periods: PGA, $S_a(0.5s)$, $S_a(1s)$, and $S_a(1.5s)$. The first half of IMs are considered representative for the short period portion of the response spectrum, while the latter half has been found adequate for moderate-to-long period structures; details may be found in section 3.4 as well as in Chioccarelli (2010) and Iervolino et al. (2011b). Results of the analyses are implemented in REXEL which now provides, for each site, together with the design spectrum, a plot of disaggregation distribution in terms of magnitude and source-to-site distance corresponding to the closer return period and structural period with respect to those of interest. This is to guide the definition of M and R bins in which to find spectrum matching sets; i.e., linking all steps of Figure 1.

3.1.5. I_D and PGV conditional hazard

When seismic analysis of structures sensitive to cyclic content of ground motion or to PGV is concerned, to match the design spectrum may be insufficient to properly characterize the set of records used as an input (e.g., Iervolino et al., 2006).

A way to account for multiple IMs (acceleration and one related to duration or PGV in this case) at the same time, is to compute the so-called vector-valued hazard analysis for the site

(e.g., Bazzurro and Cornell, 2002). Although computationally demanding, this analysis allows to compute the joint hazard for more than one IM. However, if in a vector of two, one IM is seen most important with respect to another, an alternative, easy yet hazard-consistent way of including secondary intensity measures in record selection is represented by the *conditional hazard* (Iervolino et al., 2010c and Chioccarelli et al., 2012). This consists of computing hazard curves of the secondary IM conditional to a specific value of the primary IM.
Conditional hazard is especially appropriate if the primary IM is the design acceleration value (for example PGA) provided by the code, and one wants to include in selection the likely value of the secondary IM conditional to it. In fact, two applications of conditional hazard are implemented in REXEL where PGA is assumed as the primary IM and the *Cosenza and Manfredi index* (I_D), which is considered a proxy for the cyclic content of ground motion, Equation (1), or PGV are the secondary IMs. In Equation (1), a(t) is the acceleration time-history, t_E is the total duration of the ground motion recording.

$$I_D = \frac{1}{PGA \cdot PGV} \int_0^{t_E} a^2(t)\,dt \qquad (1)$$

Having conditional hazard implemented for Italian sites, REXEL suggests to the user the distribution (in terms of complementary cumulative density functions) of I_D, or PGV, given the design PGA. This allows improving seismic input selection including, in a probabilistically consistent manner, care for PGV or cyclic content, without having to change the code design hazard.

3.1.6. Set size and set modification

In the previous releases of REXEL, it was possible a selection of seven records each of those featuring one, two or three components of motion. This was because seven is a reference set size in many codes. Now REXEL allows the search for sets comprised of one, seven, or thirty records. The new option has been included as one may be interested in one record individually matching a spectrum, as well as large group of records (i.e., thirty) for an analysis in which structural response is assessed with more confidence with respect to use seven records only.
After a spectrum matching set if any size is found, the analyst may want to exclude a particular record. In fact, REXEL finds sets matching the target spectrum via their average, thus a particular record may have a spectral shape the user wants to disregard for some reason. To address this issue, REXEL now includes the option *Repeat search excluding a station*, which allows to repeat the performed analysis by excluding any record from an already found solution. This allows to refine the selection in an iterative yet very fast way as it is guided by visual inspection of individual spectra in a found set.

3.2. REXELite

The availability of internet strong-motion databases as the ITalian ACelerometrica Archive (Luzi et al., 2008), developed by INGV for seismological purposes, is of certain interest also to the earthquake engineering community, as it facilitates seismic input definition for dynamic structural analysis by means of real records. However, as discussed, seismic structural codes, regarding the ground motion selection issue, often require that the suite of records has to “match” the design spectrum for the site and for the limit state of interest. This, along with other provisions, makes selection of real records hardly feasible for the practitioner if not adequately aided, as demonstrated by REXEL. While REXEL is a standalone software, in the RELUIS 2010-2013 project a web-based version, REXELite, with the same record selection algorithms (yet optimized for its inclusion in the a web portal) operating online on the ITACA database, was developed.

REXELite has the significant advantage to be constantly synchronized with the continuing evolution of ITACA, including new records, updated site classifications and new or revised information on existing waveforms.

Figure 4. REXELite user interface.

REXELite (Figure 4) allows to search for horizontal combinations of seven 1- or 2-horizontal components strong motion records, compatible on average with a specified target (code) spectrum, in a range of periods of interest and with arbitrary tolerances. The target spectra may be defined according to NIBC or to EC8. For this purpose, it is necessary to enter the geographical coordinates of the site, *latitude* and *longitude* in decimal degrees, and to specify the *Site Class* (according to EC8 classification), the *Topographic Category* (as in EC8), the *Nominal Life*, the *Functional Type*, and the *Limit State* of interest. For EC8 spectra, it is necessary to specify only the anchoring value of the spectrum (and therefore may be used for engineering projects outside Italy) and the site class because, as mentioned, the design spectrum is only function of a_g and S.

REXELite allows one to search for records within ITACA belonging to the same site class of the defined spectrum or to *any site class* (i.e., records referring to different site conditions may show up in the same set matching the target spectrum; see next section) and corresponding to magnitudes and epicentral distances of interest. In fact, the intervals [M_{min}, M_{max}] (moment and/or local magnitude) and [R_{min}, R_{max}] (epicentral distance, in km), in which to search for sets of accelerograms, have to be defined. Because when selecting a set of accelerograms for structural analysis the main objective is to reflect the relevant hazard scenarios at the site, for example from disaggregation (e.g., Iervolino et al., 2011b), REXELite allows to select suites which also belong to user-defined magnitude and source-to-site distance bins, and to the same site class of the location of the structure, or to any site class. Also the recording instrument features may be specified, e.g., whether *late-triggered* and/or analogue recordings should be or not included in the search. For an introduction and discussion on the quality of ITACA records, and specifically of the late-triggered records, see Paolucci et al. (2011) and Pacor et al. (2011). Once these options have been defined, REXELite returns the number of records (and the corresponding number of events and recording stations) available in ITACA. The spectra of the records returned by this preliminary search are used by REXELite to find a combination of seven accelerograms, whose average is compatible with the defined target spectrum and some tolerance in an arbitrary interval of periods [T_1, T_2] between 0s and 4s (Figure 4). ITACA may be searched for spectrum-matching sets of records which are original (unscaled) or ground motions linearly scaled in amplitude. Finally, REXElite not only ensures the set resulting from the search has its average matching the target spectrum, but also that it is the one with the smallest individual record-to-record variability (as in REXEL).

3.3. REXEL-DISP

Recent performance-based approaches to seismic design gave an increasing emphasis on the definition of the seismic demand at long periods. This is even more important when one refers to the definition of seismic demand in terms of displacement response spectra, such as in the capacity-spectrum (FEMA, 2005) and the direct displacement-based design approaches (Priestley et al., 2007), where the availability of reliable earthquake ground motions up to long periods is required.

This stimulated several research works concerning, on one hand, the development of improved displacement design spectra based on independent evaluations of long period spectral ordinates rather than on the use of the standard pseudo-spectral rule (Faccioli et al., 2004) and, on the other hand, the definition of simple criteria to assess the reliability of digital strong motion data (Paolucci et al., 2008). Such advances supported the calibration of up-to-date empirical ground motion prediction tools extending to long periods (Cauzzi and Faccioli, 2008), the improved quantification of site effects at long periods (e.g., Figini and Paolucci, 2009), the formulation of new seismic hazard maps at long periods in Italy (Faccioli and Villani, 2009).

In the framework of performance-based seismic design and assessment, a relevant issue is the selection of a *suitable* set of ground motion records to represent the design seismic excitation for nonlinear dynamic analysis. According to the vast majority of international codes, such as the Eurocode 8, or EC8 (CEN, 2003), the current Italian seismic code, or NTC08 (CS.LL.PP, 2008), and US ASCE 7-10 (ASCE, 2010) provisions, the selected suite of records needs to match, within prescribed tolerance limits, the target design spectrum. Moreover, acceleration histories shall be obtained from records having magnitudes, distances and source mechanisms consistent with those controlling the target spectrum in the range of periods of interest for a given application. While tools designed for the selection of earthquake ground motions compatible with design acceleration response spectra for earthquake engineering purposes are progressively becoming available, e.g., REXEL and REXELite, tools for the selection of displacement-spectrum compatible accelerograms are still very limited and at research stage. Therefore, a user-friendly software, namely REXEL-DISP (Figure 5), which allows to automatically select suites of real ground motion records compatible with a target displacement spectrum, was developed.

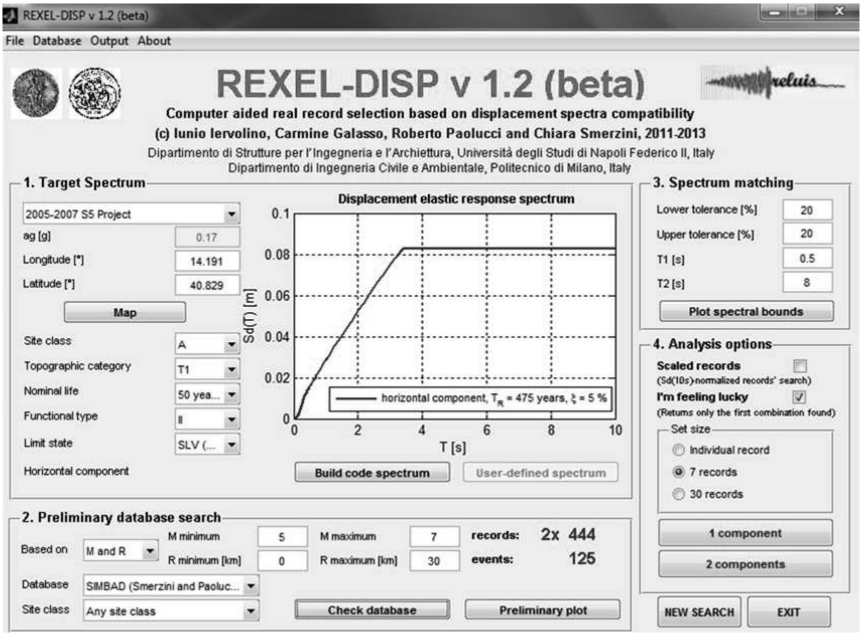

Figure 5. Image of the graphic user interface of REXEL-DISP.

The need to provide reliable displacement spectral ordinates over a broad range of vibration periods, e.g., up to about 10 s, led to embed in REXEL-DISP an *ad hoc* strong motion database, the discussed SIMBAD (Selected Input Motions for displacement-Based Assessment and Design) dataset, consisting of high quality three-component acceleration recordings from shallow crustal earthquakes worldwide in near field conditions. An innovative feature of REXEL-DISP is the definition of the target displacement spectra: besides the design spectra from the NTC08 and EC8 codes, an alternative target displacement spectrum for Italian sites is introduced. The latter combines the norm prescriptions at short periods and the main results of the PSHA study for Italy of Faccioli and Villani (2009) at long periods.

REXEL-DISP, available at http://www.reluis.it/index_eng.html, allows one to search for combinations of horizontal accelerograms whose average is compatible with a target displacement spectrum, and in which individual records have the shape as similar as possible with that of the target in a prescribed period range. Records may also reflect desired magnitude and source-to-site distance bins or specific ranges of some peak and integral ground motion intensity measures.

In the definition of the analysis constraints, REXEL-DISP allows to search for records within SIMBAD belonging to magnitudes and epicentral distances from pre-defined bins or to selected ranges of: i) PGA; ii) peak ground velocity or *PGV*; iii) I_D; iv) Arias Intensity (I_A).

Once the selection options are defined, REXEL-DISP returns the number of records (and the corresponding number of events) available in SIMBAD, which match these. The spectra of the records returned by this preliminary search are used by REXEL-DISP to find sets of record (one, seven, or thirty), whose average is compatible with the defined target spectrum in an arbitrary interval of periods [T_1, T_2] between 0 s and 10 s. Spectrum matching is ensured with some tolerance also defined by the user as an analysis' option.

The compatible record set found can be either comprised of single-component accelerograms, hence to be applied in one horizontal direction for analysis of bi-dimensional (plan) structures, or of pairs of horizontal components (therefore, for example, if one searches for a set, of seven ground motions, the software actually returns fourteen records) for the analysis of three-dimensional structures.
REXEL-DISP allows one to obtain combinations of accelerograms compatible with the code spectrum that do not need to be scaled, but it also allows one to choose sets of accelerograms compatible with the target spectrum, if scaled linearly (in this case, the individual records scale factors (SFs)are also provided). If this second option is chosen, the user has to check the *Scaled records* box, and the spectra of the preliminary search are normalized dividing the spectral ordinates to the value at 10 s. Combinations of these spectra are compared to the non-dimensional target spectrum. If this option is selected, it is also possible to specify the maximum mean scale factor allowed.

3.4. Engineering design earthquakes for Italy

It was illustrated that REXEL has embedded, to drive record selection, disaggregation of seismic hazard of four different spectral ordinates for the whole Italy. The disaggregation was developed in a specific study (Iervolino et al., 2011b) that is described in this section.
Given the characterization of seismic sources and once a ground motion IM is chosen, PSHA allows to identify, for each considered site, the probability of exceedance of different IM values in a time interval of interest. Choosing a return period, and assuming as IM the elastic spectral acceleration at different structural periods, it is possible to build the UHS; i.e, the response spectrum with a constant exceedance probability for all ordinates (Reiter, 1990). In the most advanced seismic codes, the UHS currently is the basis for the definition of design seismic actions on structures. On the other hand, for example when dealing with record selection, accelerograms not only are recommended to match such a spectrum, but also to be compatible with the earthquakes *dominating* the hazard at the site (e.g., Eurocode 8); see Figure 1.
PSHA, for its integral nature, combines the contribution to the hazard from all considered sources. The event most important for the occurrence or exceedance of an IM value may be identified via disaggregation of seismic hazard (Convertito et al., 2009). In fact, once the UHS has been defined, it is possible to identify one or more earthquakes; i.e. the values of magnitude source-to-site distance and ε (number of standard deviations that the ground motion parameter is away from its median value estimated by the assumed attenuation relationship) providing the largest contributions to the hazard in terms of exceeding a specified IM value. These events may be considereds as the earthquakes dominating the seismic hazard in a probabilistic sense, and may be used as design earthquakes; e.g., Iervolino (2010).
Analytically, disaggregation results is the joint probability density function (PDF) of magnitude, distance and ε given the exceedance of an IM level; i.e., the values of these parameters most frequent in those cases the IM level chosen is exceeded, as described in the following equation:

$$f\left(m,r,\varepsilon \middle| IM > IM_0\right) = \frac{\sum_{i=1}^{N} \nu_i \cdot I\left[IM > IM_0 \middle| m,r,\varepsilon\right] \cdot f_{M,R,\varepsilon}\left(m,r,\varepsilon\right)}{\sum_{i=1}^{N} E_i\left(IM > IM_0\right)} \tag{2}$$

where: N is the number of seismic sources which affect hazard at the site of interest; ν_i is the mean annual rate of occurrence of earthquakes at each of the considered sources; $E_i\left(IM > IM_0\right)$

is the mean annual rate of exceedance of a given IM_0 value (result of the hazard integral) and I is an indicator function that equals to 1 if IM exceeds IM_0 for a given distance r, a given magnitude m and a given ε, whose joint PDF is represented by $f_{M,R,\varepsilon}(m,r,\varepsilon)$. From equation (2) it is possible to observe that disaggregation depends on IM_0 (i.e., the hazard level being disaggregated, or the return period of the IM) and on the definition of the IM itself. If the spectral acceleration of interest is Sa(T), then disaggregation, and therefore the design earthquakes, also depends on T. In fact, UHS for different return periods is characterized by different design earthquakes, and, within a given UHS, short and long period ranges may display different M, R and ε from disaggregation (Reiter, 1990; Convertito et al., 2009).

In Italy, INGV provides disaggregation, for nine return periods between 30 and 2475 year, but for PGA only (see http://esse1-gis.mi.ingv.it/). Within this project, disaggregation of all Italian sites for structural periods equal to 0 sec (PGA), 0.5 sec, 1.0 sec and 1.5 sec was carried out. Disaggregation for these four periods is intended to help in identifying design earthquakes for the short, moderate and long period ranges of the UHS related to the life safety limit-state of ordinary constructions. Four different return periods were considered (2475, 975, 475 and 50 years) corresponding to the main limit states for civil and strategic structures, however, only results for Tr equal to 475 years will be shown in the following.

Disaggregation requires PSHA first; then, exceedance probabilities were computed for thirty values of the IMs equally distributed between 0.001g and 1.5g. All the analyses have been performed by a FORTRAN program specifically developed and also used in Convertito et al. (2009). The modelling of seismogenic zones is that proposed by Meletti et al. (2008), also adopted by INGV. Seismicity parameters of each zone are those used by Barani et al. (2009). The considered grid for Italy is the same of that from INGV and used in the Italian seismic code. All the analyses refer to rock site conditions. According to Ambraseys et al. (1996), which is the GMPE considered, magnitude is that of surface waves (M_s). Because of seismogenic zones modelling, the hazard software assumes a uniform distribution of possible epicentres, then epicentral distance is converted (Gruppo di Lavoro, 2004) in closest distance to the projection of the fault rupture (R_{jb}), as defined by Joyner and Boore (1981). Because the used GMPE is valid for R_{jb} up to 200 km, the influence of sources with larger R_{jb} was neglected in the hazard analysis for each site.

The hazard results, computed in terms of PGA and spectral acceleration at T = 1.0 sec are in fair agreement with those of INGV, and they are considered as the basis for disaggregation analyses presented in this section. The joint PDFs of M, R and ε given the exceedance of IM_0 with an exceedance return period of 475 years were computed, for each site of the grid, via simulation and using bins of M, R and ε equal to 0.05, 1.0 and 0.5, respectively. Minimum and maximum values used for ε are -3 and +3. Subsequently the first two modes of the joint PDF from disaggregation were extracted. The first mode is identified as the M, R and ε vector giving the maximum contribution to the hazard, while the second mode corresponds to second higher relative maximum contribution, identified if the differences between first and second mode are 5.0, 0.25 or 0.25 in terms of M, R, or ε respectively. Figure 6 and Figure 7 show the modes of disaggregation distributions. In the map referring to the second mode, white zones indicate that the hazard contribution of second mode is negligible or zero.

Looking at disaggregation results for PGA, it was possible to identify general trends: (i) the first mode corresponds to an earthquake caused by the closer source (or the source the site is enclosed into) and with low-to-moderate magnitude, and (ii) the influence of the more distant zones is accounted for by the second mode, which is usually a larger magnitude one. For a few sites, the particular combination of geometrical condition and seismic parameters of each source can determine an inversion of disaggregation results, and in such sites the sources influencing the first mode can be more distant than that related to the second mode. Other exceptions are represented by sites with a single mode; i.e., one design earthquake. These sites are enclosed or close to zones with high seismicity with respect to the surrounding zones and the hazard contribution from other zones is negligible (see also Convertito et al., 2009). Considering T = 1.0sec disaggregation results, the general conclusions of PGA are confirmed. However, changing from PGA to Sa(T=1.0) the contribution of the second mode increases. Finally, analyses show that almost all sites are characterized by two different modal values of disaggregation. This means that, from a design point of view, for each site may be useful to know not only the first mode, but also the second one, in definition of seismic action on structures.

It is finally to note that this disaggregation information are available to the public also through the ReLUIS website (http://193.206.66.17/egeos/web/reluis; user name and password "reluis"). Once the design earthquakes are selected for a specific site in Italy then they can be automatically passed to the REXELite website and used as a selection constraint for spectrum-matching records.

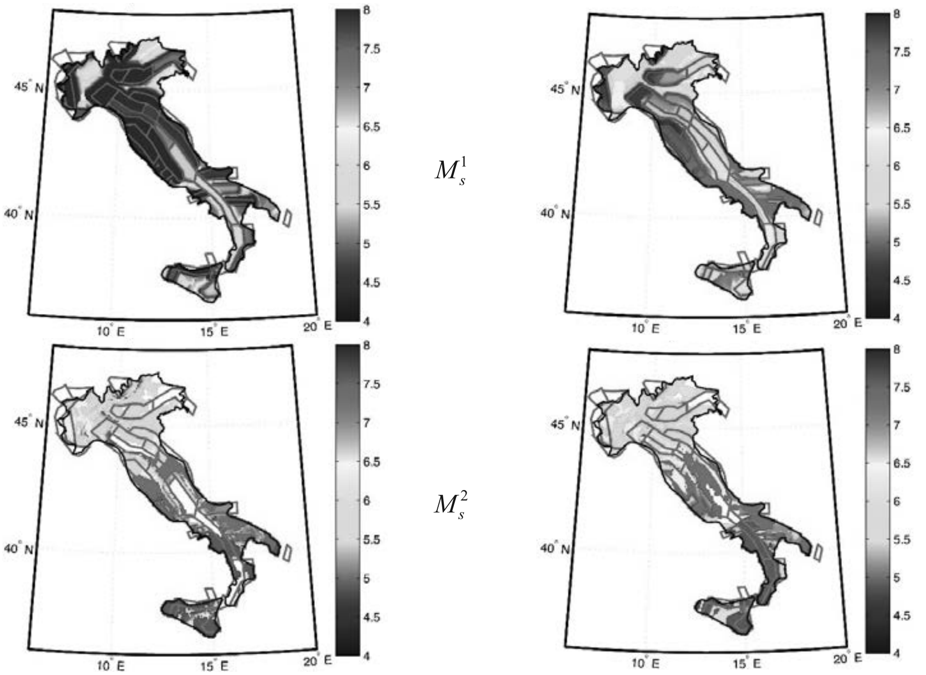

Figure 6. Map of disaggregation results represented by first (1) and second (2) modal values of M_s for PGA (*left*) and Sa(1.0sec) (*right*) and for Tr = 475year.

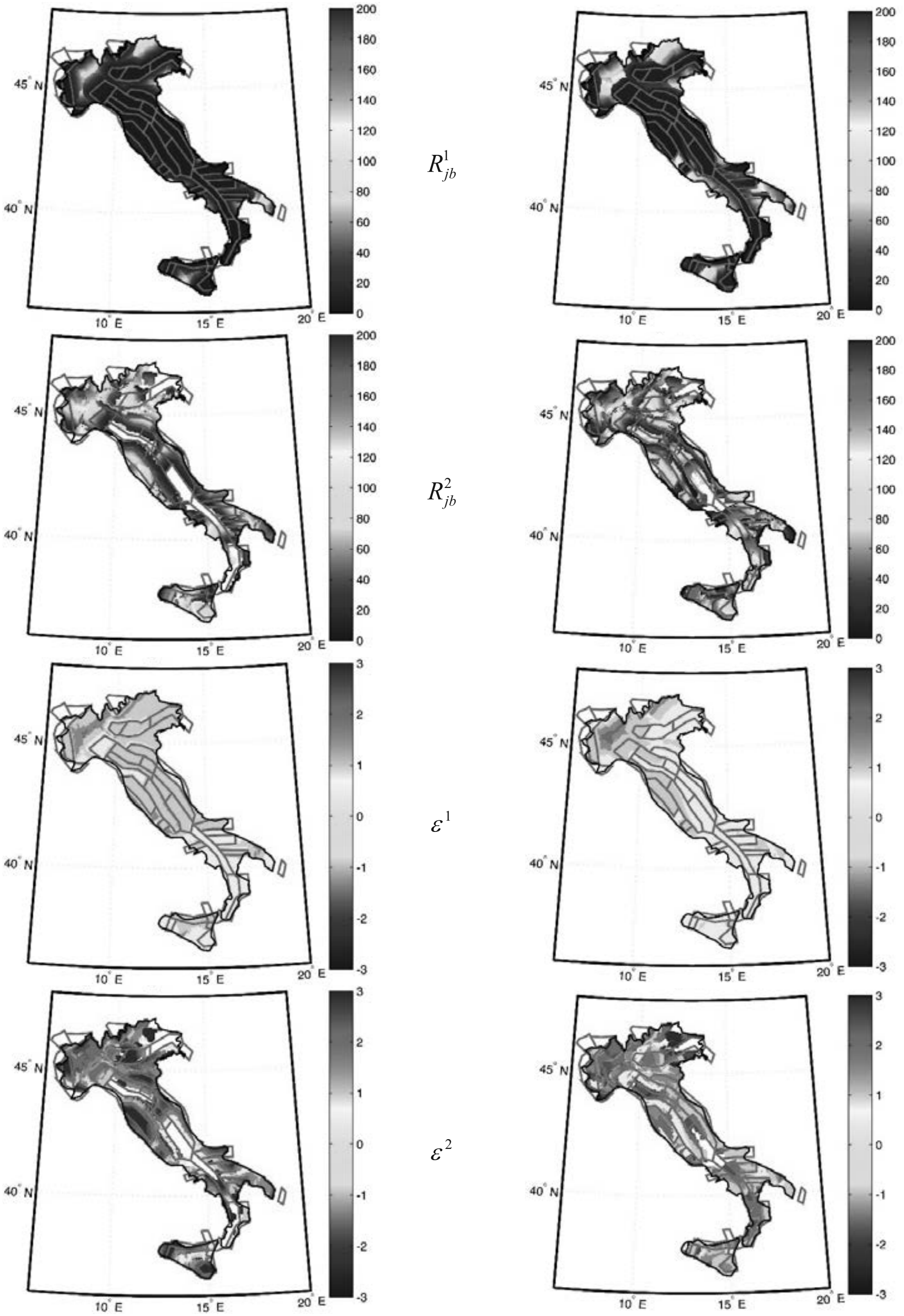

Figure 7. Map of disaggregation results represented by first ([1]) and second ([2]) modal values of R_{jb} and ε for PGA (*left*) and Sa(1.0sec) (*right*) and for Tr = 475year.

3.5. Spectrum-based evaluation of seismic response to artificial and real-manipulated records

Engineering practitioners have several options to get (spectrum-matching) input signals for their analysis; e.g., real or real manipulated records and various types of synthetic and artificial accelerograms. All these options are usually acknowledged by codes which may provide additional criteria or limitations for some of them. In the Italian seismic code, for example, artificial records, generated recurring to random vibration theory, should have duration of *at least 10s in their pseudo-stationary part*, and cannot be used in the assessment of geotechnical structures. Synthetic records, generated by simulation of earthquake rupture process, should refer to a characteristic scenario for the site in terms of magnitude, source-to-site distance and seismological source characteristics; finally real records should reflect the earthquake *dominating* the hazard at the site. However, practitioners not always can accurately characterize the seismological threat to generate synthetic signals or it is not possible to find a set of real records that fits properly code requirements in terms of a specific hazard scenario. In fact, despite in the last decades the increasing availability of databanks of real accelerograms has determined a spread use of this type of records, it may be very difficult to successfully apply code provisions to obtain code-compliant real record sets. In particular, provisions regarding spectral compatibility are hard to match if appropriate tools are not available. This is why the relatively easy and fast generation of artificial records, perfectly compatible with an assigned design spectrum, is still very popular for both practice and research purposes. More recently, procedures to get the spectral compatibility of real records by wavelets adjustment were proposed (e.g., Grant et al., 2008). This kind of manipulation is conceptually an extension of the more simple linear scaling of real records to modify (e.g., to amplify) the spectral shape to get a desired intensity level.

Although several studies tried to assess the *reliability* of each of these procedures (e.g., Schwab and Lestuzzi, 2007), many of them are relatively narrow in scope without giving a general overview of the spectral compatibility issue. Iervolino et al. (2010b) tried to address the spectral matching matter from the structural point of view in terms of ductility and cyclic response, having as reference a code-based design spectrum. To this aim six classes of 28 accelerograms, each of those comprised of four sets of 7, were considered: (1) unscaled real records (URR - Figure 8a); (2) moderately linearly scaled real records (SF5 - Figure 8b); (3) significantly linearly scaled real records (SF12 - Figure 8c); (4) wavelet-adjusted real records (RSPMatch - Figure 8d; Abrahamson, 1992); (5) (Belfagor - Figure 8e; Mucciarelli et al., 2004) stationary artificial records; (6) (SIMQKE - Figure 8f; Gasparini and Vanmarke, 1976) non stationary artificial records. All sets are compatible with the elastic design spectrum for a case study in southern Italy.

The seismic responses of a large number of SDOF systems, with different backbones, hysteretic relationships, and with various strength reduction factors (R), were considered. As structural response measures, or engineering demand parameters (EDPs), the ductility normalized with respect to the strength reduction factor and the equivalent number of cycles were considered to relate the ground motions to both peak and cyclic structural demand (Manfredi, 2001). Analyses aimed at comparing the differences, if any, in the EDPs associated to each class of records with respect to the unscaled real records, considered as a benchmark. Hypothesis tests on selected samples were also carried out to assess the statistical significance of the results found in terms of both peak and cyclic response.

