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editor

Eurocode 8 Perspectives from the Italian Standpoint Workshop

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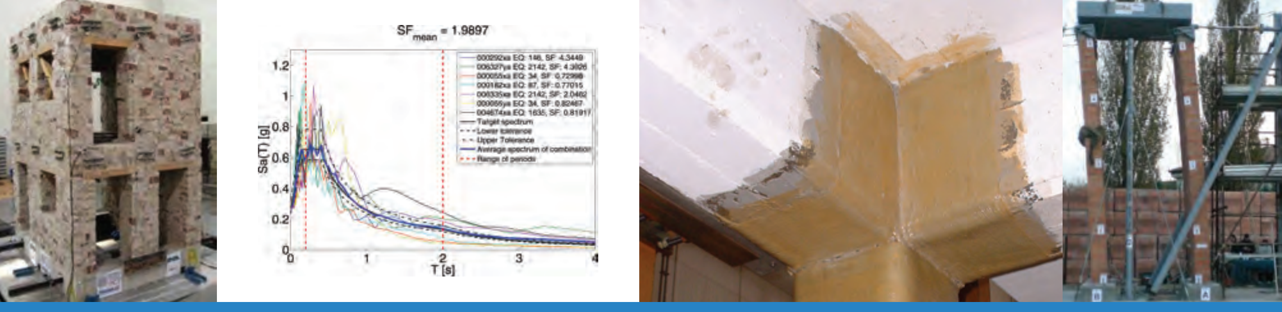

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FOREWORD

Naples, Italy – July 2009

Two non independent events significantly affected Italian earthquake engineering community in the last few years. First of all the Italian code was superseded by one comparable, for quality and technical content, to the last generation of seismic codes at international level. Second, the ReLUIS (*Rete dei Laboratori Universitari di Ingegneria Sismica*, http://www.reluis.it) consortium, networking the institutions with the largest facilities, both experimental and numerical, for earthquake engineering research, was born. ReLUIS was founded by the Italian Department of Civil Protection (DPC) via a 15 billion of Euros project developed between 2005 and 2008; the largest project in earthquake engineering ever in Italy for both funding and number of researchers involved, about six hundreds.

Needless to say, the new Italian code is based on Eurocodes, still it has benefitted of some state-of-the-art advances brought in the community by the ReLUIS project, which actually had as one of the main purposes the development of the seismic code. I had the opportunity of following such a process from a close standpoint as a past president of the consortium, and personally think this proximity with alive and active research was successful and visible in the code. This is especially true with respect to those aspects related to assessment and retrofit of existing, both reinforced concrete and masonry, structures (it is to recall that existing buildings are certainly the largest issue regarding structures in Italy which has the most of seismic risk carried by these type of constructions), but also for what concerns geotechnical earthquake engineering, and finally seismic actions on structures. This latter goal, could have not been achieved without the advanced probabilistic seismic hazard analysis provided by INGV (Instituto Nazionale di Geofisica e Vulcanologia) and DPC, which is now available for the whole national territory allowing to determine design seismic actions on a rational basis, yet manageable by practitioners.

The osmosis between the two processes is even clearer if one thinks that two consecutive ReLUIS chairs, I and Mauro Dolce, were also in the new seismic code committee. Conversely, the ReLUIS research was stimulated by many code-based issues.

On the other hand, earthquake engineering research itself not only developed results to be taken in by the new code, but also performed large experimental tests distributed all over the country still non-overlapping and coordinated at national level. The amount and quality of experimental data gathered within the ReLUIS project is something unseen before, probably not only for Italy.

The results are not only represented by the new code, which was enforced on July 1st 2009, but also by the step ahead of earthquake engineering as a whole, ranging from the mentioned improvement in understanding of seismic risk of existing structures to new design paradigms and innovative approaches to seismic risk reduction as well as emergency management, directly employed in the recent L'Aquila earthquake in which ReLUIS was side-to-side with DPC acting as one of its centers of competency.

It is my belief that part of the advances implemented in the code and supported by consistent research, may be useful for the developments of Eurocodes, and this motivated the publications of the proceedings of the workshop giving the title to this book, which is divided in chapters reflecting the Eurocode 8 structure: i.e., *Ground Conditions and Seismic Action*, *Concrete Buildings*, *Steel and Concrete Composite Structures*, *Masonry Buildings*.

I finally can't skip to thank those enthusiastically participating to all of this, Gaetano Manfredi current president of ReLUIS who managed to integrate the workshop at the end of the final meeting of the ReLUIS project where the main results were presented, and Iunio Iervolino who has helped in organizing the workshop and the proceedings.

Edoardo Cosenza

GROUND CONDITIONS AND SEISMIC ACTION

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SHEDDING SOME LIGHT ON SEISMIC INPUT SELECTION IN EUROCODE 8

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ABSTRACT
Eurocode 8 allows the use of real records as an input for nonlinear dynamic analysis; nevertheless, it has been found hardly applicable by practitioners. This is related to both the difficulty in rationally relating the ground motions to the hazard at the site and the required selection criteria, which may favor the use of various types of spectral matching signals rather than real records. To overcome, at least the latter problem, a specific software, namely REXEL, freely available at http://www.reluis.it/index_eng.html, was developed by the authors. It allows to search for suites of waveforms compatible to any arbitrary reference elastic response spectrum, according to Eurocode 8 criteria. Combinations of records are also *optimal* with respect to other selection criteria found important by recent research on the topic. In the paper, record selection in Eurocode 8 is briefly reviewed first, then, via some simple examples, it is shown how REXEL can solve most of the issues related to real records selection for seismic structural analysis with respect to Eurocode 8 provisions.

KEYWORDS
Eurocode 8, record selection, seismic hazard, seismic design, response spectrum, REXEL.

1 INTRODUCTION

One of the key issues in non-linear dynamic analysis of structures is the selection of appropriate seismic input, which should allow for an accurate estimation of the seismic performance on the basis of the seismic hazard at the site where the structures is located.
If the probabilistic risk assessment of structures is concerned, procedures have been recently developed to properly select the seismic input. The basic steps are: (i) choosing a ground motion (GM) parameter considered to be representative of the earthquake potential with respect to the specific structure (i.e., a GM intensity measure, or IM[1]); (ii) to obtain the probabilistic seismic hazard analysis (PSHA) at the site, and disaggregation, for the chosen IM; (iii) to determine probability of collapse in terms of one or more engineering (structural) demand parameters, or EDP, as a function of the IM, i.e., the fragility function; (iv) to average

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[1] For various reasons, one of the main being that hazard is easily computable, the IM is often related to the response spectrum of the record; e.g., the peak ground acceleration or PGA, and the spectral acceleration at the first mode, or some function of the spectral shape in a range of period of interest.

the fragility over the hazard to obtain the overall failure probability; i.e., the seismic risk (Cornell, 2004).
Consistently with the load-resistance factor design (LRFD), code-based procedures apparently approximate this procedure in a semi-deterministic fashion (Iervolino and Manfredi, 2008). They often require to define a design (reference) spectrum whose ordinates have a small probability of exceedance[2] during a given time period. Secondly, a scenario event or design earthquake has to be defined referring to the local seismicity (although the link of the design spectrum with the hazard at the site may be very weak). Then, in the case of nonlinear dynamic analysis, codes basically require a certain number of records to be chosen consistently with the design earthquake and the code spectrum in a broad range of periods. Finally, the performance of the structure is assessed verifying whether the maximum or average response of the structure to the record exceeds the seismic capacity.
Studies show how the most of practitioners may experience difficulties in handling code-based record selection, first of all because determining the design earthquakes may require hazard data often not readily available to engineers, or, when these are provided by authorities (e.g., in Italy), it may still require seismological skills beyond their education. Furthermore, if real records are concerned, to find a suite matching a design spectrum in broad range of periods, may be hard or practically unfeasible if appropriate tools are not available (Iervolino et al., 2008; Iervolino et al., 2009a). These issues traditionally favored the use of spectrum matching accelerograms, either artificial or obtained through manipulation of real records. On the other hand, real records are recognized by many as the best representation of seismic loading for structural assessment and design, motivating attempts to develop tools for computer aided code-based record selection, one of which being that of Naeim et al. (2004).
The recent years' work of the authors in this direction resulted in REXEL, a computer software freely distributed over the internet, which allows to build design spectra according to Eurocode 8 (EC8) (CEN, 2003), the new Italian seismic code (NIBC) (CS.LL.PP., 2008), or completely user-defined, and to search for sets of 7, 14 or 21 groups[3] of records, from the European strong motion database. These sets are compatible to the reference (i.e., target) spectra with respect to codes' prescriptions, but reflect also some research-based criteria considered relevant for seismic structural assessment.
REXEL searches for sets of records for a various range of structural applications and seems to actually make the selection quick and effective in most of cases. In the following, the procedures concerning determination of seismic action and record selection, according to EC8, are briefly reviewed along with findings of other studies on the topic. Then, the software algorithms are described; the use of REXEL is illustrated via some examples which show how it can effectively aid code-based record selection for seismic structural analysis.

[2] This is, in principle, analogous to choose a conservative value of the action in the LRFD in which actions are amplified and the capacity is reduced on a probabilistic basis.
[3] Each group may be made of 1, 2 or 3 component GMs. This means in the case of size 21, for example, the set features 63 GMs.

2 RECORD SELECTION IN EC8

2.1 EC8 Part 1 – Buildings

In EC8 – Part 1 (CEN, 2003) the seismic action on structures is defined after the acceleration elastic response spectrum. In Part 1, which applies for buildings, the spectral shapes are given for both horizontal and vertical components of motion. In *section*[4] *3.2.2* of the code two spectral shapes, *Type 1* and *Type 2*, are defined, the latter applying if the earthquake contributing most to the seismic hazard has surface wave magnitude not greater than 5.5, otherwise the former should be used. All shapes have a functional form which depends, a part of the soil class, on a single value, a_g, anchoring the spectrum to the seismicity of the site. a_g refers to the seismic classification of the territory in each country; it is basically related to the hazard in terms of peak ground acceleration (PGA) on rock for the site.
Once the reference spectrum has been defined, EC8 – Part 1 allows the use of any form of accelerograms for structural assessment; i.e., real, artificial or obtained by simulation of seismic source, propagation and site effects. To comply with Part 1 the set of accelerograms, regardless its type, should basically match the following criteria:

a) a minimum of 3 accelerograms should be used;

b) 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 S for the site in question (S is the soil factor)*;*

c) in the range of periods between $0.2T_1$ and $2T_1$, where T_1 is the fundamental period of the structure in the direction where the accelerogram will be applied, no value of the mean 5% damping elastic spectrum, calculated from all time histories, should be less than 90% of the corresponding value of the 5% damping elastic response spectrum.

According to the code, in the case of spatial structures, the seismic motion shall consist of three simultaneously acting accelerograms representing the three spatial components of the shaking, then *3* of condition (a) shall be considered as the number of translational components of motion to be used (the two horizontal and the vertical one).
In *section 4.3.3.4.3*, the code allows the consideration of the mean effects on the structure, rather than the maxima, if at least seven nonlinear time-history analyses are performed. Moreover, the vertical component of the seismic action should be taken into account only for base-isolated structures, and for some special cases in regular buildings, if the design vertical acceleration for the A-type site class (a_{vg}) is greater than 0.25g. Finally, some prescriptions regarding duration are given for artificial accelerograms, and real or simulated records should be *adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site*.

[4] References to sections and verbatim quotations of codes are given in Italic hereinafter.

2.2 EC8 Part 2 – Bridges

EC8 – Part 2 (CEN, 2005) refers to the same spectral shapes of Part 1 in order to define the seismic input for time-history analysis of bridges. The requirements for the horizontal components are somehow similar to those for buildings but not identical. The relevant points are:

a) *for each earthquake consisting of a pair of horizontal motions, the SRSS spectrum shall be established by taking the square root of the sum of squares of the 5%-damped spectra of each component;*

b) *the spectrum of the ensemble of earthquakes shall be formed by taking the average value of the SRSS spectra of the individual earthquakes of the previous step;*

c) *the ensemble spectrum shall be scaled so that it is not lower than 1.3 times the 5% damped elastic response spectrum of the design seismic action, in the period range between $0.2T_1$ and $1.5T_1$, where T_1 is the natural period of the fundamental mode of the structure in the case of a ductile bridge.*

As Part 1, Part 2 also allows the consideration of the mean effects on the structure when non-linear dynamic analysis is performed for at least seven independent GMs and the vertical action has to be considered in special cases only. Part 2 has specific prescriptions for near-source conditions and cases in which the spatial variability of GM has to be considered.

2.3 Findings of previous investigations about record selection in EC8

In other studies the authors investigated the actual applicability of EC8 prescriptions about record selection. In particular, in Iervolino et al. (2008) it was investigated whether it is possible to find unscaled real record sets fulfilling, as much as possible, the requirements of EC8 – Part 1. The investigations were based on the former Italian classification in seismic zones now superseded by the new seismic code (see following section), but still enforced in many countries.
Combinations were found for sites featuring moderate-to-low seismicity, while it was not possible to find suitable results for the more severe design spectra. Moreover, the condition of having un-scaled record sets strictly matching EC8 spectra resulted in a large record-to-record variability in the spectral ordinates within the same set. Both shortage of compatible sets for the severe spectra and large individual spectra scatter, could be avoided searching for records with a spectral shape as similar as possible to that of the code after rendering the spectra non-dimensional dividing their ordinates by the PGA. Nevertheless, this implies amplitude scaling of the records and may lead to large scaling factors.
As a general conclusion it was found that prescriptions do not easily allow to select suitable real record sets, factually favoring the use of records obtained either by computer techniques or manipulation of real records to have a spectral shape coincident to that of the reference in a broad range of periods. This is mainly because:

1) it is almost unfeasible for practitioners to search in large databases to find suites of seven real records (eventually multi-component) whose average matches closely the design spectral shape without a specific software tool;

2) the spectra based on seismic zonation may be too severe in a way that do not exist suites of unscaled records whose average has such spectral shape;

3) it is not easy to control the variability (very large) of individual spectra in a combination, and this, in the fortunate case combinations are found, may impair the confidence in the estimation of the seismic performance using such set;

4) the requirement of selecting records consistent with the earthquake events dominating the hazard at the site (i.e., the design earthquakes) requires PSHA/disaggregation data and skills seldom available to practitioners;

In Iervolino et al. (2009a) a similar study concerning EC8 – Part 2 was carried out. It was found that, although seemingly different, the requirements of the two part of the code are substantially equivalent and lead to similar results in terms of combinations found (i.e., combinations complying with Part 2 are likely to comply also with prescriptions of Part 1) and limitation of applicability to real records. As discussed in the following, tools as REXEL may help to overcome some of the issues 1 to 4 above.

3 REXEL 2.31 BETA

To enable record selection according to both approaches of EC8 and NIBC, a specific software tool was developed (Iervolino et al., 2009b). It features a MATHWORKS-MATLAB® graphic user interface (GUI, Figure 1) and a FORTRAN engine, based on the software developed for the studies in Iervolino et al. (2008 and 2009a). In particular, the computer program was developed to search for combinations of seven[5] accelerograms compatible in the average with the reference spectra according to code criteria discussed above. It is also possible to reflect in selection the characteristics of the source (if available) and site, in terms of magnitude (M), epicentral distance (R), and EC8 soil site classification. In fact, REXEL 2.31 beta, freely available on the internet on the website of the Italian consortium of earthquake engineering laboratories (*Rete dei Laboratori Universitari di Ingegneria Sismica – ReLUIS*; http://www.reluis.it/index_eng.html), currently contains the accelerograms belonging to the European Strong-motion Database, or ESD, (http://www.isesd.cv.ic.ac.uk/; last accessed July 2007) (Ambraseys et al., 2000 and 2004).
The procedure implemented for record selection deploys in 4 basic steps:

1. definition of the design (reference) horizontal and/or vertical spectra the set of records has to match on average; the spectra can be built according to EC8, NIBC, or user-defined;

2. list and plot of the records contained in the ESD and embedded in REXEL which fall into the magnitude and distance bins specified by the user for a specific site class;

[5] Seven has to be intended as the size of the set which may include 1-component, 2-components, or 3-components records, which means 7, 14 or 21 waveforms respectively.

3. 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;

4. 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 3.

Other functions are related to visualization of results, return of selected waveforms to the user, and secondary options, as search for combinations of size larger than 7. For the complete user guide one should refer to the REXEL tutorial (Iervolino and Galasso, 2009).

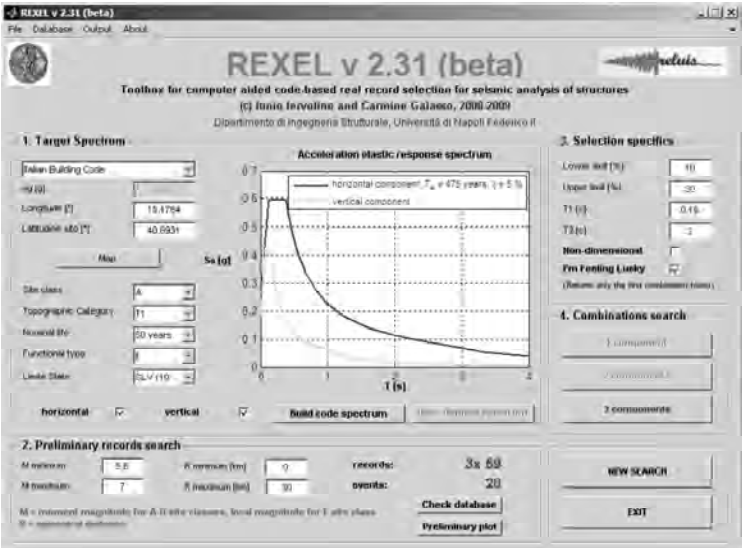

Figure 1. Image of the software GUI.

As an example of the software use let's consider selection of horizontal accelerograms according to EC8 for the life safety limit state of an ordinary structure on soil type A with a nominal life of 50 years (which corresponds to design for a 475 years return period according to the code) and located in Sant'Angelo dei Lombardi (15.1784° longitude, 40.8931° latitude; close to Avellino in Southern Italy). Setting the coordinates of the site and the other parameters to define the seismic action according to EC8, the software automatically builds the elastic design spectrum. Consider also that selection should reflect disaggregation of PGA hazard[6] on rock with a 10% probability of exceedance in 50 years (Figure 2) at the site, which may be easily obtained by the INGV website (http://esse1-gis.mi.ingv.it/s1_en.php). Specifying the M and R intervals to [5.6, 7] and [0km, 30km] respectively, REXEL 2.31 beta

[6] Note that it is often recommended to consider as design earthquakes the results of hazard disaggregation for the spectral ordinates in the range of interest for the nonlinear structural behavior. This may differ from disaggregation of PGA hazard, especially when M and R joint distribution has multiple modal values (Bazzurro and Cornell, 1999).

founds 177 records (59 x 3 components of motion) from 28 earthquakes. REXEL will search among these spectra.

Assigning, as tolerances for the average spectral matching, 10% lower and 20% upper in the period range 0.2s ÷ 2s and selecting the option to stop the search after the first combination is found (i.e., *I'm feeling lucky*), REXEL immediately returns the combinations of accelerograms in Figure 3a if 1-component search is performed. In the figure, which the software automatically plots, thick solid lines are the average of the set and the code spectrum, while the dashed are the tolerance and period range bounds where compatibility is ensured. Solid thin lines are the seven individual spectra of the combination. In the legend the ESD station and component codes, along with the earthquake code, are given.

Selecting the search for set of seven pairs of horizontal components (e.g., for the analysis of spatial structures), instead, the software returns the 14 records of Figure 3b. Note that in this case the records are 7 pairs of X and Y components of 7 recordings only.

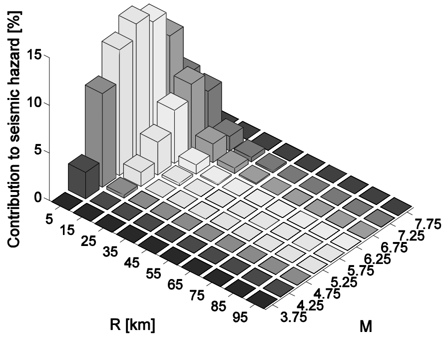

Figure 2. Disaggregation of the PGA with 475 years return period on rock for Sant'Angelo dei Lombardi.

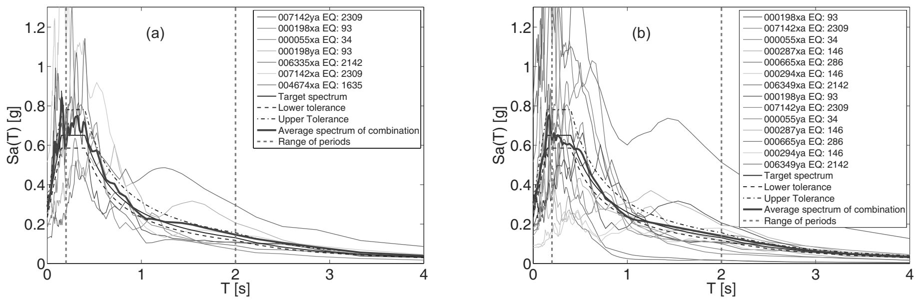

Figure 3. Unscaled combinations found for the assigned example in Sant'Angelo dei Lombardi using the *I'm feeling lucky* option in the case of horizontal 1-component (a) and 2-components (b) GMs.

It is was claimed in the description of the software that the first combination has a low scatter with respect to subsequent combinations eventually found. This may be shown, for example, not selecting the *I'm feeling lucky* option and limiting the maximum number of combinations to 1000, for simplicity, and repeating the same two searches above. Then, REXEL returns, in about one minute with a standard personal computer, 1000 1-component compatible combinations, and in a few minutes, 374 combinations featuring both the two horizontal components of motion. Figure 4a and Figure 4b show the last combination (no. 1000 and no. 374 respectively) of the output list in the two cases.

Although all average spectra of Figure 3 and Figure 4 match with similar good approximation the code spectrum, it is evident that the preliminary ordering of accelerograms according to δ_i enables the initials combinations to have individual spectra with the smallest dispersion with respect to the reference spectrum.

Nevertheless, the presented results show that the deviation of the individual spectrum compared with the reference can still be large (e.g., Figure 3b). To reduce the scatter of individual records further, the *Non-dimensional* option can be used, which means that records found have to be linearly scaled to be spectral matching in the average. In this case, repeating the search for Sant'Angelo dei Lombardi simply considering accelerograms with $M \geq 6$ and R within 0km ÷ 25km, with the same compatibility criteria as the previous case and using the option *I'm feeling lucky*, the software returns immediately combinations shown in Figure 5a and Figure 5b, which feature records less scattering with respect to those unscaled of Figure 3. In this case the individual scale factor is given in the legend. Note also that the code asks for the maximum value the average scale factor (SF_{mean}) can assume, which in this case was limited to 2.

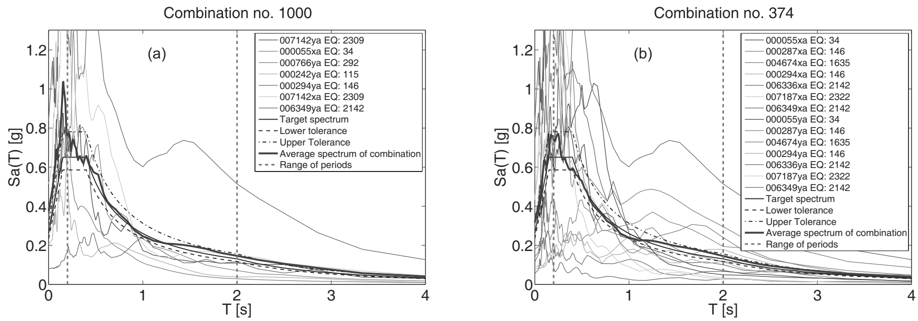

Figure 4. Last of 1000 unscaled combinations found for the assigned example in Sant'Angelo dei Lombardi using the *I'm feeling lucky* option in the case of horizontal 1-component (a) and 2-components (b) GMs.

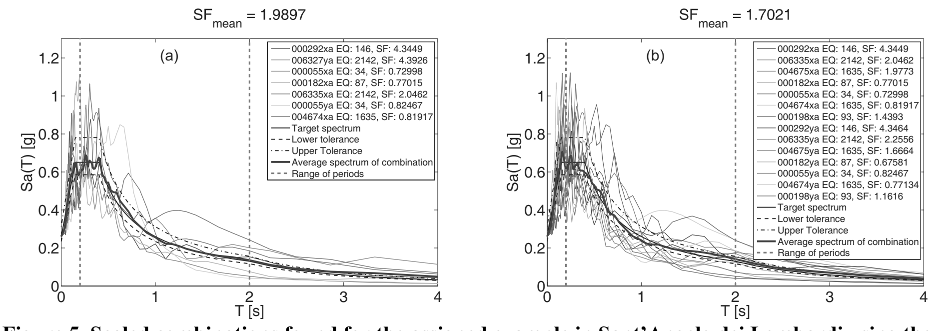

Figure 5. Scaled combinations found for the assigned example in Sant'Angelo dei Lombardi using the *I'm feeling lucky* option in the case of horizontal 1-component (a) and 2-components (b) GMs.

REXEL 2.31 beta allows selecting combinations of accelerograms that include the vertical component of the records, although EC8s require to account for it only in particular cases. Considering again the same example in Sant'Angelo Lombardi, and specifying as the M and R intervals as [6,7.8] and [0km, 50km] respectively, REXEL founds 58 groups of accelerograms from 23 events.

Assigning a tolerance compatibility of the average of 10% lower and 20% upper for the horizontal component, and 10% lower and 50% upper for the vertical component, in the range of periods 0.2s ÷ 2s (for the horizontal components) and 0.2s ÷ 1s (for the vertical component), the software returns 23773 scaled combinations compatible with the horizontal reference spectrum (the maximum number of combinations was limited to 100000, for simplicity), 172 of which are also compatible with the vertical reference spectrum. The maximum value the average scale factor can assume was limited to 3.

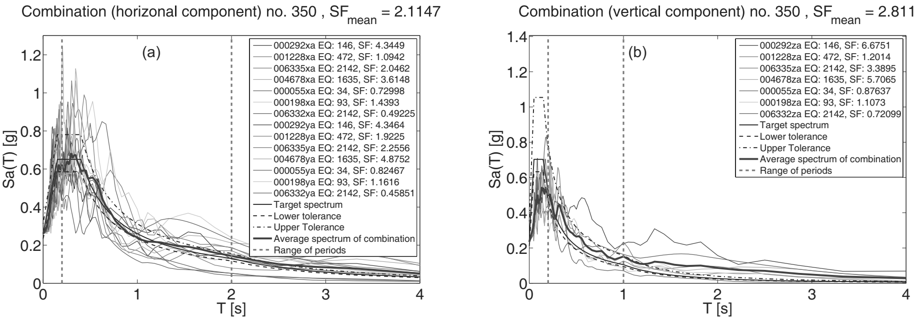

Figure 6. First scaled combination found for the assigned example in Sant'Angelo dei Lombardi which includes the two horizontal (a) and the third vertical (b) components of GM matching the respective reference spectra.

Finally, note that when searching for combinations that include the vertical component, it may not be appropriate to use the *I'm feeling lucky* option. In fact, the first combination compatible with the horizontal code spectrum (returned by the software) may not necessarily satisfy the compatibility criteria with the vertical spectrum. For example, in the considered case the first combination which has all the three components matching the reference spectra (Figure 6) only comes after 350 combinations found which match the horizontal spectrum.

4 CONCLUSIONS

A software tool developed for automatic selection of seven recordings including 1, 2 or 3 components of ground motion was presented. The main selection criterion, for unscaled or scaled sets, is the compatibility, in broad period ranges, of the average spectrum with the design spectra (which the program automatically builds) of Eurocode 8, the new Italian building code, or user-defined. REXEL 2.31 beta, freely available on the internet on the RELUIS website, allows multiple selection options that also may account for design earthquakes in terms of magnitude and distance. Moreover, it ensures that individual records in the combination have a spectral shape like that of the code as much as possible, which is important as spectral shape is currently seen as the best proxy for earthquake damage potential on structures. The current version of the software relies on records from the European strong motion database; nevertheless, data from other repositories could be easily implemented.

As the illustrative applications demonstrate, also with respect to previous studies which found difficult to practically perform code-based record selection, the determination of spectrum-compatible sets of records can be significantly improved and facilitated. REXEL, therefore, may prove to be an useful tool for practitioners to select the input of seismic input for code-based seismic structural assessment via nonlinear dynamic analysis.

5 ACKNOWLEDGEMENTS

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EVALUATION OF EC8 SITE-DEPENDENT ACCELERATION RESPONSE SPECTRA USING STRONG-MOTION ITALIAN RECORDS

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ABSTRACT
The Eurocode 8 (EC8) elastic acceleration spectra are compared with those which were derived from a representative sample of acceleration records from Italy. The source of the records is the recently developed strong-motion database SISMA (Site of Italian Strong Motion Accelerograms). The data were carefully selected based on magnitude, distance, focal depth and free-field conditions, and grouped in the A, B and C subsoil categories as defined in EC8. Average normalized spectra were computed for each soil category and for two levels of earthquake magnitude ($M_L \leq 5.5$ and $M_L > 5.5$) for both horizontal and vertical components of motion. It has been found that average normalized horizontal spectra of records match satisfactorily Type1 and Type 2 EC8 provisions for all the subsoil classes whereas average normalized vertical spectra of records, unlike EC8 provisions, show spectral shapes dependent on earthquake magnitude and ground conditions.

KEYWORDS
EC8, accelerograms, SISMA database, acceleration spectra, horizontal and vertical motion.

1 INTRODUCTION

The Eurocode 8 (EC8) building code (CEN, 2004) recommends the use of two design horizontal acceleration spectra, i.e. the Type 2 spectrum for regions where maximum magnitudes are not expected to exceed 5.5-6.0 (low to moderate seismicity) and the Type 1 spectrum for regions where maximum magnitudes are expected to exceed 5.5-6.0 (high seismicity). For both seismicity levels, different site-dependent spectral shapes are assigned, according to the EC8 classification system, for five subsoil categories (A, B, C, D and E) defined in terms of stratigraphic conditions and representative geotechnical parameters. Conversely, the design vertical acceleration spectral shape is unique, independently on magnitude and ground conditions.
Earlier version of EC8 provisions is based on the work by the SC8 Project Team 1 (1999) whereas spectral shapes adopted by EC8 are mainly based on the background study of Rey et al. (2002). The dataset used by Rey and co-authors is derived from the European Strong Motion Database (Ambraseys et al., 2000) and it includes records from all over Europe (e.g. Greece, Italy) and adjacent countries (e.g., Turkey) with different seismotectonic environments and different fault mechanisms (normal, thrust, strike-slip, etc.). Therefore, it is of practical interest to see whether the EC8 spectra are in agreement with those that are

constructed on the basis of a representative set of records from the Italian region which is mostly characterized by normal faults. Further, in the work by Rey et al. (2002) many Italian records were not used because of the lack of knowledge of geotechnical information at several instrumented sites. Finally, one central objective of the work is to serve as a background study for the national application document required for the introduction of EC8 in Italy.

In this paper a carefully selected and uniformly processed sample of Italian strong motion records is used as a basis for the evaluation of average acceleration spectra. These spectra are computed for the three components of motion (two horizontal and one vertical) and for the three soil classes A, B and C of EC8, for which the available sample of records is sufficiently large. The response spectra derived from the records were then compared to the EC8 spectral shapes in order to investigate spectral shapes as well as ground effects for different levels of seismicity, for both horizontal and vertical components of motion.

2 THE “SISMA” WEB DATA BANK

A database of Italian accelerograms recorded between 1972 and 2002, developed in the framework of a joint project between the University of Rome *La Sapienza* and the University of California at Los Angeles, was used for the analysis (Scasserra et al., 2008a). These data can be freely accessed through the SISMA (Site of Italian Strong Motion Accelerograms) website (http://sisma.dsg.uniroma1.it) whose main features are illustrated in Scasserra et al. (2008b). The principal objective of SISMA is to provide high quality Italian strong motion records whose associated parameters are consistent and reliable and can be used for most engineering applications.

The database is composed of 247 three-component accelerograms from 89 earthquakes and 101 recording sites. The accelerometric data were uniformly processed by the same team of seismologists responsible for the PEER (*Pacific Earthquake Engineering Center*) data processing. Pseudo-acceleration response spectra (5% structural damping) were then computed.

Appropriate source parameters (magnitude, hypocenter location, fault mechanism, etc.) associated with the seismic events were included. The magnitude of the events is always available as local magnitude M_L. Moment magnitudes M_W are available for 60% of the earthquakes from moment tensor solutions. Surface wave magnitudes M_S is also available for 36% of the events. Re-calculated epicentral and hypocentral distances are available for all recordings while Joyner & Boore distances (r_{jb}) and closest distance from the rupture (r_{rup}) have been re-calculated only when fault solutions were available, corresponding to about 45% of the recordings. About 85% of the records have been obtained at distances of less than 50 km from the source while the remaining 15% data are essentially concentrated at distances between 50 and 100 km. Records from normal rupture mechanism dominate with 44 earthquakes belonging to this category; for 24 earthquakes fault rupture mechanism is unknown while remaining events are related to strike-slip, oblique and thrust ruptures.

A major effort was undertaken to improve the characterization of subsoil conditions at the ground motion stations. The site databank includes for every recording site the surface geology, a measurement or estimate of average shear wave velocity in the upper 30 m (V_{s30}), and information on instrument housing. In particular seismic velocities were extracted from the literature for 33 sites. Additional seismic velocities were measured using the seismic analysis of surface waves (SASW) technique for 17 sites that recorded the 1997-1998 Umbria and Marche earthquake sequence (Kayen et al., 2008). The compiled velocity measurements provided data for 51 of the 101 sites. For the remaining sites, the average seismic velocity in the upper 30 m (V_{s30}) was estimated using a hybrid approach, based on correlations with surface geology. Using the geotechnical data available for each station soil classification according to the ground categories prescribed in EC8 was achieved.

3 DATA SELECTION CRITERIA

In order to consistently compare the EC8 response spectra with those derived from Italian records, a representative subset of SISMA database was considered. The following criteria were applied in the selection: (i) focal depth $h \leq 30$ km; (ii) earthquake magnitude $M_L \geq 3.5$; (iii) Joyner & Boore distance ≤ 50 km; (iv) free-field accelerograms.
Local magnitude M_L was considered because available for all records; it is also approximately representative of surface wave magnitude M_S in the dataset magnitude range. Joyner & Boore distance was considered where available (generally for higher-magnitude events), epicentral distance was considered otherwise (generally for small-magnitude earthquakes); this assumption is reasonable considering that small-magnitude events generally correspond to small fault dimensions.
The final record sample used in the present study consists of 200 free-field three-component accelerograms from 77 earthquakes and 83 recording sites. Earthquake distribution versus M_L and focal mechanism is illustrated in Figure 1. The distribution of recordings with respect to local soil conditions, local magnitude and Joyner & Boore distance is summarized in Table 1. In this table the records selected were divided into three range of magnitude (3.5-4.5; 4.5-5.5; 5.5-6.5) and into two groups, near-field and far-field, according to the threshold value r_{jb}=15 km. The records were also classified into the three EC8 subsoil classes A, B and C considering that very few recordings (4) are available from very soft soils sites (class D of EC8) and no data for subsoil class E. In total, 34 records were selected from subsoil class A, 114 from subsoil class B and 48 from subsoil class C. A further subdivision according to the level of seismicity led to: (a) for $M_L \leq 5.5$, 19 records in class A, 90 in class B and 35 in class C; (b) for $M_L > 5.5$, 15 records in class A, 24 in class B and 13 in class C. The distribution of selected data versus M_L, r_{jb} and V_{s30} is illustrated in Figure 2.

Table 1. Distribution of records by local soil conditions, magnitude and distance.

Subsoil class in EC8	**Local magnitude M_L**			**Joyner & Boore distance r_{jb} (km)**		**Total**
	$3.5 \leq M_L \leq 4.5$	$4.5 < M_L \leq 5.5$	$5.5 < M_L \leq 6.5$	≤15	>15	
A	5	14	15	15	19	34
B	47	43	24	67	47	114
C	18	17	13	24	24	48
D	-	2	2	4	-	4
Subtotal	*70*	*76*	*54*	*110*	*90*	*200*
Total	*200*			*200*		*200*

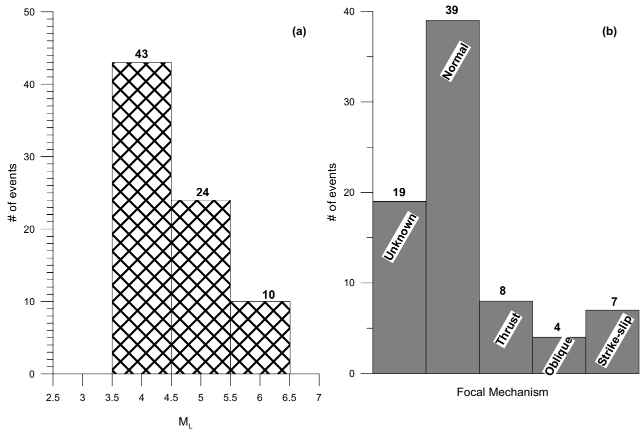

Figure 1. Distribution of events vs. (a) local magnitude and (b) focal mechanism.

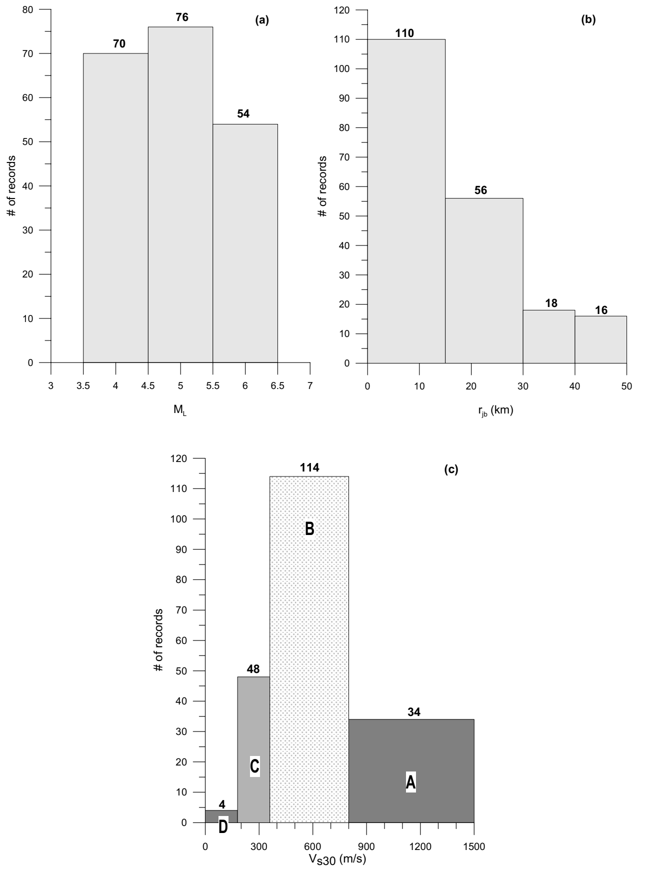

Figure 2. Distribution of records vs. (a) local magnitude, (b) Joyner & Boore distance and (c) V_{s30}.

4 ACCELERATION RESPONSE SPECTRA

4.1 Computed acceleration spectra from recordings

Acceleration response spectra (5% structural damping) were computed separately for each seismic record for the three components of motions. According to EC8 (Rey et al., 2002), the derivation of horizontal spectra was made using the envelope of the two horizontal components for each record intended as the larger of the two spectral ordinates at every period. Each envelope spectrum was normalized to the larger of the two values of the peak ground acceleration PGA. Average values of the corresponding normalized acceleration spectra were then computed for the two levels of earthquake magnitude, $M_L \leq 5.5$ and $M_L > 5.5$, each grouped in the A, B and C subsoil classes, as illustrated in Figure 3. As expected, ground conditions affect the frequency content of the spectra, which become richer of low frequencies moving from rock and stiff soils to soft soils, independently on the magnitude interval considered. The ratio between maximum spectral ordinate and PGA ($S_{a,max}$/PGA) is on average equal to 2.5 with the only exception of stiff soils (class B) for $M_L > 5.5$ which exhibit $S_{a,max}$/PGA=3, as shown in Figure 3b.
A further analysis was carried out in which the derivation of spectra was made treating the two horizontal components of each record as independent. This issue was investigated to ascertain whether the approach followed by EC8 for derivation of spectra using the envelope would have a significant effect on their shape. As an example, Figure 4 shows that differences for subsoil class C using both approaches are not significant. Similar conclusions can be drawn for subsoil categories A and B.
For the vertical component of motion, average values of the normalized acceleration spectra computed for the two levels of seismicity and for each subsoil class are presented in Figure 5. As shown in the plots, different spectral shapes can be recognized for the two intervals of magnitude investigated and for the different subsoil categories (Figures 5a and 5b), similarly to the horizontal case. In fact, the $M_L > 5.5$ spectra are enriched in long periods while for $M_L \leq 5.5$ spectra exhibit a much smaller long period content.

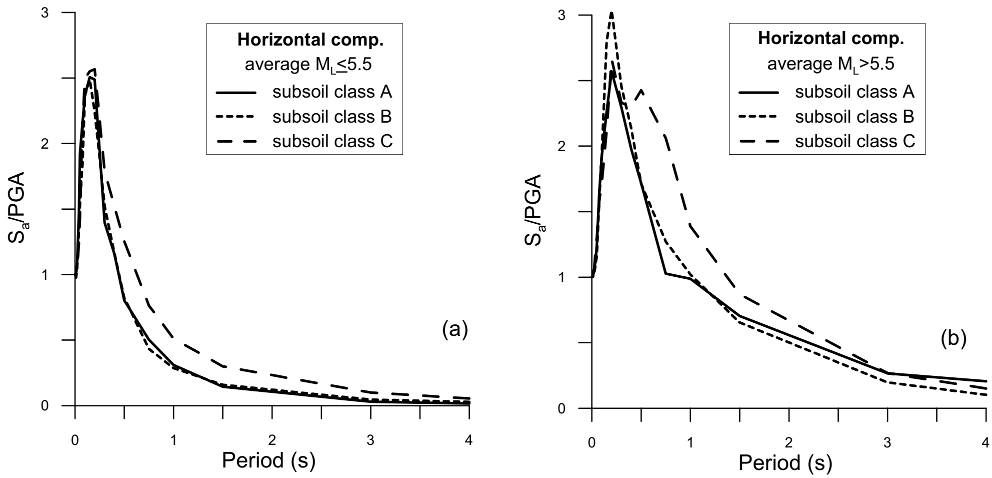

Figure 3. Average normalized horizontal spectra derived from records for A, B and C subsoil classes for (a) $M_L \leq 5.5$ and (b) $M_L > 5.5$.

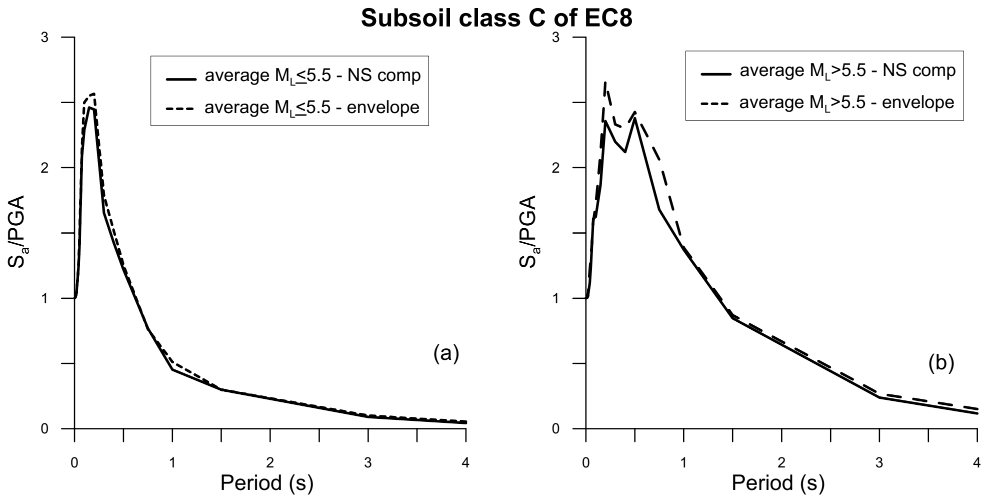

Figure 4. Comparison between average normalized acceleration spectra derived from records for NS horizontal component vs. envelopes of both components: (a) $M_L \leq 5.5$ and (b) $M_L > 5.5$.

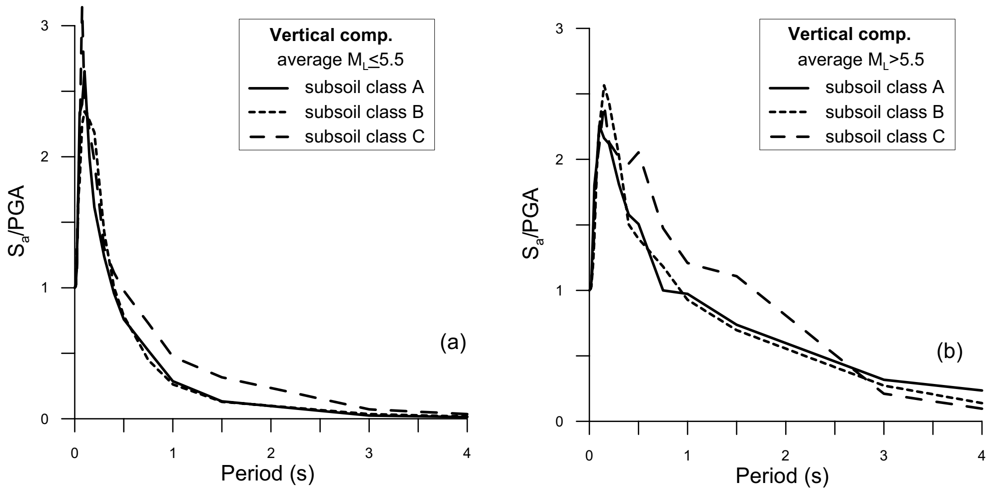

Figure 5. Average normalized vertical spectra derived from records for A, B and C subsoil classes for (a) $M_L \leq 5.5$ and (b) $M_L > 5.5$.

4.2 Comparison with EC8 provisions

The normalized acceleration response spectra derived from recordings were compared with those of EC8 (Type 1 and Type 2) for both horizontal and vertical components of motion.

Horizontal component

In Figure 6 a comparison is given between the acceleration response spectra suggested by EC8 and the average spectra derived in the present study for the two levels of seismicity and for each subsoil class. In the Figures 6a, 6c and 6e Type 2 spectra of EC8 are compared with the average spectra of records from earthquakes with $M_L \leq 5.5$ respectively for subsoil class A (19 records), B (90 records) and C (35 records). In the Figures 6b, 6d and 6f Type 1 spectra

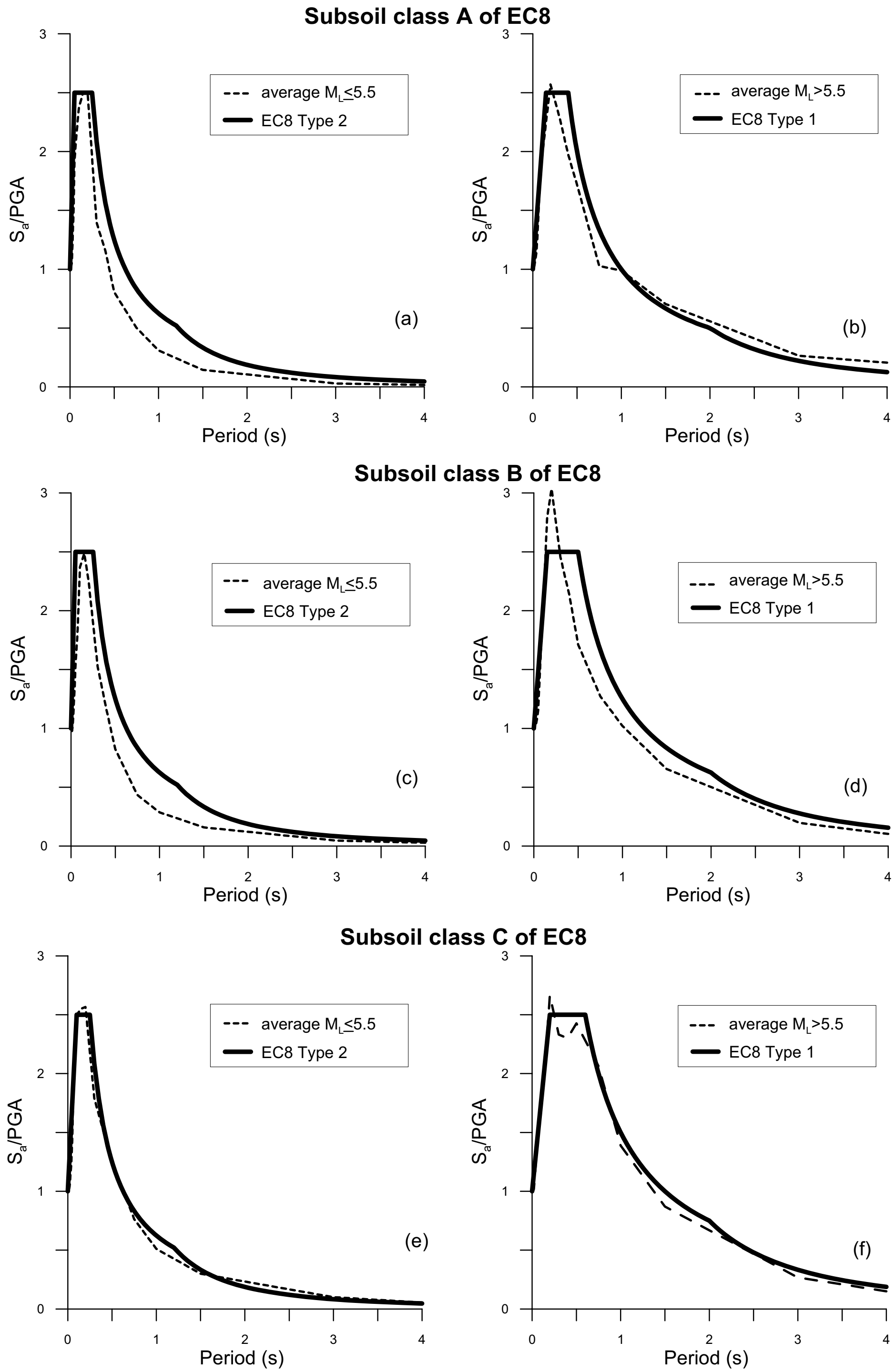

Figure 6. Normalized EC8 spectra and average spectra derived from records.

are compared with spectra of records from earthquakes with M_L >5.5 for subsoil class A (15 records), B (24 records) and C (13 records). It can be observed that for subsoil classes A and B recorded spectra are in good agreement, event though measured data for $M_L \leq 5.5$ show a faster decay of the spectral shape than EC8's. For subsoil class C, the mean values of the computed spectra almost overlap the corresponding EC8 spectra, for Type 1 as well as for Type 2 conditions in the whole range of periods.

Vertical component

In Figure 7 a comparison between the normalized vertical spectrum proposed by EC8 and the average normalized spectra of records for the different ground conditions is carried out. As already said in the Introduction, EC8 ordinates of the vertical response spectrum are independent on the magnitude level and on the subsoil class while the recorded data do not show this feature. EC8 provisions seem adequate, on average, for Type 2 case whereas significantly underestimate average recorded spectra for all the subsoil classes for Type 1 case, almost in the whole range of periods. These latter findings are in agreement with the study by Ambraseys et al. (2005) that provides attenuation relationships for the vertical spectra acceleration. As an example, in Figure 8 spectra by Ambraseys and co-authors, for an average value of r_{jb}=15 km and for appropriate magnitude values, are compared with average spectra from records for subsoil classes A and C. The comparison strongly confirms the trend of measured data for $M_L \leq 5.5$ (Figure 8a); for $M_L > 5.5$ (Figure 8b) spectra from records overestimate spectral values from attenuation relations.

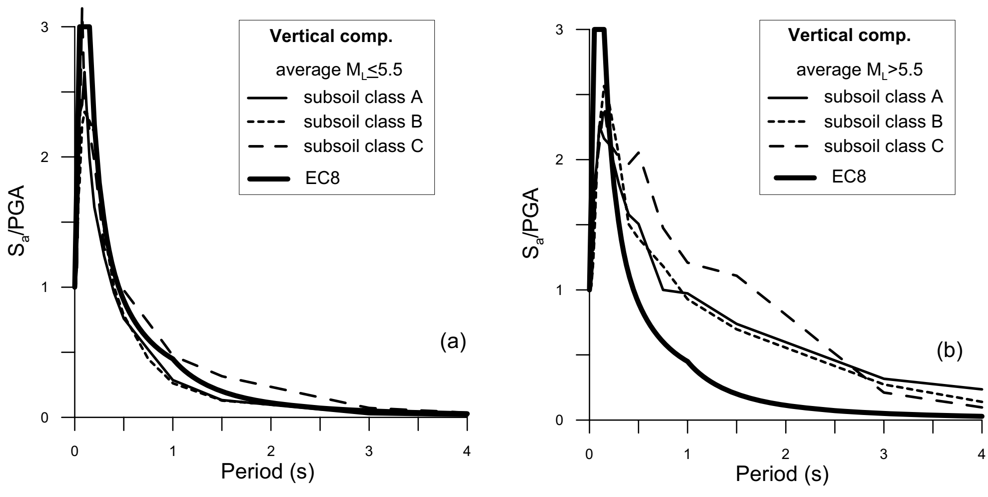

Figure 7. EC8 normalized vertical spectrum and average normalized vertical spectra derived from records for the three subsoil classes A, B and C and for (a) $M_L \leq 5.5$.and (b) $M_L > 5.5$.

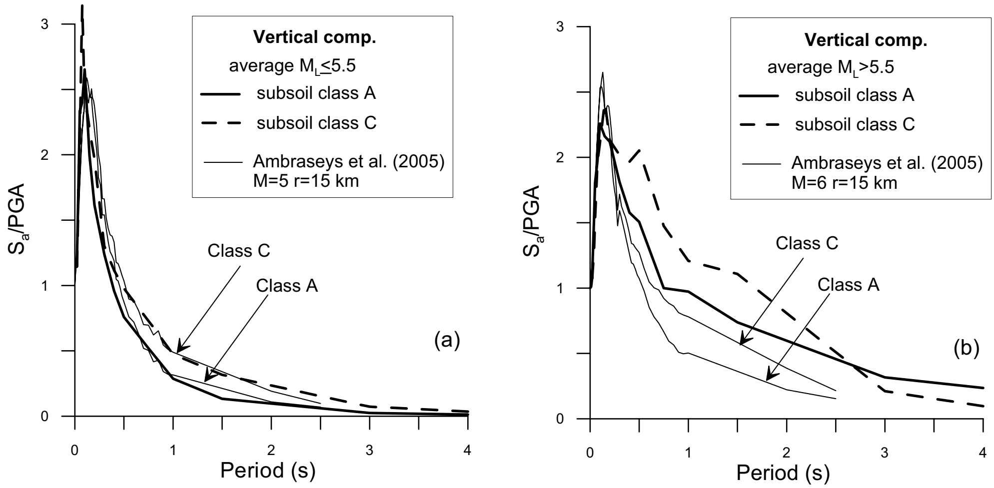

Figure 8. Comparison between spectral attenuation relationships by Ambraseys et al. (2005) and average normalized vertical spectra from records for subsoil classes A and C for (a) $M_L \leq 5.5$.and (b) $M_L > 5.5$.

5 CONCLUSIONS

In this study a comparison between both horizontal and vertical elastic acceleration spectra proposed by EC8 and corresponding acceleration spectra derived from Italian strong-motion records have been undertaken. The source of the records is the recently developed strong-motion database SISMA (Site of Italian Strong Motion Accelerograms). The comparison was carried out in terms of average normalized spectra distinguished into the three ground categories A, B and C, according to EC8, whose number of available data can be considered sufficiently large.

It was found that generally the average normalized spectra for the horizontal component are in good agreement with the spectra adopted by EC8 for Type 1 and Type cases as well as for the A, B and C subsoil classes. As vertical motion is concerned, the analysis has shown that EC8 spectral shape reasonably matches the mean response spectra for low to moderate seismicity context whereas it significantly underestimates the average spectra from recordings for high seismicity context, independently on the ground conditions. Further, it was found a dependency of average vertical spectrum on ground conditions, meaning that in the intermediate-to-long period range spectral ordinates corresponding to soft soils are higher than those pertaining to the stiffer materials A and B.

6 ACKNOWLEDGEMENTS

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CONCRETE BUILDINGS

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CONFIDENCE IN THE CONFIDENCE FACTOR

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ABSTRACT
EC8-3, devoted to assessment/retrofitting of existing buildings, accounts for epistemic uncertainty with an adjustment factor, called "confidence factor (CF)", whose value depends on the knowledge of properties such as geometry, reinforcement layout and detailing, and materials. This solution, plausible from a logical point of view, cannot yet profit from the experience of use in practice, hence it needs to be substantiated by a higher level probabilistic analysis accounting for and propagating epistemic uncertainty (i.e., incomplete knowledge of a structure) throughout the seismic assessment procedure. The paper investigates the soundness of the proposed format and pinpoints some problematic aspects that would require refinement. The approach taken rests on the simulation of the entire assessment procedure and the evaluation the distribution of the assessment results conditional on the acquired knowledge. Based on this distribution a criterion is proposed to calibrate the CF values. The obtained values are then critically examined and compared with code-specified ones.

KEYWORDS
Reinforced concrete, testing and inspection, reinforcement details, material properties, modelling assumptions, analysis method, knowledge level.

1 CURRENT RESEARCH AND NORMATIVE GAP

The obvious fact that the major proportion of seismic risk, in terms of human lives and economic loss, is posed to the society from existing structures is a surprisingly recent acquisition. It took a few disastrous events in California and Japan in the '90s to make the vastness of the problem apparent, and to expose the unpreparedness of the scientific-technical community. Research and code-writing in particular had been occupied with progressing the state-of-the-art in the design of new, well-behaving structures, a task much simpler than the assessment of existing, defective ones.

In this vacuum the first document aligned with the modern anti-seismic philosophy can be considered to be the NEHRP guidelines, prepared in 1997 under the sponsorship of the FEMA (FEMA, 1997), followed in 2000 by the FEMA 356 (FEMA, 2000). In the same years work started on Eurocode 8 Part 3 which was finally approved in 2005 (CEN, 2005). Of course, it could not be asked of these documents to provide a knowledge that did not exist and, given the relatively short period during which they were developed, it could also not be expected that they were validated through a sufficiently long experience of application. As a result, they should still be looked at as experimental and subject to further progress.

This paper focuses on one particular aspect of the assessment procedure put forward in EC8-3: the so-called confidence factor (CF), analogous to the knowledge factor in FEMA 356. Actually, the role of this factor is central in the context of the overall procedure. The first part of the paper presents a conceptual discussion on the nature and the limited reach of this factor, and clarifies the fundamental difference between it and the usual partial factors γ. The discussion shows, at least from a theoretical standpoint, the inadequacy of the present format. Next, taking a more pragmatic stance, the second part of the paper investigates the possibility that, in spite of its theoretical limitations, the CF concept is still capable of providing useful results. In particular, this is done by comparing the code-specified CF values with those obtained through a rational calibration procedure. This latter, attributing to CF the role of constraining the amount of un-conservative assessments within acceptably small limits, is applied to a reduced number of realistic case-studies. The results seem to indicate that the CF format currently specified in the code requires modification.

2 THE RESEARCH CARRIED OUT

2.1 Confidence factor in Eurocode 8 Part 3

EC8-part 3 specifies the values of the confidence factor as a function of the amount of knowledge available at the time of assessment, indicated as *knowledge level* (KL). These values have been defined mainly using expert judgment, without the formal calibration versus higher-level reliability methods that has been employed in the past for the usual partial factors. For the purpose of the following discussion, it is worth recalling how, back in the '70s, the partial factors format for the design of new structures has been calibrated.

In its simplest terms, the reliability problem (determination of failure probability) can be formulated with reference to a single component (e.g. a member in flexure, or in shear). If the uncertainty on the *action effect* and the corresponding *resistance*, denoted by S and R, respectively, is expressed by two independent normal distributions, the failure probability can be shown to be given by:

$$P_f = \Phi\left(\frac{-(\mu_R - \mu_S)}{\sqrt{\sigma_S^2 + \sigma_R^2}}\right) = \Phi(-\beta) \tag{1}$$

where μ, σ, β and Φ are the mean, standard deviation, reliability index and standard normal distribution, respectively.

Equation (1) establishes a relation between β (or, equivalently, P_f) and the so-called central factor of safety γ_0, defined as the ratio of the means μ_R/μ_S. Dividing the argument of $\Phi(\cdot)$ by μ_S one gets:

$$\beta = \frac{\gamma_0 - 1}{\sqrt{\delta_S^2 + \gamma_0^2\delta_R^2}} \rightarrow \gamma_0 = \frac{1 + \beta\sqrt{\delta_R^2 + \delta_S^2 - \beta^2\delta_R^2\delta_S^2}}{1 - \beta^2\delta_R^2} \tag{2}$$

where $\delta = \sigma / \mu$ is the coefficient of variation.

To account not only for the distance between the means, but also for the dispersion of the two random variables S and R, in the traditional partial factor (or LRFD) format use is made of the characteristic values of the variables R_k and S_k, a *lower* fractile (usually 5%) of the resistance and an *upper* fractile (usually 95%) of the action effect. Their ratio, also known as *characteristic safety factor*, can also be uniquely expressed in terms of β:

$$\gamma_k = \frac{R_k}{S_k} = \frac{\mu_R(1-k_R\delta_R)}{\mu_S(1+k_S\delta_S)} = \gamma_0 \frac{1-k_R\delta_R}{1+k_S\delta_S} = \frac{1-k_R\delta_R}{1+k_S\delta_S} \cdot \frac{1+\beta\sqrt{\delta_R^2+\delta_S^2-\beta^2\delta_R^2\delta_S^2}}{1-\beta^2\delta_R^2} \tag{3}$$

For use in practice, the factor γ_k is split into two factors, affecting S and R, respectively. The corresponding (standard) verification expression is:

$$S_d = \gamma_S S_k \leq R_d = \frac{R_k}{\gamma_R} \tag{4}$$

The values of the partial load and resistance factors have been calibrated through a large number of numerical investigations and are now engraved into the codes.
Eurocode 8 Part 3 adopts the verification format in Eq. (4). The confidence factor is prescribed as an additional partial factor on the resistance side. Hence, it might be thought of using Eq.(3) to calibrate CF, as it has been done for γ_S and γ_R. There are essential differences, however, between *design* and *assessment* that prevent this to be done.
The first difference is that in the assessment case safety (β) is not a known target to be attained by suitably proportioning the structure (R), but is an unknown quantity, that varies greatly from case to case, whose evaluation is the purpose of the procedure.
The second important difference lies in the behaviour of existing structures, as contrasted to that of new, properly designed ones, for which a number of design/detailing rules ensure (in a probabilistic sense) that the members will behave as intended, i.e. showing a stable, non-degrading dissipative response.
On the contrary, existing structures, in the majority of cases, exhibit a completely different behaviour, characterized by a progressive deterioration of strength/stiffness until the ultimate state is reached. As a consequence, in principle, it is no more feasible to separate S and R as it is done in Eq. (4), since the response (S) of the structure depends on its continuously varying properties (R).
The above reasoning points to the need for an analysis tool that can adequately capture all the relevant aspects of the degrading response of a defective structure. In assessment no failure mode can be excluded a priori: members can have brittle response in shear, joints can fracture, bars can slip and buckle, etc. Assuming such a tool were available (which is not yet fully the case), it would then be incorporated into an explicitly probabilistic assessment procedure in which all relevant sources of uncertainty could be accounted for.
A fully probabilistic method coupled with a non-linear dynamic analysis incorporating cyclic damage is too advanced to be proposed as the mainstream tool for seismic assessment of existing structures.
Hence, with the limitations of Eq.(4) clearly recognized, pragmatism obliges the adoption of the format, and all the effort must be directed in order to incorporate into the analysis the uncertainties of different nature (epistemic and aleatoric) that characterize the assessment problem. These are: the random variability of the materials (though this may well not be the most relevant one); the uncertainty associated with the approximate modelling of the behaviour (response and capacity) of defective members; the (large) epistemic uncertainty on the geometry, the mass, the detailing of the structure (in extreme but no so rare cases, uncertainty on the very presence of reinforcement layers).

The described sources of uncertainty may lead different analysts to widely different results on the same structure to be assessed. This is illustrated in some detail in the next section.

2.2 Dispersion of the assessment results and role of CF

The assessment procedure starts with the information base initially available about the existing building. From this point on, at each step of the procedure, analysts are faced with a number of options that, as it will be discussed below (with reference to Figure 1), cannot but lead to different outcomes.

The first choice to be made is on the amount and type of additional information to be collected to complement the initial set.

Several degrees of freedom, or of arbitrariness, characterize this step. First of all, the analysts may choose to attain different knowledge levels, for which different minimum amounts of tests are required by the code (Figure 1a). For the same target KL, the same percentage of tests per floor may be obtained with different test types and locations (Figure 1b). Each test type involves a different measurement error and, for indirect tests, a different dispersion in the associated correlation equation. Further, once the results have been collected, these have to be integrated with the initial data set (Figure 1c): what to do then if the additional information contradicts the design documents? One analyst might accept the discrepancy, within certain limits, while another may choose to rely entirely on in-situ information adopting a full survey, together with extended test/inspection plans, i.e. moving up in the knowledge scale to KL2.

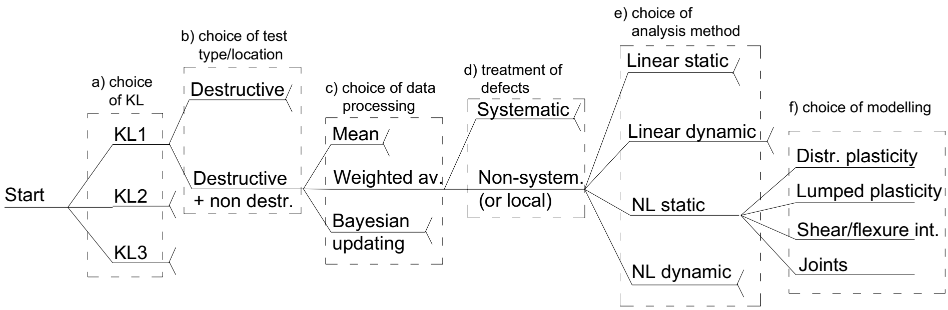

Figure 1. Degrees of freedom left to the analyst.

Another issue is related to the two higher levels, KL2 and KL3, for which the two options: "initial information plus verification" and "complete reliance on in-situ information" are given as equivalent alternatives. It is quite likely that they are not exactly equivalent and this represents one further source of difference in the final assessment results.

An important further aspect to be considered in planning tests and post-processing results is whether the collected data can be regarded as a sample from a single population, i.e. in other words, whether the structure is homogeneous or, as it often happens due to the construction history of the building, more than one homogeneous portion can be identified. If the situation is the latter, it would appear that the minimum test number required by the code should be referred to each of the homogeneous portions. This situation should also be reflected in the modelling. The code indicates that the material properties entering into the analysis model should be the mean ones. If the structure is made up of distinct portions, the mean value of properties within each portion should be used, but it can be argued that this practice is seldom adopted.

For all of the considerations above, already at the end of this first step (data collection and processing) every analyst will have a different picture of the same structure.
The next branching point has to do with the so-called defective details, such as, for example, insufficient anchorage length of rebars, 90° hooks and inadequate diameter/spacing in stirrups, absence of joint reinforcement or wrong detailing of the anchorage of longitudinal bars into the joint, etc. This is a multi-faceted problem.
Once a type of defect is discovered, the question arises whether its presence should be considered systematic over the structure or a portion of it only, or as an isolated local feature (Figure 1d). An informed answer to this question would require extensive and intrusive investigations that are seldom compatible with the continued use of a building.
The next choice of the analysts is the method of analysis to be employed (Figure 1e), which is intimately related to that of the modelling (Figure 1f). Obviously, if the selected analysis is linear, the cyclic degradation due to defects cannot be included at this stage. But even if it is nonlinear static, such behaviour cannot be easily included. Exclusion of these defects from modelling may lead to a response (S) quite different from the real one, which would then be compared to a resistance (R) affected by large uncertainty (model uncertainty on the capacity formulas for defective members). On the other hand, nonlinear dynamic analysis including behavioural models for defective members would trade the model uncertainty on capacity with that on hysteretic degrading response.
The different sources of uncertainty and multiple choices facing the analysts during the assessment, all contribute to a relatively large dispersion in the estimated state of the structure.
The above discussion makes it overwhelmingly clear that the nature of the confidence factor is different from that of the partial factors recalled in Section 1.
The interpretation that is proposed herein for the CF is that of a factor which aims at ensuring that, out of a large number of assessments carried out in accordance with EC8-3, only a predefined, acceptably small fraction of them leads to an unsafe result, i.e. to overestimating the actual safety.
Admittedly, the idea that a single factor, with values depending only on the knowledge level, and not on all the aspects recalled above, may achieve the stated objective may appear as unrealistic. The paper represents an attempt to investigate to what extent this idea maintains some value. Further it provides a limited exploration on the magnitude of the CF values needed to reach the stated goal.

2.3 Proposed procedure for the evaluation of CF values

The proposed procedure consists of a *simulation* of the *entire EC8-3 assessment process* with the purpose of quantifying the dispersion in the assessment results due to the many choices/uncertainties described in the previous section.
The starting point of the procedure is to imagine an existing building, with all its properties, including the defects and spatial fluctuation of materials, geometry, etc. completely known. This ideal state of perfect knowledge can never be obtained in practice and it represents a state of knowledge higher (the highest possible) than the state of so-called *complete knowledge* described in the code (KL3).
In each simulation run *choices* (knowledge level, type and position of tests, how to process the results, analysis method, modelling options, etc) are made randomly to reflect the arbitrary choices made by different analysts. This obviously requires the spelling-out of all the steps described in the previous section, discretizing the possible choices in a finite number of options and filling the gaps of the code with practices coming from common-sense and experience in real-case assessments. It is imagined that the generic analyst will follow his trail

down the procedure arriving at a different evaluation of the safety of the structure. This simulation is carried out without employing the confidence factor (i.e. CF=1). By repeating the process for a sufficiently large number, say *n*, of analysts a statistical sample (of size *n*) of the structural safety is obtained and can be used to estimate its distribution.
At this stage the statistical sample of structural states, quantified by the global state variable *Y* (a *critical* demand to capacity ratio, see Jalayer *et al* 2007) is compared with the *true* state of the structure, i.e. the state of the structure evaluated based on the ideal perfect state of knowledge. It is expected that a portion of the assessments will result in a *conservative* estimate (i.e. in a state worst than the real one) while the remaining will be on the *un-conservative* side.
The goal of the last part of the procedure is that of reducing the fraction of un-conservative estimates to an acceptably small value. This is done by re-evaluating the structural state, using the same sets of choices of the previous evaluation, with a value of CF larger than one (i.e. decreasing capacities). If the procedure works as intended the new sample of structural states will have the predefined target fraction of un-conservative estimates.
The procedure can be split into the following steps:

- Step 1: Generation of the existing and perfectly known structure (termed *reference* structure)
  Once all the material properties and possible defects have been assigned a probability distribution, a structure can be generated by sampling a set of parameter values from the above distributions. This structure is by definition completely known and is termed the *reference* structure.

- Step 2: Generation of a sample of imperfectly-known structures from the reference structure
  A number N_{VA} of virtual analysts is given the task of assessing the structure. This step consists of simulating the process of inspection/information-collection, and produces N_{VA} different states of (imperfect) knowledge from the reference structure. These states are the starting point for the assessment by the virtual analysts. In order to reflect the different test plans designed by different analysts, this step requires the randomization of the test locations and test types.

- Step 3: Assessment of the reference structure
  The reference structure is assessed according to the code and the seismic intensity that induces the attainment of the limit-state (LS) under consideration is recorded. The attainment of the limit state is marked by a unit value of the global variable Y=1. This result is considered the *true state* of the structure.

- Step 4: Assessment of the imperfectly known structures
  The virtual analysts apply the code-based assessment procedure with a unit value of the CF and the same intensity as determined in Step 3. This produces a sample of N_{VA} values of the global state variable *Y*. This step requires a further randomization, reflecting the freedom left to the code-user in choices such as inclusion/exclusion of defects from modelling and the selection of the analysis method (linear vs. nonlinear, static vs. dynamic).

- Step 5: Statistical processing of the sample states and determination of CF
  Statistical processing of the sample of values of *Y* produces a distribution that exhibits a certain amount of variability around the value Y=1. This is shown in Figure 2a. The value of the CF can now be determined by enforcing the condition that a chosen lower fractile of *Y* (say, 10%) is equal to 1, i.e. the *true* state of the structure (as shown in Figure 2b).

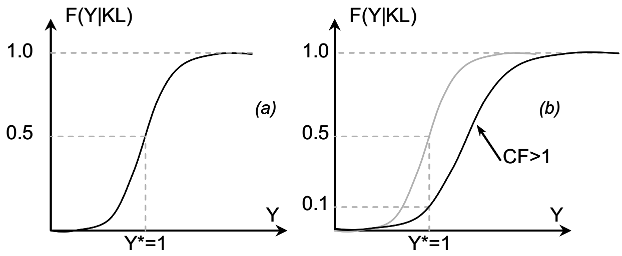

Figure 2. Distribution of the assessment results made by the N_{VA} analysts: a) with CF = 1, b) with CF>1.

2.4 *Application to three RC plane frame structures*

2.4.1 Geometry and modelling of the structures

The calibration procedure has been applied to three RC frame structures selected in order of increasing "complexity" (Figure 3). They have been selected to investigate whether CF values, which according to the proposal are a function of the distribution (spread) of structural response, are sufficiently stable with increasing structural size.

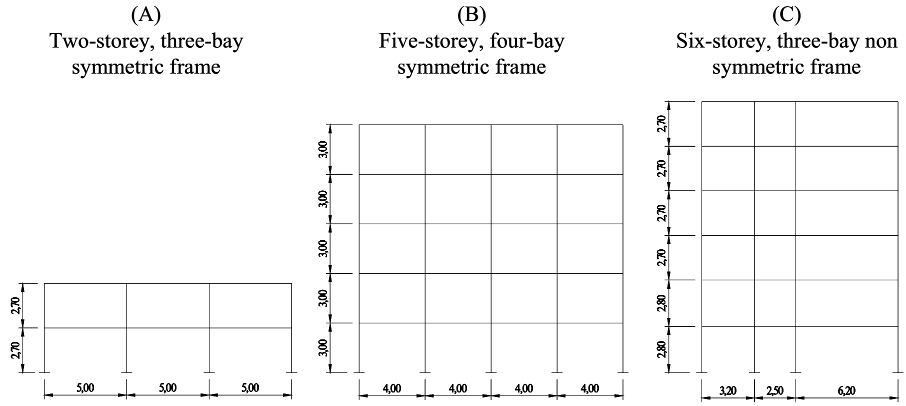

Figure 3. The analyzed structures.

For the purpose of data collection (material tests, reinforcement details etc.) and post-processing the structures are considered homogenous, in the sense that the spatial distribution of the properties/defects belongs to a single population.

The assessment has been carried out with the nonlinear static and dynamic methods. CF values have been evaluated both separately with each of the two methods, and jointly, to investigate dependence on the analysis method. For the purpose of dynamic analysis the seismic action is represented by seven recorded ground motions selected to fit on average, with minimum scaling, the EC8-specified spectral shape scaled to a PGA of 0.35g for soil class A (Iervolino *et al*, 2008). In the case of static analysis, the average spectrum of the recorded ground motions is considered as the demand spectrum.

In terms of modelling the nonlinear degrading response of the structure, account has been taken of flexure-shear interaction and joint hysteretic response. The models are set up in OpenSEES, employing flexibility-based elements for the members with section aggregator to

couple a fibre section (flexural response) with a degrading hysteretic shear force-deformation law. Joints have been modelled with a "scissor-model" with a degrading hysteretic shear force-deformation law. Tangent-stiffness proportional damping has been used, calibrated to yield a 5% equivalent viscous damping ratio on the first elastic mode.
Since the effect of brittle failure modes, such as shear in members and joints, has been included in the modelling (for both static and dynamic analysis), the structural performance is checked in terms of deformation quantities only. Hence, $Y = \theta_{max} / \theta_C$, where θ_{max} is the demand peak inter-storey drift ratio, and θ_C the corresponding capacity. Detailed information on the characteristics (geometry, reinforcement, etc) of the structures and the adopted response models can be found in (Rajeev, 2008).

2.4.2 Modelling of uncertainty
For each case (structures A, B or C), for the purpose of generating the *reference structure*, material properties (concrete strength f_c, and steel yield stress f_y and hardening ratio b) have been modelled as Lognormal r.v.'s, and structural defects (transverse reinforcement spacing in columns and beams, s_c and s_b, and column longitudinal reinforcement ratio ρ) as Uniform r.v.'s, with different parameters for each case.
The above random variables are collectively denoted as Group 1. These r.v.'s are sampled only once, during Step 1, for the purpose of generating the reference structure. The parameters relative to Case C (chosen for detailed presentation of the results) are reported in Table 1.

Table 1. Distribution type and parameters for Group 1 random variables.

Random variable	Distribution	Mean(or Min)	CoV (or Max)
Column stirrup spacing s_c	Uniform	200 mm	330 mm
Beam stirrup spacing s_b	Uniform	150 mm	250 mm
Long. Reinforcement ratio ρ	Uniform	0.008	0.014
Concrete strength f_c	LN	20 MPa	0.10
Steel yield stress f_y	LN	275 MPa	0.05
Hardening ratio $b=E_h/E_s$	LN	0.04	0.25

A value of concrete strength f_c has been sampled at each integration point along a member, while a single pair of values of steel properties (f_c, b) has been sampled for all (points within) members of each floor. Correlation has been introduced amongst the concrete strength values according to an exponential decay model. The decay of the correlation coefficient is faster in the vertical direction than in the horizontal one, to account for the floor-wise casting of concrete. The decay parameters have been arbitrarily calibrated so as to have a correlation coefficient of 0.7 at an horizontal distance of 5.0 m and 0.4 at a vertical distance of 3.0 m. The Nataf joint distribution has been adopted for simulation of the concrete strength field values (Liu and Der Kiureghian, 1986).
For the purpose of generating the *imperfectly known structures* (Step 2 of the procedure), the data collection procedure, consisting of tests on material samples from the structure and verification of reinforcement details, is randomized.
The *number* of test/inspection locations is determined based on the minimum requirements in the code. These latter are specified as a function of the target KL. Test/inspection levels for KL1, KL2 and KL3 are denominated as *limited*, *extended* or *comprehensive*, respectively, when initial information is poor (relative to each KL requirements). This is the assumption

made herein. The actual number of tests together with the code minima for structure c are reported in Table 2. The number of tests results always slightly larger than the minimum, with only one (negligible) exception.
The actual test location chosen by each analyst is determined by randomly sampling (uniform integer distribution) first the member and then the location within the member (for this purpose each integration point is regarded as a possible test location). At each location, the testing/inspection consists of reading the value of the sought property from the reference structure (value generated during Step 1). Measurement errors are not considered. Since the reference structure is homogeneous by assumption, all the data gathered are *averaged* to obtain the values to be employed in the assessment.

Table 2. Number of test/inspections (minima for construction details are meant per element type: columns, beams).

KL		Testing (material)		Inspection (details)	
		Minimum	Actual	Minimum	Actual
1	Limited	1 per floor	1 per floor (6)	20%	1 × floor × type, 6/24 col.'s = 25%, 6/18 beams = 33%
2	Extended	2 per floor	2 per floor (12)	50%	2 × floor × type, 12/24 col.'s = 50%, 12/18 beams = 66%
3	Comprehensive	3 per floor	3 per floor (18)	80%	3 × floor × type, 18/24 col.'s = 75%, 18/18 beams = 100%

The assumed scarceness of initial information, and in particular the lack of a complete set of construction drawings, influences the knowledge of the geometry of the structure. In particular, this may refer to the presence/absence of elements (a typical case being represented by beams in flat-slab structures) or the actual cross-section dimensions (significant variations in plaster thickness or the presence of cavities for ducts are common and cannot practically be ascertained for all members), or, finally, the precise unit-area weight of the floor system.
To model this kind of "geometrical" uncertainties (denoted as "residual" in the following) two types of additional random variables are introduced: the unit-area weight of floors (one variable per floor typology, e.g. typical floor and roof) and the cross section height of elements (one variable per element type: beams and columns). These random variables are collectively denoted as Group 2 and are sampled for each imperfectly known structure during Step 2. Table 3 shows the corresponding distribution types and parameters for case C.
To illustrate Steps 1 and 2, Figure 4 shows for Structure C the histograms of the relative frequency of a number of quantities. The leftmost column reports the histograms of the concrete strength f_c and of the columns' reinforcement ratio ρ for the reference structure, chosen to represent material properties and construction details, respectively. The plots show also the original distribution from which the values have been sampled (see Table 1). The sample size is 162 (one value per integration point, 24 columns with 3 points each, 18 beams with 5 points each) and 24 (one value per column), respectively.
The following three columns report for the three KLs the histograms of the values $\bar{f}_c$ and $\bar{\rho}$ obtained averaging the test results by each analyst (hence the sample size for both quantities equals the number N_{VA} = 200 of virtual analysts). The average values $\bar{f}_c$ and $\bar{\rho}$ are obtained from samples of size increasing with KL: for instance, 6, 12 or 18 f_c values for KL1, 2 and 3. Clearly a larger sample leads to an estimate of the mean closer to the true mean. This latter, of course, is the mean of the values sampled for the reference structure, not the mean of the

sampling distribution. It can be anticipated, on the basis of the histograms, that following this averaging rule, the structure analyzed by each analysts does not get closer with increasing KL to the actual (reference) structure, but rather to a *mean* structure. The consequences of this fact are discussed later on.

Table 3. Distribution type and parameters for Group 2 random variables: for h_b and h_c values reported are the variation with respect to the mean (specified in the drawings).