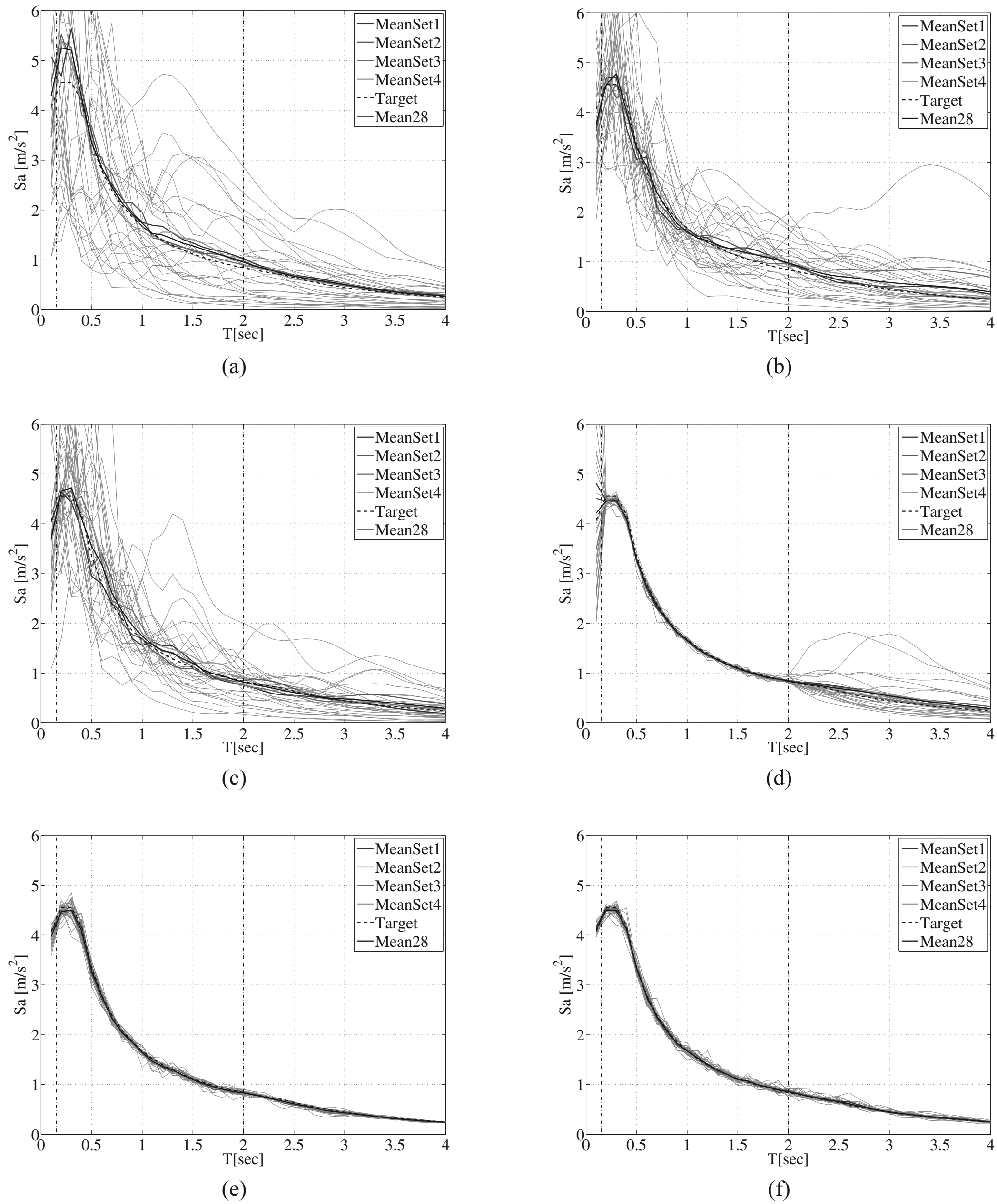

Figure 8. URR (a), SF5 (b), SF12 (c), RSPMatch (d), Belfagor (e), and Simqke (f) acceleration elastic spectra, compared to the target spectrum.

While a more comprehensive summary of the work and the results may be found in Iervolino et al. (2010b), as an example of the results of this study Figure 9 shows kinematic ductility demand (D_{kin}) normalized with respect to the different R values investigated referring to one of the SDOFs investigated; i.e., the EPH (elastic-plastic with hardening) system. For low R, normalized ductility seems to be similar for all six classes of records. The cases for high R values (Figure 9c and Figure 9d) emphasize an apparent underestimation of ductility for artificial records with respect to real records classes. In particular, results for R equal to 10 show different underestimation levels for adjusted and artificial classes of records: Belfagor class is followed by Simqke and RSPMatch. Ductility response indicates that wavelet adjusting procedure gives a lower bias. On the other hand, it should be recalled that RSPMatch records are the same records as URR to which the adjustment procedure was applied. Linearly scaled records, indifferently if moderately or significantly, seem to show no trends with respect to URR. The large scattering of real records with respect to the target, leads to large variability of the average estimated response from class-to-class of real records; e.g., Figure 9c and Figure 9d.

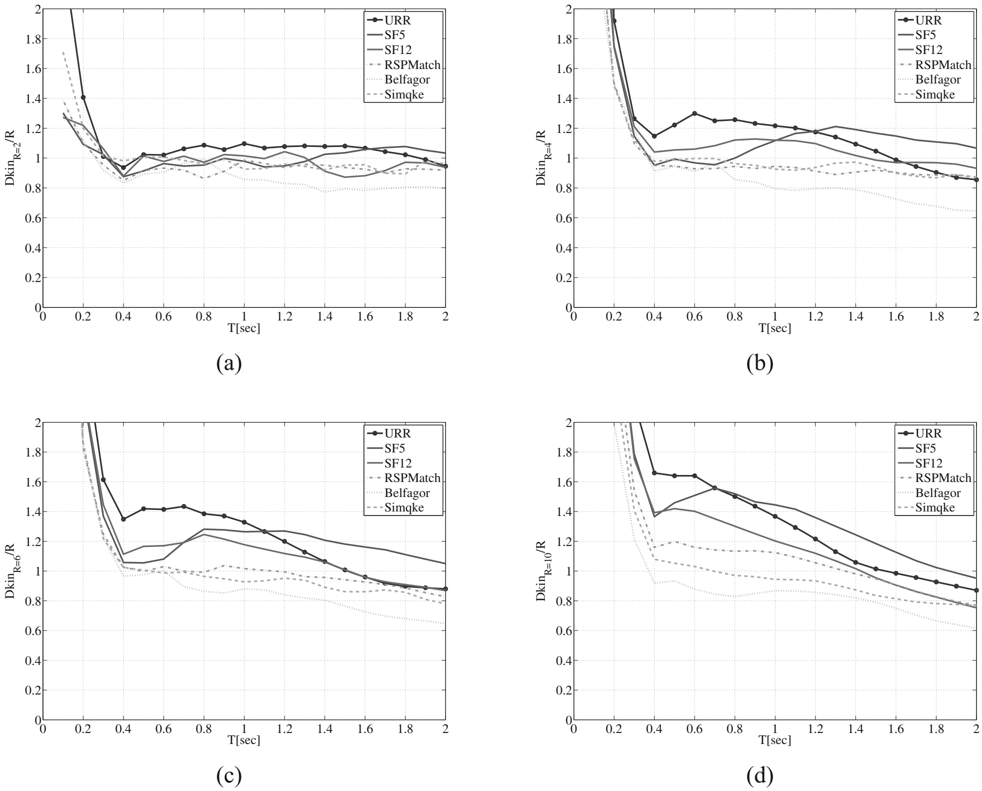

Figure 9. Average values of kinematic ductility demand for EPH system computed as mean value of 28 records.

3.6. Scenario- and GMPE-based evaluation of seismic response to simulated records and the Synthesis repository

Modern common earthquake resistant design procedures are based on inelastic deformation of structures as the primary source of seismic energy dissipation. On the other hand, design actions are based on PSHA, which refers to elastic acceleration response spectrum ordinates.

According to the analysis method employed, the elastic acceleration response spectrum represents the link between hazard and vulnerability of structures. Recently is earning interest the possibility to develop PSHA directly in terms of nonlinear structural response to improve accuracy in definition of structural design targets. This may require a prediction equation (also referred to as attenuation law) for the structural response measure of interest. The possibility to develop an attenuation law for nonlinear SDOF systems' responses based on the Italian Accelerometric Archive was explored. Peak and cyclic inelastic structural response parameters were considered for the development of such attenuation laws, useful for both design or assessment aims (see De Luca, 2011).

Prediction equations in terms of inelastic peak and cyclic inelastic response can also be employed as a benchmark for the engineering validation of simulated records; i.e., those ground motions obtained simulating source, path, and site effects in one earthquake. The validation made in terms of prediction equations has its main basis in the fact that as long as simulated records are employed in nonlinear dynamic analyses as substitutes of real records, the main target is that they lead to the same conclusion in terms of risk assessment.

A preliminary validation example, referred to the case of four sites for the 1980 Irpinia, M 6.9 earthquake was carried out focusing on the comparison of validation made by means of prediction equations with the more typical simulated-to-real validation approach. The synthetic records refer to 54 simulations of the Irpinia event at those seismic station (BGI, BSC, BNV, BVN in ITACA) where a real counterpart was recorded in the event. Three finite-fault simulation techniques were adopted to produce synthetics: purely stochastic, hybrid deterministic-stochastic with approximated Green's, broadband integral-composite. For all simulations, a normal fault-plane, embedded in 1D propagation medium, 35 km long and 15 km wide, with 315° strike and 60° dip, was assumed. On this fault, 54 different rupture models were simulated by varying the main source kinematic parameters such as: position of the nucleation point, rupture velocity, and final slip distribution (Ameri et al., 2011).

In the example provided, SODF's inelastic displacements, and equivalent number of cycles (Manfredi, 2001), for different strength reduction factors, were controlled as informative intensity measures for peak and cyclic response to be employed in the validation (see Figure 10 and Figure 11). In fact, for these, inelastic GMPEs were developed (those of De Luca, 2011). The first, expected, conclusion is that considering nonlinear response rather than the only elastic one can provide additional significant results to the validation procedure (see De Luca et al., 2014, for dtails).

The preliminary validation shows that the simulations have an underestimation trend mostly in terms of cyclic response with respect to the median estimates of the prediction equations. On the other hand, in the case of inelastic displacements the simulation are between the one standard deviation bands of the prediction equations. The underestimation trends of the simulated records, have different causes for equivalent number of cycles and spectral displacements. For cyclic response, the underestimation can be partially ascribed to the simplified simulation model with respect to the actual 3-dimensional crustal structure. For peak values, the simulations are lower than mean predictions but in better agreement with the observed data, thus suggesting that synthetics capture specific features related to the Irpinia earthquake, that are not predicted by the GMPE, developed for the whole national territory. The underestimation of observed peak spectral displacement is similar, while the underestimation trend in terms of cyclic response is opposite, to what found in previous studies for spectrum-compatible artificial records. Notwithstanding the preliminary character of the results provided, given the small sample of records considered, the validation approach through prediction equations of peak and cyclic inelastic response is a first step towards a systematical engineering validation of physics-based simulated accelerograms.

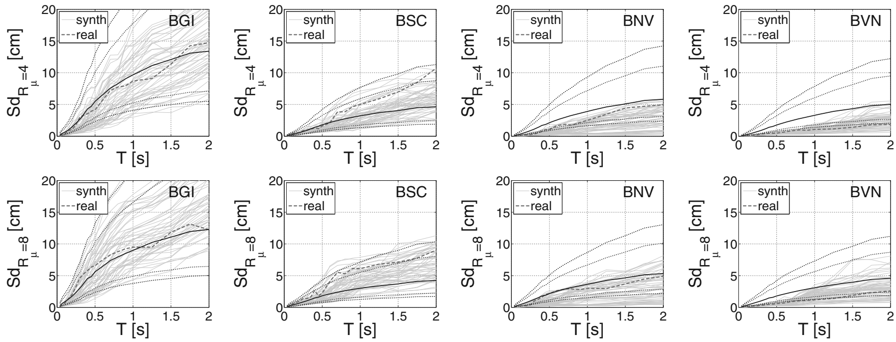

Figure 10. Median spectra of the prediction equation for inelastic spectral displacement (Sd_i), EPH SDOF system, strength reduction factor (R_μ) = 4 and 8, with their total (black dotted lines) and intra-event (blue dotted lines) standard deviation bands compared with the corresponding spectra of the 54 simulated (synth) and the real (real) records at the stations BGI, BSC, BNV, and BVN.

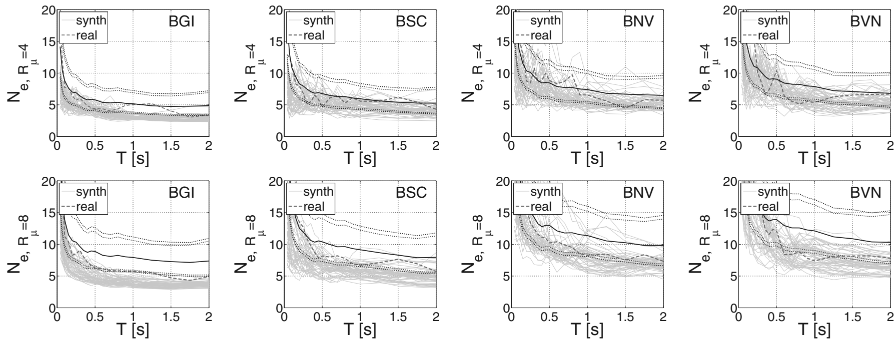

Figure 11. Median spectra of the prediction equation for the equivalent number of cycles (N_e), EPH system, compared with the corresponding spectra of the 54 simulated (synth) and the real (real) records at the stations BGI, BSC, BNV, and BVN.

Finally, the activity related to simulated records produced a prototype for synthetic seismograms database, called SYNTHESIS (http://synthesis.mi.ingv.it/); Figure 12. It is based on the structure and the tables of ITACA, made and managed to distribute strong-motion data recorded in Italy. SYNTHESIS is a fully relational data base, which can be accessed through user friendly interfaces which allow the user to perform queries to select scenarios, sites, and waveform parameters, in order to download the synthetic seismograms. The main interfaces are three: one for the seismic scenarios, one for the sites and one for synthetic seismograms. The database can be explored through searchable key fields: 13 for the stations, 11 for the seismic scenarios and 13 for the waveforms. The basic idea is that separated queries can be performed for three distinct data base blocks (stations, events and waveforms). Each query (event, station and waveform) returns a list and the single outcome can be explored in detail. Figure 8 shows and example of detail-page relative to the selection

of a seismic scenarios. Both sites and events are mapped through the Google-Map data©, which allows to display point data alternatively on a satellite image or a basic map (or both). A gallery of images is also included in the database, illustrating input data and results from the scenarios study, such as distribution of slip on the fault or map of simulated peak values corresponding to a specific scenarios.
Although synthesis is intended to host records from eventually simulated earthquakes, it was originally populated with three sets of simulated accelerograms, each of them composed by about 4500 waveforms, computed for the Irpinia fault, that cuased the 1980, M 6.9, Irpinia earthquake. A common name and file format were selected to write the files containing the synthetics.

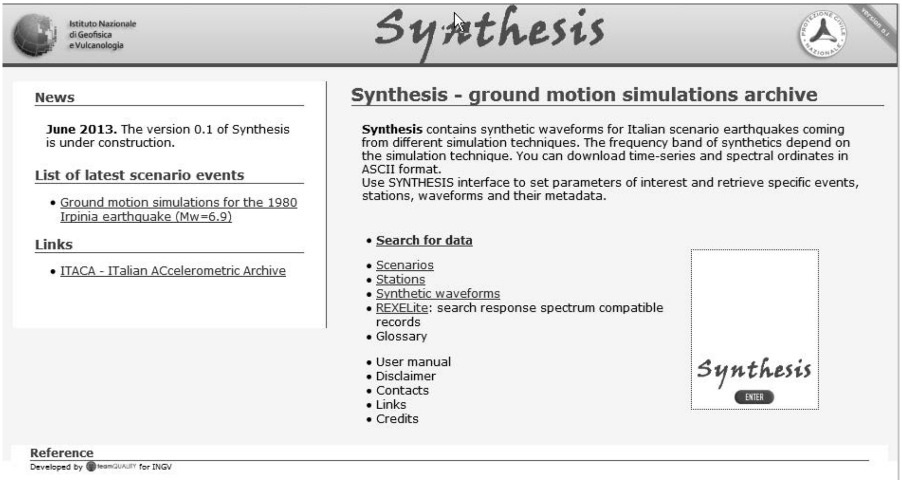

Figure 12. Home page of SYNTHESIS.

It is worth to mention also that the work related to validation of simulated ground motion, thanks to a collaboration with the University of California at Irvine (for what concerns the structural analysis part) and the Southern California Earthquake Center (for the part concerning the simulation of ground motions), also referred to simulations for historical events recorded in the US. In particular, a comparison between simulated and recorded GMs in terms of elastic and post-elastic seismic response was carried out. This was pursued by considered elastic and inelastic SDOF systems and generalized linear MDOF buildings. Four historical earthquakes in California have been considered (see Figure 12). Several engineering demand parameters have been considered and an example of comparison is shown in Figure 13; details may be found in Galasso et al. (2012a-c and 2013).

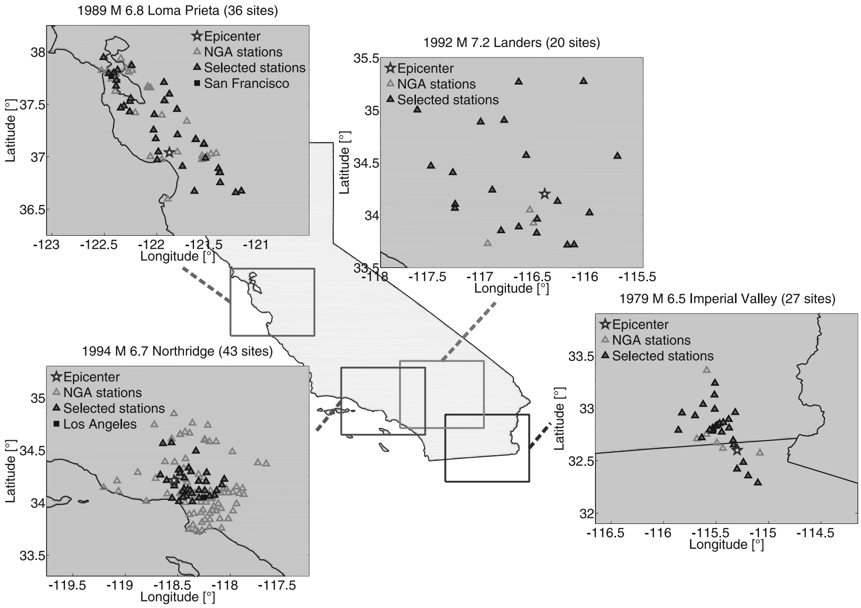

Figure 12. Maps of the considered earthquakes. The star is the epicenter and the grey triangles are recording stations of the NGA database for which the simulations are available. The red triangles are recording stations considered in this study.

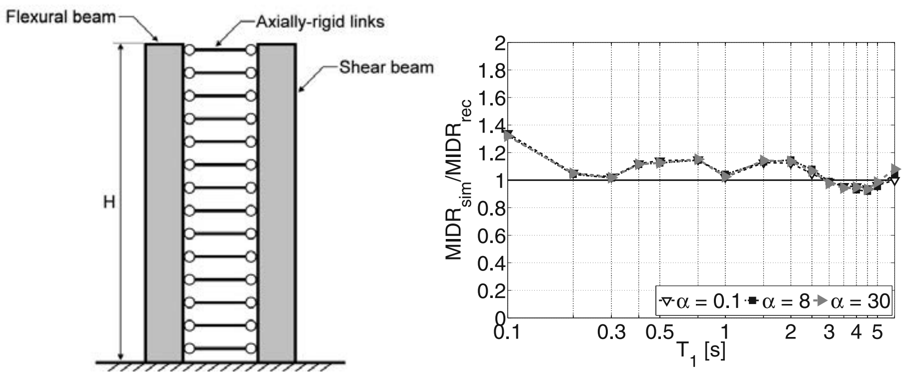

Figure 13. Simplified model used (left) and ratios of the medians of the generalized maximum interstory drift ration (MIDR) spectra for simulated GMs to the corresponding quantity computed for the recorded GMs in the Imperial Valley earthquake.

4. DISCUSSION

The work described herein was an integrated effort to comply with code-based requirements for record selection, yet rendering it as much as possible hazard-informed. In other words, the aim of the bulk of the research carried out was entirely devoted to enrich seismic-code-related record selection with relevant research results on the topic. Another driver was the development of tools that may be directly applicable by the practitioners. Therefore, large part of the research was devoted to the update and extension of the REXEL software. The latter

was released with new features, which integrate new hazard-related information such as disaggregation and conditional hazard. Moreover, other versions of REXEL with specific purposes, that is to work online on the ITACA records repository, or to comply with a displacement spectrum for displacement-based earthquake engineering were developed.
This effort of having practice-ready tool also produced an online repository of disaggregation of seismic hazard to provide design earthquakes for the whole Italy. This database is directly linked to REXELite so that once the site is selected the design earthquakes are automatically input as magnitude and distance constraints for record selection in ITACA.
The second part of the work was aimed at understating the features of simulated and artificial records that may impact structural response. This is important, indeed, because, once proven, applicable these records may provide an alternative for all those cases in which real records are not available. Two issues were faced in particular. First of all, it was assessed the bias, if any, induced by artificial records when the criterion is matching a design spectrum, that is code-based selection. The response of different kinds of artificial records and manipulated real records were compared to a reference real records case. Different structural response measures were considered, believing that artificial records may induce different seismic response with respect to real records also depending on the engineering demand parameters. Finally, simulated ground motions (i.e., those obtained via simulation of source path and site effects) were also considered and tested in terms of non-linear structural response versus their real counterpart. This was carried out for some Italian and US historical earthquakes and in terms of single and multiple degrees of freedom systems in a joint effort with the University of California at Irvine and the Southern California Earthquake Center. Finally, the prototype of a repository for simulated ground motions for engineering uses was developed. The long term goal is to have a database of simulated earthquake scenarios for earthquake engineering practice that would allow one to select specific situations that may not always be represented by recordings.

5. VISIONS AND DEVELOPMENTS

The described research takes for granted the selection criteria given in the seismic codes (with a specific focus on Italy) and tries to account as much as possible for the research results on the topic of seismic input for seismic structural analysis without changing those. However, it is recognized that there are some issues which may require update of selection criteria and consequently research in the field. For example, the intrinsic limits of the uniform hazard spectrum, which does not represent the spectrum of any specific event, may be overpassed by more recent alternatives. Also the control of the uncertainty in the estimated structural response via the number of ground motions employed should be explicitly investigated. Moreover, some specific, yet relevant, situations, such as near-source pulse-like effects are not yet accounted for by record selection practice. Also record selection for geotechnical engineering requires further effort. In particular the consistent consideration of relevant intensity measures beyond spectral ordinates as well as the use of artificial or simulated ground motions.
To address all these issues substantial aid may come from simulated ground motions, which in principle may provide great flexibility to address a broad variety of engineering needs. Therefore, a large coordinated effort of seismologist and earthquake engineers (both structural and geotechnical) should be devoted to the validation of simulations for engineering purposes

as well as to render easily available for practice this kind of ground motions. On the other hand, the selection of real records is nowadays a quite established methodology, which may be refined but will unlikely undergo under changes of paradigm in the near future.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 527-568, doi: 10.14599/r101315, © 2015 Doppiavoce, Napoli, Italy

SEISMIC ACTION AND STRUCTURAL EFFECTS OF NEAR-SOURCE GROUND MOTIONS

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INTRODUCTION

In has been known for some time now, in both the field of seismology and that of earthquake engineering, that earthquake ground motions recorded at sites located in proximity to active faults exhibit particular characteristics which duly affect structural response. These exceptional ground motions are said to be subject to phenomena collectively termed near-source (NS) effects. Perhaps the most important among these, is forward rupture directivity (FD): during fault rupture, shear dislocation may propagate at velocities similar to the shear wave velocity and as a result at sites aligned along the direction of rupture propagation, shear wave-fronts generated at different points along the fault may arrive at the same time, delivering most of the seismic energy in a single double-sided pulse registered early in the velocity recording. Such impulsive behavior, which is actually the result of constructive interference of horizontally polarized waves, is typically most prominent in the fault-normal component of ground motion (Somerville et al., 1997).

As engineering interest in this type of impulsive ground motions continues to grow, it is the objective of this research task to study the effects of NS ground motions on the response of structures, with the intention of being able to better define seismic actions and structural design rules for regions situated near known seismic sources. This entails not only analytical treatment of the problem but also close scrutiny of such effects as have been registered and observed in recent earthquakes, with particular emphasis on the April 2009 L'Aquila event.

The research presented herein has been divided into two parts. Part A, which is more focused on the definition of NS design seismic actions, comprises research which follows three principal directions: firstly, existing semi-empirical models are put to use in order to define NS elastic design spectra by means of well-established probabilistic seismic hazard procedures. Secondly, modification factors to derive inelastic structural demand from the elastic one are developed, appropriate for NS pulse-like ground motions; such modification factors are potentially applicable in the implementation of nonlinear static procedures of structural assessment in NS environments. Thirdly, the effect of damping on the displacement demand of single degree of freedom (SDOF) systems subjected to such pulse-like excitations is investigated.

Part B is dedicated to the L'Aquila case study, where research saw intense interaction among seismologists, geologists and geotechnical and structural engineers, culminating in the end

[1] Part B of this chapter (to follow) was prepared with the contribution of professor Roberto Paolucci (Politecnico di Milano, Italy).

result of a comprehensive simulation which includes source effects, propagation and site effects. This case study is oriented towards evaluating the feasibility of modelling NS effects via numerical simulation.

PART A: NEAR-SOURCE SEISMIC HAZARD AND STRUCTURAL EFFECTS OF NEAR-SOURCE GROUND MOTIONS

1 NEAR-SOURCE SEISMIC HAZARD AND DESIGN SCENARIOS

Most advanced seismic codes worldwide define structural design actions based on probabilistic seismic hazard analysis or PSHA (Cornell 1968, McGuire 2004), which allows the building of hazard curves starting from seismic source models and ground motion prediction equations (GMPEs). PSHA is, to date, a consolidated procedure; however, the need for adjustments for sites close to a seismic fault is emerging. In fact, in NS conditions, both ground motions and seismic structural response, may show systematic spatial variability which classical PSHA is not able to explicitly capture.
Recent attempts to explicitly account for directivity effects in probabilistic hazard assessment are aimed at modifying classical PSHA to highlight the pulse-like features of ground motion rather than formulating a new procedure (e.g., Abrahamson 2000, Tothong et al. 2007, Iervolino and Cornell 2008).

1.1 Near-Source PSHA

The standard approach for computing the mean annual frequency (MAF, λ) of exceeding a ground motion intensity measure (IM) threshold is shown in Equation (1) for a single seismic source. The chosen IM is the elastic spectral acceleration (Sa) at a fixed spectral period (T^*) exceeding an intensity level, $Sa(T^*) = s_a^*$:

$$\lambda_{Sa}\left(s_a^*\right) = \nu \cdot \int_m \int_r G_{Sa|M,R}\left(s_a^* | m,r\right) \cdot f_{M,R}\left(m,r\right) \cdot dm \cdot dr \qquad (1)$$

where M is the magnitude and R is the source-to-site distance, ν is the mean annual rate of occurrence of earthquakes on the source within a magnitude range of interest, $f_{M,R}$ is the joint probability density function (PDF) of M and R, and $G_{Sa|M,R}$ is the complementary cumulative distribution function (CCDF) of Sa (usually lognormal if obtained by a GMPE).
NS-PSHA requires the MAF to be a linear combination of two hazard terms which account for the absence or the occurrence of the pulse, $\lambda_{Sa,NoPulse}$ and $\lambda_{Sa,Pulse}$ respectively, as reported in Equation (2). In fact, the problem of estimating seismic hazard in near-source conditions may be posed as if two faults are present at the same location: one producing ordinary ground motions, and one producing pulse-like records.

$$\lambda_{Sa}\left(s_a^*\right) = \lambda_{Sa,NoPulse}\left(s_a^*\right) + \lambda_{Sa,Pulse}\left(s_a^*\right) \qquad (2)$$

The two terms of Equation (2) are implicitly weighted by the pulse occurrence probability. Moreover, two other tasks, which are not faced in traditional hazard analysis, appear: (i) pulse period prediction, and (ii) pulse amplitude prediction. In Equation (3) and Equation (4), which expand Equation (2) in the case of a single fault with undefined rupture mechanism, $\underline{Z}$ is a vector with all the required information (to follow) about the relative position between the seismic source and the site (e.g., Tothong et al. 2007, Iervolino and Cornell 2008).

$$\lambda_{Sa,NoPulse}\left(s_a^*\right) = \nu \cdot \int_m \int_{\underline{z}} P\left[NoPulse\middle|m,\underline{z}\right] \cdot G_{Sa|M,\underline{Z}}\left(s_a^*\middle|m,\underline{z}\right) \cdot f_{M,\underline{Z}}\left(m,\underline{z}\right) \cdot dm \cdot d\underline{z} \tag{3}$$

$$\lambda_{Sa,Pulse}\left(s_a^*\right) = \nu \cdot \int_m \int_{\underline{z}} \int_{t_p} P\left[Pulse\middle|m,\underline{z}\right] \cdot G_{Sa,\mathrm{mod}|M,\underline{Z},Tp}\left(s_a^*\middle|m,\underline{z},t_p\right) \cdot f_{Tp|M,\underline{Z}}\left(t_p\middle|m,\underline{z}\right) \times f_{M,\underline{Z}}\left(m,\underline{z}\right) \cdot dm \cdot d\underline{z} \cdot dt_p \tag{4}$$

Equation (3) refers to the case of pulse absence and it is weighted by the corresponding probability, $P\left[NoPulse\middle|m,\underline{z}\right]$. All other terms are equal to those of Equation (1) in which pulse-like effects are not considered. Conversely, Equation (4) refers to the case of pulse occurrence as indicated by the pulse occurrence probability, $P\left[Pulse\middle|m,\underline{z}\right]$. To account for the peculiar spectral shape of pulse-like records, it is possible to specifically calibrate a new GMPE or to modify an existing one: the latter is considered herein, thus the $G_{Sa,\mathrm{mod}|M,\underline{Z},Tp}$ symbol. Because modification of ordinary GMPEs depends on the pulse period, the $f_{Tp|M,\underline{Z}}$ distribution is required in the analysis. Finally, $f_{M,\underline{Z}}$ is the joint distribution (of magnitude and geometrical parameters) similar to the ordinary PSHA, but with a more detailed description (by means of $\underline{Z}$) of relative source-to-site position, with respect to the simple distance variable of Equation (1).

The complexity of rupture and wave propagation phenomena makes directivity prediction difficult if based only on physical parameters: in fact it is not always observed in the sites where it is expected, and may also occur at sites apparently not prone to pulse-like ground motion (e.g., Bray and Rodriguez-Marek 2004). Thus, stochastic models for the prediction of the pulse occurrence probability, $P[pulse]$, were developed (e.g.,Iervolino and Cornell 2008). These models depend only on geometrical parameters depicted in Figure 1, which are slightly different in the case of strike-slip (SS) or dip-slip (DS) faults, see Somerville et al. (1997). Such parameters in SS [DS] case are: (i) distance (s) from the epicenter to the site [d, from hypocenter to the site] measured along the rupture direction, (ii) θ angle between the fault strike and the path from epicenter to the site [ϕ angle between the fault plane and the path from hypocenter to the site], and (iii) minimum distance R between the rupture and the site. (For SS, some additional parameters, are shown in Figure 1.)

Equation (5) and (6) report the two models used in this study; note that the geometrical variables for DS were used in Iervolino and Cornell (2008) to fit generic non-strike-slip (NSS) data.

$$P\left[Pulse\middle|R,s,\theta\right] = \frac{e^{0.859-0.111\cdot R+0.019\cdot s-0.044\cdot\theta}}{1+e^{0.859-0.111\cdot R+0.019\cdot s-0.044\cdot\theta}} \tag{5}$$

$$P\left[Pulse\middle|R,d,\phi\right] = \frac{e^{0.553-0.055\cdot R-0.027\cdot d-0.027\cdot\phi}}{1+e^{0.553-0.055\cdot R-0.027\cdot d-0.027\cdot\phi}} \tag{6}$$

Equation (5) and Equation (6) are defined for R (SS), R (DS), s, d, θ or ϕ varying in the intervals of [0 km, 30 km], [5 km, 30 km], [0 km, 40 km], [0 km, 20 km], [0°, 90°], and [0°, 90°], respectively.

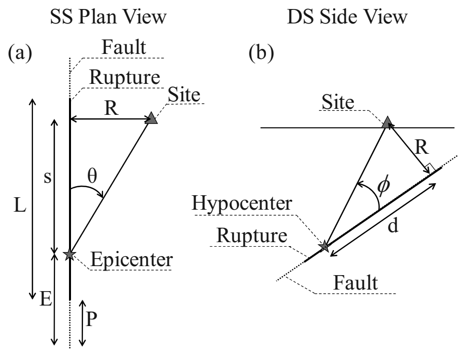

Figure 1. SS (a) and DS (b) geometrical parameters.

An example to illustrate the pulse occurrence probability model of Iervolino and Cornell (2008) is given in Figure 2 with reference to the rupture characteristics of the L' Aquila 2009 event (normal faulting). It is to point out that the occurrence probability is never larger than 0.5; this is because the model was developed generically for non-strike-slip earthquakes, which are often complex and in which it is not easy to identify rupture directivity effects. Nevertheless, it may be used to highlight sites comparatively more likely to be affected by velocity pulses given the source geometry. From this point of view, results of pulse occurrence probability model are in general agreement (except ORC) with the results of algorithm for the identification of pulse-like ground motions.

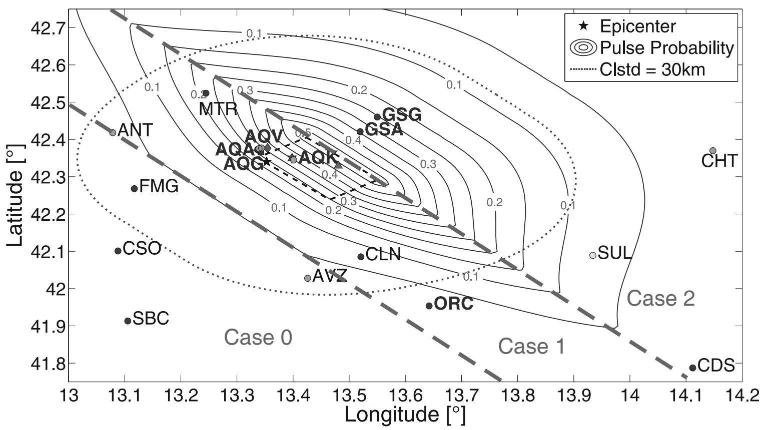

Figure 2. Contours of occurrence probability and accelerometric stations with pulse-like signals (blue).

In the case of an individual SS source of fixed dimensions and geographical location, the necessary geometrical parameters to compute hazard are the rupture length (L, assumed to be lower or equal to the fault length), the position of the rupture on the fault (P), and the epicenter location (E) as reported in Figure 1a. A deterministic relationship between these parameters and {R,s,θ} vector exists. Thus, the $\underline{Z}$ vector in Equation (3) and Equation (4) can be replaced by a vector comprised of L, P, and E.

In the analysis, L can be considered as a function of magnitude (e.g., Wells and Coppersmith 1994), while P and E may be associated to a uniform probability distribution on the fault and on the rupture length, respectively. Given these assumptions, Equation (3) and Equation (4) are specialized for the SS particular case, obtaining Equation (7) and Equation (8).

$$\lambda_{Sa,NoPulse}\left(s_a^*\right)=\nu\cdot\int_m\int_l\int_p\int_e P\left[NoPulse\left|m,l,p,e\right.\right]\cdot G_{Sa|M,L,P,E}\left(s_a^*\left|m,l,p,e\right.\right)\times f_{P,E|M,L}\left(p,e\left|m,l\right.\right)\cdot f_{L|M}\left(l\left|m\right.\right)\cdot f_M\left(m\right)\cdot dm\cdot dl\cdot dp\cdot de \tag{7}$$

$$\lambda_{Sa,Pulse}\left(s_a^*\right)=\nu\cdot\int_{t_p}\int_m\int_l\int_p\int_e P\left[Pulse\left|t_p,m,l,p,e\right.\right]\cdot G_{Sa,\text{mod}|Tp,M,L,P,E}\left(s_a^*\left|t_p,m,l,p,e\right.\right)\times f_{Tp|M,L,P,E}\left(t_p\left|m,l,p,e\right.\right)\cdot f_{P,E|M,L}\left(p,e\left|m,l\right.\right)\cdot f_{L|M}\left(l\left|m\right.\right)\cdot f_M\left(m\right)\cdot dt_p\cdot dm\cdot dl\cdot dp\cdot de \tag{8}$$

It appears that in order to account for all the geometrical parameters, a fifth order integral is necessary, leading to a computational effort significantly higher than ordinary PSHA. In fact, Equations (7) and (8) can be simplified neglecting some stochastic dependencies. More specifically:

- pulse probability is a function of geometrical parameters and is independent on magnitude and pulse period, that is, $P\left[Pulse\left|t_p,m,l,p,e\right.\right]=P\left[Pulse\left|l,p,e\right.\right]$;
- pulse period PDF (probability density function) is a function of magnitude only, that is, $f_{Tp|M,L,P,E}\left(t_p\left|m,l,p,e\right.\right)=f_{Tp|M}\left(t_p\left|m\right.\right)$;
- PDF of rupture and epicenter's position depends on the rupture length, that is, $f_{P,E|M,L}\left(p,e\left|m,l\right.\right)=f_{P,E|L}\left(p,e\left|l\right.\right)$;
- $G_{Sa,\text{mod}|Tp,M,L,P,E}$ is the modified GMPE, and $G_{Sa|M,L,P,E}$ is the original GMPE. Using R_{jb}, both functions are independent on the location of the epicenter, E;
- Pulse occurrence and absence are complementary: $P\left[NoPulse\left|l,p,e\right.\right]=1-P\left[Pulse\left|l,p,e\right.\right]$.

Regarding the DS case, pulse occurrence probability model considers such a rupture in its two-dimensional representation, which is easy to render analogous to the SS case (Figure 1). In fact, dip-slip three-dimensional representation is given in Figure 3 by two orthogonal sections, which may be useful to identify all the geometrical variables needed, and those that can be neglected in the hazard integrals.

For simplicity, the hypotheses of rectangular fault and rupture are taken here. B and L variables represent rupture surface sides (Figure 3a and Figure 3b). As for SS, they can be computed using Wells and Coppersmith (1994), which provide relationships as functions of

M for each of the two linear dimensions and the total rupture area. A possible option (used in the following applications) is to assume only one of the three mentioned parameters as a variable (dependent on event magnitude) and forcing a constant ratio for B and L.

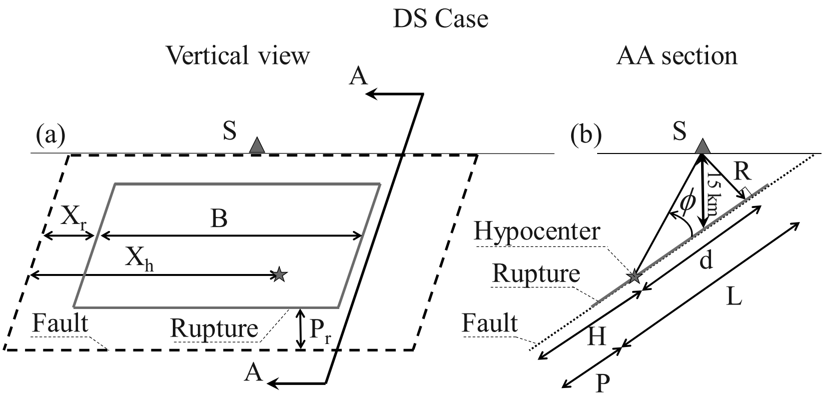

Figure 3. Dip-slip rupture representation.

X_r and X_h are the distances of the rupture and of the hypocenter from the fault bounds, respectively. The former is necessary for the identification of R_{jb} of the site for which hazard is computed, and it has to appear in the hazard integral, while the latter can be neglected. In fact, the hypocenter's position is important for parameters represented by section AA in Figure 3, where X_h does not appear.

Equations (7) and (8) can be adapted to the dip-slip case, that is, Equation (9) and Equation (10), considering the following differences:

1) position of the epicenter, E, has to be replaced by the position of the hypocenter (H);
2) the rupture's geometrical parameter dependent on event magnitude is assumed to be the rupture length L, in analogy with the SS case (in principle it should be the area of the rupture from which B and L are assumed to be deterministically dependent in the hypothesis of a constant B/L ratio);
3) to compute pulse occurrence probability X_r is also necessary (for computation of R, along L, P, and H);
4) the introduction of X_r requires knowledge of its PDF conditional to L, $f_{X_r|L}(x_r|l)$. Such distribution is, in principle, also conditional on the dimension and position of the fault, however they are considered as known.

$$\lambda_{Sa,NoPulse}\left(s_a^*\right)=\nu\cdot\int_m\int_l\int_p\int_h\int_{x_r}P\left[NoPulse|l,p,h,x_r\right]\cdot G_{Sa|M,X_r,L,P}\left(s_a^*|m,l,p,x_r\right)\times f_{P,H|L}\left(p,h|l\right)\cdot f_{X_r|L}\left(x_r|l\right)\cdot f_{L|M}\left(l|m\right)\cdot f_M\left(m\right)\cdot dm\cdot dl\cdot dp\cdot dh\cdot dx_r \tag{9}$$

$$\lambda_{Sa,Pulse}\left(s_a^*\right)=\nu\cdot\int_{t_p}\int_m\int_l\int_p\int_h\int_{x_r}P\left[Pulse|l,p,h,x_r\right]\cdot G_{Sa,\mathrm{mod}|Tp,M,X_r,L,P}\left(s_a^*|t_p,m,l,p,x_r\right)\times f_{Tp|M}\left(t_p|m\right)\cdot f_{P,H|L}\left(p,h|l\right)\cdot f_{X_r|L}\left(x_r|l\right)\cdot f_{L|M}\left(l|m\right)\cdot f_M\left(m\right)\cdot dt_p\cdot dm\cdot dl\cdot dp\cdot dh\cdot dx_r \tag{10}$$

1.2 NS-PSHA Applications

Because the marginal pulse occurrence probability is generally fairly low according to Iervolino and Cornell (2008), some applications of the proposed approach are required to quantitatively assess the effects of these modifications on seismic hazard estimates, and whether NS-PSHA is able to represent the pulse-like directivity threat adequately. To this aim, some applications are developed in terms of hazard, disaggregation and definition of design spectra. The geometrical configuration of the examples is analyzed in detail what follows, yet there are some common assumptions and working hypotheses that can be given beforehand:

1) because modification factor of GMPE accounting for pulse-like effects was fitted on the model of Boore and Atkinson (2008), this GMPE is used;
2) chosen IMs are the elastic spectral accelerations at all the spectral periods provided by used GMPE (from 0 sec, that is, Peak Ground Acceleration or PGA, to 10 sec); thus, results are first represented in terms of elastic response spectrum characterized by the same exceedance probability in a fixed time window for all ordinates (i.e., the uniform hazard spectrum, UHS);
3) a return period (Tr) equal to 475 years is assumed: in other words, computed intensity measures have an exceedance probability in 50 years equal to 10% (assuming a homogeneous Poisson process for earthquake occurrence; Cornell 1968, McGuire 2004);
4) the annual rate of earthquake occurrence (ν) on each fault is assumed to be equal to 0.05.

1.2.1 Applications

In the first application a SS fault and two different construction sites are considered: site S_1 is aligned with the fault strike and located five kilometers far from its upper edge, while site S_2 is on the center of the fault (Figure 5a). Both sites are expected to be prone to directivity, having a θ angle equal to zero (e.g., Somerville at al. 1997, Iervolino and Cornell 2008). Fault length is assumed equal to 200 km while rupture length (L) and rupture location on the fault (P) are considered as random variables. The distribution of the former, conditional to M, is lognormal (e.g., Wells and Coppersmith 1994), while that of the latter is uniform and limited by the fault dimension and the rupture length itself. In fact, for a given size, the rupture can be located in all the possible positions with a uniform probability distribution constrained by the fault limits. R_{jb} (Joyner-Boore distance) of the site is univocally defined once the rupture position is known. Also the hypocenter can ideally be located at any point within the rupture plane, but in order to reduce the numerical effort of the illustrative analyses, only three possible positions of the hypocenter were considered, that is, in the middle of the rupture plane or located at 30% of the total rupture length, measured from each of the two rupture extremities. Once the hypocenter location is defined, the s-parameter is known.

As a first case, the assumption that all the earthquakes generated by the source have a fixed (*characteristic*) magnitude equal to 7, was considered, and analyses were performed for both sites. Then, referring only to S_1, a Gutenberg-Richter-like distribution (Gutenberg and Richter 1944) for M was assumed with a negative slope equal to 1, and minimum and maximum M equal to 4.5 and 7.5, respectively. In order to reduce the computational effort, magnitude distribution was lumped by three discrete values of 5, 6 and 7; the corresponding associated discrete probabilities are respectively 0.9, 0.09 and 0.01.

Before discussing results, it may be useful to plot PDFs of pulse period and rupture length conditional on those magnitudes considered. Figure 4a shows that, if generated earthquakes have M equal to 5, it is very unlikely that forward directivity effects affect spectral periods higher than 3 sec, while a M 7 earthquake may influence a very large range of structural periods. This is because, according to the considered model, standard deviation of the logarithms is 0.59, which means about 60% coefficient of variation of Tp. Moreover, a M 5 earthquake has an associated rupture length significantly lower than the total length of the considered fault (Figure 4b), thus its probability of being located near the considered sites is lower than that associated to a M 7 event.

Results of NS-PSHA were compared to the corresponding ordinary hazard estimates; i.e., Equation (1). In Figure 5c, UHS are reported for ordinary $\left(Sa_{PSHA}\right)$ and modified $\left(Sa_{NS-PSHA}\right)$ analyses, and for characteristic and multiple M cases (magnitude distribution is applied only for S_1 and hereafter results for S_2, with characteristic magnitude, will be indicated with a cross superscript: Sa^{+}_{PSHA} and $Sa^{+}_{NS-PSHA}$). In Figure 5d the increments due to forward directivity effects, with respect to the ordinary case, are reported for each case.

Figure 5c shows that three analyzed cases yield different results: referring to PGA (for which directivity effects are negligible according to the assumed framework), it is apparent and expected that characteristic earthquakes generate higher accelerations for S_2 because of the lower distance from the rupture. The lower response spectrum for S_1, in the case of multiple-magnitude distribution, with respect to the case of characteristic magnitude, is of less intuitive understanding, but can be explained recalling that frequency of occurrence of M 5 is much higher than those of M 6 and M 7 (see Figure 4b).

Because of the aim of the study and working hypotheses, more attention is given to hazard increments with respect to the ordinary case, rather than on absolute acceleration values, and from Figure 5d the following may be pointed out.

(i) Hazard increments vary from about 25% to about 100% depending on the characteristics of the investigated case. This is mainly because of the applicability range of pulse occurrence probability model. For site S_1, a zero pulse probability is associated to a number of rupture's positions larger than S_2 (directivity effects are likely to occur more frequently for S_2 rather than for S_1 site). It can also be inferred that, under the hypotheses of uniform distribution of rupture position on the fault, geometry and magnitude occurrence may have significant effects on increment values. In fact, a similar quantitative application is provided in Shahi and Baker (2011) assuming a different geometrical configuration and a different magnitude occurrence model. In that case, amplification of hazard at 5 sec spectral acceleration was found equal to 40% for a site subjected to a SS fault and located on the fault.

(ii) The shape of hazard increments, the range of spectral periods in which increments are significant, and the period corresponding to the maximum directivity effect, can be directly derived from the model of magnitude occurrence on the fault. Such a dependency derives from the pulse period prediction model; e.g., the PDF of Tp conditional to M 7, has a median value equal to 3.7 sec, which is a good approximation of the period corresponding to maximum increment.

(iii) Similarly, because cases with multiple magnitudes are mostly affected by smaller and more frequent events, the period of maximum increments for the multiple-magnitude distribution is well correlated with the median value of *Tp* distribution for M 5 (0.43 sec), while increments are negligible for spectral periods higher than 4 sec.

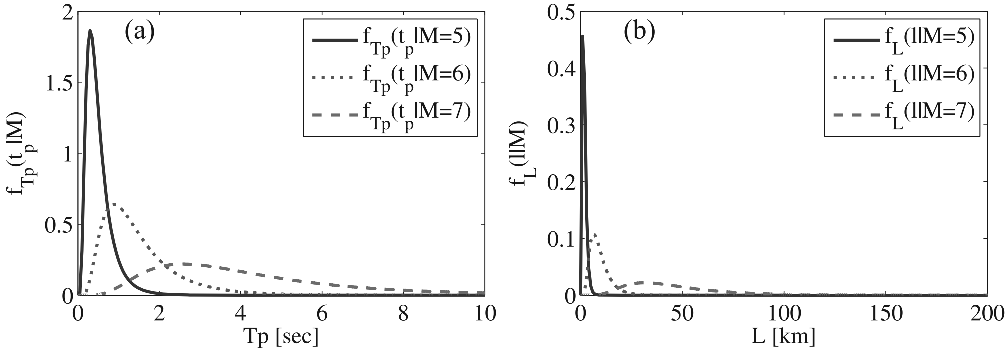

Figure 4. Probability density function of *Tp* (a); and rupture length L (b) conditional to M equal to 5, 6 and 7.

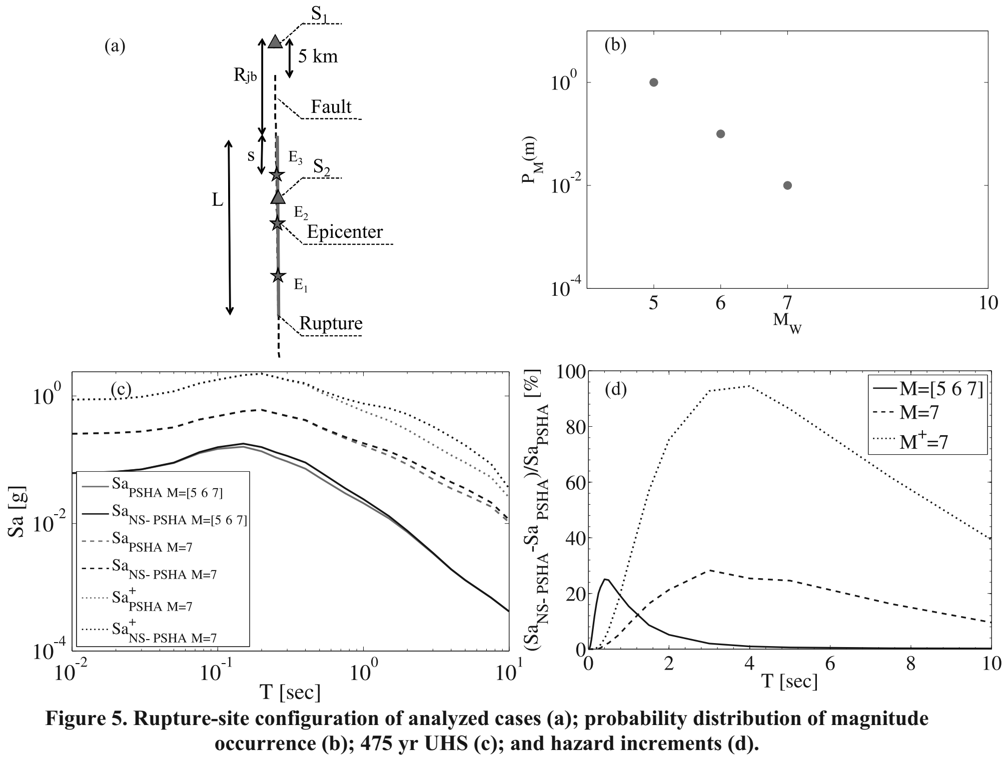

Figure 5. Rupture-site configuration of analyzed cases (a); probability distribution of magnitude occurrence (b); 475 yr UHS (c); and hazard increments (d).

A second example application, this time considering a DS rupture is also performed and similar to the SS examples, fixed planar dimensions (4000 km^2) and position are assumed for the seismic source. In fact, the fault is identified by the following angles: 50° and 90° for dip and rake, respectively; i.e., a *normal* fault. Both fault and rupture areas are assumed to be rectangular with the B/L ratio equal to two. An individual possible event magnitude, M 7, is assumed.

A site (S) placed within the surface projection of the rupture is considered and reported in Figure 3. In the same figure, size and position of the fault are represented by dotted lines. The rupture's area (A) (or its sides B and L) and rupture location (identified by distance to fault sides, X_r and P_r), are random variables. A-area is assumed to be a lognormally distributed conditional to M (Wells and Coppersmith 1994), and it is limited by the fault dimensions. For a given A-value, the rupture can be located in all the possible positions with a uniform PDF, yet constrained by the fault boundaries. R_{jb} for the site is univocally defined once the rupture is known. Ideally, the hypocenter can also be located everywhere on the rupture, but in order to reduce the computational demand, only three possible positions on the diagonal of the rupture were assumed. In Figure 3, the generic position of the hypocenters is identified as H. Once the hypocenter is also known, the d-parameter can be determined.

The hazard integral in case of dip-slip ruptures is reported in Equation (9) and Equation (10). UHS computed with ordinary and modified PSHA are reported in Figure 6a, while increments are shown in Figure 6b. The shape of hazard increments is analogous to SS, because it depends only on distribution of Tp given magnitude, which is the same in the two cases. Conversely, values of such increments are different because of the different geometrical configuration.

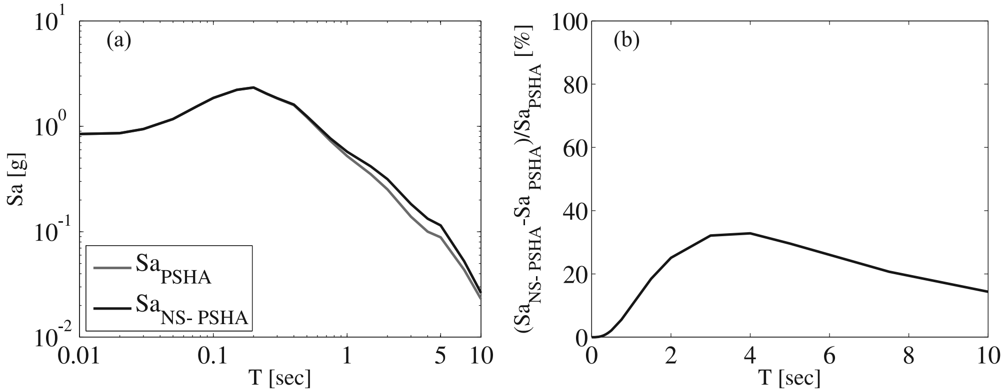

Figure 6. 475 yr UHS with modified and classical PSHA (a); and increments due to directivity effects (b).

1.2.2 Pulse-like disaggregation

Disaggregation is complementary to hazard analysis (McGuire 1995), and it is useful in identifying probability-based design scenarios (e.g., *design earthquakes*) and providing a rational basis for the selection of representative seismic input (e.g., ground motions) to be used in dynamic analyses of structures. In fact, it is typically used to compute the distribution of magnitudes, distances, and ε values contributing to occurrence or exceedance of some ground motion intensity level $\left(s_a^*\right)$. This issue is especially important in near-source conditions in which criteria for design scenarios and ground motion record selection are not well established yet. In fact, classical disaggregation equations can be modified in accordance with the expressions of NS-PSHA, to provide contribution to hazard of the main variables; i.e., the probability that a ground motion intensity level is caused by a pulse-like ground

motion, and the distribution of pulse periods associated to it, or the probability that a set of geometrical parameters determines the exceedance of a hazard threshold.
Referring to the hypotheses of a single fault, and comprising site-source geometrical parameters with the $\underline{Z}$ vector, disaggregation's most synthetic result is:

$$f\left(m,\underline{z},\varepsilon \middle| Sa > s_a^*\right) = \frac{\nu \cdot P\left[Sa > s_a^* \middle| m,\underline{z},\varepsilon\right] \cdot f\left(m,\underline{z},\varepsilon\right)}{\lambda_{s_a^*}} \tag{11}$$

in which $P\left[Sa > s_a^* \middle| m,\underline{z},\varepsilon\right]$ is the probability of exceeding the hazard level s_a^* given magnitude, ε and all the geometrical variables of the problem.
Marginal disaggregation distribution of pulse period can also be obtained considering only the case of pulse occurrence as reported in Equation (12):

$$f\left(t_p \middle| Sa > s_a^*, Pulse\right) = \frac{\nu \cdot P\left[Sa > s_a^* \middle| t_p, Pulse\right] \cdot f\left(t_p \middle| Pulse\right)}{\lambda_{s_a^*|Pulse}} \tag{12}$$

where $\lambda_{s_a^*|Pulse}$ is the MAF of exceeding the s_a^* value given that the pulse occurs, while $\lambda_{s_a^*,Pulse}$ of Equation (2) is the MAF of the joint event of exceeding s_a^* and occurrence of pulse.
As a final result, probabilities of observing pulse occurrence or absence given the exceedance of s_a^* can be computed. They give information about how likely exceedance is due to forward directivity effects. The two terms are mutually exclusive and complementary to one; analytical expression of the former is reported in Equation (13).

$$P\left[Pulse \middle| Sa > s_a^*\right] = \frac{\nu \cdot P\left[Sa > s_a^* \middle| Pulse\right] \cdot P\left[Pulse\right]}{\lambda_{s_a^*}} \tag{13}$$

Referring to the hazard result of the SS case with a multiple-magnitude distribution, the disaggregation distributions in Equation (14) and Equation (15) were computed for Sa (1 sec):

$$f\left(m,r,\varepsilon,t_p \middle| Sa > s_a^*, Pulse\right) = \frac{\nu \cdot P\left[Sa > s_a^* \middle| Pulse, m, r, \varepsilon, t_p\right] \cdot f\left(m,r,\varepsilon,t_p \middle| Pulse\right)}{\lambda_{s_a^*|Pulse}} \tag{14}$$

$$f\left(m,r,\varepsilon \middle| Sa > s_a^*, NoPulse\right) = \frac{\nu \cdot P\left[Sa > s_a^* \middle| NoPulse, m, r, \varepsilon\right] \cdot f\left(m,r,\varepsilon \middle| NoPulse\right)}{\lambda_{s_a^*|NoPulse}} \tag{15}$$

where $f\left(m,r,\varepsilon,t_p \middle| Sa > s_a^*, Pulse\right)$ is the probability of $\{m,r,\varepsilon,t_p\}$ being the causative vector for s_a^* in the case of pulse occurrence, and $f\left(m,r,\varepsilon \middle| Sa > s_a^*, NoPulse\right)$ is the probability of $\{m,r,\varepsilon\}$ being the causative vector in the case of no pulse occurrence in ground motion.

Because some of the considered PDFs cannot be clearly represented (being defined in spaces of dimensions larger than $\Re^3$) numerical integration was used in order to obtain marginal distribution of easier graphical handling; e.g., Equation (16).

$$f\left(m,r\middle|Sa>s_a^*,Pulse\right)=\int_{t_p}\int_{\varepsilon}f\left(m,r,\varepsilon,t_p\middle|Sa>s_a^*,Pulse\right)\cdot d\varepsilon\cdot dt_p \tag{16}$$

In Figure 7 the following disaggregation PDFs are reported: magnitude and distance conditional on pulse occurrence (a), and non-pulse occurrence (b); ε values conditional on pulse occurrence (c), and non-pulse occurrence (d); pulse period in case of pulse (e). All PDFs refer to Tr = 475 yr.

Distance disaggregation, conditional to pulse occurrence, is limited by the definition domain of the pulse probability model. Conversely, the same disaggregation plot, but conditional to absence of pulse, shows non-negligible hazard contributions for larger distances (however, data for distances larger than 50 km are not reported).

Mean ε, conditional to pulse occurrence, is lower than ε conditional to pulse absence (0.5 and 1.0, respectively), because the first disaggregation is computed by the modified GMPE in which Tp effects are applied on the modified predicted median. Finally, the Tp disaggregation distribution has a similar shape of the PDF of Tp conditional to M 5; however, because disaggregating hazard refers to 1 sec spectral acceleration, the mean is moved from 0.5 sec to 0.9 sec.

1.2.3 Design scenarios

It is well known that the uniform hazard spectrum (UHS) does not account for correlation of spectral ordinates. In fact, UHS is an *envelope* spectrum, which may not be representative of any specific ground motion. The problem was studied for ordinary conditions by Baker and Cornell (2006). The proposed solution consists of the *conditional-mean spectrum considering* ε (CMS-ε); i.e., a spectrum in which the spectral ordinate associated to the structural period of interest has a defined exceedance probability and all the others are computed from disaggregation for the considered structural period, and account for correlation.

In NS-PSHA, resulting UHS is the envelope of many seismic scenarios in which cases of pulse occurrence are combined with cases of pulse absence weighted by the pulse probabilities. Moreover, terms characterized by pulse occurrence account for many different pulse periods, with the PDF of *Tp*, conditional on event magnitude, having quite a flat shape. As a consequence, the effect of pulses on the final spectrum is spread over a large range of periods as shown in the previous examples. Thus, the well-known limitations of UHS may be even worse in NS cases. On the other hand, CMS-ε is not of easy direct application in the NS-PSHA framework because, to date, an assessment of correlation of spectral ordinates fitted on pulse-like records is not yet available (if not indirectly via the GMPE modification factor), and because it is not univocally defined how to account for Tp in the CMS-ε procedure.

Referring to the hazard result of the SS case with a multiple-magnitude, all the mean values of disaggregation distributions necessary to compute discussed design spectra have been computed for three values of spectral periods representative of short (0.5 sec), medium (1.0 sec) and long (2.0 sec) structural periods, as reported in Table 1.

Table 1. Average disaggregation values for application 1.