Random variable	Distribution	Mean(or Min)	CoV (or Max)
Left span permanent load G_k	Uniform	6.5 kN/m	8.5 kN/m
Middle span permanent load G_k	Uniform	6.5 kN/m	8.5 kN/m
Right span permanent load G_k	Uniform	27.5 kN/m	29.5 kN/m
Beam cross-section height h_b	Discrete Uniform	KL1 (-50;0;+50) mm KL2 (-25;0;+25) mm	
Column cross-section height h_c	Discrete Uniform	KL1 (-50;0;+50) mm KL2 (-25;0;+25) mm	

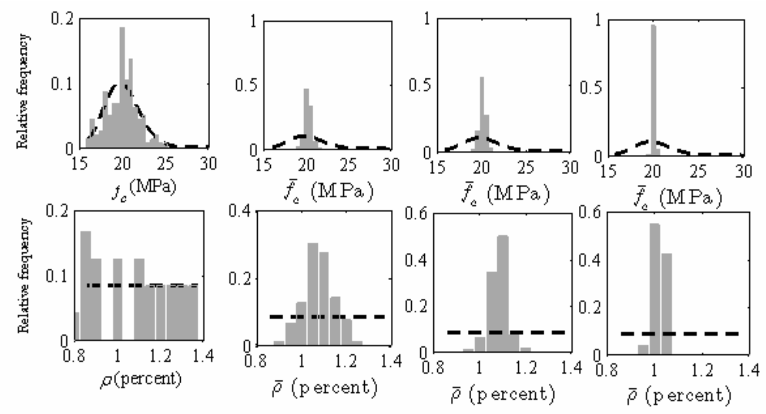

Figure 4. Histograms of concrete strength and reinforcement ratio.

2.4.3 Results

Step 3 of the procedure consists of the assessment of the reference structure by the most accurate method (herein inelastic time-history). This has been done with IDA (Vamvatsikos and Cornell, 2002) subjecting the structure to the seven natural ground motions selected to match EC8 spectrum. Consistently with the code indication of using 7 records and taking the average of the maxima, the intensity (PGA of 0.216g) where the mean IDA curve crosses Y=1 is recorded and used in Step 4. The capacity has been set for this structure to the deterministic value of θ_c=2.5%.

Step 4 of the procedure consists of the assessment by each virtual analyst of its imperfectly-known structure (the result of Step 2). As already mentioned, the number of analysts has been set to N_{VA} = 200, and each of them can choose between nonlinear static and dynamic analysis for the assessment. Actually, in this application each analyst has performed both analyses (dynamic and static). The results are first presented separately by method (200 samples each) and then *mixed* (400 samples). This, as anticipated, serves the purpose of investigating the dependence of CF on the analysis method.

For structure C, results (in terms of Y values, i.e. the state of the structure) from static and dynamic analyses are close. In the following only “static” results are shown in detail. Figure 5 shows the N_{VA} = 200 curves obtained at each KL. The decreasing dispersion with increasing KL is apparent.

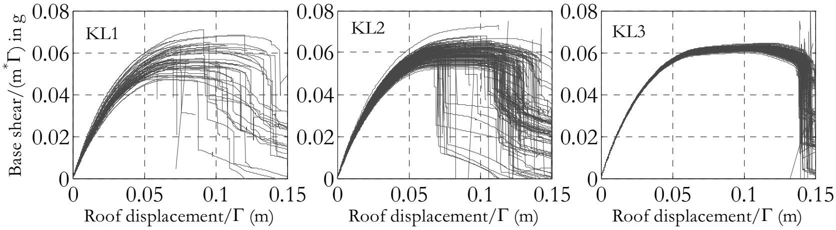

Figure 5. Structure C, pushover curves for the sample structures at each KL.

The empirical distributions (conditional on KL) of the 200 Y-values obtained from the curves in Figure 5 is shown in Figure 6a. In the figure the value Y=1 is marked by a dashed vertical line. For the employed seismic intensity of PGA = 0.216g this is the state of the reference structure. A second vertical (solid) line marks the value Y= 0.79. This is the state of the *mean* structure, i.e. a structure identical in geometry to the reference one, but with spatially homogenous properties (f_c, f_y, b, s_c, s_b, ρ) equal to the average values of the samples generated in Step 1. As it can be seen, with increasing KL the distributions get steeper (lower dispersion) and closer to the *mean* rather then the *reference* structure. In all cases a large proportion of the analysts overestimates the safety of the structure (i.e. they find Y<1): roughly 40% with KL1, 70% with KL2 and 100% with KL3.

Next, the analysis is repeated with CF-values larger than one in order to reduce the above percentages to the same acceptably low value. For the purpose of this application this value has been set to 10%. Sensitivity of the results to this choice can be found in (Rajeev, 2008). Figure 6a shows the corresponding distribution for KL1 only, for clarity (CF=1.34).

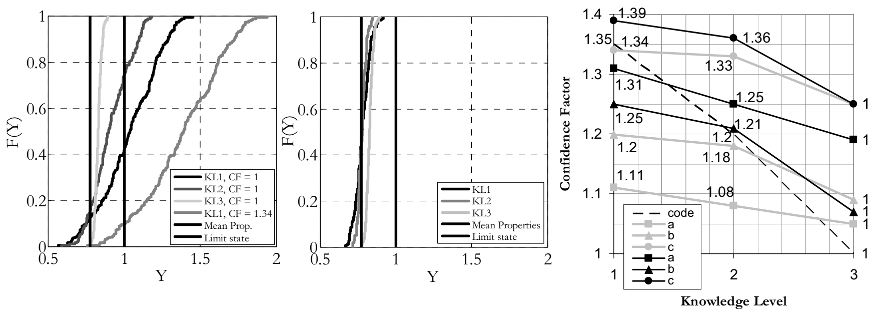

Figure 6. a) Structure C, distribution of N_{VA} Y-values obtained by static analysis, b) distribution by static analysis neglecting residual geometric uncertainty, c) CF-values for all structures and analysis types.

The relevance of the residual geometric uncertainty can be appreciated by comparing the curves in Figure 6a with those in Figure 6b, obtained disregarding this contribution (the difference between the structures analyzed by the virtual analysts is only due to material properties and construction defects).

The CF values obtained for the three considered structures are summarized in Figure 6c. The figure reports separately the values obtained by static (grey) and dynamic (black) analysis, together with the code-specified values. It can be observed how the dependence of CF on KL is in all cases milder than that specified in the code, and that CF depends on the analysis method and structural type/size.

3 DISCUSSION, CONCLUSIONS AND TENTATIVE CODE DEVELOPMENTS

The CF values given in EC8-3 depend only on the knowledge level acquired complementing the initial information by tests and inspections. After discussing the conceptual limits of the approach it has been checked whether, in spite of them, CF can still lead to useful results in practice, and whether the code-specified CF values are reasonable.

The approach taken considers a number of RC frames meant to be realistic examples, i.e. characterized by spatial variability of material properties and of construction details/defect typical of older type RC structures, and subjects them to seismic assessment by means of the most accurate available method (identified within this context as NLTH analysis with an hysteretic degrading model). This assessment is assumed to provide the *true* state of the structure. Each structure is then given to a large number of virtual analysts, that carry out their assessment according to the code prescriptions.

As discussed in Section 3, at each step of the assessment, analysts are faced with a number of options that inevitably lead to different outcomes. The choices are:

- amount and type of additional information to be collected (ie. choice of target KL and, within each KL, of the tests/inspections plan). At the end of this step every analyst will already have a different picture of the same structure;
- consideration to be given to construction defects. Once a defect is detected, it can be considered as systematic over the entire structure, a portion, or as a local feature. The corresponding defective behaviour could be included in modelling or treated through a modified capacity formula, a choice that depends also on the adopted analysis method. This choice is one of the most consequential on the end results.
- the method of analysis and modelling options to be employed.

All the aspect listed above contribute to a relatively large dispersion in the estimated state of the structure (Figure 1). The quantification of this dispersion, as obtained by a statistically significant number of virtual analysts applying the code, is already in itself an important intermediate result.

Based on the obtained distribution of assessment results, this paper proposes to employ CF to ensure that, out of a large number of assessments carried out in accordance with EC8-3, only a predefined, acceptably small fraction leads to an unsafe result, i.e. to overestimating the actual safety.

The procedure has been applied to three RC frame structures of increasing size, employing the nonlinear static and dynamic analysis methods, and considering all three KLs. The results, in terms of CF values, have been used to assess the dependence of CF on: KL, analysis method and structural size. The most relevant findings are:

- In the code CF values are specified as a function of KL only, implying that KL is the single most important factor influencing CF. Results appear not to clearly support this expectation of the code. The dependence is found to be generally mild. This can be clearly seen from the results in Figure 9.

- The code does not differentiate CF values with respect to the analysis method, implying that epistemic uncertainty has the same effect with all analysis methods. Results appear again not to clearly support this assumption (see Figure 9). When considered separately (i.e. assuming that all analysts will chose the same analysis method) nonlinear static results show a much reduced dispersion than those obtained by nonlinear dynamic analysis. This, according to the proposed procedure, leads in general to smaller values of CF to be employed with static than with dynamic analysis.
- The code does not differentiate the CF values with respect to the structural typology, size or other characteristics. The investigation carried out cannot provide decisive evidence on this aspect due to the number of structures analyzed. However, as it can be seen from Figure 9, the results show a dependence of the CF value on the structure, of the same order as that specified in the code to differentiate between KL1 and KL2.
- The code specifies that the geometry of the structure must be completely known before setting up a model for the analysis. Experience with real-case assessments shows that it is usually not possible to obtain accurate measurements over the entire structure and that even when member centrelines are known, a residual uncertainty on the cross-section dimensions is unavoidable. This source of uncertainty has been modelled in the applications. Results show it to be, for the examined cases, at least of the same order of importance of that associated with material properties and defects.
- The observed importance of the residual geometric uncertainty, indicates the need for some sort of sensitivity study on the corresponding modelling assumptions. It is recalled that the geometric uncertainty has been introduced as a random variation around the mean cross-section height of value ±5cm for KL1, ±2.5cm for KL2 (for KL3 it has been assumed no uncertainty). This is admittedly arbitrary, both in the values themselves and in their graduation with KL. One might well argue that an imprecise evaluation of the cross-section dimensions of ±5cm is still compatible with such a complete state of knowledge as that denoted as KL3. Results under this latter assumption (i.e. no differentiation of geometric uncertainty with KL) have been obtained and reported in (Rajeev, 2008). As expected, the undifferentiated treatment of geometric uncertainty with KL, levels down the differences between KLs. This type of uncertainty (residual on geometry after the data collection process) is not mentioned in the code.
- Finally, there is an interesting aspect not immediately appreciable when following the code prescriptions. The indication of using mean values within the analysis model implies that with increasing KL the average (or median) result of the assessments will not converge to the *true* state of the structure but, rather, to the state of a *mean* structure, i.e. a structure identical in geometry to the reference structure but with properties that are spatially homogeneous and equal to the average of those of the reference structure. This fact is quite relevant because while strong spots are lowered towards the mean, the weak spots (which are the critical ones) are raised towards the same mean. Thus, the averaging operation produces in general a structure stronger than the reference one. Hence, on average, the assessment results will converge to a state that is illusorily safer than the actual one.

In conclusion, within the limits of the analyses carried out, it appears that the present state of development of EC8-3 should be improved since:

- CF values are not differentiated with respect to analysis method/modelling options;

- CF values are not differentiated with respect to structural type (size, regularity, construction material, load-resisting system, etc);
- the so-called *complete* knowledge (KL3) does not actually correspond to a state of *perfect* knowledge, hence, it should be penalized with a CF value larger than one;
- there is an important "hidden assumption" in the prescription of using mean values of material properties within the model. This is generally un-conservative, leading on average to underestimating the demand-to-capacity ratio of the structure.

Finally, in more general terms, an aspect that has been highlighted by the study is the large dispersion in assessment results presently allowed by the code. Independently of the CF format, though it is obvious that the code cannot and should not cover prescriptively all the details of the assessment process, many consequential degrees of freedom presently remain in the hands of the analysts, hence stricter guidance would be needed to achieve the goal of more uniform/reliable assessments.

4 ACKNOWLEDGEMENTS

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CONFIDENCE FACTORS AND STRUCTURAL RELIABILITY

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ABSTRACT

The recent European codes such as Euro Code 8 seem to synthesize the effect of structural modeling uncertainties in the so-called confidence factors (CF) that are applied to mean material property values. However, the effect of the application of the confidence factors on structural reliability is not explicitly stated. An alternative approach featured in the SAC-FEMA guidelines, considers the effect of both ground motion uncertainty and the structural modeling uncertainties on the global performance of the structure, in a closed-form analytical safety-checking format. This work strives to have a critical look at the confidence factors from the point of view of the characterization of uncertainties and structural reliability assessment. Moreover, an efficient Bayesian method is presented that can estimate both the *robust* structural reliability and also the joint probability distributions for structural fragility parameters, based on a small sample of structural model realizations and ground motion records. Based on findings featured in this work, a set of perspectives for the future European codes are outlined.

1 INTRODUCTION

Many European countries are subject to a considerable risk of seismic activity. Quite a few of these countries enjoy a rich patrimony of existing buildings, which for the most part were built before the specific seismic design provisions made their way into the constructions codes. Therefore, the existing buildings can potentially pose serious fatality and economic risks in the event of a strong earthquake. One main feature distinguishing the assessment of existing buildings from that of the new construction is the large amount of uncertainty present in determining the structural modeling parameters. The recent European codes seem to provide a level of conservatism in the assessment of existing buildings, in the application of the (inverse of the larger than unity) *confidence factors* (*CF*) to mean material property values. These confidence factors are determined as a function of the *knowledge levels* (*KL*). The knowledge levels are determined based on the amount of tests and inspections performed on the existing building. Table 1 illustrates the three KL's, namely, limited, extended and comprehensive, based on the amount of in-situ tests and inspections performed.

Table 1. Recommended minimum requirements for different levels of inspection and testing.

	Inspection (of details)	Testing (of materials)
	For each type of primary element (beam, column, wall):	
Level of inspection and testing	Percentage of elements that are checked for details	Material samples per floor
Limited	20	1
Extended	50	2
Comprehensive	80	3

At a first glance, the application of the confidence factor seems to be a deterministic method for addressing an inherently probabilistic problem. With the emerging of probability-based concepts such as life-cycle cost analysis and performance-based design, the question arises as to what the CF would signify and would guarantee in terms of the structural seismic reliability [Jalayer et al. 2008, Franchin et al. 2008]. This would not be possible without a thorough characterization of the uncertainties in the structural modeling parameters [Monti and Alessandri 2008 and Jalayer et al. 2008]. Another issue regards the definition of the KL. The current code definition in Table 1 leaves a lot of room for interpretation; it is independent of the spatial configuration and the outcome of the test results. Moreover, the logical connection between the numerical values for the confidence factors and the onset of the knowledge levels is not clear.

An alternative probabilistic and performance-based approach is adopted in the American Department of Energy Guidelines DOE-1020 and in SAC-FEMA guidelines. This simplified approach leads to an analytic and closed-form solution which compares the factored demand against factored capacity. The factored demand and capacity are respectively equal to median demand and capacity multiplied by some factors. The magnifying demand factors and the de-magnifying capacity factors take into account all sources of uncertainty, such as record-to-record variability, structural modeling uncertainty and the uncertainty in the capacities. This approach that is recently known as the *Demand Capacity Factor Design* (DCFD) [Cornell et al., 2002] takes into account the overall effect of the various types of uncertainties on a global structural performance parameter. Therefore, in the case of existing buildings, there is a need for a method that can evaluate the global parameters reflecting the overall effect of structural modeling uncertainties.

The Bayesian framework for probabilistic inference seems to be a perfect basis for taking into account the results of tests and inspection in updating the structural model. The authors in a previous work [Jalayer et al. 2008] have demonstrated how the advanced simulation methods based on Bayesian updating can be used to both update the structural reliability and also the probability distribution for the modeling parameters, in the presence of test and inspection results. However, the application of the advanced simulation schemes requires a large number of structural analyses and there seems to be a need for less computationally intensive methods for updating the structural model and structural reliability. The authors [Jalayer et al., 2009] have employed an efficient Bayesian simulation-based method for robust estimation of structural reliability. This method exploits a relatively small number of structural analyses in order to yield the *robust reliability* for the structure in question. The term robust herein refers to the fact that the reliability is calculated taking into account all possible structural models and their relative plausibilities.

2 THE STRUCTURAL PERFORMANCE PARAMETER

The structural performance parameter in the context of this work is a particular kind of demand to capacity ratio. This parameter which denoted as Y, assumes the value of unity on the verge of the limit state LS. In the case of static analyses, the capacity spectrum method [Fajfar, 1990] is used to obtain the global demand to capacity ratio. In the case of dynamic analyses, the *cut-set* concept in reliability theory is employed to find the critical component demand to capacity ratio that takes the structure closer to the onset of the limit state LS. This critical demand to capacity ratio corresponds to the strongest component of the weakest structural mechanism [Jalayer et al., 2007].

3 THE SAC-FEMA-TYPE FORMULATION

In the case of static analyses, the SAC-FEMA formulation reduces to the following:

$$\eta_Y \cdot e^{\frac{1}{2}\beta^2{}_Y} \leq 1 \tag{1}$$

Where η_y is the median and β_Y is the standard deviation of the logarithm for the probability distribution for the structural performance parameter *Y*. If *Y* is described by a Lognormal distribution, this is equivalent to checking whether the mean value for the structural performance parameter is less than unity. The parameter β_Y represents the overall effect of uncertainties on the probability distribution for the structural performance parameter. When record-to-record variability is considered, the formulation is modified as:

$$\eta_Y(P_o) \cdot e^{\frac{1}{2}\frac{k}{b}(\beta^2{}_{Y|S_a} + \beta^2{}_{UC})} \leq 1 \tag{2}$$

Where P_o is an acceptable threshold for structural failure probability and $\eta_Y(P_o)$ is the median structural performance parameter corresponding to the acceptable probability P_o. The terms $\beta_{Y|Sa}$ and β_{UC} represent the effect of record-to-record variability and structural modeling uncertainties, respectively, on the total dispersion in the structural performance parameter given spectral acceleration.

4 CHARACTERIZATION OF THE UNCERTAINTIES

It is assumed that the vector $\underline{\theta}$ represents all the uncertain parameters considered in the problem. The vector $\underline{\theta}$ can include the uncertainties in the mechanical properties of the materials, in the structural construction details (a.k.a., *defects*) and in the representation of the ground motion uncertainty. One of the main characteristics of the construction details is that possible deviations from the original configurations are mostly taken into account in those cases leading to undesirable effects. This justifies why the uncertainties related to construction details are usually referred to as the structural defects.

Three types of uncertainties are considered herein, namely, the uncertainty in the ground motion input, the uncertainty in the material mechanical properties, and the uncertainties in the structural detailing parameters. A set of 30 ground motion records are chosen from the European strong motion database for soil type B ($400 < V_s < 600$ m/s), with moment magnitude between 5.3 to 7.2 and the epicentral distance between 7 and 87 km. Moreover, a set of 7 ground motion records are chosen compatible with the spectrum of Euro Code 8. The parameters identifying the prior probability distributions for the material mechanical properties (concrete strength and the steel yielding force) have been based on the values typical of the post world-war II construction in Italy [Verderame et al. 2001a,b]. Table 2a shows these parameters that are used to define the Lognormal probability distributions for the material properties. The prior probability distributions for the structural detailing parameters are defined based on qualitative prior information coming from expert judgment or based on *ignorance* in the extreme case [Jalayer et al., 2008]. Table 2b shows the specifications used to construct the prior probability distributions for the structural detailing parameters. It shows a list of possible defects, their probability distribution and correlation characteristics.

4.1 Updating the probability distributions

The probability distributions for the structural modeling parameters can be updated employing the Bayesian framework for inference. It assumed that the material properties are homogeneous across each floor or construction zone. Therefore, the material property value assigned to each floor can be thought of as an average of the material property values across the floor/zone in question. The results of tests and inspections for each floor can be used to update the probability distribution for the mean material property across the floor. Figure 1a illustrates an example where the test results have verified the nominal value. It can be observed that the updated curve has the same median but has its dispersion reduced.
Assuming that the probability not having a construction defect in a member is equal to *f*, the probability distribution for *f* can be updated using the test results. If the test results indicate that of *n* cases observed n_d of them demonstrate a defect, the probability distributionfor *f* can be updated according to the Bayes formula:

Table 2. a. The list of structural defects and their probability distribution. b. The list of material properties and their probability distribution

The Defects		
The distance between shear reinforcement	uniform	Systematic per floor and typology
The concrete cover (beams)	uniform	Systematic per floor
The concrete cover (columns)	uniform	Systematic per floor
Beam anchorage	Uniform	Systematic per floor
Column superposition	Uniform	Systematic per floor
Positioning of bars (columns)	Uniform	Systematic per floor and typology
Missing bar (beams)	Uniform	Systematic per floor and typology
Error in diameter (columns)	Uniform	Systematic per floor and typology

The material properties			
f_c	LN	165	0.15
f_y	LN	3200	0.08

$$p(f \mid D) = \frac{p(D \mid f)p(f)}{\sum p(D \mid f)p(f)} \tag{3}$$

Where *p*(*f*) is the prior probability distribution for *f* and *p*(*D*|*f*) is the likelihood function for the data *D* given the value of *f*. In the absence of prior information it can be assumed that *p*(*f*) is a uniform distribution from 0 to 1. Use can be made of expert judgment and experience in order to limit the lower and the upper bounds for the defect probability *f*. The likelihood function can calculated using the binomial distribution:

$$p(D \mid f) = \binom{n}{n_d}(1-f)^{n_d} f^{n-n_d} \tag{4}$$

Figure 1b illustrates the prior information on the distance between the shear reinforcement together with updated distribution based on the test results that verify the design value. It can be observed that the consideration of the test data focuses more narrowly the probability distribution around the design value.

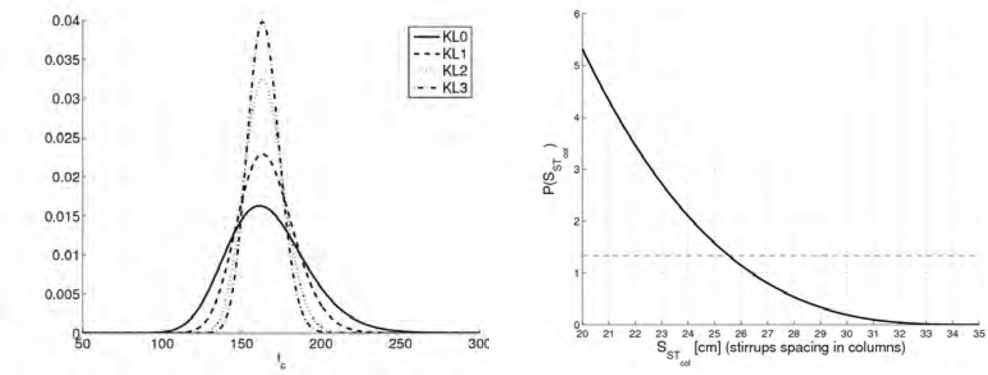

Figure 1. a. The prior and the updated probability distributions for concrete for different knowledge levels. b. The uniform prior and the updated probability distribution for the stirrup spacing.

5 THE CONFIDENCE FACTOR

The confidence factors specified by Euro Code 8 are applied to the mean material properties. Obviously, the approach based on the application of the CF does not take into account explicitly the uncertainties. It would be interesting to investigate what would the application of CF achieve in terms of the seismic performance of the structure.

Consistent with the definition of the KL in the code, one can define the KL_o or the knowledge level before performing the tests. Therefore, for a structure in KL_o the application of the confidence factor implies utilizing smaller material property values. Figure 2a illustrates different values of the material property in question for different values of the CF. It can be observed from Figure 2b that increasing the confidence factor decreases the percentage of values smaller than the nominal value or in other words increases the *confidence* in the nominal value.

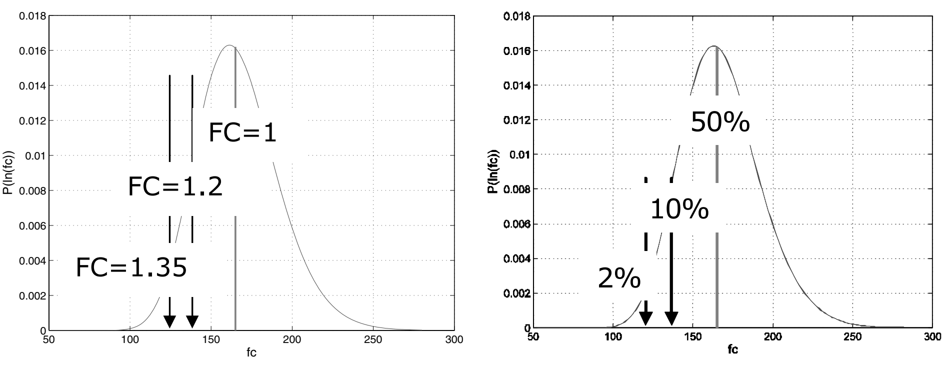

Figure 2. a. The prior probability distribution for concrete and the confidence factors. b. The percentiles corresponding to each CF on the prior probability distribution for concrete.

It is important to note that upon updating the probability distributions for material properties for each knowledge level KL, the percentiles corresponding to each CF are going to change. For example, if the test results verify the nominal value, as is the case in Figure 1a, the percentiles corresponding to each CF are going to decrease; in other words, the *confidence* in the CF is going to increase upon increasing the knowledge level.
In order to map the above discussion into the global performance of the structure, it is assumed for simplicity that the percentiles in the material properties map out invariantly into the structural performance parameter. This approximation would have been exact if the non-linear structural analysis was *a strictly monotonic function*. Figure 3 illustrates the structural fragility curve –built deterministically- by calculating the structural performance variable for different values of CF and plotting them versus their corresponding confidence. It can be seen from figure that with increasing the CF, the structural performance parameter increases and exceeds unity for CF > 1. Therefore, the $Y_{CF>1}$ value corresponds to a Y value with a higher confidence compared to $Y_{FC=1}$. It should be noted that for KL levels higher than KL_o, the CF will map out to even higher confidences.

6 AN EFFICIENT METHOD FOR ESTIMATION OF ROBUST RELIABILITY

The probability of failure given the set of parameters β (e.g., median and standard deviation of the fragility curve) is denoted by $P(F|\beta)$, the expected value (or the robust estimate) for the probability of failure given a set of values Y for the structural performance index can be expressed as [Jalayer et al., 2009]:

$$E[P(F \mid D] = \int_{\Omega} P(F \mid \beta) p(\beta \mid D) d\beta \tag{5}$$

where $p(\beta|D)$ is the posterior probability distribution for the set of parameters β given the data D and Ω is the space of possible values for β. In a similar way, the robust variance for the probability of failure can be calculated as:

$$\sigma_{P(F|D)} = E[P(F \mid D)^2] - E[P(F \mid D)]^2 \tag{6}$$

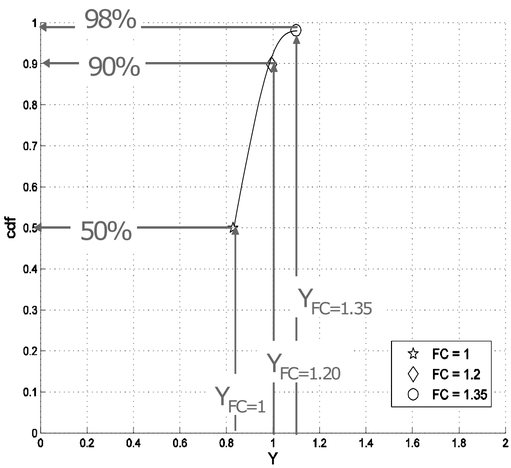

Figure 3. The confidence associated with the application of CF.

The structural reliability or the probability of failure in the case of a structure with modeling uncertainties (no uncertainty in the ground motion) can be expressed by a LogNormal cumulative distribution function (CDF) as following:

$$P(Y(\underline{\theta}) > y) = 1 - \Phi\left(\frac{\log \frac{y}{\eta_Y}}{\beta_Y}\right) \tag{7}$$

Where Y is the structural performance index and η_Y and β_Y are the median and the standard deviation (of the logarithm) for the probability distribution of the structural performance index. Using Bayesian inference, the posterior probability distribution for median and standard deviation based on data Y can be written as [Box and Tiao, 1999]:

$$\begin{aligned} P(\eta_Y, \beta_Y \mid Y) &= k\beta_Y^{-(n+1)} \exp\left(-\frac{\nu s^2 + n(\log \eta_Y - \overline{\log Y})^2}{2\beta^2{}_Y}\right) \\ k &= \sqrt{\frac{n}{2\pi}}\Gamma(\nu/2)^{-1}\left(\frac{\nu s^2}{2}\right)^{\nu/2} \end{aligned} \tag{8}$$

where $Y=\{Y_1, \ldots, Y_n\}$ is the vector of n different realizations of the structural performance index, $\nu=n-1$, logY (overbar) is the mean value for logY and ns^2 is sum of the squares of the deviations from the mean value. The expected value and the standard deviation for the probability of failure can be calculated from Equations 5 and 6 based on the posterior probability distribution $p(\eta_Y, \beta_Y|Y)$ in Equation 8. Otherwise, the best-estimate values for the median and standard deviation can be calculated either as the maximum likelihood pair for the posterior probability distribution function or based on a given (e.g., 84%) confidence.

The structural reliability in the presence of modeling uncertainties and uncertainties in the representation of the ground motion can be calculated from the following LogNormal CDF:

$$\begin{aligned} P(Y(\underline{\theta}) > y \mid S_a) &= 1 - \Phi\left(\frac{\log y - \log \eta_{Y|S_a}}{\beta_{UT}}\right) \\ \beta^2{}_{UT} &= \beta^2{}_{Y|S_a} + \beta^2{}_{UC} \end{aligned} \tag{9}$$

where $\eta_{Y|Sa}$ is the median for the probability distribution of the structural performance index and β_{UT} is the standard deviation for the probability distribution of the structural performance index. The terms $\beta_{Y|Sa}$ and β_{UC} represent the effect of the uncertainty in the ground motion representation, the uncertainty in the material properties and the structural details, respectively. It should be noted that Equation 9 yields the structural fragility; after integrating it with the hazard function for the spectral acceleration, the hazard function for the structural performance variable Y can be obtained.

Suppose that a selection of n ground motion records are used to represent the effect of ground motion uncertainty on the structural performance index. Let $S_{a,i}$ and Y_i represent the spectral acceleration and the performance index for the ground motion record i, respectively. The posterior probability distribution for standard deviation can be calculated as:

$$P(\beta_{UT} \mid Y) = \Gamma(\nu/2)^{-1}\left(\frac{\nu s^2}{2}\right)^{\nu/2} \beta_{UT}{}^{-(\nu+1)} \exp\left(\frac{-\nu s^2}{2\beta^2{}_{UT}}\right) \tag{10}$$

The data pairs (Y, Sa) are gathered by calculating the structural performance measure for the set of n ground motion records applied at the structural model generated by different realizations of material mechanical properties and structural detailing parameters. νs^2 is equal to the sum of the square of the errors for a linear regression of $\log Y$ on $\log S_a$ and a and b are the regression coefficients. The joint posterior probability distribution for the coefficients of the linear regression θ = (log a, b) can be calculated as:

$$\begin{gathered} P(\theta \mid Y, S_a) = k\left[1 + \frac{(\theta-\hat{\theta})^T X^T X(\theta-\hat{\theta})}{\nu s^2}\right]^{-\frac{\nu+1}{2}} \\ k = \frac{\Gamma\left(\frac{n}{2}\right)\sqrt{n\sum \log S^2{}_{a,i} - (\sum \log S_{a,i})^2}}{\nu s^2 \Gamma\left(\frac{1}{2}\right)^2 \Gamma\left(\frac{n}{2}-1\right)} \end{gathered} \tag{11}$$

which is a *bivariate t-distribution* where X is a nx2 matrix whose first column is a vector of ones and its second column is the vector of log $S_{a,i}$ and θ is the 2x1 vector of regression coefficients log a and b. The median and the standard deviation for the probability distribution for $Y|S_a$ can be taken equal to the maximum likelihood estimates $\eta_Y = aS_a{}^b$ and $\beta_{Y|Sa}=s$. The robust estimates for the expected value and the standard deviation of the failure probability can be obtained from Equations 5 and 6 based on the product of the posterior probability distributions $p(\theta|Y,S_a)$ and $p(\beta_{UT}|Y,S_a)$ in Equations 10 and 11, assuming they are independent.

7 NUMERICAL EXAMPLE

As the case-study, an existing school structure located in Avellino, Italy is considered herein. The structure is situated in seismic zone II according to the Italian seismic guidelines OPCM. The structure consists of three stories and a semi-embedded story and its foundation lies on soil type B. For the structure in question, the original design notes and graphics have been gathered. The building is constructed in the 1960's and it is designed for gravity loads only, as it is frequently encountered in the post second world war construction.

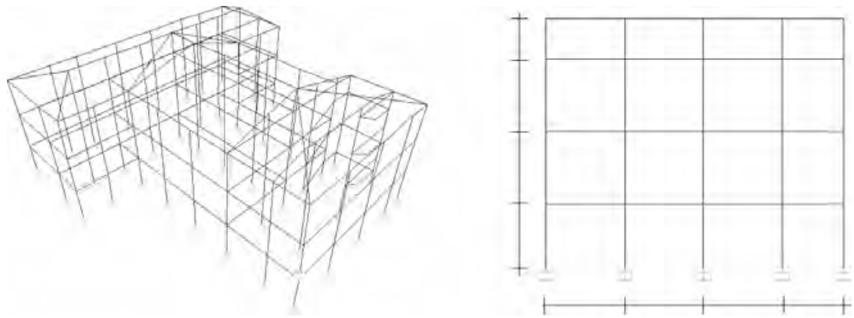

Figure 4. a. The tri-dimensional view of the scholastic building. b. The central frame of the case-study building.

In Figure 4a, the tri-dimensional view of the structure is illustrated; it can be observed that the building is highly irregular both in plane and elevation. In order to reduce the computational effort, the main central frame in the structure is extracted and used as the structural model (Figure 4b). The columns have rectangular section with the following dimensions: first storey: 40x55cm^2, second storey 40x45cm^2, third storey: 40x40cm^2, and forth storey: 30x40 cm^2. The beams, also with rectangular section, have the following dimensions: 40x70cm^2 at first and second storey, and 30x50 cm^2 for the ultimate two floors. It can be inferred from the original design notes that the steel re-bar is of the type Aq40 and the concrete has a minimum resistance equal to 180 kg/ cm^2 [DL1939]. The finite element model of the frame is constructed assuming that the non-linear behavior in the structure is concentrated in plastic hinges.

7.1 The structural performance index

When only the structural modeling uncertainties are considered, the definition of structural capacity in this work is based on the limit state of severe damage as proposed by the Italian Code. That is, the onset of critical behavior in the first element, characterized by member chord rotations larger than 3/4th of the corresponding ultimate chord rotation capacity. The structural demand is characterized by the intersection of the code-based inelastic design spectrum and the static pushover curve transformed into that of the equivalent SDOF system. As an index for the global structural performance, the ratio of structural demand to capacity is used. The component shear failure demand to capacity ratios are also considered; they are combined with the CSM demand to capacity using the *cut set theory* (see below).

When the ground motion uncertainty together with the modeling uncertainties are taken into account, the structural performance index is characterized based on the concept of *cut-sets* in structural reliability. A structural cut-set is defined as a set of structural components that, once all of them have failed, they can transform the whole structure or part of it into a mechanism. Among the set of all possible cut-sets, the critical cut-set is the one that first forms a global mechanism. Therefore, the performance index is taken as the demand to capacity ratio of the strongest component of the weakest cut-set. In the current work, three types of global mechanism are considered: (a) ultimate rotation capacity in the columns (b) formation of soft stories (c) shear failure in the columns. The component yield rotation, ultimate rotation and shear capacities are calculated according to the new Italian Unified Code (MIN.LL.PP 2008a,b). It should be noted that the structural performance in both cases signals failure when it is great than unity and signals no structural failure when it is less than or equal to unity.

7.2 Calculating the structural Fragility: CSM

The structural fragility based on the capacity spectrum method is estimated employing the efficient Bayesian method described above based on the structural performance parameter for a set of 20 Monte Carlo (MC) realizations of the structural model. These realizations take into account the uncertainties in the material properties and the structural defects. The probability distributions for the uncertain parameters are updated according to the increasing knowledge levels KL_o, KL_1, KL_2 and KL_3. As stated before, these knowledge levels are achieved based on the EC8 specifications tabulated in Table 1. Thus, for each knowledge level, the 20 realizations of the structural model are generated from the (updated) probability distributions corresponding to the KL's and based on the results of tests and inspections. Since the results of tests and inspections available for the structure in question did not exactly match the EC8 criteria, the test results used herein are simulated assuming that all the inspections performed verify the original design values. Figure 5 demonstrates the robust fragility curves (the probability of failure for a given value of *Y*) obtained using Equations 5, 7 and 8 for knowledge levels KL_1, KL_2 and KL_3.

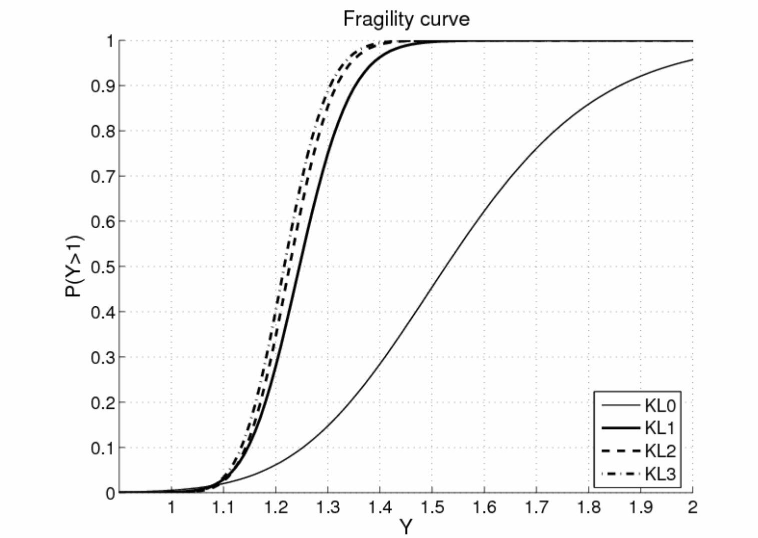

Figure 5. The structural fragility curves for the knowledge levels KL_o, KL_1, KL_2 and KL_3.

It can be observed that the upon increasing knowledge levels both the median and the dispersion in the fragility curves (η_Y and β_Y in Equation 1) decrease as the test results all verify the nominal values. However, it can be seen that the structure does not verify the SAF-FEMA criteria in Equation 1 in none of the knowledge levels. That is, because the median η_Y is larger than unity. This can be attributed to the fact that shear failure is almost everywhere the predominant failure cause; since the structure in its original design had not been designed to resist earthquake-induces shear forces.

7.3 Mapping out the CF into Structural Fragility: Dynamic Analyses

It is shown previously in this work how the CF can be viewed in an approximate way from the stand-point of structural reliability using the non-linear static analyses. In a similar way, it can be shown how the CF can be viewed in the dynamic case. A set of 7 records are chosen compatible with the code-specified spectrum [EC8]. For each CF specified in the code, the structural performance variable for the set of records is calculated for a structural model (without defects) with material properties divided by that CF. The structural performance variable is related to the spectral acceleration using linear regression with parameters $\eta_{Y|Sa}$ and $\beta_{Y|Sa}$. The structural fragility is calculated from Equation 9 setting β_{UC} equal to zero. Finally, the structural fragility is integrated with hazard in order to calculate the probability of failure. Figure 6a illustrates the fragility curve at Y=1 for CF={1,1.2,1.35} It can be observed that with increasing values of CF, the probability of failure (Y>1) increases. The resulting hazard curves corresponding to different values of CF is plotted in red in Figure 7. It should be noted that dispersion in these hazard curves reflects only the record-to-record variability. In a way, similar to Figure 3 for the static case, using increasing values of CF is equivalent to taking into account the structural modeling uncertainties by taking hazard curves (including only the ground motion uncertainty) corresponding to higher confidence levels.

7.4 Calculating the Structural Reliability: The Dynamic Method

The structural hazard curve for increasing levels of knowledge is calculated in this stage by integrating the robust fragilities and the spectral acceleration hazard curve at the site of the structure (extracted from the site of INGV). For each level of knowledge, the robust fragility is calculated from Equations 5, 9, 10 and 11 using a set of 30 MC realization of the structural model. The set of MC realizations for each KL are generated based on the corresponding (updated) probability distributions.

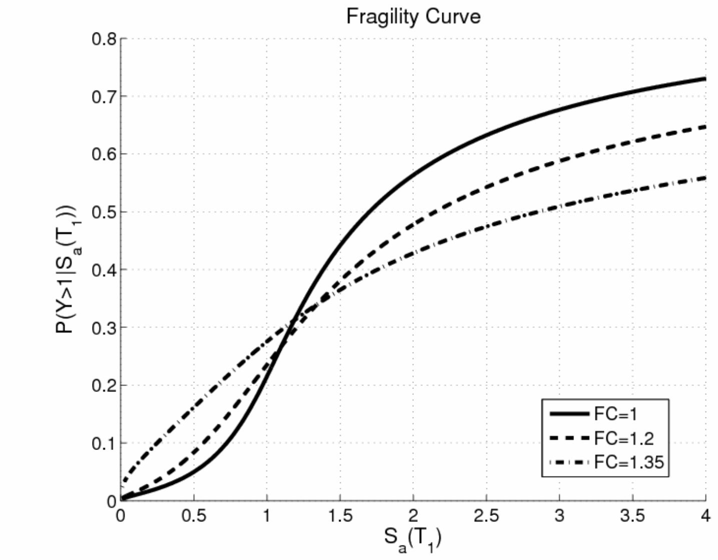

Figure 6. The Fragility curves corresponding to each CF value, the code-compatible dynamic method.

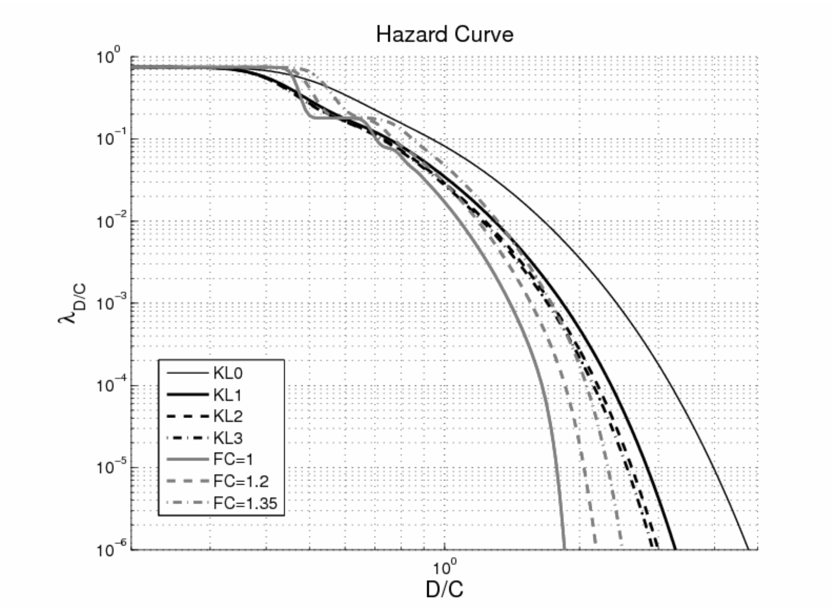

Figure 7. The Hazard curves corresponding to different values of CF and to different knowledge levels.

The resulting hazard curves are plotted (in black) in Figure 7. It can be observed that the with increasing the knowledge level KL, the mean annual frequency of exceeding the structural performance parameter Y decreases. It can be shown (Jalayer and Cornell 2008) that calculating the left-hand side of Equation 2 for a given acceptable probability P_o is equivalent to finding the value corresponding to P_o from the hazard curve for structural performance parameter. For example for an acceptable probability of P_o=0.002 or 10% in 50 years, the structure does not verify for none of the KL. Moreover, in the same manner following the CF approach, the structure does not verify for any of the KL.

8 SOME PERSPECTIVES FOR EC8 IN LIGHT OF THE ITALIAN EXPERIENCE

In this section, the observations made in this work and the previous ones by the authors is used to offer some perspectives for EC8 in the light of this case-study.

The knowledge levels (KL) defined by the code leave a lot of room for interpretation. In other words, the code-based definition for KL does not lead to a unique configuration of tests and inspections. Moreover, it is not clear what level of structural reliability does the application of the confidence factors guarantee. Hence, with the emerging of performance-based design and life-cycle cost analysis in earthquake engineering, there seems to be a need for a code-based method that bridges the different knowledge levels to structural reliability and probabilistic structural performance assessment. A proper evaluation of the structural performance needs to take into account directly the uncertainties in the structural modeling parameters. Thus, the suitable approach for assessment of existing buildings is the probabilistic one which accounts for all the uncertainties. In this sense, the approach of CF can be seen as a deterministic way of dealing with a probabilistic problem.

Intuitively speaking, the relation between the confidence factors and the knowledge levels seems to be highly dependent on the results of in-situ tests and inspections. Therefore, it is necessary to adopt a general probabilistic framework for updating the probability distribution for the uncertain parameters based on the test results. The Bayesian framework for inference seems to be perfectly suitable for this end; as it can sequentially incorporate the incoming tests and inspection results without discarding any prior information available.

There seems to be a need for simple and approachable probabilistic performance-based alternatives to the CF method. These methods can be incorporated in increasing levels of sophistication depending on the importance of building under assessment. The simplified safety-checking format adopted by the American SAC/FEMA guidelines for the assessment and retrofit of existing buildings seems to be an interesting example. This simplified method takes into account the effect of all sources of uncertainty (GN, structural modeling) in the global performance of the structure. This format is expressed as a function the statistical parameters of the structural performance parameter.

In the context of a simple performance-based assessment approach, different classes of existing buildings can be identified and analyzed. The prior probability distributions for the structural modeling uncertainties can be classified and tabulated based on the surveys of expert opinion and experience. It is important to identify those uncertain parameters that affect the structural response in a dominant way (e.g., the material properties, the distance between the shear reinforcement). These prior probability distributions are going to be updated based on the results of tests and inspections. The updated probability distributions are constructed for various KL's, based on special cases of tests and inspection results. Finally, for different classes of structures and different levels of knowledge (and a few special cases of inspection results), the best-estimate values for the parameters defining the adopted safety-checking format/structural fragility can be tabulated. In the case of strategic buildings, it would be useful to recommend some relatively simple and approachable methods suitable for case-specific estimation of the parameters defining the safety checking format and/or structural fragility. This work is a preliminary effort in classifying (for different levels of analysis sophistication) different methods suitable for the performance-based assessment of existing buildings.

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APPLICATION OF BAYESIAN TECHNIQUES TO MATERIAL STRENGTH EVALUATION AND CALIBRATION OF CONFIDENCE FACTORS

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ABSTRACT

A fundamental phase in the assessment of existing reinforced concrete buildings and in their strengthening design is the knowledge procedure. This is based on the collection of different kinds of information regarding: a) the structural system configuration, b) the materials strength, c) the reinforcing steel details, and d) the conditions of the structural elements.

The Italian Code (OPCM 3431, 03-05-05, Allegato 2) as well as the most advanced International Codes (FEMA 356, EC8 Part 3) specifies data collection procedures and ensuing Confidence Factors (CF) to apply to the mean values of the materials strength, based on the completeness and reliability of the information gathered (the so called Level of Knowledge).

Difference in the knowledge procedure about the single structural parameters and the actual possibility of propagation of information gathered on single members unlikely can be accounted for by a single CF. This paper proposes a method for evaluation of material strength and calibration of the relevant CF based on a bayesian procedure; the procedure takes into account the reliability of the results of tests executed by different methods and gives an indication on how to join such information.

KEYWORDS

Confidence Factors, reinforced concrete, strength, non destructive testing methods.

1 INTRODUCTION

Uncertainties on seismic performances of existing buildings are taken into account in the Italian Code OPCM 3431, 3-05-05, Allegato 2, as well as the most advanced International Codes (FEMA 356, Eurocode 8 Part 3) by different safety factors and analysis procedures than new buildings. The existing structures are characterized by uncertainties of intrinsic and epistemic kind; they mainly depend on completeness of information on geometrical and mechanical elements characteristics and also on the possibility they could contain hidden gross errors and may have been submitted to previous earthquake or other accidental actions with unknown effects. The codes specify data collection procedures about the configuration of the structural system, as well as material strength and condition of the structural elements comprising the building. This data shall be obtained from available drawings, specifications, and other documents for the existing construction, and shall be supplemented and verified by on-site investigations, including destructive and non-destructive examination and testing of building materials and components. As a function of the completeness of as-built information

on buildings (Level of Knowledge) the Codes specify different Confidence Factors (CF) to be applied to mean strength values of materials.
OPCM 3431 and EC8-Part 3 follow the same procedure; they define three knowledge levels:

- KL1 : Limited knowledge
- KL2 : Normal knowledge
- KL3 : Full knowledge

The factors determining the knowledge level are:

- geometry: geometrical properties of the structural system and of nonstructural elements that may affect structural response;
- details: amount and detailing of reinforcement in reinforced concrete, connections between steel members, connection of floor diaphragms to lateral resisting structure, bond and mortar jointing of masonry and nature of any reinforcing elements in masonry;
- materials: mechanical properties of the constituent materials.

The input data shall be collected from a variety of sources, including:

- available documentation specific to the building in question,
- relevant generic data sources (e.g. contemporary codes and standards),
- field investigations and in-situ and/or laboratory measurements and tests.

The classification of the levels of inspection and testing depends on the percentage of structural elements that have to be checked for details, as well as on the number of material samples per floor that have to be taken for testing. For ordinary situations recommended minimum values are given. Mean value properties of the existing materials obtained from *in-situ* tests and from the additional sources of information must be scaled by the Confidence Factor, accounting for the level of knowledge attained and, implicitly, the reliability of all the information collected on the building.
Difference in the knowledge procedure about the single structural parameters and the actual possibility of propagation to the structure as a whole of information gathered on single members unlikely can be accounted for by a single CF to be applied to mean materials strength values. Material strength is characterized by an intrinsic spatial variability and an epistemic uncertainty caused by workmanship (for instance not compliance with the original project, execution of structural elements in different times with different materials strength), reliability of testing methods and degradation of material properties with the time. Otherwise amount and detailing of reinforcement, defective detailing, etc., neglecting the intrinsic uncertainties, are characterized by epistemic uncertainties only, mainly due to lack of the original project and/or not compliance with it; collected data on one structural element are certain but don't allow to erase uncertainties about other elements. Therefore, owing to the different nature of the knowledge process and data type for a reinforced concrete structure, it should be better to distinguish between information obtained on materials strength and information obtained on amount and detailing of reinforcement and to define two different procedures: a) a procedure for data processing and evaluation of a Confidence Factor for materials strength; b) a procedure for the assessment of reinforcing details.
Another aspect of the problem is that the use of different testing methods (destructive and nondestructive) on the same concrete gives information with different reliability. Therefore the matter is how to joint such information, taking into account that the reliability of concrete strength given by non-destructive testing is greatly influenced by reliability of the used regression curves. These haven't got a general validity and should be calibrated every time.

The Codes define the minimum number of destructive and non destructive tests that can be executed on a single building but don't take explicitly into account the reliability of each testing method and don't specify how to joint test results.
In this paper a procedure for evaluation of material strength and calibration of the relevant CF is developed, based on the application of the bayesian method. The bayesian method allows to take into account the reliability of the information collected on the material strength; destructive and non-destructive testing results are separately employed, taking into account the reliability of each testing method, to up-date a prior probability distribution function. By the developed method a reference parameter for materials strength is evaluated as the lower bound of a confidence interval for the Bayesian mean. A correlation equation is calibrated to evaluate the CF as a function of the number, the kind and the reliability of each testing employed and of the reliability of prior information, so that it can be applied to a design value of material strength parameter to make it equal to the reference parameter. The design value is a weighting mean of strength values obtained by testing and by prior information.

1.1 Prior knowledge

Prior knowledge of material strength is based on construction documents, reports, reference standards and codes from the period of construction. From this data a mean value of the material strength, μ'_f, and the relating variance, $\sigma'^2_{\mu_f}$ are evaluated.

1.2 Posterior knowledge

The material strength, f, can be statistically described by a lognormal distribution function, that is usually used to describe the probabilistic model of the concrete material properties.
If the variable f is lognormal its natural logarithm, $x = \ln(f)$, is a normal random variable with mean value λ and standard deviation ζ.
The posterior distribution, $f_\Lambda(\lambda)$, of λ is normal with the following statistics:

$$\mu''_\lambda = \frac{\mu'_\lambda \left(\zeta^2_{\lambda,p} / n_p\right) + \bar{x}_{\lambda,p}\sigma'^2_\lambda}{\left(\zeta^2_{\lambda,p} / n_p\right) + \sigma'^2_\lambda} \tag{1}$$

$$\sigma''^2_\lambda = \frac{\left(\zeta^2_{\lambda,p} / n_p\right) \cdot \sigma'^2_\lambda}{\left(\zeta^2_{\lambda,p} / n_p\right) + \sigma'^2_\lambda} \tag{2}$$

where: μ'_λ and σ'^2_λ are the prior mean value and variance, respectively; $\bar{x}_{\lambda,p}$ and $\zeta'^2_{\lambda,p}$ are the mean value and variance of the natural logarithm of test results, respectively.
More than one test method can be employed performing consecutive up-dating of the probability distribution function. Destructive and non-destructive testing results are separately employed, taking into account individual testing reliability.
If n_{DM} is the destructive tests number performed and $f_{i,DM}$ is the strength value from the i-*th* test, the mean sampling value of destructive testing is:

$$\overline{x}_{DM} = \frac{\sum_{i=1}^{n_{DM}} \ln(f_{c_{i,DM}})}{n_{DM}} \tag{3}$$

The evaluation of the sampling variance ζ^2_{DM} can take into account testing errors and errors in regression curve that provides the material resistance as a function of the testing parameter:

$$\zeta^2_{DM} = \zeta^2_{s,DM} + \zeta^2_{t,DM} \tag{4}$$

where $\zeta^2_{s,DM}$ the variance of the natural logarithm of data, given by:

$$\zeta^2_{s,DM} = \frac{\sum_{i=1}^{n_{DM}} \left[\ln(f_{c_{i,DM}}) - \overline{x}_{DM}\right]^2}{n_{DM} - 1} \tag{5}$$

and $\zeta^2_{t,DM}$ is the variance of the regression curve, given by:

$$\zeta^2_{t,DM} = CoV_{t,DM} \cdot \overline{x}_{DM} \tag{6}$$

where $CoV_{t,DM}$ is the coefficient of variation of the regression curve.
When the mean value and variance are known the first updating of the statistics of the distribution, $f_\Lambda(\lambda)$ of λ is possible:

$$\mu''_\lambda = \frac{\mu'_\lambda \left(\zeta^2_{DM} / n_{DM}\right) + \overline{x}_{DM} \sigma'^2_\lambda}{\left(\zeta^2_{DM} / n_{DM}\right) + \sigma'^2_\lambda} \tag{7}$$

$$\sigma''^2_\lambda = \frac{\left(\zeta^2_{DM} / n_{DM}\right) \cdot \sigma'^2_\lambda}{\left(\zeta^2_{DM} / n_{DM}\right) + \sigma'^2_\lambda} \tag{8}$$

A second updating is possible by using non destructive testing results.
If n_{NDM} is the non-destructive tests number performed and $f_{i,NDM}$ is the strength value from the i-*th* test, the mean sampling value of destructive testing is:

$$\overline{x}_{NDM} = \frac{\sum_{i=1}^{n_{NDM}} \ln(f_{c_{i,NDM}})}{n_{NDM}} \tag{9}$$

The evaluation of the sampling variance ζ^2_{NDM} can take into account testing errors and errors in the regression curve that provides the material resistance as a function of the testing parameter:

$$\zeta^2_{NDM} = \zeta^2_{s,NDM} + \zeta^2_{t,NDM} \tag{10}$$

where $\zeta^2_{s,NDM}$ the variance of the natural logarithm of data, given by:

$$\zeta^2_{s,NDM} = \frac{\sum_{i=1}^{n_{NDM}} \left[\ln(f_{c_{i,NDM}}) - \overline{x}_{NDM} \right]^2}{n_{NDM} - 1} \tag{11}$$

and $\zeta^2_{t,NDM}$ is the variance of the regression curve, given as:

$$\zeta^2_{t,NDM} = CoV_{t,NDM} \cdot \overline{x}_{NDM} \tag{12}$$

where $CoV_{t,NDM}$ is the coefficient of variation of the regression curve.
A second updating of the statistics of the distribution, $f_\Lambda(\lambda)$ of λ is now possible:

$$\mu'''_\lambda = \frac{\mu''_\lambda \left(\zeta^2_{NDM} / n_{NDM} \right) + \overline{x}_{NDM} \sigma''^2_\lambda}{\left(\zeta^2_{NDM} / n_{NDM} \right) + \sigma''^2_\lambda} \tag{13}$$

$$\sigma'''^2_\lambda = \frac{\left(\zeta^2_{NDM} / n_{NDM} \right) \cdot \sigma''^2_\lambda}{\left(\zeta^2_{NDM} / n_{NDM} \right) + \sigma''^2_\lambda} \tag{14}$$

Replacing the value of μ''_λ and σ''^2_λ in the previous equation the posterior statistics value are sought:

$$\mu'''_\lambda = \frac{\dfrac{\mu'_\lambda}{(\sigma'_\lambda)^2} + \dfrac{n_{DM} \cdot \overline{x}_{DM}}{(\zeta_{DM})^2} + \dfrac{n_{NDM} \cdot \overline{x}_{NDM}}{(\zeta_{NDM})^2}}{\dfrac{n_{DM}}{(\zeta_{DM})^2} + \dfrac{n_{NDM}}{(\zeta_{NDM})^2} + \dfrac{1}{(\sigma'_\lambda)^2}} \tag{15}$$

$$\sigma'''^2_\lambda = \frac{1}{\sqrt{\dfrac{n_{DM}}{(\zeta_{DM})^2} + \dfrac{n_{NDM}}{(\zeta_{NDM})^2} + \dfrac{1}{(\sigma'_\lambda)^2}}} \tag{16}$$

1.3 Confidence interval on mean value

It's possible to improve mean value statistics reliability by applying the confidence interval for the mean. What is of interest is the lower confidence limit $<\mu)_{1-\alpha}$, which is the value that the population mean will be larger with a confidence level of $(1-\alpha)$. For a Normal distribution function with unknown variance it is given by:

$$P\left(\frac{\overline{x} - \mu}{s / \sqrt{n}} \le t_{\alpha/2, n-1} \right) = 1 - \alpha \tag{17}$$

where: $\bar{x}$ and s are the sampling mean and standard deviation, respectively; n is the sampling dimension; μ is the mean of the population from which the sample is observed; $(1-\alpha)$ is the specified level of confidence and $t_{\alpha/2,n-1}$ is the value of the t-Student variate with $n-1$ degrees of freedom having a cumulative probability level $\alpha/2$. Rearranging the Eq. (17) the lower limit of the confidence interval $(1-\alpha)$ is obtained:

$$\bar{x} \geq \mu - t_{\alpha/2,n-1} \cdot s/\sqrt{n} \tag{18}$$

A 95% lower confidence level is here considered:

$$\mu'''_{\lambda,\inf} = \mu'''_{\lambda} - t_{\alpha/2,n-1}\sigma'''_{\lambda} \tag{19}$$

Introducing the expression of μ'''_{λ} in the Eq. (19) we get:

$$\mu'''_{\lambda,\inf} = \sigma'''_{\lambda}\left\{(\sigma'''_{\lambda})\left[\frac{\mu'_{\lambda}}{(\sigma'_{\lambda})^2} + \frac{n_{MD}\cdot\bar{x}_{MD}}{(\zeta_{MD})^2} + \frac{n_{MND}\cdot\bar{x}_{MND}}{(\zeta_{MND})^2}\right] - t_{\alpha/2,n-1}\right\} \tag{20}$$

The parameter $\mu'''_{\lambda,\inf}$ is related to the variable $\lambda = E\left[\ln(f)\right]$; from it it's possible to evaluate the parameter $\breve{m}'''_{\inf,\breve{m}_f}$, the lower confidence limit for the Bayesian median value, $\breve{m}'''_{\breve{m}_f}$, of median value, $\breve{m}_f$, of the material strength, f, that is the value with a 50 % probability:

$$\breve{m}'''_{\inf,\breve{m}_f} = e^{\mu'''_{\lambda,\inf}} \tag{21}$$

1.4 Definition of the design material strength

The parameter $\breve{m}'''_{\inf,\breve{m}_f}$ represents the value for the structural assessment; in order to facilitate its evaluation a simplified procedure is defined. The value $\breve{m}'''_{\inf,\breve{m}_f}$ can be obtained by scaling with an opportune Confidence Factor a weighted mean, μ, of the sampling mean vales obtained by the different testing methods and the prior information:

$$\mu_D = \frac{\mu}{FC} \cong \breve{m}'''_{\inf,\breve{m}_f} \tag{22}$$

where:

$$\mu = \left[\frac{\mu'_f + n_{DM}\cdot\bar{x}_{DM} + n_{NDM}\cdot\bar{x}_{NDM}}{1 + n_{DM} + n_{NDM}}\right] \tag{23}$$

where $\bar{x}_{DM}$ and $\bar{x}_{NDM}$ are the sampling mean of the destructive and non destructive tests, respectively; n_{DM} and n_{NDM} are the corresponding sampling dimension. Generally, if M_i is the *i*-th testing method adopted, the material strength for the assessment is:

$$\mu = \left[\frac{\mu'_f + \sum_i n_{M_i} \cdot \bar{x}_{M_i}}{1 + \sum_i n_{M_i}} \right] \tag{24}$$

where $\bar{x}_{Mi}$ is the sampling mean of the i-th testing method and n_{Mi} its dimension.

1.5 Calibration of Confidence Factors for concrete strength

The CF can be expressed in an explicit form as a function of the Bayesian coefficient of variation V_μ for the median value of the material strength:

$$FC = a + c \cdot V_\mu^\omega \tag{25}$$

The parameter V_μ, which estimate the reliability of available information, is defined as:

$$V_\mu = \frac{\sigma_\mu}{\mu_\mu} = \frac{\sqrt{\sum_i \frac{n_{Mi}}{s_{s,Mi}^2 + s_{t,Mi}^2}}}{\sum_i \frac{n_{Mi} \cdot \bar{x}_{Mi}}{s_{s,Mi}^2 + s_{t,Mi}^2}} \tag{26}$$

where $s_{s,Mi}^2$ and $s_{t,Mi}^2$ are the sampling variance and the variance of the regression curve of the *i*-th testing method, respectively. The Eq. (25) has been calibrated for concrete strength by applying the least squares method. A Monte Carlo method has been used to simulate sampling with destructive and non destructive testing. The simulated samplings are extracted from a population with median concrete strength $\breve{m}_{f_c}$ ranging from 10 MPa to 40 MPa and hypothesizing the possible range of all the parameters ($V_{f_c}, \mu'_{f_c}, V'_{f_c}, \mu'_{f_c}, n_{DM}, V_{t,DM}, n_{NDM}, V_{t,NDM}$). The parameters a, c and ω have been evaluated by applying the least squares method to the set of values μ_D calculated by Eq. (22) with FC given by (25), and the set of $\breve{m}'''_{\inf,\breve{m}_{f_c}}$ calculated by the Bayesian procedure described above, so that the condition given in Eq. (22) is met. The resulting equation for the CF is the following:

$$FC = 0.9 + \sqrt{V_\mu} \tag{27}$$

The equation (27) is effective if samples have been extracted from homogeneous zones of the structure. If in the structure potential non homogeneous zones are identified, the t-Student test can be executed on the mean vales extracted from the two zones.
For two independent samples, with homogeneous variance, the t-Student test is executed by verifying the following condition:

$$\frac{\left(\bar{X}_A - \bar{X}_B\right) - \left(\mu_A - \mu_B\right)}{\sqrt{S_p^2 \cdot \left(\frac{1}{n_A} + \frac{1}{n_B}\right)}} \le t_{\alpha/2,(n_A + n_B - 2)} \tag{28}$$

where:

- $\overline{X}_A$ and $\overline{X}_B$ are the sampling means of the sample extracted from A and B zones, respectively;
- μ_A and μ_B are the expected mean values; if A and B are homogeneous zone the following condition is met: $\mu_A - \mu_B = 0$;
- n_A and n_B are the sampling dimensions of A and B;
- S_p^2 is the pooled variance given by:

$$S_p^2 = \frac{(n_A - 1)s_A^2 + (n_A - 1)s_B^2}{n_A + n_B - 2} \tag{29}$$

- $t_{\alpha/2,(n_A+n_B-2)}$ is the value of the t-Student variate with $n_A + n_B - 2$ degrees of freedom having a cumulative probability level $\alpha/2$.

If the t-student test identify non homogeneous zones these must be separately evaluated considering two different median value for concrete strength with two CF.

1.6 Summary of the Procedure

Procedure for evaluation of CF for material strength is based on the following steps:

- acquisition of prior information;
- determination of possible non homogeneous zones;
- choice of destructive and non destructive testing methods to be applied;
- definition of coefficient of variation for each testing method in relation to the used regression curves;
- execution of destructive test in each homogeneous zone and evaluation of the mean value $\overline{x}_{DM}$ and variance $s_{s,DM}^2$ for each zone;
- execution of non destructive tests in each homogeneous zone and evaluation of the mean values $\overline{x}_{NDM,i}$ and variance $s_{s,NDM,i}^2$ for each zone;
- evaluation of V_μ by equation (26);
- evaluation of FC by equation (27).

1.7 Evaluation of the correlation equations between concrete strength and non-destructive testing parameters and determination of CoV for material strength

Functional relations that give the concrete strength value from non-destructive testing results are defined by regression analysis on data from destructive testing results (cores).

Concrete strength, f_c, is evaluated as a function f of testing parameters $\mathbf{X} = (x_1, \ldots, x_k)$:

$$f_c = f(\mathbf{X}, \boldsymbol{\theta}) + \varepsilon \tag{30}$$

where $\boldsymbol{\theta}$ is the parameter vector and ε is a r.v., with unit mean value and standard deviation σ, taking into account errors in the definition of the functional relation f. The function f is usually defined as a polynomial in $\mathbf{X}$, with $\boldsymbol{\theta}$ coefficient vector ($\boldsymbol{\theta} = (\beta_1, \ldots, \beta_k, \sigma^2)$); the probability distribution function of f is studied in the context of a set experiments $i = 1, \ldots n$

on which f_c and $\mathbf{X}$ are measured. The vector f_c is the vector of outcomes of destructive testing (cores) by which the correlation equations of non-destructive testing methods (rebound, ultrasonic pulse velocity, Sonreb, etc.) are calibrated; the f_c vector has n elements, corresponding to the n test outcomes (observations). $\mathbf{X}$ is a $n \times k$ matrix, with $k =$ predictors number, corresponding to the testing methods included in the correlation equation. If the regression includes an intercept, one of the columns of X is a column of ones. The parameters in $\boldsymbol{\theta}$ are the regression coefficients β_i and the error variance of the fitted model, σ^2. The $i-th$ element in f_c is given by:

$$f_{ci} = \beta_1 x_{i1} + \ldots + \beta_k x_{ik} + \varepsilon \tag{31}$$

The probability distribution function of the r.v ε is assumed to be Normal; under the hypothesis of independent errors and with equal variance, the probability distribution function of f_c given parameters β and σ^2 and predictors $\mathbf{X}$ is a normal distribution with mean $\mathbf{X}\beta$ and variance σ^2:

$$\left(f_c \mid \beta, \sigma^2, \mathbf{X}\right) \sim N\left(\mathbf{X}\boldsymbol{\beta}, \mathbf{I}\sigma^2\right) \tag{32}$$

$$P\left(f_c \mid \beta, \sigma^2, \mathbf{X}\right) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left[-\frac{1}{2}\frac{\left(f_c - \mathbf{X}\boldsymbol{\theta}\right)^2}{\sigma^2}\right] \tag{33}$$

where $\mathbf{I}$ is the identity matrix $n \times n$.

Coefficients in $\boldsymbol{\theta}$ are generally unknown and can be estimated by a regression analysis on the in-situ test results; advantages in application of the bayesian inference method lies mainly in deriving conclusion on the parameters $\boldsymbol{\theta}$ and on the data in a probability statement.

The method presented below come from the Bayesian theory for normally-distributed random variables (Gelman et al. 1995).

By the bayesian inference, once the regression model has been defined the probability distribution of parameters conditional on the data, $p\left(\boldsymbol{\theta} \mid f_c\right)$, and the predicted distribution of unobserved data, $p\left(\tilde{f}_c \mid f_c\right)$, can be evaluated. By applying the Bayes' rule, the posterior distribution function, $p\left(\boldsymbol{\theta} \mid f_c\right)$, is given by:

$$p\left(\boldsymbol{\theta} \mid f_c\right) = \frac{p\left(\boldsymbol{\theta}, f_c\right)}{p\left(f_c\right)} = \frac{p\left(\boldsymbol{\theta}\right) p\left(f_c \mid \boldsymbol{\theta}\right)}{p\left(f_c\right)} \tag{34}$$

where $p\left(\boldsymbol{\theta}\right)$ is the prior distribution function of the parameters, $p\left(f_c \mid \boldsymbol{\theta}\right)$ is the sample distribution and $p\left(f_c\right) = \sum_{\theta} p\left(\boldsymbol{\theta}\right) p\left(f_c \mid \boldsymbol{\theta}\right)$. An alternative form of the Eq. (34) omits the term $p\left(f_c\right)$ which doesn't depend on $\boldsymbol{\theta}$ and can be considered constant:

$$p\left(\boldsymbol{\theta} \mid f_c\right) \propto p\left(\boldsymbol{\theta}\right) p\left(f_c \mid \boldsymbol{\theta}\right) \tag{35}$$

Alternatively the joint posterior distribution of β and σ^2 can be expressed as the conditional probability of β given σ^2 times the marginal posterior probability of σ^2:

$$p\left(\beta,\sigma^2 \mid f_c\right) = p\left(\beta \mid \sigma^2, f_c\right) p\left(\sigma^2 \mid f_c\right) \tag{36}$$

Under the assumption of Normal errors, independent and with equal variance, the coefficients estimates are also normally distributed:

$$\beta \mid \sigma^2, f_c \sim N\left(\hat{\boldsymbol{\beta}}, \mathbf{V}_\beta \sigma^2\right) \tag{37}$$

where:

$$\hat{\boldsymbol{\beta}} = \left(\mathbf{X}^T\mathbf{X}\right)^{-1} \mathbf{X}^T \mathbf{f}_c \tag{38}$$

$$\mathbf{V}_\beta = \left(\mathbf{X}^T\mathbf{X}\right)^{-1} \tag{39}$$

The marginal posterior distribution of σ^2 is an $Inv-\chi^2$ with $\nu = n-k$ degree of freedom:

$$\sigma^2 \mid f_c \sim Inv-\chi^2\left(n-k, s^2\right) \tag{40}$$

where:

$$s^2 = \frac{1}{n-k}\left(\mathbf{f}_c - \mathbf{X}\hat{\boldsymbol{\beta}}\right)^T \left(\mathbf{f}_c - \mathbf{X}\hat{\boldsymbol{\beta}}\right) \tag{41}$$

The posterior marginal distribution of $\beta \mid f_c$ is a multivariate t-Student with $n-k$ degree of freedom:

$$p\left(\beta \mid f_c\right) = \int p\left(\beta \mid \sigma^2, f_c\right) p\left(\sigma^2 \mid f_c\right) d\sigma^2 \tag{42}$$

The predictive conditional posterior distribution for new data $\tilde{f}_c$, given σ^2, is also Normal with mean value:

$$\begin{aligned} E\left(\tilde{f}_c \mid \sigma^2, f_c\right) &= E\left(E\left(\tilde{f}_c \mid \beta, \sigma^2, f_c\right) \mid \sigma^2, f_c\right) \\ &= E\left(\tilde{X}\beta \mid \sigma^2, f_c\right) \\ &= \tilde{\mathbf{X}}\tilde{\boldsymbol{\beta}} \end{aligned} \tag{43}$$

and variance:

$$var\left(\tilde{f}_c \mid \sigma^2, f_c\right) = \left(\mathbf{I} + \tilde{\mathbf{X}}\mathbf{V}_\beta\tilde{\mathbf{X}}^T\right)\sigma^2 \tag{44}$$

The marginal posterior distribution for new observations, $\tilde{f}_c$, $p\left(\tilde{f}_c \mid f_c\right)$, averaging over σ^2, is a multivariate t-Student with mean value $\tilde{\mathbf{X}}\tilde{\boldsymbol{\beta}}$, squared scale matrix $\left(\mathbf{I}+\tilde{\mathbf{X}}\mathbf{V}_\beta\tilde{\mathbf{X}}^T\right)s^2$ and $\nu = n-k$ degree of freedom:

$$p\left(\tilde{f}_c \mid f_c\right) = \text{multivariate } t\left[n-k, \tilde{\mathbf{X}}\tilde{\boldsymbol{\beta}}, \left(\mathbf{I}+\tilde{\mathbf{X}}\mathbf{V}_\beta\tilde{\mathbf{X}}^T\right)s^2\right] \tag{45}$$

Therefore the variance of $\tilde{f}_c$ is given by:

$$var\left(\tilde{f}_c \mid f_c\right) = \frac{\nu}{\nu-2}\left(\mathbf{I}+\tilde{\mathbf{X}}\mathbf{V}_\beta\tilde{\mathbf{X}}^T\right)s^2 \tag{46}$$

1.8 Application to Existing Buildings

The proposed method has been applied to evaluation of median concrete strength for an existing building.
Destructive (cores) and non destructive (sclerometer and ultrasonic pulse velocity) testing has been executed. No prior information was available. Non destructive testing results have been combined by the Sonreb method, using a regression curve calibrated on data from destructive tests results, as previously illustrated, and two regression curves taken from literature with relevant coefficients of variation:

$$f_c = 7.695\cdot 10^{-10}\cdot I^{1.4}\cdot V^{2.6} \quad (Kg/cm^2; m/s) \qquad \text{RILEM 43 CND} \tag{47}$$

$$f_c = 1.2\cdot 10^{-9}\cdot I^{1.058}\cdot V^{2.446} \quad (MPa; m/s) \qquad \text{Di Leo et al.} \tag{48}$$

where I is the rebound number and V is the ultrasonic pulse velocity.
For the Sonreb regression curve evaluated on tests results, the mean value and the standard deviation have been calculated as in § 1.7; for the regression curves given by Eq. (47) and Eq. (48), under the hypothesis that the two parameters I and V are independent, the mean value and the variance of concrete strength are evaluated by the following equations:

$$\overline{x}_{NDM} = f\left(\overline{I}, \overline{V}\right) \tag{49}$$

$$s_{NDM}^2 = \left(\frac{\partial f_c}{\partial I}\right)_{\overline{I}}^2 \cdot s_I^2 + \left(\frac{\partial f_c}{\partial V}\right)_{\overline{V}}^2 \cdot s_V^2 + V_{NDM}^2\cdot \overline{x}_{NDM}^2 \tag{50}$$

where $\overline{I}$ and $\overline{V}$ are the mean value of the rebound number and of the ultrasonic pulse velocity, respectively; $\overline{x}_{MND}$ is the mean value of concrete strength obtained from application of Sonreb method, $f\left(\overline{x}_{MND}\right)$ is expression of the regression curve; s_I^2 and s_V^2 are the variance of the non-destructive parameters; finally, V_{NDM} is the coefficients of variation of Sonreb curves, taken form Giannini e al. (2003).

Figure 1 and Figure 2 show the CF variation compared to the variation of mean values of concrete strength obtained by destructive and nondestructive methods and to the relevant coefficient of variation; they point out that the CF correctly reflect the information reliability.

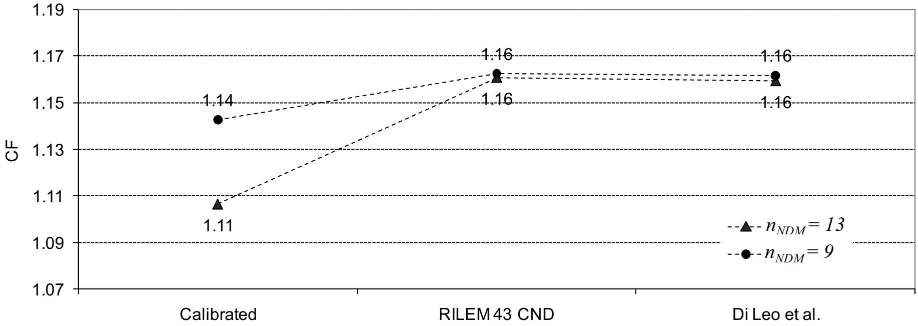

Figure 1. CF vs n_{NDM} for different Sonreb equations.

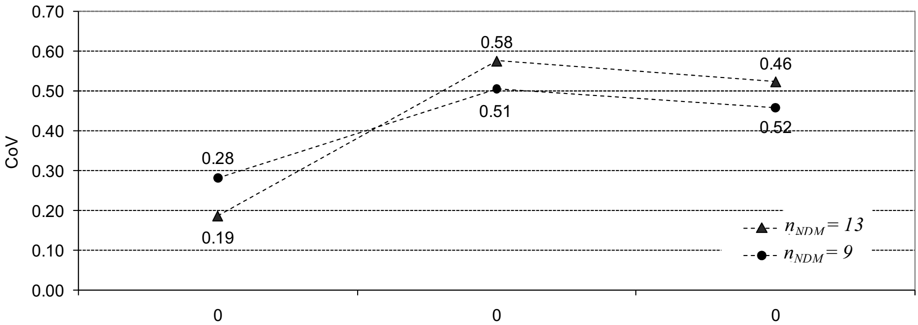

Figure 2. CoV vs n_{NDM} for different Sonreb equations.

2 CONCLUSIONS

This paper proposes a procedure for evaluation of CF which goes beyond the definition given by Italian (OPCM 3431) and European (EC8-Part3) Codes. This is because of the different nature of the knowledge process and uncertainties which characterize material strength and reinforcement detailing. The paper proposes the definition of different CF and an expression to evaluate CF for material strength as a function of scattering of testing data and prior information. The proposed procedure has been calibrated on simulated cases and tested by an existing building material strength evaluation. The application points out that the CF correctly reflects the informations reliability.

3 ACKNOWLEDGEMENTS

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ESTIMATION OF THE IN-SITU CONCRETE STRENGTH: PROVISIONS OF THE EUROPEAN AND ITALIAN SEISMIC CODES AND POSSIBLE IMPROVEMENTS

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ABSTRACT
Evaluation and possible retrofitting of existing RC buildings require specific procedures being set up. Many seismic codes have been developed with reference to this topic around the world. In Europe, Eurocode 8 – part 3 is specifically devoted to this subject. Investigations have a crucial role to adequately know the structures to be subjected to evaluation. For this reason, there is an increasing need to put at disposal sufficiently reliable as well as not very expensive methods to estimate in-situ material properties. The results presented in the paper confirm that a suitable combination of Non Destructive and core tests provides an effective solution from both the economical and technical point of view. Further, based on the experience deriving from widespread in-situ and laboratory experimental investigations, some possible improvements of the current code provisions are given. Finally, an outline of future research developments is provided.

KEYWORDS
RC structures, assessment, Eurocode 8, concrete strength, in-situ tests.

1 INTRODUCTION

A large quantity of Reinforced Concrete (RC) buildings both private and public, now placed in seismic zones, were originally designed taking into account only gravity loads and without explicitly provide ductile detailing. Evaluation and possible retrofitting of such buildings require specific procedures being set up, where investigations have a crucial role to get an adequate knowledge of the structure to be evaluated. Many seismic codes have been developed with reference to this topic around the world, e.g. FEMA 356, (2000) in the United States and the 2006 Recommendations NZSEE, (2006) in New Zealand. In Europe, Eurocode 8 – 3 (CEN, 2005) is specifically devoted to this subject, to pursue the following main objectives:

- to provide criteria for the evaluation of the seismic performance of existing individual building structures;
- to describe the approach in selecting necessary corrective measures;
- to set forth criteria for the design of the retrofitting measures (i.e. conception, structural analysis including intervention measures, final dimensioning of structural parts and their connections to existing structural elements).

Structural evaluation and possible structural intervention of existing structures are typically subjected to a different degree of uncertainty (level of knowledge) than the design of new structures. Different sets of material and structural safety factors are therefore required, as well as different analysis procedures, depending on the completeness and reliability of the information available. To this purpose, codes require that a knowledge level (KL) is defined in order to choose the admissible type of analysis and the appropriate confidence factor (CF) values in the evaluation. The design strength to be used in the safety verifications is computed on the basis of the mean value obtained from tests and other additional sources of information, divided by the achieved CF value. Among the factors determining the KL, there are the mechanical properties of the structural materials. In RC structures, the compressive strength of concrete has a crucial role on the seismic performance and is usually difficult and expensive to estimate. Reliable procedures to take into account the factors influencing the estimation of in-situ concrete strength, particularly in case of poor quality concrete, are not currently available. According to various codes (e.g. CEN EC8-3, 2005; ACI 228, 1998) estimation of the in-situ strength has to be mainly based on cores drilled from the structure. However, non-destructive tests (NDTs) can effectively supplement coring thus permitting more economical and representative evaluation of the concrete properties throughout the whole structure under examination. The critical step is to establish reliable relationships between NDT results and actual concrete strength. The approach suggested in most codes (e.g. in CEN EC8-3, 2005) is to correlate the results of in-situ NDTs carried out at selected locations with the strength of corresponding cores. Thus, NDTs can strongly reduce the total amount of coring needed to evaluate the concrete strength in an entire structure. In this paper the characteristics of the most usual methods (core testing, rebound number, ultrasonic pulse, …) has been shortly examined. Particularly, the combined Sonreb method has been described, and a specific procedure to estimate concrete strength has been proposed. Further, the provisions reported in some seismic codes to achieve information on mechanical properties and conditions of concrete are described. Based on the results from experimental in-situ or laboratory programs carried out on existing building structures designed only for gravity loads, the variability of in-situ concrete strength for populations of RC structures and within single structures has been examined, thus providing some provisions (number and type of test, locations for sampling, etc.) to get the better knowledge of in-situ strength.

2 STRENGTH ESTIMATION IN EUROPEAN AND ITALIAN SEISMIC CODE

Three knowledge levels are defined in both European (CEN EC8-3, 2005) and Italian Code IC (Ministero delle Infrastrutture, 2008, 2009), that is limited, normal and full knowledge, in order to choose the appropriate CF values in the evaluation. A certain knowledge level regarding material properties can be obtained complementing test results and information derived from standards at the time of construction or provided by original design specifications or test reports. When the test results do not confirm such information a higher level of testing is required (e.g. from limited to extended) and the available information has to be given up. This consideration is not clearly stated both in the EC8-3 and in the Italian Code. While in the EC8 it is specified when a limited knowledge is pursued, this does not happen for the KL2; the contrary happens in the IC. Particularly, EC8 specifies that:

– *"KL1 (Limited knowledge): ... default values should be assumed according to standards at the time of construction, accompanied by limited in-situ testing in the most critical elements. ... However, if values from tests are lower than default values according to standards of the time of construction, an extended in-situ testing is required.*

- *KL2 (Normal Knowledge): information on the mechanical properties of the construction materials is available either from extended in-situ testing or from original design specifications. In this latter case, limited in-situ testing should be performed.*
- *KL3 (Full Knowledge): information on the mechanical properties of the construction materials is available either from comprehensive in-situ testing or from original test reports. In this latter case, limited in-situ testing should be performed."*

Experience shows that information from original design specifications or test reports have usually poor reliability when related to concrete properties, thus strength estimation shall be always based on, or at least complemented by, testing. For this reason, using the term "should" in these circumstances, seems inappropriate and substitution with the term "shall", or at least "have to", is suggested. An analogous consideration can be made as for the use of non destructive testing in estimating concrete properties. EC8 specifies that: *"Use of non-destructive test methods (e.g., Schmidt hammer test, etc.) should be considered; however such tests should not be used in isolation, but only in conjunction with destructive tests (i.e. tests on material samples extracted from the structure)"*. On the contrary, experience clearly shows the need that NDTs shall be always used in conjunction, as clearly stated in IC. The classification of the levels of testing is dependent on the number of material samples per floor that have to be taken for testing. For ordinary situations the recommended minimum values in EC8 are given in Table 1.

Table 1. Recommended minimum requirements for different levels of testing.

	For each type of primary element (beam, column, wall)
Level of testing	Material samples per floor
Limited	1
Extended	2
Comprehensive	3

In the IC the same requirements are recommended, but there is a specification relating the minimum number of samples to the dimensions of the structure, that is a floor area equal to 300 m^2, beyond which such minimum number needs to be increased. Further, to effectively apply the rigid provisions of Table 1, IC explains that recommended requirements have to be considered as reference values to be adapted to each single particular case, also providing some directions. These directions derive from a widespread experience in estimating concrete properties, thus their use could be suggested also in the EC8-3.

3 REVIEW OF TESTING METHODS

In-situ concrete strength can be estimated through non-destructive (NDT) and destructive methods. The most widespread methods for existing buildings include core testing, rebound number, ultrasonic pulse velocity, combined non destructive methods. Specifications to use core testing are given in several standards (e.g. in Italy UNI EN 12504, 2002). Although core testing is the most direct and reliable method to estimate concrete strength in a structure, it has to be taken into account that there are many differences between the strength measured on core specimens and the actual in-situ strength. The main factors are the size and geometry of the cores, the coring direction, the presence of reinforcing bars or other inclusions, the effect of drilling damage. To this purpose, a relationship to convert the strength of a core specimen f_{core} into the equivalent in-situ value f_c is given in (Dolce *et al.*, 2006):

$$f_c = \left(C_{H/D} \cdot C_{dia} \cdot C_a \cdot C_d\right) \cdot f_{core} \tag{1}$$

where:

- $C_{H/D}$ = correction for height/diameter ratio H/D, equal to $C_{H/D} = 2/(1.5 + D/H)$;
- C_{dia} = correction for diameter of core D, equal to 1.06, 1.00 and 0.98 for D, respectively, equal to 50, 100 and 150 mm;
- C_a = correction for the presence of reinforcing bars, equal to 1 for no bars, and varying between 1.03 for small diameter bars (ϕ 10) and 1.13 for large diameter bars (ϕ 20);
- C_d = correction for damage due to drilling.

The correction coefficient C_d asks for a particular attention: whereas constant value equal to 1.06 is suggested in ACI, 2003, in the technical literature also C_d = 1.10 is proposed provided that the extraction is carefully carried out by experienced operators. However, taking into account that the lower the original concrete quality the larger the drilling damage, it appears more suitable to put C_d = 1.20 for f_{core} < 20 MPa, and C_d = 1.10 for f_{core} > 20 MPa, as suggested in Dolce *et al.*, 2006. More recent results are provided in Masi, 2008, where the possible reduction amount of core specimen strength due to drilling damage has been examined on the basis of a wide experimental database, thus providing more accurate correction coefficients.

Rebound number and ultrasonic pulse velocity methods are quick and little expensive. Specifications to apply them in concrete structures are given in several standards (e.g. in Italy UNI EN 12504-2, 2001; UNI EN 12504-4, 2005). Rebound number test consists in measuring the rebound distance of a plunger pulled by a spring against the surface of the concrete specimen. Because the test investigates only the surface layer, the result may not represent the interior concrete. For example, the carbonation process typical of old concretes heavily affects the rebound numbers, providing high values, which do not correspond to actually high strengths. Ultrasonic test requires the determination of the velocity of propagation of ultrasonic longitudinal waves in concrete, using two transducers placed at a known distance, and then correlating this value to the concrete properties by using curves provided with the test device or in other references (e.g. Masi, 2008). Really, the correlation may be affected by a number of factors, such as the water/cement ratio, the moisture content, the presence of reinforcement, the age, etc. For this reason a general correlation cannot be proposed, but the specific characteristics and conditions of the concrete under test have to be taken into account, as recommended by several international standards (e.g. ACI standard 228.2R-98, 1998). On the contrary, ultrasonic method is particularly suitable for the detection of local defects (voids, cracks, etc.). Measurements can be made by placing the two transducers on opposite faces (direct transmission), on adjacent faces (semi-direct transmission), or on the same face (indirect or surface transmission) of a concrete structure or specimen.

Combined non destructive methods are treated in the RILEM NDT4, 1993, recommendation. They aim to increase the accuracy of the estimation, compared with that from any single method. SONREB method, based on the combination of ultrasonic pulse velocity V and rebound number S measurements, is the best known and widely used of combined methods. In RILEM NDT4, 1993, iso-strength curves for a reference concrete are suggested, where the compressive strength can be estimated by knowing the rebound number and pulse velocity values. When estimating the strength of a specific in-situ concrete, in order to improve the accuracy of prediction, a number of correction coefficients that allow for the differences in composition compared to the reference concrete, have to be evaluated and applied to the iso-strength values. These coefficients of influence take into account differences in cement type and content, aggregate types and size, presence of admixtures. In practice it is very rare to know the composition of the concrete under test, thus a total coefficient of influence needs to be estimated by using the results of some core tests.

4 ASSIGNATION OF CONCRETE STRENGTH

NDTs are not satisfactory methods to estimate concrete strength, unless their results are correlated to core tests. On the contrary, they can be effectively used as a means to determine the uniformity of concrete properties in a structure. In estimating in-situ concrete strength some statements needs to be preliminarily made:

a) core tests are as more reliable as more intrusive and expensive they are, but only a limited number of them can be carried out in practice; this results in estimates which can be not representative of the in-structure property variations;
b) on the contrary, NDTs are very simple and little expensive, but they provide unreliable predictions of concrete strengths;
c) a suitable combination of cores and non destructive tests is the best solution, providing as much reliable estimates as widespread tests are made all over the structure.

In the RILEM NDT4, 1993, standards a procedure based on the determination of a total coefficient of influence is proposed to correlate non destructive and destructive test results. An alternative procedure can be used to obtain a relationship between the in-situ strength f_c and the NDT measurements, based on the following equation:

$$f_c = a \cdot S^b \cdot V^c \tag{2}$$

where the coefficients a, b and c are experimentally derived for the specific concrete under test. The first step is the execution of a non destructive testing program in N_{NDT} points, aimed at verifying the homogeneity of the concrete under examination. In such a way the possible presence of portions of structure representing different concrete batches can be acknowledged. After, in a limited number of points $N_{core} \subset N_{NDT}$, randomly selected between each homogeneous portion, some cores are extracted and after tested in laboratory to evaluate their cylinder compression strength f_{core}. Core test values are then converted into the equivalent in-situ values f_c by using Eq. (1). Finally, a multivariable regression is performed to compute the values of coefficients a, b and c providing the best correlation between destructive and non destructive results, i.e. the Eq. (2) specifically applicable to the concrete under examination. By applying the obtained Eq. (2), the in-situ concrete strength f_c also in points where only non destructive measurements were made can be estimated, thus permitting to determine design strengths in a more representative and reliable way.

5 REVIEW OF EXPERIMENTAL RESULTS

In this section the results of some experimental campaigns are reported and discussed. The main objective is to investigate the variability of the in situ concrete strength for populations of structures and within individual structures. Further, criteria for planning in-situ test programs and analysing their results are discussed.

Firstly, the main results from a wide experimental campaign carried out on the structures of school and hospital buildings in the framework of a seismic vulnerability evaluation program in Basilicata region, Italy, are reported. The buildings under examination were designed and constructed in the period ‘40s – ’90s, taking into account only gravity loads, according to old Italian codes in effect in the period. Working on more than 200 RC buildings, about 3600

NDTs (rebound number, ultrasonic velocity) and more than 800 core tests have been globally carried out on both column and beam members of the structures. The work is currently in progress then in the next years more data will be available.
Analysis of test results on the entire population of buildings show mean values of $f_{c,core}$, as converted in the equivalent in-situ strength f_c through the Eq. (1), equal to 22.8 MPa and relatively high values for the rebound number S (mean value=36) and the ultrasonic velocity V (mean value=3519 m/s). High values of the coefficient of variation (CV) relevant to core strengths (equal to about 46%), while a lower variability of S and V values (CV=17% for both S and V) has been found (table 2 and figure 1).

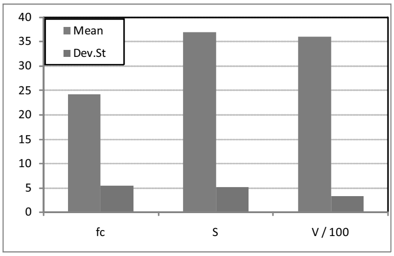

Figure 1.

	f_c (MPa)	S	V (m/s)
N. of tests	846	1647	2009
Mean value	22.8	36	3519
Dev.St	10.6	6	614
CV	46%	17%	17%

Table 2.

Being available the period of design and construction of the buildings in the dataset, test results have been examined separately for 4 periods, whose interval has been identified on the basis of significant modifications in design or construction practice in Italy. The results show that in-situ concrete strength is remarkably dependent on the period of construction. Mean values of f_c are in good accordance with the expected values relevant to the standards of the time of construction (table 3). Dispersion of values is variable with the considered construction period, even though CV values are averagely high, being not lower than about 35% in the periods 1946-1960 and 1982-1991. In figure 2 the frequency distributions of the in-situ strength values have been compared with the mean default values $f_{cm,def}$ assumed for the various construction periods. Generally, the mean value of the measured in-situ strength is greater than the mean default strength.

Table 3. Main statistical values of f_c in various construction periods.

Construction period	$f_{cm,def}$ (MPa)	N. of tests	f_c – Mean value (MPa)	f_c – Dev.St (MPa)	f_c – CV (%)
'46÷'60	12-16	93	16.74	5.67	33.8
'61÷'71	16-20	361	21.47	9.65	44.9
'72÷'81	20-24	261	25.54	12.05	47.2
'82÷'91	24-28	109	25.37	9.08	35.8

As for the dependence of mean strength values from the type of structural element where cores have been extracted, significant differences were not found between columns and beams, as reported in table 4. It is worth noting the large difference in the number of tests available for either columns or beams, demonstrating that cores are typically extracted from columns. Both technical reasons, that is columns are considered more important in determining the seismic resistance, and practical difficulties, particularly in case of embedded beams, give rise to that large difference.

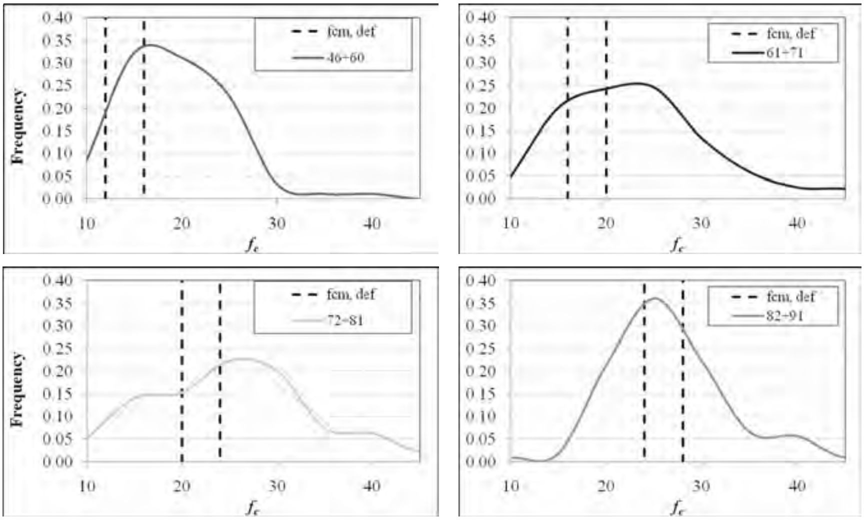

Figure 2. Distribution of in-situ strength values f_c vs mean default values $f_{cm,def}$ [MPa] in various construction periods.

Table 4. Main statistical values of f_c in beam and column elements.

Type of structural element	N. of tests	f_c – Mean value (MPa)	f_c – Dev.St (MPa)	f_c – CV
Beams	78	22.87	10.64	47%
Columns	767	22.21	9.76	44%

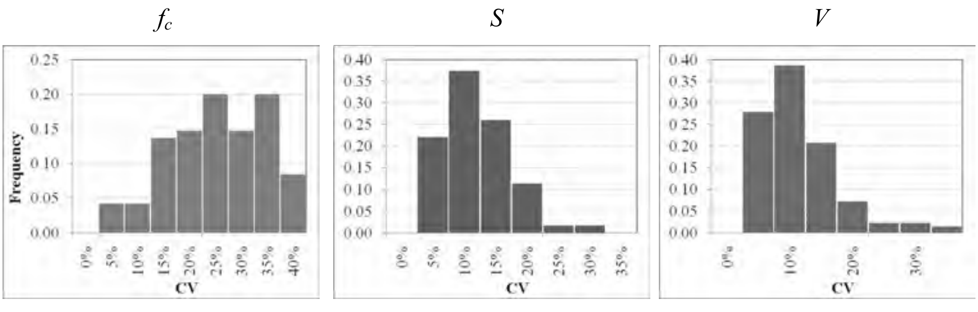

Figure 3. Distribution of CV values for core test and NDT results computed within individual buildings.

In the following, the test results within individual buildings are examined. Analysis of results shows a large scatter of the strength values from cores extracted by the same building, while lower scatters have been detected for the NDT results, as already found in previous investigations (Masi *et al.*, 2008). In fig. 3 the frequency distributions of the CVs calculated with regard to each building are displayed. CV relevant to core strengths is strongly variable

and its more probable values range between 15-35%. As for NDTs, more probable CV values are in the range 5-15% for both rebound and ultrasonic velocity measurements.
Some scatter in the test results is unavoidable, given the inherent randomness of in-place concrete properties and the additional uncertainty attributable to the preparation and testing of the specimens. Scatter of the in-situ concrete properties, and specifically of strength, within a single structure can be caused by some factors, among which: (i) random variation of concrete properties, both within one batch and among batches; (ii) systematic variation of in-situ properties along a member or throughout the structure.
To better understand such issue, the results of an experimental program (carried out at the Laboratory of Testing Materials and Structures of the University of Basilicata) on RC members extracted by existing old structures designed only for gravity loads, are shortly described. More details on the experimental program and obtained results are reported in (Dolce *et al.*, 2006; Masi *et al.*, 2008). Experimental data show a low variability of rebound number and direct velocity values. On the contrary, as a consequence of the microcracking condition due to past applied loads, a high variability was detected for drilled core strengths. Really, cracking due to flexure (perpendicular to the member axis) can affect core strength, provided that cores are usually extracted perpendicularly to the member axis. On the other hand, transverse cracking does not affect the velocity measured by direct transmission (wave direction parallel to the crack plane). Further, the procedure to estimate the in-situ concrete strength based on the Sonreb method (see section 3) has been applied to the experimental data and has been compared with other procedures given in the literature showing its higher efficiency (Masi *et al.*, 2008).

6 POSSIBLE IMPROVEMENTS OF CODE PROVISIONS

Analysis of the experimental results on the in situ concrete strength reported in this paper enables to make some remarks and to suggest some possible improvements of the current code provisions.
Investigation programs have to be planned taking into account a "milestone": core tests are unavoidable but their amount should be limited as much as possible. In fact, core tests are generally more expensive than NDTs and, chiefly, cause damage on structural elements. In some cases such a damage can determine a remarkable reduction of the load bearing capacity of the structures under investigation, immediately after the extraction and also after restoration interventions if badly performed (Masi and Vona, 2009b). For this reason, underlying again that core tests are necessary to directly estimate in-situ concrete strength, the number and location of cores needs to be accurately defined. Keeping in mind such objective, some suggestions can be given in planning and performing investigations relevant to the following steps:

- definition of concrete portions to be separately investigated (concrete areas having homogeneous properties);
- amount of testing (minimum number of samples and measurements);
- location of sampling;
- assignation of concrete strength value for safety evaluations.

<u>Definition of concrete areas having homogeneous properties</u>
Recommended requirements regarding testing of concrete provide minimum sample numbers per floor and per primary element (see table 1). Experimental evidence shows that the variability of the in situ concrete strength cannot be dependent on the specific floor and type

of element. In real buildings one or more concrete areas showing homogeneous properties, that is having sufficiently low values of the CV relevant to strength values, can be defined. These areas can be effectively identified through non destructive measurements carried out along the building floors and elements and, after, core tests can be referred to there areas, as shown in Masi and Vona (2009a). Specifically, homogeneous areas can be identified on the basis of the mean values of NDT results achieved in the various building floors. Tentative values of the number of NDTs to be performed in identifying the homogeneous areas are suggested in Table 5. NDTs need to be adequately distributed among all the primary elements. It is advisable to performing at least 30-40% of total measurements per floor in each different type of structural element.

Table 5. Minimum number of NDTs tentatively suggested in identifying homogeneous concrete areas.

	Per each floor	
Level of testing	Minimum Number of NDTs	Percentage of NDTs (on the total N. of primary elements)
Limited	6	8%
Extended	8	12%
Comprehensive	10	15%

<u>Amount of testing (minimum number of samples and measurements)</u>

Within the homogeneous areas defined on the basis of the NDT results, the amount of cores to be extracted is dependent on the knowledge level to be achieved, as well as on the number and reliability of other additional sources of information. In order to have a minimum number of values to calculate a mean strength value and to make possible the application of the combined Sonreb procedure as explained in section 3, it appears necessary that the at least 3 cores are extracted from each homogeneous area (also for the limited KL). When higher KLs have to be achieved, such minimum number needs to be suitably increased in absolute values as well as in percentages of the total amount of structural elements within each homogeneous area being investigated. Such procedure has been already adopted in several applications, such as that one described in Masi and Vona (2009a), displaying its ability to provide a sufficiently reliable estimation of the concrete strength even though keeping as low as possible the required number of cores.

Table 5. Suggested tentative minimum requirements for different levels of testing.

	For each homogeneous area	
Level of testing	Number of cores	Percentage of cores (on the total N. of primary elements)
Limited	3	5%
Extended	5	8%
Comprehensive	8	12%

Tentative values are suggested in Table 5, however a more reliable definition of number and percentages of samples to be taken requires further experimental studies aimed at achieving more general results. Also cores need to be adequately distributed among all the primary elements, drawing at least 25% of samples per homogeneous area from each different type of structural element.

Location of sampling

Experimental experience (e.g. Dolce *et al.*, 2006) shows that measurement points, need to be carefully located within the structural members. They have to be placed in zones without apparent damage and/or cracking, where stresses due to applied loads are absent or the lowest ones, and representative of the average conditions of the concrete taking into account casting-in-place and ageing effects. In the beams that were subjected only to gravity loads during their service life, the best points are located in the lower part of the member ends, provided that in the central upper part the presence of the adjacent slab usually does not permit drilling. In the columns, taking into account that the static pressure due to consolidation after placement makes strength variable along the member height, the technical literature suggests that the best points are placed at member mid-height.

Assignation of concrete strength value for safety evaluations

Firstly, results from core tests need to be adequately corrected taking into account the main differences between the strength measured on core specimens and the actual in-situ strength. To this purpose, Eq. (1) reported at section 3 can be used. Further, the procedure based on both destructive and non-destructive measurements, also reported at section 3, can be applied. It requires that the relationship between the in-situ concrete strength and the NDT measurements is experimentally derived for the specific concrete under test. By applying this relationship, the in-situ concrete strength also in points where only non destructive measurements were made can be estimated, thus permitting to estimate the design values of concrete strength in a more reliable way.

7 FINAL REMARKS AND FUTURE DEVELOPMENTS

Investigations have a crucial role to adequately know the structures to be subjected to evaluation and possible retrofitting. For this reason, there is an increasing need to set up and put at disposal of technicians and other involved stakeholders sufficiently reliable as well as not very expensive methods to estimate in-situ material properties. Number of tests required to suitably apply these methods have to be as low as possible, thus making the total required budget sustainable to building owners and, consequently, further encouraging their use. To this regard, the results presented in the paper confirm that a suitable combination of NDTs and core tests provides an effective solution from both the economical and technical point of view.

Based on the experience deriving from widespread in-situ and laboratory experimental investigations, some directions are drawn able to suggest some possible improvements of the current code provisions. Particularly, a procedure to develop the investigation plan and, subsequently, estimate in-situ concrete strength, alternative to that one in current codes, is provided. It is made up of the following main steps: (i) definition of concrete portions to be separately investigated (concrete areas having homogeneous properties), (ii) amount of testing (minimum number of samples and measurements), (iii) location of sampling, and (iv) assignation of concrete strength value for safety evaluations. Tentative values of number and percentages of tests (NDTs and cores) to be performed are suggested, however further studies are required to achieve a more general and reliable definition.

To this purpose, future research work has to be devoted to provide methods more and more able to achieve effective results in terms of prediction capability of concrete properties taking into account both aleatory and epistemic uncertainty. Further, NDTs currently available on concrete do not provoke damage on structural members but on some other building

components (e.g. partitions, infills, plaster, etc.) thus determining remarkable repair costs: new methods are necessary to make them really not very expensive.
Greater attention needs to be devoted also to the estimation of the properties of reinforcing bars. Mechanical properties of reinforcement, being it an industrial product, have a very small variability compared to those ones of concrete. Therefore, a different number of tests should be required to attain a certain knowledge level, conversely to the current EC8-3 and IC provisions, that also appear excessively onerous as far as reinforcement is concerned. In the same way, revisiting the significance of the confidence factor, different values for concrete and reinforcement steel could be suggested. Finally, taking into account the heavy damage caused by the extraction of steel bars, non destructive methods to estimate its mechanical properties need to be set up.

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ELASTIC PERIOD OF EXISTING RC-MRF BUILDINGS

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ABSTRACT

The fundamental period structures has a primary role in the seismic design and assessment and design as it is the main feature of the structure allowing to determine, at least, the elastic demand and is the basis to assess the required inelastic performance in static procedures. In fact, the definition of easy to manage relationships for the assessment of the elastic period has been the subject of a significant deal of research of both experimental and numerical/analytical studies, some of which has been acknowledged by codes and guidelines worldwide. Moreover, this kind of information is useful for territorial-scale seismic loss assessment methodologies. In the most of the cases the assessment of the period is considered as function of the structural system classification and number of storeys.

Reinforced concrete (RC) buildings constituting the most of the building stock in Italy and in seismic prone areas in Europe, were built after the second world war and are designed with obsolete seismic codes if not for gravity loads only. Therefore, a significant variability of the structural system may affect a class of buildings featuring the same height and/or number of storeys. This, along the contribution of the stair module, may affect the elastic periods in the two main directions of a three-dimensional building.

In the study presented these issues are investigated with reference to a population of existing RC structures designed via the practice at the time of supposed construction (e.g., simulated design) and with reference to relative enforced code. Elastic period is evaluated for both main directions of the buildings of the considered sample, and regression analysis is employed to capture the dependency of the elastic dynamic properties of the structures as a function of mass and stiffness.

KEYWORDS

Elastic period, sub-standard, RC buildings, structural system.

1 INTRODUCTION

In static procedures for seismic structural assessment the fundamental period is one of global characteristics to determine effects of seismic action in terms of horizontal forces. In the dynamic procedures (e.g., nonlinear) it is necessary to select the appropriate hazard information and input ground motions, especially for first-mode dominated structures. Generally speaking, the period relates seismic demand to capacity allowing to determine the seismic performance and therefore the safety level.

The most of the seismic codes worldwide propose to easy to apply relationships to determine the elastic period as a function of height or number of storeys given the structural typology (SEAOC, 1998, Eurocode 8, 2004). Such relationships, especially those for moment-resisting RC frames, have been calibrated on experimental studies, which have become a standard

reference at an international level (ATC, 1978, Goel and Chopra, 1997). These important studies are based on seismic monitoring of buildings subjected to seismic actions, eventually repeated because of more earthquakes hit the structure, in significantly seismic prone areas in which earthquake resistant design is well established since long time.
Recently, research effort was devoted to the attempt of develop similar relationships, for European "typical" frames and not buildings, for the so-called effective- or yield- period which is that of interest for determining the non-linear demand in those cases where the capacity derives from static push-over analysis (Crowley and Pinho, 2004). This period, which is longer than the elastic has more to do with the yielding and/or cracked stiffness of the structure and is more easily assessed via analytical/numerical procedures, although results strictly depend on the structural modeling assumptions.
It is well known as the existing RC buildings in Italy, significant portion of the building stock, have been designed and erected mainly after the second world (e.g., in the 1940-1980 period) war when only a fraction of the territory was considered as seismically prone. Design was carried out by for gravity loads only, and also when consideration of seismic action was required, it resulted in the application of period-independent horizontal forces without regard to capacity design which is the fundamental of earthquake resistant design nowadays.
A class of gravity-load designed buildings may feature a structural system which may be heterogeneous as the plan distribution of resisting frames may not follow the regularity principle established in the seismic case. This is also useful to point out that may be not appropriate, when analyzing these building to refer to frames as the two main directions may have dissimilar dynamic properties reflecting the variability of the structural system. Moreover, the stair-module cannot also be neglected when investigating this topic. Similarly, the "seismic" buildings of the time-span given, although presenting a more rational structural system in respect to seismic actions, are expected to not show the stiffness and regularity features of a modern earthquake resistant building.
The study presented in this paper investigated these issues for classes of existing RC buildings and how they reflect on the elastic properties. In particular, the variability of the elastic period in the two directions is assessed analytically in respect to the variability of some parameters of characterizing the structural configuration. To this aim rectangular buildings with number of storeys between 2 and 8 have been considered, which are very common in Italy. The building are bare-frames, according to the codes, for which the stair is also considered.
As the study refer to a numerical analysis of a class or population of buildings those had to be specifically designed. The simulated design was carried out via an automatic procedure (Verderame et al., 2008) which implements the design rules and professional practice at the time when the building are supposed to date. In particular, in the study refers to the Italian design principles, which represent the European and Mediterranean practice (Carvalho et al., 1999; Bal et al., 2007, Verderame et al., 2009).
Two different populations of buildings have been considered: (*i*) gravity-loads only; and (*ii*) sub-standard seismic design accounting for seismic action via statically-equivalent horizontal forces. From these two populations have been split in 14 classes of buildings with fixed number of storeys. The periods in the two main directions have been regressed versus the height, as suggested by codes and existing literature on the topic, to compare and to see how much of the variability is captured by the independent variable. Subsequently, other covariates which may explain the variability of mass and stiffness, related to the global dimensions of the building, have been included to better assess the influence on the elastic period of the design practice and structural peculiarities.
In the following, after a state-of-the-art review, of simplified relationships for elastic period estimation for RC buildings, the analytical procedure considered is described along with the

population of building considered as a sample for the analyses. Finally, results are presented and discussed with the aim of assessing the elastic periods and explaining its variability in respect to existing and code approaches.

2 RESEARCH BACKGROUND

Period is depending on those factors directly affecting structural mass and stiffness. Globally, proxies for the mass may be building's global dimensions (e.g., plan area and number of floors), stiffness may be related with structural features and height.

The most of the relationships to estimate the period are a function of global height (H) as it is a simple parameter, known before detailed design, which may explain the ratio between stiffness and mass of the building. Formulation of period-height relationships is typically of the type in Eq. (1) where α is depending on the structural system

$$T = \alpha H^{\beta} \tag{1}$$

It appeared in ATC3-06 (ATC, 1978) first with β equal to 0.75, while α was calibrated as 0.06 (if H in measured in meters or 0.025 if it is in feet), based on periods measured on some buildings during the 1971 San Fernando earthquake.

A similar relationship may be computed via the Rayleigh method (Chopra, 1995) with the following seismic design assumption: (*i*) horizontal forces linearly distributed along the height of the building; (*ii*) mass distribution constant along the height, (*iii*) linear deformed shape; (*iv*) base shear proportional to $1/T^{\gamma}$. If these conditions are met the period is expressed as:

$$T = \alpha H^{1/(2-\gamma)} \tag{2}$$

If γ equals 2/3, as established in US codes (UBC, 1997):

$$T = \alpha H^{0.75} \tag{3}$$

In SEAOC-88 commentary (SEAOC, 1998) α is 0.073 (if H in measured in meters or 0.030 if it is in feet). The formulation with these values of the parameters was adopted by International codes as Eurocode 8 (EC8, 2004) rounding α to 0.075.

Alternatively, NEHRP-94 (1994) includes a relationship as a function of the number of storeys (N), T = 0.1N, limited to buildings up to 12 storyes with inter-storey height not smaller than 3m. This relationship was frequently adopted by codes before Eq. (1).

More recently, calibration of coefficients is based on experimental data; e.g. the monitoring of buildings during earthquakes. Goel and Chopra (1997), collected data on 37 reinforced concrete buildings, featuring seismic design and with height ranging from 10m to 100m. For each of the building the periods in the two principal directions were measured; in particular, the periods in the two directions are very similar, showing an average 10% difference, as shown in Figure 1. This may most likely be attributed to earthquake resistant design of the buildings, which should give uniform lateral stiffness in both directions.

Note that this kind of approach renders, of course, the period estimation depending on the history of the shaking at the site for each structure for two reasons: (1) if the shaking is strong enough to crack the structure the period measured is longer than if the structure remains uncracked; (2) if the buildings are subjected to multiple earthquakes the period measured after

the first cracking ground motion is always related to cracked stiffness also in subsequent lower intensity shaking.
The buildings of that study were subjected to 8 main Californian earthquakes from San Fernando (1971) to Northridge (1994). According to the authors, 22 buildings experienced a peak ground acceleration (PGA) lower than 0.15g, while the others were subjected to larger acceleration at the base. As expected, the latter buildings show a larger period given height.

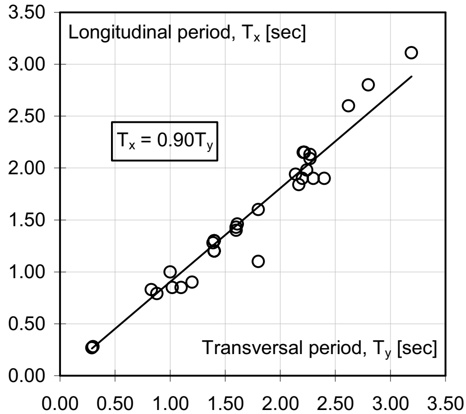


Figure 1. Correlation of elastic periods measured in the two main directions of the buildings of the study from Goel and Chopra (1997).

Comparing experimental results to those deriving from Eq. (3) a underestimation of the period is observed, especially for larger height buildings and for those which experienced a PGA larger than 0.15g. Therefore alternative formulas were proposed resulting from a semi-empirical analysis. One, Eq. (4), features the best fit coefficient for Eq. (1); Eq. (5), is that fitting data plus one standard deviation; and Eq. (6) is that conservatively works at minus one standard deviation and, therefore, is proposed for estimation of the period in seismic design.

$$T = 0.052H^{0.9} \tag{4}$$

$$T=0.065H^{0.9} \tag{5}$$

$$T = 0.044H^{0.9} \tag{6}$$

A similar study was carried out by Hong e Hwang (2000) for 21 RC seismic buildings in Taiwan subjected to 4 events claimed to not yield the structures. The coefficients proposed in that study lead to the following expression:

$$T = 0.029H^{0.804} \tag{7}$$

Comparing semi-empirical relationships significant differences in estimations are found. In Figure 2 such comparison is given, and it may be noted how the estimation according to Eq. (4) leads to an 130% average overestimation in respect to Eq. (7). Such a difference may be related with the different design criteria and construction practice in the two countries. Closer agreement is found between code-suggested relationships (Eq. (3), Eq. (6) and that function of

the number of storeys) for H ≤ 40m, which is the range of interest for the building stock in southern Europe (e.g., Italy).
From the comparison it may be argued that the calibration for Eq. (1) via a numerical or experimental approaches is conditioned on assumptions on the dynamic response and on peculiarities of seismic design. It is therefore to investigate whether period-height relationships for sub-standard seismic design or gravity loads design buildings could lead to different estimation in respect that of codes and/or existing literature. These may be common conditions for existing buildings and, therefore, it is the focus of the following analyses.

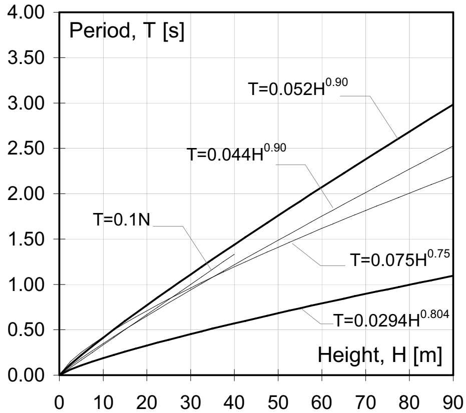


Figure 2. Literature period-height relationships

3 SUB-STANDARD RC BUILDINGS

Existing RC buildings do not reflect regularity of strength and stiffness which a building conceived with capacity-design principles shows. In fact, most of them may be considered sub-standard being designed for gravity loads only in areas subsequently considered seismically hazardous or designed with inadequate seismic provisions. Herein samples from both categories are considered, and is to underline that may be significant structural differences even among them. Design for gravity loads does not require regularity of the structural system in the plan view and therefore these buildings show disuniform distribution of the resisting substructures which may lead to different elastic responses in the two main directions; in other word, the number and orientation of frames is determined by elements carrying gravity loads. Building designed also accounting for horizontal forces show a more distribution of frames because the action was assumed equal (and period-independent) in the two directions. This lead to a first three-dimensional conception of the structure with frames specifically devoted to resist seismic actions.
Within this framework load design was carried out with approximate methods. In the gravity-load design simplified schemes instead of plane frames were used. In fact, columns were proportioned based on the axial load only, while the beams' design reflect the continuous multiple-supports scheme.
On the other hand, seismic load effects are evacuate via frame modeling, although the distribution of seismic actions to elements is still approximated being based on the floor masses' distribution or on the columns' inertia, the latter known as shear-type model. Seismic actions during the considered time span was determined imposing horizontal forces

depending on fixed acceleration levels: 0.10g or 0.05g (increased to 0.07g later on) depending on the assumed seismic potential of the site; in other words, period-independent seismic forces. Design criteria did not consider limit-states expressed in terms of maximum inter-storey drift ratios as modern codes, therefore, the storey-stiffness indirectly depends on the lateral load resistance which should be provided as a consequence of the horizontal forces above.

Therefore the following structural features are expected for this kind of buildings:

- existing buildings, independently of gravity loads or seismic design, feature a lower lateral stiffness in respect to modern code-designed structures;
- among existing buildings, those provided of seismic design have a larger global stiffness in respect to those designed for gravity-loads only as horizontal forces should imply larger dimensions of elements constituting the seismic resisting systems (i.e., frames);
- the eventual variability of structural system in gravity load design may lead to significant differences in the natural period in the two principal directions of these kind of buildings.

These issues are investigated in the following referring to a widely diffused typology of existing buildings in Italy built in the year approximately between 1950 and 1975.

4 METHODOLOGY

To investigate the elastic dynamic features of the buildings describe above as a function of various types of mechanical parameters was based on the simulated design of a population of buildings which is made of the following phases (Verderame et al., 2008):

- *building definition*. Depends only on the 3D dimension of the building;
- *identification of possible structural system for the building*. At this stage the structural parameters number of frames, bay lengths, column orientation are defined;
- *simulated design of the structural systems* in terms of cross sections' dimensions and reinforcements both longitudinal and transversal of the elements;

In the case of gravity loads design the number of frames is determined by the distribution of vertical forces only, while in the case of seismic design additional frames derive from the lateral forces. The design of the elements is carried out referring to the recommendations of the codes enforced at the time in terms of reinforcement rations, material design strengths and so on. Note that, steps (ii) and (iii) lead to multiple structures being associated to a single buildings as several structural systems and design alternatives correspond to the same global dimensions.

4.1 Considered buildings' population

Considered buildings refer to a rectangular plan shape and a moderate number of storey. In this type of buildings typically there are one or two units for each floor and one a stairway which is assumed centered in respect to the longitudinal direction of the building (the longer).

In generating the population of analyzed buildings the considered variable parameters are its longitudinal length (L_x), transversal length (L_y)and the global height (H) excluding the foundations. Interstorey height is constant and equal to 3.0m.

Structural configuration adopted in simulation refers to structural designed for gravity loads only subsequently adjusted (adding frames) to also withstand horizontal actions. Its principal feature is that frames, to support the slabs, have all the same direction, which is typically the transversal one of the building, this lead to call this resisting system as *parallel plane frames*.

In the transversal direction, then, only two frames at the two ends of the building and the stair module exist. In Figure 3a this scheme is illustrated. On the other hand the structural configuration in the case of seismic design corresponds with an integration of the former with multiple frames in the same direction, one per bay (Figure 3b).
For this kinds of buildings the bay lengths have been assumed to be comprised in the 3.0-5.0m; this lead to the variability of the structural configuration among the population.
Gravity loads design was carried out according to the design values of dead and live loads of the codes in the time-span considered. As discussed design of elements refers to simple sub-structuring schemes in which the design of columns is driven by axial load and design of beams refers to a multiple supports model.
Seismic design was carried out by static linear analysis, the most common tool at the time (and still widely used today in the practice). Three accelerations are considered for seismic design equal to 0.10g, 0.07g e 0.05g according to the evolution of seismic classification of the territory in Italy up to 1975. The seismic forces distribution considered is that proportional to the storey masses (R.D.L. 640, 1935; R.D.L. 2105, 1937; Legge 1684, 1962). The statical location of horizontal forces refers to the flexible-slab assumption; in the transversal direction the contribution of the stair is not considered.
The ranges of variability of building dimensions considered is that as follows:

- longitudinal length L_x = [15.0, 20.0, 25.0, 30.0]m;
- transversal length L_y = [8.0, 10.0, 12.0]m;
- building height (H) is comprised between (6.0÷24.0)m corresponding to 2- 8 storey

All possible combinations of these values lead to 84 buildings for each of the two possible design categories (seismic and gravity loads); while the structural configuration variability and lead to 175 structural systems, two per building on average.
The allowable stress design leads to proportions of the elements which may be defined as minimal in respect to the actions induced by gravity and seismic loads. Therefore the lateral stiffness is generally to be considered minimal in the two directions.
Considering also the 4 design options (one gravity load and three seismic design levels) a population of 700 structures was analyzed. The cylindrical compressive mean strength of concrete f_c is constant among the structures and equal to 15 MPa.
In particular two linear analyses have been carried out for each structure to investigate the elastic period in the two principal directions of the buildings to which the structure considered correspond to.

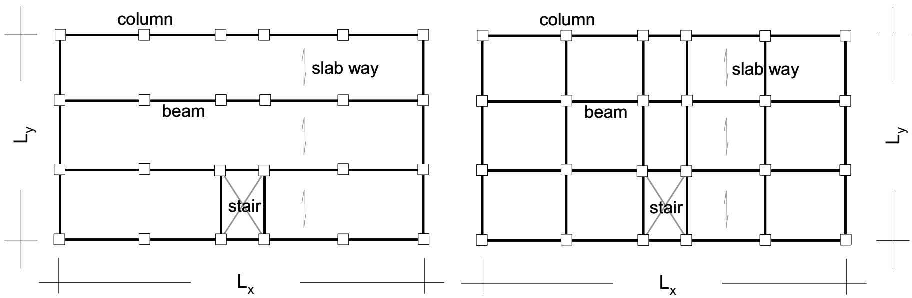


Figure 3. Structural configurations of the considered building types: gravity-loads designed (a) and seismic design (b).

5 IMPLICATIONS OF STRUCTURAL FEATURES ELASTIC PROPERTIES

The elastic characteristic of the different directions of each considered structures are evaluated via the classical eigenvalues analysis, i.e., Eq. 8:

$$([K]-\omega^2[M])\{\phi\}=\{0\} \tag{8}$$

where [K] is the stiffness matrix of the MDOF structural system, [M] is the seismic storey masses matrix, $\{\phi\}$ is the displacement vector of vibration mode, and ω is the associated circular frequency. [K] is determined starting from the cross section stiffness (E_cI), which has been evaluated referring to the inertia (I) of the uncracked section and the elastic modulus of concrete, E_c, defined as in Eq.9 in which f_c is the cylindrical compressive strength of concrete expressed in MPa (EC2, 2004).

$$E_c = 22000(f_c/10)^{0.3} \text{ (MPa)} \tag{9}$$

The storey masses in the diagonal [M] matrix have been evaluated according to Eurocode 8 (EC8, 2004) based on the analysis of dead and live loads for the structure for which the elastic periods are evaluated.
From Eq.8 the ω_i, $\{\phi\}_i$ and m_i^*, associated to the i-th mode defined as the *effective mass* and evaluated as:

$$m_i^* = \sum m_k \phi_{k,i} \tag{10}$$

where m_k is the seismic mass of the k-th storey, $\phi_{k,i}$ is the displacement of the k-th floor in the i-th mode.
The periods considered are those corresponding only to the fundamental modes, primarily translational, associated to the principal directions of the structure. Therefore, for each of the two directions the fundamental $\{\phi\}$, ω, and m^* are determined and the corresponding elastic period, T_{el}, defined as:

$$T_{el} = \frac{2\pi}{\omega} \tag{11}$$

Figure 4 shows the trends of the periods as a function of the height for the four categories of buildings investigated. As expected the design options affect the elastic periods. The periods corresponding to gravity load design are generally smaller than those proportioned to resist also to horizontal force, among which to a higher reference acceleration correspond lower natural periods. Moreover the longitudinal direction is generally stiffer than the transverse, although the transverse to longitudinal periods ratio is larger in the case of design for gravity loads in respect to seismic.

5.1 Gravity loads designed buildings

In these structures the period in the short direction is more variable in respect to that longitudinal given height. This depends on the peculiar structural configuration, Figure 3a, which significantly affects, the ratio between *effective mass* m^* and *lateral stiffness*, K_{el}, of the building. In fact, in Figure 5, the trends of these quantities are given as a function of the height for the two directions.

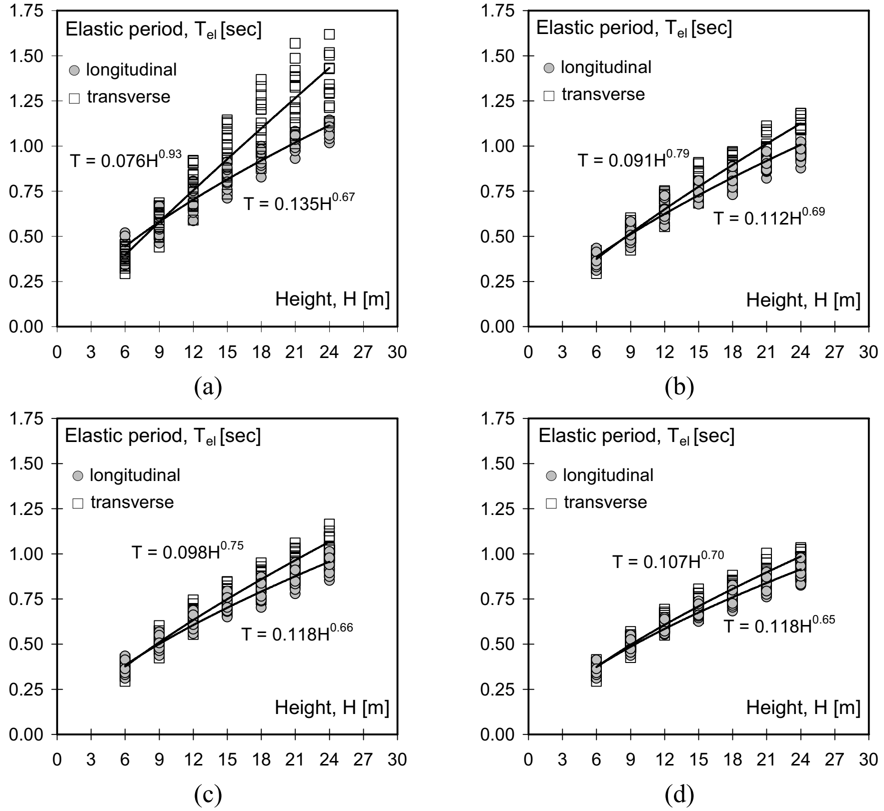


Figure 4. Period-height relationships for the analyzed populations: (a) gravity-loads design (b) seismic design (0.05g) (c) seismic design (0.07g) (d) seismic design (0.10g).

The effective mass varies almost linearly with the height, and it is variable in both directions because of the ranges of global dimensions, L_x and L_y, considered which lead to a wide range of plan areas for the populations analyzed, as it can be observed in Figure 6a . In general, the buildings feature effective masses similar in the two directions, therefore the differences in elastic periods depend on the different lateral stiffness.

As observed in Figure 5, the long direction has lateral stiffness larger than that in the short direction, this discrepancy is increasing with height and is up to 50% for buildings with more than 3 storeys. Vice versa, the longitudinal direction has a larger variability of the stiffness in respect to the transversal one. This has to be attributed to the different lateral resisting system of the two directions (Figure 3a).

The stair sub-structure significantly affects the stiffness in the short direction. Despite this direction has only the two perimetral frames, this is magnified reflected for buildings with 3 storey or less (H equal or less than 9m), for which the transverse stiffness ins larger than the longitudinal. The stair module effect, rapidly decreases with height.

The variability of the area covered by the buildings is differently reflected in the elastic properties of the two directions (Figure 6b). In the longitudinal direction the increase of L_x implies an increase in the number of bays, while an increase in L_y lead to increase the number of longitudinal frames. Therefore, the longitudinal direction has an increasing stiffness with

area. Vice versa, the resisting system in the transversal direction is only marginally affected by an increase in the area. In fact, it does not imply modification in the stair module and the slight trend observed is due to the variation of number of bays or of the proportions of the elements of the two perimetral frames.
It may be concluded, that for gravity load buildings, the stair module plays a determinant role in the transversal period determination. The lower stiffness in the short direction in respect to the longer and its comparatively small variability given height clarify the trend of the period reported in Figure 4a.

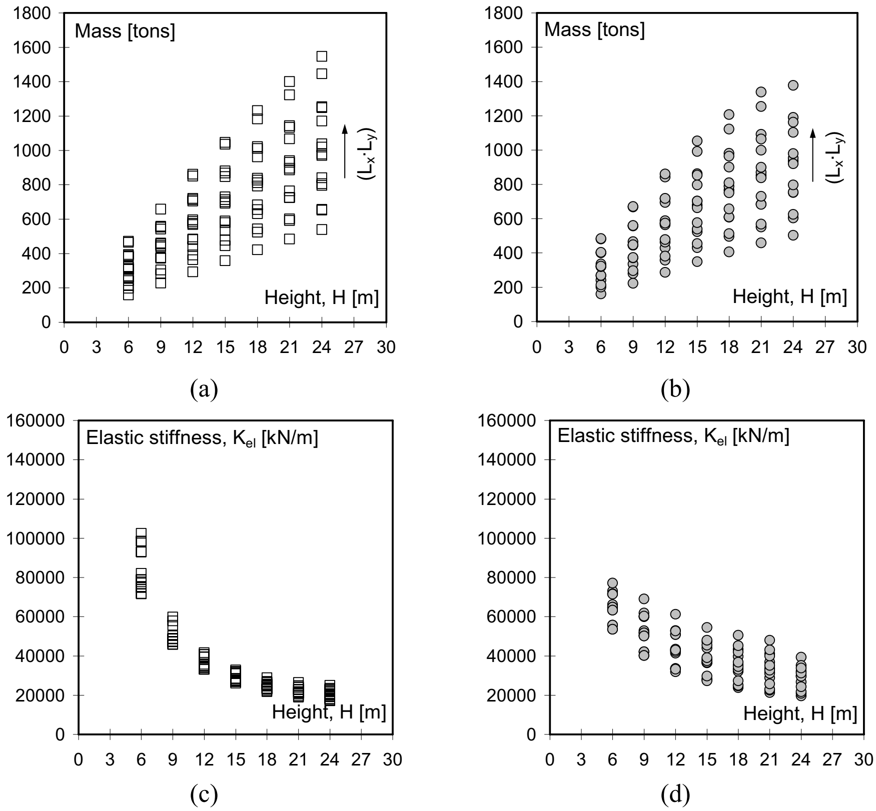
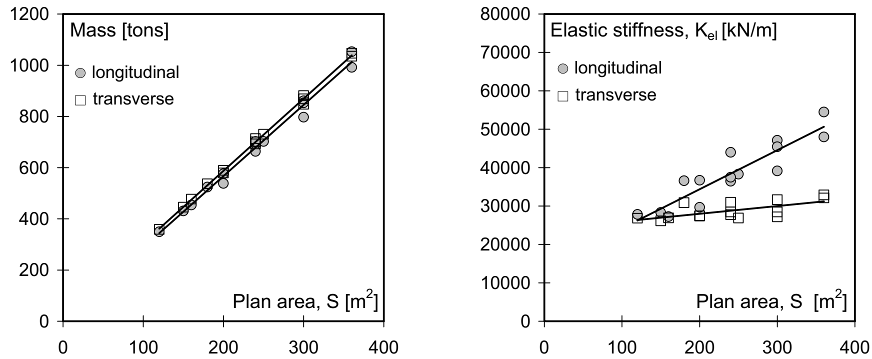


Figure 5. Gravity-loads design: (a) transverse elastic stiffness versus height; (b) longitudinal elastic stiffness versus height; (c) transverse effective mass versus height; (d) longitudinal effective mass versus height.

5.2 Seismic buildings

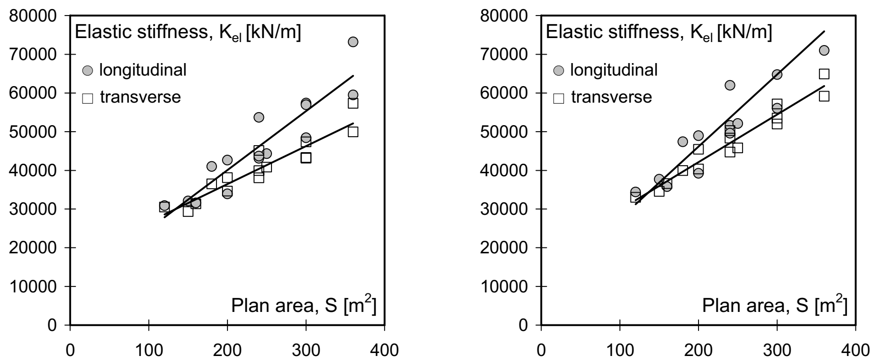
Seismic building have a defined resisting system also in the transversal direction and are also characterized by a more uniform distribution of structural sub-systems, e.g., Figure 3b. This reflect in different trends of stiffness in respect to gravity loads designed buildings.
The stiffness in the two direction is similar. In fact the stiffness in the longitudinal direction is never larger more than 20% with respect to that transversal. In particular, 2 storey buildings have a stiffness constant, on average, in respect to the design acceleration. The moderate height leads to actions that can be taken by the minima proportions of structural elements.



(a) (b)

Figure 6. Effective mass (a) and elastic stiffness (b) versus area for a 15.0m buildings designed for gravity loads only.

Therefore, the structures result generally similar independent of the design acceleration. Moreover, for this kind of buildings the stiffness reduces with global height in both directions and variability is also comparable. The more uniform distribution of resisting systems in respect to gravity-load designed also reflects in a different trend of stiffness as a function of plan area which now affects also the transversal direction. In Figure 7a and 7b K_{el} is given for the two directions of buildings with H equal to 15.0m and designed for 0.05g and 0.10g respectively.



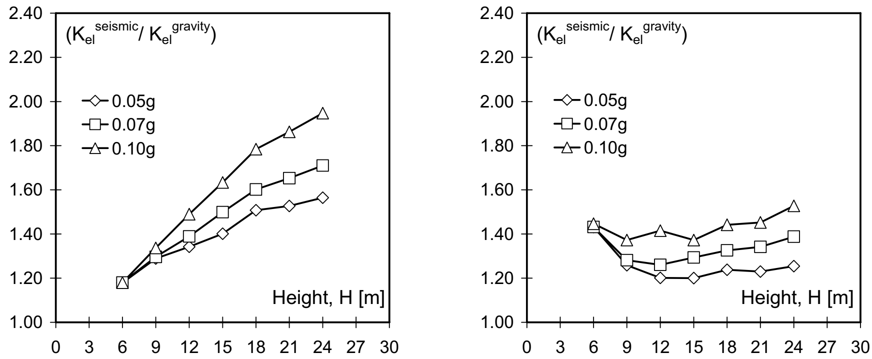
(a) (b)

Figure 7. Seismic buildings: elastic stiffness versus area for a 15m building designed with acceleration equal to 0.05g (a) and 0.10g (b).

Seismic designed buildings are generally stiffer that those designed for gravity loads only. In Figure 8 the ratio of the average seismic to gravity loads stiffness is given for the two directions. This ratio is increasing in the transversal direction while it is almost constant in the longitudinal. This is mainly because the different structural systems is shown main in the former direction rather than in the latter. The plane frames added in the seismic design contributes to the stiffness in an increasing manner with respect to the height of the building.

In the longitudinal direction, an increase in the design accelerations also implies a increment in the stiffness leading, on average, to a ratio in respect to the gravity-loads design case of

1.25, 1.35, and 1.45 for 0.05g, 0.07g and 0.10g. In the transverse direction, 20% is the minimum stiffness increment, a number found for 2 storey buildings, while 95% is the scored by 8 storey buildings, corresponding to the maximum global height considered in the study, and designed for 0.10g.
These results reflects in the trends of Figure 4b, 4c, 4d.



(a) (b)

Figure 8. Average seismic to gravity elastic stiffness ratio in transverse (a) and longitudinal (b) direction

6 PERIOD PREDICTORS FROM REGRESSION ANALYSIS

Although, not an experimental sample, on the analyzed populations simple regression analysis allowed to see how results in terms of fundamental period display in respect to height. For comparative purposes, the same power-law formulation of Eq. (1) was assumed and the coefficients estimated via ordinary least square regression.
For the case of gravity load design the relationships for transverse and longitudinal directions are respectively:

$$T_{el} = 0.076H^{0.93} \qquad T_{el} = 0.135H^{0.67} \tag{12}$$

Moreover, for the transverse direction, the same relationship was retrieved also not considering the contribution of stair sub-structure, Eq. (13).

$$T_{el} = 0.105H^{0.94} \tag{13}$$

Analogous relationships were determined for the three populations of seismic buildings:
design acceleration 0.05g

$$T_{el} = 0.091H^{0.79} \qquad T_{el} = 0.112H^{0.69} \tag{14}$$

design acceleration 0.07g

$$T_{el} = 0.098H^{0.75} \qquad T_{el} = 0.118H^{0.66} \tag{15}$$

design acceleration 0.10g

$$T_{el} = 0.107H^{0.70} \qquad T_{el} = 0.118H^{0.65} \tag{16}$$

Figure 9 serves to compare the relationships found for the two considered directions. The trends generally reflect what observed for stiffness. The largest periods is observed for gravity-loads designed buildings while it decreases if the design acceleration is increased. In the longitudinal direction the period of seismic buildings is lower by 10 to 20% in respect to gravity loads. In the transversal direction, the reduction in period or seismic buildings may be as large as 45% for 8 storey buildings. Comparing the period-height relationship including and not-including the stair module for gravity load design buildings, the contribution of the sub-structure may be appreciated as the stir leas to a reduction of the latter as large as 40%.
Finally, only as a reference the EC8 (EC8, 2004) period-height curve is also given in the figure. The EC8 period is systematically lower than what found because it is a lower bound itself (Goel and Chopra, 1997) and also because it is expected to refer to buildings featuring a different design philosophy.

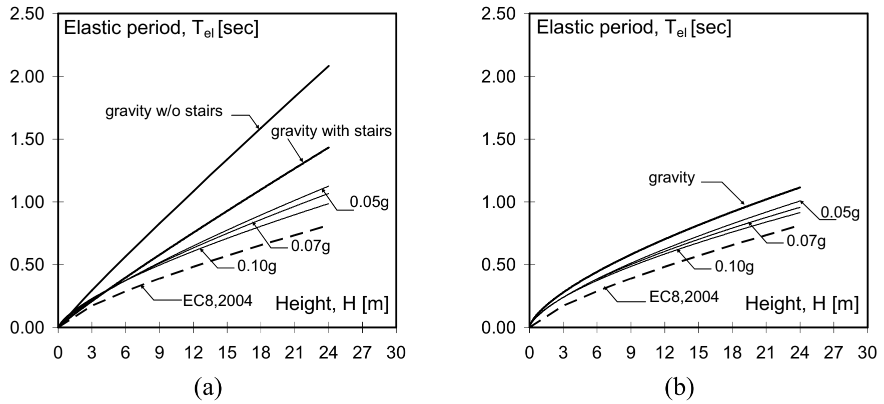


Figure 9. Comparison of period-height relationships for the buildings analyzed in the transverse (a) and longitudinal (b) directions.

Because Figure 6 and 7, show that the effective mass and translational stiffness is also correlated with the plan extension of the building is expected that this variable has some prediction power in respect to the period. Therefore, an expression which includes also the plan area is considered Eq. (17):

$$T = \alpha H^{\beta} S^{\gamma} \tag{17}$$

where S is the product of the two principal plan dimensions of the building L_x and L_y. Least squares regression lead to the relationships for the transversal and longitudinal directions respectively given in Eq. (18):

$$T_{el} = 0.009H^{0.93}S^{0.39} \qquad T_{el} = 0.044H^{0.67}S^{0.21} \tag{18}$$

the same kind of analysis for the seismic directions provides:
design acceleration 0.05g

$$T_{el} = 0.029H^{0.79}S^{0.21} \qquad T_{el} = 0.059H^{0.69}S^{0.14} \tag{19}$$

design acceleration 0.07g

$$T_{el} = 0.033H^{0.75}S^{0.20} \qquad T_{el} = 0.062H^{0.66}S^{0.12} \tag{20}$$

design acceleration 0.10g

$$T_{el} = 0.039H^{0.70}S^{0.19} \qquad T_{el} = 0.068H^{0.65}S^{0.10} \tag{21}$$

How much S contributes to explain the period is assessed simply by analyzing the standard error for the regressions' residuals, σ_T :

$$\sigma_T = \sqrt{\frac{\sum (\log T - \log T_i)^2}{n-1}} \tag{22}$$

In Eq. (22) T is the period from the regression model for the building having the computed value T_i and n is the size of the sample. In Table 1 the values of σ_T for all cases analyzed are reported, for the two conditions of including S along to H or not. Results lead to conclude that only for gravity load design S add information on the period as for this building typology adding S reduces the standard lead to a 60% reduction of the standard error in respect to formulation which only account for the period height.

Table 1. Standard error for the regressions' residuals.

Design type	direction	standard error, σ_T	
		$T=\alpha H^{\beta}$	$T=\alpha H^{\beta}S^{\gamma}$
Gravity	transverse	0.131	0.051
	longitudinal	0.078	0.045
Seismic 0.05g	transverse	0.086	0.055
	longitudinal	0.072	0.058
Seismic 0.07g	transverse	0.082	0.051
	longitudinal	0.066	0.055
Seismic 0.10g	transverse	0.073	0.044
	longitudinal	0.059	0.050

7 CONCLUSIONS

The elastic period has a primary role in the seismic assessment of buildings. Main codes propose simplified equations, retrieved on semi-empirical basis, expressing the fundamental period as a function of height, which represents the relationship between mass and stiffness of the structure. Nevertheless, the most of these relationships are based on data of buildings reflecting seismic design criteria very different from those of the European existing structures. In the study presented two populations of reinforced concrete buildings have been investigated: the first one being designed for gravity load only, the second one designed with obsolete seismic design criteria. Modal analyses allowed to assess the influence of design

criteria, structural system and global dimensions (area and height) on the elastic stiffness, the effective mass, and the elastic period for both principal axes of the building.
Results are different for the two classes:

- Gravity load design buildings feature periods in the two directions which have an increasing difference with height and as large as 50%. The period shows large variability in the short direction due to the variability of plan area.
- Seismic design buildings show a lower period in the longitudinal direction with respect to the corresponding gravity load buildings; this reduction, obviously, increases with design acceleration and is up to 20%. In the short direction the reduction of fundamental period is more significant (50%) because it is due not only to different design criteria, but also (mainly) to the different structural system.

Finally, based on the results of the analyses a power-law regression was carried out as a function of height. In the comparison with EC8 formulas existing buildings show systematically larger periods. In particular, gravity load designed buildings, featuring a 3D structural system, seem to require a twofold definition of period referring to the two directions. Therefore, height alone is inadeguate to explain period variability. In fact, also a global parameter (e.g., plan area) should be added in simplified relationships for rapid period evaluation.

8 ACKNOWLEDGEMENTS

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REVISITING EUROCODE 8 FORMULAE FOR PERIODS OF VIBRATION AND THEIR EMPLOYMENT IN LINEAR SEISMIC ANALYSIS

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ABSTRACT

The period of vibration is a fundamental parameter in the force-based design of structures as this parameter defines the spectral acceleration and thus the base shear force to which the building should be designed. This paper takes a critical look at the way in which seismic design codes around the world have allowed the designer to estimate the period of vibration for use in both linear static and dynamic analysis. Based on this review, some preliminary suggestions are made for updating the clauses related to the estimation of the periods of vibration in Eurocode 8.

KEYWORDS

Eurocode 8, period of vibration, RC frames, linear analysis.

1 INTRODUCTION

Provisions for the linear static and dynamic design of reinforced concrete buildings are included in almost all seismic design codes around the world. Although the names of these procedures vary from code to code, the basic principles are the same. The linear static (or lateral force) method allows engineers to predict the fundamental period of vibration in a simplified manner and calculate the design base shear force from the response spectrum (see Figure 1 for an example Eurocode 8 spectrum [CEN, 2004]), this base shear force is then distributed along the height of the building in a linear manner. Period-height relationships which have been obtained for different building typologies from the measured periods of vibration during earthquake ground shaking are generally used, though Rayleigh analysis is also often allowed. The linear dynamic (or modal response spectrum) method requires a simple analytical model of the structure to be produced (often using structural sections of reduced stiffness) and the periods of vibration and modal shapes of a number of significant modes to be calculated. The forces resulting from each mode are applied to the building using the appropriate modal shape and the seismic actions resulting from these forces are combined using specified combination rules.

As can be deduced from Figure 1, the period of vibration is a fundamental parameter in the force-based design of structures as this parameter defines the spectral acceleration and thus the base shear force to which the building should be designed. For the usual range of structural periods, higher periods of vibration lead to lower design forces. This paper takes a critical look at the way in which seismic design codes around the world have allowed the designer to estimate the period of vibration for use in both linear static and dynamic analysis. The influence of the period of vibration on the design will be briefly discussed and some preliminary proposals for updating the periods of vibration in linear analysis in Eurocode 8 will be made.

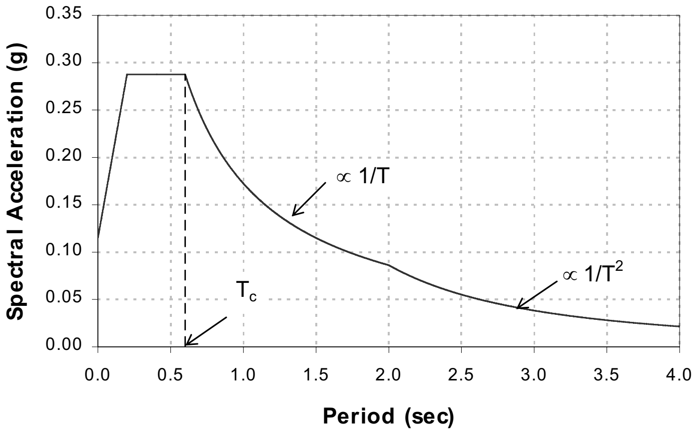

Figure 1. Type 1 acceleration response spectrum in Eurocode 8 for a peak ground acceleration of 0.1g and site condition C [CEN, 2004].

2 PERIOD OF VIBRATION IN DESIGN CODES

2.1 Period-height relationships in seismic design codes for moment resisting frames

The fundamental period of vibration required for the simplified design of reinforced concrete structures has been calculated for many years using a simplified formula relating the period to the height of the building. One of the first formulae of this type was presented almost 30 years ago in ATC3-06 [ATC, 1978] and had the form:

$$T = C_t H^{3/4} \tag{1}$$

where C_t was a regression coefficient and H represented the height of the building in feet.
As discussed in Goel and Chopra [1997], the particular form of Eq. (1) was theoretically derived by assuming that the equivalent static lateral forces are linearly distributed over the height of the building and the distribution of stiffness with height produces a uniform storey drift under the linearly distributed lateral forces. Furthermore in ATC3-06 [ATC, 1978] the base shear force was assumed to be inversely proportional to $T^{2/3}$ and thus these two assumptions led to Eq. (1), as shown in the workings below.
The period of vibration (T) of a single degree of freedom oscillator can be obtained from Eq. (2) where m is the mass of the oscillator and k is the stiffness:

$$T = 2\pi\sqrt{\frac{m}{k}} \tag{2}$$

The stiffness of the oscillator can be obtained from the base shear (V) divided by the lateral displacement (Δ). From the response spectrum in early design codes, the base shear for the usual range of periods of structures was taken as inversely proportional to the period to the power of two-thirds, with the coefficient of proportionality defined as C_1 herein. As mentioned previously, if one assumes that the distribution of stiffness with height produces a uniform storey drift under the linearly distributed lateral forces, then the lateral displacement, Δ is given by the interstorey drift, ϑ, multiplied by the height, H:

$$k = \frac{V}{\Delta} = \frac{C_1}{T^{2/3}\Delta} = \frac{C_1}{T^{2/3}\vartheta H} \tag{3}$$

By replacing Eq. (3) in Eq. (2), the relationship shown in Eq. (1) between period and height can be obtained, as outlined in the workings of Eq. (4) to (6):

$$T = 2\pi\sqrt{\frac{mT^{2/3}H\vartheta}{C_1}} \tag{4}$$

$$T^2 = (2\pi)^2 \frac{mT^{2/3}H\vartheta}{C_1} \tag{5}$$

$$T^{4/3} = C_t H \tag{6}$$

In ATC3-06 [ATC, 1978], the coefficient C_t in Eq. (1) was given equal to 0.025 for reinforced concrete moment resisting frames. This coefficient was identified from a study by Gates and Foth [1978] based on the measured periods of vibration of reinforced concrete frames during the 1971 San Fernando earthquake. A subsequent re-evaluation by SEAOC-88 [SEAOC, 1988] found that a value of Ct=0.03 was more appropriate for reinforced concrete buildings. The coefficient Ct was generally calibrated such that the derived fundamental period would underestimate the period by approximately 10-20% at first yield to obtain a conservative estimate for the base shear [Goel and Chopra, 1997].

Bertero *et al*. [1988] studied in greater detail the fourteen buildings considered by Gates and Foth [1978] and found that four of the buildings and the longitudinal direction of a fifth building could not be considered as moment-resisting frame (MRF) structures and they thus excluded them from the database. They further identified two buildings with structural damage and two others that had non-structural damage; they discuss how damage leads to stiffness degradation and thus an increase in the period of vibration. Gates and Foth [1978] did not relate the building damage to the period of vibration and thus Bertero *et al*. [1988] re-evaluated the time histories of building response for the moment-resisting frame structures and identified times where a sudden increase in the period of vibration took place, which was then correlated to the onset of non-structural and structural damage. The period of vibration at the second increase in period was considered to be the stage when the non-structural components were no longer contributing significantly to the stiffness and interpreted as the period at which the building was essentially vibrating as a bare structural frame. They added a further four buildings to the database and evaluated the bare frame period of vibration for all buildings in a uniform manner at the aforementioned second increase in period. The conclusions of their study was that the formula of Eq. (1) with C_t equal to 0.03 does not constitute a reliable estimate of the during earthquake period of reinforced concrete moment-resisting frames, and a better fit was found with C_t =0.04 (0.097 with H in metres). For a lower bound estimate of the period, Bertero *et al*. [1988] recommend the use of C_t = 0.035 (0.085 with H in metres).

In the buildings used in the Bertero *et al*. [1988] study, the partition walls were generally plaster board/dry walls whilst the outer infill was often constructed with glass curtain walls with spandrels built into the outer frames. Bendimerad *et al*. [1991] found that the participation of these non-structural components on the stiffness of the building was minimal compared to the stiffness of the frame and thus had practically no effect on the building period beyond the first 5 seconds of earthquake motion. Hence, the use of the equation derived from these buildings for the design of "moment resisting frame systems of reinforced concrete in which the frames resist 100 percent of the required seismic force" appears to be justified.

The use of the form of period-height equation shown in Eq. (1), along with the SEAOC-88 recommended 0.03 coefficient, has been adopted in many design codes since 1978, for example in UBC-97 [UBC, 1997], in SEAOC-96 [SEAOC, 1996], in NEHRP-94 [FEMA, 1994] and in Eurocode 8 [CEN 1994; 2004]. In Eurocode 8, the C_t coefficient has simply been transformed considering that the height is measured in metres, leading to C_t = 0.075. In a similar way that the use of a 475 year return period to define the seismic actions was adopted in seismic design codes around the world [see e.g. Bommer and Pinho, 2006], the period-height equation of Eq. (1) with C_t=0.075 has also spread around the world (see for example the design codes of the following countries: Algeria '88; Cuba '94; El Salvador '89; Israel '95; Korea '88; Panama '94; Philippines '92) [IAEE, 1996; 2000].

Goel and Chopra [1997] used the fundamental period of vibration of buildings measured from their motions recorded during eight California earthquakes from 1971 to 1994 to update the period-height formula in UBC-97. They found that the best fit lower bound curve (i.e. the mean –1 standard deviation) for reinforced concrete frames was given by:

$$T = 0.0466 H^{0.9} \; (H \text{ in metres}) \tag{7}$$

This period-height formula has recently been included in ASCE 7-05 [2006]. As can be noted from Eq. (7), Goel and Chopra [1997] decided to move away from the 0.75 power regression and found the best-fit regression to be 0.9. Simplified period-height equations can only be applied in Eurocode 8 for buildings up to 40 metres, and thus the period of vibration obtained with such equations will generally be within the range of inverse proportionality between base shear and period (see Figure 1). By repeating the calculations in Eq. (2) to (6) with the base shear force inversely proportional to T (i.e. $V=C_1/T$), the form of the period-height equation becomes linear:

$$T = C_t H \tag{8}$$

The reason that Goel and Chopra [1997] did not arrive at a linear relationship between period and height is probably because they did not just focus on the buildings whose base shear should be inversely proportional to period, but included taller buildings in the range of "non-proportionality". Based on the fact that an updated period-height equation for RC frames has now been proposed, as well as the possibility that a linear equation might be more valid for frames designed to Eurocode 8, an examination of the periods of vibration of newly designed European reinforced concrete bare frames is thus warranted; this issue is considered further in Section 2.3.

2.2 *Period-height relationships in seismic design codes for structures with reinforced concrete or masonry walls*

The first period-height relationship for buildings with concrete shear walls had the form presented in Eq. (9) and, as with the previous equations, was also calibrated using the measured motions of buildings recorded during the 1971 San Fernando earthquake:

$$T = \frac{C_t H}{\sqrt{D}} \tag{9}$$

where D is the dimension of the building at its base in the direction under consideration. With the height and dimension D of the building measured in feet, C_t was proposed in ATC3-06 [ATC, 1978] as 0.05 (which would be 0.09 with these dimensions measured in metres). This formula comes from the equation of the frequency of vibration of a cantilever (considering shear deformations only), with the thickness of the wall considered to be more or less constant and thus only the width/length of the building is an input parameter, as demonstrated in Eq. (10):

$$T = 4\sqrt{\frac{m}{\kappa G}}\frac{H}{\sqrt{A}} = \frac{\alpha H}{\sqrt{A}} = \frac{\alpha H}{\sqrt{D t_w}} = \frac{\alpha_1 H}{\sqrt{D}} \tag{10}$$

where *m* is the mass per unit length, *G* is the shear modulus, κ is the shape factor to account for non uniform distribution of shear stresses, *D* is the length of the cantilever and t_w is the thickness. This formula is used in many design codes around the world, but the type of structure to which it is applied varies from code to code, as illustrated in Table 1; it is noted that the text used in each code to describe the structures to which Eq. (9) applies has been maintained. Some codes use this formula specifically for buildings with both frames and shear walls, some use the equation for reinforced concrete MRF with masonry infill panels, but many specify it for use with any building except moment resisting space frames.

Table 1. Buildings to which Eq. (9) is applied in different codes from around the World [IAEE, 1996; 2000].

Country, Year	Type of Structure to which Eq. (9) is applied
Albania, 1989	RC framed structures with brick masonry infill walls participating in seismic force resistance
Algeria,1988	Steel or RC moment resisting frames with infilled masonry and partial or total RC shear walls, braced frames, and masonry walls
Canada, 1995	Other structures (i.e. not moment resisting frames)
Colombia 1984	Other structures (i.e. all except framed structures where the frame is not braced by rigid elements that tend to impede the free deflection)
Cuba, 1995	RC buildings with frames and shear walls
Egypt, 1988	All buildings except moment resisting space frames
El Salvador, 1989	All buildings except frames
Ethiopia, 1983	All buildings except those with moment resisting space frames capable of resisting 100% of the required lateral forced and not enclosed by or adjoined by more rigid elements
India, 1984	Other structures (not moment resisting frames without bracing or shear walls for resisting lateral loads)
Iran, 1988	All buildings except moment resisting frames, if other elements do not create an obstacle to the movement of the building frame
Israel, 1975	Multi-storey structures in which horizontal forces are carried by RC frames
Italy, 1986 & 1996	For framed structures (N.B. C_t from Eq. (9) is taken as equal to 0.1)

Country, Year	Type of Structure to which Eq. (9) is applied
Peru, 1977	For buildings whose structural elements are exclusively open frames and shear walls of the elevator, without other elements for providing rigidity to the structure
Venezuela, 1982	Structures consisting of frames and structural walls of reinforced concrete or braced frames

The UBC-97 code [UBC, 1997] did not use Eq. (9) for shear wall buildings, but instead reported empirical equations of the form of Eq. (1), where C_t was taken equal to 0.02 with the height measured in feet and 0.05 with the height measured in metres. This formula has become the "other structures" formula in Eurocode 8 [CEN, 2004] and is also present in the Algeria 1988 code as another possible formula to be used for steel or RC moment resisting frames with infilled masonry and partial or total RC shear walls, braced frames, and masonry walls. The Algerian code is much more explicit about the coefficient to be used in Eq. (1) for RC buildings with and without infilled frames; it is believed by the authors that many designers following Eurocode 8 would use Eq. (1) with C_t equal to 0.075 for all reinforced concrete buildings regardless of the details of the masonry infills. Hence, considering that many reinforced concrete buildings in Europe are constructed with stiff masonry infill panels which are often not isolated from the RC frame, the period of vibration is probably being overestimated by the designers and thus the forces are subsequently being underestimated. This issue is considered further in Section 3.

Another value of C_t to be used in Eq. (1) was permitted for buildings with shear walls in the UBC-97 and SEAOC-96 [SEAOC, 1996] documents, based on the following formula:

$$C_t = \frac{0.1}{\sqrt{A_c}} \tag{11}$$

where A_c, the combined effective area (in square feet) of the shear walls is defined as:

$$A_c = \sum_{i=1}^{NW} A_i \left[0.2 + \left(\frac{D_i}{H} \right)^2 \right]; \; D_i / H \le 0.9 \tag{12}$$

in which A_i is the horizontal cross-sectional area (in square feet); D_i is the dimension in the direction under consideration (in feet) of the i^{th} shear wall in the first storey of the structure; and NW is the number of shear walls. The numerator of Eq. (11) becomes 0.075 when the dimensions of the structure are measured in metres. This equation has also found its way in Eurocode 8 [CEN, 2004] as the formula to be used for structures with concrete or masonry shear walls; however, the formula for A_c, given below in Eq. (13), appears to be slightly different to the original formula in Eq. (12) which the authors believe might be due to an editing error.

$$A_c = \sum_{i=1}^{NW} A_i \left[\left(0.2 + \left(\frac{D_i}{H} \right) \right)^2 \right] \tag{13}$$

Goel and Chopra [1998] and Lee *et al*. [2000] both show how Eq. (9) and (11) are too conservative for shear wall buildings when compared with the measured periods of vibration of buildings during earthquakes. Goel and Chopra [1998] discuss how there is little correlation between the $H/\sqrt{D}$ value of Eq. (9) and the period of vibration. This could be because the shear walls do not extend for the whole dimension "D" of the buildings, but for just a small proportion. On the other hand, Eq. (11) which includes explicitly the dimensions of the walls, appears to be better correlated to the period of vibration, but was nevertheless found to be too conservative. Both Goel and Chopra [1998] and Lee *et al*. [2000] decided to calibrate an equation for the period of vibration of shear wall structures by considering the fundamental period of a uniform cantilever beam with both flexural and shear deformations. Figure 2 shows the period of vibration of a cantilever with both flexural and shear deformations (based on Dunkerley's method – see Inman [1996]) divided by the period of vibration of a pure-flexural cantilever as a function of the ratio of height to depth (H/D). This plot shows that the period approaches the period of a pure-shear cantilever as the height to depth ratio becomes smaller, and the period of a pure-flexure cantilever as the ratio increases. For shear walls with H/D ratios between 0.2 and 5, the contribution of both flexure and shear to the period of vibration should be considered.

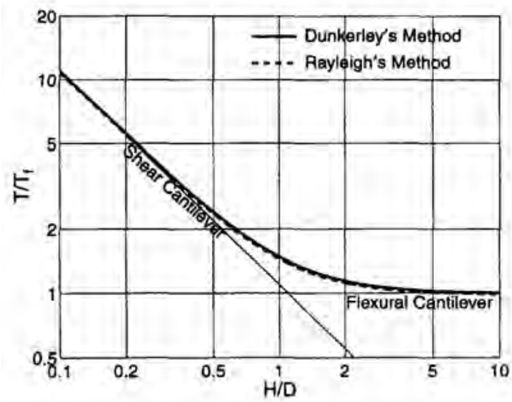

Figure 2. Fundamental period of cantilever beams as a function of height to depth ratio [Goel and Chopra, 2008].

Goel and Chopra [1998] calibrated Dunkerley's equation using the measured periods of vibration of shear wall buildings and obtained the formula which is shown in Eq. 14; this equation has recently been included in ASCE 7-05 [2006].

$$T = \frac{0.0019}{\sqrt{C_w}} H$$

$$\text{where } C_w = \frac{100}{A_B} \sum_{i=1}^{NW} \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} \tag{14}$$

2.3 Periods of vibration of case study buildings

The period-height relationships derived in the past, and described in the previous sections, have been obtained from the measured period of vibration of buildings built over a long period of time to different design codes; this variation in the age of the buildings, and thus design regulations, might be part of the reason for the large scatter in the results. Buildings designed to more recent seismic design codes with higher lateral force requirements and capacity design principles are more likely to have larger columns and are thus likely to be stiffer. A number of European moment resisting reinforced concrete frames were modelled by Crowley [2003] using SeismoStruct [SeismoSoft, 2008], a fibre-element finite elements structural analysis package, and Eigenvalue analysis was carried out to compute their fundamental periods of vibration. The results showed that there was a large difference in the stiffness of the buildings designed pre- and post-1980 (Figure 2), most likely due to the changes in design philosophy mentioned previously. The Eurocode 8 equation [CEN, 2004] which was obtained by converting Eq. (1) from feet to metres appears to match well the period of vibration of these newer European buildings, though as mentioned in Section 1 there have been a number of criticisms of the post-processing of the data which led to this equation and it was found to be too conservative. However, considering that more recent European buildings are stiffer than older pre-1980 buildings, the equation now appears to be more reliable. The recently proposed upper and lower bound equations by Goel and Chopra [1997] which were obtained from a number of buildings subjected to earthquakes in California from 1971 to 1994 are also shown in Figure 2. The characteristics of the buildings in California may not have changed so extensively during the last 40 years as they appear to have done in Europe, and so the scatter in the Goel and Chopra data may not be due to age, but it would be an interesting study to see if the scatter could be reduced by including this parameter.

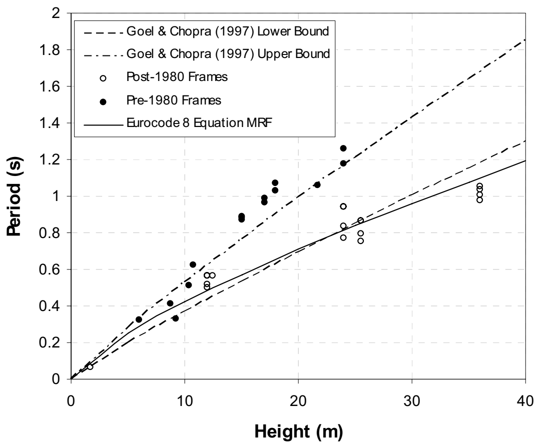

Figure 3. Upper and lower bound period-height formulae by Goel and Chopra [1997] and the EC8 [CEN, 2004] for bare MRF compared with the gross stiffness periods of vibration from European bare MRF designed pre- and post-1980 [see Crowley and Pinho, 2004; Crowley, 2003].

In Europe, it is very common to add rigid masonry infill panels to reinforced concrete moment resisting frames, both internally and externally; these non-structural elements will influence both the lateral stiffness and strength of the building. Eurocode 8 allows the designer to ignore the contribution of the infill panels to the strength of the building (by considering them as non-structural elements), and the authors believe that many designers following the recent Eurocode 8 regulations would also ignore their contribution to the stiffness. Eurocode 8 states in the modelling section that "*infill walls which contribute significantly to the lateral stiffness and resistance of the building should be taken into account*" but there is no guidance on how they should be modelled. If the designers use the linear static method to design the building then they may mistakenly use a moment resisting space frame equation to calculate the period of vibration, or otherwise use the Rayleigh method, where the deformed shape is obtained from a numerical model where the infill panels are not included. For what concerns the linear dynamic method, where Eigenvalue analyis is required to calculate the periods of a number of modes of vibration, again it is probable that these rigid elements are not being included in the numerical model.