	$M_{p,\mu}$	$R_{p,\mu}$	Tp_{μ}	$M_{np,\mu}$	$R_{np,\mu}$	$P\left(Pulse \middle\vert Sa > s_a^*\right)$	$P\left(NoPulse \middle\vert Sa > s_a^*\right)$
T=0.5	5.2	10.9	0.71	5.8	27.9	0.54	0.46
T=1.0	5.3	10.5	0.88	5.9	37.0	0.42	0.58
T=2.0	5.4	10.2	1.12	6.1	48.7	0.27	0.73

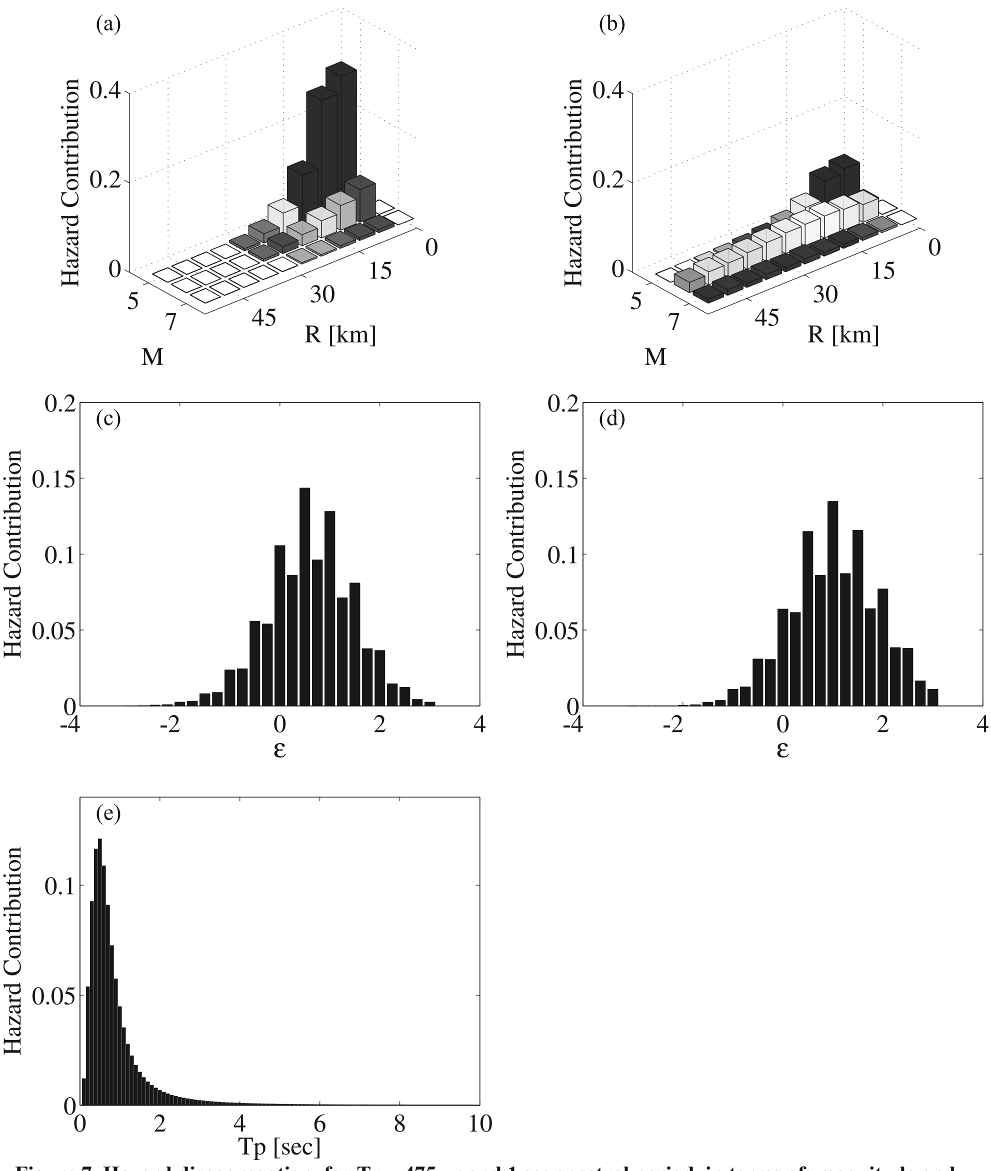

Figure 7. Hazard disaggregation, for Tr = 475 yr and 1 sec spectral period, in terms of: magnitude and distance conditional to pulse occurrence (a) and pulse absence (b); ε values conditional to pulse occurrence (c) and absence (d); pulse period (e).

Here a procedure for identification of design spectra is preliminarily proposed. It consists of the identification of two different spectra representative of the ordinary scenario (i.e., in which no forward directivity effects occur) and the pulse-like scenario (i.e., which accounts entirely for the effects of forward directivity). The latter will be referred to as *close-impulsive spectrum* or CIS.

In Figure 8 UHS is compared with the two computed spectra defined before for pulse-like, $Sa_p\left(M_{p,\mu},R_{p,\mu},Tp_\mu\right)$, and non-pulse-like, $Sa_{np}\left(M_{np,\mu},R_{np,\mu}\right)$, scenarios. Comparisons are reported in terms of pseudo-accelerations (in logarithmic scale) and displacements $\left(Sd\right)$ spectra. It is noted that Sa_p and Sa_{np} have significantly different causative magnitude and distance, therefore they are representative of different earthquakes. In order to underline the characteristic shape of CIS, a spectrum computed from ordinary GMPE with magnitude and distance values of pulse-like scenario $Sa_{np}\left(M_{p,\mu},R_{p,\mu}\right)$, and scaled to the same PGA value of CIS, is also reported in each plot.

Significant ranges of periods are affected by differences between $Sa_p\left(M_{p,\mu},R_{p,\mu},Tp_\mu\right)$ and $Sa_{np}\left(M_{p,\mu},R_{p,\mu}\right)$ spectra, this is because of the shape of modification factor. Moreover, for spectral periods different to the one disaggregation refers to, differences between the proposed spectra and UHS can be significant.

These near-source scenarios may also help in assessing structural performance by means of non-linear dynamic analysis. In fact, record selection for near-source sites should account for pulse-like and non-pulse-like records. The more straightforward way to address this issue would be to select records with a required M, R and Tp , from disaggregation.

1.3 Sensitivity analysis of directivity effects on PSHA

In the preceding sections, the subject of *hazard increments* (HIs) due to NS-PSHA with respect to the ordinary PSHA was broached. Results therein show that numerical values of such increments are dependent on geometrical parameters determining the source-to-site configuration, such as fault dimensions and site location. In order to deepen this issue, additional illustrative applications are presented retaining the hypothesis of known fault geometry which is common to all the previous works regarding this topic.

As already discussed, rupture length and rupture position are, in principle, random variables; however, these applications are implemented under the simplifying hypothesis of fixed rupture dimension and position. Such hypothesis appears to be acceptable if a single magnitude can be generated by the considered fault (as assumed in the following). The hypothesis of uniform distribution of the hypocenter position on the rupture is retained.

Two cases of single generated magnitude, equal to 5.0 and 6.0, are considered. Starting from the Wells and Coppersmith (1994) prediction model which assumes a lognormal distribution (base 10) of rupture dimensions, two fixed source lengths are associated to each magnitude scenario. More specifically, fifth and fiftieth percentiles of rupture length distributions are chosen: 2.0 and 3.4 km SS lengths are associated to *M* 5.0, while 8 and 14 km are obtained for the *M* 6.0 scenarios.

In the following, seismic hazard is estimated via the UHS for a return period (*Tr*) equal to 475 years; used GMPE is again that of Boore and Atkinson (2008); mean annual rate of earthquake occurrence on the fault is 0.05. Attention is focused on *HI*s if ordinary PSHA is replaced by its modified version for NS sites. In most of the advanced seismic codes worldwide, seismic action is characterized by UHS computed by ordinary PSHA and quantification of HIs in NS conditions is useful to understand the consequences of neglecting pulse-like directivity effects in hazard analysis. For each spectral period T and for the chosen $Tr = 475$, hazard increments $HI(T)$ are analytically defined by the percentage factor in Eq. (17).

$$HI(T) = \frac{S_a(T)_{NS-PSHA} - S_a(T)_{PSHA}}{S_a(T)_{PSHA}} \cdot 100 \qquad (17)$$

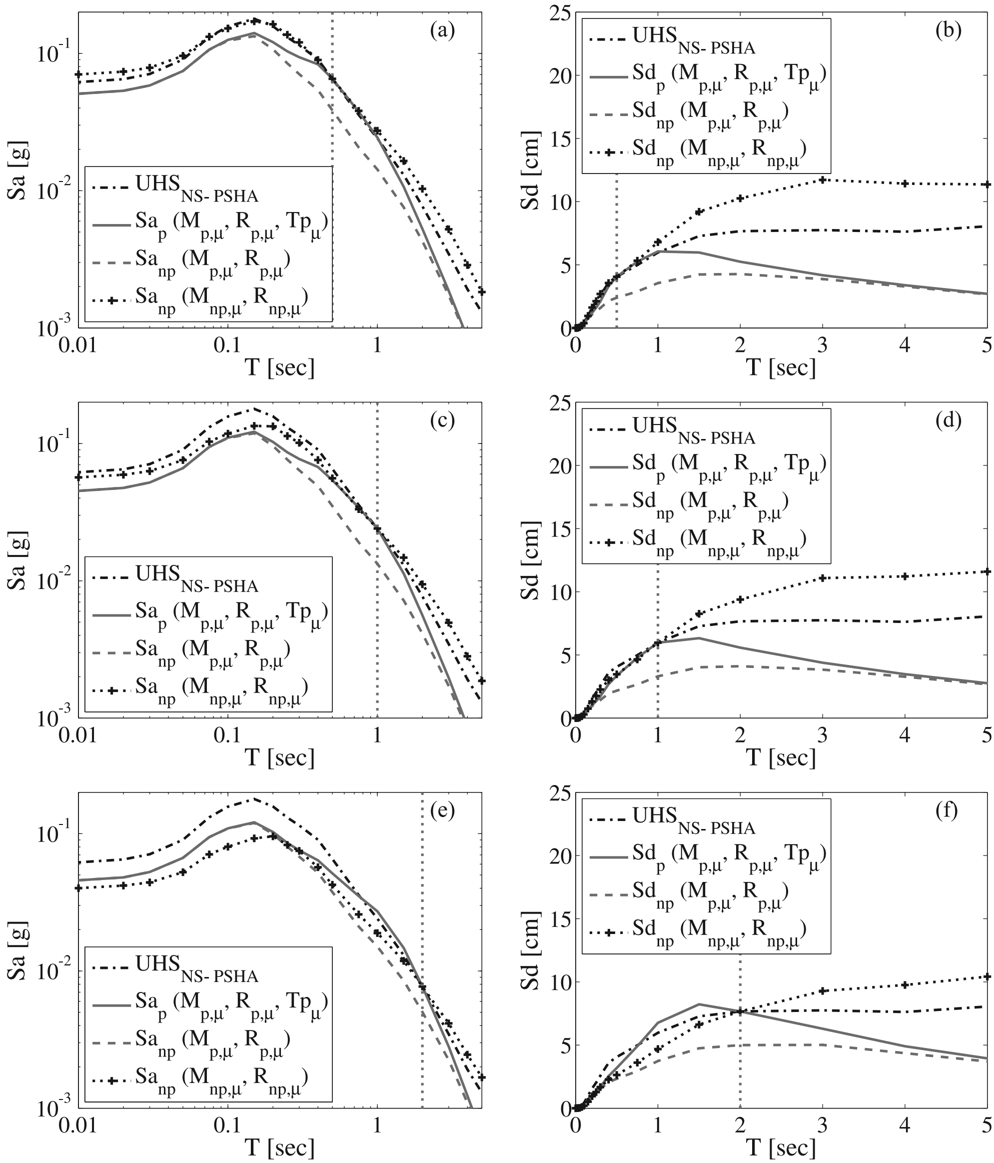

Figure 8. UHS and proposed conditional mean spectra for T equal to 0.5 (a and b); 1.0 (c and d); and 2.0 sec (e and f), in terms of accelerations and displacements.

More specifically, contours of *HI*s in a wide area around the seismic sources are studied, in order to avoid being constrained to site-specific results. In fact, such contours are also useful for identification of the distance from the fault beyond which NS-PSHA provides negligible differences with respect to PSHA.
UHS and HIs were computer for 64 sites influenced by the 8 km SS fault. Studied sites were identified by a 5 km by 10 km lattice over a 40 km by 60 km area. HIs are represented in terms of contours of maximum values at each site (Figure 9a). Similarly, contours of maximum HIs due to a 14 km fault generating M 6.0 events are reported in Figure 9b. In both cases, maximum HIs correspond to 1 s spectral period.
The case of *M* 5.0 generating fault is also studied with the fault length of 2.0 and 3.4 km (Figures 9c and 9d, respectively) chosen from Wells and Coppersmith (1994). In these cases, maximum HIs are computed for 0.4 s spectral period.
In all the plots, sites with distance from the rupture higher than 30 km are shaded because a zero pulse probability is associated to them (ordinary and NS-PSHA are formally equivalent).
It is also to note that contours of HIs are similar even if different generated magnitudes are considered. The reason is in the absence of magnitude dependency in the used pulse occurrence probability model. Two symmetry axes are apparent for the contours, while the elongated shape of HI contours comes from the dependency of pulse occurrence probability on the θ parameter. The minor differences between Figs. 9a, 9b, 9c and 9d depend only on the different rupture lengths. In all cases, the red dotted lines approximate an area near the fault with HIs higher than 10%: these lines are 15 km far from the fault and are parallel to it. This means that, at least in these applications, directivity effects can be considered *negligible* for all the sites external to the selected zone (the threshold is arbitrarily chosen).
Such results apply, without any additional computation, to all SS faults with length between 2 and 14 km. Furthermore, these seem to allow to identify the zone in which directivity effects are relevant, according to the considered NS-PSHA procedure, via a preliminary analysis, and independently of the specific characteristics of the considered fault.

1.4 Discussion on NS-PSHA and design scenarios

NS-PSHA was applied to cases of sites subjected to single seismic sources with SS or DS rupture mechanisms, and different magnitude distributions. It was found that the range of spectral periods in which hazard increments due to forward directivity effects are significant, shapes of such increments, and periods corresponding to the maximum increment, can be directly derived from the model of magnitude occurrence on the fault. Such a dependency derives from the relationship between pulse period and event magnitude. Thus, if earthquakes are generated with a Gutenberg-Richter relationship, lower structural periods are those most influenced by directivity effects, due to the higher recurrence frequency of smaller magnitudes.
The amount of hazard increments seems to be largely dependent on the characteristics of the studied cases (geometry above all). Because the pulse period prediction model depends on the event magnitude with a significant heterogeneity, it was also shown that HIs often affect a large range of periods.
HIs are found to be strongly dependent on the considered spectral ordinate. Periods to which maximum *HI*s are associated depend on the magnitude of generated events. Thus, more *hazardous* spectral periods can be predicted with the knowledge of the fault's characteristics in terms of generated magnitudes. Shapes of contours are similar even if different generated magnitudes are considered and minor differences depend only on the different rupture lengths. For investigated cases, it was also possible to geometrically identify the area affected by relevant directivity effects, independently of the specific characteristics of the considered fault in terms of event magnitude. Results suggest that, increasing the number of analyses, more general rules can be identified.

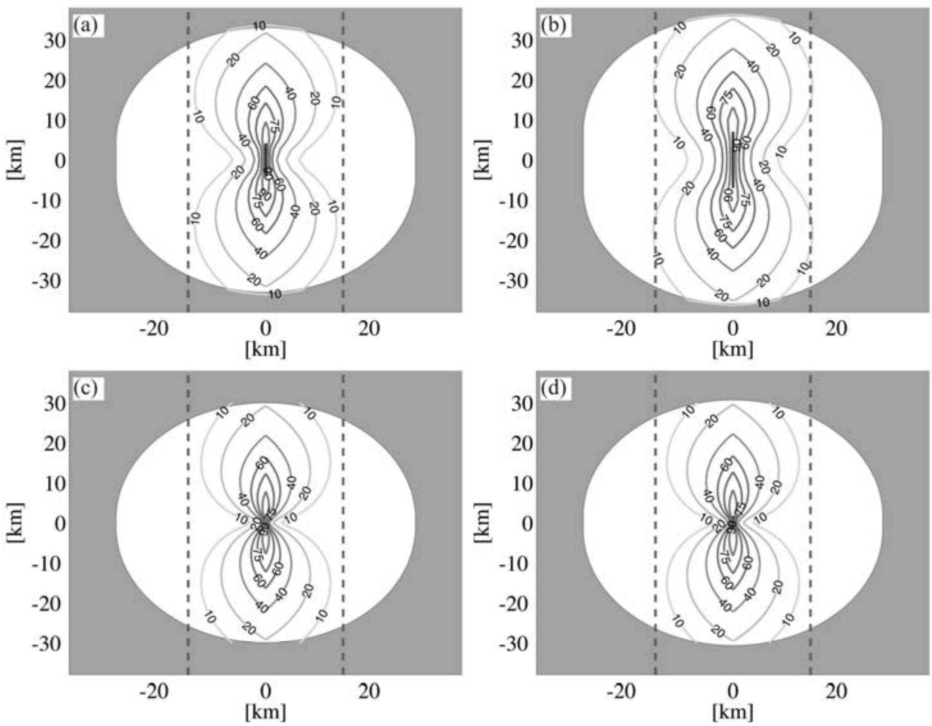

Figure 9. Contours of HI(1s) for the 8 (a) and 14 km (b) faults generating M 6.0 events and contours of HI (0.4s) for the 2.0 (c) and 3.4 km (d) faults generating M 5.0 events.

Regarding design scenarios in near-source conditions, known limits of UHS were found of larger importance in near-source conditions with respect to ordinary PSHA, and a procedure was explored based on NS hazard disaggregation. A *close-impulsive spectrum* was preliminarily discussed for the pulse-like hazard part, to complement an *ordinary* spectrum for the non-pulse-like case. These attempts may be helpful in the research for the identification of engineering ground motion characteristics at near-source sites.

2 INELASTIC DISPLACEMENT RATIO OF NS PULSE-LIKE GROUND MOTIONS

Another particular feature of NS pulse-like ground motions, which was not stressed in the preceding discussion, is the fact that the inelastic to elastic seismic spectral displacement ratio for the fault-normal component of such records may visibly depart from the *equal displacement rule*, and can be higher than that of ordinary motions. Such increments are mostly observed in a range of periods between about 30% and 50% of pulse period.

Having already discussed issues related to elastic seismic demand and how to account for them in probabilistic seismic hazard analysis, the subject will now turn to inelastic demand. To this end, the inelastic to elastic displacement ratio, or C_R, is studied by means of semi-empirical relationships. In Equation (18), $S_{d,e}(T)$ is the elastic spectral displacement at period T and $S_{d,i}(T)$ is its inelastic counterpart for a given strength reduction factor (usually indicated as R or R_s).

$$C_R = S_{d,i}(T) / S_{d,e}(T) \tag{18}$$

Current static nonlinear structural assessment procedures (e.g., Fajfar 1999) rely on prediction equations for this kind of parameters to estimate inelastic seismic demand given the (elastic) seismic hazard. Because such relationships have to be estimated semi-empirically, in those cases where peculiar features in ground motions are expected, it is necessary to investigate whether they may show special trends. As a matter of fact, inelastic displacement for near-source conditions was studied already by a number of researchers (e.g., Baez and Miranda 2000, Akkar et al. 2004). A more up to date study with respect to this issue and dealing with pulse-like records is that of Ruiz-Garcia (2011), which in part also motivated this study by pointing out the need for further investigation on the C_R functional form.

This being the scope of the study presented in this section, a series of bilinear (with 3% post-elastic stiffness) single-degree-of-freedom systems were analyzed when subjected to: (i) sets of FN (fault normal) impulsive records; (ii) the corresponding FP (fault parallel) components; and (iii) a set of ordinary ground motions. The SDOF systems where designed to cover different nonlinearity levels, measured by means of R. The latter is given in Equation (19), where: $S_{a,e}(T)$ is the elastic spectral acceleration, *m* is the mass of the SDOF system, and F_y is the yielding strength in the case of bilinear hysteresis' backbone (yielding strength was changed record by record to have uniform strength reduction factor, that is, a constant R approach). Results were employed to fit the observed trends, which were found to be different if compared to those of ordinary and FP records (at least in terms of amplitude in this latter case), as a function of the T over T_p ratio.

$$R = S_{a,e}(T) \cdot m / F_y \qquad R = \{2, 3, ..., 8\} \tag{19}$$

2.1 Dataset and empirical evidence

Pulse-like records considered are a set, already assembled in Chioccarelli and Iervolino (2010) who made use of the algorithm of Baker (2007) to identify individual records as impulsive. That algorithm is based on wavelets to extract the pulse at the beginning of a record and to determine its pulse period T_p. It also provides a score, a real number between 0 and 1, which is function of the energy and amplitude of the pulse with respect to the recorded ground motion. In fact, the dataset considered herein is comprised of impulsive FN components from the NGA database (http://peer.berkeley.edu/nga/) within 30 km from the source and with pulse score equal or larger than 0.85. This is the dataset also employed by Iervolino and Cornell, to which L'Aquila records analyzed by Chioccarelli and Iervolino (2010) were added, plus the recording of the same event by AQU station of the Mediterranean Network (MedNet, http://mednet.rm.ingv.it/) which has been made available more recently.

For comparison, also records identified as non-pulse-like (i.e., ordinary) according to the discussed procedure, yet within 30 km from the source, were considered. In Table2 datasets, in terms of number of earthquake events and records, are summarized. Table 3 reports the distribution of pulse-like records in T_p bins. Moment magnitude ranges from 5.2 to 7.9 and the vast majority of records was from C and D NEHRP site classification.

Table 2. Pulse-like and ordinary datasets.

Mechanism	Earthquakes	Records	Earthquakes with Pulse-Like Records	Pulse-Like Records
Strike-Slip	22	133	12	34
Non-Strike-Slip	23	242	12	47
Total	*45*	*375*	*24*	*81*

Table 3. Distribution of pulse-like records in Tp bins.

T_P	[0s, 1s[	[1s, 2s[	[2s, 3s[	[3s, 4s[	[4s, 5s[	[5s, 6s[	[6s,12s[
Number of records	22	20	8	11	10	5	5

The number of records from strike-slip events is 133, the records identified as pulses in the given dataset are 34. Records from other faulting mechanisms are in a unique category due to their relative paucity summing up to 375, 81 of which are identified as containing pulses. Note that in the following no distinction of ground motion with different source parameters is considered, as results of in Chioccarelli and Iervolino (2010) do not support such a distinction. In Figure a FN elastic spectra, normalized to peak ground acceleration (PGA), are given for pulse-like records considered herein with T_P between 1s and 2s (*Average Pulse*) and for ordinary ground motions (*Average Non Pulse*). In Figure 10**Figure** b, C_R for R equal to 4 is also given for pulse-like and non-pulse-like records (*Pulse - FN* and *Non Pulse – FN*, respectively). For comparison, also C_R for the FP components of the pulse-like FN records (which not necessarily are pulse-like, even if indicated as *Pulse - FP*), are shown. The figures allow to appreciate the systematic differences among the considered classes, which have already received mention in the above sections. Moreover it appears that FP records have a shape similar to FN in the low T/T_p range, yet with lower amplitudes. Same results hold for other R-values not shown.

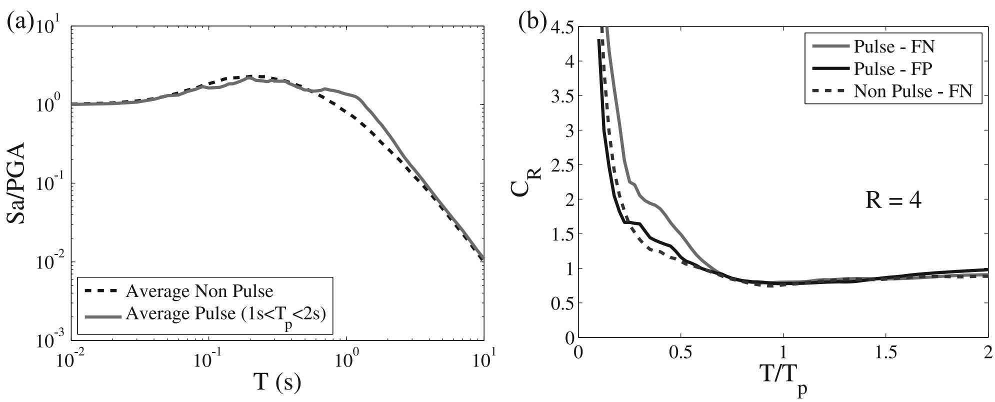

Figure 10. (a) Elastic 5% damped spectra for FN pulse-like with 1s<T_P<2s and ordinary records; (b) empirical C_R for FN pulse-like records, for their FP components, and for ordinary records, at R = 4.

2.2 Functional form and regression strategy

As already mentioned, among other researches who have looked at near-source spectral amplification, attention here is focused on the work of Ruiz-Garcia (2011), who, based on empirical evidence, proposed a functional form of C_R of the type in Equation (20) to account for a dominant frequency in ground motion.

$$C_R = 1 + \theta_1 \cdot \left(T_g/T\right)^2 \cdot \left(R-1\right) + \theta_2 \cdot \left(T_g/T\right) \cdot \exp\left\{\theta_3 \cdot \left[\ln\left(T/T_g - 0.08\right)^2\right]\right\} \tag{20}$$

In this equation T_g is the predominant period of ground motion, that is, the one corresponding to the peak of the 5% damped velocity spectrum. Because of the strong correlation that exists

between the two period measures, T_p and T_g , in the following T/T_p will be used instead of T/T_g.
As was noted by Ruiz-Garcia (2011) and is also confirmed in the following, Equation (20) is able to capture the shape of inelastic to elastic displacement ratio at $T/T_p \approx 1$, while it is not able to capture the bump in the low T/T_p range. This calls for a modification of the prediction equation for C_R, which is investigated herein. Equation (21) consists of adding another term, to reflect the C_R trend in the low T/T_p range (R dependency in the argument of last term is explained later). The resulting relationship has another bump (shifted and representing a peak rather than a valley). This equation has the same analytical form of that proposed by Ruiz-Garcia and Miranda (2006) for C_R in the case of soft soil sites. As a matter of fact, SDOF response also is dominated by specific frequencies of ground motion in that case too, yet of a different nature.

$$\begin{aligned} C_R = 1+\theta_1\cdot\left(T_p/T\right)^2\cdot(R-1)+\theta_2\cdot\left(T_p/T\right)\cdot\exp\left\{\theta_3\cdot\left[\ln\left(T/T_p-0.08\right)\right]^2\right\}+ \\ +\theta_4\cdot\left(T_p/T\right)\cdot\exp\left\{\theta_5\cdot\left[\ln\left(T/T_p+0.5+0.02\cdot R\right)\right]^2\right\} \end{aligned} \tag{21}$$

To determine the coefficients of Equation (21) for each of the R-values considered, nonlinear-segmented regressions were applied for $0.1\leq T/T_p\leq 2$. Fitting was performed in two steps, such that the first three terms of Equation (21) were determined in the initial phase, then the residuals were computed and fitted via the fourth term; this was also to compare with Equation (20), and to determine the efficiency of the considered functional form.
The initial phase of the two-step procedure was to get coefficients for Equation (20), that is, first three terms of Equation (21), Table 4, for the bilinear SDOF systems herein investigated. This was carried out not considering data within the]0.35,0.775[T/T_p range. In fact, it fitted those segments of the forward-directivity data that seem to be captured by a relationship of the type in Equation (20); Figure 11a.

Table 4. Coefficient estimates for Equation (20).

	R = 2	R = 3	R = 4	R = 5	R = 6	R = 7	R = 8
θ_1	0.0151	0.0209	0.0211	0.0198	0.0184	0.0170	0.0157
θ_2	-0.146	-0.230	-0.293	-0.343	-0.384	-0.417	-0.445
θ_3	-2.878	-2.360	-2.375	-2.437	-2.444	-2.441	-2.434

The second step was to derive the residuals $\left(\varepsilon_{C_R}\right)$ of actual data with respect to Equation (20) and to fit them by the term in Equation (6), in which $\overline{C_R}$ is the data average, and $\hat{C}_R$ is the estimate from the model. This is similar to what was done by Baker (2008) to fit pulse-like ground motion elastic residuals to modify ordinary ground motion prediction equations. Table 5 reports resulting coefficients.

$$\varepsilon_{C_R}=\overline{C_R}-\hat{C}_R\approx\theta_4\cdot\left(T_p/T\right)\cdot\exp\left\{\theta_5\cdot\left[\ln\left(T/T_p+0.50+0.02\cdot \mathrm{R}\right)^2\right]\right\} \tag{22}$$

Table 5. Coefficient estimates for Equation (22).

	R = 2	R = 3	R = 4	R = 5	R = 6	R = 7	R = 8
θ_4	0.066	0.146	0.193	0.217	0.224	0.232	0.242
θ_5	-47.931	-40.966	-32.697	-27.173	-20.973	-17.211	-15.177

Based on Figure 11b, it should be noted that the amplification observed in pulse-like records when compared to ordinary ground motions, is around a T/T_p value whose location is a function of R. To capture this effect the linear term $\left(0.50{+}0.02 \cdot \mathrm{R}\right)$ appears in Equation (21) and Equation (22).

Standard deviation $\left(\sigma_{C_R}\right)$ was also fitted as a function of T/T_p and R. In fact, functional form of the same type of Equation (21) was fitted on C_R plus one standard deviation data. Then, the relationship for σ_{C_R} the standard deviation was derived, Equation (23), whose coefficients are given in Table 6. This may be considered the statistic of a lognormal random variable as it was found to be a more appropriate probability density function, rather than Gaussian, for the observed data.

$$\sigma_{C_R} = 0.1 + s_1 \cdot \left(T_p/T\right)^2 \cdot \left(R-1\right) + s_2 \cdot \left(T_p/T\right) \cdot \exp\left\{\theta_5 \cdot \left[\ln\left(T/T_p + 0.50 + 0.02 \cdot R\right)\right]^2\right\} \quad (23)$$

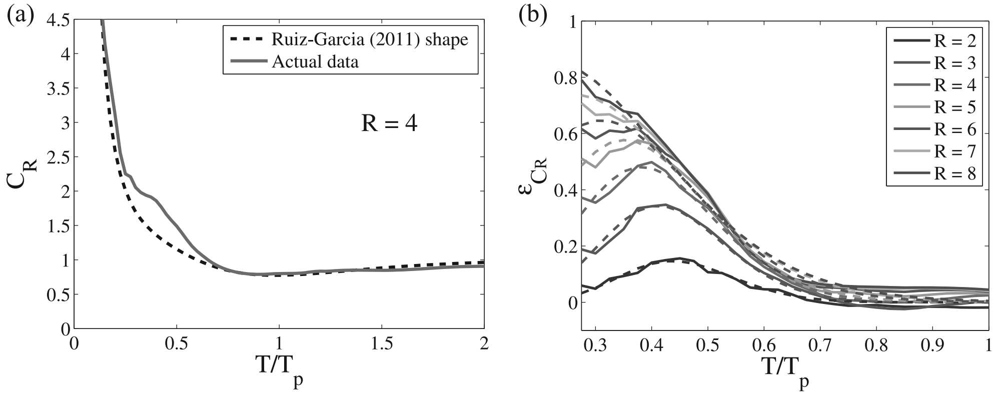

Figure 11. (a) Fitting of Equation (20) for pulse-like FN data (R = 4) outside the]0.35,0.775[T/T_p range, and (b) fitting of Equation (22) for selected R-values.