In order to consider the influence of this type of rigid masonry infill on the period of vibration of reinforced concrete moment resisting frames, Angel [2007] added equivalent struts to the post-1980 European frame models mentioned previously; the thickness of the infill was taken as either 100mm or 250mm and the panels were modelled as either fully infilled, or with a number of openings. The periods of vibration of these four types of infilled frames are shown in Figure 3, together with the period of vibration of the bare frames.

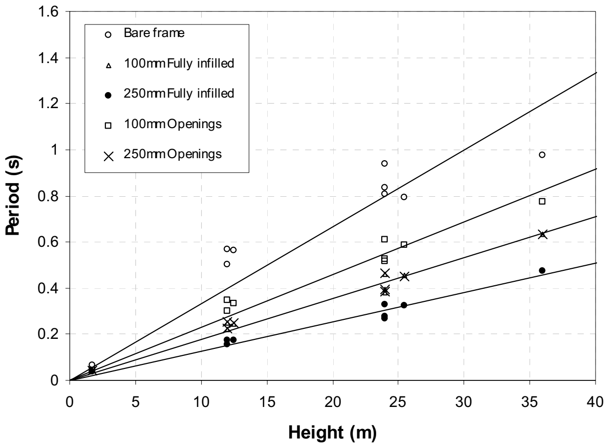

Figure 4. Periods of vibration of post-1980 reinforced concrete moment resisting frames modelling with infill panels with varying characteristics.

The distribution of the infill panels will vary within a building with some frames fully infilled, some with openings, whilst some frames will remain bare. Bal *et al.* [2008] studied the characteristics of Turkish reinforced concrete buildings and found that the proportion of bare frames, fully infilled frames and partially infilled frames in a sample of Turkish buildings to be 34%, 28% and 38%, respectively. These ratios have been used to calculate a weighted mean period of vibration for each frame presented in Figure 3, considering 50% with 100mm infills and 50% with 250mm infills. The period of vibration of the same frames has also been calculated using Eq. (9) and with the formula for “other structures” in Eurocode 8 (Eq. (1) with C_t equal to 0.05); a comparison of these code equations with the analytically calculated periods of vibration is given in Figure 4.

The results in Figure 4 show that the periods of vibration for these infilled post-1980 European buildings from numerical analysis match well both Eq. (9) and Eq. (1) (with the C_t coefficient suggested in Eurocode 8 for “other” structures). The reason that these infilled buildings agree with these equations, whereas other authors have found that they are too conservative for shear wall structures, is probably due to the low height to width ratios which arise when you have infill walls in all bays of the structure. A low height to width ratio implies a higher shear deformation contribution to the period of vibration and so this may be the reason why Eq. (9) works well.

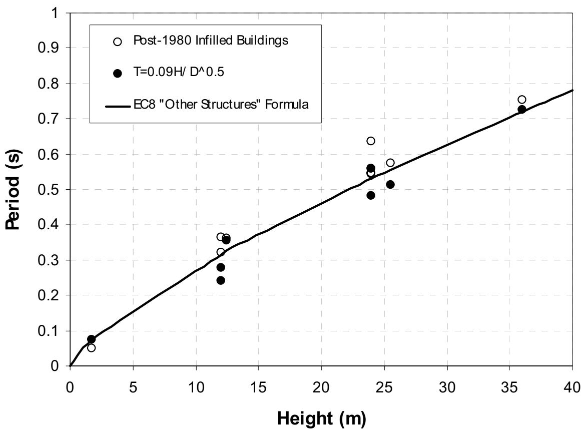

Figure 5. Periods of vibration of post-1980 "infilled buildings" based on a weighted mean of the results in Figure 2, compared with the periods predicted with Eq. (9) and the formula in Eq. (1) with C_t equal to 0.05.

3 PERIODS USED IN LINEAR STATIC AND DYNAMIC ANALYSES IN EC8

Eurocode 8 [CEN, 2004], as with most design codes, allows linear analyses to be carried out for the design of new structures. Linear analysis includes the lateral force method (static) and the modal response spectrum (dynamic) method: both types of analysis use smooth response spectra specified in the code in order to characterise the earthquake actions (see Figure 1). The modal response spectrum analysis is applicable for all types of buildings, whilst the lateral force method of analysis has many restrictions on its use due to the 'fear' that it would provide unconservative results in certain conditions; however, in spite of this disadvantage the method is still widely used due to its ease of application.

Essentially, the lateral force method of analysis may be applied to buildings whose response is not extensively affected by the contribution of higher modes of vibration. According to Eurocode 8, such buildings follow two conditions: firstly, the fundamental period of free vibration in the principal directions is smaller than either four times the corner period ($4T_c$, where T_c varies from 0.25s to 0.8s) or two seconds; and secondly, the building is regular in elevation. The lateral force method allows the use of the simplified relationships described in Section 2 for the estimation of the period of vibration of the building under design. For those buildings for which the lateral force method cannot be applied, in many cases it is possible to apply the modal response spectrum method. In order to calculate the periods of a number of modes of vibration, the designer would generally produce a numerical model of the building and carry out Eigenvalue analysis.

Recent studies [Angel *et al.*, 2006; Pinho *et al.*, 2007] have shown through a number of examples how the application of these two methods to a given building leads to different design base shear forces and shear force profiles. Examples of the shear force profiles obtained with the two methods for two post-1980 European frames are given in Figure 5.

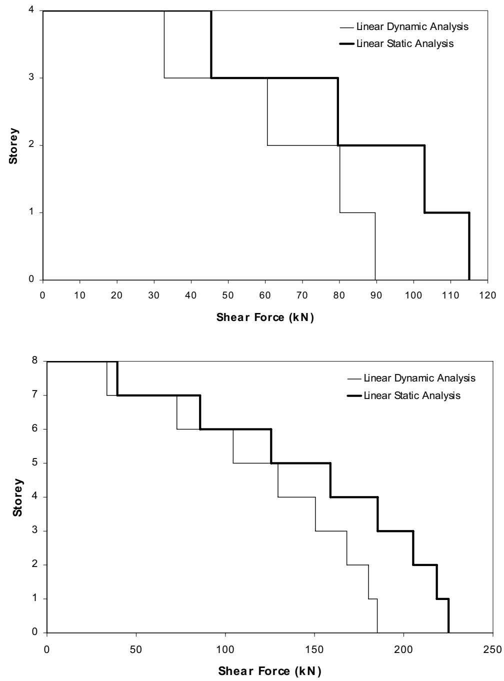

Figure 6. Shear force profiles for two reinforced concrete buildings designed to Eurocode 8 obtained with the linear dynamic and static procedures (Pinho *et al.*, 2007).

The main reason for the differences in the base shear force in the plots is Figure 5 is the disparity between the fundamental period of vibration from the period-height equation and the period of vibration from Eigenvalue or Rayleigh analysis of a bare frame model, where the stiffness of the sections may be reduced by up to 50% [CEN, 2004]. Many codes recognise that the period of vibration from the simplified period-height equation is more realistic, having been directly obtained from the measured periods of vibration of buildings subject to earthquake ground motions, but that when higher modes are important (in tall and/or irregular structures) the modal response spectrum method gives a more realistic profile of the lateral forces. Hence, these codes (e.g. ASCE [2006]; NBCC [2005]) require the designer to check whether the modal base shear force is less than 85% of the base shear force from the equivalent lateral force method. If this is the case then the modal forces, but not the drifts, should be multiplied by $0.85V/V_t$ where V is the base shear from the lateral force method and V_t is the base shear from the required modal combination.

Even when higher modes are not important and the designers are allowed to use the linear static method, but they decide to calculate the period of vibration from the Rayleigh method, many codes apply an upper bound to the period of vibration from the Rayleigh method. This is another procedure which is used to safeguard against unrealistically high periods of vibration used in the design to lower the base shear forces. In ASCE 7 (ASCE, 2006) the period of vibration of the building calculated by the designer (perhaps with a numerical model with members of reduced stiffness) should not be higher than C_u multiplied by the period from the standard period-height equations such as those presented in Section 1. This coefficient C_u varies from 1.4 to 1.7 as a function of the design spectral response acceleration

at 1 second; the higher the spectral acceleration the lower the coefficient C_u. The empirical equations have been derived using buildings in areas with higher lateral force requirements and the aforementioned variation of the coefficient C_u is intended to reflect the likelihood that buildings in areas with lower lateral force requirements will be more flexible and hence it is not necessary to be so stringent on the period to be used for design. Furthermore, it results in less dramatic changes from present practice in lower risk areas [FEMA, 2003].
Eurocode 8 [CEN, 2004] does not include either of these clauses, which the authors believe provide a rational method to avoid unrealistically low seismic design forces, whilst at the same time allowing the drifts and the higher mode effects to be realistically modelled.

4 RECOMMENDATIONS FOR FUTURE DRAFTS OF EUROCODE 8

The main recommendation of this paper is that future revisions of Eurocode 8 [CEN, 2004] should consider the following points related to the periods of vibration and forces obtained in linear analysis:

- When presenting the period-height equations for reinforced concrete moment resisting frames (MRF) there should be two formulae which depend on whether (a) the infill panels are to be isolated from the MRF or (b) the infill panels are to be rigidly connected to the MRF.
- The period-height equation for bare MRF that is currently in Eurocode 8 appears to be reasonable and should not necessarily be updated with the Goel and Chopra [1997] equation, especially considering that this equation is based on buildings constructed over many years and thus to different design codes.
- The period-height equation for "other structures" that is currently included in EC8 [CEN, 2004] could be used for the period of vibration of the MRF will rigid infill panels, as could Eq. (9) with C_t =0.09.
- The error which the authors appear to have found in the period of vibration equation for shear wall buildings should be rectified and this equation could be updated using the more recent equation proposed by Goel and Chopra [1998]. However, it would be interesting to compare this equation with analytically calculated periods of vibration for modern shear wall buildings; this is something which the authors hope to carry out in future research.
- The period of vibration calculated with the Rayleigh method (in linear static analysis) should be limited based on the period of vibration from the period-height equations.
- The base shear force obtained with the modal response spectrum method should be scaled up using the base shear force from the lateral force method; this will safeguard against low forces from the use of analytical models with unrealistically high periods of vibration.

5 ACKNOWLEDGEMENTS

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ASYMMETRIC-PLAN BUILDINGS: IRREGULARITY LEVELS AND NONLINEAR SEISMIC RESPONSE

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ABSTRACT
Results from both linear and nonlinear dynamic analyses exploring the seismic torsional behaviour of a multi-storey RC frame structure asymmetric in plan and regular in elevation are reported. Based on the obtained findings, the conceptual gaps in the current prescriptions given by the OPCM3431 and the EC8 on the evaluation method for the irregularity level produced by the mass and stiffness asymmetry and on the definition of the range of applicability of the linear analyses methods are identified. Comments on the code prescriptions relative to the number of accelerograms needed in the nonlinear time-history analyses and on the proposed method for modifying the seismic response obtained with a pushover analysis in order to account for the torsional behaviour of asymmetric-plan buildings are also presented. Possible proposals for a code provision are finally suggested.

KEYWORDS
Asymmetric-plan, irregularity level, evaluation, nonlinear dynamic response.

1 INTRODUCTION

Asymmetric-plan buildings, namely buildings with in-plan asymmetric mass and strength distributions, are systems characterized by a coupled torsional-translational seismic response. This particular behaviour under earthquake excitation usually leads to classify such type of buildings as irregular systems. Difficulties related to the study of their seismic response can be identified as follows: first, the identification and measure of the irregularity level produced by the asymmetry; second, the selection of the appropriate linear or nonlinear analysis method that can be used to evaluate the seismic demand; finally, the improvement and the extension of the pushover methods, originally proposed for regular structures, to the case of systems characterized by torsional seismic behaviours. The prescriptions on these issues proposed by the Italian code OPCM3431 "Technical Code for Seismic Design, Assessment and Retrofit of Buildings" (2005) are basically the same as those given in EC8 part 3 "Assessment and Retrofitting of Buildings" (2004). More specifically, as for the identification of the asymmetry, only general and qualitative rules are given by OPCM3431, which classifies buildings as regular only if both lateral stiffness and mass distributions are "approximately symmetrical" in plan with respect to two orthogonal axes. Furthermore, EC8 prescribes a limit on the value, both, of the structural eccentricity e_0 between the centre of mass and the centre of stiffness, and of the ratio between the torsional radius r and the radius of gyration l

of the floor mass in plan. As for the applicability of the linear analysis methods, both OPCM3431 and EC8 prescribe that the strengths should be distributed so as to produce uniform inelastic demands in the resisting elements of the structure. An estimation of the inelastic demand can be obtained by evaluating, for all the primary elements of the building, the ratio ρ_i between the bending moment demand calculated from a linear analysis and the corresponding capacity. In the evaluation of such uniformity, EC8 includes all yielded elements (with ρ_i values greater than 1), while OPCM3431 only those that go well into the nonlinear range (with ρ_i values greater than 2). It is important to note that this applicability condition for linear analysis methods applies to all buildings, regardless of the shape of their plan; as a matter of fact, no specific requirements for the case of asymmetric-plan buildings are given by either code.

Finally, on the use of pushover methods, while OPCM3431 does not apply any kind of restriction to asymmetric-plan buildings, EC8 prescribes to increase the displacements of the stiff/strong side obtained with a pushover by amplification factors based on the results of an elastic analysis. This prescription was supported by studies reported in Peruš and Fajfar (2005), Marušić and Fajfar (2005), and Fajfar (2005), which demonstrated that the elastic amplification of the displacements at the edges of the plan with respect to the displacement at the centre of mass can be used as a rough estimate also in the inelastic range.

The objective of the present paper is to show, through the analysis of a selected case study, the conceptual gaps in the current prescriptions given in OPCM3431 and EC8 on asymmetric-plan buildings. Suggestions for possible improvements of the identification and evaluation methods are also presented.

2 CASE STUDY

The case study selected for the investigation is the two-storey RC building shown in Figure 1, comprising a shear-beam frame structure with rigid diaphragms at each floor. The system is regular in elevation and asymmetric in plan because of the eccentricity in the x-direction at each floor between the centre of stiffness CS, located at the geometrical centre of the plan, and the centre of mass CM. Periods and participating mass ratios of the elastic modes of the system, whose shapes are shown in Figure 2, are reported in Table 1. Because of the asymmetry in the y-direction only, the first and third modes are coupled, while the second one is pure translational in the x-direction.

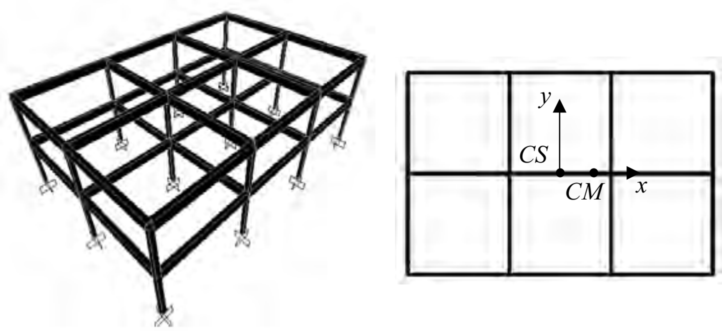

Figure 1. Studied two-storey frame RC structure: 3D view and plan.

The strengths of the resisting elements are designed for the seismic load combination so as to generate a uniform ρ_i distribution, characterized by a ratio between the maximum and minimum values ρ_{max} and ρ_{min} lower than the limit of 2.5 prescribed by OPCM3431. More specifically, the two ρ_{max}/ρ_{min} values corresponding to earthquakes exciting the structure in the *x* and *y* directions are equal to 1.89 and 1.73, respectively.

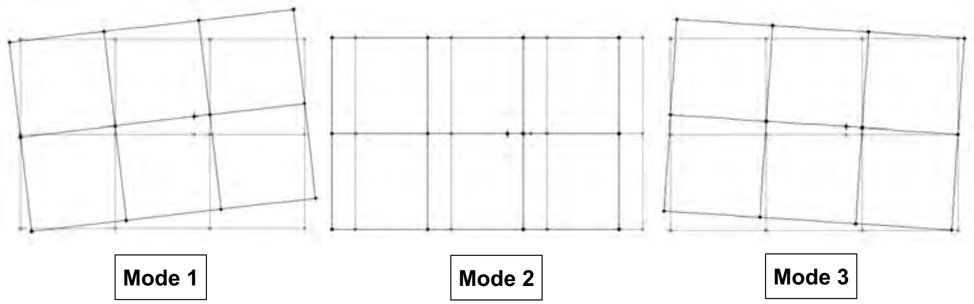

Figure 2. Deformed shapes at the roof of the first three modes of the structure.

Table 1. Periods and modal participating mass ratios.

	Mode 1	Mode 2	Mode 3
T [s]	0.70	0.62	0.56
M_{ux} [%]	0	93	0
M_{uy} [%]	52	0	41
M_{rz} [%]	78	0	15

Table 2. Natural accelerograms selected for the dynamic analyses.

Earthquake	Date	Station	Primary component *PGA* [g]	Secondary component *PGA* [g]
Friuli	1976-05-06	Tolmezzo-Diga Ambiesta	0.34	0.30
Montenegro	1979-04-15	Ulcinj-Hotel Albatros	0.35	0.28
Campano Lucano	1980-11-23	Bagnoli-Irpino	0.28	0.21
Campano Lucano	1980-11-23	Sturno	0.31	0.21
Umbria-Marche	1997-09-26	Nocera Umbra	0.51	0.45
Izmit	1999-08-17	Izmit-Meteoroloji Istasyonu	0.32	0.23
Duzce	1999-11-12	LDEO Station n° C0375 VO	0.74	0.48

In the nonlinear analyses, performed to evaluate the inelastic response of the structure, the resisting elements are modelled as beams with plastic hinges at the ends, with elasto-plastic hysteretic behaviours. The yielding bending moment values of the column hinges in the *x* and *y* directions are considered as non-interacting between them and with the axial force.

In both linear and nonlinear analyses, a Raleigh model is used for the viscous damping matrix, whose mass and tangent stiffness matrices coefficients are set to have modal damping ratios equal to 5% at the first and third period of the elastic system. The acceleration time-histories selected for the dynamic analyses are the seven unscaled natural accelerograms defined in Iervolino *et al.* (2007), compatible with the response spectrum type A with *PGA* intensity equal to 0.35g. The main properties of the records are briefly reported in Table 2.

3 ANALYSES RESULTS

On the selected case study two sets of analyses are carried out. First, linear modal response spectrum analyses are used to evaluate the coupling level between the rotational and translational response of the system produced by the in-plan asymmetry. Then, linear and nonlinear time-history analyses are used to study the seismic behaviour evolution from elastic to inelastic range, and also to evaluate the response sensitivity to the exciting accelerograms.

3.1 Response spectrum analyses

In general, the asymmetric-plan building irregularity results in the coupling of the torsio-translational behaviour, producing non-uniform seismic demands in the resisting elements of the system. Such coupling depends, in turn, by the significance of the higher modes contribution: in fact, it can be easily observed that, with the increase of the coupling, more than one mode is needed to excite the translational mass of the system.
However, it is worth noticing that, on the basis of this evidence, the irregularity in the demand does not depend solely on the properties of the building, as indicated both in OPCM3431 and EC8, but also on the properties of the seismic excitation. The single mode contribution, in fact, depends both on the associated participating mass ratio and on the frequency content of the seismic excitation as well.
This can be easily seen in the plot of Figure 3, where the OPCM3431 response spectra of three different ground types are shown. Looking at the two periods T_2 and T_3 of the coupled modes of the studied building, it is observed that for two different spectra, *e.g.*, type D and type A, the relative contribution of the modes is different: for type D the two modes have the same contribution, while for type A the two modes have different contributions.

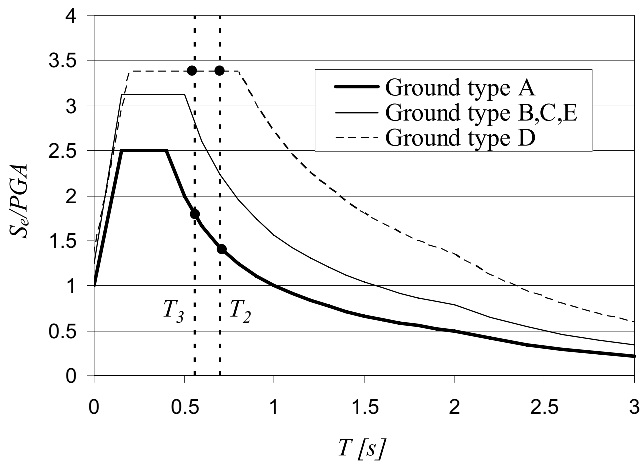

Figure 3. OPCM3431 response spectra with the periods of the coupled modes of the studied building.

One way for estimating the importance of the higher modes in global terms could be to evaluate the ratio between the seismic base shear associated to the first coupled torsional-translational mode in one direction and the total seismic base shear acting in the same direction.

Thus, a simple modal response spectrum analysis would suffice to measure the irregularity level of the building, while weighing the seismic excitation effects on the asymmetry of mass and stiffness distributions.

In the studied case, values of the base shear ratio ranging between 65% and 75%, depending on the ground type considered, are obtained.

3.2 Time-history analyses

According to EC8, if ρ_{max}/ρ_{min} is lower than a certain value (2.5 for OPCM3431), then the distribution of the inelastic demands, as measured by the ρ_i, is considered as uniform. This allows to use a linear analysis to evaluate the seismic demand in the resisting elements of the building. In this prescription, there is the implicit assumption that, if the inelastic demands are uniformly distributed, then the building behaves regularly, in the sense that the elastic response shape does not significantly change when the building enters the nonlinear range. This implies that the seismic demand evaluation (in terms of deformations) obtained from the linear analysis can be considered as representative of that in the nonlinear range.

One of the peculiarities of the seismic response of asymmetric-plan buildings is that the maximum demand is not attained at the same time in all the elements of the structure: this is due to the higher modes effects, that is, to the torsio-translational behaviour characterizing their seismic motion. For such systems, even if their ρ_i distribution is uniform, the structure does not enter the nonlinear range in a uniform fashion. In such case, it can be seen that the seismic response cannot be identified as regular. This is the case of the investigated asymmetric-plan building, which, notwithstanding its ρ_{max}/ρ_{min} largely lower than 2.5, shows a seismic response shape in the nonlinear range different from the linear one.

In Figure 4, the response of the building to the Montenegro earthquake (Yugoslavia, April 15, Ulcinj-Hotel Albatros station) is shown. In the plots, the envelopes of the maximum displacements in the direction of asymmetry (the *y*-direction) of the four frames at the second floor of the structure are reported. More specifically, in the two plots the absolute displacements u_y and the normalised values with respect to the displacement at the centre of mass $u_{y,\,CM}$ are shown, respectively. In addition to the nonlinear displacements, in each plot the elastic response of the building to the same accelerogram is also reported for comparison.

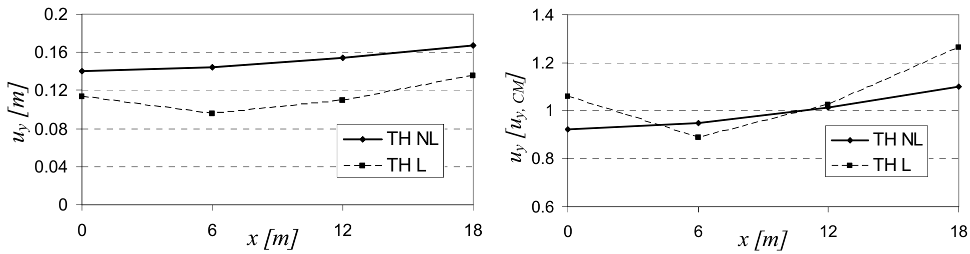

Figure 4. Response of the system to the Montenegro earthquake: envelopes of the roof maximum displacements in the *y*-direction obtained from nonlinear (TH NL) and linear (TH L) analyses, shown as absolute values (left) and normalised with respect to the centre of mass displacement (right).

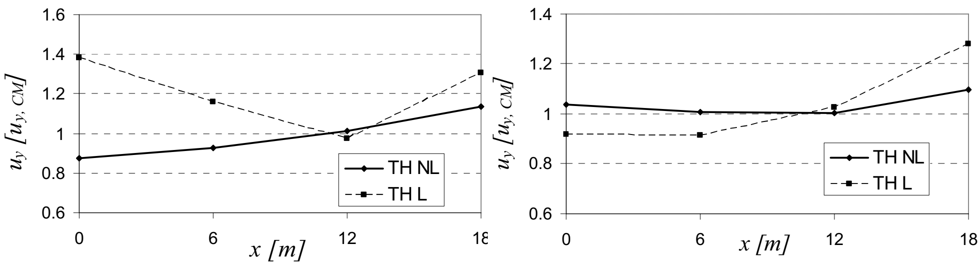

Figure 5. Comparison of the responses under different earthquakes normalised with respect to the centre of mass displacement: Campano Lucano (left) and Umbria Marche (right).

On the basis of these results, general observations on the behaviour of asymmetric-plan buildings can be done. First, in some cases the inelastic deformations of the system can result higher than the elastic ones, meaning that the linear analysis can underestimate the actual inelastic demands in the elements of the structure. Second, the shape of the system response changes between linear and nonlinear range. The elastic one, in fact, is usually more affected by in-plan rotations (see, for example, Lucchini et al., 2009): the more curved envelope shows amplifications with respect to the displacements at the centre of mass on both the rigid and flexible side of the plan, which are usually absent in the nonlinear response.
From such considerations, it can be concluded that, for asymmetric-plan buildings, if the inelastic demand is estimated as significant (*e.g.*, as proposed by OPCM3431, if the ρ_i are greater than 2), regardless its distribution (as measured by ρ_{max}/ρ_{min}), a nonlinear analysis should always be used to evaluate the seismic demand.
In Figure 5, the envelopes of the maximum displacements produced by the Campano Lucano and the Umbria Marche earthquakes, respectively, are reported. From the comparison between the two curves the dependence of the shape of the system response on the selected exciting accelerogram can be clearly noticed: on the flexible side of the plan, in fact, either amplifications or deamplifications can be experienced by the system in the nonlinear range.
Such dependence can be also observed in the plot of Figure 6 that reports, both for the linear and the nonlinear response (denoted with solid and dashed lines, respectively), the mean and the mean plus/minus one standard deviation values of the *normalised* maximum displacements (which shows the displacement *shape*), evaluated in the *y*-direction for all the seven earthquakes in Table 2.

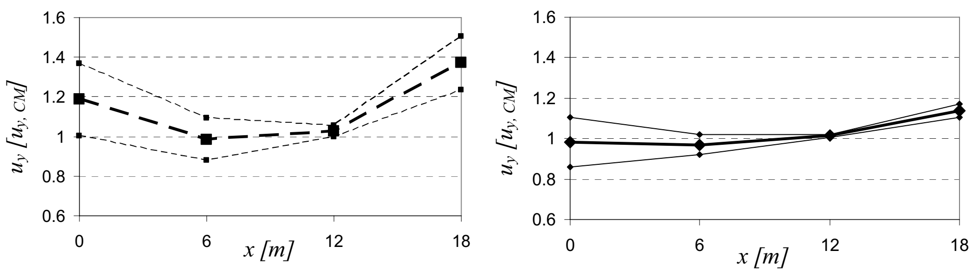

Figure 6. Mean and mean plus/minus one standard deviation values of the *normalised* maximum displacements in the *y*-direction evaluated for the seven earthquakes in Table 2, as obtained from linear (left) and nonlinear (right) analyses.

Even if the response *shape* tends to show less variability in the nonlinear range – *i.e.*, its standard deviations decreases – however, it still depends on the excitation characteristics: it is important to notice, in fact, that for regular buildings characterized by pure translational behaviours the standard deviation of the normalised displacements is always equal to zero, as the in-plan response *shape* does not depend on the excitation.
Based on this finding, it can be stated that when nonlinear time-history analyses are used to evaluate the seismic demand of asymmetric-plan buildings, because of the significant sensitivity of the response shape to the selected record, a larger number of accelerograms than that requested for regular buildings should be prescribed by the code.
Observing the plot of Figure 6, another interesting result can be found: the mean value of the normalised displacement evaluated with the linear analysis is always higher than the nonlinear one. This means that the inelastic displacements amplifications at the edges of the plan can be conservatively approximated by the elastic ones. This finding actually confirms the EC8 prescription that requests to increase the displacement of the stiff/strong side obtained with a pushover with an amplification factor based on the results of an elastic analysis. However, based on this code prescription, while the displacements are clearly obtained through the amplification factors, it is not clear how other quantities of interest (*e.g.*, element forces or chord rotations), nonlinearly dependent on the displacement, can be obtained.

4 POSSIBLE PROPOSALS FOR A CODE PROVISION

Dynamic analyses carried out on a selected case study of a mass-asymmetric multi-storey frame building led the authors to highlight conceptual gaps of both OPCM3431 and EC8 seismic codes in the following general issues related to the evaluation of asymmetric-plan buildings:

- the estimation of the irregularity level of the seismic behaviour of the building;
- the use of the ρ_i uniformity as a parameter evaluating the regularity in the resistance distribution and identifying the range of applicability of linear analyses;
- the evaluation of the number of accelerograms to be used in nonlinear time-history analyses;
- the modification procedure of the pushover method to account for the torsional behaviour.

Based on the results obtained in the investigations, the following suggestions are proposed by the authors for improving the code prescriptions:

- the irregularity produced by the asymmetric elastic properties of the building could be measured by evaluating the ratio between the contribution $V_{b,1}$ given by the first coupled mode in one direction and the total seismic shear $V_{b,tot}$ acting at the base of the structure in the same direction; this parameter, which can be calculated with a simple modal response spectrum analysis, can measure both the coupling level of the building behaviour and the influence of the earthquake properties on the significance of the higher modes effects;
- when significant nonlinearity is expected (*e.g.*, as proposed by OPCM3431, if $\rho_i \geq 2$), regardless the uniformity of the estimated inelastic demand distribution (as measured by ρ_{max}/ρ_{min}), the building should be evaluated with a nonlinear analysis; apart from the regularity of the resistance distribution with respect to the seismic demand, mass and stiffness asymmetric configurations produce an irregular response, namely an irregular evolution of the nonlinear damage that cannot be evaluated with a simple linear analysis;

- the minimum number of accelerograms requested in the nonlinear time-history analyses should be larger than that prescribed by the code for regular symmetric-plan buildings; in fact, unlike symmetric systems, those asymmetric in plan show a response shape significantly sensitive to the exciting record; however, the authors believe that further investigations are still needed to define such minimum number;
- more exhaustive explanations on how to modify the pushover analysis results based on the elastic analyses should be given by the code when dealing with response quantities that do not depend linearly on the displacements (*e.g.*, the element forces or the chord rotations) need to be evaluated.

A possible procedure for selecting the analysis type based on the first two points listed above, is shown as a flow-chart in Figure 7.

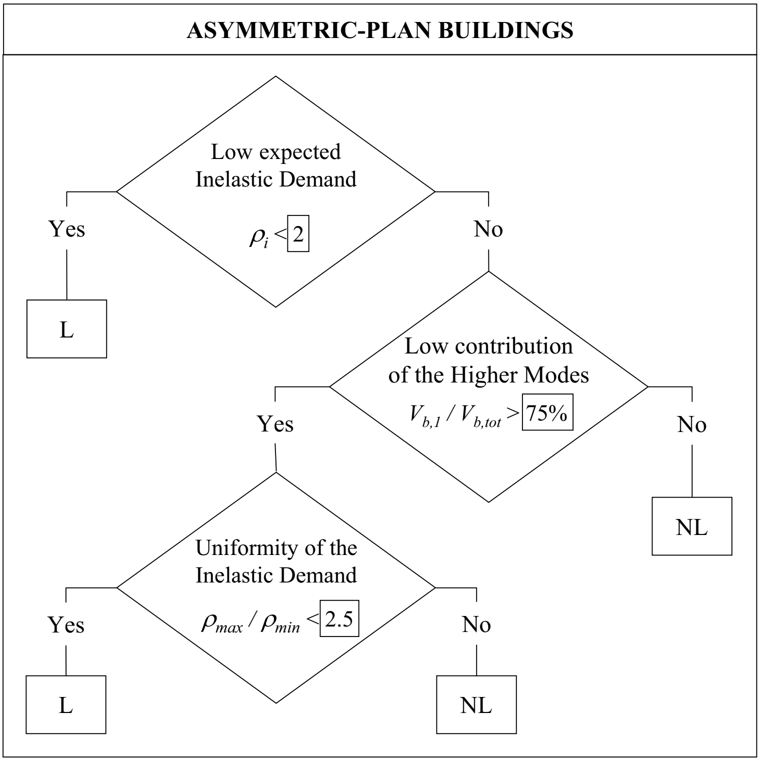

Figure 7. Proposal of requirements for selecting linear (L) or nonlinear (NL) analysis methods in the evaluation of asymmetric-plan buildings.

5 ACKNOWLEDGEMENTS

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AN UPDATED MODEL OF EQUIVALENT DIAGONAL STRUT FOR INFILL PANELS

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ABSTRACT
Recently the interest of the scientific community in infills in framed structures has increased, recognising their non-negligible effects on the primary structure. Although different codes, as a reflex, stress the need to take the infills into account in the analysis of framed structures, often this invitation does not correspond to proper and detailed code rules. In this connection, in this paper, the European and American codes are discussed, stressing that a defined and appropriate approach to this problem is missing in them. Further, a simplified tool for the evaluation of the effects of the infills is suggested, to be adopted by technical codes and used in practical applications. This approach, recalling the well-known equivalent strut model, is based on the usual assumptions, but considers some aspects of the problem of the frame-infill interactions, which are recognised as basic in the literature but not included in the available models.

KEYWORDS
Infills, framed structures, equivalent pin-jointed strut.

1 INTRODUCTION

Today it is generally accepted that infills modify the behaviour of framed structures under lateral loads; nevertheless, infills are generally neglected in structural analysis, and this can lead to an unreliable evaluation of the response in several cases.
Considering only the principal effects, it can be observed that: - the infills modify the lateral stiffness of the infilled frames, and, consequently, the modal characteristics of a framed building, producing a change in the base excitation-structure interactions; - a non-uniform infill distribution in plan and in elevation can produce torsional effects and soft storeys, respectively, with possible premature collapse in the case of a severe earthquake; - infills can produce increased shear stresses on the columns that may modify the mode of collapse due to reduction in structural ductility.
Although the negative effects of infills are generally stressed, it is also true that uniformly distributed infills give a lateral strength contribution, which can prove to be useful in the case of weak frames, not designed following seismic capacity criteria.
To sum up, infills have to be considered in structural analysis, because they can change the safety level of bare structures, making apparently safe structures unsafe and apparently unsafe structures safe.
A successful way to take infills into account in framed structures is based on their substitution with a couple of equivalent pin-jointed struts, alternatively effective in relation to the

direction of the lateral loads. This approach is also suggested by some technical codes, but the problem of the definition of the strut characteristics is not adequately treated (the definition of the geometrical and mechanical characteristics of this equivalent element is a question that each designer must solve before a reliable structural analysis is carried out) or non-updated models are proposed, which do not take into account recent advances in research on this topic.
If, on one side, technical codes give insufficient or inappropriate indications, on the other side it is not possible to find a univocal approach to the problem of infills in the literature. By contrast, one can find several approaches, based on different analytical, numerical or experimental investigations, leading to very different results.
For example, by using the historically first approach proposed by Stafford Smith, 1966, for evaluating the width of the equivalent strut, one finds a value that is very different from those deducible from subsequent researches (e.g. Stafford Smith and Carter, 1969, Mainstone, 1971, 1974, Klingner and Bertero, 1978, Durrani and Luo, 1994). Many more details on this aspect can be found in Papia *et al.*, 2003.
This discrepancy is due to the fact that the stiffening effect of an infill depends on many parameters (mechanical characteristics of the infill, stiffness ratio between frame and infill, etc) that have usually been considered, but it is also influenced by some other parameters, whose effect has been recognised by the scientific community but is not included in the proposed models: axial stiffness of columns of the frame, vertical load occurring after the construction of the infills, transverse strain ratio of the material constituting the infill panel.
Stafford Smith, 1966, first stressed the effect of vertical loads, on the basis of experimental observations; nevertheless, his calculus tool does not allow a generalized model including this effect. Subsequently, a specific resolution adopted during the NCEER workshop on Seismic Response of Masonry Infills (1994) stated that vertical loads have to be taken into account for the identification of the equivalent strut. In Papia *et al.*, 2004 and Amato *et al.*, 2008a the effect of vertical loads is once again discussed together with those produced by the further parameters mentioned above, previously considered in Papia et *al.*, 2003.
In this framework, this paper proposes an updated general model of pin-jointed equivalent diagonal strut, which could be utilized for practical applications and suggested by technical codes, by virtue of its simplicity of use.

2 PROVISIONS GIVEN BY EC8 AND FEMA 356

As mentioned above, many technical codes recognise the need to take the infills into account in the evaluation of the response of infilled frames. Eurocode 8 and FEMA 356 can be mentioned among the most representative codes for the extension of the territories in which they are in force and because they are considered a reference for many others codes. For this reason their content regarding the effect of infills will be commented on here.

2.1 Eurocode 8

In the section devoted to modelling in structural analysis of the last version of Eurocode 8 (UNI EN 1998-1:2004), one reads: *infill walls which contribute significantly to the lateral stiffness and resistance of building should be taken into account.*
Then, in the section regarding irregularities in plan, it is stated that: *infills should be included in the model and a sensitivity analysis regarding the position and the properties of the infills should be performed.* Then, with reference to non uniform distribution of infills in elevation, *if a more precise model is not used*, one can calculate the seismic action effects on columns by amplifying them by a magnification factor.

Hence, many times the use of a reliable model is recommended. Nevertheless, no models for the infill are included in Eurocode 8 as a support for practical applications, leaving designers free in choosing a criterion for modelling infills and identifying the complex frame-infill interactions.

2.2 FEMA 356

Unlike Eurocode 8, the Federal Emergency Management Agency (FEMA) code 356 explains clearly enough how to take infills into account: the effect of infills has to be considered by a FEM analysis or, alternatively, by introducing a diagonal pin-jointed strut equivalent to the infill.

For the first option no more is said, unlike the second one, which is derived from an experimental observation: under lateral forces the frame tends to separate from the infill near the windward lower and leeward upper corners of the infilled mesh.

For FEMA 356 the equivalent strut is to have the same thickness and modulus of elasticity as the infill panel (but it is not clear along which direction the modulus of elasticity must be calculated) while the width w is given by the following equation

$$\frac{w}{d}=0.175\left(\lambda'h'\right)^{-0.4} \tag{1}$$

where

$$\lambda'=\sqrt[4]{\frac{E_d\, t\, \sin(2\theta)}{4\, E_f\, I_c\, h}} \tag{2}$$

in which t is the thickness of the infill, h and ℓ are its height and length, respectively, $\theta = \text{atan}\ (h/\ell)$, I_c is the moment of inertia of the columns, h' is the height of the frame, measured between the centrelines of the beams; E_d and E_f are Young's modulus of the infill and of the material constituting the frame, respectively.

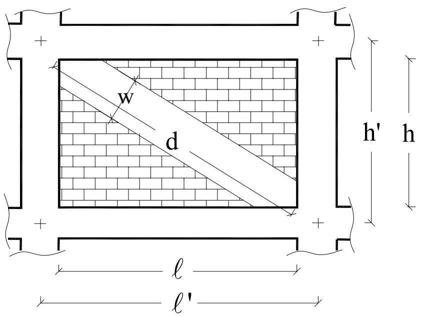

Figure 1. Geometric characteristics in Eqs. (1) and (2).

Eq. (1) was proposed in Mainstone, 1974, to identify the mean lateral stiffness of the infilled frame before the cracking of the infill.

The equivalent strut can be modelled as a concentric element connecting the intersections between beams and columns. This scheme does not give evidence of the local effects produced by the infill on beam and column regions near the nodes. For this reason, although the force acting on the strut is evaluated with concentrically located struts, it must be considered to be acting eccentrically, in agreement with the schemes in Figure 2.

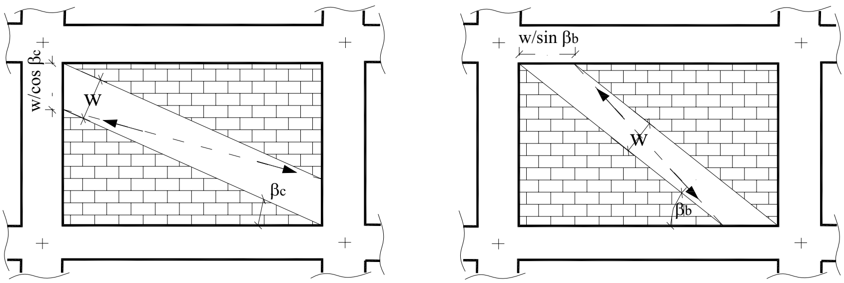

Figure 2. Schemes with eccentric reaction of strut.

Finally, the strength of the strut can be simply obtained as a projection of the shear strength of the infill, as specified at section 7.5.2.2 of FEMA 356.
Although FEMA gives many more details about the model to be used for the infills, some aspects of the frame-infill interaction are not treated: for example, the effect of vertical loads, which modify the capacity of the infills to stiffen the surrounding frames because of a different length of contact between frame and infill under a lateral load (Figure 3). This effect should lead to a greater value of w.

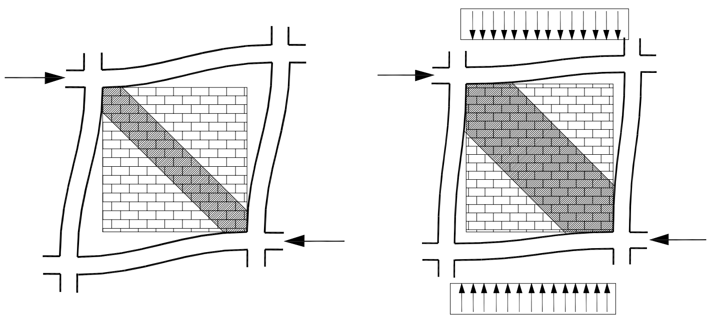

Figure 3. Influence of vertical load on effectiveness of infill.

Another aspect of the problem, which has been observed by means of FEM analyses and experimental tests, is the influence of the transverse strain ratio (Poisson's ratio) of the material constituting the infill. High values of this parameter produce an extension of the frame-infill contact region because of the transverse dilatation of the diagonally compressed infill panel. It follows that the width of the equivalent diagonal strut should be higher.
As a further remark, the modulus of elasticity of the infill inserted in Eq. (2) should be referred to the diagonal direction. The evaluation of this modulus in practical applications is a problem that should be specifically addressed.
What has been said highlights that the model assumed by FEMA is not wholly appropriate, since the calculus tools that were available when Mainstone formulated the model were not so

powerful as the present ones. Therefore, the use of modern calculus tools should be encouraged for an improvement of the models proposed by the codes in force.
In the next sections, an attempt is made to show the inappropriateness of Mainstone's model and an implemented model is proposed for practical applications.

3 PROPOSAL FOR IDENTIFYING EQUIVALENT PIN-JOINTED STRUT

A tool for the identification of the strut equivalent to the infill belonging to the generic mesh of a framed structure can be simply obtained by extending the study of a single-bay single-storey infilled frame.
Referring to this structural system, the section of the equivalent pin-jointed strut can be identified by imposing the condition that the initial stiffness of the actual system is equal to the initial stiffness of the equivalent braced frame (Figure 4).

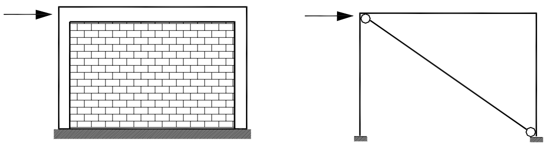

Figure 4. Actual and equivalent schemes of infilled single-bay single-storey frame.

It can be shown that the stiffness of the equivalent braced frame can be expressed as

$$D_i = \frac{k_d \cos^2\theta}{1 + \frac{k_d}{k_c}\text{sen}^2\theta + \frac{1}{4}\frac{k_d}{k_b}\cos^2\theta} + 24\frac{E_f I_c}{h'^3}\left(1 - 1.5\left(3\frac{I_b}{I_c}\frac{h'}{\ell'} + 2\right)^{-1}\right) \tag{3}$$

where k_d, k_c, k_b are the axial stiffnesses of the equivalent strut, of the column and of the beam, respectively:

$$k_d = \frac{E_d\, t\, w}{d}; \quad k_c = \frac{E_f A_c}{h'}; \quad k_b = \frac{E_f A_b}{\ell'} \tag{4}$$

In Eq. (4), I_b and A_c are the moment of inertia and the area of the cross-section of the columns, while A_b is the area of the cross-section of the beam.
The stiffness $\tilde{D}_i$ of the actual system can be evaluated by a FEM analysis taking into account the level of the vertical load and the frame-infill interaction along the contact surface. In the present work the contact surface was modelled by using finite elements with no tensile strength and able to transmit frictional shear stresses proportional to the compression stresses. In this way it was possible to include the effects of the frame-infill detachment produced by the lateral load.
The value of the width w can be calculated by imposing the condition that

$$D_i(w) = \tilde{D}_i \tag{5}$$

Repeating this procedure for different characteristics of infill and surrounding frame a set of widths can be obtained.
It was recognised that these data can be related to a parameter, denoted as λ^*, which depends on some mechanical characteristics of frame and infill and has a similar meaning to the parameter λ' introduced by Mainstone (the details about the identification of λ^* as a parameter can be found in Papia *et al*, 2003). Further relations can be obtained as a function of the level of vertical load, of Poisson's ratio and of the aspect ratio of the infill.
Considering that the parameter λ^* is expressed by

$$\lambda^* = \frac{E_d}{E_f} \frac{t\,h'}{A_c} \left(\frac{h'^2}{\ell'^2} + \frac{1}{4} \frac{A_c}{A_b} \frac{\ell'}{h'} \right) \tag{6}$$

and that the level of the vertical load F_v is expressed by the dimensionless parameter

$$\varepsilon_v = \frac{F_v}{2 A_c E_f} \tag{7}$$

the numerical experimentation gave the results partially contained in Figure 5.

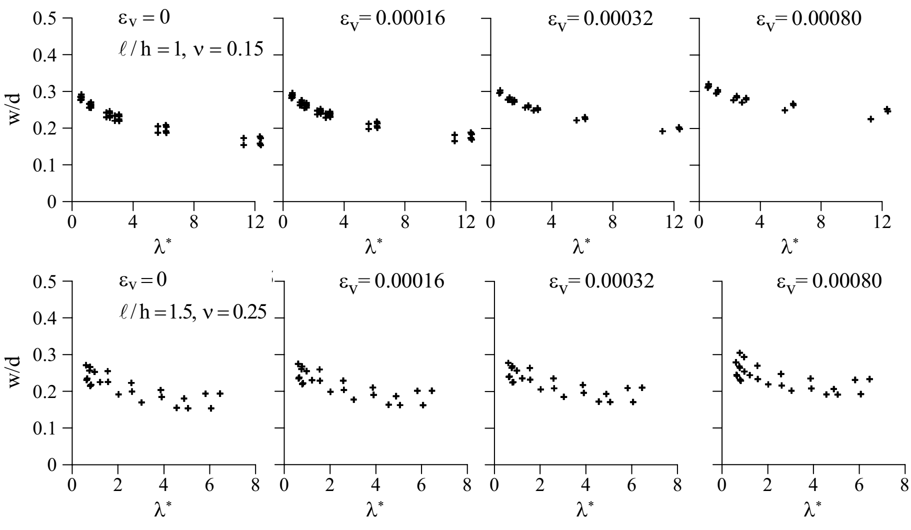

Figure 5. Dimensionless width of equivalent strut cross-section for different infilled frames.

The results highlight the effect of the vertical loads and of Poisson's ratio on the behaviour of an infilled frame and, consequently, on the identification of the width of the equivalent strut cross-section.
The points that were obtained numerically can be fitted by the following equation:

$$\frac{w}{d} = k \frac{c}{z} \frac{1}{\left(\lambda^*\right)^{\beta}} \tag{8}$$

where c and β depend on Poisson's ratio ν of the infill masonry along the diagonal direction,

$$c = 0.249 - 0.0116\ \nu + 0.567\ \nu^2 \tag{9}$$

$$\beta = 0.146 + 0.0073\ \nu + 0.126\ \nu^2 \tag{10}$$

the parameter z is a function of the aspect ratio of the infill,

$$z = 1 + 0.25\left(\ell / h - 1\right), \quad 1 \le \ell / h \le 1.5 \tag{11}$$

while k is a function of ε_v and λ^*,

$$k = 1 + \left(18\lambda^* + 200\right)\varepsilon_v \tag{12}$$

Figures 6-7 show the curves obtained by Eq.(8) for two values of Poisson's ratio and of the aspect ratio of the infill.

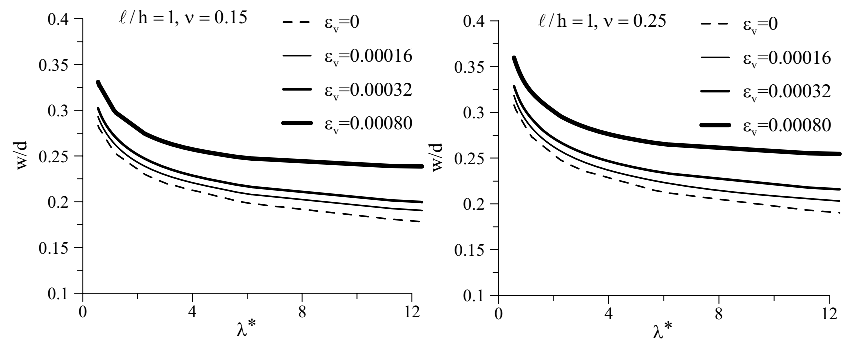

Figure 6. Curves of Eq.(8) for two values of Poisson's ratio and ℓ/h =1.

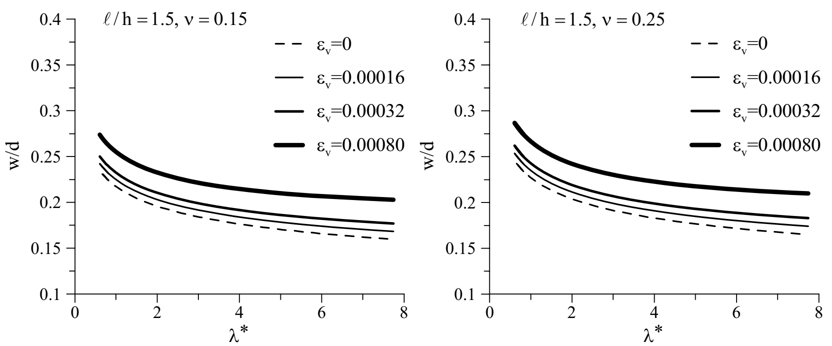

Figure 7. Curves of Eq.(8) for two values of Poisson's ratio and ℓ/h =1.5.

A comparison was made between the results of the numerical investigation described here and the results obtainable by means of Mainstone's model (Eq. (1)).

For this comparison, because of the different expression of λ' and λ^*, an *ad hoc* procedure was followed: once an infilled frame was fixed, the values of λ' and λ^* were calculated; then, the corresponding value of w/d was obtained by using Mainstone's model, and this value of w/d was plotted versus the parameter λ^*. The above sequence was repeated for different infilled frames.

Figure 8 shows the results of this procedure. It must be observed that the effect of vertical loads, not included in the model proposed by Mainstone, 1974, has not been considered.

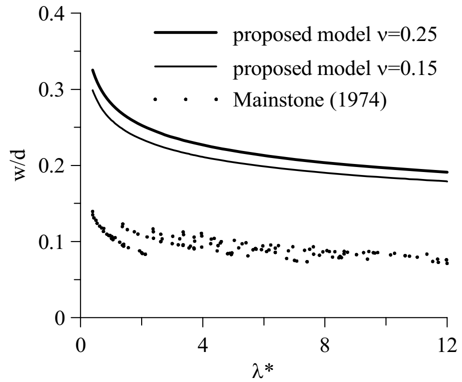

Figure 8. Comparison between proposed model and model adopted by FEMA 356 (ℓ/h =1).

By observing Figure 8, one notes that the values of w/d obtainable using Mainstone's model are much lower than those obtainable with the model proposed here. A similar underestimation of the width of the equivalent strut can be found after a comparison with some other models available in the literature, as discussed in Papia *et al*, 2003.

The distance between Mainstone's model and the model proposed here increases when a level of vertical loads different from zero is considered.

4 VALIDATION OF PROPOSED MODEL

Some experimental tests made it possible to validate the model that is proposed here. Single-bay single-storey infilled frames were tested for the measure of their lateral stiffness. The specimens were subjected to a constant value of vertical load (400 kN) and to a monotonically increasing lateral force. Figure 9 shows two specimens during the test.

In Table 1 some geometric and mechanical characteristics of the specimens that were tested are included, while the results in Table 2 allow comparison of the numerical and experimental values.

The experimental values of the lateral stiffness were calculated considering the first phase of the response, in which elastic behaviour was observed. The values of the elastic modulus and Poisson's ratio of the infill along the diagonal direction were obtained starting from the results of experimental tests on masonry samples and by applying the procedure proposed in Amato *et al*, 2008b.

Comparison shows good agreement between the experimental stiffness D_{is}, and the values of stiffness D_i obtained by the proposed model, confirming the reliability of the model itself.

Figure 9. Tests on frames with different kinds of masonry infills.

Table 1. Characteristics of specimens tested.

Frame	Kind of Infill masonry	E_f (kN/mm^2)	Cross-section of columns (mmxmm)	Cross-section of beam (mmxmm)	Height and width of infill (mm, mm)	Thickness of infill (mm)
1	calcarenite	28	200x200	200x400	1600, 1600	200
2						
3	clay brick		200x200	200x400	1600,1600	150
4						
5	lightweight concrete		300x300	300x400	1600, 1600	300
6						

Table 2. Lateral stiffness of infilled frames: numerical and experimental values.

Kind of Infill masonry	E_d (N/mm^2)	ν	λ^*	w/d	w (mm)	D_i (kN/mm)	Specimen	D_{is} (kN/mm)
calcarenite	4292	0.30	1.55	0.27	610	110	1	125
							2	111
clay tile	5303	0.07	1.43	0.25	565	122	3	128
							4	125
lightweight concrete	1795	0	0.40	0.29	655	127.5	5	115
							6	125

5 CONCLUSIONS

This paper shows that the influence of infills on the behaviour of infilled frames is not adequately treated by technical codes although infills may be basic for the structural response. In particular, while EC8 leaves the designer absolutely free, not giving specific instructions and models to be applied to include the presence of infills in the analysis model, FEMA 356 suggests the substitution of the infills with equivalent struts identified using the model proposed by Mainstone, 1974. This model fails to take some parameters into account, which

could substantially change the lateral response of an infilled frame; further, it seems to underestimate the stiffening effect of the infill.
In this context, confirming the criteria proposed by FEMA for the evaluation of the forces transmitted by the infill to the surrounding frame, a new model has been presented and proposed for identifying the equivalent strut, taking these parameters into account. It is the result of a numerical experimentation carried out with a FEM approach, in which the frame-infill interface has been adequately characterized mechanically.
The results obtainable with the proposed model were compared with the results of experimental test on infilled frames with different kinds of infills, confirming its reliability.
The proposed model makes it easy to identify the width of the equivalent strut thanks to an analytical expression containing the parameters mentioned several times.

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CAPACITY MODELS OF RC MEMBERS WITH EMPHASIS ON SUB-STANDARD COLUMNS WITH PLAIN BARS

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ABSTRACT
Current code prescriptions allow to evaluate the ultimate rotational capacity from hybrid (mechanical-empirical) or empirical formulations, for R.C. members with deformed bars and seismically detailed. These formulations can be extended to non-conforming elements by applying correction coefficients calibrated on experimental data. These coefficients, for members with plain bars, imply a reduction of 40% at least; this reduction increases if lapping of longitudinal bars is present. The experimental campaign conducted at the University of Naples on 16 real-scale columns with plain bars allowed to extend the experimental database for this typology. Experimental results highlight the excessive conservativeness of the current code proposal. Based on these results, new correction coefficients are proposed.

KEYWORDS
Existing RC building, non conforming, ultimate capacity, plain bars, lapping.

1 INTRODUCTION

The present Italian technical regulations (D.M. 2008), on a level with the most modern of international codes (CEN, 2005), allow us to determine the seismic capacity of existing RC buildings with recourse to non-linear analysis methodologies. The use of such methods of analysis, however, requires knowledge of the real post-elastic rotational capacities of each element of the construction (beams, columns) both in monotonic field, for non-linear static analysis, and in cyclical field, for non-linear dynamic analysis. In monotonic field, a series of parameters (yielding, peak resistance, ultimate state) has to be defined, in order to define the response curve of the element. In cyclic field, hysteretic rules and strength and stiffness degradation models have to be defined; they significantly influence the assessment of ultimate rotational capacity. Nevertheless, these rules are not easy to define, due to the number of geometrical and mechanical parameters and to the uncertainties involved. For example, the type of loading influences in a not negligible way the response of the r.c. element. Most of the code prescriptions only define the deformation capacity at the elastic limit (yielding) and at ultimate (collapse); therefore, based on these prescriptions, it is not possible to completely

define the strength degradation of the monotonic envelope, nor the hysteretic behaviour through appropriate rules.
Generally, deformation at yielding is evaluated as a chord rotation, accounting for different contributions corresponding to bending, shear and fixed-end rotation deformation mechanisms.
The rotational capacity is generally evaluated referring to a fixed strength decay (20%) respect to the peak resistance, evaluated on the envelope force-displacement curve. It is clear that this definition is strongly influenced by the maximum resistance condition, as well as the post-peak degradation, monotonic or cyclic. It is difficult to define a relationship between the element parameters and the rotational capacity, due to the complex phenomena influencing the post-elastic deformation behaviour and to the natural variability affecting this phenomena.
The code, consistently with the methodologies developed in literature, proposes two main approaches: a mechanical-empirical approach, based on plastic hinge length concept, and a purely empirical approach.
Referring to the purely empirical formulation proposed in (CEN, 2005), in the present work, based on experimental data, the applicability of this formulation to under-designed elements with plain bars is evaluated. In particular, correction coefficients applied to the code formulation are proposed for elements with plain bars, with or without lapping of longitudinal reinforcement.

2 EVALUATION OF ULTIMATE CHORD ROTATION

In this section, the theoretical background of current European code (CEN, 2005) formulas for the ultimate rotational capacity of reinforced concrete members is presented. Principles and methodologies standing behind the two main approaches to the assessment of this value (mechanical and empirical) are introduced.

2.1 Code formulas (EC8 part 3.3)

Eurocode 8 – Part 3 at section A.3.2.2 (Limit state of near collapse) provides expressions for the evaluation of ultimate element capacity of R.C. elements. The value of total chord rotation capacity under cyclic loading, following a mechanical approach, is given by [EC8 - Eq. (A.1)]:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016 \cdot (0.30^{v}) \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c \right]^{0.225} \left(\frac{L_V}{h} \right)^{0.35} 25^{\left(\alpha\rho_{sx} \frac{f_{yw}}{f_c} \right)} (1.25^{100\rho_d}) \tag{1}$$

where γ_{el}, equal to 1.5 for primary seismic elements and to 1.0 for secondary seismic elements, is meant to convert mean values of chord rotation to mean-minus-one-standard-deviation ones. The code also provides another expression for the evaluation of the plastic part of the ultimate chord rotation [EC8 - Eq. (A.3)]:

$$\theta_{um}^{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0.0145 \cdot (0.25^{v}) \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} \right]^{0.3} f_c^{\,0.2} \left(\frac{L_V}{h} \right)^{0.35} 25^{\left(\alpha\rho_{sx} \frac{f_{yw}}{f_c} \right)} (1.275^{100\rho_d}) \tag{2}$$

In this expression the coefficient γ_{el} equal to 1.8 for primary elements and to 1.0 for secondary ones. To evaluate the total chord rotation, the plastic part calculated according to this formula should be added to the yielding rotation [EC8 - Eq. (A.10)].
The values of chord rotation calculated according to (1) and (2) apply to elements with deformed bars, seismically detailed and without lapping of longitudinal bars in the vicinity of the end region where yielding is expected (plastic hinge region).
The correction coefficient applied to members with deformed bars without seismic detailing is equal to 0.825 for both formulas. If the longitudinal deformed bars are lapped, expressions (1) and (2) should be applied doubling the mechanical compression reinforcement ratio (ω'). Moreover, if the lap length is less than the minimum value $l_{ou,min}$:

$$l_{ou,min} = d_{bL} f_{yL} / \left[(1.05 + 14.5 \alpha_1 \rho_{sx} \frac{f_{yw}}{f_c}) \sqrt{f_c} \right] \tag{3}$$

another reduction factor equal to $(l_o / l_{ou,min})$ should be applied, calibrated only for expression (2), that is only for the plastic part of chord rotation. Corrections applied to the yielding chord rotation are given at section A.3.2.4(3) of the code; they are omitted here for the sake of brevity.
In elements with plain bars the chord rotation evaluated according to (1) should be multiplied by 0.575, while the plastic part of chord rotation given by (2) should be multiplied by to 0.375. It's worth noting that both coefficients already include the reduction factor equal to 0.825, accounting for the lack of seismic detailing. If longitudinal bars are lapped in members with plain bars, another coefficient has to be adopted, depending on the lap length (l_o) and the shear span (L_V). For total chord rotations, it is given by:

$$0.0025(180 + \min(50, l_o / d_{bL}))(1 - l_o / L_V) \tag{4}$$

while for the only plastic part it is:

$$0.0035(60 + \min(50, l_o / d_{bL}))(1 - l_o / L_V) \tag{5}$$

Moreover, shear span in expressions (1) and (2) should be reduced by the lap length l_o, assuming that the ultimate condition is controlled by the region right after the end of the lap.
The ultimate rotation may also be calculated following an equivalent mechanical approach through the evaluation of the ultimate section curvature, assumed to be constant over the plastic hinge length, which is empirically calibrated. Hence, the ultimate rotational capacity may be evaluated according to [EC8 - Eq. (A.4)]:

$$\theta_{um} = \frac{1}{\gamma_{el}} \left(\theta_y + (\phi_u - \phi_y) L_{pl} \left(1 - 0.5 \frac{L_{pl}}{L_V} \right) \right) \tag{6}$$

Section curvatures at ultimate and at yielding are calculated based on the first principles, with the constitutive relationships given by Eurocode 2 (CEN, 2004). If the concrete confinement model given in 3.1.9 in Eurocode 2 is assumed, the plastic hinge length is equal to [EC8 - Eq. (A.5)]:

$$L_{pl} = 0.10L_V + 0.17h + 0.24\frac{d_{bL}f_y}{\sqrt{f_c}} \tag{7}$$

If the confinement model proposed by Eurocode 8 – part 3 is adopted, better representing the effects of confinement under cyclic loading, the plastic hinge length is given by:

$$L_{pl} = \frac{L_V}{30} + 0.20h + 0.11\frac{d_{bL}f_y}{\sqrt{f_c}} \tag{8}$$

For expressions (7) and (8) no correction factor accounting for the above mentioned deficiencies is given. Therefore, they should only be applied to members with deformed bars, seismically detailed and without lapping of longitudinal bars.

2.2 Mechanical approach: background theory

From a phenomenological standpoint, the plastic hinge region can be identified with the zone of the element where yielding of reinforcement and concrete crushing take place. The plastic hinge length used in the evaluation of the element rotational capacity is, instead, purely conventional. It only represents the length over which ultimate section curvature, assumed to be constant, is integrated, following an equivalent bending approach, to calculate the effective chord rotation including shear and fixed-end rotation contributions to the overall deformability of the member; the curvature is calculated based on Bernoulli's plane section assumption.

The plastic hinge length can not be evaluated based on a purely mechanical approach. As a matter of fact, based on section equilibrium conditions and full-interaction hypothesis, in a post-peak phase the curvature should increase only at the base section of the element ("zero length hinge problem)" (Daniell et al., 2008; Haskett et al., 2009). Moreover, a purely mechanical approach, leading to the evaluation of flexural deformability, would not account for other deformation mechanisms such as shear deformability and slippage of reinforcing bars from the connection element. These contributions are not negligible at all. Shear mechanisms may contribute in the overall post-elastic member deformability up to 30 % (Fenwick et al., 1993), whilst the end rotation due to the slippage of reinforcing bars may contribute up to 40 % (Sezen, 2002).

Therefore, researchers over years have empirically calibrated the plastic hinge length over which theoretical ultimate section curvature is integrated, aiming at achieving the best agreement with experimental values of ultimate chord rotation.

Following this approach, rotational capacity of an element may be expressed as:

$$\theta_u = \theta_y + (\phi_u - \phi_y)L_{pl} \tag{9}$$

where length L_{pl} is made up of three terms, corresponding to different deformation mechanisms:

$$L_{pl} = L_{pl,flex} + L_{pl,shear} + L_{pl,slip} \tag{10}$$

Table 1 reports main formulations that have been proposed over years, starting from the first fundamental work by Baker (Baker et al., 1956). These expressions show that the shear span L_S and the section depth h are the major variables influencing the plastic hinge length, while the term corresponding to fixed-end rotation is generally proportional to diameter and yielding strength of longitudinal reinforcement bars. First proposed formulations are mainly calibrated based on experimental tests on beam elements, therefore the fixed-end rotation contribution is not clearly evaluated. In recent formulations, calibrated also on column elements, this contribution is clearly represented instead.

Moreover, in (9) the ultimate condition is given in terms of curvature ϕ_u, depending, based on plane section hypothesis, on steel or concrete failure. Nevertheless, the evaluation of ultimate curvature is not easy or univocal, due to the influence of some aspects as concrete confinement, spalling of the concrete cover or buckling of compressive reinforcing bars. For example, the use of different confinement models may significantly influence the determination of the ultimate curvature, therefore the plastic hinge length can assume very different values.

The plastic hinge formulation proposed in (Panagiotakos et al., 2001) is the most interesting among the expressions presented in literature. It is based on an extensive experimental database, which will be discussed in the next paragraph.

Table 1. Empirically derived hinge lengths.

Reference	Hinge Length (L_{pl})
(Baker et al., 1956)	$k_1 k_2 k_3 \cdot (z/d)^{1/4} \cdot d$
(Mattock, 1964)	$\frac{d}{2} \cdot \left[1 + \left(1.14\sqrt{\frac{z}{d}} - 1\right)\left(1 - \left(\frac{q - q'}{qb}\right)\sqrt{\frac{d}{16.2}}\right)\right]$
(Corley, 1966)	$\frac{d}{2} + 0.2\frac{z}{\sqrt{d}}$
(Mattock, 1967)	$\frac{d}{2} + 0.05z$
(Park, 1982)	$0.4h$
(Priestley et al., 1987)	$0.08L_v + 6d_b$
(Paulay et al., 1992)	$0.08L_v + 0.022 d_b f_y$
(Panagiotakos et al., 2001)	$0.12L_v + 0.014\alpha_{sl} d_b f_y$ for cyclic loading $0.18L_v + 0.021\alpha_{sl} d_b f_y$ for monotonic loading
(Fardis, 2007)	$0.09L_v + 0.2h$ for cyclic loading $0.04L_v + 1.2h$ for monotonic loading

The ultimate chord rotation is given by:

$$\theta_u = \frac{\phi_y L_V}{3} + (\phi_u - \phi_y) L_{pl}\left(1 - 0.5\frac{L_{pl}}{L_V}\right) \tag{11}$$

and the plastic hinge length L_{pl} is given as a linear function of shear span L_V (bending contribution) and of the product $(f_y d_{bL})$ (fixed-end contribution):

$$L_{pl} = \alpha L_V + \beta (f_y d_{bL}) \tag{12}$$

Coefficients α=0.12 e β=0.0014 are derived from a regression analysis on experimental data from cyclic tests. The ultimate curvature ϕ_u is evaluated accounting both for the concrete confinement and for the spalling of the concrete cover. The mean and median of the experimental-to-predicted ratio for expression (11), using (12), are equal to 1.23 and 0.99 respectively, with a Coefficient of Variation (CoV) of 83%.
The last plastic hinge expression proposed by (Fardis, 2007), based on a more extensive experimental database, is depending not on the shear span L_V but also on the height h of the section. Moreover, the fixed-end rotation contribution is evaluated with a separate term:

$$\theta_u = \theta_y + (\theta_{u,slip} - \theta_{y,slip}) + (\phi_u - \phi_y)L_{pl}\left(1 - 0.5\frac{L_{pl}}{L_V}\right) \tag{13}$$

with:

$$L_{pl} = 0.09L_V + 0.20h \tag{14}$$

where:

$$\theta_y = \frac{\phi_y L_V}{3} + 0.0013\left(1 + 1.5\frac{h}{L_V}\right) + \frac{\phi_y d_{bL} f_y}{8\sqrt{f_c}} \tag{15}$$

$$\theta_{y,slip} = \frac{\phi_y d_{bL} f_y}{8\sqrt{f_c}} \tag{16}$$

$$\theta_{u,slip} = \frac{\phi_u d_{bL} f_y}{16\sqrt{f_c}} \tag{17}$$

The use of the illustrated relationships, together with the confinement model showed in the same work, leads to an experimental-to-predicted ratio with mean and median, on a database of 1307 experimental tests, equal to 1.105 and 0.994 respectively, with a CoV of 53.6%.
Expressions (11) e (13), although providing a different evaluation of the fixed-end contribution, present the same control variables of the code expression (6), which directly shows, in the calculation of plastic hinge length, the dependence on all the above mentioned parameters.

2.3 Empirical approach: background theory

Formulas for the evaluation of rotational capacity can also be obtained with a totally empirical approach, based on experimental data, with pure numerical regression analyses. Different empirical expressions are proposed in literature (Rossetto, 2002; Zhu et al, 2007); among them, the expression proposed in (Panagiotakos et al., 2001) is certainly based on the most extensive database. Therefore, it is the most representative and it represents a reference for code formulas (CEN, 2005).
This experimental database consists in 633 cyclic tests and 242 monotonic tests on beams, columns and walls, which do not present brittle failure mechanisms. The relationship is a linear regression of the logarithm of θ_u on the control variables or their logarithms, without

coupling. Only control variables which turn out to be statistically significant for the prediction of θ_u are retained. Separate regression analyses for monotonic tests and for cyclic ones are performed. To obtain a more representative experimental database, with particular regard to members with unsymmetric reinforcement well represented in monotonic tests, another regression analysis on all 875 tests is performed, carrying to the following expression:

$$\theta_u = \alpha_{st} \cdot \alpha_{cyc} \cdot \left(1 + \frac{\alpha_{sl}}{2.3}\right)\left(1 - \frac{\alpha_{wall}}{3}\right) \\ (0.20^{v})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.275} \left(\frac{L_V}{h}\right)^{0.45} 1.1^{\left(100\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.30^{100\rho_d}) \tag{18}$$

where α_{cyc} is a binary coefficient given equal to 1 for monotonic loading and equal to 0.6 for cyclic loading. The ratio between the experimental ultimate rotation and the numerical value provided by (18) has mean equal to 1.06, median equal to 1.00 and CoV of 47%.
During years, together with the extension of the experimental database, the coefficients in this expression have been slightly modified. The last proposal, given in (Fardis, 2007), is based on 1307 monotonic and cyclic tests:

$$\theta_u = \alpha_{st} \cdot (1 - 0.43\alpha_{cyc}) \cdot \left(1 + \frac{\alpha_{sl}}{2}\right)\left(1 - \frac{3}{8}\alpha_{wall}\right) \\ (0.30^{v})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.25^{100\rho_d}) \tag{19}$$

where α_{st} is equal to 0.0185 for hot-rolled ductile steel, 0.0115 for heat-treated (tempcore) steel, and 0.0090 for cold-worked steel. The mean value of the ratio between the experimental ultimate rotation and the numerical value provided by (19) is 1.05, the median is equal to 0.995 and the CoV is of 42.8%. The comparison between the coefficients of variation clearly shows the better prediction capacity of (19), given by the growth of experimental knowledge state.
In the same work, a regression analysis for the only plastic part is also presented, which was already proposed in (CEB-FIB Bulletin 24, 2003) based on 1100 experimental tests. The expression is:

$$\theta_u^{pl} = \alpha_{st}^{pl} \cdot (1 - 0.52\alpha_{cyc}) \cdot \left(1 + \frac{\alpha_{sl}}{1.6}\right)(1 - 0.4\alpha_{wall}) \\ (0.25^{v})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.30} f_c^{\,0.20} \left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.275^{100\rho_d}) \tag{20}$$

The mean value of the ratio between the experimental ultimate rotation and the corresponding numerical prediction is 1.05, the median is equal to 0.995 and the CoV is of 42.7%, against the 47% in the first proposal (see Eq. 18).

Expressions (1) and (2) proposed in EC8 almost perfectly agree with (19) and (20), assuming $\alpha_{cyc}=1$ (cyclic loading), $\alpha_{sl}=1$ (with slip), $\alpha_{wall}=0$ (only beams and columns) e α_{st} e α_{st}^{pl} equal to 0.0185 (hot-rolled ductile steel).
Consistently with the characteristic of tests included in the experimental database, the proposed expressions for the ultimate rotational capacity should be applied only to members with deformed bars, with seismic detailing and without lapping of longitudinal bars in the vicinity of plastic hinge region, that is, to members which are not representative of existing buildings. Authors define correction coefficients allowing to extend the use of these expressions to members with different characteristics. These coefficients are calibrated to counterbalance the mean error evaluated through the comparison between values from expressions (19) and (20) and results of experimental tests on under-designed members, not included in the original (primary) database. This approach, certainly approximated, is necessary because of the small number of experimental data for these members. Because of the low number of these data, it seems to be allowed to suppose that their inclusion in the database would have not led to any significant change in the regression expression. Moreover, applying the primary expression to members of different typologies, only using a multiplicative coefficient, is the same as postulating that the ultimate rotation depends on the control parameters by the same way, independently on the specific characteristics of considered elements. Nevertheless, the assumed methodology seems to be the only one that can be followed, due to the few experimental data now available for this kinds of elements. A higher reliability can be obtained only by extending the experimental database, so that a wider range of loading conditions and geometrical and mechanical characteristics can be covered. In Table 2 correction coefficients and the extension of the corresponding experimental database used for calibrations are reported.

Table 2. Correction factors for non-detailed members.

Element Type	Correction Factor	# of Data	Mean - Median CoV	Reference
w/o seismic detailing and continuous ribbed bars	0.85	27	0.81 - 0.85 42%	(Panagiotakos et al., 2002; CEB-FIB Bulletin 24, 2003)
w/o seismic detailing w/ hooked plain bars and w/ or w/o lap-splicing over plastic-hinge length	$0.015 \cdot \left(10+\min\left(40;l_o/d_b\right)\right)$	15	1.07 - 0.975 32%	(Fardis, 2006)

2.4 *Critical review*

The expressions for the ultimate rotational capacity, as clearly shown in the previous paragraphs, are necessarily calibrated on experimental data, due to the complex nature of mechanisms affecting the post-elastic behaviour of reinforced concrete members and their interaction.
Both the approaches presented in literature and in Code are characterized by high values of the coefficient of variation of the experimental-to-predicted capacity ratio.
The high CoV affecting expressions (19) and (20) – or (1) and (2) - is not only due to the natural experimental variability, but also to the difficulty in completely modelling with a simple formulation the interaction between the complex phenomena influencing the post-elastic deformation behaviour of reinforced concrete element. Panagiotakos e Fardis in (Panagiotakos et al., 2001), based on the analysis of subgroups of tests, homogenous for geometrical and mechanical characteristics and for loading conditions, quantify the CoV associated with the only natural variability in 12.5 %.

The limited prediction capacity of these expressions is also due to impossibility of introducing in the control variables some parameters which certainly affect the rotational capacity. The major among these parameters is the load path, that is the energy dissipated in hysteretic cycles. This aspect has been experimentally investigated by (Pujol et al., 2006), who analyzed the influence of displacement history on the decay of element resistance capacity. The experimental tests show that, given equal the geometrical and mechanical characteristics and the applied axial load (that is, all the input parameters of code and literature regression formulations), it is possible to *predetermine* the value of element chord rotation corresponding to a conventional drop of 20 % of peak resistance, by imposing a given load path (cfr. Figure 1).

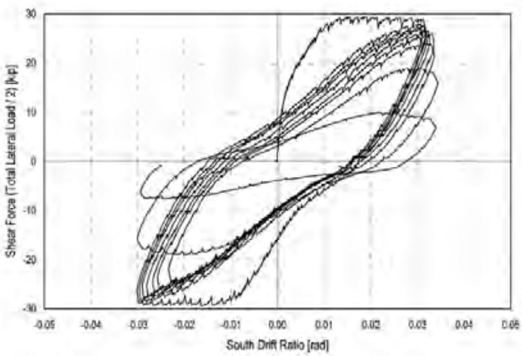


Figure 1. Influence of displacement history on ultimate chord rotation (Pujol et al., 2006).

Panagiotakos and Fardis, in the above mentioned work, try to explicitly account for the effect of cyclic loading by another regression, where the type of loading is evaluated with a variable expressing the equivalent number of inelastic imposed cycles ($\sum|\theta_i|\theta_u$), instead of the coefficient (α_{cyc}). Nevertheless, contrary to expectations, the inclusion of this parameter makes worse the prediction capacity of the formulation. The coefficient of variation (CoV) of the ratio between the experimental and the predicted value, in fact, increases up to 51 %. On the other hand, the usual structural modelling approaches do not allow to introduce the dissipated energy in the control variables.

A critical analysis of expressions (19) and (20), based on mechanical considerations regarding the absence of a direct relationship between the median estimation of the ultimate rotation and some parameters that certainly influence the member capacity, seems to be without foundation. Due to the purely statistical nature of the expression, in fact, the retaining of these variables turns out to be not significant because of their strong correlation with other parameters, already present in the formulation (Panagiotakos et al., 2001).

It's worth noting that the higher coefficient of variation affecting the hybrid mechanical-empirical formulation (plastic hinge length) with respect to the purely empirical one is probably related to the difficulty in expressing the ultimate rotation as a function of element characteristics based on a statistical regression analysis restrained to a mechanical relationship.

3 DEFORMATION CAPACITY OF RC MEMBERS WITH PLAIN BARS

Plain reinforcing bars have been widely used in the construction of European reinforced concrete buildings. In Italy and in the whole Mediterranean area, their use was widely spread up to 1970s, in north-American countries and in New Zealand construction with plain bars are present until 1950. The high spreading of reinforced concrete buildings with plain bars among existing buildings can be deduced if it is considered that 50 % of Italian existing buildings has been constructed between earliest 1940s and latest 1970s, when reinforced concrete structures with plain bars were the prevailing construction typology.

The correct evaluation of deformation capacity of R.C. elements has to account for the effective bond capacities between reinforcing bars and the surrounding concrete. For members with plain bars, low bond capacities directly influence the three main deformation mechanisms: bending, shear and fixed-end rotation.

As shown by experimental evidence, the scarce capacities of load transfer between the reinforcing bars and the surrounding concrete makes the deformation contribution associated with the fixed-end rotation effect very important. This contribution, in fact, due to the cyclic and post-elastic decay of bond capacities, may represent up to 80-90 % of overall deformability of the element (Verderame et al., 2008a; Verderame et al., 2008b).

Bond capacities also influence the development of cracks along the element. A lower number of wider cracks is observed when bond decreases. This greatly influences both shear and bending deformability, reducing the former and increasing the latter.

Therefore, formulations able to provide a reliable assessment of ultimate deformation capacity of elements with plain bars are of a particular interest for assessment of existing buildings.

The ultimate rotational capacities for members with plain bars, according to code, as already shown at paragraph 2.1, is evaluated by applying a correction coefficient to the capacity formulations calibrated on members with deformed bars and seismically detailed. In the following a new calibration of these coefficients, which result to be too conservative, is proposed, based on an experimental database of columns with plain bars extended with recent experimental results from tests executed in the laboratory of the Department of Structural Engineering at the University of Naples "Federico II", in the research project ReLUIS-DPC 2005-2008 Linea 2.

3.1 Experimental data set

Most of literature data about the experimental behaviour of R.C. elements comes from test executed on members with deformed bars. During last years, the need for a reliable assessment of seismic capacity of existing structures has produced an increasing number of experimental campaigns aimed at the study of behaviour of under-designed elements. This allowed to extend the experimental database for the calibration of correction coefficients applied to the regression relationships for the evaluation of ultimate rotational capacity.

Coefficients proposed in (CEN, 2005), reported at 2.1, are calibrated on very few experimental tests. Expression proposed in (Fardis, 2006), reported in Table 2, are, instead, based on 15 experimental tests; 9 of them without lapping ($l_o / d_{bL} = 100$) and 6 with a lap length l_o varying between 15, 25 and 40 times the diameter d_{bL} of longitudinal reinforcing bars. The comparison between the code correction coefficient and the latest one proposed in (Fardis, 2006) shows the considerable conservativeness of the prescription proposed by Eurocode 8.

In recent times, in the Department of Structural Engineering at the University of Naples "Federico II", a great attention has been addressed to the experimental study of members with plain bars, both through test aimed at the characterization of bond capacities in cyclic (Verderame al., 2009a; Verderame et al., 2009b) and post-elastic field (Verderame et al., 2008c) and through tests on real-scale columns elements under monotonic and cyclic loading. The first phase of the experimental activity 6 monotonic test e 6 cyclic ones have been performed, on elements with square section (300×300)mm^2, for different values of the applied axial load. In this phase particular attention has been addressed to the detail of longitudinal bars, by executing tests on elements without lapping of longitudinal bars at the base of the element and on elements with a lap length l_o equal to 40 times the diameter d_{bL} of longitudinal bars.

The second phase of the experimental campaign, just finished and still unpublished, is focused on the comparison between rotational capacity and deformation mechanisms of R.C. elements with plain and ribbed bars. In particular, 4+4 tests have been executed on elements equal for the geometry of the transverse section, the geometric ratio of longitudinal and transverse reinforcement, the axial load level and the load path, varying the geometry of the transverse section. The characteristics of tested elements are reported in Table 3, where the drift corresponding to the 20 % decay of the peak resistance is also given

Table 3. Geometrical and mechanical characteristics of tested elements.

n test	Reference	b [mm]	L_V/h	P/A_gf_c	ρ_l [%]	l_o/d_{bL}	reinforcement Type	loading	$\theta_{u,\,exp}$ (20%) [%]
1	Verderame et al., 2008	300	5.23	0.12	0.8	40	Plain	cyclic	6.23
2		300	5.23	0.12	0.8	40	Plain	cyclic	5.82
3		300	5.23	0.12	0.8	100	Plain	cyclic	6.49
4		300	5.23	0.24	0.8	40	Plain	cyclic	3.72
5		300	5.23	0.24	0.8	100	Plain	cyclic	3.81
6		300	5.23	0.24	0.8	100	Plain	cyclic	2.77
7		300	5.23	0.12	0.8	40	Plain	monotonic	6.83
8		300	5.23	0.12	0.8	40	Plain	monotonic	6.88
9		300	5.23	0.12	0.8	100	Plain	monotonic	10.72
10		300	5.23	0.12	0.8	100	Plain	monotonic	7.87
11		300	5.23	0.24	0.8	40	Plain	monotonic	7.87
12		300	5.23	0.24	0.8	100	Plain	monotonic	4.29
13	Reluis (2005-2008)	300	5.00	0.2	1.0	100	Plain	monotonic	8.53
14		300	5.00	0.2	1.0	100	Plain	cyclic	5.43
15		300	3.00	0.1	0.9	100	Plain	cyclic	5.27
16		500	5.00	0.1	0.9	100	Plain	cyclic	6.23
17		300	5.00	0.2	1.0	100	Ribbed	monotonic	6.86
18		300	5.00	0.2	1.0	100	Ribbed	cyclic	3.87
19		300	3.00	0.1	0.9	100	Ribbed	cyclic	3.65
20		500	5.00	0.1	0.9	100	Ribbed	cyclic	4.66

By adding these tests, the database for the evaluation of the correction coefficient applied to the ultimate rotational capacity of elements with plain bars consists of 26 tests, 7 of which monotonic. It's worth noting that tests (#1,2,3,5,6) were already included in the database used by Fardis for calibrating expressions reported in Table 2 (Fardis, 2006).

3.2 Calibration of correction factor

In this paragraph the correction coefficient applied to code expressions for the ultimate rotation of members with plain bars is calibrated, based on experimental data introduced at 3.1.

The correction coefficients will be calibrated according to the methodology already illustrated at 2.3, with regard to the following expression:

$$\theta_u = \theta_y + \theta_u^{pl} \tag{21}$$

with:

$$\theta_u^{pl} = 0.03 \cdot (1 - 0.52\alpha_{cyc})(0.25^{v})\left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)}\right]^{0.3} f_c^{\,0.2}\left(\frac{L_V}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}(1.275^{100\rho_d}) \tag{22}$$

It is to be noted that a factor accounting for the type of loading (α_{cyc}) has been added to the code expression (2) for the plastic part of the ultimate rotation θ_u^{pl}. This assumption is considered to be allowed because of the almost perfect agreement between the code expression and the one proposed in (Fardis, 2007), as already shown at 2.3.
Table 4 reports, for all experimental tests in the database, the ratios between experimental ultimate rotation and the corresponding theoretical value ($\theta_{u,exp}/\theta_u$), according to (22).

Table 4. Ratios between experimental ultimate rotations and corresponding theoretical values.

n test	Reference	loading	l_o/d_{bL}	$\theta_{u,exp}/\theta_{u,m}$
1	University of Patras	cyclic	15	0.33
2	University of Patras	cyclic	15	0.62
3	University of Patras	cyclic	25	0.39
4	University of Patras	cyclic	25	0.41
5	University of Patras	cyclic	100	0.58
6	University of Patras	cyclic	100	0.60
7	Other sources	cyclic	100	0.54
8	Other sources	cyclic	100	0.74
9	Other sources	cyclic	100	0.83
10	Other sources	cyclic	100	1.25
11	University of Naples (Verderame et al., 2008)	cyclic	40	1.26
12	University of Naples (Verderame et al., 2008)	cyclic	40	0.83
13	University of Naples (Verderame et al., 2008)	cyclic	40	0.60
14	University of Naples (Verderame et al., 2008)	cyclic	100	1.21
15	University of Naples (Verderame et al., 2008)	cyclic	100	1.13
16	University of Naples (Verderame et al., 2008)	cyclic	100	0.81
17	University of Naples (Verderame et al., 2008)	monotonic	100	0.69
18	University of Naples (Verderame et al., 2008)	monotonic	100	0.70
19	University of Naples (Verderame et al., 2008)	monotonic	100	0.92
20	University of Naples (Verderame et al., 2008)	monotonic	40	1.09
21	University of Naples (Verderame et al., 2008)	monotonic	40	0.80
22	University of Naples (Verderame et al., 2008)	monotonic	40	0.50
23	University of Naples (Reluis)	monotonic	100	1.20
24	University of Naples (Reluis)	cyclic	100	1.41
25	University of Naples (Reluis)	cyclic	100	1.42
26	University of Naples (Reluis)	cyclic	100	1.76

The ratio ($\theta_{u,exp}/\theta_u$) for members without lapping of longitudinal bars (conventionally reported as $l_o/d_{bL} = 100$) has mean equal to 0.99 and median equal to 0.87, with a CoV of 37%. Hence, based on the experimental database, it can be deduced that the assessment of the ultimate rotation with (22) overestimates the median value of rotational capacity by about 13%.

The use of expression (22) for members with lapping of longitudinal bars overestimates even more the experimental rotational capacity. A linear regression performed on the ratio ($\theta_{u,exp}/\theta_u$) gives the following expression for the correction coefficient:

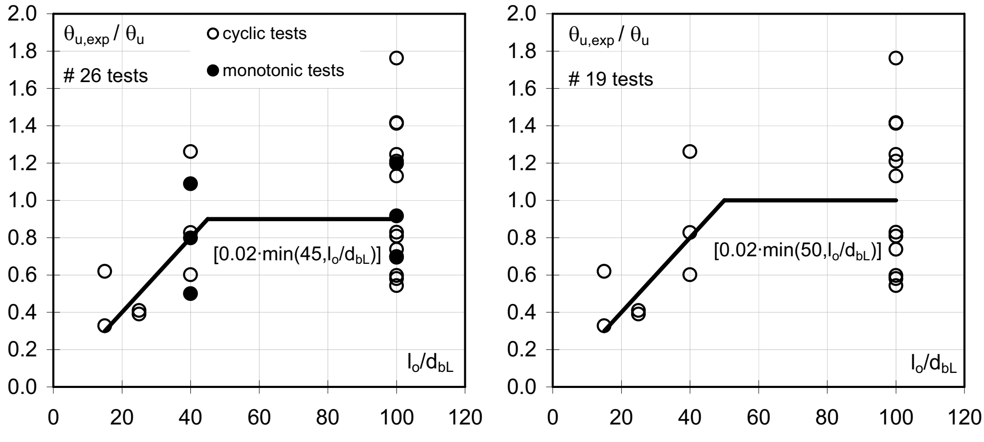
$$k = 0.020 \min(45, l_o / d_{bL}) \tag{23}$$

This coefficient, applied also to elements without lapping, allows to account for the overestimate of the rotational capacity given by (22); in particular, the rotational capacity evaluated according to (22) is reduced by 10%. The ratio [$\theta_{u,exp}/(k\theta_u)$], calculates on all tests in the experimental database, has mean equal to 1.10 and media equal to 1.01, with a CoV of 37%.

Figure 2a reports, for each experimental test, both cyclic and monotonic, the ratio between the experimental ultimate rotation and the corresponding theoretical value ($\theta_{u,exp}/\theta_u$), together with the correction coefficient given by (23), which should be applied to (22).

It is noted that including monotonic tests in the evaluation of the correction factor (k) is the same as postulating that the reducing of rotational capacity due to cyclic loading, evaluated in (22) by the coefficient $(1-0.52\alpha_{cyc})$, is, on average, not depending on bond capacities. As a matter of fact, this coefficient, as previously illustrated, is calibrated on a database made up of members with deformed bars; therefore, the evaluation of the correction coefficient (k) has been executed supposing that the reduction given by $(1-0.52\alpha_{cyc})$ can also be extended to members with plain bars.

Nevertheless, from a theoretical standpoint, a member with deformed bars, given equal the geometrical and mechanical characteristics, should show a higher cyclic degradation with respect to a member with plain bars, because of the micro-cracking of the concrete surrounding the reinforcing bar due to the higher bond performances, which emphasizes the strength degradation of concrete alternatively in compression and in tension.



(a) (b)

Figure 2. Proposed correction factor: (a) cyclic and monotonic tests, (b) only cyclic tests.

However, due to the uncertainties related to the inclusion of monotonic tests, the correction coefficient will now be calibrated based on the only cyclic tests. For these tests, the ratio ($\theta_{u,exp}/\theta_u$) for elements without lapping of longitudinal bars has mean equal to 1.02 and median equal to 0.98, with a CoV of 39%. Therefore, based on the experimental tests, expression (22) shows a very good agreement with the cyclic rotational capacity of elements with plain bars without lapping of longitudinal reinforcement.

A linear regression performed on the ratio ($\theta_{u,exp}/\theta_u$), for elements with lapping of longitudinal reinforcement, gives the following expression for the correction coefficient:

$$k = 0.020\min(50, l_o/d_{bL}) \tag{24}$$

Figure 2b reports, for each cyclic experimental result, the ratio between the experimental ultimate rotation and the corresponding theoretical value ($\theta_{u,exp}/\theta_u$), together with the correction coefficient given by (24), applied to (22).

3.3 Discussion of results

The extension of the experimental database allowed to re-calibrate the correction coefficients applied to the assessment of the ultimate rotational capacity of elements with plain bars, with or without lapping of longitudinal reinforcement.

The choice between the correction coefficient calibrated only on cyclic tests or on the whole experimental database is not easy. The numerical results, in fact, are rather different. Although the inclusion of monotonic tests in the database is affected by the previously highlighted uncertainties, it is to be noted that the consideration of the only cyclic tests would imply a further reduction in the extension of the experimental database, which is already limited.

However, both the expressions of correction coefficients proposed in the present work highlight the conservativeness of EC8 proposal, which is based on very few experimental tests. Moreover, EC8 assumes that, when lapping of longitudinal reinforcement is present, the ultimate condition is controlled by the region right after the end of the lap, so that the shear span and, therefore, the rotational capacity are further reduced, but this assumption is not confirmed by the experimental results. The highest plastic demand, in fact, always concentrates at the base section of the element.

Despite the difficulties in the choice of the most reliable expression for the correction coefficient, recent experimental results clearly highlight the higher rotational capacity of members with plain bars with respect to ones with deformed bars, equal for structural characteristics and details. As a matter of fact, the comparison between the ultimate rotations of the elements from the second phase of the experimental campaign, briefly illustrated at 3.1, highlight that the capacity of members with plain bars are higher, on average, by 35 % compared with the corresponding members with deformed bars (see Table 3).

From a mechanical standpoint, the higher ultimate rotational capacity of columns with plain bars may be explained with the comparison between two opposite mechanisms: the increase of deformability caused by the fixed-end rotation mechanism, particularly exalted due to the low bond capacities; on the other hand, the higher degradation of global resistance due to the increase of deformation demand on concrete in compression, localized at the base of the element and associated with the concentrated rotation (“rocking effect”). According to experimental evidence, the former seems to prevail on the latter, leading to an overall increase of ultimate rotational capacity compared with members with higher bond capacities.

4 CONCLUSIONS

In this work, the theoretical background to code formulas for the assessment of ultimate rotational capacity of reinforced concrete members has been briefly presented. Most recent literature contribution, together with advantages and deficiencies of the approaches to the calibration of this relationships, have been illustrated.
Special attention has been addressed to the calibration of correction coefficients used for the assessment of ultimate rotational capacity of under-designed elements, with emphasis on members with plain bars.
Main conclusions drawn form this work are:

- The evaluation of post-elastic deformation capacity of r.c. elements may only be based on experimental data; any mechanical approach would not allow to evaluate accurately the complex interaction phenomena influencing the deformability of the element.
- The reliability of regression expressions proposed in literature, some of which have been adopted in code, is a direct result of the extension and the correct sorting of the database.
- The estimate of rotational capacity of under-designed elements is strongly influenced by the low number of experimental data related to this typologies.
- Recent experimental tests on columns with plain bars, executed at the University of Naples (DIST), allow to extend significantly the database used for the calibration of correction coefficients applied to the assessment of these elements, with or without lapping of longitudinal reinforcement.
- The re-calibration of correction coefficients, even within the limits of the adopted methodology, has allowed to highlight the excessive conservativeness of current code prescriptions for elements with plain bars; this is confirmed by the experimental evidence, showing that the ultimate rotation of members with plain bars is higher compared with members with deformed bars, on average, by 35 %, given equal the structural characteristics and details.

5 ACKNOWLEDGEMENTS

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CAPACITY MODELS OF BEAM-COLUMN JOINTS: PROVISIONS OF EUROPEAN AND ITALIAN SEISMIC CODES AND POSSIBLE IMPROVEMENTS

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ABSTRACT
More reliable assessment procedures of existing RC buildings are currently available, and have been introduced in the Italian and European codes reporting new rules for seismic design and analysis. However, further studies are necessary in order to upgrade such procedures and, specifically, to test the effectiveness of the capacity evaluation methods relevant to beam-column joints. To this purpose, a literature review on the subject and a wide experimental program on exterior beam-column joint specimens were carried out in the framework of the DPC-ReLUIS Project (Research Line 2, Task NODI). Some results are reported in the present paper to highlighting the role of the key behavioural parameters of RC beam-column joints, thus giving useful suggestions on the reliability of current code expressions and on possible improvements.

KEYWORDS
Existing buildings, reinforced concrete, beam-column joint, capacity model.

1 INTRODUCTION

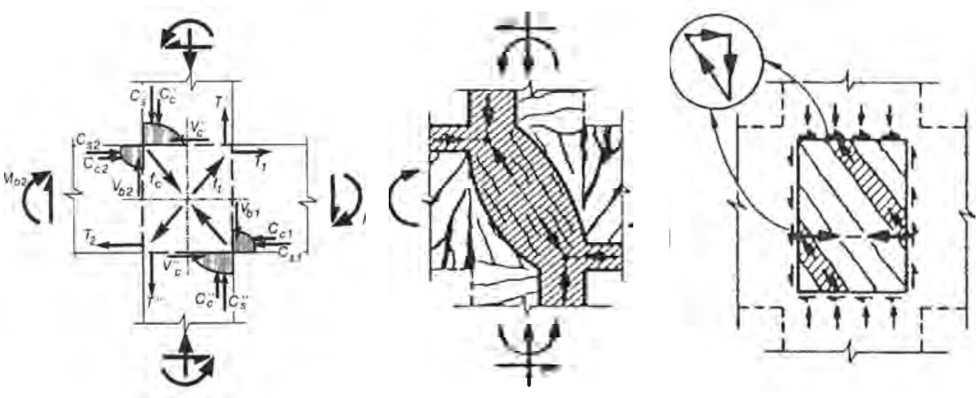
In the framework of DPC-ReLUIS 2005-2008 Research Project, a Research Line (RL 2) was devoted to the “Assessment and Reduction of Seismic Vulnerability of RC Existing Buildings”. In this RL, a task was specifically devoted to the “Behaviour and Strengthening of Beam-Column Joints”. In fact, in spite of a more reliable assessment of this type of structures, and simultaneously with the promulgation of new rules for seismic design and analysis in Italy and Europe, further work is necessary to evaluate the effectiveness of the capacity evaluation methods relevant to beam-column joints contained in the codes. To this purpose, the work was firstly devoted to an accurate literature review on the available capacity evaluation methods, pointing out the most appropriate ones to the Italian and European building stock. Then, such capacity models have been applied to test results deriving from experimental programs on beam-column joints reported in the technical literature, highlighting the models able to better predict the experimental results.

As for the ReLUIS experimental program, although the total number of tests up to now performed is not so high and is relevant only to exterior joints, some useful indications have been derived on the prediction capability of code expressions. This has been made comparing the real behaviour in beam-column joints detected during experimental tests, with respect to the conventional performances determined strictly applying the codes.

2 OVERVIEW OF CURRENT RESEARCH STATE

2.1 Capacity models

A reliable evaluation of strength and deformability of beam-column joints is a crucial aspect in the framework of performance-based design or evaluation of Reinforced Concrete (RC) buildings, as confirmed by recent experimental activities and damage observations from recent earthquakes. Nowadays, a large consensus has not been found on a single joint modeling technique neither in the scientific literature nor in the Codes, in spite of the fact that many research groups worldwide, during the last three decades, performed wide experimental and theoretical studies on this topic to evaluate the cyclic behavior of beam-column joints. As shown by many experimental programs, the failure of joint panels is induced usually by shear or bond flaws. The stress distribution due to flexural and shear forces transferred through the joint produces a wide diagonal crack pattern in the panel leading to a crushing failure of the compressed strut and consequently to strength and stiffness deterioration. The cyclic deterioration of bond performance, on one side, yields to reduced flexural strength and ductility of framing elements (Hakuto et al., 1999; Manfredi et al., 2008), while it yields, on the other, to a noticeable increase in the story drift (Soleimani et al., 1979).



(a) (b) (c)

Figure 1. Mechanisms of shear transfer at an interior beam-column joint: (a) Force from beam and columns acting on joint core; (b) Strut-mechanism; (c) Truss-mechanism; (Paulay et al., 1992).

In Figure 1a the forces acting on an interior beam-column joint panel are reported. The horizontal shear force V_{jh} is equal to:

$$V_{jh} = C_{s2} + C_{c2} + T_1 - V_c^{'} = T_1 + T_2 - V_c^{'} \tag{1}$$

because the horizontal equilibrium equation referred to the end section of the beam yields to:

$$C_{s2} + C_{c2} = T_2 \tag{2}$$

The vertical shear force can be similarly given by an equilibrium equation, but an accurate evaluation can be given by:

$$V_{jv} = \frac{h_b}{h_c} V_{jh} \tag{3}$$

where h_b is the beam depth and h_c is the column depth.
The shear transfer mechanisms allowing for the joint force transfer, after the diagonal cracking of the joint panel, are shown in Figure 1b-c (Paulay et al., 1992). In the first mechanism, named strut-mechanism, the joint shear is concentrated in a single compressed concrete strut. In this case the transverse steel reinforcement provides confinement to the concrete allowing for higher deformability of the strut, but only before steel yielding. In the second mechanism, named truss-mechanism, the portion of the shear force due to the bond stress along the longitudinal steel reinforcement inside the joint is in equilibrium with a truss mechanism given by concrete struts and vertical and horizontal ties corresponding to joint panel reinforcement. The shear capacity is given by the sum of the shear contributions according to these two mechanisms. It is worth noting that if bond deterioration occurs, the shear contribution due to the strut-mechanism increases, while the total shear force acting on the joint panel remains constant. Assuming a full deterioration of the bond capacity between steel bars and concrete, it is:

$$C_{s2} = -T_1 \tag{4}$$

so that the following equation is given, that is equal to (1):

$$V_{jh} = C_{s2} + C_{c2} + T_1 - V_c^{'} = -T_1 + T_1 + T_2 + T_1 - V_c^{'} = T_1 + T_2 - V_c^{'} \tag{5}$$

A first approach to evaluate the shear capacity of a beam-column joint without transverse reinforcement consists in principal stress limits according to concrete strength. Direct limits on the shear stress, according to (Hakuto et al., 2000) and reported in many International Codes (ACI 352, 2002; AIJ, 1999) are not accurate because they do not account for the vertical axial force in the column. The principal stresses p in the joint panel can be given by Mohr's Circle assuming uniform normal and transverse stresses, f_a e v_{jh} respectively, according to the following equation:

$$p = \frac{-f_a}{2} \pm \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \tag{6}$$

Equation (6) allows the horizontal joint shear to be evaluated at the first development of diagonal cracking:

$$v_{jh} = p_t \sqrt{1 + \frac{f_a}{p_t}} \quad \Rightarrow \quad V_{jh} = k_1 \sqrt{f_c} \sqrt{1 + \frac{f_a}{k_1 \sqrt{f_c}}} b_j h_j \tag{7}$$

where the tensile limit stress p_t is assumed to be proportional to k_1 times the square root of concrete compressive strength, where k_1 is empirically evaluated. It is clearly shown in (7) that axial load delays the diagonal cracking in the joint panel.
The joint shear causing compressed concrete strut crushing, assuming the compressive limit stress p_c to be proportional to k_2 times the concrete compressive strength, is equal to:

$$v_{jh} = p_c\sqrt{1-\frac{f_a}{p_c}} \;\Rightarrow\; V_{jh} = k_2 f_c\sqrt{1-\frac{f_a}{k_1 f_c}}b_j h_j \tag{8}$$

It is worth noting that a joint failure criterion based on tensile principal stress limit results to be over conservative. In fact the joint panel is able to transfer noticeable shear forces also in a cracked phase due to the diagonal strut mechanism. The joint failure should be in fact always related to the compressed strut crushing. In the case of high axial loads, the compressed strut crushing can be attained before the joint panel cracking (Paulay et al., 1992).
The evaluation of the horizontal joint shear determining the compressed strut crushing, according to equation (8), needs the experimental evaluation of k_2 coefficient, accounting for the real stress field in the joint, that is complex to be evaluated in cracked phase, for the compressive strength deterioration due to diagonal tensile strains (Collins et al., 1980), and for the detailing of steel bars anchorage, that is a crucial aspect in the case of exterior joints, to guarantee the development of the compressed strut (Priestley, 1996).
In the case of interior joints without transverse reinforcement, the values for k_1 e k_2 (Priestley, 1996) are 0.29 e 0.50, respectively. In the case of exterior joints, the proposed value for k_1 according to Priestley depends on the longitudinal bars anchoring details and it is 0.29 if the longitudinal bars are bent at 90° outside the joint or 0.42 if they are anchored inside the joint. In the case of exterior joints with smooth bars the value for k_1 equal to 0.20 is suggested (Calvi et al., 2002). In the cited works, deterioration models are provided to account for k_1 and k_2 variability based on ductility demand.
The strength capacity of joints with transverse reinforcement can be evaluated according to the cited model (Paulay et al., 1992). The minimum horizontal and vertical reinforcement requirements (A_{jh}, A_{jv}) are herein reported, while details on the complete model and on the simplified assumption on shear partition in the two mechanisms are given in the original paper. In the case of interior joints, it is:

$$A_{jh} = \left(1.15 - 1.3\frac{N_{col}}{f_c A_g}\right)\frac{\lambda_0 f_y}{f_{yh}}A_s;\;\; A_{jv} \geq \frac{1}{f_{yv}}\left(0.5V_{jv} - N_{col}\right) \tag{9}$$

while in the case of exterior joints, it is:

$$A_{jh} = \beta\left(0.7 - \frac{N_{col}}{f_c A_g}\right)\frac{f_y}{f_{yh}}A_s;\;\; A_{jv} \geq \frac{1}{f_{yv}}\left(0.5V_{jv} - N_{col}\right) \tag{10}$$

where $\beta = A'_s/A_s$ is the tensile over compressed beam reinforcement ratio, f_c is the concrete compressive strength, N_{col} is the minimum axial load in the column and A_g is the column cross sectional area.
Many other models are available in literature (Sarsam et al., 1985; Vollum et al. 1999; Bakir et al., 2002) to evaluate the shear strength as the sum of concrete and steel reinforcement contributions. However, such models are based on experimental calibration, while the model

according to Paulay is based on force equilibrium. In this respect, even though sometimes these models seem to be more accurate, it is obvious that their reliability depends on the extent and completeness of the experimental database of tests used for their calibration.
During last years, many other works proposed different models to evaluate the strength and deformability of beam-column joints. Amongst the well known models, there is the model (Pantazopolou et al., 1992) based on simple equilibrium equations and extended by (Antopoulos et al., 2002) to FRP strengthened joints. Strut-and-tie models (Hwang et al., 1999; Mitra, 2007) are well known also to analyze the joint panels with or without transverse reinforcement. The "Quadruple flexural resistance in reinforced concrete beam-column joints" (Shiohara, 2001) model accounts for the equilibrium of four rigid bodies forming the joint panel. It allows the different failure modes to be determined accounting for bond behavior of reinforcement in exterior or interior joints. Amongst the models accounting for the inelastic cyclic behavior of beam-column joints and those based on experimental calibration of plastic hinges and/or rotational springs, it can be cited the macro-model proposed in (Lowes et al., 2004) to account for the beam-column joint behavior and implemented in the structural code OpenSees. This algorithm is not time-consuming, but it allows to reliably and objectively account for the main mechanisms determining the inelastic cyclic behavior of the joints: namely, anchorage failure of the longitudinal reinforcement both in the columns and beams, shear failure of the joint panel and failure of the shear transfer mechanism at the joint interfaces.

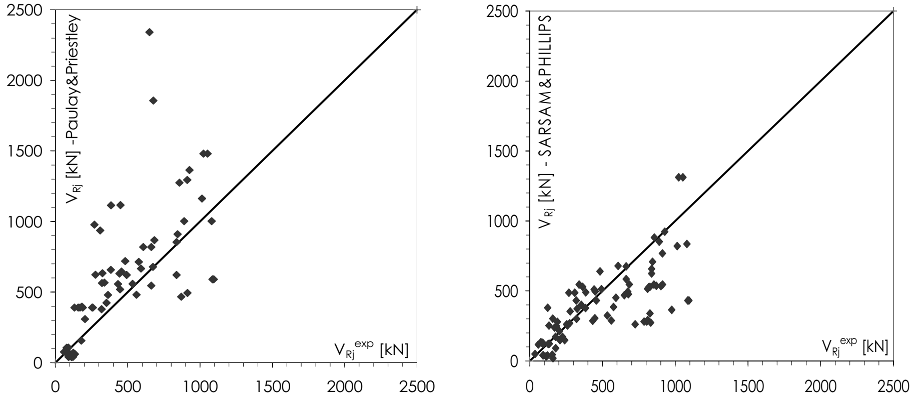
2.2 Experimental results from the scientific literature

The mentioned models for the shear capacity of RC joints can be assessed by comparing their theoretical results with the experimental data derived from the scientific literature purposely collected in a wide database. In particular, 87 results of experimental tests have been considered in this database (Table 1): 66 tests were carried out on specimens with stirrups within the joint panel, while the remaining 21 tests deal with unreinforced RC joints. The former ones basically reproduce the behaviour of joints in new seismically designed structures while the latter ones aim at reproducing the response of existing RC members. As far as the loading modality, a large part of the tests were carried out under cyclic conditions.

Table 1. Experimental data collected in the database.

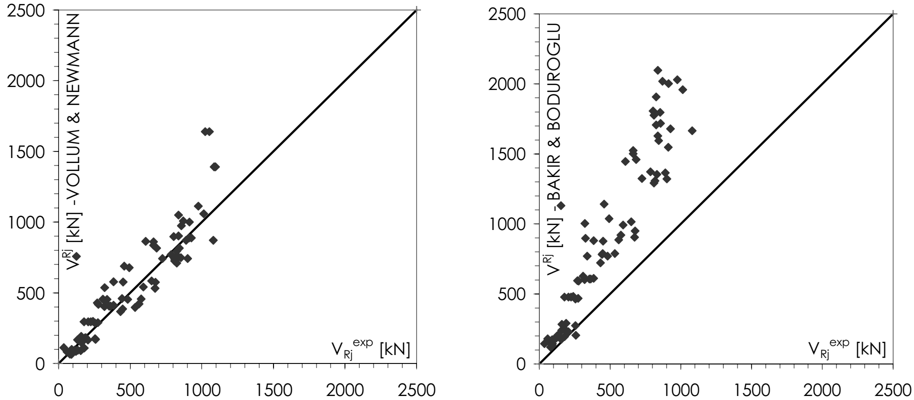
Authors	No. of tests	Authors	No. of tests
Chun & Al. (2007)	2	Lee & Ko (2005)	3
Chun & Kim (2004)	2	Pampanin & Al. (2002)	1
Chutarat & Aboutara (2003)	2	Pantelides & Al. (2000)	4
Durrani & Zerbe (1987)	4	Pantelides & Al. (2002)	6
Ehsani & Al. (1987)	5	Parker & Bullman (1997)	12
Ehsani & Alameddine (1991)	6	Salim (2007)	3
Ehsani & Wight (1985)	10	Scott (1996)	12
Hakuto & Al. (2000)	1	Tsonos & Al. (2002)	6
Hwang & Al. (2004)	1	Tsonos (2007)	2
Hwang & Al. (2005)	5		

The four diagrams in Figures 2 and 3 point out the comparison between the experimental results in terms of shear strength with respect to the corresponding results derived by applying four of the above mentioned theoretical models. The experimental values of shear strength of the joint panel (V_{Rj}^{exp}) can be directly derived by the ultimate force applied on the beam through simple equilibrium equations.



a) Paulay & Priestly (1992) model b) Sarsam & Phillps (1985) model

Figure 2. Shear capacity of RC joints: comparison of results from models and experimental tests.



a) Vollum & Newmann (1999) model b) Bakir & Boduroglu (2002) model

Figure 3. Shear capacity of RC joints: comparison of results from models and experimental tests.

Figure 2a is devoted to the model by Paulay & Priestley (1992) resulting in generally scattered and often unconservative prediction of the experimental values. On the contrary, the model by Sarsam & Phillips (1986) is generally more conservative, albeit resulting in significant dispersion with respect to the experimental values (Figure 2b). The model by Vollum & Newmann (1999) results in a more precise prediction of the experimental results (Figure 3a), while the model by Bakir & Boduroglu (2002) is generally unconservative, yet highly correlated to the experimental results (Figure 3b).

3 CAPACITY EVALUATION IN SEISMIC CODES

As for the European Code EC8, both for new (CEN, 2004) and existing (CEN, 2005) buildings, the evaluation of the horizontal maximum shear acting in the joint panel (shear demand) can be performed through the following two expressions, respectively for exterior and interior joints:

$$V_{jhd} = \gamma_{Rd} \cdot A_{s1} \cdot f_{yd} - V_C \tag{11}$$

$$V_{jhd} = \gamma_{Rd} \cdot (A_{s1} + A_{s2}) \cdot f_{yd} - V_C \tag{12}$$

where A_{s1} is the area of the beam top reinforcement, A_{s2} is the area of the beam bottom reinforcement, V_c is the column shear force, obtained from the analysis in the seismic design situation, γ_{Rd} is a factor to account for overstrength due to steel strain-hardening and should be not less than 1.2.
The EC8 formulation for predicting the joint shear capacity is made up of two separated steps. Firstly, there is an expression to evaluate the compression capacity of the strut that can be recognized in the joint panel under seismic actions and, then, an expression devoted to verify the tensile strength of the joint in order to avoid diagonal cracking.
The horizontal shear demand should not exceed a value that could cause the compression failure of the joint:

$$V_{jhd} \leq \eta\, f_{cd} \sqrt{1 - \frac{\nu_d}{\eta}}\, b_j \cdot h_{jc} \tag{13}$$

where η = 0.60 (1-f_{ck}/250) for interior joints and η = 0.48 (1-f_{ck}/250) for exterior joints, practically meaning that the strength of exterior joints is 0.8 (0.48/0.60) times that one of interior joints (assuming the same joint materials and detailing); ν_d is the normalised axial force in the column above the joint, f_{ck} is given in MPa, h_{jc} is the distance between the extreme layers of column reinforcement, b_j is the effective width of the joint. Further, EC8 provides an expression to evaluate the joint transverse reinforcement (left hand term in (14)) needed to avoid the diagonal cracking caused by the achievement of the concrete tensile strength f_{ctd}, as follows:

$$\frac{A_{sh} \cdot f_{ywd}}{b_j \cdot h_{jw}} \geq \frac{\left(V_{jhd} / b_j \cdot h_{jc}\right)^2}{f_{ctd} + \nu_d f_{cd}} - f_{ctd} \tag{14}$$

where, A_{sh} is the total area of the horizontal hoops in the joint, V_{jhd} is the horizontal joint shear demand, h_{jw} is the distance between top and bottom reinforcement of the beam.
The Italian Code IC (Ministry of Infrastructures, 2008) deals separately with joints belonging to new and existing buildings, the former ones being evaluated as in EC8. As for existing buildings, IC contains two expressions devoted to verify beam-column joint without seismic provisions, that is without hoops in the panel (paragraph C8.7.2.5). These expressions allow to calculate the maximum diagonal compression (15) and tensile (16) stresses in the concrete joint core that must be below given values related to the concrete strength f_c:

$$\sigma_{nc} = \frac{N}{2A_g} + \sqrt{\left(\frac{N}{2A_g}\right)^2 + \left(\frac{V_n}{A_g}\right)^2} \leq 0{,}5 f_c \quad (15) \qquad \sigma_{nt} = \left| \frac{N}{2A_g} - \sqrt{\left(\frac{N}{2A_g}\right)^2 + \left(\frac{V_n}{A_g}\right)^2} \right| \leq 0{,}3\sqrt{f_c} \quad \left(f_c \text{ in MPa}\right) \quad (16)$$

where N is the axial force acting on the upper column, A_g is the gross area of the joint panel horizontal section and V_n the horizontal shear acting in the joint panel evaluated accounting both the column shear and the shear transmitted by the beam reinforcing bars.

4 CRITICAL REVIEW OF CODE PROVISIONS

Results about joint behaviour, obtained principally from the experimental tests carried out during the ReLUIS project, provided some useful information to critically review and analyse some expressions reported in the European Code EC8 and in the Italian Code IC.

4.1 Evaluation of the shear demand V_{jhd}

Expressions (11) and (12), regarding exterior and interior joints, respectively, may underestimate the shear demand that the beam can really transmit to the joint. Indeed, the amplified tension that appears in the expressions, $\gamma_{Rd} \cdot f_{yd}$, is a design value that the steel certainly exceeds when the plastic hinge develops in the beam.
By considering that:
(i) the real yielding strength f_y usually exceeds the nominal yielding strength f_{yk}, even though it should not exceed over the 25% (EC8 par. 5.5.1.1(3)P and IC Tab. 11.3.Ib), and
(ii) the characteristic value of the ratio between the ultimate strength f_t and the yielding strength f_y must be in the range 1.15-1.35,

it can be considered that steel bars, in case of large strains, can reach, on the average, a tension value up to f_{tm} =1.45 f_{yk}, instead of the value suggested by EC8 and IC that is equal to 1.04f_{yk} achieved by multiplying f_{yk} by γ_{Rd} =1.20 and dividing by γ_s = 1.15.
An experimental proof of that statement has been obtained for the exterior joints tested by the Research Unit (RU) of University of Udine (Russo et al., 2007 and 2008) during the ReLUIS project, where the shear demand was calculated through the expression (11) using:

$$f_{yd} = \frac{f_{yk}}{\gamma_S} = \frac{1}{\gamma_S} \cdot \frac{f_{y,mean}}{(1+1.25)/2} \tag{17}$$

since only the mean value $f_{y,mean}$ of reinforcing steel was available and the characteristic value $f_{yk} = f_{y,nom}$ was unknown. The values of V_{jhd} calculated as described above and the maximum ones obtained by experimental results, V_{exp}, are significantly different, as shown in Table 2.

Table 2. Calculated (V_{jhd}) and measured (V_{exp}) shear values.

Joint	V_{jhd} (kN)	V_{exp} (kN)
12-6 (+)	50.4	99.3
12-6 (-)	49.6	103.2
12-8 (+)	50.7	97.6
12-8 (-)	48.7	110.0
16-6 (+)	49.0	108.4
16-6 (-)	90.6	147.2
16-8 (+)	52.1	108.9
16-8 (-)	88.5	162.2

(+) positive acting moment (-) negative acting moment

The V_{exp} values shown in Table 2 have been determined using the expression (11) where γ_{Rd} $A_{s1} f_{yd}$ has been substituted by the actual force value provided by the reinforcing bars. This value is obtained by dividing the maximum experimental moment at the beam-column interface by $0,9d_b$, with d_b the effective depth of the beam. Further, V_c is the maximum shear in the column corresponding to the maximum experimental beam moment.
From the above considerations and on the basis of the results shown in Table 2, it can be deduced that the expressions (13) and (14) (either 5.33 and 5.35 of EC8 or 7.4.8 and 7.4.10 of

IC) for resistance verification (maximum diagonal compression and tension in concrete core) of joints with or without hoops can be not conservative, being the acting shear V_{jhd} (Eq.(11)) underestimated.
In addition, the calculation of demand in EC8 for joints of existing buildings is the same as for new buildings applying the equations (11) and (12), with the difference that, as reported in (CEN, 2005), the mean values of material resistance must be divided by the Confidence Factor (CF) and, in case of brittle elements as the beam-column joints are, by the partial factor γ_s. This can lead to a greater underestimation (with respect to the joints of new buildings) of the shear demand, using a very low tension in beam reinforcing bars equal to:

$$f_{yd} = \gamma_{Rd} \cdot \frac{f_{ym}}{CF \cdot \gamma_s} \tag{18}$$

where f_{ym} is the mean value of the yielding strength of steel.
Use of CF in expression (18) does not appear correct, as it should provide lower values of f_{yd} to be used in equations (11) and (12), thus lower demand values on joints at decreasing knowledge levels. It is then advisable, as typically suggested when strength values need to be used in calculating action effects delivered to brittle component/mechanism by ductile components, that mean values of material properties are multiplied and not divided by CF in order to appropriately account for the attained knowledge level. Moreover, the safety factor γ_s should be assumed equal to 1.0, thus expression (18) can be effectively rewritten as follows:

$$f_{yd} = \gamma_{rd} \cdot CF \cdot f_{ym} \tag{19}$$

where the term $\gamma_{Rd} \cdot CF$ could be limited to the value of 1.45 taking into account what above reported real yielding and ultimate strength values.

5 ANALYSIS OF PREDICTION ABILITY OF CODE FORMULATIONS THROUGH ReLUIS EXPERIMENTAL TESTS

In order to verify the estimation capability of the joint shear strength provided by the expressions contained in IC and EC8, they have been applied to the specimens tested at Laboratory of Structures of the University of Basilicata (UniBas RU) in the framework of the DPC-Reluis Project.
During the experimental program, 10 quasi-static cyclic tests on exterior full scale beam-column joints, provided with different Earthquake Resistant Design (ERD) Level, axial force values and type of steel reinforcement, have been carried out. The main characteristics of the specimens under test and of the obtained results are summarized in Table 3. The specimens had three different ERD levels: design for seismic zone 2 (Z2), for seismic zone 4 (Z4) and with respect to gravity loads only (NE). The normalized axial load applied during test was equal either to 0.15 (NL) or 0.30 (NH). More details on the experimental program are reported in (Masi et al., 2008), (Masi et al., 2008b) and in (Masi & Santarsiero, 2008).
As it can seen in Table 3, 7 out of 10 specimens showed a failure mechanism that involved only the beams because of their small amount of longitudinal reinforcement, particularly the bottom one. Adopting the expressions (13)-(16), joint strength values (compression and tension capacities) have been determined for each of the 10 tested joints accounting for amount of transverse reinforcement, axial force value, concrete and steel strengths. No

conventional safety factors have been taken into account. Further, mean values of material strengths have been considered, thus f_{ck} has been assumed equal to f_{cm}=21 MPa (achieved as the mean value from compression tests on cube specimens purposely tested during the experimental program), as well as f_{ctd} has been assumed equal to f_{ctm}=2.28 MPa. The tension value exhibited by reinforcing bars has been determined for each tested specimen imposing the equilibrium of the sub-assemblage under the maximum applied horizontal force. Therefore, for each joint a different value of steel strength has been determined according with the variability of the material characteristics and the amount of the slippage effects, as reported in Table 4. In Table 3 the failure mode is indicated as “B” in the cases in which the specimen showed a flexural failure in the beam, while “J” indicates the occurrence of a diagonal cracking in the joint panel. As it can be seen, joint cracking is always accompanied by flexural cracking and yielding of beam longitudinal reinforcement.

Table 3. Main results from UniBas experimental tests.

Test #	Design type	Axial load	Failure mode	Maximum column shear (kN)	Collapse drift (%)
T1	NE	NL	B	18.9	2.75
T2	Z2	NH	B	40.2	3.36
T3	Z2	NH	B	38.9	4.96
T4	Z4	NH	B	42.9	3.45
T5	Z2	NL	J+B	39.8	3.25
T6	NE	NH	B	21.3	2.85
T7	NE	NL	B	21.3	3.28
T8	Z4	NH	B	42.8	3.40
T9*	Z2	NH	J+B	48.3	3.30
T10*	Z2	NL	J+B	48.9	3.65

*specimens with reinforcing bars having higher strength and lower deformation capacity

In Figure 4, as an example, the force-drift relationships and the damage pattern occurred to the specimens during the tests T3 and T5 are shown. The specimen of test T3 shows no cracks in the joint panel and all the damage is concentrated at the beam-column interface with a depth flexural crack. Collapse of this joint was caused by the tensile failure of the bottom longitudinal bars in the beam.

The specimen of the test T5, identical to the previous one, showed a heavy damage into the joint panel in addition to the flexural cracks in the beam. The different behaviour is attributable to the value of the axial force acting in the column that was lower in the test T5. The occurrence of the joint failure caused less satisfactory performance, as displayed in Figure 4: test T5 shows greater stiffness and strength deterioration as well as more pronounced pinching of hysteresis loops.

Results of the application of code formulations reported at the paragraph 3 for estimating the joint shear capacity, are shown in Figure 5. Design strength value of steel bars are evaluated assuming CF, γ_s and γ_{Rd} equal to 1.0.

For each test, the first (blue) bar is relevant to the joint shear V_{exp} experimentally determined, that is by using expression (11) as already explained at par. 4.1. For each of 10 tests both compression strength V_{jc} and tensile strength V_{jt} have been evaluated through the code expressions (13) and (14), and, only for NE joints, also applying IC code expressions (15) and (16) for existing buildings.

Results in Figure 5a show that the shear strengths provided by code expressions are greater than V_{exp}, except for the tests T5, T9 and T10. In particular, tests T5 and T10 show a predicted tensile strength V_{jt} lower than the experimental shear V_{exp} in agreement to the observed failure modality of the joints that involved both the beam and the joint panel (see Table 3). As for test T9, it can be observed that the predicted compressive strength V_{jc} is lower than the experimental joint shear V_{exp}, highlighting the occurrence of a compression failure of the joint panel. The EC8 expressions are, in these cases, able to predict the joint shear failure.

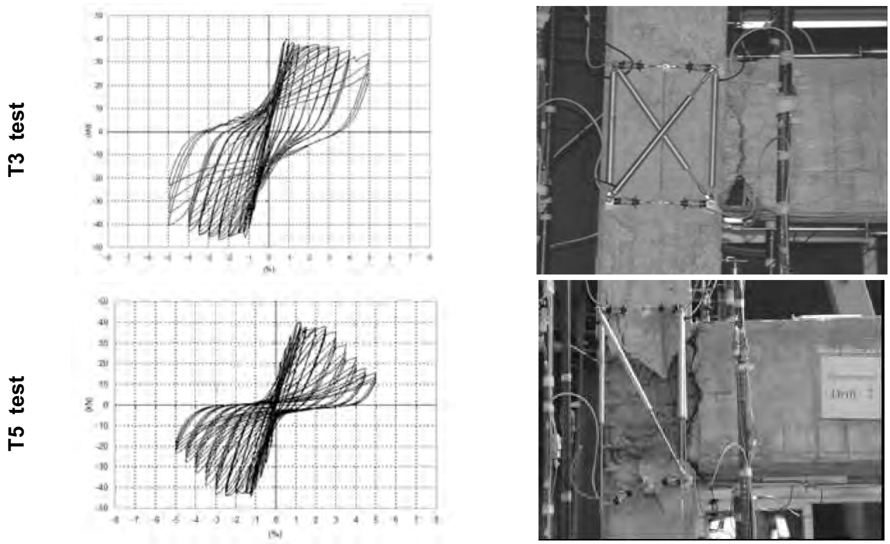


Figure 4. Examples of mechanical behaviour and damage state (UniBas experimental program).

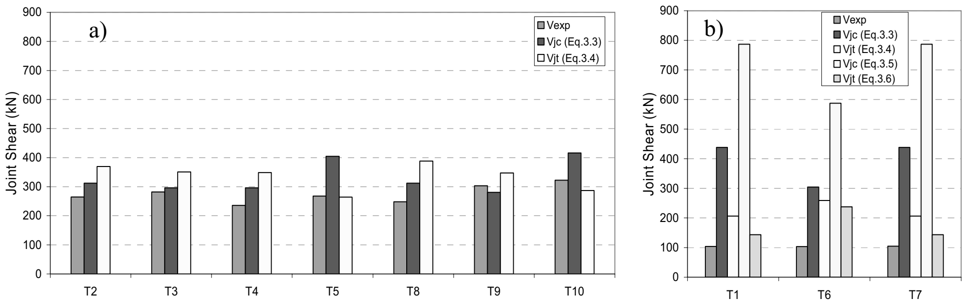


Figure 5. Comparison between experimental and analytical joint shear strength values: a) specimens with seismic design, b) specimens w/o seismic design (UniBas experimental program).

The diagram in Figure 5b shows also the values of the joint shear strength obtained applying expressions (15) and (16) provided in IC to verify joints without transverse reinforcement belonging to existing buildings, thus applicable only to NE joints. In this case V_{jt} and V_{jc} (green and yellow bars) are always greater than V_{exp} in accordance with the experimental result, although it cannot be stated whether the expressions (15) and (16) would be able to predict joint failure.

As for the estimation of the shear demand, in Table 4 a comparison between the shear values experienced by the joints during the tests V_{exp} and the shear demand V_{jhd} calculated according to the code expression (11), is reported. As can be seen, the "real" shear demand values are always higher than the theoretical ones: $V_{exp} > V_{jhd}$. It is worth specifying that V_{jhd} has been calculated blindly applying EC8, referring to the design yield strength of reinforcing bars used to build the specimens, that is $f_{yd} = f_{yk} / \gamma_s = 430/1.15 = 373.9$ MPa, where f_{yk} is the nominal yield strength. The mean value of the ratio V_{exp}/V_{jhd} is about 1.26, highlighting the need of a correction of the strength value to be used in calculating the shear demand, as already pointed out at par. 4.1, in order to avoid a remarkable underestimation of the joint shear demand. Finally, the estimated steel strength, showed in Table 4, has a mean value 1.24 times the nominal yield stress f_{yk} and 1.08 times the mean value of yield stress f_y deduced by tensile tests performed on bars actually used to build the specimens.

Table 4. Comparison between experimental and code shear in joints tested at UniBas.

Test N.	V_{jhd} [kN]	V_{exp} [kN]	Estimated steel strength (MPa)
T1	82.6	103.8	542.4
T2	230.3	264.8	505.9
T3	231.7	282.0	532.1
T4	196.6	235.6	521.6
T5	230.7	267.9	510.4
T6	80.1	103.3	551.1
T7	80.1	104.6	557.0
T8	196.7	248.5	545.6
T9	222.2	303.2	582.9
T10	221.7	322.5	615.9

6 FINAL REMARKS AND IMPROVEMENT PROPOSALS

The main objective of the present paper was to investigate on the experimental behaviour of RC beam-column joints, thus providing a contribution to a more reliable evaluation of the seismic vulnerability of RC existing buildings. In particular, the understanding and the validation of capacity models reported in the current seismic codes is of great interest.

To this purpose, a wide bibliographic research on available capacity models and experimental investigations on beam-column joints have been firstly carried out. Literature analysis has been devoted to carefully describe some capacity evaluation models and to compare their prediction ability applying them to a database of experimental results. The capacity model of Vollum & Newmann (1999) appeared more effective in predicting shear capacity of the analysed joints. Use of this model could be suggested to improve the code expressions on joint shear strength, although further work needs to be made in order to enlarge the database of analysed experimental results.

The validation of the code expressions (reported in EC8 and IC) has been made on the basis of the ReLUIS project results. The main result is that the shear demand computed by using code expression (11) can be unconservative if a suitable value of the steel strength is not assumed. Experimental programs performed by University of Basilicata and Udine RUs demonstrated that the actual shear demand in joints is always greater than the theoretical one, being the difference dependent on steel type and bond conditions. Specifically, for existing buildings the underestimation of shear demand can be greater with respect to joints belonging to new buildings because, strictly applying the current code provisions, the estimated steel strength should be divided also by the confidence factor (CF). A more suitable application of code expressions is proposed in the present paper where mean values of material properties are multiplied by CF, as typically suggested when strength values need to be used in calculating action effects delivered to brittle component/mechanism by ductile components. Further, the actual stress values exhibited by the steel during the tests were calculated and used for safety verifications using code expressions (13) and (14) for joint shear capacity. As a result, the code capacity models appeared able to predict whether or not the joint cracking occurred, provided that the right steel strength values were used.

Due to the limited amount of tested specimens, further work is required to get a more reliable proof of the effectiveness of code expressions, skilfully combining purposely designed experimental investigations, review of experimental campaigns reported in the literature, and accurate numerical simulations. Particularly, extensive experimental programs on joint specimens having different characteristics (e.g. interior or exterior, bi- or tri-dimensional, beam type, etc.) well targeted on the types representative of the Italian and European built environment, need to be performed.

7 ACKNOWLEDGEMENTS

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RETROFITTING OF EXISTING RC BUILDINGS WITH FRP

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ABSTRACT
This work focuses on current research developments in the field of FRP strengthening of RC buildings. The main outcomes of these activities have been analyzed to provide possible recommendations towards a future update of EC8 – Part 3. The main strategies and driving principles for the seismic retrofit of existing structures have been discussed with due attention to both local and global interventions. In this framework, FRP confinement has been analyzed as well, focusing in details on rectangular columns with high aspect ratio.

KEYWORDS
Confinement, FRP, guideline, RC buildings, seismic retrofit.

1 INTRODUCTION

The most common strategies adopted in the field of seismic retrofit of existing structures are the restriction or change of use of the building, partial demolition and/or mass reduction, removal or lessening of existing irregularities and discontinuities, addition of new lateral load resistance systems, local or global modification of elements and systems.
In particular, local intervention methods are meant to increase the deformation capacity of deficient components, so that they will not attain their specified limit state as the building is acted upon by the design seismic excitation. Common approaches mainly include steel jacketing and externally bonded Fiber Reinforced Polymers (FRP) wrapping. On the other hand, global intervention methods involve a thorough modification of the structural system; such modification is designed so that the design demands (often identified in a target displacement) on the existing structural and non-structural components are less than structural capacities. Common approaches mainly include: Reinforced Concrete (RC) jacketing, insertion of walls, steel (dissipative) bracing and base isolation.
The above brief overview of possible rehabilitation strategies shows that the structural performances of an existing building can be enhanced in different ways by acting on ductility, stiffness or strength (separately or, in many cases, at the same time); in each case, a preliminary assessment of the existing structure performances and the evaluation of the analysis results are necessary to select the rehabilitation method that meets the required performance targets. Nevertheless, numerous factors influence the selection of the most appropriate technique and therefore no general rules can be defined.

The present paper focuses on the potential of FRP for seismic strengthening of RC buildings, by highlighting the criteria for selecting the type of intervention and by discussing the outcomes of some related research activities lately performed by the authors.

2 FRP STRENGTHENING IN SEISMIC ZONES

EC8 – Part 3 offers the possibility, as an alternative to the more traditional strengthening techniques of RC and steel jacketing, to use composite materials such as FRP for seismic retrofit of under-designed RC structures. However, EC8 – Part 3 only indicates that FRP can be used to: a) increase shear strength of members, b) provide ductility to concrete, and c) prevent lap splice failure. The authors believe that preliminary insights should be given about the overall objective of FRP interventions on buildings; a possible strategy outline is proposed herein.

First, it is worth recalling that stiffness irregularities cannot be solved by applying FRP. Strength irregularities can be modified by strengthening a selected number of elements, however, attention should be paid that the global ductility be not reduced.

From the seismic standpoint, FRP strengthening could be regarded as a selective intervention technique that could allow:

a) increasing the flexural capacity of deficient members, with and without axial load, through the application of composites with the fibers placed parallel to the element axis;
b) increasing the shear strength through the application of composites with the fibers placed transversely to the element axis;
c) increasing the ductility (or the chord rotation capacity) of critical zones of beams and columns through FRP wrapping (confinement);
d) improving the efficiency of lap splice zones, through FRP wrapping;
e) preventing buckling of longitudinal rebars under compression through FRP wrapping;
f) increasing the tensile strength of the panels of partially confined beam-column joints through the application of composites with the fibers placed along the principal tensile stresses.

The driving principles of the FRP intervention strategies should be:

a) all potential brittle collapse mechanisms should be eliminated: failures such as shear, lap splice, bar buckling and joint shear should be prevented;
b) the global deformation capacity of the structure should be enhanced, either by: b1) increasing the ductility of the potential plastic hinge zones without changing their position, or, b2) relocating the potential plastic hinge zones by applying capacity design criteria. In this latter case, the columns should be strengthened in flexure with the aim of transforming the framed structure into a highly dissipating mechanism with strong columns and weak beams.

For case a), when eliminating potential brittle failure mechanisms, the relative strengthening modalities are quite straightforward. The most common case is potential shear failure, for which a strengthening of the shear mechanism should be sought. More peculiar cases are those of longitudinal bars lap splices and buckling. In the former case, due to either bond degradation in splices or insufficient splice length, the relevant regions of potential plastic hinge formation should be adequately confined through FRP wrapping; in the latter case of bar buckling, the strengthening intervention should consist in confining the potential plastic hinge zones where the existing transverse reinforcement cannot prevent the bars post-elastic buckling.

For case b), when all possible brittle and storey mechanisms have been prevented, it is necessary to assess to which extent the structure could exploit its ductility. This can be done, for example, through a nonlinear pushover analysis, now adopted and codified in the most modern seismic codes. Usually, it is requested to check if the structure can actually ensure a given ductility, expressed by a pre-selected behavior factor, or, which is the same, if it is able to attain a given target displacement. Such analysis allows identifying the elements whose local collapse, due to ductility exhaustion, prevents the structure from exploiting its global ductility and from reaching the target displacement.

Thus, the required global deformation capacity can be obtained either by b1) or b2) strategies.

In the former case, the deformation capacity of elements that collapse before the global target displacement is attained has to be increased. A possible measure of the deformation capacity of beams and columns is the chord rotation θ, that is, the rotation of the chord connecting the element end section with the contraflexure section (shear span). Generally, the plastic deformation capacity is controlled by the compressive behavior of concrete. An intervention of FRP-confinement on such elements (usually columns) increases the ultimate compressive strain of concrete, thus determining a ductility increase of the element.

In the latter case, the overall resisting mechanism should be changed in order to distribute the ductility request over a larger number of elements. This can be achieved by relocating all potential plastic hinges by applying the capacity design criteria. The application of the capacity design criteria implies the elimination of all potential plastic hinges in columns. In "weak column-strong beam" situations, typical of frame structures designed for gravity loads only, the columns' cross sections are under-designed both in terms of geometry and reinforcement. In such case, it is necessary to increase their flexural strength with the objective of changing the structure into a "strong column-weak beam" situation. It should be noted that this strategy implies an increase of shear demand on columns due to the flexural capacity increase. It is therefore necessary to perform the required shear verifications, and to eventually increase the shear strength in order to avoid brittle failure modes. Moreover, attention must be paid to the foundation systems as the increased seismic strength capacity leads to an overturning moments increase.

Within the outlined strategies, FRP confinement is a key technique to increase the seismic capacity of RC members. Existing analytical models for predicting the stress–strain behavior of FRP-confined concrete are mostly derived for cylindrical plain concrete columns. Square- and rectangular-section columns were found to experience less increase in strength and ductility than their circular counterparts. This is because the distribution of lateral confining pressure in circular sections is uniform, in contrast to square and rectangular sections, in which the confining pressure varies from a maximum at the corners and diagonals, to a minimum in between. In particular, in the case of wall-like columns (e.g. with aspect ratio higher than 3), the effectiveness of FRP jackets is even more reduced (Prota et al. 2006). To determine the effective lateral confining pressure, some researchers proposed to transform the rectangular section into an equivalent circular section (e.g. circumscribed, inscribed, or with an equivalent cross-sectional area). A more refined iterative approach has been proposed by Lignola et al. 2009a, based on solid mechanics. Confinement models based on regression analyses are very sensitive to the value adopted for the ultimate FRP strain: in fact, FRP ultimate tensile strain determined experimentally according to flat coupon tests is not reached at the rupture of the FRP jacket in confined concrete columns compression tests (reasons for this have been provided by many authors, a summary is in Lignola et al. 2008a). The ratio between the two strain values is termed "efficiency factor", β. If the effective lateral confining pressure is inserted in a confinement model, whatever the material of the confining device, the scattering between theoretical and experimental results can be drastically reduced and a single

expression can be formulated to predict benefits provided by confinement, e.g. for concrete, independently of the materials used as confinement device. It is highlighted that also more refined iterative confinement models may need a stop criterion given by the effective failure of the FRP jacket.
In the case of rectangular cross-sections with high aspect ratio (structural walls), the failure is strongly affected by the occurrence of premature mechanisms (compressed bars buckling and unrestrained concrete cover spalling), while nowadays slender structural wall members are usually designed without special prescriptions (Lignola 2006).

3 RESEARCH DEVELOPMENTS

The present section discusses two specific aspects of FRP seismic rehabilitation. First, the confinement of RC members is analyzed and significant research outcomes on the behavior of rectangular cross-sections with high aspect ratio are dealt with. Then, the strategy based on increasing the global displacement capacity without relocalizing plastic hinges is discussed; the design procedure is outlined and its validation by comparison to the experimental results on a real scale structure is reported.

3.1 Confinement

The analysis of the behavior of hollow RC piers, peculiar of bridge constructions to maximize structural efficiency of the strength-mass and stiffness-mass ratios, allowed the confinement of circular and non circular and also of slender structural wall members, peculiar of buildings, to be studied in details. A refined numerical iterative procedure and a detailed nonlinear confinement model was provided for the analysis of hollow RC columns (Lignola et al 2008b). Nevertheless, to provide a direct, practical tool, oriented to the profession more than a nonlinear refined iterative analysis, the opportunity was evaluated to simplify the analysis, considering the effect of confinement on the walls composing the hollow cross-section. A preliminary Finite Element Method (F.E.M.) analysis has been conducted (Lignola et al. 2009b) in the elastic range to evaluate the stress field generated by external wrapping on confined wall members. The arch-shaped paths of the confining stresses rapidly changes in a straight distributed confinement stress field moving away from the corner.
The results of the previous works suggest that a reliable numerical procedure to predict structural wall behavior under combination of flexure/shear and compression should include appropriate models for compressed bars buckling, concrete cover spalling and, of course, confined concrete behavior. If compressed bars buckling and concrete cover spalling are neglected, inaccurate ductility predictions may be obtained. In these cases confinement models may be successfully used to predict essentially the strength of the column, if the evaluation is limited to the occurrence of buckling of the compressed steel reinforcement bars. Usually confinement does not change the failure mode for walls, but it is able to delay bars buckling, restraining also concrete cover spalling, and to let compressive concrete strains attaining larger values, thus resulting in higher load carrying capacity of the member and in significant ductility enhancement. The strength increase in confined concrete due to FRP wrapping turns into load carrying capacity increases, mainly in the elements loaded with small eccentricity (it is clear that at higher eccentricities the effect of concrete strength enhancement is not relevant because failure moves to the tension side, and also the influence of reinforcement buckling is less significant).
A confinement model, recently proposed, is based on solid mechanics in plane strain conditions and able to predict the fundamentals of the behavior of solid and hollow circular

(Lignola et al. 2008b, 2009c) and solid square (Lignola et al. 2009a) members confined with FRP. A secant approach is used to account for the nonlinear behavior of concrete. The key innovative aspect of the proposed model is the evaluation of the contribution of confining stress field neither equal in the two transverse directions *x* and *y*, nor uniform along those directions. The effect of confinement is evaluated in each point of the cross-section explicitly considering a plasticity model for concrete under triaxial compression. The model traces the different confinement effectiveness and lateral stress field inside the cross-section and it allows to evaluate, at each load step, the multiaxial state of stress, and eventually the failure, of the concrete or the external reinforcement: i.e., the effective FRP strain at failure (Lignola et al. 2008a, and Zinno et al. 2009). The lateral-to-axial strain relationship provides the essential linkage between the response of the concrete column and the response of the FRP jacket in a passive-confinement model. The ultimate strength surface (Figure 1), $\rho=r(\theta,\xi)\cdot f'_c$ with failure parabolic meridians *r* (Elwi and Murray 1979) is formulated in the Haigh–Westergaard stress space defined by the cylindrical coordinates of hydrostatic length (ξ), deviatoric length (ρ) and Lode angle (θ). In the ultimate surface equation the only unknown is the confined concrete strength f_{cc} and it can be iteratively evaluated. It is noted that the cited model is the basis for the equation reported, for instance, in the ACI 440.2R (2008) code or Mander et al. (1988), to evaluate the cylindrical triaxial confined concrete strength f_{cc} given a uniform confining pressure f'_l:

$$\frac{f_{cc}}{f_c^{'}} = 2.25\sqrt{1+7.9\frac{f'_l}{f_c^{'}}} - 2\frac{f'_l}{f_c^{'}} - 1.25 \quad (1)$$

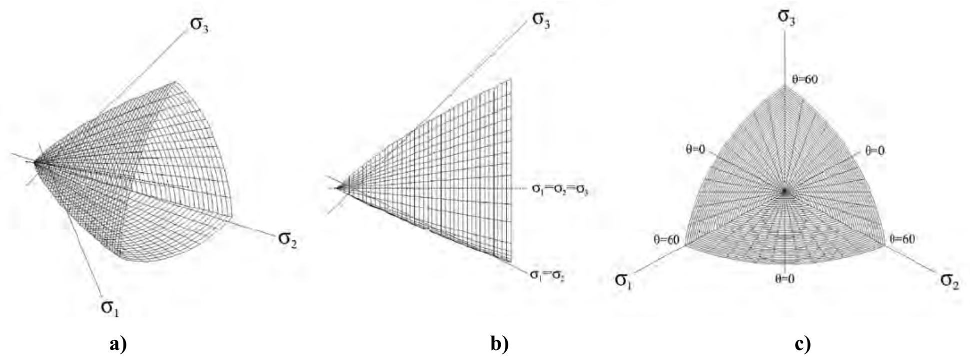


Figure 1. Ultimate strength surface (a); in the Rendulic plane (b); in the deviatoric plane (c).

Again, even though a refined nonlinear confinement model was provided for the analysis of circular and noncircular RC columns (e.g., Lignola et al. 2009a, 2009c), to provide a direct, practical tool, oriented to the profession, a simplified confinement model was also provided for wall-like cross-sections (the arch-shaped path of confining stresses was seen to rapidly change in a straight field moving away from the corners (Lignola et al. 2009b)). According to this alternative simplified approach, which gives rather accurate results despite the heavily reduced computational effort (no iterations are needed), the confining stress field is only parallel to the longer side of the cross-section, thus neglecting the confinement in the shorter direction and the confining pressure can be assumed equal to:

$$f'_l = 2\frac{tE_f \varepsilon_{FRP}}{h} \tag{2}$$

assuming cross-section height h < base b. Assuming zero stress for the minimum principal stress, confining pressure f'_l equal to the intermediate principal stress, f_{cc} as the maximum principal stress, then the following approximated equation is derived by the failure surface:

$$\frac{f_{cc}}{f'_c} = 1 + 1.42\frac{f'_l}{f'_c} - 1.40\left(\frac{f'_l}{f'_c}\right)^2 + 0.30\left(\frac{f'_l}{f'_c}\right)^3 \tag{3}$$

where $f'_l/f'_c<1.3$. Eq. (3) can be also used to evaluate the stress-strain relationship for confined concrete in slender walls according to the procedure proposed by Spoelstra and Monti (1999), relying on an iterative procedure through which the stress-strain curve crosses a family of curves at constant confinement pressure, at each point induced by the FRP jacket subjected to the corresponding lateral expansion. It is highlighted that, in case of wall confinement, the response of concrete may show a high load carrying capacity loss (e.g., more than 20%) before FRP failure and therefore numerical simulation can be concluded due to the high capacity loss rather than due to the failure of the confining material.

Experimental campaign conducted on wall-like columns (Prota et al. 2006) confirmed that significant strength increases can be achieved by FRP wrapping: the number of plies does not play a major role on the axial strength, while it gives improvements in terms of axial ductility. The failure of these walls determines the bulging of the FRP laminates occurring at fiber strains far below the ultimate values provided by the manufacturers.

A theoretical model has been proposed in Lignola et al. (2008a) suggesting an upper bound of the efficiency factor β (because it neglects stress localization and premature failures). To avoid an iterative procedure, the bond between concrete and FRP is also neglected providing a direct closed form solution (assuming the three-dimensional Tsai-Wu failure criterion):

$$\beta = \frac{\left(1 - \upsilon_{TL}\upsilon_{LT} + \upsilon_{TL}\upsilon_{LT}\frac{t}{R} + \upsilon_{LT}\frac{t}{R}\right)}{\sqrt{1 + \left(\frac{f_\theta}{f_r}\frac{t}{R}\right)^2 + \left(\frac{f_\theta}{f_z}\right)^2\left(\upsilon_{TL} - \upsilon_{TL}\cdot\frac{t}{R}\right)^2 + \frac{f_\theta}{f_r}\frac{t}{R} - \left(\upsilon_{TL} - \upsilon_{TL}\frac{t}{R}\right)\frac{f_\theta}{f_z} + \left(\upsilon_{TL} - \upsilon_{TL}\frac{t}{R}\right)\frac{t}{R}\frac{f_\theta^2}{f_r \cdot f_z}}} \tag{4}$$

The sensitivity of the involved parameters has been discussed in Lignola et al. (2009d), where it was shown that the main parameter driving coefficient β is the FRP composite relative strength (f_θ/f_r) and GFRP presents the highest dependence on the analyzed parameters. In this sense the proposed Eq. (4) can be simplified assuming typical values for Poisson's ratios (ν_{TL} and ν_{LT}). There is a research need to collect and publish in future confinement experimental works also those FRP mechanical (orthotropic) properties.

To better limit the range of variability of the effective FRP strain in confinement, a second model was proposed (Zinno et al. 2009) to analyze the effect of the stresses concentration at the free edge of the FRP jacket. Interlaminar stresses can cause premature failure of the FRP wrapping due to separation or delamination, thus limiting the confinement capacity of the FRP wrapping. This second model directly provides the effective FRP strain depending on the maximum interlaminar shear, τ_{max}, or on the normal tensile interlaminar peel stress, σ_{nmax}, capacity:

$$\varepsilon_{FRP} = \min\left\{\frac{e^{2\gamma L}-1}{e^{2\gamma L}+1}\tau_{\max};\frac{\sigma_{n\max}}{t_a\gamma}\right\}\cdot\frac{1+n}{Etn\gamma} \tag{5}$$

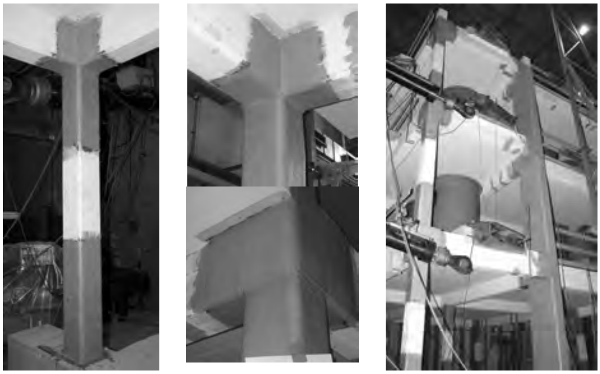
parameters are described in detail in the original paper and their typical values are provided.

3.2 Seismic retrofit without relocalization of plastic hinges

In the case of structures designed for gravity loads only, the overall deformation capacity is usually governed by the limited rotation capacity in the plastic hinge at columns ends (inadequate cross-sectional dimensions and amount of longitudinal steel reinforcement). A seismic upgrade intervention targeted at increasing the overall structure deformation capacity can be pursued by FRP columns confinement. Indeed, columns wrapping allows enhancing the ultimate concrete compressive strain; this corresponds to an increase of curvature ductility that, assuming a plastic hinge length not significantly affected by the upgrade intervention, determines a proportional increase of the plastic hinge rotation capacity. Because confinement using composite materials at columns ends induces, for intervals that are typical of normal stress levels, a considerable increase in terms of sections ductility, but does not lead to a significant increase in strength, such kind of retrofit does not modify the strength hierarchy of the structure.

The outlined seismic strengthening strategy effectiveness was experimentally investigated within the European research project SPEAR (Seismic PErformance Assessment and Rehabilitation). Such project involved a series of pseudo-dynamic bi-directional tests carried out on a three-storey RC structure with an irregular layout at the ELSA laboratory of Joint Research Centre (JRC) in Ispra (Italy). The structure under examination was designed and built with the aim of creating a structural prototype featuring all the main problems normally affecting most existing structures: plan irregularity, dimensions of structural elements and reinforcement designed by considering only gravity loads, smooth reinforcement bars, poor local detailing, insufficient confinement in the structural elements and weak beam column joints. The structure was subjected to pseudo-dynamic tests, both in its original configuration and retrofitted by using GFRP. The structure in its original configuration was subjected to experimental tests with maximum peak ground acceleration (PGA) of 0.20g. Since both theoretical and experimental results showed that the 'as-built' structure was unable to withstand a larger seismic action, a retrofit intervention by using FRP laminates was designed. Once the design of the GFRP retrofit was provided, the structure was subjected to a new series of two tests with the same input accelerogram selected for the 'as built' specimen but scaled to a PGA value of 0.20g and 0.30g, respectively. The design of the rehabilitation was based on deficiencies underlined by both the test on the 'as-built' structure and the theoretical results provided by the post-test assessment (nonlinear static pushover analysis). They indicated that a retrofit intervention was necessary in order to increase the structural seismic capacity; in particular, the theoretical results showed that the target design PGA level of 0.30g could have been sustained by the structure if its displacement capacity was increased by a factor of 48% (Di Ludovico et al. 2008a). In order to pursue this objective, the retrofit design strategy focused on two main aspects. First, it was decided to increase the global deformation capacity of the structure and thus its dissipating global performance; such objective was pursued by confining column ends with two plies of GFRP laminates. In particular, the amount of FRP plies to be installed to provide the required ductility increase of plastic hinges at columns ends was determined based on the following steps: 1) maximum theoretical ratio between ultimate chord rotation demand and capacity, $\gamma = \theta_{u,demand}/\theta_{u,capacity}$ was determined; 2) the target rotation capacity was computed as $\gamma\cdot\theta_{u,capacity}$ and thus the corresponding design cross-section ultimate curvature, $\phi_{u,\ target}$ was evaluated; 3) the concrete

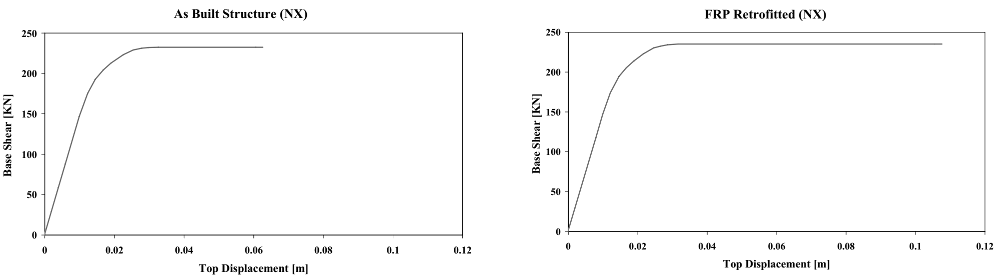
ultimate strain, $\varepsilon_{cu,target}$, to achieve such curvature was computed based on cross-section analysis; 4) the amount of FRP plies ensuring the attainment of $\varepsilon_{cu,target}$ was evaluated.
Moreover, the second design key aspect was to allow the structure to fully exploit the increased deformation capacity by avoiding brittle collapse modes. To achieve this goal, corner beam column joint panels were strengthened by using two plies of quadri-axial GFRP laminates as well as a wall-type column for its entire length with two plies of the same quadri-axial GFRP laminates used for the above joints (see Figure 2).



(a) (b)

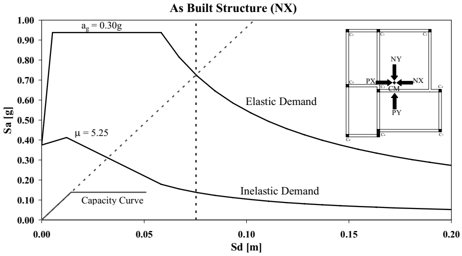
Figure 2. Column confinement and shear strength of corner joints (a); shear strength of wall-type column and retrofitted structure overview (b) (Di Ludovico et al. 2008a).

The assessment of structural global performance, before and after the strengthening intervention, was performed by nonlinear static pushover analysis in longitudinal direction (positive and negative X-direction, PX and NX, respectively) and in transverse direction (positive and negative Y-direction, PY and NY). In Figure 3, the theoretical base shear-top displacement curves for the 'as built' and FRP retrofitted structure are depicted with reference to direction NX (where the maximum capacity-demand gap was recorded for the 'as-built structure at the significant damage limit state LSSD).
Figure 3b clearly shows that the FRP retrofit is able to greatly increase the global deformation capacity of the structure, slightly affecting its strength. The comparison between the seismic structural capacity and both elastic and inelastic demand is reported in Figure 4 for direction NX by using the Capacity Spectrum Approach (CSA) (Fajfar 2000).

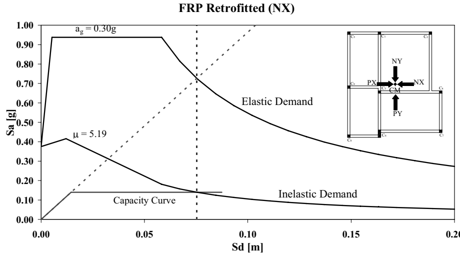


(a) (b)

Figure 3. Theoretical base shear – top displacement curves for 'as-built' (a) and FRP retrofitted structure (b), (Di Ludovico et al. 2008a).



(a)



(b)

Figure 4. Theoretical seismic performance comparison at 0.3g PGA between 'as-built'(a)and FRP retrofitted structure (b), (Di Ludovico et al. 2008a).

Figure 4 clearly shows that column confinement provides the structure with significantly enhanced ductility, allowing it to achieve the theoretical inelastic demand by only modifying the plastic branch of the capacity curve. After that columns and joints were wrapped with GFRP, the retrofitted structure was able to withstand the higher (0.30g PGA) level of excitation without exhibiting significant damage. After tests, FRP was removed and it was shown that the RC core was neither cracked nor damaged. The comparison between the experimental results provided by the structure in the 'as built' and GFRP retrofitted configurations highlighted the effectiveness of the FRP technique in improving global performance of under-designed RC structures in terms of ductility and energy dissipation capacity without significantly affect its strength (Di Ludovico et al. 2008b).

4 CONCLUSIVE REMARKS AND RECOMMENDED GUIDELINE UPDATE

The following recommendations can be made in order to update the existing EC8 – Part 3 provisions:

1) It is suggested to include an introductory section to the list of the three potential interventions using FRP. This section could provide principles about the main strategies that can be pursued when facing the retrofitting of RC framed structures. This could help the engineer to set the target of the intervention prior to designing specific FRP strengthening. This would also imply the addition of a section about shear strengthening of joints and flexural strengthening of columns.
2) It seems important to standardize the calibration process of confinement models by using the efficiency factor β, because the average absolute error of confinement models for circular cross-sections shows a remarkable decrease when the effective strain is considered. In particular, with respect to Eq. A.34 of EC8 – Part 3, it is recommended to provide more information about how to determine the adopted FRP jacket ultimate strain, ε_{ju}. This strain could be recommended to be the minimum among the following values:
   a) ultimate strain of the FRP jacket determined by means of flat coupon tests;
   b) strain value ensuring integrity of concrete with respect to mechanisms contributing to shear capacity of the member;
   c) strain of the jacket corresponding to the attainment of tridimensional Tsai Wu failure criterion. According to the discussion presented above, this strain can be obtained by multiplying the ultimate strain of FRP by the efficiency factor β;

   d) strain value corresponding to inter-laminar failure due to stress concentration at free edge of the jacket overlap, as discussed in the previous sections.
3) It is highlighted that simple geometrical considerations show that the confinement effectiveness factor (see Eq. A.36 in EC8 – Part 3) tends to have no physical meaning if the parabolas overlap, which occurs if $h<(b-2R)/2$ (assuming that $h<b$). With reference to these cross-sections, a recommendation could be added to compute the effective lateral pressure and the confined concrete strength according to Eq. (3) reported above.
4) It is also recommended to address the issue of FRP strengthening limits. When dealing with existing structures located in seismic zones, the engineer could find deficiencies due to either gravity loads or seismic loads. With this respect, the code should provide provisions about maximum strength increases depending on the type of actions.

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STEEL AND CONCRETE COMPOSITE STRUCTURES

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EUROCODE 8 PROVISIONS FOR STEEL AND STEEL-CONCRETE COMPOSITE STRUCTURES: COMMENTS, CRITIQUES, IMPROVEMENT PROPOSALS AND RESEARCH NEEDS

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ABSTRACT
This paper presents a summary of those aspects which are deemed to represent key issues for the correct maintenance of the current Eurocode 8 provisions for the design of steel and composite steel/concrete structures. The general design rules are first commented and, subsequently, aspects specific of each structural type are discussed. Based on the knowledge acquired in the last few years, weaknesses of the current code are then highlighted and improvement proposals presented. Topics requiring further research are eventually identified aiming at paving the way for the next generation of the European seismic code.

KEYWORDS
Seismic design, steel structures, composite steel-concrete structures, moment resisting frames, braced frames.

1 INTRODUCTION

Eurocode 8 (EC8) (CEN 2005*a*) is a prescriptive code, implementing a set of principles and rules to be satisfied in order to meet compliance of the structural performance with the code requirements. Recently, a different idea has encountered the favour of many people in both the scientific and practicing engineering communities: structural codes should be "performance-based" and they should only give the general requirements that a structure should meet. However, the role and value of strictly technical documents, such as the current version of EC8, remains intact, since designers do always need guidance as to how reach general performance objectives.

In the last few years, a strong and passionate discussion has taken place on the subject of the development of rules for seismic design of structures, especially in Italy, where the evolution of the seismic Code has often encountered the resistance of conservative positions. A recent history of this evolution can be found in Landolfo (2005). It seems that a "binary" way of thinking has taken dominance: (i) the "optimistic" view, sustaining the validity of the recent regulations without any doubt against the "old" regulations; (ii) the "pessimistic" view, emphasizing only the "bad" things of the new regulations. According to the Authors' opinion, the time is matured for a more equilibrated approach, which must recognize that the current

version of the Code is a good starting platform, over which modifications for improving the quality and the effectiveness of the new Code can be made.
This paper is a list of shortcomings and/or drawbacks of the current version of EC8 with reference to steel and steel-concrete composite structures. Although it could give the impression that the code is weak, it must be borne in mind that any code has inherent limitations and is susceptible of improvements, as much as the advancement of knowledge proceeds (Landolfo, 2008).
Therefore, the objective of this paper is only to highlight those aspects that, according to the Authors' view, should deserve either further investigation or modification. The discussion presented herein is intended for those specialised people working in the field, since no detailed explanation of the presented issues is given.

2 GENERAL DESIGN CONCEPTS

2.1 Behaviour factors and structural typologies

The force-based design procedure implemented by EC8 relies on the correctness of the behaviour factors. While concerns could be raised about the real background information behind the values of the behaviour factors currently fixed by the code, the lack of information for some important more recent structural types is easily recognized.
Table 1 highlights those aspects which should be either checked or added to the similar table implemented in EC8. Namely, bold characters highlight both those structural types which are currently not dealt with and those aspects which should be checked and eventually adjusted. The following paragraphs discuss very shortly each of the items highlighted in Table 1.
As far as the aspects to be improved are concerned, one controversial issue is about the design of V bracings, where one unique value of the behaviour factor is specified for both ductility class "medium" (DCM) and ductility class "high" (DCH). Indeed, recent research has shown that appropriately designed V-braced frames may reach design values of the behaviour factor of about 4 or even more (Della Corte & Mazzolani 2008). Therefore, the possibility to improve current design rules for V-bracings does exist.
The behaviour factors of eccentric bracings (EBs) is set equal to the values of moment resisting frames (MRFs). Generally speaking, MRFs possess a larger plastic redistribution capacity than EBs. Furthermore, no distinction is made between the use of short, intermediate or long links, even though the plastic deformation capacity of the frame is markedly affected by the type of link.
The mixed reinforced concrete (RC) walls and steel MRF structures are not explicitly dealt with, while they are one attractive solution to designers. Recent studies have shown that the plastic demand to MRFs coupled to RC walls is relatively small (Reyes *et al.* 2009). Consequently, the seismic design of the steel moment frames could significantly be relaxed in this case, meaning that capacity design rules do not need to be fully applied, thus saving the costs. This is one area where further research could profitably be carried out.
Similar to the previous case is the one of MRFs coupled with bracing systems. There are many studies showing that the combination of the two systems may be advantageous, mainly because the large plastic redistribution capacity of the MRFs allows damage concentration in the braced bays to be strongly reduced or even completely avoided.
One recent and very successful application in the field of seismic resistant steel structures is represented by buckling restrained braces. They are not mentioned in EC8 and this is recognized as one main gap in the current version of the code.

Recent research has also proved that one simple but effective way to avoid damage concentration in V-braced frames is to use vertical ties connecting the braces over the frame height ("zipper" bracing). This could also represent one area for the improvement of the code.
Finally, the case of MRFs with infills represent one controversial point of the code. In case of "unconnected" MRFs a very small value (q = 2) is assigned to the behaviour factor. This can be interpreted as the result of the brittle response supposed for the (masonry) infill panels. However, the code should recognize that the infill panels usually add stiffness and strength to the frame, thus reducing the displacement demand to the structure. It is frequently found that infilled frames behave better than bare frames. The presence of infill panels should explicitly be considered in the structural model and rules and suggestions should be given as to how this can be done. Besides, the different properties of different infill panels should be taken into account. It is likely that the steel frame is enveloped by a modern cladding system, with larger displacement capacity than classic masonry infills. The case of "connected" infills is correctly thought of as a case of composite structural action, but actually no specific rule is found in the code.

Table 1. Behaviour factors.

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames (MRFs)	4	$5\alpha_u/\alpha_y$
b) Concentrically braced frames (CBFs)		
Diagonal bracings	4	4
V-bracings	2 **(?)**	2 **(?)**
c) Eccentrically braced frames (EBFs)	4 **(?)**	$5\alpha_u/\alpha_y$ **(?)**
d) Inverted pendulum	2	$2\alpha_u/\alpha_y$
e) Mixed RC walls and steel MRF structures	**?**	**?**
f) Dual MRFs and CBFs	**4**	$\mathbf{4\alpha_u/\alpha_y}$
g) Dual MRFs and EBFs	**?**	**?**
h) Frames with buckling restrained braces (BRBs)	**?**	**?**
i) Frames with metallic shear panels (SPs)	**?**	**?**
j) "Zipper" bracing	**?**	**?**
k) MRFs with infills		
Unconnected	2 **(?)**	2 **(?)**
Connected	See Section 7 **(?)**	
Isolated	4	$5\alpha_u/\alpha_y$

There is a number of novel structural types which are spreading all over the World and should be included in the next generation of EC8.
The following is a list of such novel types:

1. Frames with buckling restrained braces.
2. Frames with metallic plate shear walls.
3. Special braced frames, with improved performance, such as the "zipper" bracing.