In Figure 12a the composition of fitted coefficients of Table and Table to obtain the prediction relationship of the type in Equation (21), is given for all R-values investigated. As an example, actual data and fitted model are compared for R equal to 4 in Figure 12b, in terms of average C_R and C_R plus one standard deviation. Goodness of fit holds for other R-values not shown.

Table 6. Standard deviation coefficients for Equation (23).

	R = 2	R = 3	R = 4	R = 5	R = 6	R = 7	R = 8
s_1	0.0170	0.0278	0.0306	0.0294	0.0262	0.0232	0.0208
s_2	0.0635	0.0837	0.0657	0.0516	0.0516	0.0485	0.0400

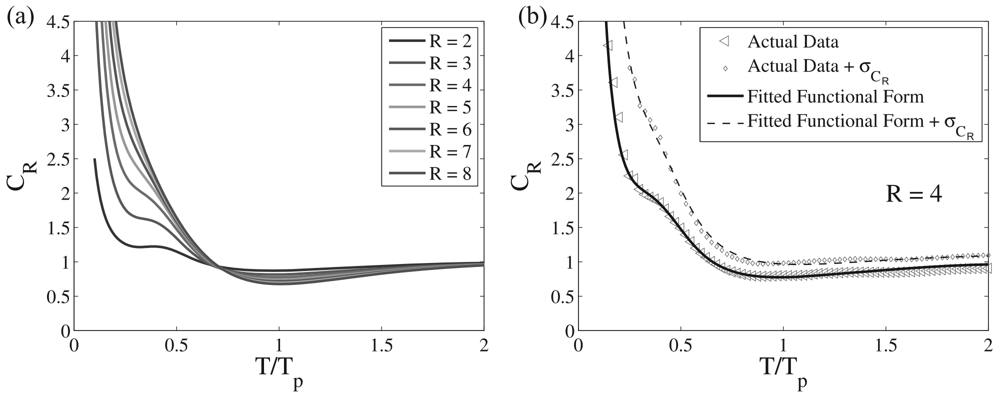

Figure 12. (a) Double-bump fitted C_R trends; (b) comparison with empirical data for R = 4.

2.3 Discussion on inelastic displacement ratios for NS pulse-like ground motions

The functional form for prediction of near-source pulse-like inelastic displacement ratio, which was investigated in this study, is potentially useful for structural assessment by means of static nonlinear procedures in near-source conditions, and complements current efforts to model effects of forward directivity on elastic seismic structural demand, which was presented in a preceding section.

An asymmetric-bell term, centered at different points depending on R, was suitable to fit C_R in the low T/T_p range. This resulted in the proposed model exhibiting two opposite bumps in two different spectral regions, and builds up consistent with recent literature on the same topic and on what observed for soft soil site records, which are also characterized by a predominant period.

These results may be of some help in investigations concerning design procedures specific for near-source conditions, given that the pulse period is available from design scenarios based on near-source probabilistic seismic hazard analysis.

3 DISPLACEMENT DAMPING MODIFICATION FACTORS FOR PULSE-LIKE GROUND MOTIONS

The effect of damping on the displacement demand of SDOF systems subjected to pulse-like excitations has been investigated. The damping modification factor (*DMF)*, defined as the ratio between the displacement demand of a SDOF system with damping ratio other than 5% and that of the corresponding 5%-damped SDOF is calculated using near-fault pulse-like records from different earthquakes. Values of the damping ratio ranging from 2% to 50% are considered. With the aim of highlighting the effect of the pulse, the pulse-like records are grouped into bins corresponding to different values of the pulse period T_p. A prediction model was then proposed to estimate the *DMF* for this type of records, and the model parameters calibrated for each considered T_p bin.

Reduction of spectral ordinates due to damping is influenced by various factors (see Mollaioli *et al.* 2013 and references therein), such as period of vibration, earthquake magnitude, ground motion duration and number of cycles, distance to the fault, site condition and near-fault effects. The large-amplitude velocity pulses, which may be observed at sites close to the fault rupture, are expected to decrease the effectiveness of damping in reducing the structural response. For this reason Priestley (2003) proposed the following expression for the *DMF* in near-fault regions:

$$\mathrm{DMF}(\xi) = \left(\frac{10}{5+\xi}\right)^{0.25} \tag{24}$$

Recently, Hatzigeorgiou (2010) focused on *DMF*s for SDOF systems subjected to near-fault records and proposed a prediction equation for the *DMF* in near-fault regions which is a function of both damping ratio and period of vibration. For the equation coefficients different (but very similar) sets of values corresponding to different types of soil are reported whereas the influence of magnitude is not accounted for.

The effect of the pulse period T_p on *DMF* had not been studied as of yet. Different studies (e.g. Alavi and Krawinkler 2001, Fu and Menun 2004, Tothong et al. 2007, Ruiz-Garcia 2011, Iervolino et al. 2012), have shown that an adequate characterization of ground motions exhibiting a distinct velocity pulse should explicitly take into account the pulse period.

3.1 Prediction model of DMF for pulse-like ground motions

The effect of damping on single-degree-of-freedom systems is studied here using 110 pulse-like ground motions. Elastic displacement spectra $S_d(T,\xi)$ are estimated for different values of the damping ratio ξ (namely, 2, 5, 10, 20, 30, 40 and 50%), and the *DMF* spectra are calculated as:

$$DMF = \frac{S_d(T,\xi)}{S_d(T,0.05)} \tag{25}$$

The records are selected from the NGA database (http://peer.berkeley.edu/nga/), and are from shallow crustal earthquakes occurred in active tectonic regions. They are near-fault pulse-like ground motions characterized by forward-directivity. The ensemble of selected ground motions contains records identified in other studies (Rodriguez-Marek 2000, Mavroeidis and Papageorgiou 2003, Bray and Rodriguez-Marek 2004, Baker 2007, Shahi and Baker 2011) as characterized by forward-directivity. Based on what reported in Rodriguez-Marek and Cofer (2007) and in Gillie *et al.* (2010), records from the Bam earthquake (2003) and the Parkfield earthquake (2004) are also added in the set. More details on the selected ground motions are reported in Mollaioli *et al.* (2014).

The *DMF* spectra are calculated up to 8 s, therefore given that the pulse-like effects are significant only at periods around the period of the pulse, only records with a pulse period lower than about 9 s were used. About the soil conditions at recording stations, with the exception of 4 ground motions recorded on soil type B (refer to the NEHRP site classification based on the V_{S30} value), all the others ground motions were recorded on soil type C or D. The two horizontal components of the ground motions were rotated to the fault-parallel (FP) and the fault-normal (FN) direction. For each near-fault ground motion, the pulse period of the velocity time history is evaluated according to the methodology proposed by Baker (2007). With the aim of highlighting the effect of the pulse on the damping modification factor, the ground motions are grouped into bins having different values of the pulse period T_p. The following seven intervals of T_p are considered: $T_p< 1$ s; $1 \leq T_p< 2$ s; $2 \leq T_p< 3$ s; $3 \leq T_p< 4$ s; $4 \leq T_p< 5$ s; $5 \leq T_p< 6$ s; $6\text{ s} \leq T_p$. In order to compare the results with the fault-parallel counterpart, the same division in T_p bins used for the FN records is also used for the FP records.

Mean *DMF* spectra obtained for the FN and FP dataset are shown in Figure 13 for each T_p bin and for three selected values of the damping ratio. The bin corresponding to $T_p \geq 6$ s is strongly dominated by signals from the same event (i.e. the Chi-Chi earthquake), however,

the trend for the variation of *DMF* with period and damping ratio observed for this group of records seems consistent with those obtained for the other groups.
The period-dependent nature of the *DMF* is highlighted in the plots of Figure 13. Such dependence is clearly influenced by the pulse period T_p. For low values of T_p the variability with the period is evident in the whole spectrum, while for high T_p values the effect of the period is significant in the short period range only (T< 0.5 s). Differences in the trends obtained for the *DMF* spectra calculated with the FN records and the FP records are negligible for T_p< 1 s and $T_p \geq$ 6 s. By looking at the plots obtained for the other T_p intervals it can be observed that the fault-normal mean spectra are characterized by a peak or a valley (depending on whether the damping ratio is lower or higher than 5%, respectively) at a period value that is about 1 sec less than that of the pulse.
Finally, due to the fact that the incidence angle may significantly affect the response of buildings (Sebastiani et al. 2014) the effect of the orientation of the ground motion components on the *DMF* is investigated by rotating in a set of different directions each pulse-like ground motion. The observed trend is that the DMF values obtained using the rotated components are between the values obtained with the fault-normal and fault-parallel components.
Based on the estimated *DMF* values, the prediction model given in Equation 26 is proposed for pulse-like ground motions. The dependence on damping ratio is given by a polynomial expression of the natural logarithm of ξ, the dependence on period is given by two terms: the first term is a quadratic expression of the logarithm of T; the second term, which is a linear function of $T^{-0.15}$, is introduced to improve the predictive capability of the equation in the case of damping ratio values smaller than 5%.

$$\mathrm{DMF} = 1-(5-\xi)\cdot\left[1+\alpha\ln(\xi)+b(\ln(\xi))^2\right]\cdot\left[c+d\ln(T)+e(\ln(T))^2\right]\cdot\left[1-\gamma\left(1-f\cdot T^{-0.15}\right)\right] \quad (26)$$

In Equation 26 the variable γ is 1 or 0 according to whether ξ< 0.05 or $\xi \geq 0.05$. The parameters *a*, *b*, *c*, *d*, *e* and *f* are regression coefficients which depend on the considered pulse period interval (Table 7). They are obtained with a nonlinear regression that minimizes the sum of squares of relative errors.
In Figure 13, the proposed prediction model is compared with the mean and the mean ± one standard deviation *DMF* spectra of the pulse-like records (FN dataset); damping ratios equal to 2%, 20% and 50%, and two different intervals of T_p are shown. Even if some differences between observed mean spectra and predicted spectra may be noticed, the model fits the data well. In the same figure the proposed model is compared with that proposed by Priestley (2003) (Equation 24), this equation tends to overestimate the *DMF* in the long period range of the spectra at low values of the pulse period, while it underestimates the effect of damping in the other cases.

Table 7. Model parameters of the prediction equation

	T_p< 1 s	$1 \leq T_p$< 2 s	$2 \leq T_p$< 3 s	$3 \leq T_p$< 4 s	$4 \leq T_p$< 5 s	$5 \leq T_p$< 6 s	6 s $\leq T_p$
a	-0.27513	-0.28068	-0.31371	-0.29172	-0.32238	-0.33944	-0.35714
b	0.01793	0.01895	0.02523	0.02032	0.02702	0.03036	0.03379
c	-0.06640	-0.07427	-0.09171	-0.08348	-0.09131	-0.10365	-0.11760
d	0.01025	0.00748	0.00224	-0.00344	-0.00072	-0.00033	-0.00652
e	0.00822	0.01001	0.00754	0.00497	0.00208	0.00258	0.00444
f	0.87902	0.92861	0.88061	0.94676	0.86887	0.87211	0.92982

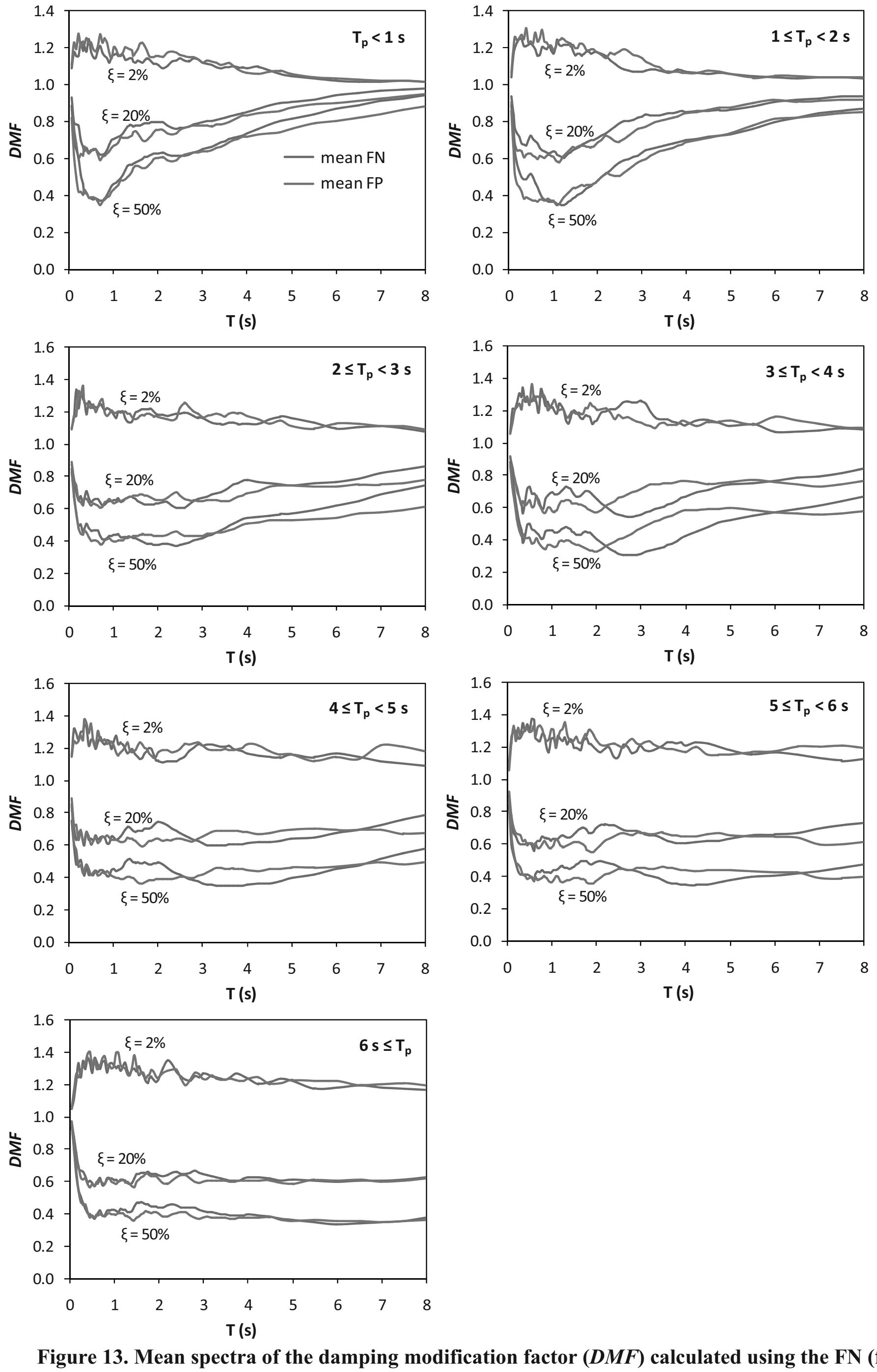

Figure 13. Mean spectra of the damping modification factor (*DMF*) calculated using the FN (fault-normal) and the FP(fault-parallel) records.

PART B: NUMERICAL SIMULATION OF THE APRIL 2009 L' AQUILA EVENT

1 OBJECTIVES

With the ongoing progress of computing power made available not only by large supercomputer facilities but also by relatively common workstations and desktops, physics-based source-to-site 3D numerical simulations of seismic ground motion will likely become the leading and most reliable tool to construct ground shaking scenarios from future earthquakes. These simulations aim at coupled modelling of seismic source, propagation path and site effects, so to provide an engine to produce, effectively and with reasonable computing efforts, plausible realizations of future earthquakes. This is for example the idea behind the ShakeOut experiment in California, where the physics-based simulations of a hypothetical M_W7.8 earthquake on the Southern San Andreas Fault were the basis to construct a comprehensive earthquake risk scenario including costs evaluations and planning of emergency response activities.
With such an engine available, many issues related to the earthquake ground motion prediction in the near-source area of large earthquakes could be addressed in a more reliable way, such as the characterization of near-source ground motion for seismic hazard studies and for seismic actions for design, including site effects, vertical components and spatial variability over short distances.
However, limitations of such numerical simulations should be carefully assessed, before numerical results may confidently be used instead of ground motion records. These limitations may typically come from (i) the high-frequency bounds of the spatial discretization of the numerical mesh; (ii) the scanty knowledge of details of the surface geology and of the seismic source and fault slip distribution.

With the main objective of providing an advanced 3D numerical simulation of earthquake ground motion in the near-source area of the M_w6.3 L'Aquila earthquake on Apr 6, 2009, to be used as an input for different earthquake engineering applications within the RELUIS projects, a pilot study was performed with the following tasks:

1. to improve the geological and geophysical characterization of the Aterno river basin, suitable to be cast in the form of a 3D numerical model suitable for earthquake ground motion simulations;
2. to invert the fault geometry and fault slip distribution based on available records;
3. to perform numerical simulations of the source-to-site seismic wave propagation during the L'Aquila earthquake.

The selected numerical tool was the SPEED code (Spectral Elements in Elastodynamics with Discontinuous Galerkin)[2], developed at Politecnico di Milano and designed for the simulation of large-scale seismic wave propagation problems including the coupled effects of a seismic fault rupture, the propagation path through Earth's layers, localized geological irregularities such as alluvial basins and topographic irregularities. As a starting point, the numerical simulations published by Smerzini and Villani (2012) were considered, having the main

[2] mox.polimi.it/it/progetti/speed.

limitations of a numerical model based on a relatively rough information about the geology and dynamic properties of the Aterno river basin.

2 GEOLOGICAL AND GEOPHYSICAL INVESTIGATIONS[3]

The research on the geological features of the Aterno valley, such as lithology, geomorphology and tectonics is addressed to the creation of a GIS database which should contain both geological-stratigraphical and geophysical data (boreholes, down-hole, samples for laboratory analysis, geology, detailed geological maps used for the seismic microzoning) and cartographical (maps at different scales), geographical (perimeter of the urban centers located in the areas of interest) and seismological data (epicenters and magnitude of the events). The main goal is the reconstruction of a geological model both at the large scale, i.e., the subsurface geology of the Aterno basin, and at the small scale, i.e., the stratigraphy of the first tens of meters, which are the more sensitive in terms of site amplification.

The first step of the study has been the collection of literature geological and geological-structural data on the Aterno River valley (which include numerous scientific papers and unpublished reports produced after the 2009 L'Aquila seismic event), and geological maps, e.g. those produced by the CARG geological mapping project, and by the Civil Protection Seismic Microzoning Project. The latter maps are related to 12 macro-areas located in the Aterno River valley. Such collection was paralleled by the collection of existing subsurface data, e.g. borehole data, geophysical data and numerous geological cross-sections, both published and unpublished, e.g. those obtained by the Civil Protection Project. The pre-existing dataset includes: (i) the official 1:50,000 geological maps of the L'Aquila area (Foglio 359 L'Aquila and Foglio 349 Gran Sasso d'Italia of the CARG Project; APAT, 2006; ISPRA, 2010); (ii) 23 geological maps at different scales (1:2000 to 1:5000), 67 geological cross-sections, 336 borehole data and 6 gravimetric maps produced during the Seismic Microzoning studies promoted by the Italian Department of Civil Protection (Gruppo di Lavoro MS–AQ, 2010); (iii) geoelectrical data (Bosi & Bertini, 1970); (iv) gravimetric data (Blumetti et al., 2002; Ge.Mi.Na., 1963) and (v) seismic tomography data (Improta et al., 2012).

Most of the collected maps were in pdf or paper format. Such data were digitalized and, subsequently, transformed in vectorial data with the aim of obtaining a geographic database. A further step of the study was the synthesis and homogenization of data reported in the various geological maps, borehole stratigraphies and cross-sections. In fact, the collected geological data are characterized by different scales and by different kinds of lithological characterization of the outcropping/buried sedimentary units. In addition, the acquisition of S waves velocity data of both the bedrock and the continental units recognized into the Aterno valley intramontane basin were carried out.

Detailed geological field surveys and a large scale geological-structural study have been also carried out, allowing a better characterization of the surface geology in some significant areas, e.g. the transect across the Aterno valley which crosses the seismic stations, and the recognition of the predisposing and causative factors responsible for surface deformation phenomena which affect strongly fractured carbonate rocky masses (sinkhole phenomena), which are in some instances related to seismicity.

[3] Prepared by Antonio Santo.

Overall information from borehole stratigraphy and from geophysics have been used to reconstruct the subsurface geological model of the middle Aterno valley (APAT, 2006; Blumetti et al., 2002; Bosi & Bertini, 1970; Ge.Mi.Na., 1963; Gruppo di Lavoro MS–AQ, 2010; Improta et al., 2012). The main result of this phase is the creation of a 40m DTM (digital terrain model) representing the distribution of the carbonate bedrock in the middle Aterno valley.

The thickness of the continental infilling is constrained by 13 drillings reaching the basin bedrock, which consists of Mesozoic–Cenozoic carbonate rocks, and Miocene– Lower Pliocene sandy– silty silicoclastic deposits. Deep drillings point to the occurrence of around 190-m thick deposits in the areas around L'Aquila and between Bazzano and Onna, respectively. However, the occurrence at some places of much larger cumulative sedimentary thickness is inferred from geophysical information.

The reconstructed subsurface geological model points to the occurrence of five main depocentres located in the NW, central and SE parts of the middle Aterno valley. These are respectively located: (i) to the SW of L'Aquila (inferred maximum bedrock depth around 350–400 m; orientation N150), (ii) to the west of the Bazzano and Monticchio carbonate ridges (maximum depth around 350–400 m; orientation N90), (iii) in the middle part of the Paganica–San Demetrio basin (depth around 250–300 m; orientation N150), (iv) in the SE part of such basin (San Demetrio area; estimated maximum bedrock depth 550 m; orientation N050), and (v) to the SW of Barisciano, in the Castelnuovo–Civitaretenga basin (again estimated maximum bedrock depth 550 m; orientation N120) (Figure 2.1).

The buried bedrock morphology only partly mirrors the spatial distribution of the present-day topographic highs and lows. In fact, while the depocentre in the middle Paganica–San Demetrio basin develops in the subsurface of the present-day alluvial basin, very thick (≥400/500 m) sedimentary bodies identified in the NW and SE parts of the middle Aterno valley overlap topographic highs interposed between the modern alluvial/lacustrine plains. Outcropping units, in such highs, are relatively old (Lower to Middle Pleistocene – alluvial/lacustrine successions). Overall evidence testifies to important changes in the perimeter of the depocentres in the middle Aterno valley over the Early Pleistocene to present time span.

The 40m DTM has been than shared with the other research groups. In particular, there has been a strong collaboration with the Geotechnical and Geophysics Groups, that has determined the creation of several new data which include:

- a 67x67 km wide box representing the surface topography of the areas surrounding the middle Aterno valley (it has been represented in a 200m DTM);
- the surface topography of the middle Aterno valley (it has been represented in a 200m DTM);
- the subsurface topography of the middle Aterno valley (the 40m DTM has been reduced to a 200m DTM).

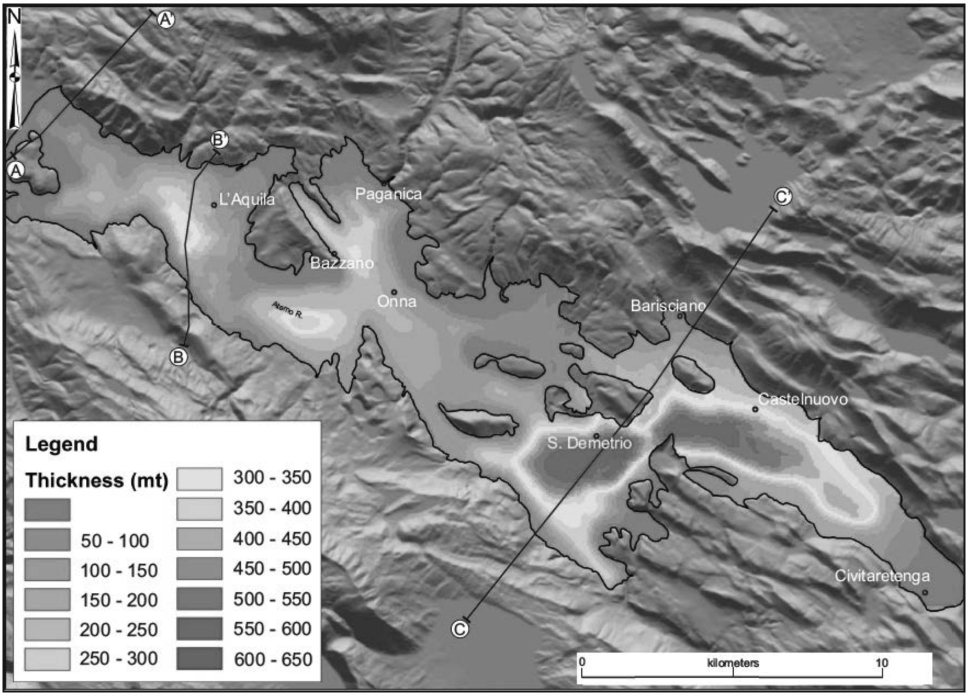

Figure 2.1. Reconstruction of the Quaternary continental deposits thickness derived by subtracting the 40m DTM representing the distribution of the carbonate bedrock in the middle Aterno valley from the actual topography of the same area. (Santo et al., 2014).

All this data have been produced into two different formats:

- ".grid" which consists in DTMs;
- ".shp" which consists in Point Shapefiles.

The following step has been the creation of the tectonic-geomorphological map of the middle Aterno valley. This phase has contributed to the recognition and the location of the main faults interesting the area, which control the geometry of the middle Aterno valley and that can be located on both the valley flanks.

The tectonic-geomorphological map of the middle Aterno River valley has been created also considering that after the Mw 6.3, 6 April 2009 normal faulting L'Aquila earthquake, several geoscience research groups focused their attention on the understanding of geological, tectonic and seismological features of the L'Aquila area and of the surroundings of the Central Apennines, publishing a considerable number of studies (see references in Giaccio et al., 2012). However, few studies addressed the morphotectonic features of the area (e.g., Blumetti, Guerrieri, & Vittori, 2013; Galli, Giaccio, & Messina, 2010; Giaccio et al., 2012) and none on extensive geomorphological mapping (only old morphotectonic maps are available; Demangeot, 1965). The map is the result of a detailed geomorphological analysis of topographic maps at 1:5000 scale, aerial photos and orthophotos at 1:5000–1:33,000 scale (provided by Ufficio Infrastrutture Geografiche, Regione Abruzzo), a GIS-aided DEM investigation, and geomorphological field surveys. The lithologies on the map are modified from the official geological map (APAT, 2006; ISPRA, 2010) and are grouped in to three main units, pertaining to the main lithological successions:

- Quaternary continental deposits (Postorogenic succession);

- Arenaceous-pelitic bedrock (Synorogenic succession);
- Calcareous bedrock (Preorogenic succession).

Mapped landforms have been grouped as it follows:

- Structural landforms (which include knickpoints, saddles, rectilinear scarps, fault scarps, fault escarpments, dip slopes, triangular facets, subsequent valleys and back-tilted valleys);
- Fluvial landforms (which include fluvial terraces, alluvial fans, alluvial scarps, hanging valleys, gorges, beheaded valleys, windgaps, river bends and alluvial plain);
- Lacustrine landforms (which include paleo-lacustrine depositional surface);
- Karst landforms (which include tectono-karst plains, dissected tectono-karst plains, sinkholes, doline fields and dolines);
- Complex landforms (which include remnants of paleosurfaces).

All the geological and geomorphological data derived by the extensive analysis of the middle Aterno River valley have been then synthesized in three tectonic-geomorphological profiles, chosen based upon available deep boreholes and geophysical data (see Figure 2.1 for profile location). The profiles clearly show the increasing thickness of continental deposits moving towards the SE sector of the basin.

In profile A (Pettino area) the bedrock is covered by only 20 –70 m of Upper Pleistocene – Holocene alluvial deposits. In the NE part alluvial deposits are separated by fault escarpments and triangular facets and dissected by subsequent valleys.

Profile B, located near L'Aquila town, shows a thick continental sedimentary body characterized, in the first _50–100 m of depth, by slope breccias (Lower Pleistocene). These breccias lie on lacustrine clay and silt deposits about 250-m thick which can be ascribed to Upper Pliocene – Lower Pleistocene time. The deepest portion (last 150 m) of this lacustrine succession (Upper Pliocene) is known only by deep borehole data and is characterised by the presence of lignite deposits (Ge.Mi.Na., 1963; Gruppo di Lavoro MS-AQ, 2010). The Quaternary continental succession is uplifted and dissected by a wide gorge; paleosurfaces are preserved on both sides of the valley, on continental deposits or bedrock. The valley is bounded by wide fault escarpments. Profile C outlines the very thick (up to 500 m) Upper Pliocene-Middle Pleistocene continental succession passing from a lacustrine environment to a deltaic and fluvial environment with a clear regressive trend. This succession is bounded toward SW by NW-SE trending and NE dipping faults characterised by a wide fault escarpment; toward NE it is displaced and uplifted by extensional tectonics along NW-SE trending and SW dipping faults outlined by fault scarps and escarpment and by a stair-like sequence of paleosurfaces on Quaternary continental deposits. The NE side of the middle Aterno valley, is characterised by remnants of high elevation paleosurfaces on bedrock, dissected by several NW-SE elongated tectono-karst basins, bounded by NE dipping and SW dipping fault escarpments.

The subsurface geological model, the tectonic-geomorphological map and the tectonic-geomorphological profiles of the middle Aterno River valley have been published on international paper (Santo et al.,2014). In this paper, besides the previously mentioned results, additional information regarding the orography and hydrography of the study area have been reported. Such informations include the elevation map of the middle Aterno River valley, the Aterno River long profile, the normalized steepness index (Ksn) of the Aterno drainage network and the drainage network ordering and azimuth distribution.