Many theoretical and numerical studies have recently been performed on these structural types (D'Aniello *et al.* 2008, De Matteis *et al.* 2007, Yang *et al.* 2007), which could be considered mature enough to be included in the code. It is worth noting that the first two types of novel systems is already included in the current AISC (2005) Seismic Provisions.

2.2 *Classification of cross sections*

Steel member cross-sections are classified by EC8 according to the same rules fixed by Eurocode 3 (EC3) (CEN 2005*b*). It has long been recognized that the classification for monotonic loading must be different from the one for seismic loading, because of strength deterioration induced by the repetition of inelastic deformations. Furthermore, it has been pointed out that shifting from a section-based to a member-based classification would represent one significant advancement of the code. More discussion about the cross section classification is provided at Section 3.1.

2.3 *Material random overstrength*

A well-established general concept in seismic design of structures is that non dissipative members must be designed on the basis of the expected material strength of the dissipative zones. The ratio between the expected (average) yield stress and the nominal yield value for a given steel class is called γ_{ov} by EC8. There is no specific information about the values to be attributed to γ_{ov}, but National Authorities have the freedom to select the most appropriate ones. However, a constant value of $\gamma_{ov} = 1.25$ is suggested, which is contradictory with the available experimental evidence of the dependence on the yield strength of the steel (Calderoni *et al.* 1994).

2.4 *Capacity design*

The rules implemented for capacity design in case of steel structures are different from those implemented for other materials, and this deserves some comments.

In case of steel structures, capacity design of non dissipative parts is regulated by a unique format applicable to all the different structural types covered by the code. Namely, earthquake-induced effects are increased by the factor $1.1\gamma_{ov}\Omega$, where γ_{ov} has previously been defined and $\Omega = \min\left(R_{pl,Rd,i} / R_{Ed,i}\right)$, where $R_{pl,Rd,i}$ is the design strength of the *i*-th plastic zone and $R_{Ed,i}$ is the required strength. Therefore, the design value of the generic internal action for non dissipative members is taken equal to $R_{Ed,i} = R_{Ed,G,i} + 1.1\gamma_{ov}\Omega R_{Ed,E,i}$, where subscripts "*G*" and "*E*" indicates the effect of gravity and earthquake loads, respectively. There are some controversial aspects in this approach. In case of small gravity load effects, amplifying the earthquake-induced counterpart of the required strength by the factor $\gamma_{ov}\Omega$ means that the internal actions corresponding to the first real plastic hinge formation are being calculated. However, in case of large gravity load effects, the proposed Ω factor markedly underestimates the real overstrength. One proposal of correction has been recently reported by Elghazouli (2008) and was formerly proposed within the first version of the recent Italian Seismic Code (OPCM 3274). The meaning of the multiplicative coefficient (1.1) is not clearly stated in the code. According to Elghazouli (2008) it is introduced to take into account strain hardening of steel and strain rate effects. Indeed, the coefficient 1.1 is proposed by the code also for capacity design of connections between a plastic zone and non dissipative parts of the structure. In this case the expected yield strength of the plastic zone $\gamma_{ov}R_{yd}$ is amplified by a factor again equal to 1.1. Alternatively, one could suppose that, in case of capacity design of columns, the coefficient tries to take account of the force redistribution occurring after the first plastic hinge formation. But, in this case using one single coefficient for any type of structure and any type of design criteria would be strongly questionable. In fact, the force redistribution capacity of MRFs is markedly different from that of braced frames and, for a given structure type, it may significantly change according to the design criteria used. Therefore, the exact meaning of this coefficient (1.1) and, especially, the rational background behind the assumed value, remains unknown. Rather, it seems to be one of those "magic" numbers which are sometimes encountered in the code, when a clear scientific background is missing. Amplifying the action effects due to earthquake loads only is also questionable, since the ratio between the actual strength of plastic zones and the strength required by all the loads should affect the internal actions to design non dissipative members and connections.

2.5 Structural regularity

The code suggests reducing by 20% the behaviour factor of buildings which are irregular in elevation. This rough estimation of the effect of vertical irregularity seems to be an oversimplification, which should deserve a deeper investigation. Many numerical results are available on this subject, which also shows that geometrical set-backs could even be beneficial in some cases (Mazzolani and Piluso 1997).

2.6 Floor diaphragms

Design rules for floor diaphragms and their connections with the vertical frames are lacking within the current version of EC8. Indeed, the code suggest multiplying the effects obtained from the seismic analysis by an overstrength factor γ_d, whose value is to be found in the National Annex. But, constant values equal to 1.3 and 1.1 are suggested for brittle and ductile failure modes, respectively. This assumption of a unique value, independent on the type of structure and its design criteria, seems to be inadequately oversimplified and should deserve further investigation.

2.7 Foundation connections

An harmonization of the design of foundation connections with the capacity design of columns is required. In particular, in case of composite steel-concrete constructions, the large flexural strength of composite columns poses serious problems to the practical implementation of capacity design rules (Di Sarno *et al.* 2007).

3 DESIGN ASPECTS SPECIFIC OF EACH STRUCTURAL TYPE

3.1 Moment resisting frames

Seismic design criteria for MRFs have long been studied (Mazzolani and Piluso 1995), but many results still needs to be fully exploited in order to justify the assumptions for the q-factor values in relation to the design criteria.

A very general problem with the current codified EC8 rules for the design of MRFs is the compatibility of maximum drifts imposed at the damage limitation limit state and the large behaviour factors for the ultimate limit state (Della Corte *et al.* 2002). It has long been recognized that the design of MRFs is dictated by drift limitations of EC8, which produces strong overstrength and, consequently, reduced ductility demand as well as increase of costs. Limitations on P-Delta effects could also be a source of significant frame overstrength (Elghazouli 2008).

Another important weakness of the current version of EC8 can be identified in the lack of any regulation about the detailing of moment resisting connections. There is a large amount of experimental and theoretical information about the response of beam-to-column connections

and panel zones, which have been basically produced in response to the Northridge and Kobe earthquakes (Mazzolani 2000, FEMA 2000). This information could profitably be used to form the basis of an upgraded code.
Analogously, one more area where the code could easily be improved, by fully exploiting the existing knowledge, is the classification of member cross sections. One useful proposal is that made by Mazzolani and Piluso (1996), who propose to establish a relationship between the member ductility and the stress ratio $s = f_c / f_y$ (i.e. the ratio between the peak collapse stress f_c and the yield stress f_y). Through a regression analysis of many experimental test results on beams with I-shaped cross-sections, the stress ratio is obtained as function of the flange and web slenderness (λ_f and λ_w), as well as the ratio between the distance of the cross section subjected to the peak bending moment from the contraflexure cross section (L^*) and the beam flange width (b_f). Recently, an upgrading and extension to tubular member cross sections has been made (Landolfo *et al.* 2008, Brescia *et al.* 2009). Based on the parameter *s*, the proposed cross section classification is given in Table 2. One additional advantage of the proposed classification is that only 3 classes are defined. Indeed, the difference between class 2 and class 3 of EC3 is somewhat troublesome from a conceptual point of view and usually a very small number of cross sections belong to class 2. Besides, the four classes based classification does not find any correspondence in other international codes.

Table 2. Classification of cross sections.

Cross section class	Cross section normalized strength (s)
Ductile	$s \geq 1.2$
Plastic	$1 \leq s \leq 1.2$
Slender	$s \leq 1$

The possibility of using slender cross sections, such as those typically characterizing cold-formed members, along with a q-factor larger than 1 has long been investigated by Calderoni *et al.* (2008). The current codification strongly penalizes the use of cold-formed members, since only a non-dissipative structural design approach is permitted for them.
The case of composite construction is one field of application where more research is needed with reference to MR frames. For example, the use of partially restrained vs. fully restrained moment connections should be clearly distinguished by the Code and specific design rules formulated for each of the two options. Significant experimental and theoretical studies have recently been carried out and some useful results are available (Thermou *et al.* 2002, Amadio *et al.* 2008, Bursi *et al.* 2008). As a further example, the currently codified rules for the classification of cross sections appear to be an oversimplification of the real problem. Recent research results (Pecce *et al.* 2009) could form the basis for an improvement of the code, by looking at consolidated knowledge for bare steel frames (Mazzolani and Piluso 1996).

3.2 Concentrically braced frames

The design of CBFs is regulated according to somewhat different criteria depending on the type of bracing. In fact, in case of diagonal bracing, the use of an elastic model with only-tension braces is prescribed, while in case of V bracings both the tension and compression braces shall be taken into account.
While using one single brace to calculate the ultimate storey shear strength could be considered as an acceptable simplification for slender braces, the adoption of an elastic model

with one single brace is difficult to justify even in this case. The procedure overestimate the elastic period of vibration, hence underestimate the elastic force demand on the braced frame. Consequently, a premature and uncontrolled compression buckling of braces will occur.

The post-buckling strength of braces is required in the design of beams in V bracings. The code assumes this post-buckling strength to be a fraction of the plastic strength in tension, i.e. $\gamma_{pb} N_{pl,Rd}$. Though the post-buckling strength coefficient γ_{pb} can be selected by each National Annex, a constant value equal to 0.3 is suggested. But, there is an extensive literature showing that the post-buckling strength is function of the brace slenderness (Tremblay 2002).

There is missing information in the code about the design of some types of unfrequent, but usable, bracing systems, such as X-bracing systems over two or more floors and bracing with vertical ties ("zipper" frames).

One point of the code deserving particular attention is the requirement about the height wise variation of the overstrength factor Ω_i. The difference between the values of Ω_i at two adjacent floors is prescribed to be not larger than 25%. The objective is clearly to avoid or, at least, to limit the occurrence of damage concentration in one or few stories. However, the coupling of this requirement with the upper limit on the brace slenderness $\bar{\lambda}_{br,i} \leq 1.3$, which is imposed in case of V-bracing, may produce strongly over-resistant braces. This occurs because the seismic shear force at the top storey is usually small, thus leading to small brace strength demand. Consequently, it would often be appropriate to select small cross section area and radius of inertia, i.e. large slenderness, at the top floor. In order to satisfy the limit on the brace slenderness, the designer is then forced to select larger cross sections at the top floor and, consequently, to every floor, because of the limitation on the brace design overstrength factor. Once braces have been overdesigned, the application of the codified capacity design rules obviously leads to large cross section areas for columns and beams, as well as to large force demand on connections. This design solution is obviously expensive, though the final overstrength will be on the safe side. One very simple way to avoid the problem, could be assigning at the top storey a conventional additional force, as percentage of the base shear force. This could also be advantageous to face higher mode effects which are pronounced at the top storeys.

Information is missing about using double-angle or double-channel cross section shapes for braces. For example, information about the stitch spacing is not provided, while experimental tests have proved that the brace ductility is significantly affected (Astaneh-Asl *et al.* 1996). Double angle and double-channel shapes are frequently encountered in the European Countries, where low-to-moderate seismic intensities are frequent.

Information is also missing about the design and detailing of brace to beam and column connections. The sensitivity of the real brace performance to the detailing of the connections is well known and documented by several experimental studies (Astaneh-Asl *et al.* 1998). Braces buckle by forming three plastic hinges, two at the ends and one at the middle of the brace. In case of out-of-plane buckling, the plastic hinges at the brace ends could be either permitted to form in the gusset plate connections or forced to occur in the brace. Though the latter option is characterized by larger energy dissipation capacity (Lee and Goel 1987), the former solution is economically advantageous, because it avoids using gusset plate transverse stiffeners. If plastic hinges are permitted to form in the gusset plates, then an appropriate length of the free space between the end of the brace and the assumed line of restraint for the gusset must be detailed (AISC 2005, Astaneh-Asl *et al.* 1986) in order to accommodate a plastic hinge but avoiding buckling . Besides, recent research has shown that alternative brace end connections could advantageously be employed (Martinez-Saucedo *et al.* 2008).

3.3 *Eccentrically braced frames*

According to EC8, capacity design of non dissipative members and connections outside the yielding link zone shall be conducted using the general rule of Section 2.4. The product of the material overstrength factor γ_{ov} and the Ω factor is shown in Figure 1*a*, as function of the normalized link length. An unacceptable and inconsistent discontinuity is observed at $\rho = 1.6$, i.e. at the transition between short and intermediate link lengths ($\rho = V_p e / M_p$ is the normalized link length). The EC8 codification is compared in Figure 1*a* with the one reported by Richards and Uang (2002). The inherent link overstrength of European shapes is considered to be larger than for US shapes. This is consistent with some experimental findings (Mazzolani *et al.* 2009). However, theoretical studies also suggest that the link overstrength should be considered length-dependent and shape-dependent (Della Corte *et al.* 2009). This is not currently recognized by the codes and could indeed represent one area of further research. Besides, the link classification is proposed for only I-shaped cross sections, what implies that the use of different cross section shapes is actually not possible. However, some documentation is available for other types of cross section, such as the tubular one (Fig. 1*b*). The available experimental and theoretical studies could form the basis for an extension of the design rules, making possible for the designer to select the preferred shape of cross section. This could be particularly important in some European Countries, such as Italy, where tubular shapes are easily available in the market as cold-formed products.

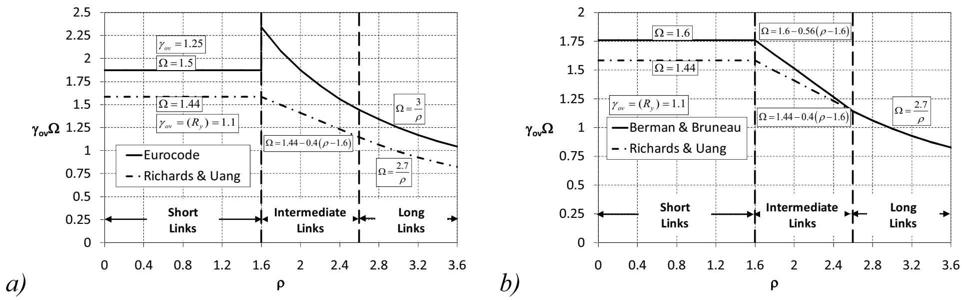

Figure 1. Link classification and overstrength.

3.4 *Dual systems*

Information on dual structural types (e.g. reinforced concrete walls and moment resisting frames, moment resisting frames with bracing) is lacking in the current version of EC8. Research results are available for such dual systems (Dubina *et al.* 2008, Reyes *et al.* 2009), which are among the most efficient structural types worth of consideration.

4 CONCLUSIONS

The paper has shortly commented a few issues related to the current Eurocode 8 regulations for the design of steel and composite steel concrete structures, highlighting those aspects deserving improvement or further research to be carried out. The Authors are aware that many other issues could/should be discussed and that the presented list is not exhaustive. However, the short list presented here could form the basis to plan the maintenance operations of the current version of the code. It is worth mentioning that such a maintenance process has

already started within the ECCS TC13 Committee about the seismic design of steel and composite steel-concrete structures (www.steelconstruct.com).
A "good" Code must necessarily have two requisites: (i) it must be equipped with a Commentary, explaining the reasons for the prescribed rules and (ii) the scientific and technical background of the Code must clearly be described (e.g. in the same Commentary). Unfortunately, both of these two requisites are not satisfied by the current version of EC8 and many of the rules prescribed in the Code are consequently obscure even to people working in this field either as researchers or practitioners. Notwithstanding, Eurocode 8 represents an advancement in the field of seismic design for European structures, with strong efforts done by many people involved into the Code development. The current version should profitably be used as the starting basis to develop more comprehensive and clear design rules.

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MASONRY BUILDINGS

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EXISTING MASONRY BUILDINGS: GENERAL CODE ISSUES AND METHODS OF ANALYSIS AND ASSESSMENT

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ABSTRACT
The paper presents first an introduction to the main differences between the EN 1998-3 approach to seismic assessment and strengthening of existing masonry buildings and the Italian norms approach. Then, issues related to the definition of material properties, to structural analysis, modelling and performance checks are discussed in more detail, pointing out the difficulties associated with the practical application of the Eurocode and reporting the Italian attempts to overcome such difficulties. The conclusions call for a thorough revision or re-draft of EN 1998-3 to take into account the specific problems of existing masonry buildings.

KEYWORDS
Existing masonry buildings, seismic assessment, analysis, modeling, codes.

1 INTRODUCTION

The problem of the seismic assessment and retrofit of existing buildings, among which a great number are unreinforced masonry, has become by now one of the main topics of interest in the world of constructions, also due to progressive relative reduction of new construction activity with respect to interventions on existing structures. The topic itself is extremely complex, due to the enormous variability of structural forms and materials that can be found in countries with a long history of civilization such as in Europe, ranging from simpler dwellings to spectacular monumental structures. Such variety constitutes a great hindrance to a strict codification of methodologies and approaches, such as it may be possible with new designs. Considering specifically masonry buildings, not only the diversity of structural forms and materials is enormous from country to country, but first of all, such structural forms very often do not lend themselves to be approached with the same engineering criteria used for reinforced concrete or steel construction, even when the simpler and regular residential buildings are considered.
The attempt of transposing Eurocode 8 part 3 (CEN-EN 1998-3) to the Italian reality (attempt made in the drafts of OPCM 3274, 2003, and OPCM 3431, 2005) presented, from the Italian viewpoint, a series of novelties that in part were a serious progress towards a safe and rational approach to assessment, in part were not compatible with the reality of the problem due to the impossibility to extend to masonry buildings concepts and procedures which would be appropriate for other types of structures such as r.c. or steel framed buildings.

An important step forward coming from EN 1998-3 was the introduction of the fundamental problem of the knowledge of the structure, and the conceptual definition of different knowledge levels and consequent confidence factors for assessment. However, in the rational definition of knowledge levels of masonry buildings it should be noted that in most real cases:

- no construction drawings neither structural design are available, neither test reports;
- the building was built in absence of design regulations, and in the best case conforming to a “rule of art”, so no “simulation of design” is thinkable;
- often the direct experimental measurement of material parameters is not feasible or, if in principle feasible, completely unreliable.

In the new Italian structural norms (now published in the NTC, 2008, document and in the relevant guidelines Circ. NTC08) it was therefore felt essential to define specific criteria for masonry regarding the different knowledge levels. A discussion of the issues pertaining to the knowledge of the building can be found in the paper by Binda and Saisi (2009) in these Workshop proceedings.

Another important issue, in which the Italian norms felt the necessity to introduce more articulation with respect to EN 1998-3, was the definition of the type of intervention in relationship with the increase in safety/performance level that is being pursued (local intervention, improvement/strengthening intervention, retrofit), taking into consideration the situations in which the complete retrofit of the building is not possible, as discussed in the paper by Borri and De Maria (2009). At the same time, it was felt that an informative and updated annex on strengthening techniques and strategies would have been an important complement to the norms, in the light of recent post-earthquake experiences in Italy. This issue is treated by Modena et al. (2009) within the present Workshop proceedings.

A considerable expansion and partial correction of the criteria for safety/performance assessment presented in chapter 4 of EN 1998-3 was considered necessary for masonry buildings regarding two main aspects:

- the need to address local mechanisms, often related to out-of-plane response of walls of or parts of the structure (Figure 1a), which could be approached by ad-hoc methods, and which constitute an essential guidance in the design of the strengthening/retrofit intervention; this issue is not addressed in EN 1998-3;
- the impossibility to apply to masonry buildings the schematic “ductile mechanism/brittle mechanism” conceptual framework which is applied to r.c. or steel structures and the consequent difficulty to apply the methodologies of analysis proposed in EN 1998-3.

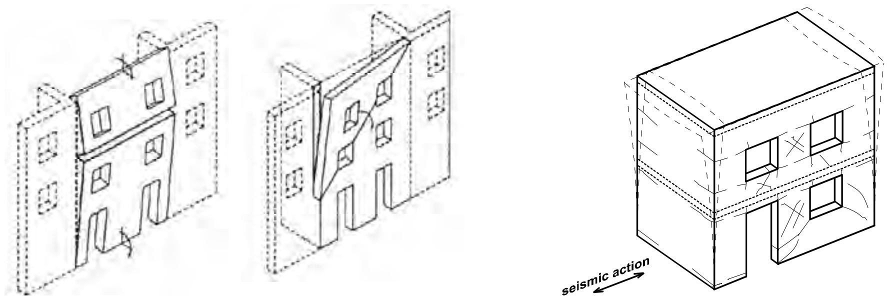
Regarding the first issue, the paper of Lagomarsino (2009) presents the approach that is presently suggested by the Italian code, and the possible use of limit analysis for the assessment of local mechanisms. The second issue will be discussed in the present work.

During an earthquake both out-of-plane and in-plane response are simultaneously mobilized, but it is generally recognized that a satisfactory seismic behaviour is attained only if out-of-plane collapse is prevented and in-plane strength and deformation capacity of walls can be fully exploited (Figure 1b). A global model of the structure is usually needed when the resistance of the building to horizontal actions is provided by the combined effect of floor diaphragms and in-plane response of structural walls (although in some cases simplified partial models could be used). In this paper, attention will be paid to the methods of “global analysis” of existing masonry buildings.

2 SEISMIC ASSESSMENT OF EXISTING MASONRY BUILDINGS

2.1 Local and global response

As demonstrated by the post-earthquake damage surveys carried out after all earthquakes affecting areas where masonry buildings are common, one of the main sources of vulnerability for such structures is associated to local failure modes, mainly due to out-of-plane response of walls. The building seismic response can be governed by such mechanisms when connections between orthogonal walls and between walls and floors are particularly poor. This is often the case in existing stone masonry buildings without tie rods and ring beams, with lack of interlocking at the connection of intersecting walls, presence of simply supported wooden floors and thrusting roofs. Only if connections are improved by proper devices (e.g. tie-rods), local mechanisms can be prevented and a global behavior governed by the wall in-plane response can develop.



(a) (b)

Figure 1. Examples of first-mode "local" damage mechanisms (a, from D'Ayala & Speranza, 2003) and global response mechanism (b).

2.2 Strength criteria for structural elements

Essential elements for global assessment of a building are suitable strength criteria for the structural elements. Eurocode 6 and 8, as well as the Italian norms make reference to walls (or piers) and to beams (or spandrels), for which strength criteria are provided.

Strength criteria for in-plane response of masonry structural elements are proposed in EN 1996 and in Annex C of EN 1998-3. They both include a strength criterion with a Coulomb-type formulation for the evaluation of shear strength:

$$V_R = f_v t l_c \tag{1}$$

where: t is the wall thickness,

l_c is the length of the compressed part of the cross section, and

$f_v = f_{vo} + 0.4\sigma_d \le f_{v,\lim}$ is the shear strength where: f_{vo} is the initial shear strength in the absence of vertical load, σ_d is the compressive stress along l_c, and $f_{v,\lim}$ is a limiting value for f_v, related to failure of units.

The definition of $f_{v,\lim}$ in Annex C of EN 1998-3 apparently is consistent with EN 1996, but in reality the latter sets $f_{v,\lim}$ equal to $0.065f_b$, where f_b is the compressive strength of the *unit*, while the former sets it equal to $0.065f_m$, where f_m is the *masonry* compressive strength. The rationale for this is unclear and the result leads to an evident underestimation of the shear strength in existing buildings.

A criterion for assessing the capacity of masonry elements subjected to normal force and in-plane bending is also reported in section C.4.2.1(3) of EN 1998-3, which is different from the procedure given EN 1996. The approach of EN 1998-3 seems more appropriate for seismic assessment, and is based on the evaluation of the ultimate moment capacity of a wall section subjected to in-plane forces. The same conceptual approach is followed by the Italian norms both for existing building assessment and for new masonry design.

In OPCM 3431 and Circ. NTC08, the possibility of applying an alternative shear strength formulation for existing masonry typologies such as undressed stone masonry is foreseen. Such a criterion, which follows the formulation by Turnšek and Sheppard (1980) as exemplified by Benedetti and Tomaževic (1984), had been introduced in the Italian standards since almost three decades, and it is representative of the common diagonal cracking failure typically observed on both regular and irregular masonry, and most of the Italian experimental literature and available data on historical masonry makes use of this criterion to interpret test results, either from shear-compression tests or diagonal compression tests.

In OPCM 3431 and NTC08, strength criteria for unreinforced masonry spandrel beams subjected to seismic action are also reported. Although their formulation is still mainly based on theoretical considerations, the inclusion of strength criteria for spandrel beams in standards is definitely a necessary reference for the assessment of multistory structures. Very few experimental data exist in this regard and a need for a number of specific experimental tests on such structural elements is here emphasized.

2.3 Element deformability

In the performance requirements and compliance criteria section, at point 2.2.1, EN 1998-3 includes a clear distinction between "brittle" and "ductile" elements and a note refers to the material-related annexes for a classification of elements and mechanisms. Annex C does not include any classification of brittle and ductile elements/mechanisms, since it is implicitly recognized that masonry failure modes can be moderately ductile to some extent, even when shear failures develop. The failure mechanism is definitely a discriminating factor in a wall displacement capacity. As shown in Figure 2, the ultimate drift capacity associated to shear failure modes is significantly lower than the one related to bending-rocking ones. Due to this experimental evidence Annex C of EN 1998-3, OPCM 3274 and NTC08 provide different ultimate in-plane drift limits for shear and flexure failure modes.

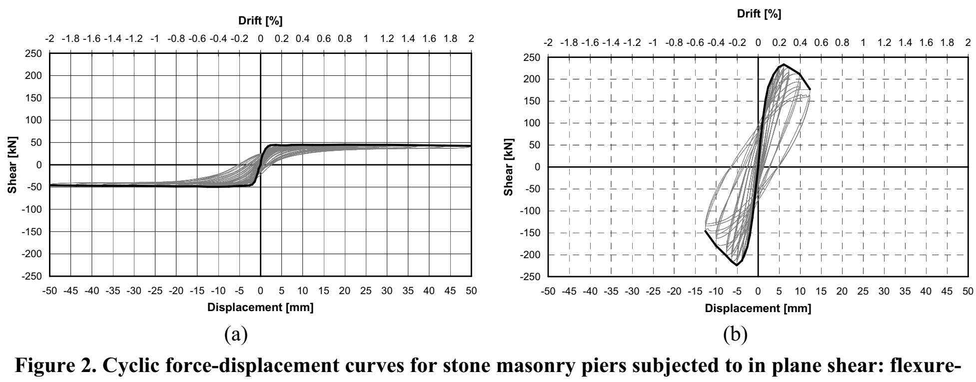


Figure 2. Cyclic force-displacement curves for stone masonry piers subjected to in plane shear: flexure-rocking failure (a) and diagonal cracking failure (b) (after Galasco *et al.*, 2009).

Nevertheless, the adopted limitations apply without distinction to all masonry typologies and levels of axial load while several experimental campaigns on full scale masonry specimens have shown that drift values corresponding to ultimate conditions (full capacity of carrying vertical loads and residual lateral stiffness) are generally quite scattered and can depend on failure mode, restraint conditions, masonry typology and applied axial load. As an example, the envelope curves of cyclic tests reported in Figure 3 show that in both cases a ductile behavior can be observed even in the case of shear failure mechanism,, but the limited drift capacity suggested in NTC08 can be non conservative in case of shear response. Recent experimental tests have shown that for double leaf stone masonry (Galasco et al., 2009) an ultimate drift capacity of 0.3% was measured. Similarly, as discussed in Magenes et al. (2009) some modern masonry typologies have shown ultimate drift capacities around 0.2-0.25%.

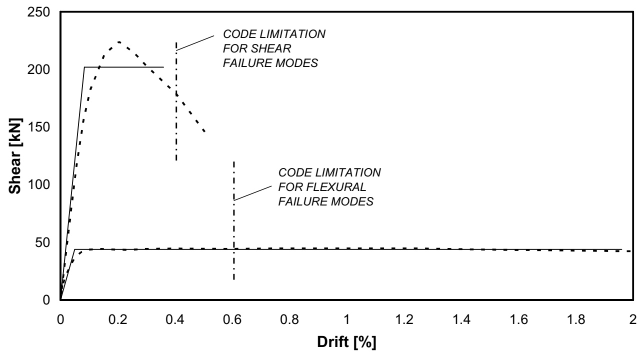


Figure 3. Envelope curves of cyclic tests on stone masonry piers (dashed lines) and equivalent bilinear curves.

While in principle similar drift limitations may also apply to spandrel beams, the results of the few available tests on such elements, although they still do not provide a statistically significant set of data for fixing the limiting values, seem to suggest that they could be higher than those for masonry piers (Gattesco *et al.*, 2008). This may be explained by the fact that the tested spandrel beams were supported by flexurally resisting lintels.

3 DEFINITION OF MECHANICAL PROPERTIES

As mentioned in the Introduction, often the in-situ direct experimental measurement of material parameters in existing masonry buildings is not feasible or, if in principle feasible, extremely unreliable. Regarding the latter comment, it is known how an in-situ measure of shear strength of stone masonry is presently possible only with destructive testing of panels of significant dimensions, rarely smaller than 1.0 x 1.0 m, through a self-equilibrated diagonal compression testing or more complex shear-compression test procedures (Sheppard, 1985, Chiostrini et al., 2003). The conditions for the feasibility of such tests may not be always satisfied, depending on the quality and texture of masonry, on the thickness of the walls, on the number of storeys of the building, on the availability of adequate experimental equipment. The use of easier, moderately destructive tests such as the “shove test” (ASTM C 1531-03) are conceivable only for regular brick masonry, which is only one of the many typologies that can be found in Italy (such as in other countries). In the new Italian norms it was therefore felt essential to define specific criteria for masonry regarding the different knowledge levels.
First, full geometric survey is always required, and information regarding structural details should specifically address: quality of connections between vertical walls, quality of

connections between floor/roof and walls, and presence of ring beams or other tying devices, presence of structurally efficient architraves/lintels above openings, presence of elements which can equilibrate horizontal thrusts, presence of structural or non structural elements of high vulnerability, typology of masonry (stone or brick, regular or irregular units, single-leaf or multi-leaf , with or without transversal ties…).
Regarding the quantification of material parameters, the category *limited in situ testing* which defines the minimum knowledge level, for which EN 1998-3 recommends at least one test per floor as a reference, has been substituted with *limited in situ investigations* where the mechanical properties of the material are estimated after visual inspections, removing plaster in selected areas to asses the texture and the connection between orthogonal wall, visual inspections through the thickness to recognize the internal level of connection of the leaves and the ability of the wall to behave monolithically through the thickness, qualitative assay of the mortar consistency. Such recognition of the typology and quality of the material is then used to associate it with the mechanical parameters reported in a reference table (an extract is given in Table 1), which was compiled on the basis of the experimental data available on the most common typologies.

Table 1. Reference values of the mechanical parameters and average specific weights for selected types of masonry (extract from Table C8A.2.2. of Circ. NTC08, 2009).

Masonry typology	f_m (N/mm^2)	τ_o (N/mm^2)	E (N/mm^2)	G (N/mm^2)	W (kN/m^3)
	min-max	min-max	min-max	min-max	
Irregular stone masonry (pebbles, erratic, irregular stone)	1.0 1.8	0.020 0.032	690 1050	230 350	19
Uncut stone masonry with facing walls of limited thickness and infill core	2.0 3.0	0.035 0.051	1020 1440	340 480	20
Cut stone with good bonding	2.6 3.8	0.056 0.074	1500 1980	500 660	21
Soft stone masonry (tuff, limestone, etc.)	1.4 2.4	0.028 0.042	900 1260	300 420	16
Dressed rectangular (ashlar) stone masonry	6.0 8.0	0.090 0.120	2400 3200	780 940	22
Solid brick masonry with lime mortar	2.4 4.0	0.060 0.090	1200 1800	400 600	18

The next level is defined as *extensive in-situ investigations*. At such level, the visual inspections described in the previous level are carried out extensively and systematically with superficial and internal samples for every type of masonry present, while tests with double flat jacks and tests for characterization of the mortar (type of binding agent, type of aggregate, binding agent/aggregate ratio, etc.) and eventually on stone and/or bricks (physical and mechanical characteristics) are required to verify the correspondence of the masonry to the typology defined in the reference table. A test for every type of masonry present in the building is required. Non-destructive testing procedures (sonic tests, sclerometer tests), penetrometer test for mortar, etc.) may be utilized as complementary to the required tests.
Finally, *exhaustive in-situ investigations* serve to obtain direct quantitative information on the material strength. Apart from the visual inspections of the internal samples and the tests

mentioned in the previous levels, a further series of experimental tests have to be carried out, both in quantity and quality, in order to be able to estimate the mechanical characteristics of the masonry. The measurements of the mechanical characteristics of the masonry are obtained by means of in-situ and laboratory tests (on undisturbed elements extracted from the structure). The tests can generally include diagonal compression tests on panels or combined tests of vertical compression and shear. Non-destructive testing methods can be used in combination, but not as a substitute, of the aforementioned tests.

The results of the tests have to be examined and considered within a general typological frame of reference which takes into account the results of the experimental tests available in the literature up to that time for the masonry typology under investigation, and that allows the estimation of an effective representative of the values found, even in statistical terms. The results of the tests have to be utilized with reference to the values reported in the reference table. The Regional governments can define specific supplemental tables for masonry typologies recurring within the regional territory.

In addition to the prevision of a knowledge level that does not necessarily foresee direct experimental testing, a very important element introduced in the Italian norms is the possibility of using experimental information obtained in other buildings: when a clear and proven typological continuity exists in terms of materials, masonry unit sizing, construction details, etc., between the building under investigation and others situated in the same zone, tests conducted on the other buildings can be utilized as a substitute. The Regional governments can define such homogeneous zones to meet this purpose, taking into account the uniqueness of structural typology within their territory.

4 ANALYSIS METHODS

4.1 Available models for global analysis

Dealing first with the problem of global analysis of the structural system, linear methods and nonlinear static methods are the ones that can be used in common practice. The last two decades have been characterized by a significant progress in nonlinear methods of analyses of masonry structures, to the extent that now a rather reliable nonlinear pushover analysis of buildings is a real possibility also for practice. Considering the problem of seismic design/assessment of masonry buildings, the need for non-linear analysis had been recognized in Italy and Slovenia as early as in the late 1970s, after the 1976 Friuli earthquake, after which an equivalent static, simplified non-linear assessment method was proposed and developed in Slovenia by Tomaževic (1978). Such method, which has undergone several refinements in the subsequent years (Tomaževic, 1999), is based on the so-called "storey-mechanism" approach, which basically consists of a separate non-linear interstorey shear-displacement analysis for each storey, where each masonry pier is characterized by an idealized non-linear shear-displacement curve (typically elastic-perfectly plastic with limited ductility).

The need for more general methods of analysis has stimulated the research on the subject and analytical methods have made significant progress in the last decades, particularly in the field of finite element analyses (Calderini & Lagomarsino, 2008, Lourenço, 2002, Milani et al., 2006; Massart et al., 2004; Cecchi & Sab, 2008; Brasile et al., 2007). Still, despite such progress, each model has a range of validity which needs to be understood with care, and the use of such tools requires high expertise, and in many cases can be applied to problems that are limited in size; therefore refined nonlinear finite element modeling does not constitute yet a suitable tool for the analysis of whole buildings in everyday engineering practice, especially when considering the task of designing/assessing ordinary low-rise residential buildings.

For this reason, several methods based on macro-element discretization have been developed, requiring a low to moderate computational burden. As a development of several basic ideas of the "storey-mechanism" approach, a nonlinear method based on an equivalent frame idealization of multistorey walls was developed and implemented at the University of Pavia (Magenes and Della Fontana, 1998, Magenes et al., 2006). The method was developed from the consideration that the distribution of internal forces at ultimate is basically governed by strength of members and by equilibrium. If a sufficient plastic deformation capacity in the piers is assumed, their initial elastic stiffness is therefore not as important as the definition of suitable and sufficiently accurate strength criteria, and simple bi-linear (elasto-plastic) or multilinear formulations can yield effective results, also when compared with more refined nonlinear f.e.m. analyses or experimental results.

When an equivalent static analysis is used within the context of seismic assessment, the strength and deformation properties of the members must be defined with reference to the experimental envelopes obtained from cyclic tests on wall elements, in particular regarding the ultimate deformation capacity (expressed in terms of chord rotation or drift capacity, θ_u). The ultimate deformation capacity of masonry is governed mainly by shear failure mechanisms. The values of θ_u =0.4-0.6 % are currently suggested by EN 1998-3 and NTC 2008, but they should be revised in the light of what said above in section 2.3 .

Other works have explored and verified the suitability of nonlinear equivalent frame modeling for unreinforced masonry (Kappos et al. 2002, Roca et al., 2005). An efficient equivalent-frame formulation can allow the dynamic global analysis of whole buildings (Lagomarsino et al. 2007), when only in-plane response of walls is considered. Software packages for nonlinear pushover analyses of masonry buildings have also become recently available to the public (among others, Magenes et al., 2006, Lagomarsino et al., 2006) in Italy.

The comparison between linear and nonlinear analysis results, which has been made possible by the progress in modeling techniques, is the best way to appreciate the limits of elastic analysis when applied to ULS seismic assessment and the reason why masonry structures were the first for which at a code level a simplified nonlinear approach was felt necessary in real applications. Examples of such comparisons under static lateral loads can be found, e.g., in (Magenes and Della Fontana 1998) and Magenes (2006). In the nonlinear f.e.m. models the calculated shears in walls show dramatic differences when compared to linear elastic f.e.m. At ultimate conditions, the moderate "ductility" of the wall piers (it would be more correct to speak of nonlinear behaviour) tends to a situation where forces are shared according to strength capacity, not according to initial elastic stiffness.

Similar considerations can also explain why, especially for the analysis of historical masonry structures, the approach of limit analysis has been proposed and used for quite a few decades now for the estimate of the ultimate static capacity of masonry systems, initially for vertical loading only, subsequently also for horizontal loads simulating seismic forces (Como & Grimaldi, 1985, Giuffré, 1993), and have been proposed in the Italian codes for the assessment of local mechanisms (Lagomarsino, 2009).

4.2 Linear vs. nonlinear analysis

In section 4.4.3, EN 1998-3 reports that the seismic action effects may be evaluated using one of the following methods:

a) lateral force analysis (static linear),
b) modal response spectrum analysis (linear),
c) non-linear static (pushover) analysis,
d) non-linear time history dynamic analysis.
e) *q*-factor approach.

Except in the *q*-factor approach, the seismic action to be used shall be the one corresponding to the elastic (i.e., un-reduced by the behaviour factor *q*) response spectrum or its equivalent alternative representations given in EN 1998-1. Despite the apparent broad spectrum of alternatives that the designer is offered, in the case of masonry buildings many of the above options are impracticable. Options a) and b), to be applied with the un-reduced elastic spectrum, assume that a clear distinction between ductile and failure mechanisms is feasible. For masonry structural elements, this is seldom the case, as can be seen from experimental tests, and in the great majority of cases, shear-dominated or hybrid shear-flexural failure mechanisms govern the in-plane response of walls. To classify such mechanisms as plainly "brittle" without whatsoever nonlinear deformation capacity would lead to a spectacular underestimation of the seismic capacity of the building, and therefore to unusable results. A moderate "ductility" has to be recognized also for shear failures.

Nonlinear time history analysis (option d) cannot be considered a viable tool for practitioners and for the standard applications (residential buildings). Non linear static analysis and linear analysis (whether static or modal response spectrum) with the q-factor approach seem the real options for designers presently. EN 1998-3 in principle recognizes the particular situation of masonry buildings, specifying that the above-listed methods of analysis are applicable subject to the conditions specified for other buildings, "with the exception of masonry structures for which procedures accounting for the peculiarities of this construction typology need to be used", and the complementary information on these procedures should be found in the relevant material related Informative Annex (Annex C). Looking at Annex C, section C 3.2, it can be seen that further additional limitations are given for the application of the various methods, especially for the linear static and linear multimodal analysis, which are then allowed only if a series of requirements are satisfied, typical of very regular buildings with stiff diaphragms. Such requirements, not satisfied by a great part of existing buildings, apparently leave only one option left to the designer, which is nonlinear analysis. The idea of favoring static nonlinear approaches can be agreed upon, nevertheless it seems that, when dealing with existing buildings, more freedom should be given to the designer in the choice of suitable methods of analysis.

In the companion paper by Magenes at al. (2009) regarding seismic design of new masonry buildings the drawbacks of linear analysis and the problems of the *q*-factor approach are discussed. Regarding linear methods, the considerations regarding overstrength ratio (OSR) and the limits of elastic analysis are applicable also to existing buildings. However, the problems of selecting appropriate values for the OSR in existing masonry buildings are magnified by the wide variety of structural configurations that can be found. As a first remark, it is essential to realize that a global analysis of the building is meaningful when presence of ties and diaphragms and appropriate connections can guarantee a global box-type action. However, diaphragms in old building are often flexible, providing a lesser degree of coupling of the walls that tend to vibrate more independently. Retrofitting interventions may lead to a stiffening of the diaphragm, but not to the extent in which it can be considered as rigid in a global analysis. In such situations, all methods of analysis currently available to designers tend to give a rather approximate picture of the response. If, on one hand, an elastic 3D model can be used to obtain a better understanding of the initial periods and modes of vibration, still the elastic strength assessment at ULS would present all the inconvenients of elastic analysis. In principle a global elastic model could be used to single out subsystems that could be analyzed in a second stage with static nonlinear models. When flexible diaphragms are present, the in-plane response of walls could be assessed by analyzing separately the capacity of co-planar systems, as discussed later.

Once the first-mode mechanisms are prevented, the redundancy of masonry buildings is such that an OSR of 1.5 should be considered adequate in elastic analysis (as suggested by OPCM 3431 and NTC 2008), so that the *q*-factor values for existing masonry buildings in NTC 2008 $q = q^{*}\cdot OSR$ can be evaluated assuming $q^{*}=2.0$ for regular buildings and $q^{*}=1.5$ for irregular buildings, yielding q=2.25÷3.0. Although further experimental and theoretical research is needed to fully support a generalized use of such an OSR value, from a pragmatic point of view its use generates results which are more consistent with experience and with the application of nonlinear analysis procedures. The possibility of applying linear elastic methods of analysis with reasonable levels of seismic loads is important especially when complicated geometries and irregular positioning of openings do not allow a reliable macro-element discretization of the building (refined nonlinear f.e.m. analyses are not an option for most practicing engineers).

As a general remark for code makers, when dealing with existing buildings, an exceedingly conservative underestimation of global seismic resistance is to be avoided possibly more than in new design. Whereas strengthening interventions aiming to increase the robustness and redundancy of the system by improving and inserting connections to avoid first mode damage mechanism are in general always positive, the increase in global strength is more difficult to achieve and it is pursued sometimes with heavy, non conservative (from the architectural point of view) interventions whose effects may be very difficult to predict, and whose effectiveness can be sometimes arguable, as past experiences have shown.

4.3 Non linear static analysis

The basic assumption of nonlinear static analysis is that the pushover curve is an envelope of the responses of dynamic analyses. The accuracy of this assumption is strongly dependent on a correct choice of the force pattern, or of the incremental control criterion of the analysis . For fixed force pattern methods, two possible distributions are commonly adopted: modal and mass-proportional. The former consists in applying a force distribution proportional to the first modal shape of the structure; it is able to represent the structural dynamic amplification, which increases the action on higher storeys. On the contrary, the latter can be consistent with a soft ground storey response. These two distributions may be assumed as bounds for seismic analyses of regular buildings: the actual result, coming from dynamic analyses, is usually assumed to be within these two solutions. However, the higher or smaller accuracy in the prediction of static analyses may depend on the evolution of the structural response due to damage (Lagomarsino *et al.*, 2007). In case of irregular structures, for example, it is possible that both distributions over- or under-predict the actual global response. In several cases the choice of a fixed distribution is a limitation: the structure indeed starts with a dynamic response which can be well represented by a modal force distribution but, as the damage progresses, the structure modifies its response and, near collapse, its behaviour may be better approximated by a mass-proportional force distribution or other. For this reason, several types of adaptive pushover analyses have been proposed, in which the control of the analysis is progressively updated to take into account the structural response evolution in terms of stiffness degradation, variations of modal properties and so on. The problems of nonlinear pushover methods counterbalance to some extent the limits of linear analysis, and in some cases it is recommendable to use different modeling approaches for the same structure and critically compare the results.

A case in which nonlinear methods, albeit static, should be favoured is when mixed masonry and r.c. buildings are considered (e.g. buildings with vertical structures made of masonry walls and members of other materials such as r.c. walls or frames). From the early 20th-century the spreading of reinforced concrete technology caused the birth of mixed solutions

starting from existing structures in order to satisfy requirements mostly related to functional purposes: masonry structures subjected to internal demolishment, column insertions, r.c. staircases insertions, plan enlargements or raisings by mean of steel or concrete structures. Furthermore, the insertion of additional r.c. walls or steel bracings is one of the current techniques which is being used for strengthening existing buildings, especially when the "global" seismic capacity of the system is deemed insufficient.
Whereas no clear guidance is given in EN 1998-3 on how to analyze mixed structures, OPCM 3431 and NTC 2008 favour the use of non linear static analysis, given the very different stiffness and deformation capacity of the structural members..

5 ISSUES RELATED TO FLEXIBLE DIAPHRAGMS

The presence of in-plane flexible diaphragms, typically wooden floors and roofs as well as thin masonry vaults, is very common in the existing masonry building stock. Even though proper connections between walls and floors allow to prevent local first mode mechanisms, in masonry buildings with flexible floors the global seismic response is quite complex. Since no or little coupling effect can be operated by the horizontal structures, vertical structures (walls) tend to behave independently.
As discussed in Galasco *et al.* (2006), the use of non linear static analysis (pushover) in case of masonry buildings with in-plane deformable diaphragms presents issues and difficulties which have not been yet taken into consideration in codes. As an example, the choice of the control node is in such cases particularly critical, where a storey center of mass can not be assumed as the control node and significant differences can be observed in the displacements of different points at the same storey. A possible solution may consist in creating an equivalent s.d.o.f. system that does not depend on the initially assumed modal shape but is a function of a conversion factor calculated on the current deformed shape. However, an acceptable approach in practice could be to analyze separately the in-plane seismic response of each masonry wall as extracted from the global structure with its pertaining loads and inertial masses (Figure 4).

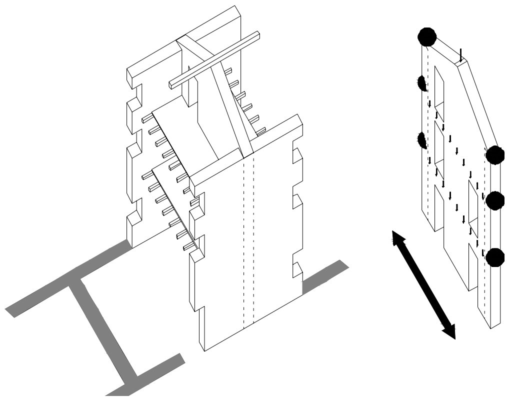


Figure 4. Wall model idealization in case of very flexible floors.

5.1 Aggregates of buildings

A specific problem of urban historical centres is the issue of the seismic vulnerability of conglomerations or blocks of masonry buildings. They are the result of the progressive

growth of the urban tissue, in which elevations are added to existing buildings and enlargements or progressive growths in plan are made by adding structural cells or units in contact with the previously existing, so that often adjacent units share the same boundary wall. Within a similar situation, the distinction of structurally independent units is problematic if not impossible, and any structural analysis that aims at evaluating the “global” response should in principle either model the whole block or model the structural unit with suitable boundary conditions that take into account the effect of the adjacent ones, but both options present a number of almost insurmountable practical hindrances for professionals.
OPCM 3431 has been the first Italian code presenting guidelines for the seismic assessment of structural units part of building aggregates, whereas this issue is not covered by EN 1998-3.
Since any method of structural analysis which considers only the structural unit of interest is clearly conventional, OPCM 3431 and Circ. NTC08 allow the use of simplified approaches.
According to these guidelines, in case of sufficiently rigid floors, the conventional verifications for the ULS and DLS of a structural unit can be performed, using the static non-linear analysis, by analyzing and verifying separately every floor of the building and neglecting the variation in axial force in the masonry piers due to the effect of the seismic action. Except for the corner or head structural unit (of an aggregate), or parts of buildings not constrained or bonded on any side to other structural units (e.g. upper floors of a building of greater height with respect to all adjacent structural units), the analysis can also be performed neglecting torsional effects.
When the floors of the building are flexible, analysis of single walls or of systems of coplanar walls can be carried out, each analyzed as an independent structure subjected to relevant vertical loads and seismic action in the direction parallel to the wall (Figure 4). Nonlinear time-history analysis of a simple aggregate (Figure 5) with flexible diaphragms allowed to verify the effectiveness of such simplified approach when looking at the transversal response. In the case studied in Figure 5, the higher displacement demand in the transversal end walls and the concentration of floor deformations in the external units can be fairly captured fairly by analyzing separately each transverse wall. Research on these issues is still strongly needed.

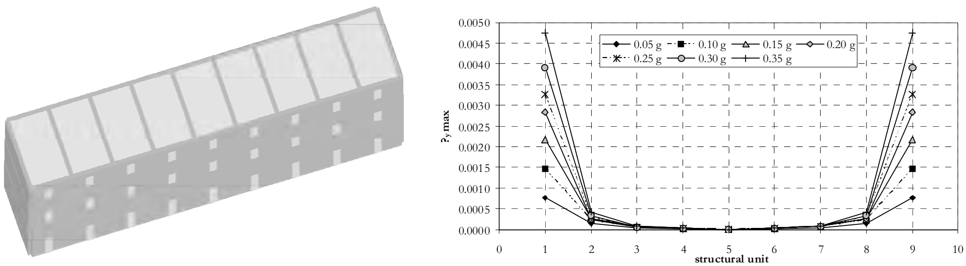


Figure 5. In-plane floor distortion computed for the different structural units of a ‘row’ configuration building aggregate with flexible diaphragms (nonlinear time-history analyses).

6 CONCLUSIONS

From what discussed in this paper, in great part related to methods of seismic analysis and assessment, it appears that the current structure of EN 1998-3 is applicable with great difficulty and only in very few cases to real existing masonry buildings. It is the authors’ opinion that a thorough revision of the code should be considered regarding some fundamental steps of the analysis and assessment procedures, to the extent that a new re-draft of the code could perhaps be more effective than amending the current text.

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SEISMIC BEHAVIOUR AND DESIGN OF NEW MASONRY BUILDINGS: RECENT DEVELOPMENTS AND CONSEQUENT EFFECTS ON DESIGN CODES

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ABSTRACT
The paper gives an overview of recent developments in the field of masonry technologies and seismic design criteria for new masonry buildings, drawing mainly from experimental researches and coordinated projects carried out in Italy and Europe. In the first part, some code-related issues are discussed with reference to the Italian norms and Eurocodes. In the second part, some significant outcomes of recent research projects on unreinforced and reinforced masonry systems are reported, with possible interaction with design code regulations. Suggestions for code improvement are given.

KEYWORDS
Seismic design, unreinforced masonry, reinforced masonry, new buildings, experimental testing.

1 INTRODUCTION

There is a rather generalized negative attitude towards the use of structural masonry for new buildings in seismic areas, since most collapses and deaths in recent earthquakes are due to inadequate performance of unreinforced masonry (URM) buildings (usually non-engineered, low-quality, old dwellings). This explains why the large majority of the current scientific and technical literature on seismic behaviour of masonry is dedicated to the study of existing structures and very seldom structural masonry is being nowadays considered as a choice for the design of new structures in seismic areas.
However, it is essential to recognize that the wide majority, if not the entirety of the collapses of URM masonry buildings in the recent earthquakes involved buildings which did not comply with most of the requirements that any new masonry building would have to satisfy according to the current seismic codes as regards code enforcement/construction control (see, e.g., Decanini et al., 2002).
On the other hand, on the base of the past observational experience and of the safety levels accepted in codes, the behaviour of structurally detailed unreinforced masonry buildings should be considered adequate with respect to the ULS (severe damage) with design PGA up to 0.2g (475 years return period), and an accurate design and construction of low rise structurally designed and engineered URM buildings should be possible also for a design PGA up to 0.3 g (Magenes, 2006). For higher seismic hazards, the solutions of confined or

reinforced masonry are available, whose competitiveness in seismic areas is however not fully recognized in several European countries.
The construction of new masonry buildings in European countries is far from being marginal, even in countries with appreciable seismic hazard, and is still a very competitive choice for low rise residential buildings.
In recent years the authors were involved in the drafting of new design codes for Italy, and in coordinated European projects on masonry design. A great part of what will be discussed in this paper draws from such an experience and more generally from the recent Italian experiences and researches on masonry structures. Some comments regarding selected issues pertaining to code implementation will first be given, with main reference to Eurocode 8 (CEN EN 1998-1, 2005) and to the recent Italian codes OPCM 3274 (2003) and OPCM 3431 (2005) and NTC (2008). Subsequently, some recent developments and results from co-ordinated researches on unreinforced and reinforced masonry systems will be discussed.

2 STRUCTURAL MASONRY IN ITALIAN CODES AND IN EUROCODES

2.1 Formulation and recommended values of q-factors

In seismic design or assessment of buildings, modern codes, including EC8 and the Italian NTC, consider four main methods of structural analysis: linear static (or simplified modal), linear dynamic (typically multimodal with response spectrum), non linear static ("pushover"), non linear dynamic.
In the design of modern structures, the structural details (e.g. slenderness limits to the walls, connections) should prevent out-of-plane collapse and the in-plane response of walls should be checked through methods of global analysis of the structural system.
Methods of global analysis that are used in common practice are essentially elastic linear (static or dynamic, usually through f.e.m.-based software) or non-linear static methods based on storey mechanism (Tomaževič, 1999) or on equivalent frame or macro-element idealizations (Magenes et al. 2006, Lagomarsino et al., 2006).
In the case of linear elastic approaches, the safety check procedure is usually based on at least two-level performance requirements (no collapse and damage control); at ultimate (ULS) the safety check consists of a strength verification, whereas for damage control (DLS) the check is made on deformation (drift) demands. According to the performance targets, each limit state (LS) is associated to a specific level of seismic action, which corresponds to a given probability of exceedance or a given return period. In general, for masonry structures the ULS verification is prevailing with respect to DLS. The ULS verification in carried out by checking that in each structural element the design resistance is not exceeded according to the strength criteria defined in codes. In other words, the ULS safety requirement is not met if the shear strength or the flexural strength of even just one element is exceeded.
The choice of the numerical values of the seismic force reduction factor, or q-factor, to be used to reduce the elastic design spectrum ordinates is obviously crucial for the linear procedures. Such choice is left by Eurocode 8 to national authorities, i.e. the values of q are Nationally Determined Parameters (NDP). For URM the current version of EN 1998-1 suggests a range between 1.5 and 2.5, keeping however as *recommended* value the lower limit, q=1.5, whereas larger values are suggested for confined and reinforced masonry
In May 2003 a new national seismic code was issued in Italy (OPCM 3274). The new code had been conceived as a document of transition from the previous national seismic code, dating 1996, towards the final adoption of Eurocode 8, and to this end many elements of the latter had been included, among which the limit state formulation and the recommended q-

values for masonry buildings (lower bound values). The first application of such design code to common real practical cases, following the standard linear analysis procedure (be it static or multimodal), without resorting to force redistribution, showed the following:
- with a q of 1.5 or even 2, it was practically impossible to satisfy strength safety checks for any configuration of URM, two- or three-storey building for a_gS greater than 0.1g; in numerous cases the strength safety checks would not be satisfied even for $a_gS = 0.05g$;
- the results of the analytical safety checks via elastic analysis clashed with the past experience and the experimental evidence;
- the results obtained via elastic analysis were in great contradiction with the results of nonlinear static procedures, which would produce results more in line with experience.

No appreciable improvement of the situation was observed resorting to force redistribution after the linear analysis, within the specified code limits (which were identical to Eurocode 8). Already in 2004, Benedetti (2004) and Magenes (2004) had shown how such contradictory panorama could be explained with the recognition of an “overstrength” ratio (OSR) also for masonry buildings. For masonry, the OSR is a consequence of the fact that the elastic analysis would predict “failure” of a structural element for a level of base seismic shear that is much lower than the ultimate strength that the structural system can provide. The formulation of the q-factor can thus be given by the product:

$$q = q^* \cdot OSR \tag{1}$$

For URM buildings, the OSR can reach quite high values since in most cases, consequent to the force redistribution that takes place as the strength capacity of different elements is progressively achieved, the internal force distribution at ultimate will differ substantially from the “stiffness-proportional” distribution resulting from linear elastic analysis. On the basis of the above considerations and studies, in the recent Italian codes (OPCM 3431, 2005, NTC, 2008) the *q*-factors for new unreinforced and reinforced masonry buildings were corrected as follow:

Unreinforced masonry buildings	$q = 2.0\ \alpha_u/\alpha_1$
Reinforced masonry buildings	$q = 2.5\ \alpha_u/\alpha_1$
Reinforced masonry buildings with capacity design principles	$q = 3.0\ \alpha_u/\alpha_1$

where α_u/α_1 is the OSR, for which the following values were suggested:

Single-storey unreinforced masonry buildings	$\alpha_u/\alpha_1 = 1,4$
Two- or more storey unreinforced masonry buildings	$\alpha_u/\alpha_1 = 1,8$
Single-storey reinforced masonry buildings	$\alpha_u/\alpha_1 = 1,3$
Two- or more storey reinforced masonry buildings	$\alpha_u/\alpha_1 = 1,5$
Reinforced masonry buildings with capacity design principles	$\alpha_u/\alpha_1 = 1,3$

However, as pointed out by Magenes and Morandi (2008) and Morandi and Magenes (2008), there are conceptual difficulties in defining a rational approach for the evaluation of a single conservative value of the *q*-factor (i.e. of the OSR) for a specific masonry. Despite the ongoing research carried out by the authors is approaching some rational criteria for a better definition of the *q*-factors for elastic design/assessment, the limitations of linear elastic models in the seismic analysis of masonry buildings are such that more consistent results could be achieved only favouring nonlinear procedures, albeit simplified, or the combined use of linear and nonlinear methods, the latter becoming more and more approachable by practitioners thanks to the advances in software tools for the analysis of masonry systems. In this perspective, deformation/displacement based design methods could allow a more rational solution of the problem.

2.2 Limits to force redistribution after linear analysis

The recognition of the necessity of the OSR in masonry design/assessment, which has been introduced in the Italian norms, was certainly an important step to rationally explain and find a rapid solution for the inconsistencies found in the application of the code. Nevertheless, the choice of a specific value of OSR, even for the same homogeneous typology of masonry buildings, does not overcome completely the intrinsic problems of the linear methods of analysis. Considering a homogeneous class of two- and three-storey buildings, the choice of a single conservative value, be it the minimum or a “sufficiently conservative” percentile (e.g. 1.4÷1.8 as proposed in the Italian norms), has the consequence that in the wide majority of the cases, in which the OSR is much higher (e.g. 2.5 or 3), the design seismic action will be much higher than it should. For such configurations, the use of a default conservative OSR could be so penalizing that the strength safety checks can never be satisfied, even if the quality of materials, the structural configuration and details, the total amount of shear walls clearly show that the design should be safe. It is very useful in such a situation, to resort to a *redistribution of internal forces* after the linear elastic analysis is carried out. This possibility was considered by design codes, including Eurocode 8, since the very early drafts. However, the limits to force redistribution have been so far so strict to make redistribution almost ineffective in many practical problems. The current limit given by Eurocode 8, states that the shear in any wall is neither reduced more than 25% nor increased by more than 33%. The origin of these limits dates back to 1985 or earlier (CIB, 1985), at times in which the experience in nonlinear analysis was quite limited, and recalls criteria originally developed for reinforced concrete structures. As shown in other works (e.g. Magenes, 2006), the problem of elastic analysis is that it does not provide a correct distribution of internal forces with reference to ultimate limit state, and that differences with more accurate nonlinear analyses tend to be much higher than the limits imposed for redistribution. A more rational approach would possibly be to allow a larger redistribution which could allocate shears approaching the available strength reserve of the walls, as would be the result of a nonlinear analysis. Nevertheless, a balance between OSR ratio and the limit to force redistribution should be sought, in order not to produce unconservative designs.

In the revised OPCM 3431 of 2005 and in NTC 2008, the limits to force redistribution have been relaxed, stating that the variation in shear in each wall should not exceed the largest value between 25% of the shear in the wall and 10% of the total interstorey shear. In the case of non rigid diaphragms the redistribution is allowed only among coplanar walls connected by ties or r.c. beams (in such case the interstorey shear is evaluated considering only the contribution of the coplanar walls). Such choice, motivated by the need to find an urgent remedy to the inconsistent overconservative results that can be obtained by using linear analysis, was based on the comparison of linear and nonlinear analyses.

It must be remarked that more rational solutions could be envisaged, such as those being explored in Morandi and Magenes, (2008), in which the possibility of overcoming the use of the OSR is attempted (e.g. free redistribution of forces compatible with members strengths followed by a deformation capacity check).

2.3 The use of nonlinear analysis and deformation limits

As mentioned above and elsewhere (Magenes, 2006, Magenes and Penna, 2009), the use of linear elastic analysis, be it static or even dynamic, has strong drawbacks when applied to masonry buildings. In many cases, a static nonlinear analysis can provide a more realistic picture of the response of the buildings than a linear dynamic one, besides avoiding the uncertainties related to the definition of the q-factor. Comments on possible nonlinear models that can be used in current practice are given in the companion paper by Magenes and Penna.

Despite the explicit possibility given by EC8 to use nonlinear static procedures, little guidance is given in EN 1998-1 to the designer for the application to new structures, as regards some important design parameters. No reference is given regarding the deformation/drift limits that should be used in the analysis, neither other directions on modelling criteria, such as the possibility to use "storey-mechanism" approaches. Such information is given however in Annex C of EN 1998-3 for existing buildings. The Italian norms NTC 2008 provide drift limits for in-plane response which are consistent with EN 1998-3 (see Table 1 for primary URM walls).

Table 1. Angular deformation limits for URM walls.

Limit state	EN 1998-3 Annex C		NTC 2008 (new buildings)	
	Shear fail.	Flex. fail.	Shear fail.	Flex. fail.
SD	0.40%	0.80%	0.40%	0.80%
NC	0.53%	1.07%	n.a.	n.a.

SD: significant damage; NC: no collapse.

Annex C of EN 1998-3 does not provide suggestions for confined or reinforced masonry, whereas NTC 2008 suggests to increase the limits of Table 1 by 50% in the case of reinforced masonry. Some of these limits should be suitably revised on the basis of the more recent available experimental information, as discussed further on in this paper.

2.4 The problem of out-of-plane seismic response of walls

The issue of out-of-plane stability of walls subjected to seismic excitation is strangely not well put in evidence in Eurocode 8, to the point that the seismic loading is not clearly defined and the engineer would have to find his own way to a safety check, resorting for instance to the seismic loading defined for non structural elements (as suggested in Tomaževič, 1999). It can be said that strict slenderness limitations, minimum thickness requirements and appropriate structural conception and detailing (rigid diaphragms and efficient floor-to-wall connection) can guarantee in most cases the prevention of out-of-plane driven failures; however, on one hand such slenderness and thickness limitations are Nationally Determined Parameters that could vary significantly from country to country, on the other hand out-of-plane stability is an issue also for "secondary" seismic elements and non-structural partitions, which may not comply with such limitations. In addition, reinforced masonry (RM) solutions have recently been proposed for single-storey buildings, such as those for commercial and industrial purposes, as they can fulfil several functions, besides the structural one. Similar structural systems, based on post-tensioned masonry, are being used in other countries (e.g. The United States), where it has been recognized that for such slender walls, the effects of out-of-plane loads, such as extreme wind loading and inertia forces from seismic excitations, are significant (Bean Popehn et al. 2007).

The experimental and theoretical research of Doherty et al., (2002), and Griffith et al., (2003) have confirmed that out-of-plane response and stability of walls under seismic excitation, when ultimate conditions are considered, is more a matter of displacement demand vs. displacement capacity rather than a strength issue.
The problem is quite complex, requiring the evaluation of:
- seismic demand on walls considering the dynamic filtering effect of building and diaphragms, and the dynamic response of wall
- strength of wall against out-of-plane forces and relevant mechanisms of resistance
- out-of-plane displacement capacity of walls

Most of the past research dedicated to wind loading has focused mainly on strength capacity, which for URM can come from three possible sources: vertical compression, apparent flexural strength in one- or two-way bending, thrust (or arching) action. Considering the simpler one-way, vertical bending condition, in an URM wall the apparent flexural strength is due to vertical tensile strength of bedjoints or bricks, whichever the lesser. The attainment of cracking, which incidentally could develop already under service loading, due for instance to eccentricity of vertical loads, does not imply necessarily collapse, and could be seen as a damage limit state. In post-cracking regime, lateral resistance is provided by the presence of vertical compression, and could be sensitive to geometric second order effects. The behaviour of the subsystem is close to elastic nonlinear, with moderate energy dissipation. An appropriate safety assessment should be based on the evaluation of the main characteristics of the lateral response, namely initial lateral stiffness of the wall, maximum force, displacement at static instability. Proposals are available for one way bending (Griffith et al. 2003), research is still under development for cyclic two-way bending (Griffith et al., 2007), in which friction and cohesion of bedjoints and tensile strength of bricks play an important role in the maximum force capacity and also on the post-cracking hysteretic behaviour.
The code approach of the Italian norms (OPCM 3431 and then NTC 2008) explicitly requires the designer to evaluate the seismic demand in the form of an equivalent static out-of-plane force proportional to the mass of the wall (and of any fixture rigidly connected) according to the expression proposed for non-structural elements (defined as per EN 1998-1, sec. 4.3.5) . This includes an approximate estimate of the filtering effect and possible resonance between the fundamental period of the building and the fundamental out-of-plane natural period of the wall. The seismic force is in turn reduced by a behaviour factor of the wall element, which for structural walls is assumed as $q_a = 3$, whereas for non-structural walls a $q_a = 2$ would be used. Such force-based approach is clearly very rough, and it is deemed not to produce consistent results, especially since it is based on the initial elastic properties (periods) and it assumes a *constant q*-factor, independent of the displacement capacity, which is a size-dependent quantity (i.e. it increases as the thickness of the wall increases, even if the slenderness of the wall remains the same).
Since the safety check is carried out in terms of strength, namely bending moment, second order effects affecting the component of the resisting moment which depends from axial load ($M_{u,N}$) could be taken into consideration by reducing the first order resisting moment $M_{u,N}$ multiplying it by a coefficient $\phi_M \leq 1.0$, which accounts for slenderness and axial load ratio. A conservative estimate of such coefficient has been evaluated by Morandi et al. (2008) for the case of simply supported URM walls in one-way vertical bending.
The out-of-plane safety check, in particular when the influence of second order effects is non negligible, can be even more limiting in the case of reinforced masonry. The Italian norms, which only introduce conservative slenderness limits, do not provide any method to take into account second order effects for RM. The European norms also fix a maximum slenderness of 15 for “primary” seismic elements. The safety check according to EC6, for elements having

slenderness higher than 12, proposes to take into account second order effects by an additional moment accounting for slenderness and axial loads, but to calculate walls as if they were unreinforced (EN 1996-1, 2005). Such method is overconservative and can be problematic in the case of single-storey industrial/commercial buildings, with tall RM walls. These buildings are very often built with deformable roofs, where the walls can be considered as cantilevers, which is a case usually not taken into account by the codes.
In general, it is felt that considerable further research is still needed before an appropriate understanding of the problem and reliable assessment procedures will be achieved.

2.5 The design of "simple" masonry buildings

Considering the results of nonlinear analyses as a more reliable reference than linear analysis, the Italian norms provide criteria for the design of "simple masonry buildings" which differ from what suggested by EN 1998-1 in terms of minimum values of total cross sectional area of walls in each direction. The main differences can be summarized in the fact that in the Italian code there is the possibility to build simple masonry buildings for higher levels of seismic hazard than in EN 1998-1, but at the same time for lower seismic hazard the minimum required cross sectional area in the Italian code (3.5-4.0% for URM buildings, 3.0 % for reinforced masonry buildings) is higher than in EN 1998-1 (2.0% for URM and reinforced masonry).
In addition, despite some construction rules for "simple" masonry buildings in EN 1998-1 are more restrictive than in NTC 2008, the European norm does not require any basic safety verification, not even the effortless verification of mean storey compressive stress required by the Italian norms.

3 DEVELOPMENTS IN RECENT AND ONGOING RESEARCH

In recent years the experimental research on seismic behaviour/design of new masonry constructions has resumed throughout Europe and in Italy, mainly through coordinated research projects and industry-sponsored experimentation.
Such projects were considering also recent masonry products for which limited testing had been available beforehand, including thin joint construction, solution with partially filled headjoints (e.g. mortar pocket), unfilled headjoints (e.g. tongue and groove blocks), and considering different materials (clay, AAC, calcium silicate, lightweight aggregate concrete). Some meaningful results are herein summarized

3.1 Deformation limits of unreinforced masonry walls, design parameters

The EC funded project ESECMaSE (2005-2007) allowed to carry out numerous tests on large (storey-high) masonry walls subjected to in-plane cyclic shear. In such tests (Magenes et al. 2088a), new information could be collected on the different deformation/displacement capacity of masonry walls, considering different geometries of the specimens, vertical compression level, boundary conditions, material and structural details. It was confirmed that the deformation capacity strongly depends on the type of failure mechanism (shear- or flexure-dominated). The lowest deformation capacities, in terms of horizontal drift $\theta = \delta/H$ (horizontal deflection/ height of the specimen) were found in correspondence of diagonal cracking failures, involving cracking of the units (Figure 1).

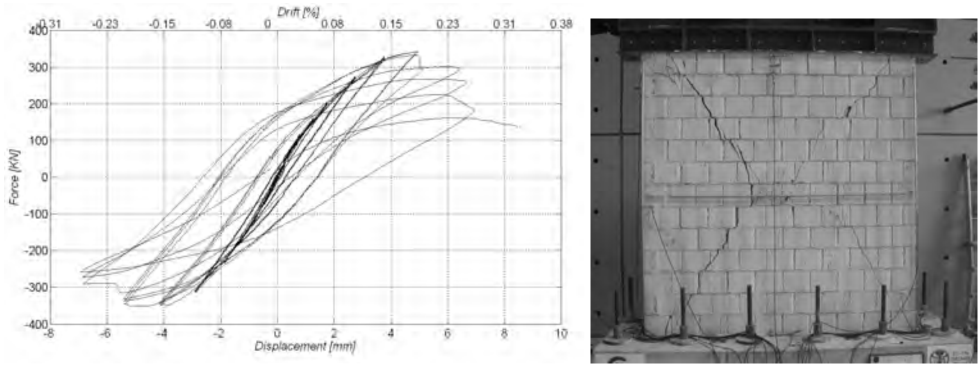

Figure 1. Shear test on a clay brick masonry wall. Left: shear force vs. horizontal displacement, right: crack pattern after the test (Magenes et al., 2008a and 2008c).

Table 2. Proposed *q*-factors for URM masonry buildings (Frumento et al., 2009), clay brick masonry.

Proposed values	F	MP	TG
q	3.00	3.00	2.50
*q**	1.75	1.75	1.50
OSR	1.70	1.70	1.70

F = fully mortared masonry, MP = mortar pocket masonry, TG = tongue and groove masonry or masonry with unfilled headjoints.

By defining the ultimate drift θ_u as the value beyond which the resisting shear measured on the cyclic envelope degrades below 80% of the maximum shear, a large variation in the data was found, ranging from ultimate drifts as low as 0.2-0.25% (diagonal cracking) to values exceeding 1.5-2.0 % (for flexural failures) were found. It must be remarked that the lowest values, found for different types of masonry (perforated clay units and calcium silicate units) are well below the reference of 0.4% (see Table 1) that was considered on the basis of the previous experimental research, mostly focused on solid brick masonry. Such results were also found when analyzing further data coming from recent tests carried out in other European labs, and a systematic review of recent clay brick masonry data (Frumento et al., 2009) has confirmed the lower bounds of ultimate drift found in the ESECMaSE project.

It was therefore felt necessary to reconsider, on the basis of these new experimental references, whether the *q*-factors recommended for design should be revised. The numerical work carried out by Frumento et al. (2009), which consisted of parametric nonlinear static analyses based on experimentally measured stiffness, strength and deformation properties of modern URM clay brick/block typologies, has led to the conclusion that conservative values of *q*-factors for URM could be defined as given in Table 2, if no differentiation is made on number of storeys.

As it can be seen, the proposed q^* coefficient gets closer to the lower bound of 1.5, giving however a global q of 2.5-3.0. The range of q^* values given in Table 2 is also in agreement with the values listed by da Porto (2005) and da Porto et al (2009a). In this case, the load reduction factor due to non-linear behaviour of mortar pocket, tongue and groove, and thin joint masonry was evaluated by means of nonlinear dynamic analyses based on experimental results. The lowest values in the range of q^* values were obtained for the latter masonry type.

In the data collection presented by Frumento et al., it is possible to recognize that one of the factors that appear to affect the experimental deformation capacity is the size of the specimens, with the lowest drift capacities obtained on storey-high walls. This fact suggests that a minimum height and length of the specimens must be used in experimental campaigns aiming to characterize the in-plane seismic behaviour of walls. Taking into account that different tests configurations, still adopted to evaluate the in-plane shear behaviour of masonry walls, show failure modes that may significantly differ from those observed on real walls (da Porto et al. 2009b), standardization of test procedures is clearly needed.
Also, from the comparison of the recent tests, the deformation capacities can vary significantly depending on the materials (clay vs. lightweight concrete vs. AAC….) and, within a given material, on the type of blocks and joints (e.g. fully mortared head joints or dry headjoints…). The latter finding has been also confirmed by extensive non-linear numerical modelling of clay unit masonry made with different type of joints and with various unit strength, again based on experimental results (da Porto et al., 2009b).
This fact may suggest that a larger differentiation of some reference parameters for design (e.g. *q*-factors, deformation limits for nonlinear analysis) for different materials/technologies could be used in design codes, where currently the only differentiation is among unreinforced, confined and reinforced masonry. Also, some basic requirements, such as the minimum strength of masonry units, may be revised and, on the basis of the latest findings, differentiated according to the different material properties.

3.2 Reinforced masonry

The EC funded project DISWall (2006-2008) focused on new solutions for reinforced masonry walls. In the context of the project, tests to characterize the in-plane cyclic shear behaviour of RM walls and the cyclic out-of-plane behaviour of tall load-bearing reinforced masonry walls in large-displacement regime were carried out (Mosele, 2009).
The in-plane cyclic tests were carried out on specimens characterized by two aspect ratios, with different types of reinforcement and under different vertical compression levels. Such tests allowed to collect information on in-plane flexural and shear behaviour of RM walls, and on the influence of the above parameters on strength and displacement capacity, energy dissipation and stiffness degradation. In the case of RM walls, the ultimate drift θ_u ranged from a minimum value of 0.7% for shear failure to values exceeding 1.7% for flexural failures (da Porto et al. 2009c). These values satisfy the limits associated to ULS for shear (0.6%) and flexural (1.2%) failures of RM walls, adopted by the Italian norms, but the European norms do not provide any drift limit for in-plane response of RM walls.
The experimental values of shear strength were compared with those provided by the European and the Italian norms, which adopt an additive approach, where the contribution of horizontal reinforcement is added to the shear strength of unreinforced masonry. The main difference is that the maximum tensile capacity of shear reinforcement is multiplied times a reduction factor of 0.6 in NTC 2008 and 0.9 in EN 1996-1-1 (2006). The first value, which was proposed by Tomaževič (1999) and Magenes (1998), and reflects experimental values of shear reinforcement effectiveness (Tomaževič, 1999; da Porto et al. 2009c), yields strength evaluations which appear to be more realistic (Mosele et al. 2009a).
According to the experimental results, a numerical research to evaluate reduction of elastic response of RM walls due to their hysteretic behaviour was carried out. In this case, the results of the non-linear dynamic analyses mainly confirmed the q^* values of 2.5 and 3.0 that the Italian norm suggests respectively to RM failing in shear and in flexure, the latter being associated to the application of capacity design principles (da Porto et al. 2008). It should be

pointed out that the same range of values, regardless of the failure mode, is also given by the EN 1998-1 (2005), but as final values of q-factors to be adopted (i.e. neglecting overstrength).

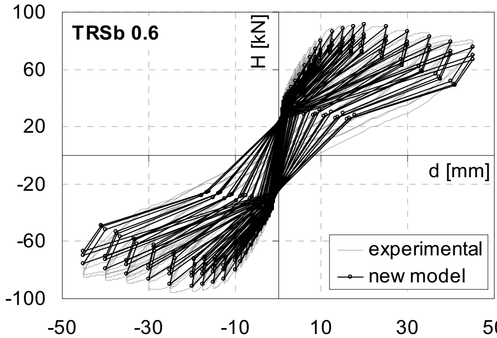

Figure 2. Shear test on a clay brick reinforced masonry wall. Left: test set-up; right: comparison between experimental and numerical hysteresis loops (Mosele 2009; da Porto et al. 2008).

The out-of-plane cyclic tests were carried out on two reinforced masonry frames, with different vertical reinforcement percentage (0.08% and 0.13%). Each frame was made of two cantilever walls, 6 m high, 2 m long and 0,38 m thick (Figure 3). Horizontal displacements and roof dead loads were applied at the top of each specimen. In such tests, new information could be collected on the out-of-plane behaviour of tall load-bearing RM walls in large-displacement regime, under the influence of vertical loads (P-Δ effects) (Mosele, 2009). The ductility of under-reinforced RM sections in out-of-plane flexure could not be exploited, as the influence of P-Δ effects dominated the behaviour as soon as the reinforcements started yielding. On the other hand, the tests showed the positive influence of higher vertical reinforcement percentage (0.13%), close to balanced failure for the masonry section. In this case, the top displacements that activated the influence of P-Δ effects, in terms of achievement of 10% stability ratio (generally adopted for reinforced concrete elements, EN 1992-1-1, 2004), were of 100 mm (1.7% of wall height). The maximum lateral load capacity, corresponding to top displacements of 5.2% of wall height, was almost twice that at 10% stability ratio, and the maximum top displacement corresponded to deflection of 6.6% of wall height (Mosele et al. 2008).

The slenderness limit of 15, fixed by the European and the Italian seismic norms assuming simple support boundary conditions, is quite severe when compared to these experimental results. This was further demonstrated by numerical analyses, which took into account both geometrical and material non-linearity and studied the influence of axial load level, wall slenderness and percentage of vertical reinforcement on the out-of-plane response of the RM walls. The models also showed that the minimum percentage of vertical reinforcement recommended to avoid failure dominated by second-order effects is 0.08%, in agreement with that given by the EN 1998-1 (2004), although the lower percentage given by the Italian norms, which is 0.05%, has been proven to be adequate for in-plane walls (Magenes, 1998; Mosele et al, 2009b). It is also significant that excessively high reinforcement percentage, in out-of-plane as well as in-plane walls, can be useless, and even harmful, as they bring the masonry section towards brittle failure modes. Nevertheless, the European norms do not provide any indication on maximum reinforcement percentage.

Figure 3. Cyclic out-of-plane tests on reinforced masonry walls. Left: view of the test set-up; right: final deflection for the walls with 0.13% vertical reinforcement (Mosele et al. 2008; Mosele 2009).

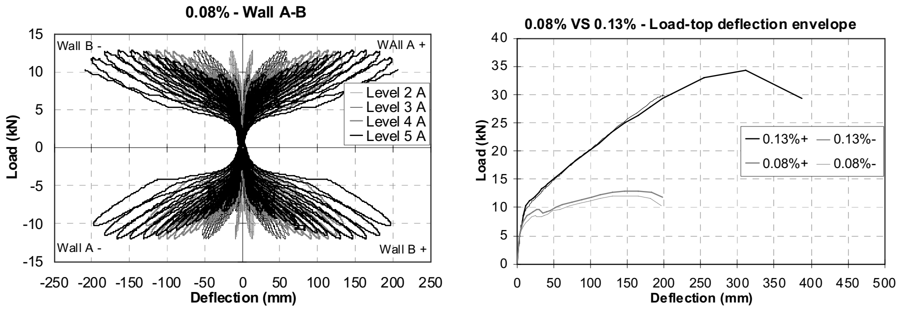

Figure 4. Cyclic out-of-plane tests on reinforced masonry walls. Left: cyclic load-deflection diagrams for the walls with 0.08% vertical reinforcement; right: load-top deflection envelopes (Mosele 2009).

The application of analytical models, usually adopted to take into account second order effects in slender reinforced concrete columns (EN 1992-1-1, 2004), gave promising results also when applied to RM walls (Mosele, 2009). The use of simplified moment magnifier methods to account for P-Δ effects in RM is thus consistent (Drysdale et al., 2008), and may be adopted by the norms to overcome some of the current limitations.

4 CONCLUSIONS

On the basis of the issues discussed above, it seems that some changes should be proposed for the improvement of chapter 9 of Eurocode 8 and also to parts of Eurocode 6.
In particular the recent research on the seismic behaviour of unreinforced and reinforced masonry buildings has led to some developments in both linear and nonlinear methods of analysis for masonry structures that still need to be implemented into Eurocode 8. These improvements mainly concern the new definition of seismic force reduction factors and more

rational approaches for force redistribution to be used in linear procedures. In addition, static non-linear analysis methods for masonry buildings have been improved, but it is felt that reference values for some basic design parameters should be provided by the norms, in order to make the methods applicable in practice by the designers.

Updated information allowing to adopt both force-based design approaches and more rational displacement- or deformation-based design approaches for masonry buildings is progressively becoming available and should be transferred to Eurocode 8.

In addition, the Eurocode 8 does not address the issue of out-of-plane stability of masonry walls subjected to seismic excitation. Some inconsistencies in Eurocode 8 and 6 are found when dealing with the problem of safety checks, not only in the case of out-of-plane loads, but also with some issues regarding in-plane strength.

The revival in several European countries of the research on seismic design of masonry buildings, of which some results were here briefly outlined, can serve a support towards the updating and improvement of design methods and construction criteria. In general, it should be kept in mind that the variety and diversity of masonry materials and construction techniques, together with the ongoing technological evolution of products calls for a continuous review and experimental verification of the structural performances and for possible further differentiations of design criteria and reference parameters for different materials/ technologies. In this context, shared and reliable procedures for evaluating code requirements of the new materials/technologies, also in the case of seismic design, should be established.