The elevation map highlights the occurrence of a succession of adjoining, and partly coalescent, morphological depressions at elevation ranging from 500 m to 900 m a.s.l. The main topographic lows are: the Barete–Pizzoli plain (700 m a.s.l.); the Scoppito basin (700–750 m a.s.l.); the Preturo plain (660 m a.s.l.); the western L'Aquila–Coppito plain (640 m a.s.l.); the Monticchio plain (590 m a.s.l.); the Paganica–San Demetrio plain (550–620 m a.s.l.); the Barisciano plain (850 m a.s.l.); the Castelnuovo–Civitaretenga plain (740 m a.s.l.); the Navelli plain (670 m a.s.l.).

The L'Aquila and Barisciano highs define three major sub basins, from northwest to southeast: the western L'Aquila – Coppito basin; the Paganica–San Demetrio basin; the Castelnuovo– Civitaretenga basin. These basins are surrounded, towards SW and NE, by NW-SE elongated ridges of carbonate rocks, with elevations ranging from 1000 m to 2000 m, up to the 2912 m a.s.l. high Gran Sasso massif (toward the NE) and to the 2204 m a.s.l. high Mt. Ocre (towards the SW). Minor ridges are located within the major basins, and separate adjacent plains outlining a very articulate physiography.

The long profiles and steepness index investigation outline several knick points along the main rivers in the study area. Some either sharp or smooth knick points are also present along the Aterno River, in the middle and lower parts, which are also depicted on the main map. These features are related to lithostructural control or to tectonic features. The overall configuration of the drainage, suggests a complex evolution of the hydrography marked by strong modifications over time (Santo et al., 2014).

Overall information shows that the modern topographic lows represent a delayed response to the extensional tectonics, which have been active in the study area at least since the Early Pleistocene. A sequence of fault escarpments bounds the basins on both sides (NE and SW) outlining step-like (SW) and ridge and basins (NE) landscapes, displacing and truncating previous paleosurfaces resting on both bedrock and Quaternary continental deposits. The formation of the western L'Aquila–Coppito, Paganica–San Demetrio and Castelnuovo–Civitaretenga basins as tectonic basins along NW-SE trending, SW-dipping Mt. Pettino, Paganica- San Demetrio and Castelnuovo fault systems, and NW-SE trending NE-dipping Bazzano- Fossa fault system, caused a significant change in the middle Aterno valley perimeter, in the spatial distribution of the main topographic highs and lows, and in the drainage basins and network geometry. Former (Early Pleistocene to early Middle Pleistocene) depocentres, outlined by the Quaternary continental deposits thickness analysis, have been affected by the mentioned fault systems, also giving rise to either dissected (i.e., the gorge cut in the L'Aquila high), or undissected, thresholds (i.e., saddles, wind gaps) which separate the present-day drainage basins from endoreic areas. Evidence of such phenomena is represented by the uplifted terraces (formed in the Lower to Middle Pleistocene lacustrine/alluvial successions), and erosional surfaces in the area spanning from Paganica to San Demetrio, in the Barisciano high, and in the L'Aquila area.

3 FROM THE GEOLOGICAL MODEL TO THE NUMERICAL SPECTRAL ELEMENT MODEL[4]

The 3D Spectral Element (SE) numerical model of the L'Aquila basin extends for 58 km in the NS and EW directions with a maximum height of 20 km, constrained by the size of the fault (Figure 3.1a). The geometrical model was constructed by overlapping the topographic layer, obtained by a 250 m Digital Elevation Model (DEM), with the layers describing the bedrock morphology as provided in section 2, and including the fault geometry, according to Chiaraluce et al., 2011 (Figure 3.1b).

[4] Prepared by Lorenza Evangelista, Anna d'Onofrio and Francesco Silvestri.

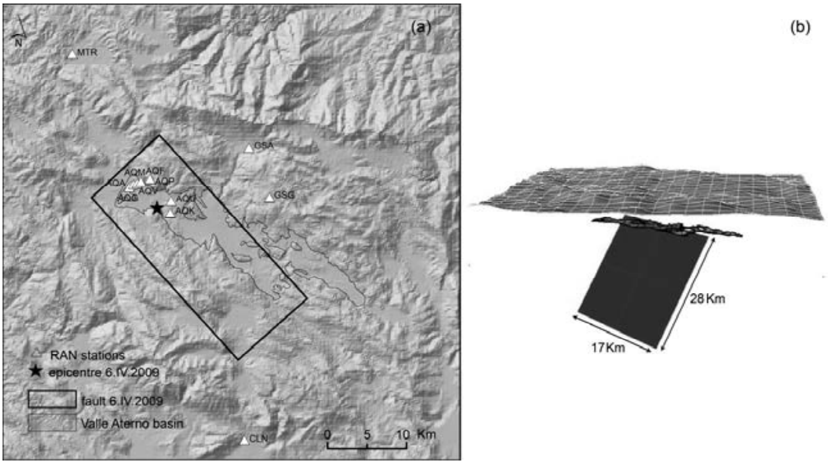

Figura 3.1. 3D model of the L'Aquila basin: (a) areal extension of the analysis domain, (b) topography, bedrock morphology and geometry of the fault.

Through the collection and interpretation of geophysical and geotechnical investigations carried out in the area within the studies for the Microzonation and the C.A.S.E. Project (Santucci de Magistris et al, 2013), a shear waves velocity model of the Aterno basin was defined. Based on these studies, the geological complexity of the quaternary formations filling the basin was simplified providing a characteristic Vs profile for each of the five depocentres. Since the SE (Spectral Element) code did not allow to consider a spatial distribution of V_S, in the preliminary analyses all the alluvial soils were assumed to behave as a linear visco-elastic medium, characterized by a unique value of density (ρ), Poisson ratio (ν) and shear wave velocity (V_S) profile. The adopted model was based on the results of several in situ and laboratory tests: a minimum V_S value of 300 m/s was assumed at surface and a shear wave velocity varying as a power function of depth was adopted for the deeper sedimentary layers. The Q factor was derived by the V_S and assumed proportional to the frequency.
The soil model parameters are summarized in the following:

$$\rho = 19 \text{ (kN/m}^3\text{)} \tag{3a}$$

$$V_s = 300 + 36 \cdot z^{0.43} \text{ e } V_p = \sqrt{4.57} \cdot V_s \text{ (m/s)} \tag{3b}$$

$$Q = V_s / 10 \tag{3c}$$

To verify the suitability of the simplified model adopted, two 1D site response analyses were carried out considering the measured (Lanzo et al., 2011) and the modeled shear wave velocity profiles at the RAN (National Accelerometric Network) station AQA (Figure 3.2a). The results were compared in terms of amplification functions in Figure 3.2b and show a good agreement in the range of frequencies (up to 2 Hz), that can be reproduced by the SE code.

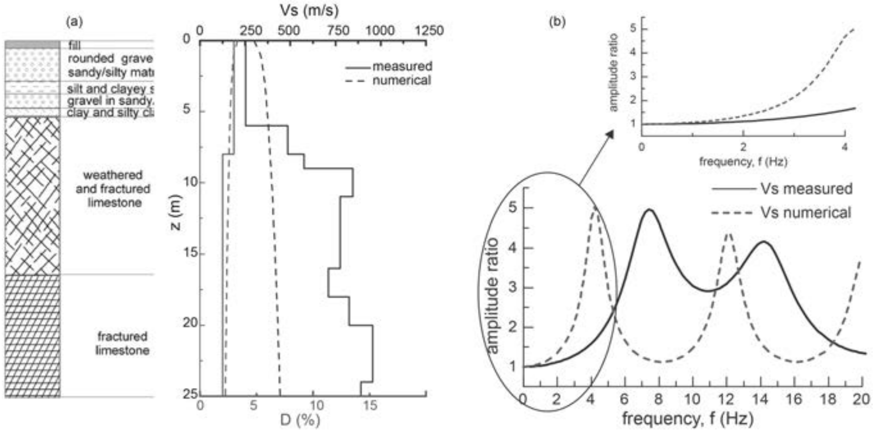

Figure 3.2. AQA station: (a) comparison between the measured and numerical Vs profile, (b) results of 1D site response analyses in term of amplification ratio.

It is worth highlighting that an outcropping bedrock was assumed outside the boundaries of the basin.

A crustal model, based on that suggested by Ameri et al. (2012), was adopted. It is characterized by five horizontal and parallel layers resting on a half-space at a depth of 20000 m. In particular the V_S of the shallow layer was reduced respect to that proposed by Ameri et al (2012) according to the results of site investigations (AQ-MS Working Group, 2010) in order to reduce the impedance ratio at the bedrock. The properties of each layer are shown in Table 1.

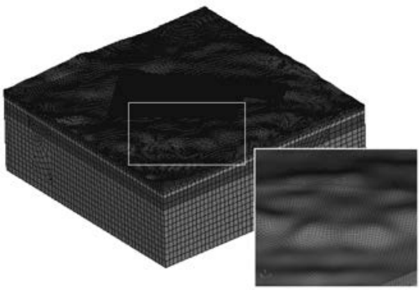

Figure 3.3. Numerical mesh consists of 263,936 hexahedral elements.

Table 1. Horizontally stratified crustal model assumed for the 3D numerical simulations.

H (m)	Vs (m/s)	Vp (m)	γ (kN/m3)	Q
1000	1700	3160	25	100
1000	2600	4830	28.4	200
3000	3100	5760	29.4	200
15000	3500	6510	31.8	200

The numerical domain was discretized according to the not-honoring approach: the mesh consists of 263,936 hexahedral elements (Figure 3.3): the size varies from a minimum of 133m, within the quaternary basin, up to 400 m in the outcropping bedrock. The mesh was generated to propagate up to about 2Hz for SD=3; the timestep for the explicit second-order finite difference time integration scheme is $\Delta t=10^{-3}$ s for a total simulated time of 30s. The simulations were carried out on the cluster ScoPe of the University of Naples using 132 parallel CPUs, resulting in a total computation time of about 3 hours for a single simulation.

4 INVERSION OF A KINEMATIC SLIP DISTRIBUTION ALONG THE PAGANICA FAULT[5]

A key input for the numerical simulation is the fault geometry, in terms of strike, dip and spatial extension, and the fault slip distribution, to be inverted based on available records. L'Aquila earthquake offers a unique opportunity to investigate the source process because of the large amount of data recorded both in near and far fault conditions. Additionally, azimuthal coverage of stations may allow in principle to capture both up-dip in-plane and along strike directivity effects.

The geometry of the fault was fixed and defined according to the focal mechanism (Chiaraluce et al., 2011). We assumed a NW-SE trending normal fault, with strike 133° and dip 54°. The dimension of the fault along the strike direction is assumed to be 28 km while the width is 20 km, as suggested by Cirella et al. (2009). According to the scaling law relating the moment magnitude to the fault size of earthquakes, length and width are significantly larger than the typical size of a magnitude 6.3 event. Nevertheless, this is not a problem because the inversion procedure for fault slip distribution excludes those zones where the retrieved slip is zero or close to zero, thus reducing the effective size of the fault.

We inverted 13 strong motion data recorded at the RAN network. Three component acceleration data were twice integrated to obtain the displacement, and then band-pass filtered between 0.05 and 0.5 Hz. The lower frequency was selected in order to insure a large signal to noise ratio on the displacement traces, while the high-frequency was selected as the largest frequency for which a 1D model can be assumed as a good approximation of the structure crossed by the seismic waves. We remember that the far field displacement spectrum is expected to be flat up to the corner frequency of the event and then to decay beyond that limit. Since the corner frequency is estimated at about 0.1-0.2 Hz, the higher frequency region of the selected spectrum is naturally diminished in the misfit of the displacement records.

Green functions were computed in the 1D model of Chiaraluce et al. (2011), using the discrete wavenumber method.

The inversion was performed by fixing the shape of the source time function (we selected a boxcar) and the rise time duration to 1 sec. It is worth to note that the spectrum of the source time function is almost flat up to its own corner frequency related to the inverse of the duration, independently of the specific selection of the function. Since this frequency (1Hz for this case) is significantly above the upper frequency limit of the used filter, this choice will not affect our results. As a drawback, we have to note that the rise time is completely unconstrained by this inversion. Moreover, to limit the number of parameters to be retrieved we also fixed the slip direction on the fault plane to the rake retrieved by the focal mechanism. The rake is -102°, indicating an almost pure normal fault.

For the slip parametrization, we used a linear superposition of overlapping Gaussian functions, as suggested by Lucca et al. (2012). The width of the Gaussian functions is compatible with the minimum wavelength we want to be represented in the data. We also assumed a variable rupture velocity model, based on a Lagrange interpolation on a regular grid. Rupture times were obtained by integration of the rupture velocities values, using the eikonal equation. Inversion is based on the minimization of the L2 norm between observed data and synthetics built on the described kinematic model. We used a two-step nested strategy, for which the rupture velocity is explored by a global algorithm, while for each fixed

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[5] Prepared by Gaetano Festa and Sergio del Gaudio.

rupture velocity model, the slip is retrieved by a non-negative linear inversion, based on the least squares technique.

The kinematic model is shown in Figure 4.1, within the misfit between real data (black records) and synthetics (red records). We found a major slip patch between 5 and 10 km southwards of the hypocenter, which is responsible for the directivity effect observed south of the fault. We also found a smaller asperity in the upper part of the fault with slip as large as 50 cm and a third one nearby the surface, with a 30 cm of slip, which is responsible of the up-dip directivity observed at L'Aquila and GSA stations (see Figure 4.1). Rupture times indicate a faster rupture in the upper part of the fault, where the average rupture velocity is about 2.5km/s. The anti-plane rupture southward of the hypocenter is delayed in time, with a resulting average rupture velocity of 2 km/s. The absolute values of the rupture velocity seem to be sensitive to the specific structural model, while the decrease of the rupture velocity in the south segment of the fault is robust.

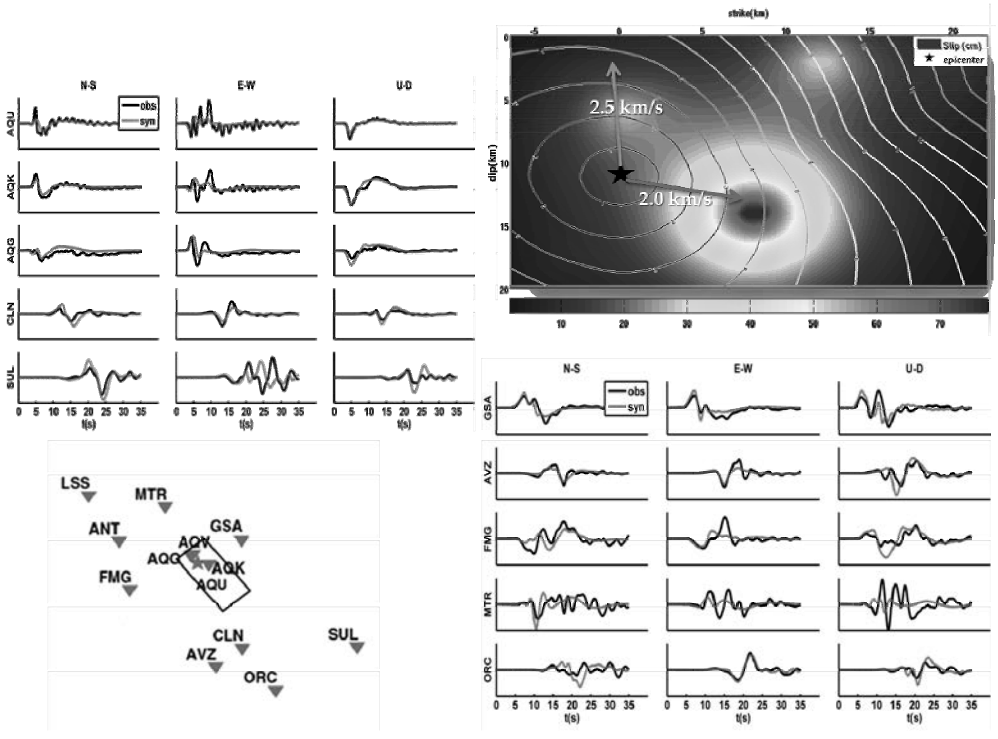

Figure 4.1. Final slip distribution and isolines of rupture time from kinematic inversion of l'Aquila earthquake data. Comparison of real data (black records) and synthetics (red records) is also shown for some stations.

We observe that in the selected frequency range the synthetics fit pretty well the real observations, at least at two of the three components, for almost all of the stations. Specifically, one of the horizontal components of the L'Aquila stations has an amplitude which is not reproduced by the synthetics, while the synthetic waveform partially follows the real one. Higher frequency wiggles are not reproduced by the model, indicating that the velocity model is depleting high-frequency signals.

To improve the kinematic model with the specific goal of better representing the strong motion data recorded at L'Aquila stations in a broader frequency range, the low frequency model was coupled with a high frequency k-2 slip distribution (Causse et al., 2009). We also allow the rise-time to be randomly selected, with a specific distribution, and rupture direction randomized at small space scales, to reduce spatial coherency. Comparison is now based only on the Fourier spectral shape at high frequency and on the value of the PGA. We found that the slip roughness does not significantly affect the spectral shape at near and far fault stations, while randomization of the rise time, with a uniform distribution between 0.1s and 1s, significantly improves the spectral fit, mostly at near fault distances. Anyhow, it is worth to note that we were not able to reproduce the pulse at about 1Hz observed at near fault L'Aquila stations.

5 PRELIMINARY RESULTS OF NUMERICAL SIMULATIONS OF THE APRIL 6 2009 EARTHQUAKE[6]

The numerical simulations were carried out assuming the two source models described in section 4. In particular, for the kinematic source model at low frequency (M1) the rise time, τ, was assumed constant and equal to 1 sec, while for the source model at high frequency (M2) a random distribution of τ was adopted with mean value equal to 0.75 sec.

In Figure 5.1 the contours of E-W velocity component are plotted at 6, 12, and 24 seconds, showing that the numerical model reproduce the up-dip rupture propagation and a wave focalization within the basin, particularly where the deepest depocenters are located.

The results of 3D analyses are shown as time histories of displacement monitored at selected stations of the Italian Accelerometric Network reported in Figure 3.1. The numerical results are then compared with the records of the mainshock in terms of velocity time histories as well as Fourier and normalized response spectra. To properly compare the observed and simulated data, the waveforms were processed with acausal band-pass filter Butterworth between 0.1 and 2 Hz, according to the maximum propagated frequency.

Figure 5.2 shows the comparison between numerical and observed signals at the stations AQU and AQG, considering the three components (EW, NS and UP).

The data clearly show that:

- ✓ whichever the source model adopted, M1 or M2, the energy content of the recorded signals is not consistently reproduced by numerical simulations;
- ✓ both models simulate with a satisfactory accuracy the frequency content coherently with the source models adopted (low and high frequency), particularly in the case of AQU (Figure 5.2a). At the station AQG (Figure 5.2b), instead, the simulations underestimate the amplitude of the signal.

  This observation could be justified by the roughness of the soil model adopted at AQG station: in the numerical model, it lies on outcropping bedrock (V_S=1700 m/s), while in the reality the station lies on weathered and fractured limestone deposit, which is characterized by a shear wave velocity profile corresponding to a 'class B' (i.e. stiff soil) site on the basis of the equivalent shear wave velocity, V_{S30} (Lanzo et al., 2011);

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[6] Prepared by Lorenza Evangelista, Anna d'Onofrio and Francesco Silvestri.

- ✓ the shape of the recorded response spectra are well reproduced by the numerical simulations, whereas the spectral amplitudes are significantly underestimated.

The comparison is summarized in terms of Goodness-of-Fit scores as suggested by Anderson (2004). For each monitored stations and for each source models, the GoF scores of PGV (peak ground velocity), PGD (peak ground displacement), RS (response spectral acceleration), FSA (Fourier amplitude spectrum), IA (Arias intensity) and SDE (Energy density) are evaluated in the bandwidth 0.1–2 Hz (Figures 5.3a-b).
The scores of the 6 metrics are assessed as average value of the 3-component for each stations.
The results clearly show a misfit between recorded and numerical waveforms, that cannot be ascribed to the simplified model adopted for the deposit filling the basin (see section 3) but could be due to lack of high-frequency energy content of both source models.

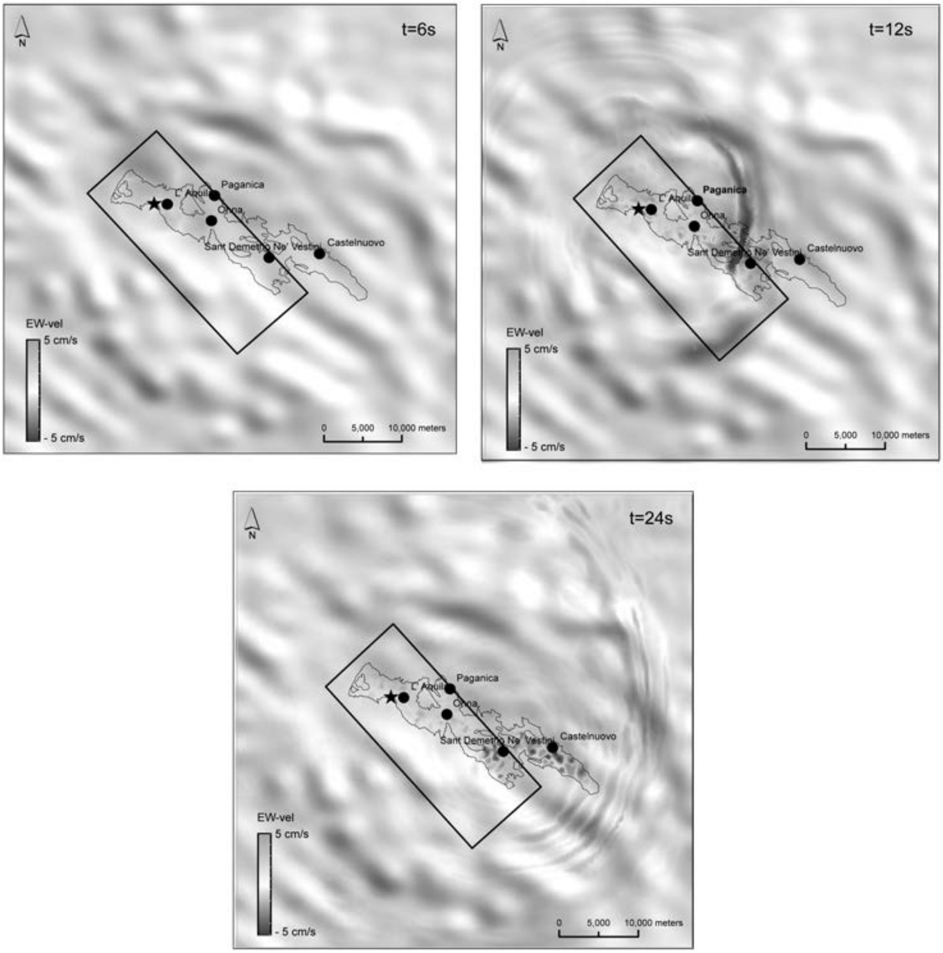

Figure 5.1. Contours of E-W velocity component at 6, 12, and 24 seconds.

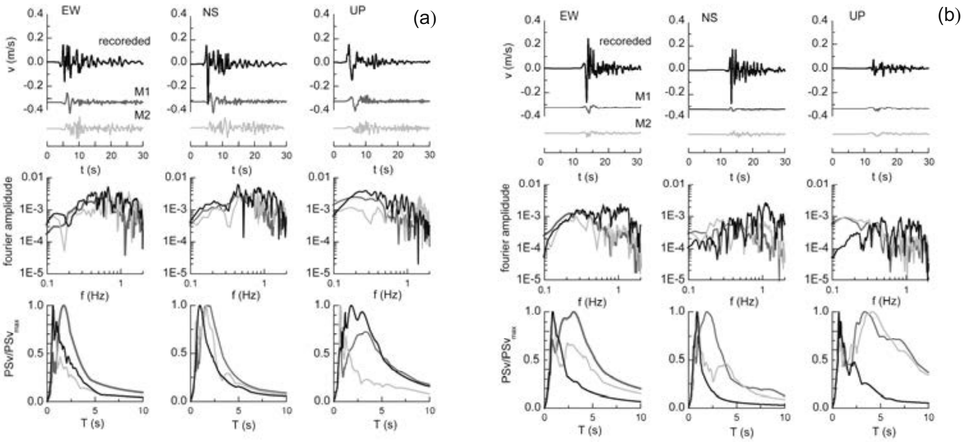

Figure 5.2. Comparison of recorded and simulated signals at the RAN stations: (a) AQU and (b) AQG.

Figure 5.3. Goodness-fit-scores in the frequency band between 0.1-2 Hz: (a) for he 5 monitored stations; (b) average values of the 6 metrics for each stations.

Finally the pulse characteristics of the recorded and numerical waveforms were evaluated adopting the procedure suggested by Baker (2007).

The application of this procedure to the Fault Normal (FN) and Fault Parallel (FP) components of the simulated waveforms confirmed the occurrence of a pulse at the RAN stations classified as "pulse-like" in Chioccarelli and Iervolino (2010), when the source model M1 is used. It is worth highlighting that the pulse periods extracted are significantly greater that 1 sec; this result is ascribed to the value of the rise time adopted in the analyses.

The comparison in terms of normalized pseudo-velocity spectrum of the fault parallel (FP) and the fault normal (FN) components (Figure 5.4), in any case, confirm the good fit between the recorded and simulated signals.

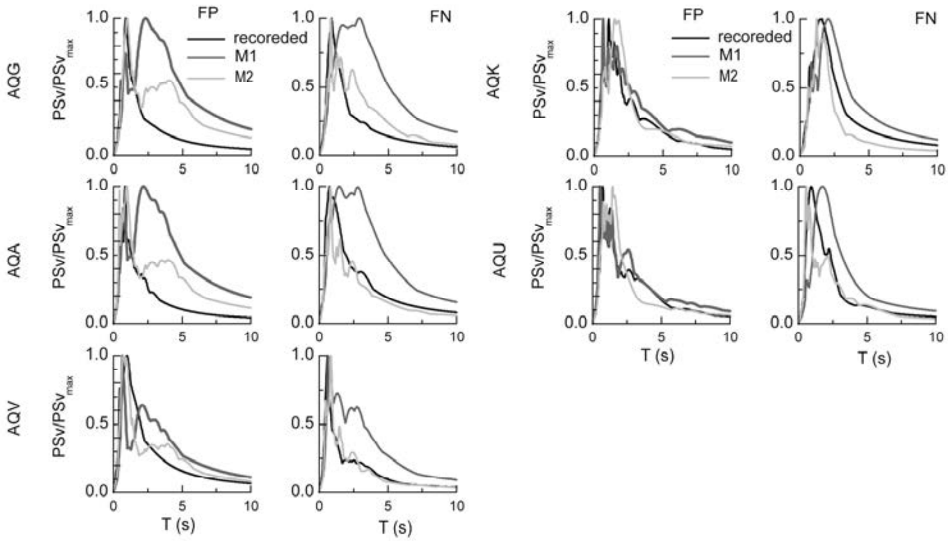

Figure 5.4. Normalized pseudo-velocity spectrum of the FP and FP components.

6 CONCLUSIVE REMARKS AND HINTS FOR FURTHER ACTIVITIES WITHIN RELUIS PROJECTS

Several important results were obtained within the research activity outlined in this contribution, related to the earthquake ground motion prediction in the near-source area of the M_w6.3 L'Aquila earthquake of April 6, 2009. First, an updated geological model of the Aterno river basin was obtained, suitable to devise a 3D numerical model for seismic wave propagation analyses; second, a contribution was given for the inversion of the fault slip distribution; third, a pilot set of numerical simulations of earthquake ground motion during the earthquake were obtained by the open source code SPEED, by exploiting the high performance computing cluster ScoPe of the University of Napoli.

Numerical simulations deserve further improvements, especially as regards the definition of the slip distribution, which turned out to be the most critical input feature the numerical results depend on. For this purpose, future research activities will be addressed (i) to provide pre-processing tools to SPEED that, based on a relatively rough information on the fault geometry, hypocenter location and spatial distribution of the main fault asperities, provide a numerical slip distribution consistent with the high-frequency (ω^2) radiation of seismic energy; (ii) to improve results of the numerical simulation of the L'Aquila earthquake by considering other inversions of the slip distribution from the literature, in order to find the one that allows for the most accurate approximation of the observed results.

These two improvements, affecting the high-frequency (item i) and low-frequency (item ii) parts of the spectrum, are expected to provide a realistic and complete picture of earthquake ground motion for the L'Aquila earthquake. The experience gained within this highly demanding numerical simulation experience, together with similar benchmarks from other Italian earthquakes such as the Po Plain earthquakes in 2012, will be of major relevance for earthquake engineering applications in near-source conditions, to better constrain the characterization of directivity pulses of earthquake ground motion, its small- and large-scale spatial variability, the relative contribution of vertical vs horizontal components of motion.

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G. Manfredi, M. Dolce (eds), *The state of Earthquake Engineering Research in Italy: the ReLUIS-DPC 2010-2013 Project*, 569-593, doi: 10.14599/r101316, © 2015 Doppiavoce, Napoli, Italy

SAFETY ASSESSMENT OF EXISTING BUILDINGS

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1 INTRODUCTION

The principle that the design/assessment of structures should be based on predefined performance objectives is accepted as self evident and explicitly adopted in most technical norms, albeit at different levels of completeness.
There is also now a general consensus on the idea that a full implementation of the concept in the area of seismic design can only be achieved within a rigorous probabilistic framework.
In the last 15/20 years research focused on achieving this objective has made substantial progresses on almost all aspects of the problem: from the description of the seismic action, to the behaviour of materials and of structural elements under cyclic deformations of amplitude close to collapse, to the elaboration of alternative approaches for the risk evaluation, etc.
On the other hand, it cannot be said that the output of this research has become part of the average background of the engineer. While it can be thus premature to issue or even draft a code on probabilistic seismic assessment (an easier task with respect to design, from many point of views), it is possible to draft guidelines to inspire revisions of current codes and to provide higher level methods and tools that could be used in practice for particular applications. Use of these higher-level methods is also encouraged in recently released international normative documents (fib, 2012).