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KNOWLEDGE OF THE BUILDING, ON SITE INVESTIGATION AND CONNECTED PROBLEMS

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ABSTRACT
Historic building, either famous monuments or "minor" architecture of historic centers need to be investigated in order to carry out repair aimed to their preservation. Non destructive techniques should be applied on site and destructive investigation limited to minor sampling. A methodology for investigation is outlined describing advantages and limits of the different techniques. Need for guidelines in Codes and Standards are also stressed.

KEYWORDS
Historic masonry buildings, diagnostic investigation, on site investigation, non destructive techniques.

1 INTRODUCTION

Historic buildings, no matter whether they are famous monuments or so called "minor" or even vernacular architectures, represent an important part of our cultural heritage. This patrimony which is the living memory of the country history and development must be preserved as much as possible as an historic document of our past. In the last decade the word "restoration" has more and more been substituted by the term "preservation". Also in the case of damages due to earthquake or other calamities the expression "to adequate" was substituted by the expression "to improve by minor repair and strengthening".
Conservation of historic buildings requires a deep knowledge of structures and materials, of their characteristics, of the eventual state of damage and its causes. Prevention and rehabilitation can be successfully accomplished only if a diagnosis of the state of damage of the building has been formulated. The diagnosis should result from an experimental investigation on site and in the laboratory; it should also be clear that the investigation on site must be non-destructive as far as possible and give information with good precision. Besides the damage investigation before the intervention, the effectiveness of the repair techniques should also be controlled during and after the repair work, as well. The investigation also may require long-term monitoring of the structure.
The on-site experimental investigation is required and recommended also by Codes of Standards in several countries. The working group WG2- Working Item 001b: Diagnosis of building structures of the EC Technical Committee TC 346, prepare Standards for on site investigation. A RILEM TC (216 SAM, Structural Assessment of Masonry) is preparing guidelines for diagnostic investigation, following the results of a European Contract named ONSITEFORMASONRY.

2 DESIGN FOR THE DIAGNOSTIC INVESTIGATION

Historic buildings belong to different typologies to which a different behaviour of the structure corresponds: (i) isolated buildings, (ii) building in a row, (iii) complex buildings, (iv) towers, (v) palaces, (vi) churches, (vii) arenas. The modelling of these structures can be very difficult. In fact, when the structure is a complex one, only linear elastic models are easily usable. Non-linear models or limit state design complex models are difficult to apply, also because the needed constitutive laws for the material are seldom available. Furthermore when the complexity of the structure is given by its evolution along the centuries, starting from a simple volume to a more and more complex volume (Figure 1), then modelling has to take into account all the vulnerabilities accumulated during the subsequent transformations

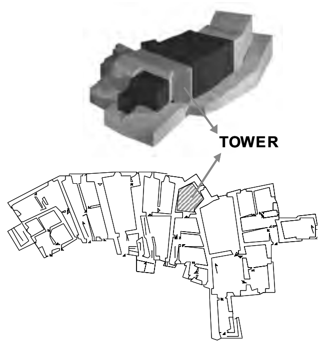


Figure 1. Complex building (aggregate) in Castelluccio di Norcia

The same difficulties can be found in choosing the techniques for repair and strengthening. Figure 2 shows which information can be available from in situ and laboratory survey and how they can constitute the input data for the structural analysis (Binda, 2007)

The structural performance of a historic masonry building can be understood provided the following aspects are known: (a) its geometry and historical evolution; (b) the characteristics of its masonry texture (single or multiple leaf walls, connection between the leaves, joints empty or filled with mortar); (c) the physical, chemical and mechanical characteristics of the components such as bricks, stones, mortar; (d) the characteristics of the masonry as a composite material; (e) the material decay; (f) the state of damage of the structure (Binda *et al.,* 2009).

Several investigation procedures have been applied in the last decades most of them coming from other research field (e.g. medicine, aerospace engineering) or from application to the study of new materials (steel, concrete, composites). Nevertheless, to apply non destructive, although advanced, techniques to masonry which is a composite, highly non homogeneous material can be frustrating due to the difficulty of interpretation of the collected data.

Most of the ND procedure can give only qualitative results; therefore the designer is asked to interpret the results and use them at least as comparative values between different parts of the same masonry structure.

It must be clear that even if there is a need of consulting experts in the field, it is the designer, or a member of the design team, who must be responsible of the diagnosis and must: (i) set up the in-situ and laboratory survey project, (ii) constantly follow the survey, (iii) understand and verify the results, (iv) make technically acceptable use of the results including their use as input data for structural analyses, (v) choose appropriate models for the structural analysis, (vi) arrive at a diagnosis at the end of the study.

In the following the methodology of investigation, as proposed by the author, also within the EU Contract "ONSITEFORMASONRY" developed between 2001 and 2004, and within GNDT and RELUIS contracts, is briefly described.

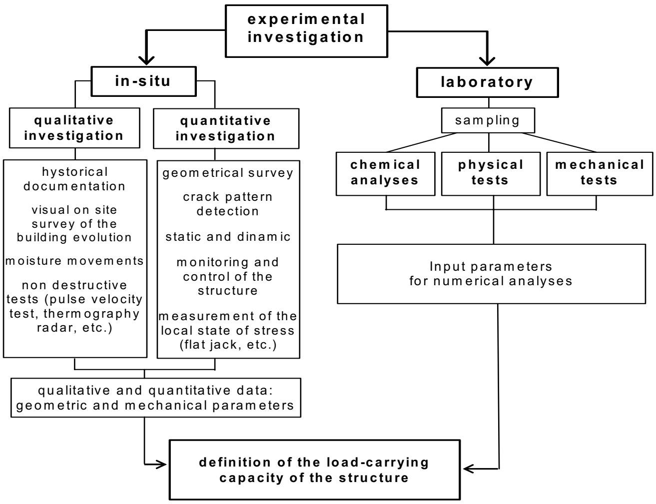
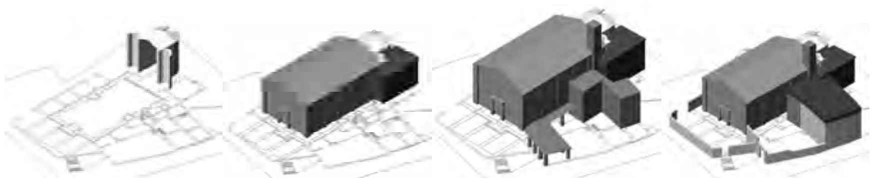


Figure 2. Finalization of the experimental survey to the structural analysis.

2.1 Historical evolution of the building

A preliminary in-situ visual survey is useful in order to provide details on the geometry of the structure and in order to identify the points where more accurate observations have to be concentrated. In the meantime the historical evolution of the structure has to be known (Cardani *et al.,* 2008a) in order to explain the signs of damage detected on the building (Figure 3).



High Middle Age 15th – 16th century Napoleonic period Early 20th century

Figure 3. Construction phases of a church (S. Michele Arcangelo at Sabbio Chiese, Italy).

2.2 Geometrical survey

The geometrical survey can be carried out with simple tools or more sophisticated ones as photogrammetry or laser scanners surveys.
The geometry of the structures has to be known in details, since it is the base both for the project of intervention and for the mathematical modelling.

2.3 Crack pattern survey

Especially important is the survey and drawing of the crack patterns (Figures 4 and 5). The interpretation of the crack pattern can be of great help in understanding the state of damage of the structure, its possible causes and the type of survey to be performed (Binda *et al.,* 2007).

The crack pattern must be reported on prospects, and plans and even in 3D in order to better interpret it.

Figure 4. Church of SS. Benedetto, Pompegnino (BS).

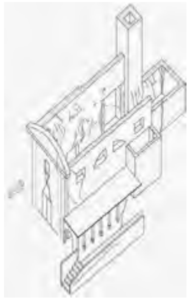


Figure 5. Church of St. A. Abate, Morgnaga (BS).

2.4 Masonry morphology survey

The structural performance of a masonry wall can be understood provided the following factors are known: (i) the geometry; (ii) the characteristics of its masonry texture (single or multiple leaf walls, connection between the leaves, joints empty or filled with mortar), (iii) physical, chemical and mechanical characteristics of the components (bricks, stones, mortar); (iv) the characteristics of masonry as a composite material.

A direct inspection can be performed by removing few bricks or stones, surveying photographically and drawing the section of the wall. This can be more efficient than coring (See also Sec.4.1.1)

2.5 Laboratory test for material characterization

The aims of these tests are the followings: (i) to characterize the material from a chemical, physical and mechanical point of view, (ii) to detect its origin, (iii) to know its composition and content in order to use compatible materials for the repair, and (iv) to measure its decay and durability to aggressive agents.

2.5.1 Tests on damaged and new mortars

Physical, chemical and mineralogical-petrographic analyses are useful (and less expensive than other more sophisticated tests) to determine: the type of binder and of aggregate, the binder/aggregate ratio, the extent of carbonation, the presence of chemical reaction, which produced new formations (pozzolanic reactions, binder-aggregate reactions, alkali-aggregate reactions) (Baronio *et al.*, 1991).

The grain size distribution of the aggregates can also be measured, particularly in the case of siliceous aggregates, by separating the binder from the aggregates through chemical or thermic treatments

2.5.2 Tests on damaged and new bricks and stones

When masonry is damaged by aggressive agents, chemical, physical and mechanical laboratory tests can give useful information for the choice of the appropriate material for substitution. The tests have to be carried out on deteriorated and on undamaged existing bricks and stones, and new ones.

2.6 On site investigation techniques

By definition the on site investigation should be carried out by using non destructive or slightly destructive techniques (Binda *et al.*, 2000), (Binda *et al.,* 2009).

2.6.1 Minor destructive tests

Several minor destructive tests are non used as the ponder drilling test, the penetration test and the Schmidt Hammer test for mortars.

Nevertheless the flat jack tests are the only ones which give valuable mechanical results up to now.

Flat jack test. The method was originally applied to determine the in-situ stress level under compression of the masonry. The firsts applications of this technique on some historical monuments, clearly showed its great potential.

The determination of the state of stress is based on the stress relaxation caused by a cut perpendicular to the wall surface; the stress release is determined by a partial closing of the cutting, i.e. the distance after the cutting is lower than before (Binda *et al.,* 1999). A thin flat-jack is placed inside the cut and the oil pressure into the jack is gradually increased to obtain the distance measured before the cut (Figure 6). The displacement caused by the slot and the ones subsequently induced by the flat-jack are measured by a removable extensometer before, after the slot and during the tests. P_f corresponds to the pressure of the hydraulic system driving the displacement equal to those read before the slot is executed. The equilibrium relationship is the fundamental requirement for all the applications where the flat-jack are currently used : $S_f = K_j\ K_a\ P_f$ when: S_f =calculated stress value, K_j =jack const (<1), K_a =slot/jack area const (<1).

The test described can also be used to determine the deformability characteristics of a masonry. A second cut is made, parallel to the first one and a second jack is inserted, at a distance of about 40 to 50 cm from the other. The two jacks delimit a masonry sample of appreciable size to which a uni-axial compression stress can be applied. Measurement bases for removable strain-gauge or LVDTs on the sample face provide information on vertical and lateral displacements. In this way a compression test is carried out on an undisturbed sample of large area (Figure 7).

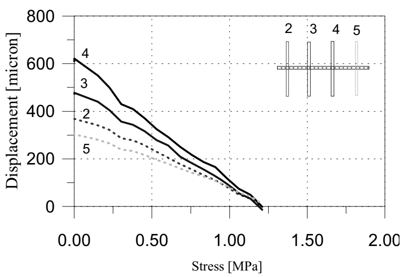


Figure 6. Single flat-jack tests carried out at the Monza Tower.

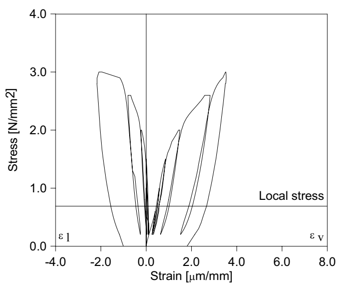


Figure 7. Double flat-jack test on West side of the Monza Tower.

Proposals for Standards on flat-jack tests have been included in the Re-luis products in 2003.

2.6.2 Non destructive tests

Many authors have mentioned the importance of evaluating existing masonry buildings by non-destructive investigation carried out in situ. ND techniques can be used for several purposes: (i) detection of hidden structural elements, like floor structures, arches, pillars, etc., (ii) qualification of masonry and of masonry materials, mapping of non homogeneity of the materials used in the walls (e.g. use of different bricks in the history of the building), (iii) evaluation of the extent of mechanical damage in cracked structures, (iv) detection of the presence of voids and flaws, (v) evaluation of moisture content and capillary rise, (vi) detection of surface decay, and (vii) evaluation of mortar and brick or stone mechanical and physical properties (Binda *et al*., 2009).

2.6.2.1 Thermovision

The thermographic analysis is based on the thermal conductivity of a material and may be *passive* or *active.* The *passive* application analyses the radiation of a surface during thermal cycles due to natural phenomena (insulation and subsequent cooling). If the survey is *active*, forced heating to the surfaces analyzed are applied.

A camera sensitive to infrared radiation collects the thermal radiation from the materials. The result is a thermographic image in a colored scale. At each tone corresponds a temperature range. Usually the differences of temperatures are fraction of degree. Applications can be to: (i) survey of cavities, (ii) detection of inclusions of different materials (Figure 8), (iii) detection of water and heating systems, (iv) moisture presence. In the diagnosis of old masonries, thermovision allows the analysis of the most superficial layers (Binda *et al.*, 2003a).

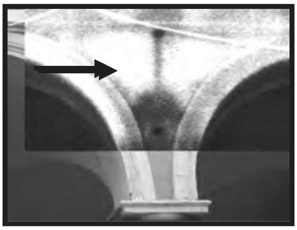


Figure 8. Investigation on hidden steel tie rods

2.6.2.2 Sonic pulse velocity test

The testing methodology is based on the generation of sonic or ultrasonic impulses at a point of the structure. An elastic wave is generated by a percussion or by an electrodynamics or pneumatic device (transmitter) and collected through a receiver, usually an accelerometer, which can be placed in various positions (Binda *et al.*, 2007b). The elaboration of the data consists generally in measuring the time the impulse takes to cover the distance between the transmitter and the receiver. The use of sonic tests for the evaluation of masonry structures has the following aims: (i) to qualify masonry through the morphology of the wall section; (ii) to detect the presence of voids and flaws and to find crack and damage patterns; (iii) to control the effectiveness of repair by injection technique in others which can change the physical characteristics of materials. The limitation given by ultrasonic tests in the case of very inhomogeneous material made the sonic pulse velocity tests more appealing for masonry. In general it is preferable to use sonic pulse with an input of 3.5. kHz for inhomogeneous masonry. Figure 9 shows the application of sonic tests to the detection of density in a stone walls.

2.6.2.3 Radar

Among the techniques and procedures of investigation which have been proposed in these last years, georadar seems from one hand to be most promising, from the other to need a great deal more of study and research (Binda *et al.* 1998) (Binda *et al.*, 2008).

The method is based on the propagation of short electromagnetic impulses, which are transmitted into the building material using a dipole antenna. When the transmitting and receiving antennas, which are often contained in the same housing, are moved along the surface of the object under investigation, radargrams (colour or grey scale intensity charts giving the position of the antenna against the travel time) are produced . The choice of the antenna frequency must be made on a site basis. It is important to show results, as radargrams and graphics, which are significant to operators like architects and engineers.

When used for masonry, the applications of radar procedures can be the following: (i) to locate the position of large voids, cracks (Figure 10) and inclusions of different materials, like steel, wood, etc; (ii) to qualify the state of conservation or damage of the walls; (iii) to define the presence and the level of moisture; (iv) to detect the morphology of the wall section in multiple leaf stone and brick masonry structures.

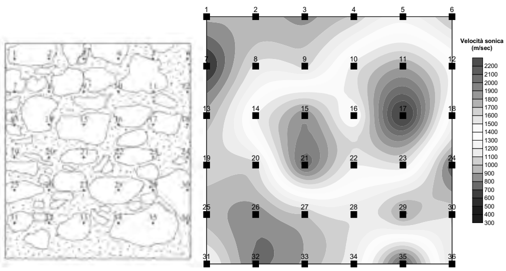


Figure 9. Distribution of sonic velocities in a wall

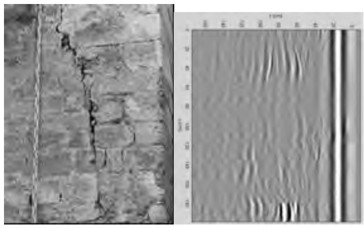


Figure 10. Localisation of cracks by radar

2.6.2.4 Radar and sonic tomography

Among the ND applications the tomographic technique is quite attractive for the high resolution that can be obtained. Tomography, developed in medicine and in several other fields, seems to be a valuable tool to give two or and three-dimensional representation of the physical characteristics of a solid. Tomography, from Greek "tomos" (slice), reproduces the internal structure of an object from measurements collected on its external surface (Binda e*t al.,* 2003b).

2.7 Static and dynamic monitoring

Where an important crack pattern is detected and its progressive growth is suspected due to soil settlements, temperature variations or to excessive loads, the measure of displacements in the structure as function of time has to be collected.

Very simple monitoring systems can be applied to some of the most important cracks in masonry walls, were the opening of the cracks along the time can be measured by removable extensometers with high resolution. This simple system can give very important information to the designer on the evolution of the damage.

In-situ testing using dynamic methods can be considered a reliable non-destructive procedure to verify the structural behavior and integrity of a building. The principal objective of the dynamic tests is to control the behavior of the structure to vibration. The first test carried out can be seen also as the starting one of a periodical survey using vibration monitoring inside a

global preventive maintenance programme. Acceptance of vibration monitoring as an effective technique of diagnosis has been supported by different studies (Niederwanger, 1997). These tests are very important to detect eventual anomalies in the diagnosis phase and to calibrate efficient analytic models (FEM).
The environmental excitation sources could be the wind, the traffic or the bell ringing in the particular case of towers. The forced vibrations could be produced by local hammering systems or by the use of vibrodines (Gentile *et al.*, 2002).

3 DIFFERENT LEVELS OF INVESTIGATION

The importance of carrying out diagnostic investigations at different levels was clear after detecting the damages to the C.H. patrimony and to ordinary old buildings since the earthquake occurred in Friuli (1976). It was first of all observed that the damages caused by in plane and out of plane actions can occur according to typical mechanisms of failure recurrent for the same building typologies (churches, palaces, etc.). The experience of investigation in fact allowed to prepare special forms for the survey of damages by teams of expert referring to known failure mechanisms (Servizio Sismico Nazionale, 1998 and Civil Protection Department templates for C.H., 2006). It was so possible to extend the investigation to the whole historic centre instead of to single buildings. The study of the effect of the earthquake that struck the Umbria and Marche regions in 1997, showed how most retrofitting, carried out after the 1979 earthquake, mainly performed with upgrading interventions (substitutions of timber floors and roofs with r.c., jacketing of walls, etc.), provoked unforeseen and serious out-of-plane effects (large collapses, local expulsions), due to the “hybrid” behaviour activated from the new and the old structures (Binda *et al.*, 2003a).
At the level of the single building (usually a listed monument), the on site investigation may be more complex due to the necessity of collecting the most possible number of data in order to help the designer preparing the project for the conservation of the building. More technologically advanced NDT can be applied to the details of the building so as more advanced techniques can be used for the geometrical survey (Binda *et al*., 2009).

3.1 Investigation at urban level

A research was carried out in Italy within the frame of a GNDT (National Group for Defence from Earthquakes) contract, involving Universities and Cultural Property Regional Offices. The main aim of the research was to set up systematic Data-bases for historic centres, able to store information useful for defining the seismic vulnerability of the buildings; preparing rescue plans; and, designing interventions for the preservation of the cultural heritage (GNDT contracts and Reluis contracts). The collected information deals with: i) the technological and constructive characteristics of the surveyed buildings; ii) the material and structure properties (with particular reference to the constructive techniques and to materials used for load-bearing masonry); iii) the materials and the techniques used for restoration before the earthquake; iv) the collapse mechanisms of buildings and structures due to the earthquake, considering also the ones already retrofitted (Binda *et al.*, 2003a).
The object of the above mentioned research was not the single building, but the whole historic centre (even if small). The strategic aim was to define a methodology for the vulnerability analysis of built heritage, wrongly considered as “minor” in the past, that holds a meaningful testimony of cultural value. The research work aimed to produce a methodology of investigation which could be applied in the future by the authorities, at municipality, province or regional levels, to support the designers in choosing the right analytical model for the

safety definition and the appropriate intervention techniques for their projects, (Binda *et al.*, 2007c).

3.1.1 A simple methodology for the quality survey of the masonry walls

The authors experience acquired on several types of Italian and European masonry structures, suggests that coupling flat-jack test with sonic test and with the observation of the masonry section by sampling, is a good and not too much invasive methodology to mechanically qualify the masonry (Binda *et al.,* 2007a). This methodology was proposed with a document as a product of Re-Luis (line 1 and 10). A systematic investigation campaign was carried out on a sample of ten churches with the aim of supplying a good level of knowledge necessary to the structural analysis and to the proper strengthening and repair interventions (Cardani *et al.,* 2008).

According to the low budget allocated by the single churches, only few points could be chosen to detect the masonry quality, i.e. its morphology and stress-strain behaviour. Systematically, the testing points were chosen in the most representative parts of the bearing walls: taking into account that the façade of a church is usually made of better masonry, the lateral bearing walls were chosen. The tests were carried out mainly on the outer face of the wall, since the inner one was usually decorated with frescos and paintings which could not be damaged.

A complete characterization of the masonry in the chosen points was achieved by measuring: the sonic velocity, the state of stress, the modulus of elasticity, the coefficient of lateral deformation, the mortar and stones chemical, physical and mechanical properties. The morphology of the wall cross section was also surveyed in order to understand whether the masonry was made of one or two leaf and the leaves were connected in some way. This survey was carried out by sampling few stones in order to visually investigate the wall texture, redraw the inner aspect of the wall, sample stones and mortars for laboratory testing.

In particular, four subsequent steps were followed in the same area: (i) sonic tests by transparency on a grid of 75x75cm, (ii) single and double flat-jack test, (iii) survey of the masonry morphology and material sampling, (iv) repositioning of the stones in the wall (Figures 11, 12).

Figure 13 shows the relationship between the sonic velocity and the modulus of elasticity, compared to other values previously obtained from tests on different stone-masonry walls in historic centres.

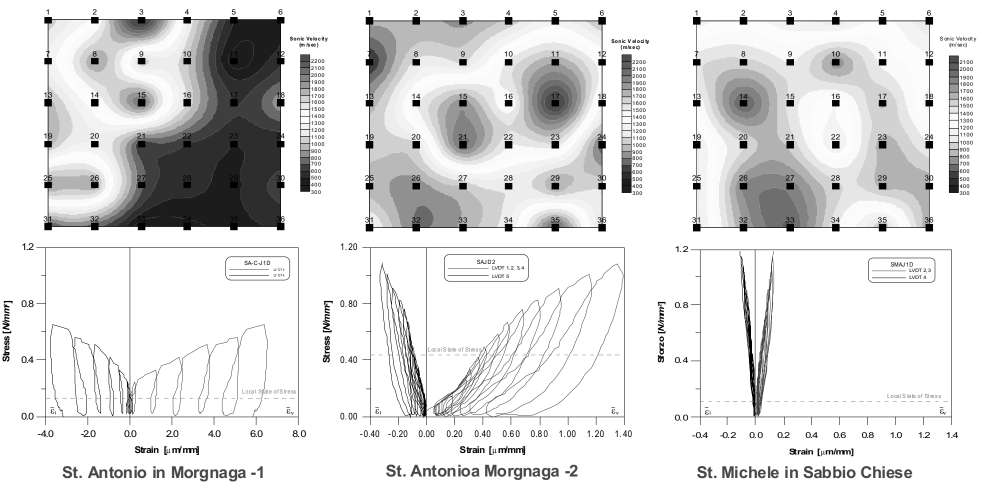


Figure 11. Results of sonic and flat-jack tests on the walls of three buildings.

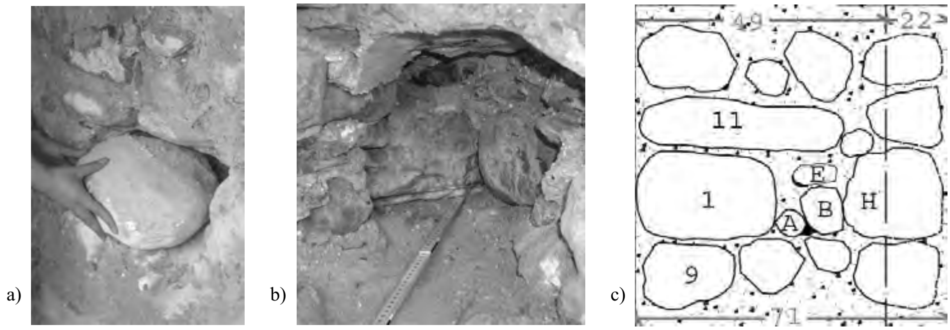


Figure 12. Survey of the wall section c) by removal of few stones (a,b).

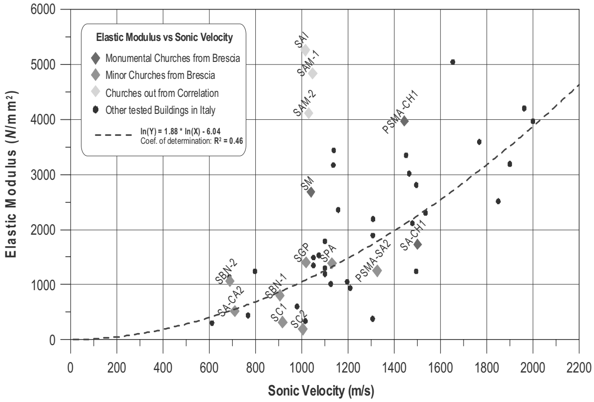


Figure 13. E measured by double flat-jack test vs. sonic velocity.

3.2 Investigation of single buildings: complementary use of tests

When a complex investigation is carried out using different techniques, the highest difficulty is represented not only by the interpretation of the results of the single technique but also by the harmonisation of all the collected data (Binda *et al.,* 2003c) .

To this purpose the development of new more appropriate software for the elaboration, interpretation and fusion of data particularly from NDT is needed (sonic, radar, flat-jack tests, static and dynamic monitoring). The production of guidelines for the correct application of investigation techniques to the different classes of masonries and of masonry structures, is also important.

The solution of very difficult problems as the detection of the morphology of multiple leaf masonry sections, the presence of voids and cracks in masonries, their mechanical characteristics, cannot be reached with a single investigation technique, but with the complementary use of different techniques (see also sec. 4.1.1).

Some typical problems were solved by the Authors with the combination of different techniques such as radar and sonic tests, flat-jack and sonic tests, sonic and radar tomography thermography, sonic ,ultrasonic and radar tests, together with static monitoring. A good application and harmonisation of the results is presented in (Binda *et al,*. 2007d) for the case of the Syracuse Cathedral.

4 CONCLUSIONS

A methodology for investigation on historic structures aimed to their preservation was outlined. Knowledge of the building details, materials and structural elements is essential in order to avoid past mistakes.
NDTs and MDTs are efficient only if their application is carefully calibrated on the studied building. Nevertheless the interpretation of the results is a difficult task and should be accomplished in a multidisciplinary approach.
Further research is needed on the complementarity of the techniques and on the development of appropriate software in order to obtain clear interpretations.
In absence of an immediate risk, the investigation can be: (i) prolonged in time and comprehensive, (ii) carried out to *calibrate* eventual *mechanical models* of the building behavior for long term actions or particular single events (hurricanes, earthquakes, etc.), (iii) set up to *control the effectiveness of the intervention* by monitoring of the parts, which were previously more at risk. Investigation is also needed in case of *long term maintenance programs* for repaired buildings.
Due to the possible high cost of MDTs and NDTs, an accurate choice has to be done by the designer, especially when diagnosis and search for vulnerability has to be applied at the level of to historic centres.
All the difficulties which have been described in the paper suggest that appropriate guidelines should be prepared within the National and International Codes and Standards (as in case of EC8 for seismic areas).

5 AKNOWLEDGEMENTS

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STRUCTURAL INTERVENTIONS ON HISTORICAL MASONRY BUILDINGS: REVIEW OF EUROCODE 8 PROVISIONS IN THE LIGHT OF THE ITALIAN EXPERIENCE

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ABSTRACT
The paper presents a critical review of Annex C, section 5, of the Eurocode 8 Part 3, which focuses on the structural interventions for existing masonry buildings. The considerations herein drawn up are developed in the light of the Italian experience after recent earthquakes, and refer particularly to the Codes and Guidelines currently in force, which are also based on the results of extensive experimental researches carried out in Italy and Europe.
The limits and the deficiencies of Eurocode 8 Part 3 (Annex C.5) are pointed out following the logical structure of the Italian norms, that are organized by defining the objectives and the performance requirements to be achieved through the strengthening interventions. Suggestions for reviews and improvements are given, considering the significance of this document and the need for general and shared rules to be applied, in the field of seismic strengthening of masonry buildings, at an European level.

KEYWORDS
Eurocode 8, structural interventions, masonry buildings, architectural heritage, restoration.

1 INTRODUCTION

Safeguard of historical buildings from the seismic risk is a difficult task regarding first the prevention, and then the whole process from building assessment through design and execution of interventions. It is essential that targeted methodologies, which allow applying the general concepts of seismic engineering to the particular case of historic buildings and are being recently developed, are applied at a larger scale and become available and viable for designers.
As an example, the adoption, for historical masonry buildings, of the same classes of predictive models developed for new constructions can mislead about the real behaviour of the structures (Binda et al., 2005; Magenes, 2006), and can bring to the choice of useless or even harmful interventions for their seismic protection (Modena et al., 2009). The most recent seismic events (Lunigiana and Garfagnana, 1995; Reggio Emilia, 1996; Umbria and Marche, 1997; Piedmont, 2000; Molise, 2002; Piedmont, 2003; Salò 2004, Abruzzo 2009) confirmed limits and consequences of some type of interventions, concurrently corroborated also by extensive experimental researches, but also the effectiveness of new methods based on the use of both traditional and innovative materials and techniques.

From observation and study of structural faults of original and repaired structures, not only information on the effectiveness of various strengthening interventions could be drawn, but it could be also highlighted that historical buildings cannot be assessed through “standard” methods. On one hand, these methods are far from understanding the real seismic behaviour of constructions and, on the other, they can lead to invasive interventions which modify permanently the cultural value and the structural behaviour of the buildings, in conflict with necessary preservation requirements. It has been also clarified that the knowledge of the typical features of each historical building, regarding, for example, the constitutive materials (i.e. masonry typology and arrangement), the structural type (common building, isolated/in aggregate, churches, towers, palaces), etc., is essential for the definition of suitable interpretative models. In addition, the preliminary diagnosis of the building, regarding history, geometry, materials, connections, etc., should constitute the basis for all safety evaluation and intervention planning (Giuffrè, 1991; Giuffrè, 1993; Doglioni et al, 1994; Binda et al, 1999). Hence, the assessment process should be considered a multidisciplinary task, taking into account also qualitative evaluations and involving different specialists to take a joined decision, with the structural engineer, about the safety level to confer to the historical construction and the type of intervention to be undertaken, as recently reasserted also by the draft version of Annex I (Heritage Structures) to ISO 13822 (2006).
In the next section, some aspects of how the topic of structural interventions on existing masonry buildings is treated by the recent Italian codes (OPCM 3431, 2005; NTC 2008) are discussed. Subsequently, some recent developments and results from researches and post-earthquake survey observations are reported, with reference to the different approach adopted in the corresponding European norm (EN 1998-3, 2005).

2 PRINCIPLES OF ITALIAN NORMS AND GUIDELINES

Since the D.M. 24/01/1986 was adopted, the concept of “seismic improvement” was introduced in Italy. As a consequence, in the case of minor interventions that do not significantly alter the overall structural behaviour, it is not necessary to undertake the “seismic upgrading”, i.e. the increase of seismic performances to the level required for new constructions. At the same time, in the Italian norms it was possible to avoid the safety checks required by standards prescriptions. Reasserted in the D.M. 16/01/1996, this concept was then applied to the assessment of cultural heritage buildings, since it was considered compatible with their preservation needs. These norms also listed some strengthening techniques to be adopted for improvement and upgrading interventions, which reflected the knowledge and state of art of the years when these were issued. From that moment onwards, the consequences of various earthquakes, listed in the introductions, led to a critical review of the technical content of these documents. The calibration of safety level to the need of an existing structure is also proposed by international standards (ISO 13822, 2006), and can be based on the concepts of minimum total expected cost, comparison with other social risks, importance of the structure, possible failure consequences and socio-economical criteria, although its influence on the type of intervention to be adopted is not clearly defined in other norms.
These efforts, through the OPCM 3274 (2003) and OPCM 3431 (2005), have led to the documents currently in force: the NTC 2008 (D.M. 14/01/2008) and the Guidelines issued by the Ministry of Cultural Heritage. In this context, a significant step ahead was moved first with the drawing up of the OPCM 3431 (2005), which enhanced significantly the entire process by defining the knowledge levels and introducing new procedures of analysis and assessment and new criteria for the intervention on existing structures. In addition, this norm

stated again that there is no need of carrying out common safety checks for seismic improvements, but required the designer to calculate the higher safety level reached by means of these improvement. Hence, the OPCM 3431 limited some of the uncertainties prior connected to the application of seismic improvement, which did not need any demonstration of their real need and effectiveness.

A second step was moved when the Guidelines (2007) for the application of the above seismic standards to cultural heritage buildings was issued. This technical document, including and extending the principles already contained in OPCM 3431, was specifically drawn up to delineate a methodology fitted to the need and features of cultural heritage. Several features in this technical document are related to the specificity of cultural heritage buildings. The innovative aspects of these guidelines emerge from the multidisciplinary approach that they propose. The outcomes of the process of assessment and reduction of seismic risk for cultural heritage buildings is thus a compromise between seismic protection requirements and respect of cultural and artistic values, according to the preservation criteria asserted in the various issued charters for the restoration of historic monuments (Athens Charter, 1931; Venice Charter, 1964) and recommendation for structural restoration of architectural heritage (ICOMOS/ISCARSAH, 2003).

In this context, it should be pointed out that the EN 1998-3 (2005), which introduced some of the concepts that have been further developed in the Italian norms (e.g., the definition of knowledge levels, see Binda and Saisi, 2009), is currently lacking some reference values in order to make the methods applicable by the designers. In other fields, such as the definition of the seismic safety levels for existing structures (Borri and De Maria, 2009) and the methods of analysis and assessment (Magenes and Penna, 2009; Lagomarsino, 2009), the Eurocode 8 is not updated with the latest findings and methods introduced in Italy. Even the established concept of “seismic improvement”, which has a significant influence on the design of interventions, is not yet recalled by the EN 1998-3 (2005).

When going more in detail to the types of intervention, some others significant differences are found between the Italian and the European norms. First of all, the Italian norms give some general principles to select and apply the interventions, which are valid regardless the specific technique being employed. One of these criteria is that interventions should be applied as much as possible regularly and uniformly on the building, so to avoid uneven distributions of strength and stiffness. Eventual increase of these factors on limited portions of the building must be carefully evaluated. In addition, particular care must be paid to the execution phase. These simple criteria derive, obviously, by the observation of damages on buildings that had been retrofitted in recent times, but have already sustained new earthquakes. The surveys indeed demonstrated that interventions carried out without paying the due attention to the above criteria were useless and even harmful.

The improvement of the building seismic performances may be achieved using traditional methodologies but also adopting innovative techniques and materials. The choice of the most appropriate approach depends on the results of the previous evaluation phases, thus the interventions listed in the Italian norms should not be considered as prescriptive and to be applied in any case, but should be targeted to the specific problem. Conservation of both materials and functionality of the structure is the main objective, therefore interventions should avoid significant alterations to the original structure and provide compatibility to the largest extent. It should be pointed out that these simple concepts have not been included in the Eurocode 8.

Strengthening interventions are grouped following a performance-based scheme in Italian norms, while EN 1998-3 does not have a consistent outline. In the Italian code, there has been an effort to understand the behaviour and the typical faults of masonry buildings, and thus proposing classes of interventions that can improve or solve specific problems. Some classes of interventions listed by NTC 2008, that will be discussed in the following, are: intervention aimed at improving the structural connections, interventions aimed at reducing horizontal diaphragm deformability, intervention to increase masonry strength, interventions on vaults and arches, interventions on pillars, etc. At this regard, the structure of EN 1998-3 is not coherent as it sometimes follows a rational path but some other times mixes or simplifies the approaches. As an example, in annex C of Eurocode 8 a sub-section is devoted to the repair of cracks. This is quite limited, if one thinks that being cracks the symptoms of more complex structural problems, say inadequate masonry strength under in-plane shear, or activation of kinematic mechanisms on vaults and arches, or presence of sustained dead-loads on pillars, etc., the solution should be aimed at solving the basic problems. In this context, the interventions listed by the Italian norms, according to the current knowledge, cover all the main aspects of masonry building behaviour, but is not deemed to be exhaustive in terms of materials and techniques. The same norm, indeed, states that other materials and techniques, when proved to be viable for the solution of a specific problem, could be adopted by the designers. In the next section, the main strengthening techniques, grouped by purpose as in NTC 2008, are presented and compared to similar interventions suggested by EN 1998-3 (2005).

3 STRUCTURAL INTERVENTIONS

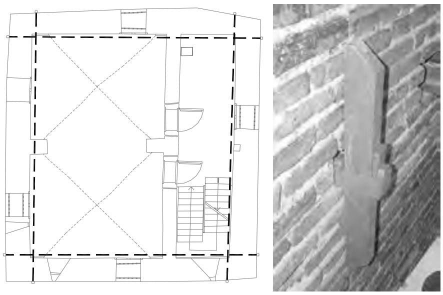
3.1 Interventions to improve connections

One of the first aspect to be taken into account when dealing with the seismic behaviour of existing masonry buildings, is the lack of good connections between structural elements. Hence, to allow the structure to manifest a satisfactory global behaviour, it is necessary to improve the connections between masonry walls, and between walls and floors and walls and roofs (Tomaževič and Weiss, 1994; Tomaževič and Lutman, 1996).

This goal may be achieved inserting ties (Figure 1), confining rings (Figure 2), and tie-beams at the top of the building (preferably in reinforced masonry or steel, also in r.c. but with restrictions, Figure 3). An effective connection between floors and walls is useful since it allows a better load redistribution and applies a restraining action towards the walls' overturning. In the case of wooden floors, a satisfactory connection is provided by fasteners anchored on the external face of the wall (Figure 4).

Conversely, introduction of tie-beams in the masonry thickness at intermediate storeys should be definitely avoided, due to their damaging effects on perimeter walls, often causing also uneven load redistribution among masonry leaves and/or pounding effects on the external masonry leaves in case of seismic excitation. The Eurocode 8 treats the problem of wall to wall intersections in sub-section C.5.1.2. The three techniques reported there are "construction of a reinforced concrete belt", "addition of steel plates or meshes in the bed-joints" and "insertion of inclined steel bars in holes drilled in the masonry and grouting thereafter". No mention is made to the wall-to-floor and wall-to-roof connection within this paragraph. It is well known that the first solution should be carefully considered, as it may worsen the overall behaviour of the structure, while the third one should be avoided in most cases, and is actually advise against in the Italian norms, because of its detrimental effects observed after recent earthquakes. Indeed, this technique has raised many doubts due to its invasiveness, to the durability issues raised by the insertion of steel bars into masonry, and to the scarce effectiveness, related to both the execution procedures and the absence of adequate experimental investigation.

Further strengthening techniques, tie-beams and addition of steel ties, are presented in C.5.1.4 and C.5.1.5. Also in this case considerations about effects and feasibility of these techniques are omitted, and some indications, like the provision of adding tie-beams if not existing in the original structure, should be reviewed taking into account the current state of art.



a) b)

Figure 1. Positioning of stainless steel ties, Vanga tower (Trento): a) tie positioning, plan; b) view of the external anchor.

a) b)

Figure 2. External confining stainless steel cables, Clock Tower, Padua: a) detail of the cable insertion between the mortar joints; b) view of the positioned cables, façade.

a) b)

Figure 3. Stainless steel tie-beam (S. Stefano church, Monselice, Padua): a) plan of the intervention; b) detail of the metallic belt, from the outside.

Figure 4. Wooden beams connections to the masonry wall.

3.2 Interventions to increase the masonry strength

Interventions, aimed at increasing the masonry strength, may be applied to re-establish the original mechanical properties of materials or to improve their performance. Techniques, employed with caution, should make use of materials with mechanical and chemical-physical properties similar to the original ones (Valluzzi, 2008).

The local rebuilding ("scuci-cuci") methodology (Figure 5) aims to restore the wall continuity along cracking lines (substitution of damaged elements with new ones, reestablishment of the structural continuity) and to recover heavily damaged parts of masonry walls. The use of materials that are similar, in terms of shape, dimensions, stiffness and strength, to those employed in the original wall is preferable. Adequate connections should be provided to obtain monolithic behaviour. This intervention, which is detailed in such a way in NTC 2008,

is also suggested by the EN 1998-3 (sub-section C.5.1.1), only to stitch cracks by means of elongated bricks or stones, without any other specification, besides those regarding the use of heterogeneous materials, such as metal clamps, plates, or polymeric grids to enhance the intervention effectiveness.
An extensive research has recently focused on the used of non cement-based mortar grouting (Valluzzi, 2000; Valluzzi et al. 2004) to increase the strength of multi-leaf masonry walls, and brought to the requirement of compatibility, in terms of chemical-physical and mechanical properties between grout admixture and substrate wall, which is currently asked for by the norms. This technique (Vintzileou and Tassios, 1995) consists in the injection of mixture through a regular pattern of drilled holes (Figure 6), for increasing the connection between masonry layers. Studies (Valluzzi, 2000; Valluzzi et al, 2003) demonstrated (Figure 7) that injections do not significantly change the stiffness of walls, differently from RC jackets, improving at the same time the strength and consistency of walls provided with voids and/or irregular morphology. The EN 1998-3 mentions the injection technique to strengthen multi-leaf walls (sub-section C.5.1.6) and to repair cracks (sub-section C.5.1.1), suggesting the use cement-based materials or, in some cases, epoxy grouting, without taking into account the most recent research findings about chemical compatibility and effectiveness of low-strength grout injections (Vintzileou and Miltiadou, 2008).
Insertion of small-sized tie beams across the wall, supplying a connective function among the wall leaves (Figure 8), is mentioned in sub-section C.5.1.6 as a supplemental solution for the injections. This intervention permits to reduce transversal deformations and local problems of out-of-plane buckling or overturning due to lack of connection. In addition, the combination of these techniques can provide a larger increase of the overall strength of the wall, permitting to carry higher loads (Valluzzi et al. 2004).
A wide research, studying the dynamic behaviour of injections and transversal ties, is in progress. Two scaled masonry structures were built using three leaves masonry stones and tested on the shaking table (Figure 9). First results (Mazzon et al., 2009) confirmed that the overall stiffness of the injected model does not significantly increase with respect to the original one. In addition, higher values of seismic input can be reached, thanks to a monolithic performance due to the connection effect provided by mixture. Several compression, shear-compression and out-of-plain test complete the investigation, and allow to understand the mechanical behaviour of strengthened structures (Figure 10).

a) b)

Figure 5. Examples of "scuci-cuci" interventions on the bell tower of the Cathedral of Monza (Modena et al., 2002).

Figure 6. Strengthening interventions using: a) mortar grouting; b) structural repointing.

The mortar bed-joint repointing is an other technique to improve deteriorated joints (Figure 6), that consists in the replacement of degraded mortar (Corradi et al., 2008). If steel bars are inserted within the joints to limit the opening of vertical cracks, this modified method is known as structural repointing (D'Ayala, 1998; Valluzzi et al., 2005). Laboratory tests and numerical models show that it is possible to use new materials as FRP laminates instead of steel, to ensure compatibility and removability as well as the control of creep deformations (Garbin, 2008). This kind of intervention is described also by EN 1998-3 sub-section C.5.1.1, although no further details, for example on the effectiveness of the intervention in relation to the masonry thickness, are given there.

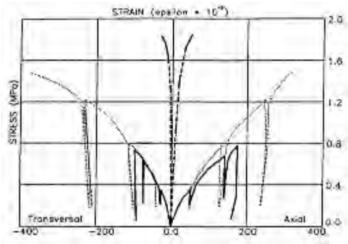
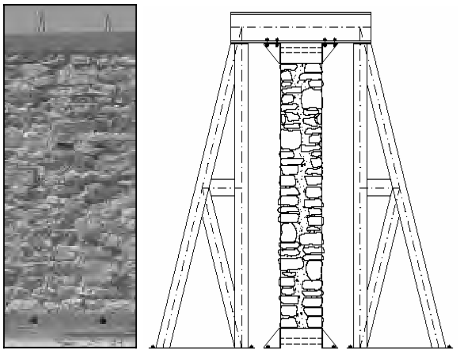


Figure 7. Comparison of structural behaviour using injections and jacketing (Modena and Bettio, 1994).

a) b)

Figure 8. Positioning of stainless steel threaded bars used as ties: a) fixing of the nut; b) external aspect of the strengthened wall.

Figure 9. Masonry model, reinforced using injections, on the shaking table (Mazzon et al., 2009).



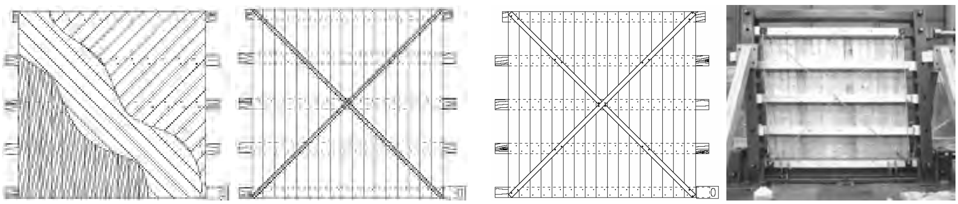
a) b)

Figure 10. Single panel (a) and steel frame (b) for shaking table tests.

Masonry strength can be also increased by means of the insertion of "diatoni" (masonry units disposed in a orthogonal direction with respect to the wall's plane), substituting damaged stones or introducing new elements to provide transversal connections between external layers of wall (NTC 2008). Other methods are mentioned in the EN 1998-3, such as the use of RC jackets or the insertion steel profiles (sub-section C.5.1.7) and application of polymeric grids jackets (sub-section C.5.1.8). A lack of critical evaluation about the proposed techniques is observed also in these cases: the suggested systems have to be carefully evaluated, since the suitability and the effectiveness should be demonstrated case by case. For instance, an incorrect application of RC jackets could easily worsen the structural behaviour because of an excessive stiffness and mass increase of portions of the structure (Modena and Bettio, 1994), or they can even be ineffective because of incorrect execution procedures or because of durability problems.

3.3 Interventions to reduce flexibility of floors and their consolidation

Interventions aiming at enhancing the in-plane stiffness of existing floors must be carefully evaluated, since it changes the redistribution of horizontal seismic action to the load-bearing walls, and this is seldom the objective of structural interventions. The role of diaphragms in the dynamic behaviour of masonry buildings consists in transferring seismic actions to the walls parallel to the earthquake direction (Tomaževič, 1991); therefore, an effective connection between floors and walls has a large importance as this can limit undesirable overturning of walls.



a) b) c) d)

Figure 11. Different strengthening interventions using: (a) rotated double planking; (b) orthogonal double planking with wooden diagonal; (c) orthogonal double planking with steel diagonal. (d) Steel frame used for laboratory tests (Valluzzi et al., 2008).

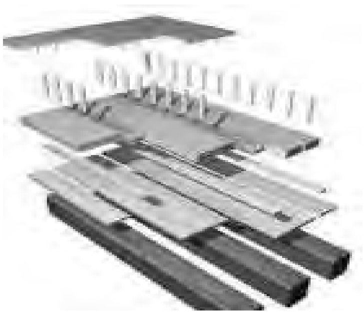


Figure 12. In-plane and Out-of-plane stiffening of an existing wooden floor: wooden planks at the extrados connected by wooden dowels.

Figure 13. In-plane stiffening of existing wooden floors: metallic belts reinforcing wooden roof.

Providing a further layer of wooden planks is a limited intervention, that does not modifies the overall behaviour and the force redistribution, and increases the wooden floors stiffening (Parisi and Piazza, 2002b). Some studies focused on the analysis of the double planking method (Valluzzi et al, 2008), as the application in orthogonal or inclined direction and the use of tongue-and-groove joints and nails or screw as connectors (Figure 11). This technique may be adapted (Figure 12) by using only wooden connectors instead of metallic ones (Modena et al. 2008). In addition, the use of metallic belts or FRP strips, disposed in a crossed pattern and fixed at the extrados of the wooden floor (Figure 13), or the use of metallic tie-beams bracings, may improve not only the stiffening effect (Corradi et al., 2006), but also the wall to floor connections.

Similar techniques are listed in the EN 1998-3 (sub-section C.5.1.3), underlining the importance of a correct connection between horizontal and vertical structures. However, the application of an overlay of concrete reinforced with welded wire mesh is proposed without pointing out the considerable increase of diaphragm self-weight and stiffness that it produces . In the same paragraph, it is correctly recommended to brace and anchor the roof trusses to the supporting walls (Piazza and Candelpergher, 2001; Parisi and Piazza, 2002a).

3.4 Interventions to reduce thrust of vaulted arches and their strengthening

Among the structural components in masonry buildings, arches and vaults deserve particular attention for being widespread in European historical centres; therefore, their preservation as part of the cultural heritage is a topical subject. These structures can suffer several types of damage, due to many causes (such as earthquakes, age, etc.). Hence, the contribution of strengthening materials and repair techniques is often required to re-establish or enhance their performances and to prevent a brittle collapse of the masonry in possible future hazardous conditions.

The EN 1998-3, differently from the Italian norms, does not provide any information concerning the interventions on this type of structures. Strengthening methods (Oliveira and Lourenço, 2004) may be applied by using the traditional techniques of tie-rods to compensate the thrust induced on the bearing walls. In addition, to absorb thrust of vaulted arches, the possibility of realizing buttresses or reinforced transverse vertical diaphragms should be considered, whilst jacketing the extrados using concrete, reinforced or not, should be avoided. Composite materials, such as FRP (Barbieri et al., 2002; Valluzzi, 2008) or SGP/SRG (Borri et al., 2008) could be a suitable option in some cases (Figure 15). In recent years, experimental researches focused on the behaviour of masonry vaults strengthened by new composite materials, as carbon or glass FRPs, placed at the intrados (inner surface) or at the extrados (outer surface) of the structure (Figure 14) (Valluzzi et al, 2001; Panizza et al, 2008). A multilayer system of adhesion based on epoxy adhesives and designed to provide a support as homogeneous as possible for the fibers has been adopted.

Figure 14. Collapse mechanism of a vault strengthened using CFRP strips (Valluzzi et al., 2001).

Figure 15. Palazzo Ducale di Urbino: reinforced transverse vertical diaphragms.

4 CONCLUSIONS

The Italian norms have introduced since a long time ago the concept of 'seismic improvements' which is a viable solutions for those buildings, where the requirement of

satisfying current safety level adopted for new buildings would imply an excessive use of strengthening measures and the complete loss of the original building concept and value. Nevertheless, this concept has not been yet defined into the European norms.
Going more in details, the recent Italian seismic norms, OPCM 3431 and NTC 2008, provide sound criteria for the application of interventions and group the strengthening techniques following a performance-based scheme, while the EN 1998-3 does not show a consistent outline, as it mixes lists of interventions collected by purpose, such as wall intersections or horizontal diaphragms, with descriptions of single techniques, without providing any general concept on how the intervention should be conceived, designed and executed.
In addition, many interventions suggested for masonry buildings are obsolete, as the past ten years of earthquakes have demonstrated their ineffectiveness. Concurrently, many experimental researches have been also carried out. However, these state of the art knowledge has affected only the Italian norms. In this context, and considering the continuous technological development, the Italian norms are open to new types of interventions and materials, that may emerge after the drawing up of the norms, provided that some simple criteria are respected and that the effectiveness of the new materials and techniques is not only asserted, but also demonstrated.
Conversely, the provisions of Eurocode 8 point out specific aspects, for instance materials to be used, but are lacking of information concerning effectiveness, feasibility, suitability, and compatibility, under the chemical-physical and mechanical point of view, of each kind of intervention. Many important aspects that would complete the description of structural interventions on historical masonry buildings are totally disregarded, such as the strengthening of columns and pillars, the interventions on arches, vaulted structures and foundation systems, etc.
Furthermore, the Italian system of seismic norms is also provided with specific Guidelines (2007) for the application of the above criteria to cultural heritage buildings, following a methodology for the application of the seismic standards which takes into account the requirements of preservation.
In conclusion, the Eurocode 8 does no deal with the topic of interventions on masonry buildings in a sufficiently consistent and organic way. Starting from these above mentioned considerations, the way to improve the document could be based on more general principles, matched with a substantial review of all the other aspects, included in the European norm, which concern the seismic assessment and improvement of existing masonry buildings, and with the explicit description of the specific features of the seismic behaviour of masonry buildings. This approach would avoid the *a priori* exclusion of technical solutions not explicitly mentioned, could simplify the update and the integration of the document and improve its suitability for application on historical structures.

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EUROCODE 8 AND ITALIAN CODE. A COMPARISON ABOUT SAFETY LEVELS AND CLASSIFICATION OF INTERVENTIONS ON MASONRY EXISTING BUILDINGS

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ABSTRACT
The Eurocode 8 will probably come into effect all over Europe in the next years, substituting all the national codes. EC8 is quite difficult to be applied in Italy due to the presence of existing masonry buildings, aggregated buildings in the characteristic historic centres and of cultural heritage. This paper deals with the problem of the safety level of these buildings that should depend on the class of the intervention and the artistic and cultural importance of the construction, just like the Italian Code permits.

KEYWORDS
Seismic upgrading, seismic improvement, aggregated buildings, preservation, safety level.

1 INTRODUCTION

Considering the forthcoming introduction of the Eurocodes as prescriptive Italian reference about constructions, there will be soon the problem of coordination of the Italian existing standards with the Eurocodes. This problem becomes particularly evident in the arguments discussed in this paper: safety levels and classification of retrofitting interventions of existing masonry buildings located in seismic zones.

The Eurocode dealing with these arguments is Eurocode n. 8, particularly:

- EN1998-1 (definition of seismic action)
- EN1998-3 (existing buildings)
- Italian national annex regarding EN1998-1

In Italy, only in the last five years, a lot of standards and guidelines dealing with the above mentioned arguments have been introduced. Specifically it's worth to remember:

- Technical Rules for Constructions, proclaimed in 2008 (shortly named NTC 2008);
- OPM n. 3274 (2003), updated with OPM. n. 3431 (2005);
- Guidelines about the preservation of historical and architectonical heritage;
- Rules DT/200 by National Research Council (CNR) dealing with the utilization of composite materials in retrofitting of existing buildings.

The whole of this standards and rules, more advanced than the Eurocodes dealing with these arguments (specifically Eurocode n. 8), is a legacy of knowledge deriving from the Italian seismic recent experiences (just like the Umbrian earthquake of 1997 and the Molise earthquake of 2002), This knowledge is not to waste because it will be precious in the next reconstruction after the last destructive earthquake that hit L'Aquila.

Among the most significant aspects faced by the Italian Code and ignored by the Eurocode n. 8, it is possible to remember, above all:

- the deep attention dedicated to the question of masonry and its quality;
- the classification of the intervention in "seismic upgrading", "seismic improvement" and "local interventions", depending on basic criteria of extension of the intervention, transformation of the original behaviour of the construction and safety levels to be performed;
- the case of aggregated buildings is discussed; this is a typical configuration in the Italian historic centres;
- the case of historical and architectonical heritage is discussed; this is another typical problem of retrofitting of buildings;
- safety factors depend on knowledge of the construction; three levels of knowledge has been defined; in such a way there is a direct correspondence between safety and knowledge;
- a range of possible values of the principal mechanical parameters are defined for masonry;
- the most common techniques of intervention are shortly explained.

The following sections deal with some of the aspects above introduced. Their aim is to show that Eurocode n. 8 needs a deep updating before being used concretely in the Italian reality.
It is remarkable that all what is written in this paper may be a problem not only in Italy. In fact there are other seismic zones in Europe with many existing masonry buildings. This is quite evident looking the Figure 1.

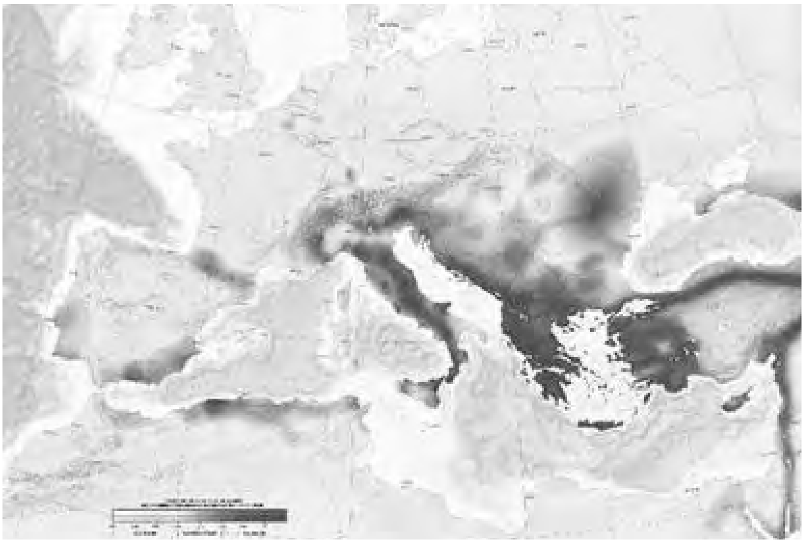
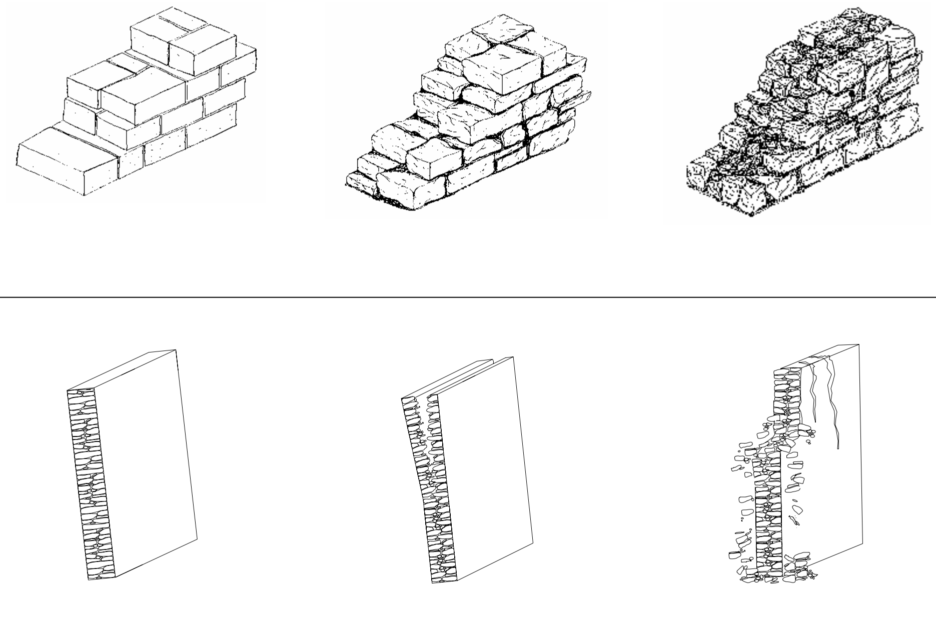


Figure 1. European seismic hazard map.

2 THE QUALITY OF MASONRY AND OF THE CONNECTIONS BETWEEN ELEMENTS OF THE BUILDING

The quality of masonry plays a fundamental role in determining the capacity of a construction to oppose to seismic action. This problem cannot be studied only in terms of stress and strain: a masonry which can resist and transfer the vertical and seismic forces without breaking up should have geometric and physical characteristics that permit a monolithic behaviour.
Among the features which the Italian code requires to consider (and model) a masonry like a “good quality” one, we can remember now: horizontal courses, not-aligned mortar vertical joints, square-shaped and big stones or bricks, presence of transversal connections in multi-leaf walls, good quality of mortar and, obviously, an adequate strength of the brick or stones.



High quality masonry	Medium quality masonry	Poor quality masonry

Figure 2. Example of out-of-plane behaviour of high, medium and poor quality. High quality masonry has a monolitic out-of-plane behaviour. In the medium quality masonry there is a lack of monolitic out-of-plane behaviour. Poor quality masonry produces a complete disintegration of the wall.

A good quality of the connections between floors and walls, between roof and walls and between crossing walls is also crucial to reach a good global seismic behaviour of the building. Good quality connections will drive the collapse of the construction to a configuration that requires a stronger seismic action.

Figure 3. In this masonry building it is possible to see a flexural wall failure. There is a good roof-to-wall connection but the middle floor is not connected to the wall. In this case we can see how important is to achieve the connection between the roof and the existing masonry.

Figure 4. In this masonry building it is possible to see the overturning of the wall caused by a poor connection of the roof to the wall.

3 SAFETY LEVELS DEPENDING ON THE CLASS OF INTERVENTION

Italian code considers different safety levels to be reached depending on the typology of intervention that is going to be realized on the building. In fact, the Italian code states a fixed safety level which has to be necessarily reached only in case of "seismic upgrading", that is in case of heavy and wide interventions (see hereinafter). In case of "strengthening" or "localized intervention" it is necessary only to prove that the safety level of the building will effectively be improved in consequence of the intervention.
The classification of interventions stated by the Italian Technical Code for Constructions (paragraph 8.4 and paragraph C8.4 of the Circular n. 617/2009) is the following:

seismic upgrading

- A determined safety level has to be pursued. It is quantitatively defined in terms of PGA depending on the estimated life of the structure, the utilization and importance of the construction, its geographic site, etc…
- The whole building has to be modelled and checked.

seismic improvement

- In consequence of the intervention it is necessary to obtain an higher safety level than before the intervention. The safety threshold to be reached is not a priori fixed.
- Safety level valuation concerns the whole building.

local intervention

- In consequence of the intervention it is necessary to obtain an higher safety level than before the intervention. The safety threshold to be reached is not a priori fixed.
- Safety level valuation is "localized" to the element or area of intervention.

It is possible to illustrate a list of some of the most frequently realized interventions on existing masonry buildings classifying each intervention in one of the three above mentioned categories. In this way it will be evident the richness and variety of the effective situation of existing masonry buildings and the need of differentiate the various situations.

3.1 Seismic upgrading interventions

Among the seismic upgrading interventions (paragraph C8.4.1 of the Circular n. 617/2009) appear the following interventions:

- Addition of stories over an existing building;
- enlargements of the building;
- introduction of a new floor at an intermediate quote;
- introduction of independent and important structures into existing constructions;
- construction of underground rooms under existing structures;
- demolition and rebuilding in a different position of a large quantity of main walls;
- realization of seismic joints in order to divide the original building into two or more separate buildings.

3.2 Seismic improvement interventions

Among the seismic improvement interventions (paragraph C8.4.2 of the Circular n. 617/2009) appear the following interventions:

- demolition and rebuilding of some main walls in the original position but with significantly different stiffness and strength, so that the global seismic behaviour of the structure will be significantly changed;
- construction of new main walls which significantly modify the seismic behaviour of the original structure;
- construction of stairs that cause a significant mass or stiffness variation;
- construction of elevators structurally connected to the existing building if the elevators have a very stiff structure;
- systematic substitution of floors and roofs involving a significant stiffness variation or a weight increase;
- systematic strengthening of the masonry walls (i.e. shotcreting by steel wire mesh attached to the existing wall with through-wall ties; grouting with cement grout) on a large number of walls so to significantly modify the stiffness-ratio between the main walls and the distribution of the seismic action.

3.3 Localized interventions

Among the seismic “localized interventions” (paragraph C8.4.3 of the Circular n. 617/2009) appear the following interventions:

- repair or substitution of a single damaged element on condition that the global seismic behaviour of the structure will be not significantly changed (i.e. substitution of lintels, beams, walls, floor or roof on a single room);
- total substitution (on the entire level) of floors or roof without a significant stiffness variation of the floor or roof and without weight-increase;
- construction of little stairs without involving significant variation of stiffness and mass;
- repair or strengthening of the floor-to-walls and wall-to-wall connections;
- installation of metallic ties;
- localized strengthening of floors or roof by the installation of a new concrete slab atop the existing floor. The slab has to be well anchored to the existing main walls.

3.4 The EC8 point of view

Compared to all this cases discussed by the Italian Code, the Eurocode n. 8 (EN1998-3), on the contrary, seems to be quite rigid and inadequate. EN1998-3 states only a single procedure for retrofitting of existing (masonry) buildings and it always states to reach a fixed safety level for every typology and extension of the retrofitting intervention.
In the section 6.1 of EN1998-3 it is possible to read:

" 6 Design of structural intervention
6.1 Retrofit design procedure
(1)P The retrofit design procedure shall include the following steps:
a) Conceptual design,
b) Analysis,
c) Verifications.
(2)P The conceptual design shall cover the following:
(i) Selection of techniques and/or materials, as well as of the type and configuration of the intervention.
(ii) Preliminary estimation of dimensions of additional structural parts.
(iii) Preliminary estimation of the modified stiffness of the retrofitted elements.
(3)P The methods of analysis of the structure specified in 4.4 shall be used, taking into account the modified characteristics of the building.
(4)P Safety verifications shall be carried out in general in accordance with 4.5, for both existing, modified and new structural elements. For existing materials, mean values from in-situ tests and any additional sources of information shall be used in the safety verification, modified by the confidence factor CF, as specified in 3.5. However, for new or added materials nominal properties shall be used, without modification by the confidence factor CF.
(5)P In case the structural system, comprising both existing and new structural elements, can be made to fulfill the requirements of EN1998-1: 2004, the verifications may be carried out in accordance with the provisions therein ".

The comparison between the Italian Code and the Eurocode n. 8 is resumed in the flow-chart reproduced in Figure 5.

4 THE CASE OF CULTURAL AND ARTISTIC HERITAGE

The Italian body of laws and codes about constructions faces the problem of the artistic buildings in the "Guidelines for valuation and decrease of seismic hazard of cultural heritage referring to technical construction code".
The purpose of the Guidelines is to match seismic-safety requirements with preservation requirements when the building is a unique and unrepeatable artistic construction. In fact, in such an eventuality, it is necessary to graduate the safety level of the intervention to be the highest possible without modify (or, at worst, damage) the nature and the characteristics of the protected building.
The Italian Guidelines for cultural heritage are based on this concept. The Guidelines, in fact, direct the attention to seismic improvement rather than seismic update interventions because seismic improvement does not imply any fixed safety threshold achievement. The only law prescription is to demonstrate that, in consequence of the intervention, the safety level increases.

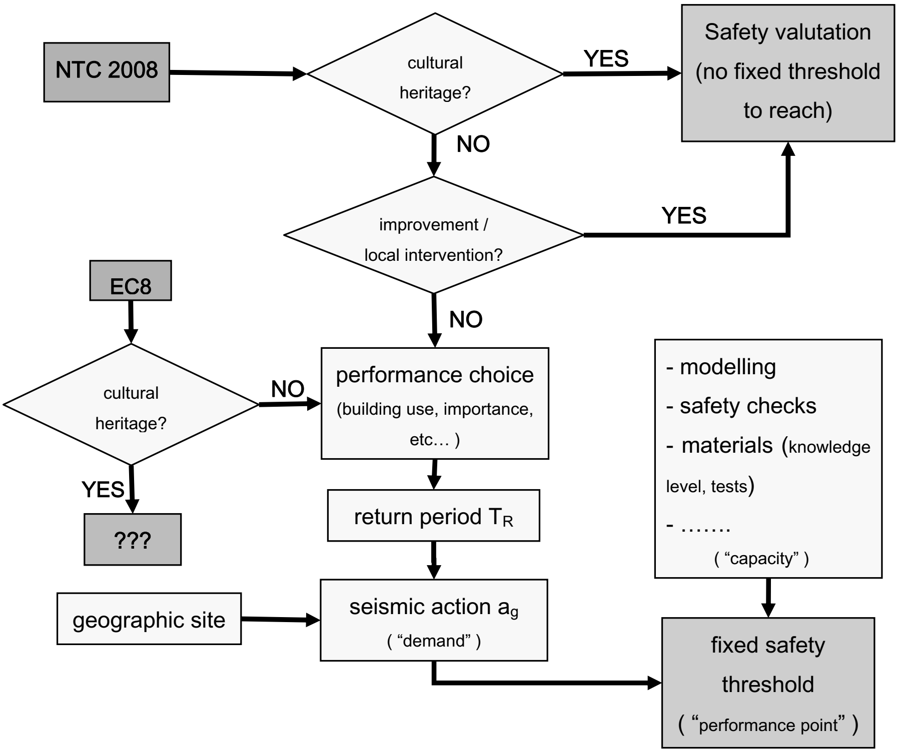


Figure 5. Flow-chart explaining the procedure to determine the safety level. It is clear that EC8 cuts off the eventuality of seismic improvement and local interventions and doesn't deal with cultural heritage.

In Guidelines much attention is pointed on relief and knowledge of the building, safety calculations to be performed, an adequate structural model (i.e. cinematic model) and monitoring the building after the realization of the intervention.
Seismic design actions depend on the importance of the artistic building and its utilization-class (occasional presence, frequent presence, very frequent presence).
In order to prevent the seismic collapse of objects and decorations in the building, an appropriate "artistic limit state" (SLA) has been introduced in the guidelines.

On the contrary, although EN1998-3 clearly admits (paragraph 1.1) that cultural heritage needs a different approach than ordinary existing buildings, no indication can be found in the European Code. It is evident from the flow chart in Figure 5.
Here the paragraph 1.1 of EN1998-3 is reproduced:

" (5) Although the provisions of this Standard are applicable to all categories of buildings, the seismic assessment and retrofitting of monuments and historical buildings often requires different types of provisions and approaches, depending on the nature of the monuments ".

It is hoped that such a gap will be corrected by releasing a specific Eurocode dealing with all the problems of cultural heritage in seismic zones, just like the Italian Guidelines.

5 AGGREGATION BUILDINGS

The Circular n. 617/2009 deals with one of the most important problems of the Italian buildings in seismic zones: the behaviour of aggregated constructions under seismic actions. It is a really frequent eventuality, specially in the historic centres deriving from a building process which lasts for centuries.
The aggregation could be defined as a construction delimited by open spaces (Figure 6). It is formed by Structural Units (U.S.) which could be defined as portions of the aggregation which have a unitary behaviour from a static and seismic point of view.
U.S. are defined thanks to structural criteria (i.e. a rigid floor defines a single U.S. or two parts with a different kind of masonry are two U.S.) and thanks to historical criteria, according to the age of construction of the several parts.

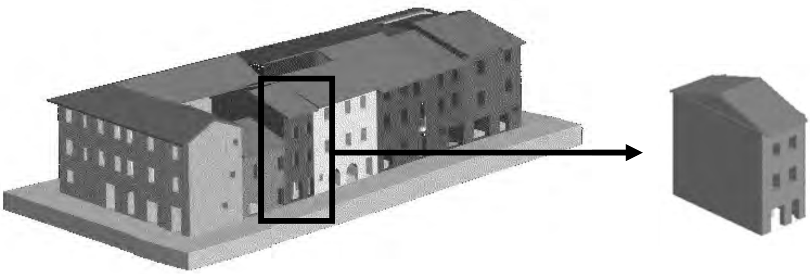


Figure 6. Aggregation and structural unit.

The analysis of a structural unit belonging to an aggregation is different than the case of an isolated building because of the several interactions that the adjacent buildings do on the structural unit which is analysed.

The basic structural interaction phenomena may be classified in two categories:
a) vertical loads or horizontal pushes (specially under the seismic action) coming from adjacent buildings;
b) buttressing or constraining effects offered by the adjacent buildings.
These interactions modify the collapse mechanism of the building introducing new different actions and changing the constrain configuration.
Hereinafter in the Figures from 7 to 10 some typical situations of interaction between aggregated buildings are listed.

Figure 7. The not aligned facade causes a wall not to be buttressed by the other buildings.

Figure 8. The U.S. in head position is not buttressed by the other buildings.

Figure 9. Adjacent U.S. of different height. It may be possible the collapse of the higher one.

Figure 10. Floors at different height in adjacent U.S. The seismic action can push on the common wall.

Even in this case, EN1998-3 appears quite inadequate. In fact, the problem of aggregation buildings is faced only in the paragraph C.2.1 where it is only possible to read:

“(1) The following aspects should be carefully examined:
[… *omissis*…]

vi. Information on adjacent buildings potentially interacting with the building under consideration ”.

6 DIFFUSION OF SEISMIC IMPROVEMENT AND INTERVENTIONS ON CULTURAL HERITAGE

With the purpose of showing how frequent are the seismic improvement interventions in the Italian building praxis, some informations regarding the Umbrian 1997 post-seismic reconstruction are reported hereinafter (data by Provincia di Perugia).
In the time-extension from 2001.08.01 to 2006.12.31, in a sample of 1479 random-checked masonry buildings which suffered damage from earthquake of 1997, the intervention-design can be classified in this way:
seismic improving: 1148 buildings (78%);
seismic updating: 153 buildings (10%);
rebuilt buildings: 178 buildings (12%).
Moreover on the total sample of 1479 buildings, the ones which are protected by special laws dealing with cultural and artistic heritage are 164 buildings (11%).

7 CONCLUSIONS

The EC8 in the current version is not appliable to all the Italian constructions. Particularly it is difficult to design a retrofitting intervention of an existing masonry buildings (the most diffused and the most seismic vulnerable typology of building in Italy). The paper focuses on three questions that should be resolved: the absence, in EC8, of a graduation of safety levels depending on the class of the intervention, the absence, in EC8, of any indication about preservation techniques of the buildings belonging to cultural heritage and, finally, the absence, in EC8, of any indication about aggregated buildings of historic centres. The paper indicates a simple solution of the mentioned problems: to update EC8 taking into account the recent Italian Codes. This should be done before EC8 come into effect, at least in Italy, where the Abruzzi reconstruction is now starting, even if the problems may concern other European nations.

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GEOTECHNICAL EARTHQUAKE ENGINEERING

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FORCE-BASED PSEUDO-STATIC METHODS VERSUS DISPLACEMENT-BASED METHODS FOR SLOPE STABILITY ANALYSIS

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ABSTRACT
The response of ground slopes to earthquake loading may be studied via effective stress dynamic analyses, displacement-based sliding block analyses or through force-based pseudo-static methods, the latter being more often used in common practice. In the pseudo-static approach, an equivalent seismic coefficient *k* is used within a conventional limit equilibrium slope stability calculation. Since *k* designates the horizontal force to be used in the stability analysis, its selection is crucial. In this work, the equivalent seismic coefficients are evaluated relating the ground motion amplitude to earthquake-induced slope displacements, using a database of Italian strong-motion records. Account is also taken of the influence of significant duration and frequency content of ground motion, as well as of the effect of soil deformability.

KEYWORDS
Slope stability, earthquake, seismic performance, displacements, seismic coefficient.

1 MECHANISMS OF SLOPE INSTABILITY UNDER SEISMIC CONDITIONS

Earthquake loading induces inertial forces within the slope that combine with the pre-existing static forces, reducing the slope stability. The main effects associated to earthquake loading consist of permanent displacements induced by temporary mobilisation of the shear strength or by accumulation of plastic deformation for stress states even far from failure conditions. Slope displacements may increase progressively during the earthquake, or be suddenly triggered at a certain instant of time during the ground motion, or develop only after the end of the earthquake. They can result from strains developed throughout the slope or by strain localisation within the failure zone.

The attainment of significant slope displacements implies that shear strength mobilisation is achieved in a significant portion of the slope. The available shear strength can also reduce during earthquake loading, due to pore water pressure increase or to degradation of shear strength parameters. Assuming undrained stress condition during the seismic action, the shear strength can be expressed in the form:

$$\tau_f = c' + (\sigma'_n + \Delta\sigma_n - \Delta u)\cdot\tan\varphi' \tag{1}$$

where σ'_n is the effective stress normal to the sliding surface, $\Delta\sigma_n$ and Δu are the changes in total normal stress and in pore water pressure induced by earthquake loading; effective

cohesion, c', and the angle of shearing resistance, φ', include eventual degradation effects associated to seismic actions.
From what mentioned above, slope instability can be essentially affected by inertial effects and/or by shear strength reduction. Inertial forces are transient actions proportional to seismic accelerations. In contrast, shear strength reduction withstands at the end of the earthquake, being mainly related to the development of excess pore water pressure and to cyclic soil degradation. Such a difference is schematically shown in Figure 1, in which the resultant of gravitational and seismic actions is schematically represented versus time by a harmonic function and the shear strength is plotted as a decreasing monotonic function.

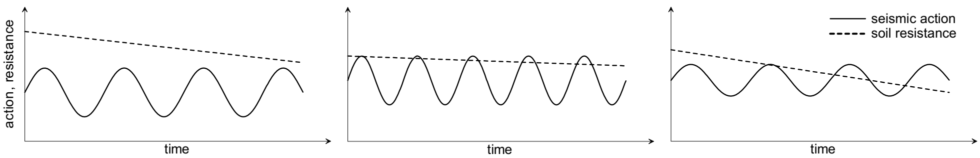

Figure 1. Schematic representation of seismic action and soil resistance versus time.

In Figure 1a, the static plus seismic actions are lower than the shear strength, both during and after the seismic event. With the simple assumption of rigid-perfectly plastic soil behaviour, no displacement is induced in the slope in this case. In Figure 1b, the total load exceeds soil resistance for limited periods of time during the earthquake, but the gravity force acting at the end of the seismic event is still lower than the available shear strength; in this case, earthquake-induced displacements develop during the event, the slope remaining stable in the static, post-seismic conditions. Finally, in Figure 1c the shear strength decreases substantially during the earthquake, resulting lower than the static action at the end of the event. In this case, the total seismic action may exceed soil resistance during the earthquake, thus inducing some slope displacement, but slope instability is mainly produced by the static forces acting on the slope at the end of ground motion. In fact, gravity forces higher than the shear strength available in the post-seismic conditions produce the onset of failure mechanisms characterised by high displacements.
Due to the transient character of the seismic actions, slope instability associated to inertial effects actually consists of cumulated displacements (Figure 1b), rather than of a failure mechanism. On the contrary, instability induced by shear strength reduction consists of a failure mechanism produced by the destabilising gravity forces that are constant with time and greater than the available strength at the end of earthquake (Figure 1c). In this case, slope displacements proceed till a new geometrical configuration of equilibrium is attained for the soil mass.
Failure mechanisms caused by shear strength reduction assume different characteristics depending on soil behaviour, ductile or brittle, and soil type, granular or cohesive, as discussed by Rampello and Callisto (2008). These mechanisms can be analysed in the static condition following the seismic event, using conventional limit equilibrium methods, eventually accounting for the increase of pore water pressure and the degradation of strength parameters induced by earthquake loading.
On the contrary, when slope instability is produced by earthquake-induced inertial forces, a progressive development of slope displacements occurs for the duration of ground motion only. Accordingly, evaluation of slope response to earthquake loading should be carried out, in principle, using analysis procedures which account for time-dependent seismic action and that allow an evaluation of the induced displacement to be obtained. If a pseudo-static

approach is adopted in this case, the equivalent seismic coefficients used in limit equilibrium calculations must be calibrated against specified levels of slope performance, in turn defined by specified threshold values of earthquake-induced displacements. In fact, in the pseudo-static approach the safety factor F_S provides an indirect estimate of the seismic performance of the slope to earthquake loading, while under static conditions it represents a measure of the distance from a potential failure mechanism.

In the following, attention is focused on slope instability induced by inertial effects and on the appropriate choice of the seismic coefficient to be adopted in the pseudo-static approach. A procedure is proposed in which the equivalent seismic coefficient is related to limit slope displacement and ground motion parameters.

2 PSEUDOSTATIC ANALYSIS AND EQUIVALENT SEISMIC COEFFICIENT

The main difficulty in evaluating earthquake-induced slope displacements is the selection of acceleration time histories representative of site seismicity. In common practice, the seismic action is represented by some parameters that describe the main characteristics of ground motion (maximum horizontal acceleration, maximum velocity, Arias intensity etc.). A number of parametric studies can be found in the literature in which permanent displacements were first computed using a large number of accelerograms; the displacement amplitude was then linked to some representative parameters of the accelerograms used in computations and to the vulnerability of the slope, as described by the critical acceleration (e.g. for European or North America seismic events: Franklin and Chang, 1977; Makdisi and Seed, 1978; Ambraseys and Menu, 1988). By best-fitting the computed results, the proposed procedures provide conservative estimates of maximum expected displacements, for a given input seismic event.

When using the pseudo-static approach, the seismic action is assimilated to an equivalent static force. In this procedure, a destabilizing seismic coefficient k is entered into a conventional slope stability analysis. The coefficient k represents the fraction of the weight of the sliding mass that is applied as an equivalent static force. In the following, two types of seismic coefficients are distinguished: the first is the seismic coefficient that reduces the pseudo-static factor of safety (F_S) for a given slope to unity, and is referred to as the yield coefficient k_y; the second is the peak value of spatially averaged horizontal acceleration (normalized by g) within the slide mass, and is denoted as k_{max}.

The pseudo-static approach provides an evaluation of a safety factor against sliding that is assimilated to a failure mechanism. However, as mentioned above, the inertial effects mainly produce a progressive development of permanent displacements in the slope rather than a failure mechanism, due to the transient and cyclic nature of seismic actions. Using the pseudo-static approach for evaluation of slope stability under seismic conditions then requires relating the maximum expected displacements to the seismic coefficient k and the safety factor F_S. Such an equivalence can be obtained using relationships of the kind mentioned above between earthquake-induced displacements and given ground motion parameters.

2.1 Existing simplified procedures

Simplified procedures have been proposed in the literature by Seed (1979) and Hynes-Griffin and Franklin (1984) for applications to earth dams, Bray et al. (1998) for solid-waste landfills and Stewart at al. (2003) for hillside residential and commercial developments. All of them define an equivalent seismic coefficient k to calibrate the pseudo-static method to a particular level of slope performance, as indexed by earthquake-induced displacements. These procedures refer to earthquake magnitudes M in the range 4–8, limit displacement d_y as high as 100 cm, and define the equivalent seismic coefficient k as a fraction (η) of the maximum acceleration at the bedrock (Table 1).

Table 1. Features of existing procedures for evaluation of seismic slope stability.