2 BACKGROUND AND MOTIVATION

In spite of Part3 of Eurocode 8 (CEN, 2005), or EC8/3, dealing with seismic assessment and retrofitting of buildings, being available in draft form since 1996, relevance of such a document escaped the attention of both the authorities and the profession in Italy, until a small earthquake in 2002 caused the complete collapse of a school with a shocking death toll. The ensuing national scandal paved the way for an awakening in the general public of the consciousness of the seismic risk potentially affecting all types of constructions, the old as well as the recent ones. It also prompted the Department of Civil Protection to take action in 2003 in two directions: preparing a technical document dealing with the analytical seismic assessment of buildings, and emanating an ordnance requiring that all important public facilities be subjected to assessment within five years time. The technical document can be regarded essentially as the translation of the EC8/3, and even if never enforced, it became part of the current code, issued in 2008 and mandatory from 2009 (NTC, 2008). The ordnance on the other hand had the effect of promoting a large scale assessment effort which resulted in a very large number of buildings being subjected to seismic assessment using basically EC8/3, so that experience on its merits and limitations rests by now on solid statistical bases.

Besides, critical aspects have emerged from the use of EC8/3 not just in Italy but also in other seismic-prone European Countries, and plans for an improved version are under way. The consensus existing on major critical aspects allows for just a brief mention to be made here.

- Performance must be checked with reference to three Limit States. While these are formulated in terms of system performance, the verifications for reinforced concrete (RC) buildings must be carried out in terms of member behaviour, independently of the number and importance of non-complying members. This inconsistency is a major cause of dispersion of the results obtained by different analysts.
- Structure-related uncertainties are grouped according to three aspects, namely: those related to geometry, to the properties of the materials and to the details of reinforcement (for RC structures). Three levels of knowledge are considered, each one characterized by a combination of the knowledge acquired on the three types of uncertainty, and a so-called "confidence factor (CF)" is associated to each level. In many practical cases, however, the achievable state of knowledge does not fit in any of the levels above, due to non-uniform quality/quantity of information on the three aspects, with the consequent uncertainty on the value of CF to be adopted.
- The CF factors are to be applied to the material properties, which are only one of the many sources of uncertainties, and in the majority of cases of comparatively much lesser relevance on the outcome of the assessment.
- Little if any guidance is given on the modelling of the structure, e.g. on the use classical fibre elements or of stiffness/strength degrading models. Yet different choices on these aspects are rather consequential on the definition of the attainment of the LS's, especially for that of collapse.

In consideration of the above mentioned limits, the need arose to prepare a document of a level higher than the one in force, in which the performance-based concept is implemented in explicit probabilistic terms, thus allowing uncertainties of all nature to be taken into consideration and introduced into the assessment process, with their relevance on the final outcome properly reflected.
For what concerns the probabilistic procedures adopted the choice has been to adhere to the now well consolidated state-of–the-art, avoiding refinements deemed as inessential, in order to make the document accessible to a larger audience.

3 RESEARCH STRUCTURE

Research within Task RS4 was carried out in close collaboration with the units of University of Naples "Federico II" (Manfredi, Iervolino, Jalayer) and University of Genoa (Lagomarsino, Cattari).
The achieved goal of the Task was the production of a guideline document on probabilistic seismic assessment of existing buildings, feasible for practical application and open to future developments. The document, issued in the form of National Research Council Provisions (CNR, 2014), is briefly illustrated in the next section 4, following closely the discussion in (Pinto and Franchin, 2015).

4 MAIN RESULTS

The main result of Task RS4 is the provisions produced under the umbrella of the DPC-Reluis and CNR. The document is subdivided into four chapters (Introduction, Methodological aspects common to all typologies, Specific provisions for masonry buildings, Specific provisions for reinforced concrete buildings) and three appendices (Commentary to the text, Example application to a masonry building, Example application to a reinforced concrete building). Herein illustration is limited to the material-independent general part and to reinforced concrete buildings.

4.1 Methodological aspects common to all typologies

4.1.1 Limit states

Limit States (LS) are defined with reference to the performance of the building in its entirety including the structural and non-structural (partitions, electrical and hydraulic systems, etc.) parts. Three LS are considered:

- Damage Limit State (SLD): negligible damages (no repair necessary) to the structural parts, and light, economically repairable damages to the non-structural ones.
- Severe Damage Limit State (SLS): loss of use of non-structural systems and a residual capacity to resist horizontal actions. State of damage uneconomic to repair.
- Collapse prevention Limit State (SLC): the building is still standing but would not survive an aftershock.

Check against the attainment of the SLC is mandatory, in consideration of the general lack of reserve ductility of non-seismically designed buildings (contrary to the proven large reserve possessed by buildings designed according to present seismic codes).

4.1.2 Performance metric and associated targets

Buildings are attributed to the same four classes of importance in the current Italian code (NTC, 2008), depending on the socio-economic consequences of their LS exceedance. The required level of protection for each Class and each LS is formulated in terms of the mean annual frequency of exceedance (MAF): λ_{LS}. A so-called IM-based approach, is adopted for the evaluation of λ_{LS}, i.e. the MAF is computed by means of the total probability theorem, as the product of the probability of LS exceedance as a function of the value of the chosen seismic intensity measure (IM), denoted by $p_{LS}(s)$ and called fragility, and the frequency of occurrence of the corresponding intensity level $d\lambda_S(s)$, summed over the range of possible intensity values:

$$\lambda_{LS} = \int_0^{\infty} p_{LS}(s)\left|d\lambda_S(s)\right| \tag{1}$$

The proposed maximum values of λ_{LS} (reported in Table 1) are such as to ensure approximately the same level of protection as currently required by the national seismic code for the different Classes and LS's for new buildings. The values in the table have been calculated using the closed-form expression for the MAF λ_{LS} due to Cornell et al (2002). For details see (CNR, 2014) or (Pinto and Franchin, 2015).

Table 1. Minimum levels of protection in terms of maximum tolerated λ_{LS} (values in the table are multiplied by 103) as a function of building class.

Limit state	Class I	Class II	Class III	Class IV
SLD	64.0	45.0	30.0	22.0
SLS	6.8	4.7	3.2	2.4
SLC	3.3	2.3	1.5	1.2

4.1.3 Seismic action

The seismic action is characterized in terms of:

- the mean hazard curve for the site $\lambda_S(s)$, whose derivative appears in (1)
- a set of seismic motion time-series, used for the calculation of the fragility $p_{LS}(s)$

A discrete hazard curve in any site in Italy can be obtained from the *median* uniform hazard spectra (UHS) provided in the national code (NTC, 2008) for nine values of the mean return period, ranging from 30 to 2475 years, at the nodes of a square grid with sides of about 5 km. The hazard in a point inside a grid is obtained by interpolation of the values at its four corners. For any given value T of the structural period the nine values of $S_a(T)$ provide a point-wise median hazard curve. Official data provide also the 16% and 84% fractiles, which can be used, as suggested in (Cornell *et al*., 2002), to account for the epistemic uncertainty on the hazard curve by using its mean value, instead of the median. Details can be found in (CNR, 2014) or (Pinto and Franchin, 2015).

Motions (by which it is meant multiple orthogonal components) to be used for response analysis, in a minimum number of 30, can be either natural records or artificially generated motions, provided these latter are able to reproduce the same mean and variance of the spectral ordinates of the natural motions.

Selection of the natural records can be made, according to the state of the practice, using the technique of disaggregation of the hazard in terms of magnitude M, distance R and epsilon: it is suggested that the above data are obtained for values of the IM characterized by a MAF in the interval from 1/500 to 1/2000. The use of more refined techniques for record selection is also allowed (Bradley, 2013)(Lin *et al*, 2013).

Selection of motions should be made among those recorded on rock or stiff soil. If the site is characterized by soft soil (e.g. V_{s30} in the interval 180-360 m/s, or less) a site response analysis is mandatory. Equivalent linear methods can be used for this purpose if significant inelastic response at the higher intensities is not expected, otherwise fully non-linear methods must be employed.

Uncertainties regarding soil profile and geotechnical parameters should be treated in the same way as those related to the structure above soil, as explained later in 4.1.5.

For sites in proximity of known active faults the probability of occurrence of pulse-like motions must be evaluated and the selection of records should proportionately reflect it.

4.1.4 Knowledge acquisition

Given that a fully exhaustive (i.e. deterministic) knowledge of an existing building in terms of geometry, detailing and properties of the materials is realistically impossible to achieve, it is required that every type of incomplete information be explicitly recognized and quantified, for introduction in the assessment process in the form of additional random variables or of alternative assumptions. Since the number and the relevance of the considered uncertainties has an obvious bearing on the final evaluation of the risk, and consequently on the cost of the upgrading intervention, the search for a balance between the cost for additional information and the potential saving in the intervention should be a guiding criterion in the Knowledge acquisition process.

Based on the above consideration the present code does not prescribe quantitative minima for the number of elements to be inspected, the number of samples to be taken, etc. It asks instead for a sensitivity analysis to be carried out on one or more preliminary models of the building (variations on a first approximation of the final model). For RC structures this analysis is of the linear dynamic type (modal with full elastic response spectrum), which is adequate to expose global modes of response (regular or less regular) and to provide an estimate of the member chord rotations demands to be compared with yield chord rotation capacities. The latter, being quite insensitive to the amount of reinforcement, can be obtained based on gross concrete dimensions and nominal steel properties. The results of these analyses would then provide guidance on where to concentrate tests and inspections.

The extension of these tests depends on the initial amount of information. If original construction drawings are available, only limited verification of the actual reinforcement details is required, through concrete removal over an area sufficient to expose longitudinal and transverse reinforcement (and estimate spacing). When drawings are incomplete or missing, the extension of test/inspections must increase to understand the “designer’s modus operandi” in view of replicating it (this is regarded as more effective than blindly applying the ruling provisions at the time in a simulated design).

4.1.5 Uncertainty modelling

All types of uncertainties are assumed to belong to either one of the following two classes:

- those describing variations of parameters within a single model, amenable to a description in terms of random variables, with their associated distribution function
- those whose description requires consideration of multiple models, to each of which a subjective mass probability function is associated.

The uncertainties belonging to the first class include: the seismic intensity at the site, governed by the hazard function, the record-to record variability, described by a set of records, all material properties, related both to the soil and to the structure, normally described as lognormal variables, and the model error terms of the capacity models, also usually described as lognormal variables.

The uncertainties belonging to the second class include, among others, the geometry of the structure (e.g. presence and dimension of certain elements whose precise identification is too demanding), the reinforcement details in important places, alternative models for the capacity of the elements, alternative models for the behaviour of the components (e.g. degrading or non degrading). Uncertainties of this class are treated with the logic tree technique, where mass probabilities are assigned to the alternative assumptions for each of uncertain factor. Details can be found in (CNR, 2014) or (Pinto and Franchin, 2015).

4.1.6 Structural modelling and analysis

Exclusive recourse to non-linear methods of analysis, accounting for material and geometric non-linear phenomena, is considered in the document. The analysis can be either static or dynamic, and guidance is given for the application, as it will be illustrated in the following.

The structural model must be tri-dimensional, with simultaneous excitation applied along two orthogonal directions.

Regarding the behaviour of the structural members (beams and columns, recall that this paper illustrates only RC buildings) under cyclic loading of increasing amplitude two modelling approaches are considered, as shown in Figure 1.

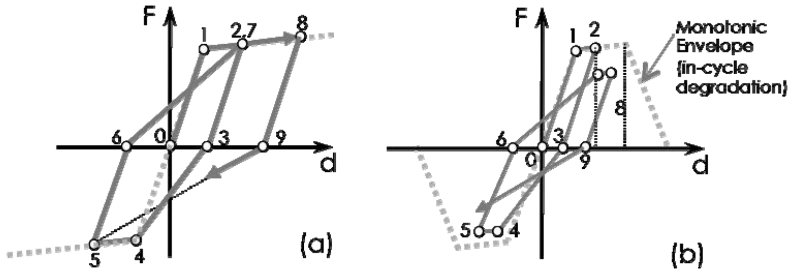

Figure 1. Non-degrading (a) vs degrading (b) nonlinear modelling.

- Non-degrading, i.e. stable hysteretic behaviour with non degradation of strength but overall degradation of stiffness (Takeda-type models)
- Degrading, where both stiffness and strength degrade with increasing cyclic amplitude down to negligible values.

The document provides in chapter 4 an overview of the state of the art on this latter type of models for RC structures.
As it shown in the next section, it is important to note how the use of the two different types of models has important reflexes in the identification of the collapse limit state of the structure.

4.1.7 Identification of limit state exceedance

Exceedance of each LS is signalled by a scalar indicator *Y* taking a value equal or larger than unity. The indicator expresses the global state of the structure as a function of that of its members. Its definition depends on the considered LS.
For the first two LS's, of light and severe damage, which pertain functionality and economic feasibility of repair actions, the choice of an appropriate threshold is left to the analyst in accordance to the owner/stakeholder requirements. The formulation of *Y* for the collapse limit state, related to safety, is stricter and does not leave space for subjective choices on the analyst side.

1.1.1 Light damage

For the purpose of the identification of the light damage LS, the building is considered as composed by N_{st} structural members and N_{nst} non-structural components:

$$Y_{SLD} = \frac{1}{\tau_{SLD}} \max\left[\sum_{i=1}^{N_{st}} w_i I\left(\frac{D_i}{C_{i,SLD}}\right); \sum_{j=1}^{N_{nst}} w_j I\left(\frac{D_j}{C_{j,SLD}}\right)\right] \tag{2}$$

In the above expression, *D* and *C* indicate the appropriate demand and capacity values, *I* is an indicator function taking the value of one when *D*≥*C* and zero otherwise, and the *w*'s are weights summing up to one, accounting for the importance of different members/components.

The indicator Y attains unity when the max function equals τ_{SLD}, a user-defined tolerable maximum cumulative damage. (e.g. something in the range 3 to 5%).

1.1.2 Severe damage

For the purpose of the identification of the severe damage LS, the indicator Y is formulated in terms of a conventional total cost of damage to structural and non-structural elements as:

$$Y_{SLV} = \begin{cases} \dfrac{1}{\tau_{SLV}}\left[\alpha_{st}\sum_{i=1}^{N_{st}} w_i c\left(\dfrac{D_i}{C_{i,SLV}}\right) + \left(1-\alpha_{st}\right)\sum_{j=1}^{N_{nst}} w_j c\left(\dfrac{D_j}{C_{j,SLV}}\right)\right] & \text{if } Y_{SLC} < 1 \\ 1 \quad \text{if } Y_{SLC} \geq 1 \end{cases} \tag{3}$$

where α_{st} is the economic "weight" of the structural part (i.e. about 20% in a low- to mid-rise residential building); $c(D/C)$ is a conventional cost function which starts from zero for $D=0$ and reaches unity, i.e. the replacement cost for the element, for $D = C_{SLS}$ (with C_{SLS} usually a fraction of the ultimate capacity of the element); as for the light damage LS, the indicator function attains unity when the quantity within square brackets equals τ_{SLS}, a user-defined fraction of the total building value over which repair is considered economically not competitive with demolition and replacement (data from past events show that this threshold depends on the social end economic context, but can be considered somewhere around 70% of total cost). Obviously if collapse occurs Y_{SLS} is set to 1.

1.1.3 Collapse

Identification of this LS depends on the modelling choices (see 4.1.6). If non-degrading elements are adopted, the system is described as a serial arrangement of a number of elements in parallel, so that the Y variable takes the expression (Jalayer *et al.*, 2007):

$$Y_{SLC} = \max_{i=1,N_s} \min_{j \in I_i} \frac{D_j}{C_{j,SLC}} \tag{4}$$

where N_S is the number of parallel sub-systems (cut-sets) in series, and I_i is the sets of indices identifying the members in the i-th sub-system. This formulation requires the *a priori* identification of the cut-sets. Carrying out this task is in general not immediate, since the critical cut-set depends on the dynamic response and changes from record to record.

If all elements are of the "degrading" type, i.e. they are able to simulate all types of failure, accounting for the interaction of bending and shear, the collapse state Y=1 is identified with the occurrence of the so-called "dynamic instability", that is, when the curve intensity-response becomes almost flat. In order to identify the point on the curve corresponding to Y=1 one can use the expression:

$$Y_{SLC} = (1+\Delta) - \frac{S'}{S'_0} \quad \text{with} \quad 0 < S' < S'_0 \tag{5}$$

with values for Δ in the interval 0.05 to 0.10, corresponding to a small residual positive stiffness, in order to avoid numerical problems.

Finally, if the elements are of the degrading type but the adopted formulation cannot account for all possible collapse modes, the indicator variable can be expressed as:

$$Y_{SLC} = \max\left[(1+\Delta) - \frac{S'}{S'_0}; \max_{nsm}\left(\frac{D}{C}\right)\right] \tag{6}$$

that simply indicates that the collapse condition is attained for the most unfavourable between dynamic instability and the series of the "non simulated (collapse) modes". Typically, this set includes the axial failure of columns. Care should be taken in selecting the columns to be included in the evaluation of (6), limiting it only to those that can really be associated with a partial/global collapse.

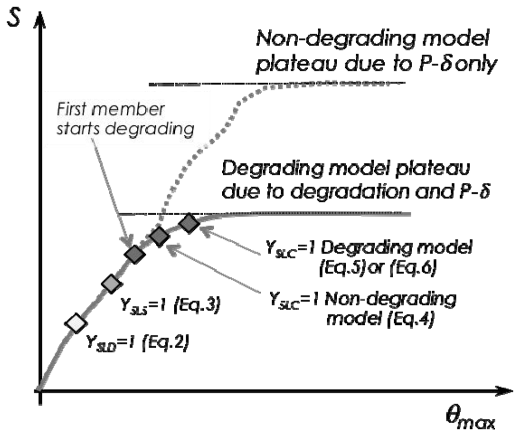

Figure 2. Response-intensity curves (also known as IDA curves, see 4.1.8.1) as a function of modeling choices.

4.1.8 Assessment methods

As already indicated in 1b), the outcome of the assessment is expressed in terms of the mean annual frequency of exceeding any of the proposed three Limit States: λ_{LS}. The integral in (1) can be evaluated numerically. However, if the hazard is approximated with a quadratic fit in the log-log plane ($\ln\lambda_S = \ln k_0 + k_1 \ln s + k_2 \ln^2 s$), and the fragility function is assumed to have a lognormal shape, closed forms for the evaluation of the integral are available. Indeed, the lognormal assumption is adopted in the provisions based on the international general consensus. The fragility thus takes the form:

$$p_{LS}(s) = p\left(Y_{LS} \geq 1 \middle| S = s\right) = p\left(S_{Y_{LS}=1} \leq s\right) = \Phi\left(\frac{\ln s - \mu_{\ln S_{Y=1}}}{\sigma_{\ln S_{Y=1}}}\right) \tag{8}$$

that requires evaluation of two parameters only: the mean and the standard deviation of the logarithm of the seismic intensity inducing the unit value of the Limit State indicator: Y=1.
The document provides three alternative methods, indicated in the following as A, B and C, for the evaluation of the fragility. All methods require a 3D model of the structure.

4.1.8.1 Method A: Incremental Dynamic analysis on the complete model

Recourse is made to the well known technique usually referred to as Incremental Dynamic Analysis, or IDA (Vamvatsikos and Cornell, 2002): it consists in subjecting the complete 3D model of the structure to a suite of *n* time-histories (each with two orthogonal horizontal components, the vertical component being normally omitted in case of ordinary buildings), each time-history being scaled at increasing intensity levels. At each level of S the value of Y is calculated, and the set of (S,Y) points are plotted to obtain a curve in the intensity-response plane, denoted as "IDA" curve. A sample of values of S leading to Y=1 is obtained from the set of *n* IDA curves, as shown in Figure 3, left: this sample is used to evaluate the parameters $\mu_{\ln S_{Y=1}}$ and $\sigma_{\ln S_{Y=1}}$.

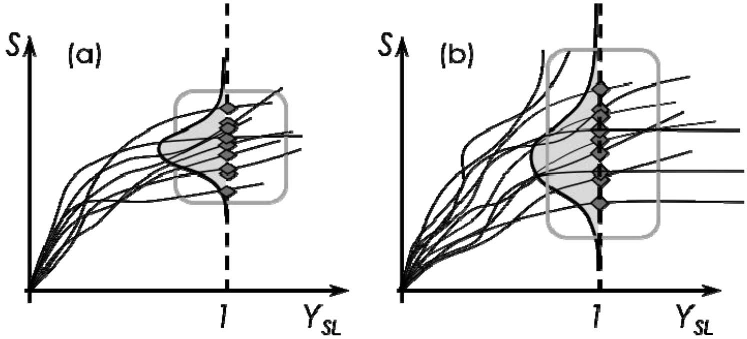

Figure 3. IDA curves and samples of the $S_{Y=1}$ intensity values: a) including record-to-record variability only, b) with structural uncertainty.

The effect of the uncertainties that can be modelled as continuous can be approximately determined by associating to each ground motion a sample of the uncertainties taken from their distributions (the approach is acceptable if the number of time-histories is adequate to describe at least approximately the distribution of the r.v.'s). The effect of the introduction of the uncertainties is visible on the IDA curves by their larger spread (Figure 3, right).

4.1.8.2 Method B: Incremental Dynamic Analysis on an equivalent single degree-of-freedom oscillator

This method differs from the previous one for the fact that the incremental dynamic analyses are carried out on a (number of) "equivalent" single degree–of–freedom (SDOF) oscillators, obtained through nonlinear static (NLS) analysis on the 3D model. Any of the available types of NLS analysis can be adopted, as appropriate for the case at hand.

The global curve relating base shear to the top displacement obtained from the pushover becomes the force-displacement relationship of a simple oscillator, which for the purpose of the response analysis is approximated with a multi-linear relationship.

The number of the needed SDOF oscillators equals the number of modes contributing significantly to the total 3D response. On each SDOF an IDA analysis is performed for all of selected time-histories: for any time-history, modal responses, obtained translating the maximum dynamic response of each SDOF in the response of the 3D structure, at the same intensity level are combined by an appropriate rule (SRSS or CQC) to yield the total response. The latter is used to compute the indicator variable for each LS. Then collection of $S_{Y=1}$ values and evaluation of the fragility parameters $\mu_{\ln S_{Y=1}}$ and $\sigma_{\ln S_{Y=1}}$ proceeds as per method A.

The effect of the uncertainties that can be modelled as continuous can be treated in the same approximate way as in Method A. In this case the pushover analyses must be repeated on different structures each one characterized by a different realization of the uncertainties, and associated one-to-one with the selected motions.

4.1.8.3 Method C: Non-linear static analysis and response surface

This method is again based on nonlinear static analysis. The main differences with respect to method B are two: demand on the SODF oscillators is determined using the response spectra of the selected time-histories (the actual response can be obtained using any of the available methods for obtaining the inelastic displacement response from an elastic spectrum), and the effect of the uncertainties that can be modelled as continuous is determined through the use of the Response Surface technique (Pinto *et al*, 2004).

The two parameters of the fragility function are determined as follows.

The log-mean is obtained from the median response spectrum of the selected time-histories, whose intensity is scaled upwards until Y=1 is obtained:

$$\mu_{\ln S_{Y=1}} = \ln S_{Y=1|S_{a,50\%}(T)} \tag{9}$$

The logarithmic standard deviation is assumed as independently contributed by two factors: the variability of the response due to the variability of the ground motions (given S=s), and the variability due to the randomness of the material properties:

$$\sigma_{\ln S_{Y=1}} = \sqrt{\sigma^2_{\ln S_{Y=1},S} + \sigma^2_{\ln S_{Y=1},C}} \tag{10}$$

The first of the two terms is evaluated from the response spectra fractiles at 16% and 84% from the selected time-histories according to:

$$\sigma_{\ln S_{Y=1},S} = \frac{\ln S_{Y=1|16\%} - \ln S_{Y=1|84\%}}{2} \tag{11}$$

The influence on $S_{Y=1}$ of the continuous random variables, denoted by X_k ,is studied by expressing $\ln S_{Y=1}$ as a linear response surface, in the space of the normalized variables $x_k = (X_k - \mu_{Xk})/\sigma_{Xk}$:

$$\ln S_{Y=1} = \alpha_0 + \sum_k \alpha_k x_k + \varepsilon \tag{12}$$

The normalized variables are assigned the values ±1 in correspondence of their fractile values of 16% and 84%. The N parameters α_k are obtained through a complete factorial combination of the variables at two levels (+1,-1). For each of the $M=2^N$ combinations the median spectrum is increased up to the value producing $Y=1$. The values attributed to the normalized variables (+1or -1) for each of the combinations are the rows of a so-called “matrix of experiments” **Z**, and the corresponding values of $\ln S_{Y=1}$ form a vector of “response” denoted as **y**.

The parameters α_k are then obtained from the expression:

$$\alpha = \left(\mathbf{Z}^T\mathbf{Z}\right)^{-1}\mathbf{Z}^T\mathbf{y} \tag{13}$$

from which the component of $\sigma_{\ln S_{Y=1}}$ related to the uncertainty in the structure (“capacity”) follows as:

$$\sigma_{\ln S_{Y=1},C} = \sqrt{\sum_k \sum_j \alpha_k \alpha_j \rho_{x_i x_j} + \sigma^2_\varepsilon} \tag{14}$$

where σ_{ε}^{2} is the variance of the residuals, and the facts that ε and $\mathbf{x}$ are independent, and the latter are correlated standard variables with correlation coefficient ρ has been used.

4.2 RC-specific provisions

This chapter complements the general chapter 2, by providing detailed indications on modelling of response and capacity for RC structures. As mentioned before the document is based exclusively on nonlinear analysis and prescribes a mandatory verification of the collapse LS. Inelastic models that describe response up to collapse, however, are still not in the average technical background of engineers, and, also, they are still evolving toward a more mature and consolidated state. In recognition of this, the document introduces formulations for the identification of the collapse LS that allow a correct use of the mainstream non-degrading models (equation 4), but leaves the door open to the use of more advanced degrading models (equation 5). Further, in order to guide the user in the selection of the latter, it provides a brief reasoned classification of inelastic response models.

4.2.1 Response models

Models for beam-columns, joints and masonry infills are presented, though the former are obviously given the major attention. In particular, collapse modes of RC columns are described, as schematically shown in Figure 4. The figure illustrates the possible modes of collapse in a monotonic loading condition, in terms of shear force-chord rotation of the member. In all three cases the plot shows with dashed grey lines the monotonic response in a pure flexural mode, with the usual I, II and III stages up to ultimate/peak strength, followed by a fourth descending branch to actual collapse, and the shear strength envelope. The latter starts with $V_{R,0}$ and decreases as a function of deformation, measured in terms of ductility μ. Depending on whether the two curves cross before flexural yield, after, or do not cross at all, the member fails in brittle shear, ductile shear or flexure. In all cases, collapse occurs due to loss of vertical load-bearing capacity (V_R=N_R=0) at the end of the degrading branch.

In cyclic loading at large amplitude the response presents a second contribution to degradation, which is cyclic degradation, as shown in Figure 5.

Available models can be classified in mechanical and phenomenological. The state of the art of purely mechanical models is not yet capable of describing the full range of behaviour of RC members illustrated in Figure 4 and Figure 5 (especially for brittle and ductile shear collapse). Currently, if the analyst wishes to incorporate degrading models, the only viable option is to use phenomenological (e.g. Ibarra et al, 2005) or hybrid models (Elwood, 2004). These models, however, also have their limitations and, for instance, rely heavily on the experimental base used to develop them, which is often not large enough (e.g. for the Ibarra et al. model, the proportion of ductile shear and flexural failures dominate the experimental base, resulting in limited confidence on the model capability to describe brittle failures). Further, computational robustness is an issue with all these models.

Figure 5 shows the monotonic backbone (e.g. for the ductile shear collapse mode) and the cyclic response. It is important to note that the deformation thresholds corresponding to state transitions and ultimately to collapse are different for monotonic and cyclic loading. This fact is highlighted in Figure 6, where the peak/ultimate and axial failure rotations are clearly identified as different in the monotonic and cyclic loading.

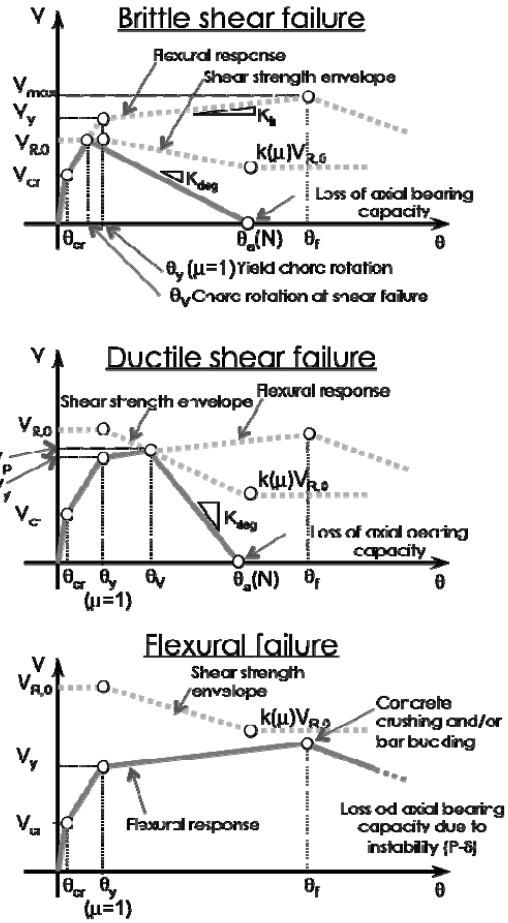

Figure 4. Collapse modes of RC columns (chord rotations at peak strength, usually denoted as ultimate values u, are here differentiated as either shear θ_V or flexural θ_f).

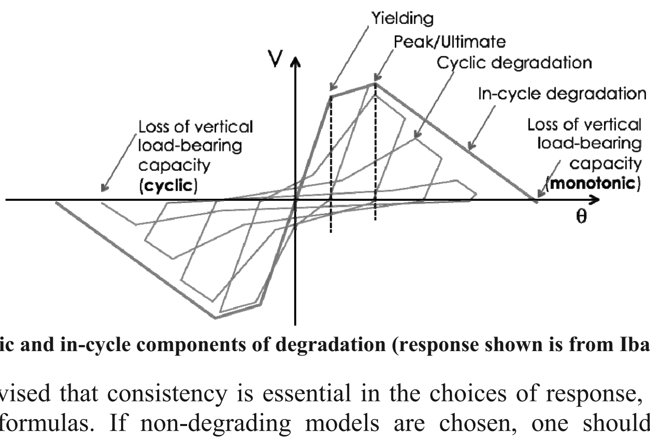

Figure 5. Cyclic and in-cycle components of degradation (response shown is from Ibarra et al model).

The user is advised that consistency is essential in the choices of response, capacity and LS identification formulas. If non-degrading models are chosen, one should use Eq.(4) for collapse identification, with peak deformation thresholds $\theta_{u,cyclic}$ that account on the capacity side for the degradation disregarded on the response side. If degrading models are used, Eq. (5) or (6) are employed, and the monotonic deformation thresholds, $\theta_{u,mono}$, $\theta_{a,mono}$, etc are used as input parameters for the response model (together with degradation parameters).

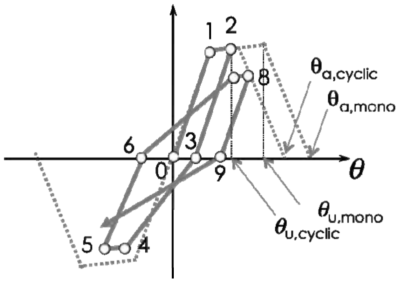

Figure 6. Deformation limits for monotonic and cyclic loading.

4.2.2 Capacity models

A survey of probabilistic models for the deformation thresholds shown before, grouped by LS, is presented in the document. Requirements for an ideal set of models are stated explicitly: consistency of derivation of thresholds of increasing amplitude (i.e. yield, peak and axial deformation models derived based on the same experimental tests, accounting also for correlations), and an experimental base covering the full range of behaviours (different types of collapse, different reinforcement layouts, etc) in a balanced manner. Such a set of models is currently not available.

One set of models that comes closer to the above requirements, and is used in the application illustrated in the next section, is that by Haselton et al (2008), which consists of predictive equations for the parameters of the Ibarra et al 2005 degrading hysteretic model. Haselton et al, however, provide only mean and standard deviation of the logarithm of each parameter, disregarding pair-wise correlation, in spite of the fact that the equations were established on the same experimental basis. Also, as already anticipated, brittle shear failures are not represented.

Figure 7 shows the tri-linear moment-rotation monotonic envelope according to the Ibarra model, with (marginal) probability density functions (PDFs) for its parameters, as supplied by Haselton et al. 2008. Not all the parameters can be independently predicted at the same time, to maintain physical consistency of the moment-rotation law. In the application the stiffness at 40% and 100% of yield, and the rotation increment $\Delta\theta_f$ and $\Delta\theta_a$ have been used (darker PDFs in the figure). Use of the latter two in place of θ_f and θ_a ensures that situations with $\theta_f > \theta_a$ cannot occur. The equation for θ_y is redundant since θ_y is obtained from M_y and K_y. As described in the application, care has been taken in ensuring that K_y is always larger than $K_{40\%}$. The latter is used as an intermediate value between I and II stage stiffness, since the model is tri-linear. Finally, Haselton et al 2008 provide also a marginal model for the parameter regulating cyclic degradation in the Ibarra model, i.e. the normalized total hysteretic energy $E_t/(M_y\theta_y)$.

The document provides also equations by Biskinis and Fardis (2010a,b), adopted since 2005 in earlier form in Eurocode 8 Part 3 (CEN, 2005) and in latest fib Model Code (fib, 2010), as well as by Zhu et al (2007). These equations, however, are calibrated to provide cyclic values of the deformation thresholds, and their use is thus appropriate for LS identification when non-degrading models are used.

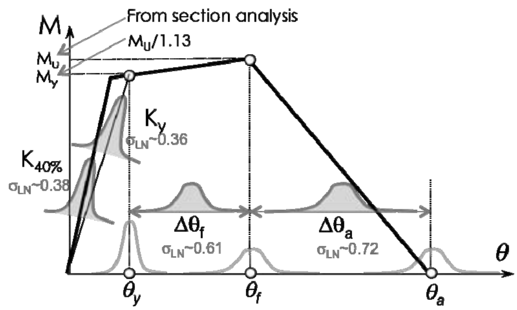

Figure 7. Deformation limits for monotonic loading with schematic indication of the marginal PDF of each parameter.

4.2.3 Biaxial verification

Most response and all available capacity models are applicable for deformation in a single plane of flexure, while the document requires mandatory use of tri-dimensional models. While this does not represent a limitation for beams and for joints, with the exception of corner ones, columns are always subjected to biaxial deformation.

If degrading models are employed, currently the only option is to use the same model independently in the two orthogonal planes of flexure, disregarding interaction.

When non-degrading models are employed, interaction can be accounted for on the response side e.g. by use of fibre-discretized sections, and on the capacity side through the use of an “elliptical” rule for the evaluation of the local, member-level capacity-to-demand ratio (Biskinis and Fardis, 2010a,b):

$$y = \sqrt{\left(\frac{\theta_2}{\theta_{2,LS}}\right)^2 + \left(\frac{\theta_3}{\theta_{3,LS}}\right)^2} \tag{15}$$

where θ_2 and θ_3 are the rotation demands in the two orthogonal planes, and $\theta_{2,LS}$ and $\theta_{3,LS}$ are the corresponding (cyclic) capacities for the LS under consideration.

4.3 Example application to an RC building

The seismic risk assessment of the RC building (one of two applications in the provisions) has been carried out twice, using both non-degrading and degrading models, denoted as A and B, respectively. This has been done to provide users with an order of magnitude of the expected differences between the two approaches. Actually, the document provides results also for a third analysis with masonry infills, not reported here.

4.3.1 Description of the building

The building, shown in Figure 8, is one of three blocks making up a school complex in southern Italy, built in the early ‘60s. The structure consists of an RC space frame with extradosed beams and one-way hollow-core slabs, developing for three storeys over a sloping site. The lower storey is constrained since it is under-ground on three sides.

Figure 8. North-east view of the building.

4.3.2 Seismic action

For the purpose of the evaluation the building has been located at a site in the Basilicata region. Seismic hazard from the current design code, in terms of uniform hazard spectra at nine return periods, has been used to reconstruct median and fractile hazard curves at the first mode period of the structure (see later). The median curve has been interpolated with a quadratic polynomial in log-log space (k_0 = 8.134x10^{-5}, k_1 = 3.254, k_2 = 0.303). Fractile curves have been used to compute a value of the hazard dispersion β_H = 0.3 (at a frequency between 1/500 and 1/1000 years, close to the value of collapse MAF).

Thirty ground motion records have been selected from an aggregated database obtained merging the European Strong Motion database, and the Italian ITACA and SIMBAD databases. Records have been selected in the M_w=[5.6;6.5] and d_{epi} =[10km;30km] ranges (Figure 9), centred around the values obtained from PSHA deaggregation in the same 1/500 and 1/1000 years frequency range.

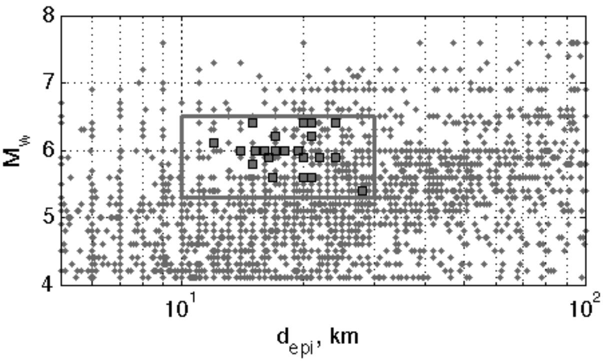

Figure 9. Magnitude and distance bin used in the selection of recorded motions.

4.3.3 Preliminary analysis and results

No construction or design drawings were available. Based on an existing architectural survey, a structural survey was conducted to reconstruct the gross concrete frame dimensions. Based on these and on values for material properties, loads and reinforcement assumed based on the ruling design code at the time of construction a preliminary model was set–up. Modal analysis with full elastic response spectrum has provided the location where the largest inelastic deformation demand is expected. The most stressed columns are framed in red in Figure 10, where actual members chosen for inspection and material sampling (at ground floor) are circled in blue. The results are reported in Table 2.

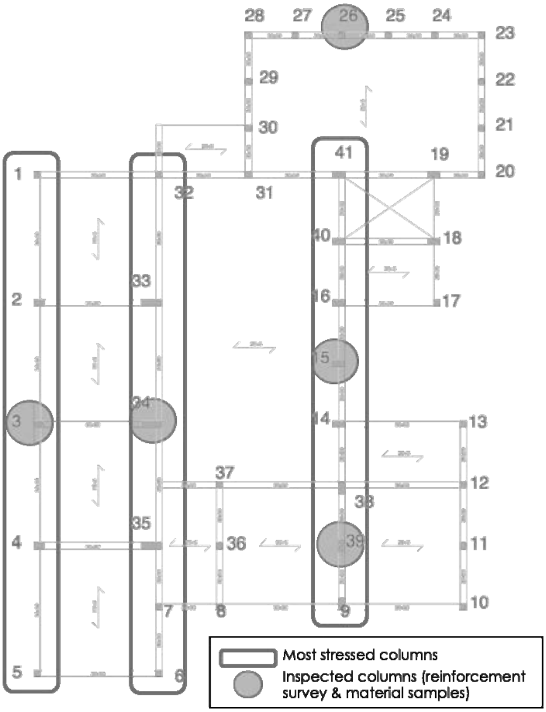

Figure 10. Plan of inspections.

Table 2. Results of tests on columns at ground floor.

Member	B (mm)	H (mm)	Long. Reinf.	Transv. Reinf.	f_c (MPa)	f_y (MPa)
P3	300	500	$6\phi20$	$2\phi6/200$	16.7	-
P15	300	600	$6\phi20$	$2\phi6/200$	15.4	-
P26	300	300	$4\phi12$	$2\phi6/200$	17.8	-
P34	300	1000	$8\phi20$	$2\phi6/200$	11.9	337
P39	300	500	$6\phi12$	$2\phi6/200$	11.6	370

4.3.4 Structural modeling

Structural analysis has been carried out using the general-purpose FE package OpenSees (McKenna et al 2010). The behaviour of RC beam-column joints has not been modelled. Beams and columns have been modelled by means of elastic frame elements with zero length at the two ends, with independent uniaxial constitutive laws on each degree of freedom. The adopted moment-rotation law is the tri-linear one by Ibarra et al (2005), in the implementation by (Lignos and Krawinkler, 2012), and shown in Figure 11 for the two orthogonal planes of flexure of one of the columns. Axial force-bending moment interaction is not included in the model, therefore a constant axial force needs to be assigned at the beginning of the analysis for determination of the model parameters. A single gravity load analysis on the median model has been used to determined axial forces in all columns, and these have been used for all random realizations of the structure (see next section).

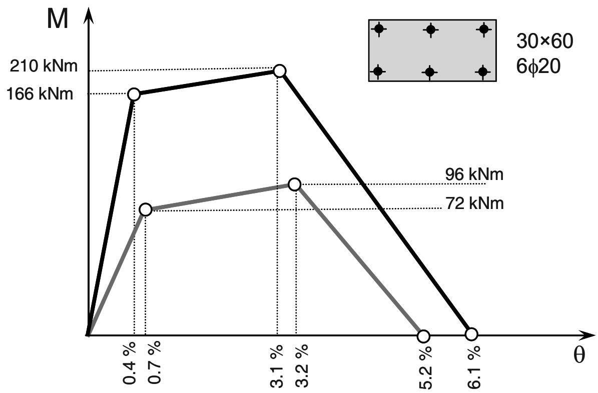

Figure 11. Moment-rotation in two orthogonal planes.

Parameters for the Ibarra model have been predicted with the set of equations calibrated by Haselton et al (2008). These equations include one that provides the degradation parameter:

$$\gamma = \frac{E_t}{M_y \theta_y} \tag{16}$$

Actually, the OpenSees implementation of the Ibarra model requires in input the degradation parameter in the form:

$$\Lambda = \lambda \theta_p = \frac{E_t}{M_y \theta_p} \theta_p = \gamma \theta_y \tag{17}$$

Since method B has been used for the assessment (see later), a unique value of the degradation parameter needs to be assigned to the equivalent oscillator of each mode. The average value of Λ over the columns has been used.

As anticipated, the risk analysis has been performed twice, for both degrading and non-degrading models. In the latter case, for the sake of simplicity, the same Ibarra model has been used, but with zero, rather than negative, post-peak stiffness (e.g. M-θ curves in Figure 11 go flat after 3.1% and 3.2%, respectively). Equation (6) has been used to check the collapse LS, and cyclic thresholds by Zhu *et al* (2007) have been used for the ductile shear (θ_V) or flexural (θ_f) peak deformation. Each member has been attributed a ductile shear or flexural threshold based on the classification criterion proposed in Zhu *et al*, i.e. shear if geometric transverse reinforcement percentage lower or equal to 0.002, or shear span ratio lower than 2 (squat member), or plastic shear V_p=2M_u/L larger than 1.05 the shear strength (according to Sezen and Mohele, 2002). Zhu *et al* model for cyclic axial failure threshold θ_a has also been used for the non-degrading model.

4.3.5 Uncertainty modeling

In this application uncertainties that require analysis of alternative models, to be treated with the logic tree technique, have not been considered.

The uncertainties included in the assessment are:

- Material strengths: f_c and f_y, and ultimate concrete deformation ε_{cu}, which determine the constitutive law of the materials and enter into: a) the stiffness of the elastic members, b) section analysis leading to M_u, c) predictive formulas for deformation thresholds;
- Monotonic incremental deformation $\Delta\theta_f = \theta_f - \theta_y$ and $\Delta\theta_a = \theta_a - \theta_f$, and the cyclic degradation parameter γ, the latter two only for the degrading model;
- Cyclic deformation thresholds θ_f, θ_V and θ_a, for the non-degrading model;

All variables have been modelled as lognormal. As anticipated, statistical dependence of parameters within the same member or between same-parameter across different members has been modelled through assumed correlation coefficients.

In particular, in order to ensure that within each member $K_{40} > K_y$, perfect correlation has been assumed, a single standard normal random variable $\varepsilon_i \sim N(0,1)$ has been sampled in each member, and then amplified by the corresponding logarithmic standard deviation to yield the factors $\exp(\varepsilon_i\, \sigma_{\ln K40})$ and $\exp(\varepsilon_i\, \sigma_{\ln Ky})$ that multiply the corresponding medians.

Similarly, in order to avoid situations where a very ductile element loses axial bearing capacity prematurely, the variables $\Delta\theta_f$ and $\Delta\theta_a$ have been considered perfectly correlated and a single normal variable has been sampled as done for the stiffness.

Finally, in a way of simplicity, same-variables across different members (stiffness, deformation thresholds and material properties) have been considered equicorrelated, independently of distance (one could have used a distance-dependent correlation coefficient, with an exponential or squared exponential model, differentiating correlation lengths in the vertical and horizontal directions), with values reported in Table 3.

Table 3. Distribution parameters for the random variables.

RV	Median	Log-std	Correlation
f_c (MPa)	14.0	0.20	0.7
ε_{cu}	0.006	0.20	0.7
f_y (MPa)	338.0	0.10	0.8
K_{40}	Haselton *et al*	0.38	0.8
K_y	Haselton *et al*	0.36	0.8
$\Delta\theta_f$	Haselton *et al*	0.61	0.8
$\Delta\theta_a$	Haselton *et al*	0.72	0.8
θ_f	Zhu *et al*	0.35	0.8
θ_V	Zhu *et al*	0.27	0.8
θ_a	Zhu *et al*	0.35	0.8
γ	Haselton *et al*	0.50	0.8

Figure 12 shows the moment-rotation law of a member for median values and one of the 30 samples of the random variables. The figure reports also in dashed line the non-degrading branch of the M-θ law, and the corresponding cyclic thresholds used for LS checking.

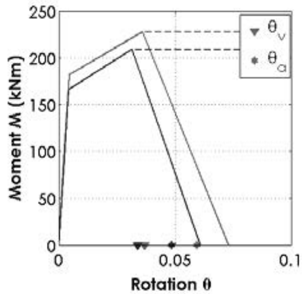

Figure 12. Moment-rotation law for median values (blue) and one sample (red) of the random variables.

1.2 Method B and response analysis via modal pushover

The assessment has been carried out with method B, which uses IDA on equivalent oscillators obtained through nonlinear static analysis to characterize response. Several proposals are available in the literature for the determination of an approximate IDA curve starting from nonlinear static analysis, e.g. (Vamvatsikos and Cornell 2005)(Dolsek and Fajfar 2005)(Han and Chopra 2006). The latter, based on the modal pushover analysis (MPA) technique (Goel and Chopra 2002), has been chosen here due to its easy implementation with commercial analysis packages, since it uses invariant force patterns, and its applicability to general spatial geometries (Reyes and Chopra 2011). Differently from (Reyes and Chopra 2011), however, herein a single excitation that accounts for both orthogonal components of ground motion has been used.

The assessment starts with modal analysis. For each significant vibration mode two nonlinear static analyses are carried out, one for each sign of the forces. The result of each nonlinear static analysis will consist of a database of local responses, i.e. matrices of nodal displacements, of size (n_{steps} x n_{nodes} x n_{dofs}), or of member deformations, of size (n_{steps} x $n_{members}$ x $n_{deformations}$), plus a curve, usually called capacity curve, linking the base shear V_b to the displacement of a control degree of freedom u_c, usually taken to be that with the largest modal coordinate. The capacity curves are approximated by tri-linear laws and transformed into *F/L-D* format. Each tri-linear equivalent oscillator is then subjected to IDA with the 30 selected motions and local responses are obtained by interpolation of the corresponding database at the maximum displacement of the oscillator (for each motion and intensity level). Total responses are obtained from modal ones, at the same intensity S=s, by a suitable combination rule (SRSS or CQC). Based on total response, LS indicator functions Y are evaluated.

1.3 Results

Modal analysis of the median model (i.e. a model with median values assigned to all random variables) shows that the first three modes cumulatively account for more than 80% of the total mass in both plan directions (Figure 13). These mode shapes are the same for models A and B, since they have the same elastic properties.

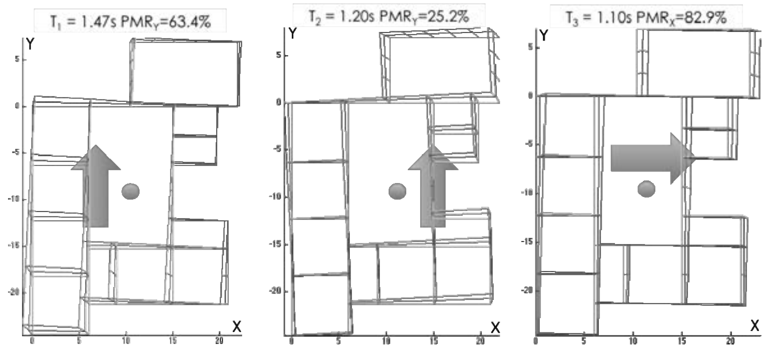

Figure 13. Plan view of the first three mode shapes, with participating mass ratios in the dominant direction of each mode ("median" model).

These three modes are chosen for nonlinear static analysis. Figure 14, left, shows the corresponding results in terms of capacity curves with reference to model A. The figure shows also the tri-linear approximations of the curves used as monotonic backbone for the equivalent oscillators. The post-peak negative stiffness for this non-degrading model is entirely due to geometric effects (P-δ). Figure 14, right, compares the capacity curves for the three considered modes obtained with model A (red) and B (black), respectively. The curves depart from each other only after some excursion in the inelastic range, when the first local failure (exceedance of the ultimate/capping deformation) occurs. The total number of pushover analysis amounts to 2 signs x 3 modes x 30 models = 270, as shown in Figure 15.

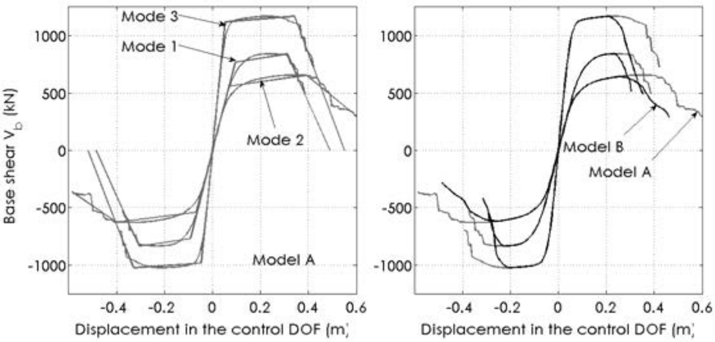

Figure 14 Pushover curves for model A and B.

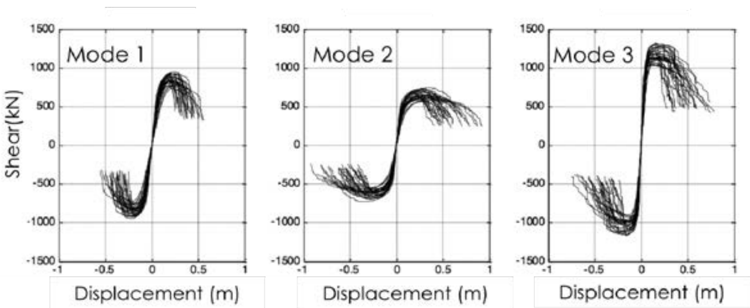

Figure 15. Pushover curves of 30 random samples of model A.

Figure 16 shows further details of the nonlinear static analysis, with the capacity curve of one of the 30 random realizations of Model B, subjected to modal forces according to its 3^{rd} mode, in the positive sign, and the deformed shapes (same scale) at three steps corresponding to increasing levels of inelastic demand. The first and second step (S1 and S2 in the figure) correspond to the yield and peak displacement in the tri-linear approximation of the capacity curve, the last step S3 is midway between the peak and the last point. The deformed shapes report also the level of inelastic demand in plastic hinges, according to the convention already used in (Haselton and Deierlein, 2007): hollow circles denote potential plastic hinge zones, blue and red circles denote inelastic demands lower and higher of the peak rotation, respectively. The diameter, for blue and red circles, is proportional to the D/C ratio. The blue circle fills completely the hollow black circle when y=1 (Eq. (10)), with $\theta_{LS} = \theta_f$ or θ_V. It can be observed that along the descending branch increases at some locations to more than three times the diameter of the black circle. This situation is numerically possible since the loss of axial load-bearing capacity is not modelled, and the analysis proceeds with redistribution of shear demand on the adjacent members. This fact, however, does not compromise the analysis since the axial collapse mode is actually checked a posteriori, using the θ_a model from Zhu et al (2007) in conjunction with (6).

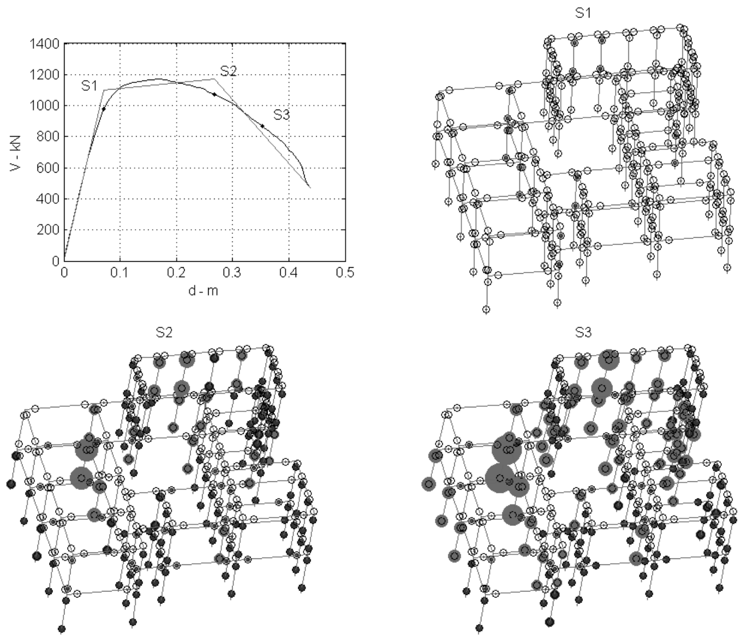

Figure 16. Model B, Mode 3, pushover curve and deformed shapes at three different displacement levels, with indication of plastic hinge deformations (hollow circles, blue circles and red circles denote potential plastic hinges, active hinges before peak/ultimate deformation and hinges in the degrading post-peak branch, respectively).

Figure 17 shows the response time-series for the equivalent oscillator (Model B, Mode 3, first random sample and associated motion) at three increasing intensity levels, shown below in terms of force-displacement loops. Depending on whether the largest response displacement has a positive or negative sign, the local responses at node/member level are interpolated from the database relative to the positive or negative pushover.

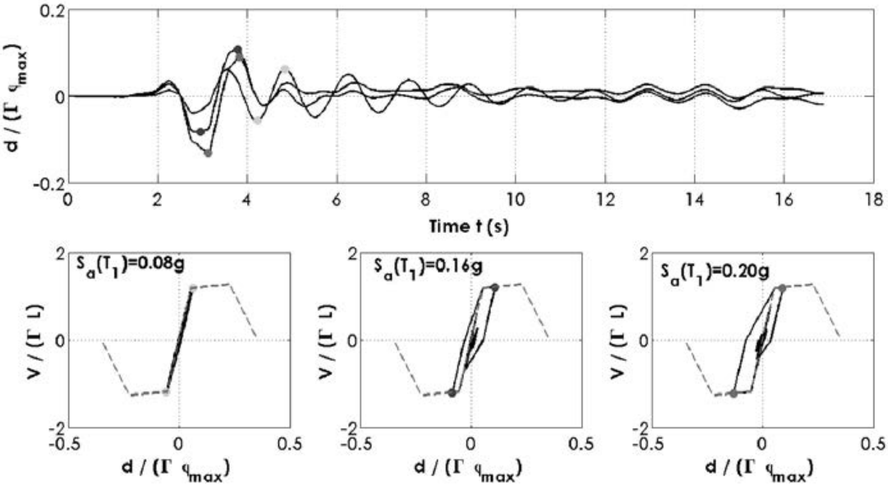

Figure 17. Model B, Mode 3, response of the equivalent oscillator to the same motion at three increasing intensity levels (top) and corresponding force-deformation loops (bottom).

Finally, Figure 18 and Figure 19 show the IDA and the fragility curves for model A (left) and B (right), respectively. Green, blue and red dots on the IDA curves mark the attainment (Y=1) of the damage, severe damage and collapse LS. Each cloud of points is used to determine the log-mean and log-standard deviation of the intensity leading to the corresponding LS: $S_{Y=1}$, parameters of the fragility curves reported below.

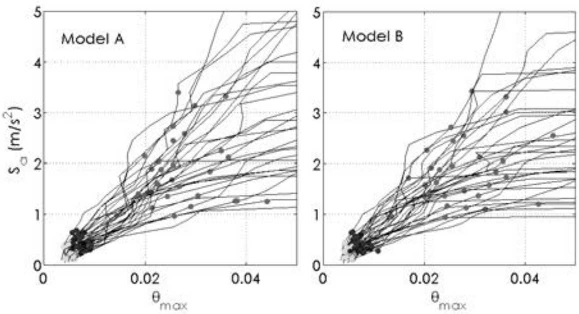

Figure 18. IDA curves with indication of intensity leading to each LS for all records.

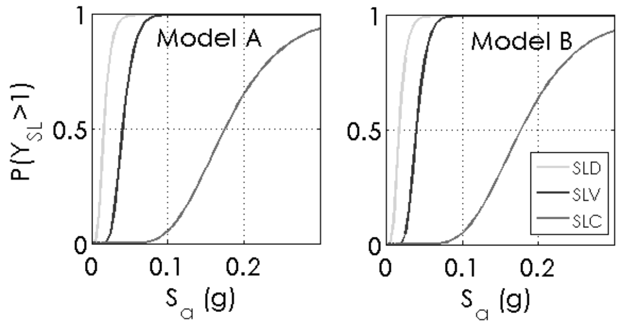

Figure 19 Fragility curves.

Convolution of the fragility curves in Figure 19 with the hazard curve for the corresponding intensity measure, $S = S_a(T_1)$, yields the values of the mean annual rate of exceedance of the three LS's reported in Table 4. The table reports also the MAF thresholds for this school building (Class III structure, Table 1). As it can be seen, for the considered example the MAFs from the two modelling approaches are practically coincident for all LSs.

Table 4. Mean annual frequencies of LS exceedance for the two models and corresponding thresholds.

Model	A	B	Threshold
λ_{SLD}	0.03150	0.03040	0.0300
λ_{SLV}	0.01270	0.01310	0.0032
λ_{SLC}	0.00119	0.00117	0.0015

In conclusion, the example shows that the method is of relatively lengthy but rather straightforward application to real buildings, requiring in sequence a modal analysis, random sampling of model realizations, pushover analyses with invariant modal patterns, tri-linear approximation of capacity curves, expeditious IDA on equivalent SDOF oscillators, interpolation in the local response databases and CQC/SRSS combination, fragility parameters evaluation by simple statistical operations on the $S_{Y=1}$ intensity values. As long as MPA can provide a reasonable approximation of the dynamic response, Method B is a computationally effective alternative to Method A (IDA on complete model), since it requires determination and handling of much smaller response databases: where Method A requires determination of $n_{responses}$ x n_{steps} x $n_{IM\text{-}levels}$ quantities per each record/model pair (with e.g. $n_{steps} = 2000$ steps and $n_{IM\text{-}levels} = 10$), Method B requires determination of $n_{responses}$ x n_{steps} x n_{modes} quantities only (with e.g. $n_{steps} = 100$ steps and $n_{modes} = 3 \div 5$), since the IDA is carried out on a SDOF oscillator.

5 DISCUSSION

In full recognition that further progresses are needed in certain areas, and that research is still very actively working to improve the present state of knowledge, Task RS4 has considered the knowledge already available sufficient for producing a document providing the necessary instructions for guiding users willing to access the reliability world in order to evaluate the seismic risk, with specific reference to existing building structures.
The absence of similar documents internationally has not permitted the healthy exercise of critical comparisons, hence the responsibility for the choices on the content of the document rests entirely with Task RS4: use of the document will suggest improvements on various aspects, and the same will do the progresses of the research in the field.

6 VISIONS AND DEVELOPMENTS

The provisions for probabilistic seismic assessment of existing buildings produced within Task RS4 and published by the National Research Council are aligned with the consolidated state of the art in the field. The overall framework, and in particular the adoption of the hazard-fragility split of the risk integral, are not deemed to be challenged soon by more efficient alternatives. The provisions are conceived as open to new developments, explicitly recognizing that specific aspects are subject of on-going research (e.g. response and capacity modelling into the nonlinear range up to collapse for non-conforming members). Alternative techniques, e.g. for the selection of natural motions, or for the derivation of the fragility functions, are already mentioned in the annexes. Improvements are to be expected in the response and uncertainty modelling with respect to site response analysis and soil-structure interaction. In sum the product of this research represents a reference that can inspire normative action with respect to the assessment of existing buildings.

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Finito di stampare nel mese di novembre 2015 per conto di Doppiavoce
presso Officine Grafiche Francesco Giannini & Figli S.p.A. – Napoli