	Seed (1979)	Hynes-Griffin & Franklin (1984)	Bray et al. (1998)	Stewart et al. (2003)
	earth dams	earth dams	landfills	residential hillside
M	6.5 , 8.25	3.8 – 7.7 (6.6)	8	6, 7, 8
d_y (cm)	100	100	15 – 30	5 – 15
η	--	0.5	0.75	$\eta(a_g, M, r, d_y)$
K	0.1 , 0.15	$0.5 \cdot a_g/g$	$0.75 \cdot a_g/g$	$\eta \cdot a_g/g$
F_S	1.15	1.0	1.0	1.0

It is worth noting that the procedures mentioned above underlie three important conditions:
- the level of displacement considered allowable for a specific application;
- the earthquake magnitude associated with the time histories used to calculate displacements;
- the level of conservatism employed in the interpretation of statistical distributions of results.

Due to the differences in both seismic activity and design practice, such procedures can be hardly considered of straightforward application to the European seismic engineering context.

2.2 Proposed simplified procedure

Following the above background, a procedure has been recently developed by Rampello et al. (2008), in which the horizontal seismic coefficient k and the corresponding safety factor F_S are evaluated using an equivalence with the results of a parametric application of the displacement method, as proposed originally by Newmark (1965). In the procedure, the seismic coefficient is expressed as a function of the maximum acceleration of the slide mass (k_{max}), the ratio of slope resistance to peak demand k_y/k_{max} and the limit displacement d_y considered as tolerable for the slope. Slope stability is satisfied for values of the safety factor $F_S \geq 1.0$. The procedure can be thought of as being related to earthquake magnitudes $M = 4 – 6.5$, typical of Italian seismic events.

The principle adopted to introduce a relationship between the earthquake-induced displacement and the corresponding seismic coefficient is schematically shown in Figure 2.

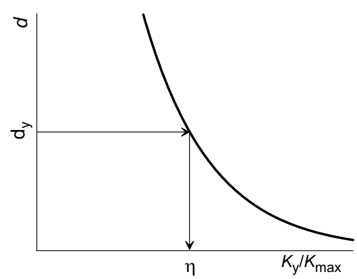

Figura 2. Equivalence between permanent displacement and seismic coefficient.

The permanent displacement d induced by an acceleration time history can be expressed as a function of the ratio k_y/k_{max}. Earthquake-induced displacement rapidly decreases with increasing k_y/k_{max}; for $k_y/k_{max} = 1$, the critical acceleration is never trespassed during the seismic event and no permanent displacement is triggered in the hypothesis of rigid-perfectly plastic soil behaviour. Once the relationship between the displacement d and the ratio k_y/k_{max} is known, for a given limit displacement d_y the corresponding value η of the ratio k_y/k_{max} can be obtained, as shown in Figure 2. If a pseudo-static analysis is carried out using $k = \eta \cdot k_{max}$ and a full mobilization of shear strength is attained ($F_S = 1$), a permanent displacement equal to d_y can be anticipated for the slope. The equivalent seismic coefficient k can then be defined as a fraction η of the maximum acceleration a_{max} of the slide mass:

$$k = \eta \cdot k_{max} = \eta \cdot \frac{a_{max}}{g} \tag{2}$$

where η decreases with the allowable displacement.

In principle, the application of the displacement method should be performed using the equivalent accelerogram acting in the sliding mass, as obtained by one or two-dimensional seismic response analyses. In the proposed approach, a rigid soil behaviour was assumed, which implies that, until full mobilisation of shear strength, the acceleration distribution is uniform through the soil mass. Under this assumption, site effects are simply taken into account using amplification factors for subsoil profile S_S and ground surface topography S_T, as specified by technical recommendations or building codes (e.g.: EN 1998-5, D.M. 14.01.2008); in eq. (2) it is then $a_{max} = S_S \cdot S_T \cdot a_g$.

Permanent displacement were evaluated through a Newmark-type integration of the portion of the accelerograms in excess of k_y; only Italian acceleration time histories were used as provided by database SISMA (Scasserra et al., 2008). A total of 214 accelerograms were used, pertaining to 47 events recorded by 58 stations. The accelerograms, with peak ground acceleration greater than 0.05 g, refer to earthquake magnitudes $M = 4 - 6.5$, epicentre distances of 1 to 87 km, focal depths in the range 2 – 24 km. They were divided into three groups, according to the subsoil underlying the recording sites, as defined by Eurocode 8 and by the Italian building code (EN 1998-5; D.M. 14.01.2008): rock or rock-like subsoil, with shear wave velocity $V_S \geq 800$ m/s (A); dense granular and stiff cohesive subsoil, with $V_S = 360 - 800$ m/s (B); medium to loose granular and medium stiff to soft cohesive subsoil, with $V_S < 360$ m/s (C, D, E). Specifically, 74 accelerograms were attributed to subsoil class A, 98 accelerograms to subsoil class B and 42 accelerograms to subsoil classes C, D and E.

For each group of accelerograms, peak accelerations were scaled to values of $a_{max} = 0.05$, 0.15, 0.25 and 0.35 g, limiting the scale factors in the range 0.5 – 2. Earthquake-induced displacements were computed integrating twice the equation of relative motion for translational sliding, using critical acceleration values equal to 10 to 80 % of the maximum acceleration ($k_y/k_{max} = 0.1 - 0.8$). Permanent displacements d computed for each subsoil class and for each level of acceleration were plotted as a function of the ratio k_y/k_{max} in a semi-logarithmic scale. An example is shown in Figure 3 for subsoil class B. Computed results were best-fitted using exponential relationships written in the form:

$$d = B \cdot e^{A \frac{k_y}{k_{max}}} \tag{3}$$

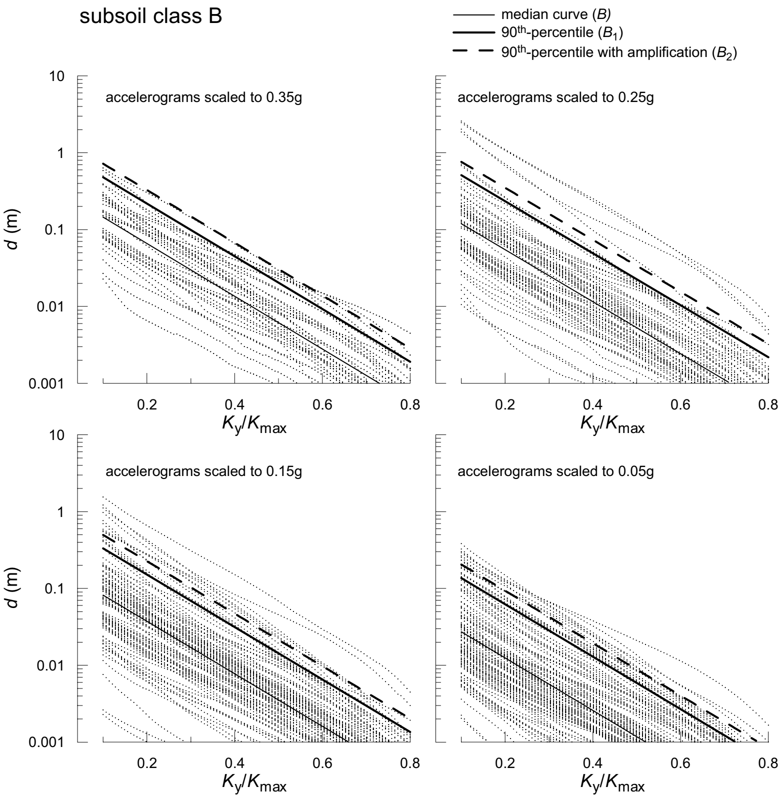

Figure 3. Permanent displacements computed using acceleration time histories recorded on class B subsoil (Rampello et al. 2008).

Assuming a log-normal distribution around the mean value, the 90th-percentile upper-bound displacements were obtained, their analytical dependence on the acceleration ratio being characterised by the same parameter A of the mean curves and by a value of $B_1 > B$.

At a constant critical acceleration ratio, the permanent displacement induced by a given accelerogram is proportional to a_{max}. It is then possible to account for an eventual amplification of ground motion, produced by site effects, multiplying the coefficient B_1 by the amplification factors S_S and S_T, thus obtaining $B_2 = S_S \cdot S_T \cdot B_1$; a further level of conservatism is in this way introduced in the procedure.

For a given threshold displacement d_y, the corresponding values of η can then be obtained by inverting eq. (3) with $d = d_y$ and $B = B_2$:

$$\eta = \frac{k_y}{k_{max}} = \frac{\ln\left(d_y / B_2\right)}{A} \qquad (4)$$

Table 2 reports values of η for threshold displacements of 5, 15, 20 and 30 cm, corresponding to levels of damage from severe to moderate (Idriss, 1985).

It is worth noting that the displacement method may be not capable of reproducing the actual deformation pattern of slopes, since the permanent strain field may be spread out over a zone, leading to bulging or lateral spreading mechanisms rather than sliding. Therefore, the threshold value of d_y, adopted for defining the equivalent seismic coefficient, may provide only the order of magnitude of the permanent displacements induced in the slope by earthquake loading, and should be considered as just a simple index of seismic performance of the slope, to be referred to given limit states.

Table 2. Values of coefficient η versus threshold displacements.

d_y (cm)	5	15	20	30	5	15	20	30	5	15	20	30
a_{max} (g)	η (subsoil class A)				η (subsoil class B)				η (subsoil class C, D, E)			
0.3 - 0.4	0.47	0.32	0.28	0.23	0.44	0.30	0.26	0.21	0.37	0.22	0.18	0.12
0.2 - 0.3	0.48	0.33	0.30	0.24	0.45	0.31	0.27	0.22	0.36	0.22	0.18	0.13
0.1 - 0.2	0.39	0.24	0.20	0.15	0.39	0.25	0.22	0.16	0.39	0.25	0.22	0.17
≤ 0.1	0.26	0.12	0.09	0.04	0.28	0.14	0.10	0.05	0.31	0.17	0.14	0.09

The obtained correspondence between earthquake-induced displacements and seismic coefficients imply that slope stability evaluated using the proposed procedure is satisfied for a values of safety factor $F_S \geq 1.0$. Whenever the pseudo-static approach yields values of $F_S > 1.0$, a critical acceleration greater than that assumed for the slope at hand can be assumed, this implying a significant reduction of the expected displacement with respect to the limit value.

The earthquake-induced displacements computed so far have been expressed as a function of the ratio k_y/k_{max}, assuming the slide mass to behave as a rigid body and accounting for the ground motion amplitude only.

The high scatter observed in Figure 3 implies that permanent displacements are affected by other ground motion parameters, in addition to the maximum acceleration of the slide mass a_{max}, that should be considered when handing out such correlations.

Actually, for a given maximum acceleration, large magnitude earthquakes will induce poorer slope performance than smaller magnitude earthquakes. One reason for this is that duration increases with magnitude, and slope displacements increase with duration. Moreover, mean period of ground motion also increases with magnitude, which implies longer wavelengths and higher spatial coherence of motion within the slide mass. As suggested by Yegian et al. (1991), the effects of duration and frequency content of the seismic event can be incorporated into the evaluation of permanent displacements and, accordingly, of the reduction factor η. This can be achieved using a statistical model that relates slope displacements d to the amplitude of the seismic shaking (a_{max}), the significant duration $D_{5\text{-}95}$ (measured as the time between 5-95% Arias intensity), the mean period of ground motion T_m and the acceleration ratio a_y/a_{max}.

Ausilio et al. (2007b) developed such a statistical model using a selection of accelerograms from database SISMA (Scasserra et al., 2008) to compute the Newmark-type displacements. A significant reduction in the scatter of data set was observed once the displacements d were normalised by the product $a_{max} \cdot T_m \cdot D_{5\text{-}95}$ and plotted against the ratio a_y/a_{max} (= k_y/k_{max}), as shown in Figure 4. Again, the computed results were best-fitted using an exponential relationship and assuming a log-normal distribution of data around the median value:

$$\log\left(\frac{d}{a_{\max} \cdot T_m \cdot D_{5-95}}\right) = -1.349 - 3.410 \cdot \frac{a_y}{a_{\max}} + \sigma \cdot t \tag{5}$$

where σ is the standard deviation (0.35 in $\log_{10}$ units) and t is the inverse normal standard distribution for a generic confidence level (t = 1.281 for the 90th-percentile).
Rigid soil behaviour is still assumed for the slide mass, this implying an uniform distribution of acceleration within the soil mass ($a_{max} = a_g$). The value of η can then be expressed as a function of limit slope displacement d_y and the site seismicity (a_g, $D_{5\text{-}95}$, T_m). Considering the 90th-percentile upper-bound relationship for non-dimensional displacements, an alternative expression for coefficient η was obtained by Ausilio et al. (2007c), defined as the ratio a_y/a_g:

$$\eta' = \frac{a_y}{a_g} = \frac{1}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{a_g \cdot E[T_m \cdot D_{5-95}]} \right) \right] + 0.237 \qquad (6)$$

in which an apex has been added to distinguish this definition from that given in eq. (4) and $E[T_m \cdot D_{5\text{-}95}]$ is the mean value of statistic distribution of the product $T_m \cdot D_{5\text{-}95}$, treated as a random variable.

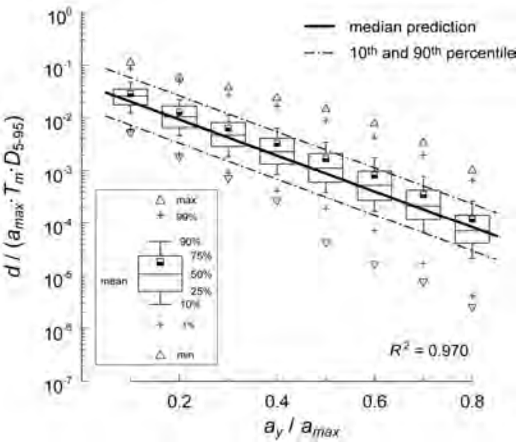

Figure 4. Non-dimensional permanent displacements versus the ratio a_y/a_{max} (Ausilio et al., 2007b).

In this case also, site effects can be accounted for in the evaluation of the seismic coefficient by introducing the amplification factors for subsoil profile S_S and for ground surface topography S_T:

$$k = \eta' \cdot k_{max} = \eta' \cdot \frac{a_{max}}{g} = \eta' \cdot S_S S_T \frac{a_g}{g} \qquad (7)$$

3 INFLUENCE OF THE DEFORMABILITY OF THE SLIDING MASS

In both procedures discussed above, a rigid soil behaviour was assumed for the slide mass, this implying uniform spatial distribution of acceleration within the slope. A more realistic description of seismic performance of slopes can be obtained taking into account the deformable behaviour of the slide mass during ground motion. To this purpose, slope behaviour can be studied using the de-coupled approach, in which a preliminary analysis is carried out to evaluate the dynamic response of the slope, and then the resulting acceleration time histories are used to compute the permanent displacements with a rigid block sliding analysis (Makdisi and Seed, 1978). The seismic response analyses can be carried out in one-dimensional (1D) or two-dimensional (2D) conditions, and the non-linear soil behaviour can often be described through the equivalent linear approximation, that is known to yield a reasonable estimate of soil response at moderate levels of seismic intensity. In a seismic response analysis, spatial variation of the seismic forces within the slope is implicitly taken into account, to a degree that depends on the accuracy of the 1D or 2D geometrical approximation of the slope. Then, in order to compute the permanent displacements with a rigid-block calculation, an equivalent (or average) accelerogram can be found that represents the overall response of the slide mass to earthquake loading (Seed and Martin 1966; Chopra, 1966). In this way, the dynamic response of the deformable soil deposit is considered, including the instantaneous distribution of the motion amplitudes within the soil mass.

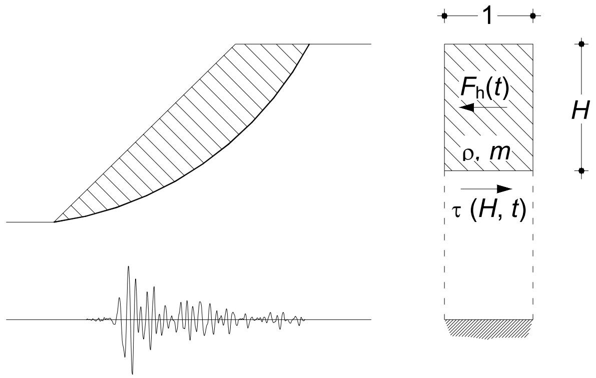

Figure 5. Evaluation of equivalent acceleration in one-dimensional conditions.

In a simplified 1D representation, the subsoil can be assimilated to a soil column with thickness H, as shown in Figure 5. The value of H can be assumed using different criteria: it can be taken as the slope height, as the average or maximum depth of the sliding surface along the slope, or even by considering an infinite slope with critical acceleration and fundamental period equal to those of the actual 2D geometry (Tropeano et al., 2008).
The equivalent acceleration time history acting on the slide mass is then obtained in the form:

$$a_{eq}(t) = \frac{F_h(t)}{m} = \frac{\tau(H,t)}{\rho \cdot H} = \frac{\tau(H,t)}{\sigma_v(H)} \cdot g \tag{8}$$

where ρ is the soil density, and $\tau(H, t)$ and σ_v are the shear stress and the total vertical stress acting at the base of the soil column, the latter assumed constant with time. Accordingly, the maximum acceleration a_{max} in eq. (2) should be intended as the maximum equivalent acceleration $a_{(eq)max}$ of the slide mass.
Following this approach, Ausilio et al. (2007a) carried out a parametric study on a set of soil columns constituted by different materials (gravel, sand and clay), with bedrock depths of 5 to 60 m and depths of sliding surfaces in the range 5 – 30 m. The subsoil models corresponded to 21 profiles of shear wave velocity, representative of the subsoil classes A to E, as identified by Eurocode 8 (EN 1998-5) and by Italian building code (D.M. 14.01.2008). The 1D

equivalent linear visco-elastic analyses were carried out selecting 124 Italian accelerograms from the database SISMA (Scasserra et al., 2008) as input motions.
Maximum values of acceleration a_s, computed at the ground surface by the seismic response analyses, were expressed by the product of the peak acceleration at the rigid outcrop a_g with non-linear amplification factors S_{NL} ($a_s = S_{NL} \cdot a_g$) associated to each subsoil class (Ausilio et al., 2007a). In Figure 6, the ratio $a_{eq(max)}/a_s$ is plotted against the ratio T_s/T_m, in which T_s is the fundamental period of the soil column, obtained from the transfer function from the sliding depth to the ground surface, and T_m is the mean period of the accelerogram, as defined by Rathje et al. (1998):

$$T_m = \frac{\sum C_i^2 \cdot \left(\frac{1}{f_i} \right)}{\sum C_i^2} \tag{9}$$

with C_i and f_i being the Fourier amplitudes and the corresponding frequencies, in the range 0.25-20 Hz.

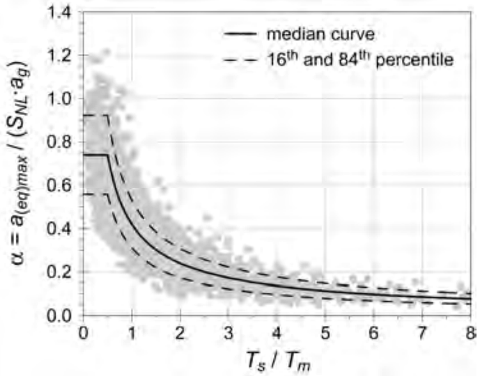

Figure 6. Ratio of $a_{(eq)max}/a_s$ versus the ratio between the fundamental period of slope and the mean period of input motion (Ausilio et al. 2007a).

The Authors recognised that the dependency of the ratio $\alpha = a_{eq(max)}/a_s$ on the period ratio T_s/T_m could be interpolated by a single relationship, irrespective of the subsoil profile:

$$\alpha = \frac{a_{(eq)max}}{a_s} = \frac{a_{(eq)max}}{a_g \cdot S_{NL}} = 0.4199 \cdot \left(\frac{T_s}{T_m} \right)^{-0.815} \tag{10}$$

Such expression holds for period ratios higher than 0.5, while for $T_s/T_m < 0.5$ it can be assumed $\alpha = 0.74$. The ratio between the wavelengths characteristic of the seismic event and the sliding depth decreases as the ratio T_s/T_m increase; as a consequence, the inertial forces in the slide mass reduce substantially, yielding equivalent accelerations lower than those computed at the ground surface. Then, values of $a_{(eq)max}/a_s$ lower than unity are due to vertical incoherence of ground motion, implicitly embedded into $a_{(eq)max}$.
Figure 6 shows that, for values of $T_s/T_m > 0.5$, the upper bound equivalent acceleration is always lower than the peak acceleration at the ground surface. Considering that most of Italian accelerograms provided by database SISMA are characterised by values of the mean period $T_m < 0.3$ s, a ratio $T_s/T_m > 0.5$ is obtained for depths of sliding surfaces greater than

about 10 m, assuming shear wave velocities V_S = 200 – 300 m/s for the soil. Then, shallow sliding mechanisms can be studied using the maximum acceleration at the ground surface, while deep mechanisms, once reduced to equivalent soil columns for seismic response analysis, will be characterised by significant reduction of ground motion amplitude due to substantial vertical incoherence of seismic shaking.

Based on the results of the analyses mentioned above, Ausilio et al. (2007c) included the influence of soil deformability in the estimate of the equivalent seismic coefficient. In this case the maximum acceleration a_{max} in eq. (2) is properly intended as the maximum equivalent acceleration $a_{(eq)max}$ of the slide mass, that is expressed in the form: $a_{(eq)max} = \alpha \cdot a_s = \alpha \cdot S_{NL} \cdot a_g$. To maintain a simplified and conservative character of the procedure, a constant value of $\alpha = 0.74$ can be assumed, irrespective of T_s/T_m. Under such an assumption, starting from eq. (5), a reduction factor η'' that accounts for both soil amplification and vertical incoherence of ground motion can be written in the form:

$$\eta'' = \frac{a_y}{a_g} = \frac{0.74 \cdot S_{NL}}{3.41} \cdot \left[-1.349 - \log\left(\frac{d_y}{0.74 \cdot S_{NL} \cdot a_g \cdot E[T_m \cdot D_{5-95}]} \right) \right] + 0.237 \qquad (11)$$

The following expression is then obtained for the equivalent seismic coefficient:

$$k = \eta'' \cdot k_{max} = \eta'' \cdot \frac{a_{max}}{g} = \eta'' \cdot S_T \frac{a_g}{g} \qquad (12)$$

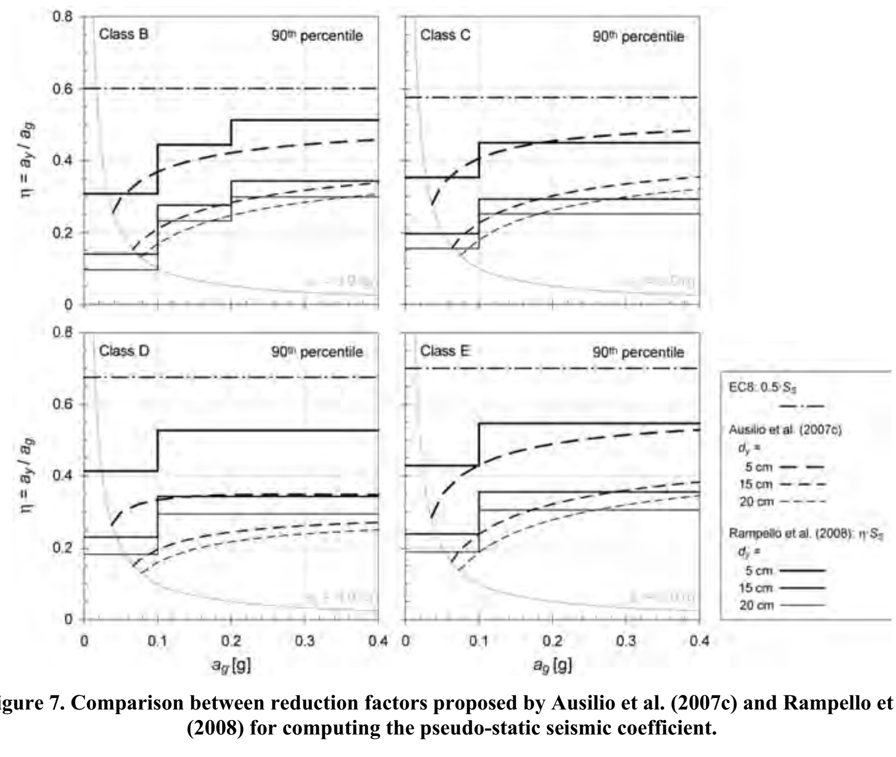

Figure 7. Comparison between reduction factors proposed by Ausilio et al. (2007c) and Rampello et al. (2008) for computing the pseudo-static seismic coefficient.

Figure 7 shows values of coefficient η'' computed using the median value of the dataset for the product $T_m \cdot D_{5\text{-}95} = 2.5\ s^2$ versus the peak acceleration at the rigid outcrop, a_g. The different plots refer to subsoil categories B, C, D and E, while different curves refer to limit displacement of 5, 15 and 20 cm.
The values of η'' are compared with the quantities $0.5 \cdot S_S$, recommended by Eurocode 8-5 (EN 1998-5), and the quantities $\eta \cdot S_S$, with values of η proposed by Rampello et al. (2008) and values of S_S specified by Eurocode 8 for different subsoil classes and for earthquake type 1 ($M_S > 5.5$). The values of $\eta \cdot S_S$ are characterised by a stepwise increase with a_g, since η was defined for given ranges of accelerations. For $a_g > 0.15\ g$ the curves describing η'' tend to about constant values, which are on the average 50% lower than those specified by EC8-5, for limit displacements d_y of the order of 15 cm. The lowest values of η'' pertain to category D, for which maximum reduction induced by soil deformability can be expected. The values of η'' proposed by Ausilio et al. (2007c) are in overall agreement with the corresponding estimates obtained by Rampello et al. (2008); these latter are increasingly higher at weak ground motion amplitude ($a_g < 0.2\ g$), due to the greater level of conservatism adopted in the procedure.

4 CONCLUSIONS

To assess the safety of slope behaviour during a seismic event, Eurocode 8-5 (EN 1998-5) suggests the use of either force-based or displacement-based procedures for common practice. In conventional displacement analysis, a rigid soil behaviour is assumed for the sliding mass, slope geometry and soil strength being described by the critical acceleration of the slope; this is typically assumed constant during earthquake loading, thus implying negligible excess pore water pressures and ductile soil behaviour. The analysis can be improved accounting for soil deformability and for progressive reduction of shear strength. Influence of soil deformability can be taken into account evaluating the equivalent accelerogram acting in the soil mass, via 1D or 2D seismic response analyses, while effects of shear strength reduction can be simply included in the analyses relying on the strength parameters attained at large strains.
In displacement-based analyses, both seismic demand and slope performance are defined by kinematic variables: the design action should be represented by the equivalent accelerogram, that describes the overall response of the slide mass to seismic shaking, while the slope behaviour is indexed by earthquake-induced displacements, to be compared with given threshold values associated to the design limit states. No limit displacements are currently specified by Eurocode 8 (EN 1998-5), nor are they expected to be included in the National Annexes.
In force-based approaches, an equivalent seismic coefficient k is used in a conventional pseudo-static stability analysis to define the seismic demand E, which should not trespass the sliding strength capacity R to let the safety requirement satisfied at the limit states. Slope stability is not satisfied for $R < E$, while it is satisfied for $R > E$. Eurocode 8-5 prescribes the use of a conventional 50% reduction of peak acceleration at ground surface to account for space and time variability of earthquake loading, whatever the size and the deformability of the sliding mass. If the seismic performance of a slope is represented by limit displacements d_y, the pseudo-static approach can withhold its validity, provided that the equivalent static action accounts for the capacity of the slope to undergo specified levels of displacements and for the effects of soil deformability.
Following this path, a parametric application of the displacement method has been used in this work to evaluate the equivalent seismic coefficient k as a function of tolerable slope

displacements and ground motion parameters. The reduction factors of the peak ground accelerations were computed by referring to limit displacements of 5 to 20 cm, which can pertain to serviceability limit states and result conservative for ultimate state design. The hypothesis of both rigid and deformable sliding mass leaded to less conservative estimates of the equivalent acceleration than what specified by Eurocode 8-5. Also, the higher the ratio between the wavelengths characteristic of the seismic event and the sliding depth, the lower the reduction factor. This implies that equivalent static actions on deformable sliding masses can be significantly reduced if the frequency content of the design earthquake and the dynamic response of the soil are taken into account. Therefore, use of seismic parameters, such as the mean period and the significant duration, as well as adequate description of soil behaviour should be introduced in common practice for reliable evaluation of seismic slope stability.

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SOIL-PILE KINEMATIC INTERACTION: NEW PERSPECTIVES FOR EC8 IMPROVEMENT

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ABSTRACT
Kinematic interaction between soil and structure originates from the incompatibility of the seismic free-field motion and the displacements of a more rigid embedded foundation. Foundation piles will actually experience deformations not only due to the loads directly transmitted by the superstructure, but also due to the passage of the seismic waves through the surrounding soil. In particular, bending moments may reveal quite important and, under certain circumstances, should be taken into account in the design, as prescribed by recent seismic codes (EN 1998-5; D.M. 14.1.2008). In this paper, the main results of a theoretical study are illustrated and some preliminary elements to be considered for seismic building codes are suggested.

KEYWORDS
Piles, kinematic interaction, code, simplified approach, bending moment.

1 INTRODUCTION

During strong earthquakes foundation piles tend to significantly modify soil deformations, since they oppose to the seismic motion of the ground. The interplay between soil and structure makes the motion at the base of the superstructure to deviate from the free-field motion, and the piles to be subjected to additional bending, axial and shearing stresses. The bending moments, usually referred to as "kinematic" ones, may result somewhat important even in the absence of the superstructure.

The kinematic interaction between soil and piles has been studied by many researchers (Fan et al., 1991; Gazetas et al., 1992; Kaynia and Mahzooni, 1996; Poulos and Tabesh, 1996; Mylonakis, 2001; Nikolaou et al., 2001; Saitoh, 2005; Cairo and Dente, 2007; Sica et al., 2007; Simonelli and Sica, 2008; Maiorano et al., 2009, and others). In spite of the big effort on such a topic, kinematic interaction is rarely accounted for in practical design. Modern seismic codes have, however, acknowledged the importance of kinematic interaction and demand piles to be designed also accounting for soil deformations arising from the passage of seismic waves. Two main issues should be also addressed by a building code:

- when has kinematic interaction to be considered (or, conversely, when can it be neglected)?;
- how has kinematic interaction to be analysed?

At present, the evidence collected is far from providing a definitive answer to the former question, but it is adequate to indicate what has still to be done for this issue. Eurocode 8 (EN 1998-5) suggests that kinematic effects should be taken into account when all the following conditions simultaneously exist: 1) seismicity of the area is moderate or high (specifying that moderate or high-seismicity areas are characterized by a peak ground acceleration a_gS>0.1g, where a_g is the design ground acceleration on type A subsoil and S is the soil factor); 2) subsoil type is D or worse, characterized by sharply different shear moduli between consecutive layers; 3) the importance of the superstructure is of III or IV class (e.g. schools, hospitals, fire stations, power plants, etc). The recent Italian building code (D.M. 14.1.2008) provides quite similar indications concerning the kinematic bending moments in piles.
The topic of this paper is to illustrate the key aspects of pile kinematic interaction and to individuate some preliminary elements to be considered for technical codes.

2 METHODS OF ANALYSIS

2.1 Overview of existing methods

Available methods for the analysis of kinematic soil-pile interaction may be classified into three groups: numerical approaches (FEM, BEM), Winkler methods (BDWF), simplified formulations. The finite element method (Wu and Finn, 1997; Cai et al., 2000; Kimura and Zhang, 2000; Maheshwari et al., 2004) provides a powerful and versatile technique, since some important effects such as soil nonlinearity and heterogeneity may be directly accounted for. Nevertheless, this method is generally very expensive from a computational viewpoint, since it requires suitable boundaries conditions being introduced to simulate the radiation damping effect. In such a context, a more attractive approach is represented by the boundary element technique (Kaynia and Kausel, 1982; Mamoon and Banerjee, 1990; Cairo and Dente, 2007). It only needs the discretization of the interfaces and permits the condition of wave propagation towards infinity to be automatically satisfied. This technique is generally formulated in the frequency domain and, in principle, is valid only under the assumption of material linear behaviour.
The methods based on the Winkler foundation model (Novak, 1974; Flores-Berrones and Whitman, 1982; Kavvadas and Gazetas, 1993) prove quite accurate and computationally time saving. They allow nonlinear behaviour of the soil to be easily incorporated if solution is envisaged in the time domain (Boulanger et al., 1999; El Naggar et al., 2005; Maheshwari and Watanabe, 2006; Cairo et al., 2008). According to this method, the pile is modelled as a linearly elastic beam, with length L and diameter d, discretized into segments connected to the surrounding soil by springs and dashpots, which provide the interaction forces in the lateral direction (Figure 1).
As a first approximation, the spring stiffness k may be considered to be frequency-independent and expressed as a multiple of the local soil Young's modulus E_s (Kavvadas and Gazetas, 1993). The dashpot coefficient c represents both material and radiation damping. The latter one may be computed using the analogy with one-dimensional wave propagation in an elastic prismatic rod of semi-infinite extent (Gazetas and Dobry, 1984).
For harmonic vertically propagating S-waves, the governing differential equation of the pile response is:

$$E_p I_p \frac{\partial^4 u_p}{\partial z^4} + m_p \frac{\partial^2 u_p}{\partial t^2} = (k + i\omega c)(u_{ff} - u_p) \tag{1}$$

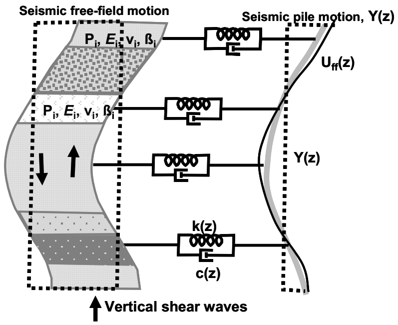

Figure 1. Beam on Dynamic Winkler Foundation model (BDWF).

where E_pI_p is the pile flexural rigidity, m_p the pile mass per unit length, u_p is the amplitude of the pile lateral displacement, u_{ff} the free-field soil displacement, ω the circular frequency of the motion, z is the depth, t the time and $i=(-1)^{0.5}$. Pile response in the time domain is then attained through standard Fourier transformations. Some results obtained using the Winkler approach (BDWF), as proposed by Mylonakis et al. (1997), will be presented in a subsequent section of the paper.

2.2 Approximate methods

Closed-form expressions (Dobry and O'Rourke, 1983; Nikolaou and Gazetas, 1997; Mylonakis, 2001; Nikolaou et al., 2001) are available in literature for approximately computing the maximum steady-state bending moment at the interface between two layers. These approaches have been derived by modelling the pile as a beam on a Winkler foundation and are based on the following simplified assumptions: each soil layer is homogeneous, isotropic and linearly elastic; the soil is subjected to a uniform static shear stress field; the pile behaves as a linear-elastic semi-infinite beam; the embedded length of the pile in each layer is greater than the so-called "active length". This latter is usually expressed by the equation (Randolph, 1981):

$$L_a = 1.5d\left(\frac{E_p}{E_s}\right)^{0.25} \tag{2}$$

The simplified design procedure proposed by Dobry and O'Rourke (1983) permits to compute the bending moment at the interface between two layers as:

$$M = 1.86(E_p\, I_p)^{3/4}(G_1)^{1/4}\,\gamma_1\, F \tag{3}$$

where G_1 and γ_1 are the shear modulus and shear strain in the upper soil layer, respectively; F is a function of the ratio $c=(G_2/G_1)^{0.25}$, with G_2 being the shear modulus of the lower layer, and expressed by:

$$F = \frac{(1-c^{-4})(1+c^3)}{(1+c)(c^{-1}+1+c+c^2)} \tag{4}$$

An improvement of the Dobry and O'Rourke (1983) solution has been achieved by Mylonakis (2001). Using more suitable dynamic parameters, the following expression has been derived:

$$M = 2\frac{E_p I_p}{d}\left(\frac{\varepsilon_p}{\gamma_1}\right)\gamma_1 \tag{5}$$

in which ε_p indicates the pile peak bending strain and γ_1 denotes the soil shear strain at the layer interface. The ratio of these parameters represents a sort of "strain transmissibility" function, which is strongly frequency-dependent. If this aspect is neglected, the strain transmissibility function takes into account only pile-soil interaction effects and is expressed as:

$$\left(\frac{\varepsilon_p}{\gamma_1}\right)_{\omega=0} = \frac{c^2 - c + 1}{2c^4}\left(\frac{H_1}{d}\right)^{-1}\left\{\left[4.7\left(\frac{E_1}{E_p}\right)^{9/32}\frac{H_1}{d} - 1\right]c(c-1) - 1\right\} \tag{6}$$

where H_1 and E_1 are the thickness and the Young's modulus of the upper layer, respectively. The effect of frequency may be introduced in terms of the ratio:

$$\Phi = \left(\frac{\varepsilon_p}{\gamma_1}\right)\Big/\left(\frac{\varepsilon_p}{\gamma_1}\right)_{\omega=0} \tag{7}$$

which is a function of H_1/d, G_1/G_2, E_p/E_1. Nevertheless, in the range of frequencies of interest Φ is generally less than 1.25.

A fitted formula has been proposed by Nikolaou et al. (2001) for harmonic excitation. It is based on the maximum shear stress τ_c induced at the layer interface by the free-field motion. The resulting expression is:

$$M = 0.042\,\tau_c\, d^3\left(\frac{L}{d}\right)^{0.30}\left(\frac{E_p}{E_1}\right)^{0.65}\left(\frac{V_{s2}}{V_{s1}}\right)^{0.50} \tag{8}$$

where V_{s1} and V_{s2} are the shear wave velocities in the upper and lower layer, respectively.

In order to account for the transient nature of the seismic excitation, the authors have introduced a reduction factor depending on the duration of the accelerograms in terms of the effective number N_c of cycles in the record, the relative frequency characteristics between earthquake and soil deposit, the effective damping of the soil-pile system. Two simplified expressions are given:

$$\eta = \begin{cases} 0.04N_c + 0.23 & \text{for } T_1 \approx T_p \\ 0.015N_c + 0.17 \approx 0.2 & \text{for } T_1 \neq T_p \end{cases} \tag{9}$$

where T_p represents the predominant period of the ground motion and T_1 the fundamental period of the deposit.

A crucial issue in the use of these approximate approaches is the evaluation of the uniform soil shear strain γ_1 in the upper layer and the maximum shear stress τ_c at the interface. As suggested by Mylonakis (2001), if the seismic peak acceleration $a_{\max s}$ is specified at the soil surface, γ_1 can be computed using the expression suggested by Seed and Idriss (1982) for liquefaction problems:

$$\gamma_1 = (1 - 0.015 H_1) \frac{\rho_1 H_1}{G_1} a_{\max s} \tag{10}$$

where, in addition to the soil parameters already defined, the mass density of the upper layer ρ_1 has been introduced. Equivalently, the maximum shear stress τ_c at the interface may be roughly estimated as (Nikolaou and Gazetas, 1997; Nikolaou et al., 2001):

$$\tau_c = a_{\max s} \rho_1 H_1 \tag{11}$$

In the next section, the accuracy of these simplified approaches will be investigated.

3 MAIN RESULTS

Several analyses have been performed referring to a fixed-head pile with length L=20 m, diameter d=0.6 m, Young's modulus E_p=2.5·10^7 kN/m², and mass density ρ_p=2.5 Mg/m³ (Figure 2). The pile is embedded in a two-layered subsoil which is 30 m thick and underlined by a stiffer bedrock. Soil shear stiffness contrast has been changed as a function of the shear wave velocities V_{s1} and V_{s2} of the two layers, in order to reproduce a subsoil of type D or C (D.M. 14.1.2008). Poisson's ratio and mass density of the soil are: ν_s=0.4, ρ_s=1.9 Mg/m³. The shear wave velocity of the rock is 1200 m/s. The analyses have been performed by adopting 18 Italian accelerograms (Table 1) scaled in amplitude to provide a rock peak acceleration consistent with the seismic zone considered. These records are provided by the database SISMA (Scasserra et al., 2008). In Table 1, the frequency content of the input motion is quantified through the predominant period T_p, corresponding to the maximum spectral acceleration in an acceleration response spectrum (computed for 5% viscous damping) and through the mean period, T_m, as defined by Rathje et al. (1998) on the basis of the Fourier spectrum of the signal. Actually, T_m should provide a better indication of the frequency content of the recordings because it averages the spectrum over the whole period range of amplification.

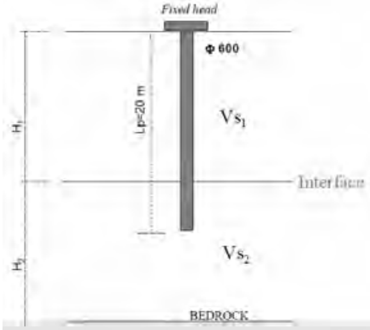

Figure 2. Reference scheme adopted in the analyses.

The main results of the whole parametric study, described elsewhere (Sica et al., 2007; Simonelli and Sica, 2008), are reported in the following. It is worth noting that the behaviour of the soil-pile system is assumed to be linearly elastic.

Table 1. Seismic records provided by the database SISMA and used in the analyses.

Label	Station name	Earthquake	Comp.	Date (d/m/y)	T_p (s)	T_m (s)
A-TMZ270	Tolmezzo-Diga Ambiesta	Friuli	EW	6.5.1976	0.64	0.500
A-TMZ000	Tolmezzo-Diga Ambiesta	Friuli	NS	6.5.1976	0.26	0.395
A-STU270	Sturno	Campano Lucano	EW	23.11.1980	0.20	0.845
A-STU000	Sturno	Campano Lucano	NS	23.11.1980	0.38	0.661
A-AAL018	Assisi-Stallone	Umbria Marche	NS	26.9.1997	0.32	0.333
E-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	6.10.1997	0.12	0.172
E.NCB000	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	NS	6.10.1997	0.14	0.165
R-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	3.4.1998	0.18	0.180
J-BCT000	Borgo-Cerreto Torre	Umbria Marche (aftershock)	NS	14.10.1997	0.10	0.167
J-BCT090	Borgo-Cerreto Torre	Umbria Marche (aftershock)	EW	14.10.1997	0.16	0.208
E-AAL018	Assisi-Stallone	Umbria Marche (aftershock)	EW	6.10.1997	0.22	0.242
B-BCT000	Borgo-Cerreto Torre	Umbria Marche	NS	26.9.1997	0.08	0.154
B-BCT090	Borgo-Cerreto Torre	Umbria Marche	EW	26.9.1997	0.12	0.198
TRT000	Tarcento	Friuli (aftershock)	NS	11.9.1976	0.10	0.215
C-NCB000	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	NS	3.10.1997	0.04	0.128
C-NCB090	Nocera Umbra-Biscontini	Umbria Marche (aftershock)	EW	3.10.1997	0.12	0.154
R-NC2090	Nocera Umbra 2	Umbria Marche (aftershock)	EW	3.4.1998	0.18	0.184
R-NC2000	Nocera Umbra 2	Umbria Marche (aftershock)	NS	3.4.1998	0.16	0.152

Figure 3 presents the envelopes of the maximum kinematic bending moments with depth, as computed in the time domain using the Winkler approach by Mylonakis et al. (1997), for each of the selected 18 accelerograms (scaled to the same peak ground acceleration of 0.35g). The thickness of the layers is assumed to be H_1=H_2=15 m; the shear wave velocities of the upper and the lower layers are V_{s1}=100 m/s and V_{s2}=400 m/s, respectively. On the basis of the average shear wave velocity $V_{s,30}$=160 m/s, the soil profile under consideration can be classified as type D. The damping ratio of the soil is β_s=0.10.

Three grey zones are also displayed corresponding to the range of reinforced concrete pile yielding moments for typical reinforcements of the cross section (8ϕ16, 24ϕ12 and 12ϕ30) and magnitude of the axial force in the pile. For each reinforcement the lower limit of the grey zone represents the cross section yielding moment corresponding to zero axial force while the higher one to an axial force equal to 1200 kN.

From the results of this study, the following remarks may be made:

- the maximum kinematic bending moment generally occurs at the soil layer interface;
- at the soil layer interface, the kinematic bending moment dramatically increases when the shear wave velocity contrast between the bottom and top layer Vs_2/Vs_1 increases from 2 to 4, for subsoil types D and C;
- for subsoil profiles corresponding to class D, the computed kinematic bending moments may be well above the assumed yielding moments of the pile cross section.

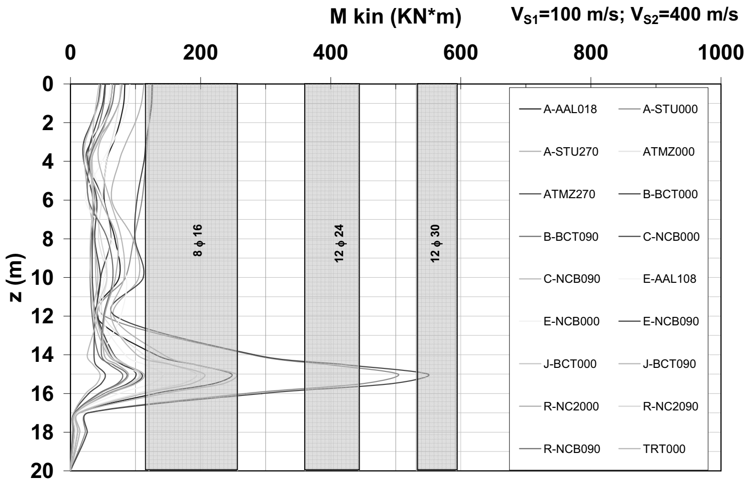

Figure 3. Kinematic bending moments for two-layer soil profile of type D and with H_1=H_2=15 m.

3.1 *Effect of the frequency content of the earthquake*

As a further interpretation of the results previously shown, the ratio between the period T_{input} of the input motion (in terms of T_p and T_m) and the fundamental period T_{soil} of the subsoil in hand (in the specific case, T_{soil}=0.62 s) is presented in Figure 4, for all the considered seismic records. It is worth noting that the accelerograms characterized by values of T_p/T_{soil} or T_m/T_{soil} close to unity provide the higher kinematic moments in Figure 3, due to the occurrence of a resonance phenomenon. This confirms what pointed out by Nikolaou et al. (2001) by frequency domain analyses: maximum effects of kinematic bending in piles occur at the fundamental period of the subsoil. Conversely, all accelerograms having T_p/T_{soil} or T_m/T_{soil} below 0.5, induce lower kinematic moments in the pile.

These results suggest that a “critical band” of the ratio T_{input}/T_{soil} in which kinematic effects could be important, may be individuated (Figure 4). This observation could be a possible criterion to select significant records from a database of local seismic events. Therefore, if the acceleration time-histories are provided for a given seismic zone (in addition to the peak ground acceleration), the “susceptibility of the site with the associated waveforms” to induce significant kinematic bending in piles may be established on the basis of the following criterion:

- if the ratio T_{input}/T_{soil} is external to the critical band, site susceptibility is low, and only inertial interaction should be accounted for;
- if the ratio T_{input}/T_{soil} is internal to the critical band, site susceptibility is high, and kinematic interaction should be analysed in addition to inertial interaction.

In the latter case, the analysis tool should be consistent to the one adopted in the design of the overall structure.

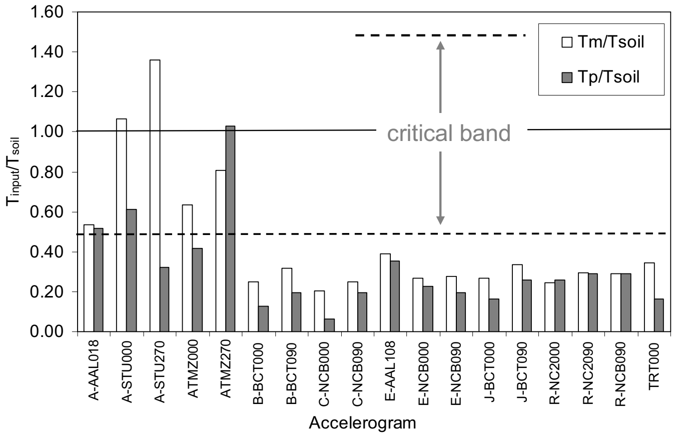

Figure 4. Ratio among the predominant period of the input motion and the fundamental period of the soil.

3.2 Kinematic moments computed with simplified methods

The simplified methods provided by Dobry and O'Rourke (1983), Mylonakis (2001), and Nikolaou et al. (2001) have been applied to compute the kinematic moments at the interface of the above specified two-layered subsoil, in order to compare their predictions to those obtained numerically by the BDWF approach of Mylonakis et al. (1997).

The formulas by Dobry and O'Rourke (1983) and Mylonakis (2001) have been applied by adopting both the shear strain at the bottom of the first layer γ_1 provided by Eq. (10) (Figure 5a) and the value of γ_1 directly computed with SHAKE (Schnabel et al., 1972) or EERA (Bardet et al., 2000) for each selected accelerogram (Figure 5b). At the same way, the equation derived by Nikolaou et al. (2001) has been applied by adopting both the shear stress at the interface using Eq. (11) as suggested by the authors (Figura 5a), and the value directly provided by EERA (Figura 5b). The formula of Nikolaou et al. (2001) has been applied without introducing any corrective factor η.

From Figure 5a and 5b, it emerges that if literature closed-form solutions - Dobry and O'Rourke (1983), Mylonakis (2001), and Nikolaou et al. (2001) - are adopted with the values of γ_1 and τ_c provided by the simplified approaches, all formulas overestimate the kinematic moments with respect to the values computed numerically by BDWF analyses. Conversely, if γ_1 or τ_c are derived from a free-field analysis, carried out with SHAKE or EERA, the kinematic moments provided by the literature formulas are quite close to those computed numerically (Figure 5b). Similar results have been obtained also by Maiorano et al. (2009).

In short, it seems well-established that it is more suitable to apply the literature formulas for estimating pile kinematic moment at the interface in combination with free-field analyses (with SHAKE or EERA) in order to get the proper value of γ_1 or τ_c, especially when the interface is quite deep. As well-known, in such case the formula provided by Seed & Idriss (1982) for computing γ_1 (or, equivalently, τ_c) is no longer reliable.

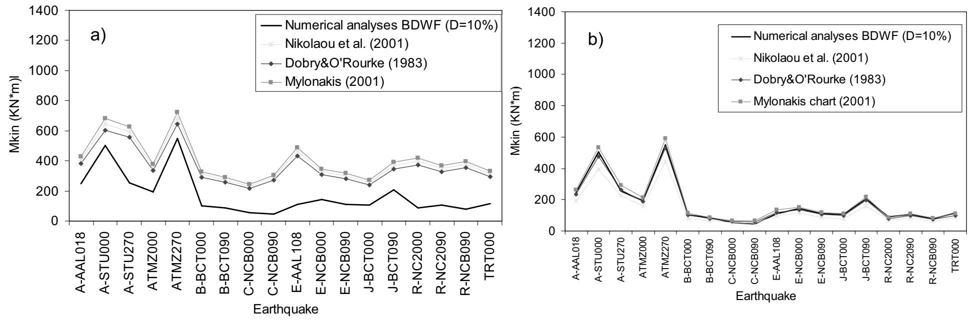

Figure 5. Kinematic moment at the interface computed with approximate formulas and the BDWF model.

3.3 An alternative simplified approach

Recently, Cairo et al. (2009) have suggested an alternative procedure that may be promptly used in current practice. This procedure (that is fully analytical) is very simple to use and needs only two parameters for defining the seismic motion: the peak ground acceleration and the mean period of the excitation. The former is directly provided by the code; the latter may be assumed on the basis of some suitable regression equations available in the literature (Rathje et al., 2004; Ausilio et al., 2007). In the proposed procedure, Eq. (8) by Nikolaou et al. (2001) has been considered the most suitable, and a "corrective" factor δ has been introduced. This latter is defined as the ratio of the maximum pile bending moment in the time domain, calculated using the BEM approach developed by Cairo and Dente (2007), to the kinematic moment obtained by Eq. (8), in which the peak ground acceleration is computed by site response analyses. The results of Figure 6 show the factor δ, calculated with reference to the case-studies previously documented, versus the ratio T_1/T_m of the fundamental period of the soil deposit to the mean period of the seismic excitation. A linear relationship between δ and T_1/T_m can be obtained with reference to the 95th percentile of the distribution. This line is described by the following expression:

$$\delta = 1.31 - 0.20\frac{T_1}{T_m} \tag{12}$$

This relation may be considered to evaluate the expected maximum bending moment. As a first approximation (Mylonakis, 2001) the fundamental period of the deposit may be evaluated by the equation:

$$T_1 = \frac{4H_1}{V_{s1}} \tag{13}$$

where H_1 and V_{s1} are the first layer thickness and shear wave velocity, respectively. Therefore, the maximum pile bending moment M_{kin} at the interface between two soil layers may be computed by:

$$M_{kin} = \delta \cdot M \tag{14}$$

with δ and M provided by Eq. (12) and Eq. (8), respectively.

As an example, this procedure has been applied to a case-study proposed by Nikolaou and Gazetas (1997) and recalled by Cairo and Dente (2007). A concrete pile is embedded 9.5 m into a top layer of soft clay and 6 m into a deep layer of dense sand. The shear wave velocities for the upper and lower layers are 80 and 330 m/s, respectively. The soil deposit is 30 m thick and rests on a rigid bedrock. The pile has E_p=25 GPa, d=1.3m, L=15.5 m, ρ_p=2.5 Mg/m^3. Two actual accelerograms, scaled to 0.10g peak acceleration, have been used as excitation at the rock level. In Figure 7 the results obtained by Eq. (14) are compared with the envelopes of the peak moments computed by Nikolaou and Gazetas (1997) and by Cairo and Dente (2007) using different methods. As can be seen, the agreement between the results is quite satisfactory.

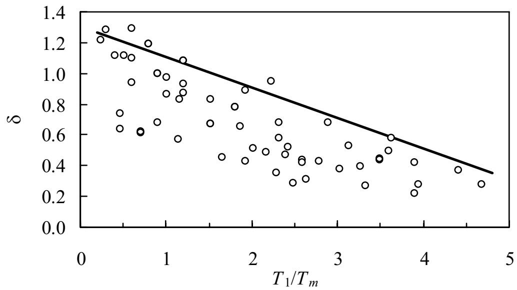

Figure 6. Corrective factor δ as a function of the ratio of the fundamental period T_1 of the soil deposit to the mean period T_m of the earthquakes.

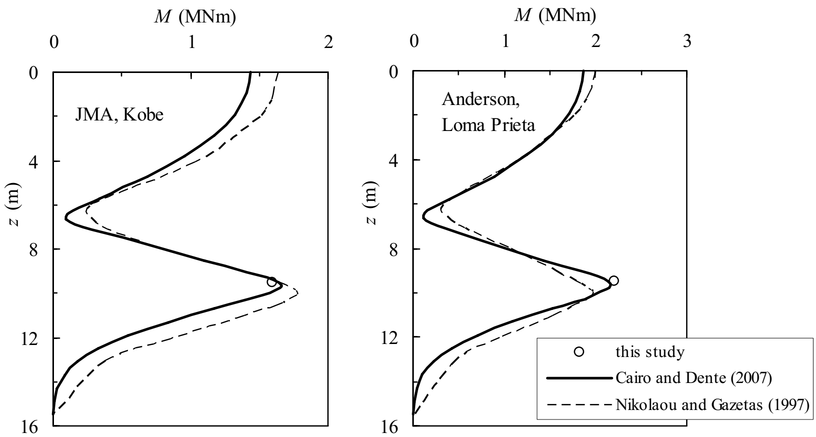

Figure 7. Envelopes of time-domain moments in a pile and peak kinematic moments at the interface.

4 CONCLUSIONS

In the paper some of the results from a comprehensive parametric study performed on a single fixed-head pile, embedded in two-layered soil deposits, have been illustrated. The analyses have been carried out using the Winkler-type model provided by Mylonakis et al. (1997), the

BEM formulation developed by Cairo and Dente (2007), and some simplified closed-form expressions available in the specific literature for evaluating, in an approximate manner, the maximum kinematic moment in the pile at the interface between two soil layers. Italian seismic records have been used as input motion for the numerical analyses.

From the obtained results, the following conclusions may be drawn:

- in high seismicity zones and especially for subsoil type D, kinematic bending moments may be very high depending on the stiffness contrast between consecutive layers;
- the maximum effects of kinematic pile bending generally occur for input motions having the fundamental period close to that of the subsoil;
- the simplified methods here examined (Dobry and O'Rourke, 1983; Mylonakis, 2001; Nikolaou et al., 2001) tend to predict conservative moments at the subsoil interface, especially when the interface is deep. In such case it is better their use in combination with free-field response analyses to get predictions closer to the ones obtained from more rigorous approaches.

On the basis of these observations, a simple criterion has been suggested to roughly estimate site susceptibility to induce significant kinematic bending in piles, and a simplified procedure for crudely evaluating kinematic moment has been presented. Yet, caution is required before proposing suggestions to be incorporated into seismic building codes. The theoretical evidence derived from the analysis needs to be extended to a wider selection of schemes, and to be compared with further experimental evidence.

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PERFORMANCE-BASED DESIGN OF GRAVITY RETAINING WALLS UNDER SEISMIC ACTIONS

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ABSTRACT
This paper is devoted to the performance-based design of retaining walls under seismic actions. EC8 takes into account this kind of approach, both encouraging the utilization of dynamic analyses, and proposing reduction factor *r* in the simple pseudostatic method, whose value depends on the amount of "displacement" tolerable by the structure. In the recent Italian Building Code (2008) a better calibration of the seismic coefficients for the pseudostatic approach has been produced, based on the results of specific displacement analyses. According to the results obtained, it would be necessary to recalibrate the seismic coefficient now proposed in EC8. In this paper a further improvement in the design procedure of retaining walls is proposed, still based on the performance evaluation, which more effectively takes into account the principle of the "capacity design", widely applied in structural design.

KEYWORDS
Retaining walls, PBD, simplified dynamic analysis, pseudostatic design, capacity design.

1 INTRODUCTION

This paper deals with the performance-based design of retaining walls under seismic actions.
In the first part of the paper the present version of Eurocode 8, and specifically EC8-Part5, is illustrated. The Code takes into account the performance criteria, both encouraging the utilization of dynamic analyses which allow to forecast the behaviour of the wall under real excitations, and proposing reduction factor *r* in the simple pseudostatic method, whose value depends on the amount of "displacement" tolerable by the structure.
In the second part of the paper, after a discussion on the effects of the application of EC8 design rules for retaining wall, the recent studies performed in Italy by the Associazione Geotecnica Italiana (AGI) on these topics are briefly referred to.
Then the recent Italian Building Code (Norme Tecniche per le Costruzioni, D.M. 14/01/2008, alias NTC2008) is illustrated in detail, since it derives from EC8 and could be practically considered as a first national application of Eurocodes, although with some significant improvements. As regards retaining walls, it propose a calibration of the seismic coefficient for the pseudostatic approach, based on the results of specific displacement analyses.
In the last part of the paper, a further improvement in the design procedure of retaining walls is proposed, still based on the performance evaluation, which more effectively takes into account the principle of the "capacity design", widely applied in structural design.

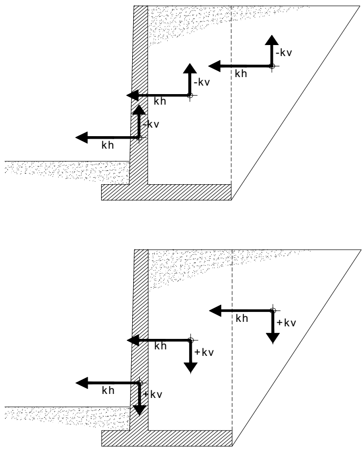

Figure 1. EC8: scheme of the pseudostatic seismic coefficients acting in the horizontal and vertical directions (by Simonelli 2006).

2 EUROCODE 8: RETAINING WALL DESIGN

The design of retaining walls is dealt with in Ch. 7 of Eurocode8-Part 5 (*EN1998-5, FINAL DRAFT*, December 2003, alias EC8-5); a detailed discussion on the whole chapter is illustrated in Simonelli (2003). At the beginning of ch. 7.1 "General requirements", clause (2) it is stated that "Permanent displacements, in the form of combined sliding and tilting, the latter due to irreversible deformation of the foundation soil, may be acceptable if it is shown that they are compatible with functional and/or aesthetic requirements". This concept is very important, and will be taken into account later in the evaluation of the pseudo-static seismic action on the structure. The methods of analysis are dealt with in Ch. 7.3, and it is stated that: "Any established method based on the procedures of structural and soil dynamics, and supported by experience and observations, is in principle acceptable for assessing the safety of an earth retaining structure" (Ch. 7.3.1, clause (1)P). After this foreword, particular attention is devoted to the pseudo-static analysis, regarded as the main simplified method (Ch. 7.3.2).

2.1 Simplified methods: pseudo-static analysis

The pseudo-static method is based on the well-known theory of Mononobe (1929) and Okabe (1926). Pseudo-static seismic actions both in the horizontal and vertical directions are taken into account (Figure 1). As for the vertical action, this may act both upwards and downwards. The total design thrust E_d, which is the thrust affected by the partial safety factors (see *EN 1997-1 – Geotechnical Design*, 2003, alias EC7; Aversa and Squeglia, 2003; Scarpelli, 2003; Frank, 2005), is given in *Annex E*, points *E3* and *E4* (see Figure 2):

$$E_d = 0.5 \cdot \gamma \cdot (1 \pm k_v) \cdot K \cdot H^2 + E_{ws} + E_{wd} \tag{1}$$

where γ =soil unity weight; k_v=vertical seismic coefficient; H=wall height; E_{ws}=static water force; E_{wd}=hydrodynamic water force; K=earth pressure coefficient (static + dynamic).

Thrust: $E_d = \frac{1}{2}\,\gamma^{*}\,(1 \pm k_v)\,K \cdot H^2 + E_{ws} + E_{wd}$

Active thrust coefficient (static + dynamic):

$$K = \frac{sin^2(\psi + \phi'_d - \theta)}{\cos\theta\, sin^2\psi\, sin(\psi - \theta - \delta_d)\left[1 + \sqrt{\frac{sin(\phi'_d + \delta_d)\,sin(\phi'_d - \beta - \theta)}{sin(\psi - \theta - \delta_d)\,sin(\psi + \beta)}}\right]^2}$$

Per $\beta \le \varphi'_d - \theta$

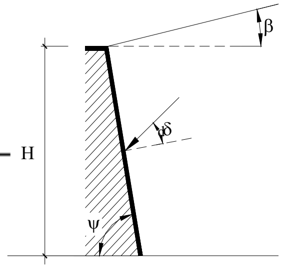

Figure 2. EC8: pseudostatic active thrust of the soil in seismic conditions.

The vertical seismic coefficient k_v is a function of the horizontal one k_h *:*

$$k_v = \pm 0.33 \cdot k_h \quad or \quad k_v = \pm 0.5 \cdot k_h \tag{2}$$

depending on the ratio between the vertical and horizontal design accelerations (a_{vg} and a_g), being respectively lower or larger than 0,6 (see *EC8-5, ch. 7.3.2.2, clause (4)P*).
The horizontal seismic coefficient k_h is:

$$k_h = a_{gR} \cdot \gamma_I \cdot S / (g \cdot r) \tag{3}$$

where γ_I = importance factor of the structure; r=factor that depends on the allowable wall displacements (in the Final Draft of EC8-5 the formula is $k_h = \alpha \cdot S / r$, where $\alpha = (a_{gR}/g) \cdot \gamma_I$). The seismic coefficient shall be taken as being constant along the height, for walls not higher than 10 m.
The values to be adopted for the factor r are listed in Table 1 (which is a copy of the EC8-5 Table 7.1). In brief the factor should be taken equal to 1 for structures that substantially cannot accept any displacement, while it assumes 1.5 and 2 values as the acceptable displacement increases. The threshold values of the displacement d_r are proportional to the peak ground acceleration ($\alpha \cdot S$) expected at the site.

Table 1. EC8: *r* factor values for the evaluation of the pseudostatic seismic coefficient.

Type of retaining structure	*r*
Free gravity walls that can accept a displacement up to $d_r = 300\ \alpha\, S$ (mm)	2
Free gravity walls that can accept a displacement up to $d_r = 200\ \alpha\, S$ (mm)	1.5
Flexural reinforced concrete walls, anchored or braced walls, reinforced concrete walls founded on vertical piles, restrained basement walls and bridge abutments	1

Nevertheless, in the Authors' opinion, it is not very clear if the threshold values d_r are the upper or the lower limit values for the acceptable displacement. In particular, if d_r identifies the upper limit values, then Table 2 should be read as shown in Figure 3a, but the doubt still remains on the r value to be adopted for walls that can accept a displacement greater than 300 $\alpha \cdot S$ (mm); probably even for these walls a factor r value equal to 2 should be adopted (and in Fig 3a the line $d_r = 300\ \alpha \cdot S$ should be eliminated). On the other hand, if d_r identifies the lower

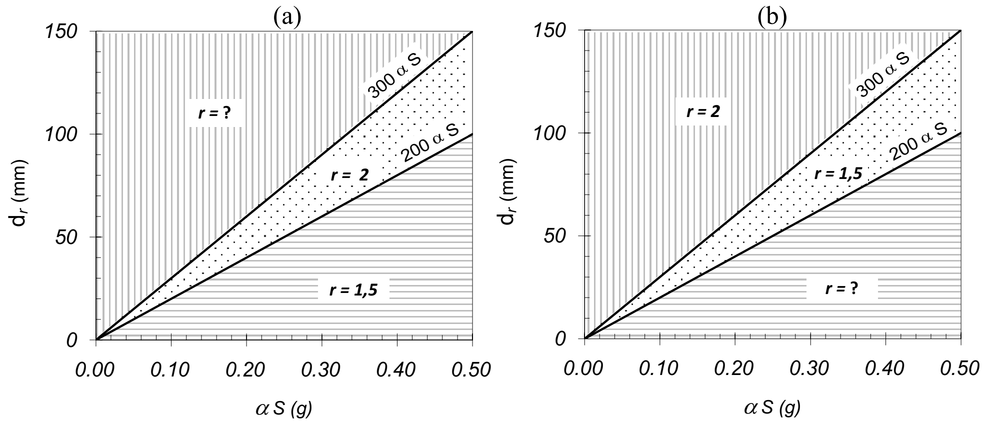

Figure 3. Graphic interpretations of the correlation among the *r* factor, the acceptable displacement d_r (*free gravity wall*) and the peak ground acceleration, listed in Table 1 (by Simonelli, 2006).

limit values for the acceptable displacement, then Table 2 should be read as shown in Figure 3b, but the doubt still remains on the *r* value to be adopted for walls that can accept a displacement lower than 200 $\alpha \cdot S$ (mm); moreover, in this case, the threshold condition d_r = 300 $\alpha \cdot S$ (mm) for applying r=2 would be quite severe, implying very large acceptable displacement values for the walls.

In summary, the intensity of the pseudostatic force depends on the value of the ground surface acceleration $a_g \cdot S$ and on the amount of allowable displacement of the wall (by means of the factor r).
As regards the point of application of the force due to the dynamic earth pressures, it must be taken at mid-height of the wall, in the absence of a more detailed study taking into account the relative stiffness, the type of movements and the relative mass of the retaining structure. Only for walls which are free to rotate about their toe, the dynamic force may be taken to act at the same point as the static force. As regards the inclination of the thrust on the wall due to the static and the dynamic action, it can not be taken greater than $(2/3)\varphi'$ for the active state, and must be taken equal to zero for the passive state.
The equation of the active earth pressure coefficient K is given in Figure 2, where the symbols are φ'_d = design value of the soil friction angle; δ_d = design value of the wall-soil friction angle; θ = inclination of the mass forces acting on the soil wedge.
For dry soil θ is given by the equation:

$$tan\ \vartheta = \frac{k_h}{1 \mp k_v} \tag{4}$$

For saturated soils the expression of θ changes for the two cases of low and high permeability soil under dynamic actions (see *Annex E*), and proper values of E_{ws} and E_{wd} must be taken into account; it is worthwhile to underline that in any case the soil strength is always computed in drained conditions.
Once the design action E_d has been determined, the wall must be verified against the sliding and bearing capacity failures: in both cases, E_d must be lower or equal to the design resistance R_d, which is the resistance affected by the partial safety factors:

$$R_d \geq E_d \tag{5}$$

2.2 Dynamic analysis

In the code, as recalled before, it is stated that any established method based on the procedures of structural and soil dynamics, and supported by experience and observations, is in principle acceptable for assessing the safety of an earth retaining structure. Obviously such procedures imply the definition of time-history seismic input motion.

In particular, according to clause (1)P of Ch. 2.2 of EC8-5, both artificial accelerograms and real strong motion recordings may be used; their peak values and frequency contents have to be in agreement with the rules specified in EC8-1, Ch. 3.2.3.1. It is worthwhile to recall that, if recorded accelerograms are utilised, the samples used must be adequately qualified with regard to the seismogenetic features of the sources and to the soil conditions appropriate to the site, and their values must be scaled to the value of the ground surface acceleration ($a_g{\cdot}S$) for the zone under consideration (Ch. 3.2.3.1.3, clause (1) P).

Returning to EC8-5, clause (2) of Ch. 2.2, it is stated that "in verifications of dynamic stability involving calculations of permanent ground deformations, the excitation should preferably consist of accelerograms recorded on soil sites in real earthquakes, as they possess realistic low frequency content and proper time correlation between horizontal and vertical components of motion." Moreover it is stated that the strong motion durations have to be selected consistently with EC8-1, Ch. 3.2.3.1; mainly the duration has to be consistent "with the magnitude and the other relevant features of the seismic event underlying the establishment of a_g" (Ch. 3.2.3.1.2, clause (2)P).

3 RETAINING WALL DISPLACEMENT ANALYSIS

The analysis of the performance of a retaining structure subjected to real dynamic actions has been studied by means of simplified approaches, which assume *a priori* a prevailing kinematism of the wall (generally tilting or sliding). Taking into account the observed behaviour of walls under seismic actions, many researcher have adopted as prevailing kinematism the sliding of the wall along its foundation base. Most of the methods for assessing the permanent horizontal displacements induced by the dynamic excitations are based on the original sliding block model proposed by Newmark (1965).

As well known, the Newmark model analyzes the sliding of a rigid block on a plane surface, assuming a rigid-plastic behaviour at the interface between them. From simple limit equilibrium considerations, the threshold acceleration a_t value can be evaluated, over which the surface moves with an acceleration higher than the block, which instead still moves according to the threshold acceleration. The displacement between the block and the surface can be computed by integrating the relative accelerations twice, until the velocity between them returns to zero again (see Figures 4 and 5, where the threshold acceleration is indicated as $a_t=N{\cdot}g$, where N is the threshold acceleration coefficient).

Newmark model has been subsequently upgraded by Zarrabi (1979), for a more effective application to retaining structures. Zarrabi model takes into account the congruency among the displacements of the backfill, the soil wedge and the wall (Figure 4): as a consequence, the threshold acceleration value is not constant, but varies with the amplitude of the input acceleration.

The effectiveness of this model has been validated by proper tests on the shaking table apparatus at the dynamic laboratory of the University of Bristol (Crewe et al., 1998 and Simonelli et al., 2000). For example, in Figure 6 the final comparison between analytical and measured permanent displacements induced on a proptotype wall (N=0,234) by Tolmezzo input motion (Friuli 1976 earthquake) scaled at increasing maximum acceleration values is illustrated.
Further, Zarrabi model allows to take account of both the horizontal and the vertical components of the seismic input motion, which according to recent studies have to be considered acting together in proper seismic analyses.

4 NTC2008: RETAINING WALL DESIGN

After the earthquake that struck Molise region in 2002, a new hazard map of the Italian territory was set out according to the EC8 criteria, providing the peak ground acceleration expected on a horizontal rock site for an earthquake having a return period of 475 years (see the Italian Seismic Code attached to the Ordinanza del Consiglio dei Ministri 3274 of March 2003, alias OPCM3274). In the same Code the site classification (A to E subsoil categories) for taking into account the local soil amplification was also introduced, and the EC8 design criteria for structural and geotechnical systems were adopted.

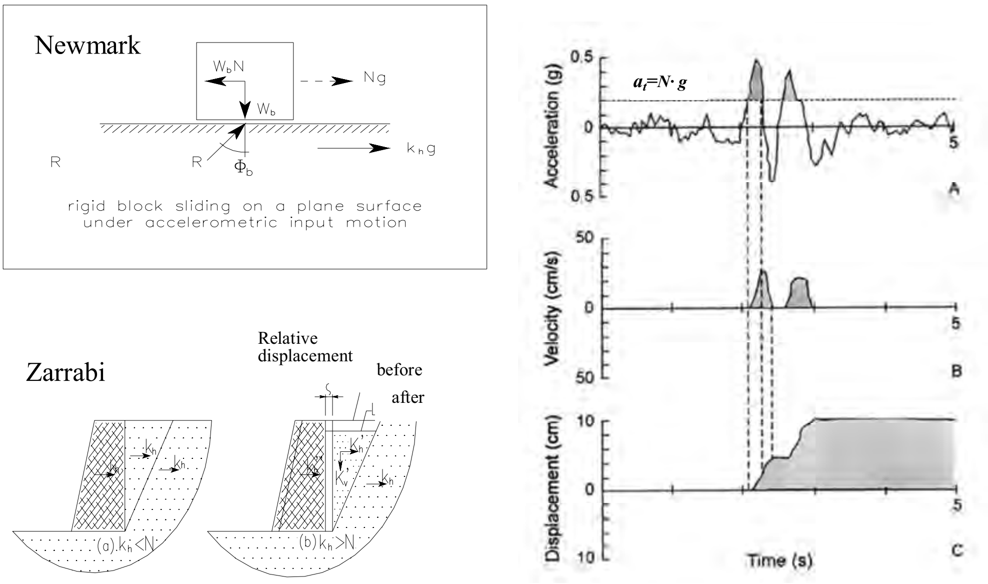

Figure 4. Newmark (1965) and Zarrabi (1979) models for displacement analysis.

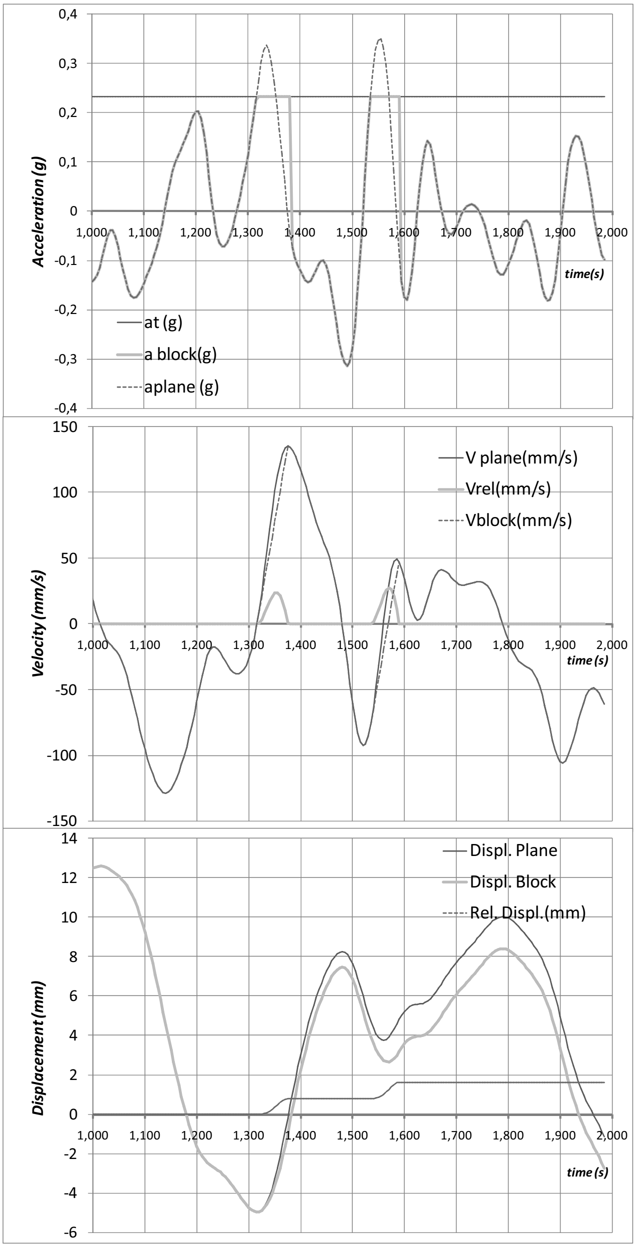

Figure 5. Example of displacement analysis results: displacements induced by the NS component of San Rocco accelerogram (1976 Friuli earthquake) on a wall having the threshold acceleration a_t=0,234g.

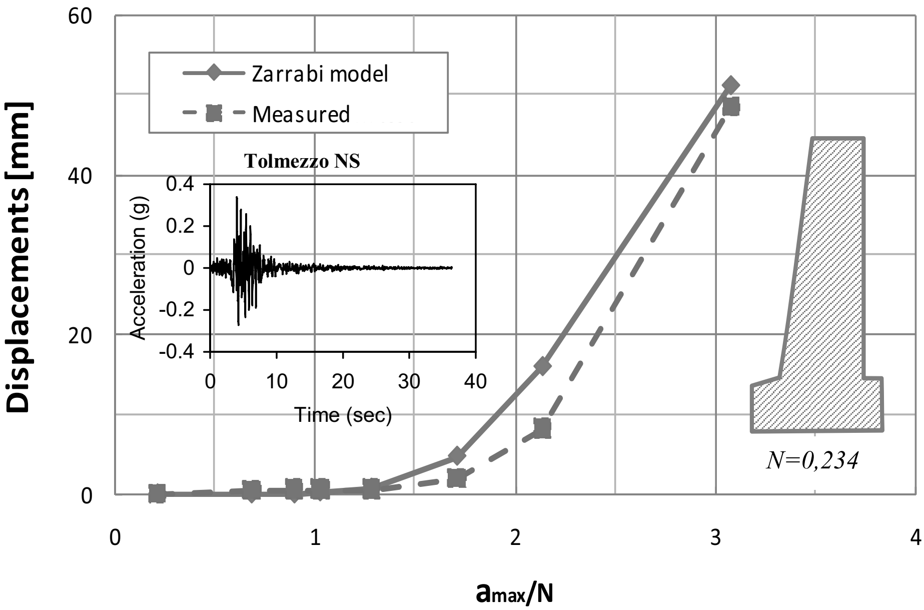

Figure 6. Shaking table tests on a prototype wall: comparison between computed and measured displacements (Crewe et al., 1998).

In the geotechnical field, the application of pseudostatic methods according to OPCM3274, coupled with the accelerations values to be adopted for the Ultimate Limit State design, showed to provide unusual and very conservative results for several systems, and in particular for retaining structures (Simonelli, 2003). As a confirmation of that, the application of more advanced dynamic analysis procedures according to the same Code indications, gave rise to reliable results, much less conservative than those provided by pseudo-static approaches (Simonelli, 2006; Callisto and Aversa, 2008) . The comparison between the two approaches put in evidence the role of the adopted pseudostatic coefficients, whose values, correlated to the peak ground acceleration, showed to be too severe and not representative of the effects induced by real seismic motion. As a matter of fact, the problem of the proper correlation between the seismic coefficient and the parameters characterising the seismic waveforms had already been studied in the past for retaining walls and slopes, with reference to Italian accelerograms recorded during Friuli 1976, Irpinia 1980 and Umbria-Marche 1998 earthquakes, and it came out that is not rational and hence quite difficult to simply correlate the coefficient to the peak ground acceleration value, especially when even small displacements can be accepted at the end of the seismic motion (Simonelli and Viggiani, 1992; Simonelli, 1993 and 1994; Simonelli and Fortunato 1996).
In the years following the emanation of OPCM3274 Code, a group of experts of the Associazione Geotecnica Italiana (AGI) have intensely worked on those topics, and first published guidelines on geotechnical seismic design (AGI; 2005), then wrote the geotechnical parts of the new national building code "Norme Tecniche per le Costruzioni" (alias NTC2008, recently published by the D.M. 14.1.2008).

As regards retaining walls (NTC2008, Ch. 7.11.6), the Italian Code still refer to EC8, stating that both dynamic, simplified dynamic and pseudostatic methods can be applied; then some more detailed indications for the pseudostatic approach are provided. The main difference with respect to EC8 is the new evaluation of the seismic coefficients for pseudostatic actions. The horizontal seismic coefficient k_h is determined as a fraction β_m of the dimensionless peak ground acceleration a_{max} :

$$k_h = \beta_m \cdot a_{max} / g \tag{6}$$

while the vertical seismic coefficient k_v is simply :

$$k_v = \pm 0.5 \cdot k_h \tag{7}$$

The peak ground acceleration a_{max} is :

$$a_{max} = S \cdot a_g = S_S \cdot S_T \; a_g \tag{8}$$

where S is the coefficient that takes into account both the stratigraphic (S_S) and the topographic (S_T) amplification, and a_g is the reference peak ground acceleration, acting on category A subsoil (it is worthwhile noting that S_S is equivalent to the coefficient S of EC8).
As a matter of fact, the peak ground acceleration a_{max} is practically equivalent to the acceleration ($a_{gR} \cdot \gamma_I \cdot S$) of equ.(3). In fact in EC8 the acceleration a_{gR} refers always to an earthquake having a return period equal to 475 years, while the importance of the structure is explicitly represented by the importance factor γ_I. In NTC2008 the factor γ_I disappears, and the importance of the structure is taken into account both in the reference lifetime of the structure and in the “coefficiente d’uso” (utilization coefficient) C_u, that give rise to different values of the return period of the earthquake, and of the correspondent maximum ground acceleration a_g. Another minor difference between EC8 and NTC is that the latter takes explicitly account of the topographic amplification S_T.

As previously said, the main difference between EC8 and NTC2008 is in the factor multiplying the peak ground acceleration for retaining wall that can tolerate displacements:
- in EC8 the factor is equal to $1/r$, and since r can be equal to 1.5 or 2 depending on the amount of the tolerable displacement, the factor $1/r$ can assume the values of 0.67 or 0.5;
- in NTC2008 the values of β_m does not depend on the amount of the displacement (provided that the wall can allow and tolerate displacements), but on the level of the expected reference peak ground acceleration (on category A subsoil) and on the subsoil classification (A or B÷E categories), according to Table 2; the values are much lower than EC8 ones.

Table 2. NTC2008: β_m factor values for the evaluation of the horizontal seismic coefficient.

β_m	Ground type	
	A	B, C, D, E
$0.2 < a_g(g) \le 0.4$	*0.31*	*0.31*
$0.1 < a_g(g) \le 0.2$	*0.29*	*0.24*
$a_g(g) \le 0.1$	*0.20*	*0.18*

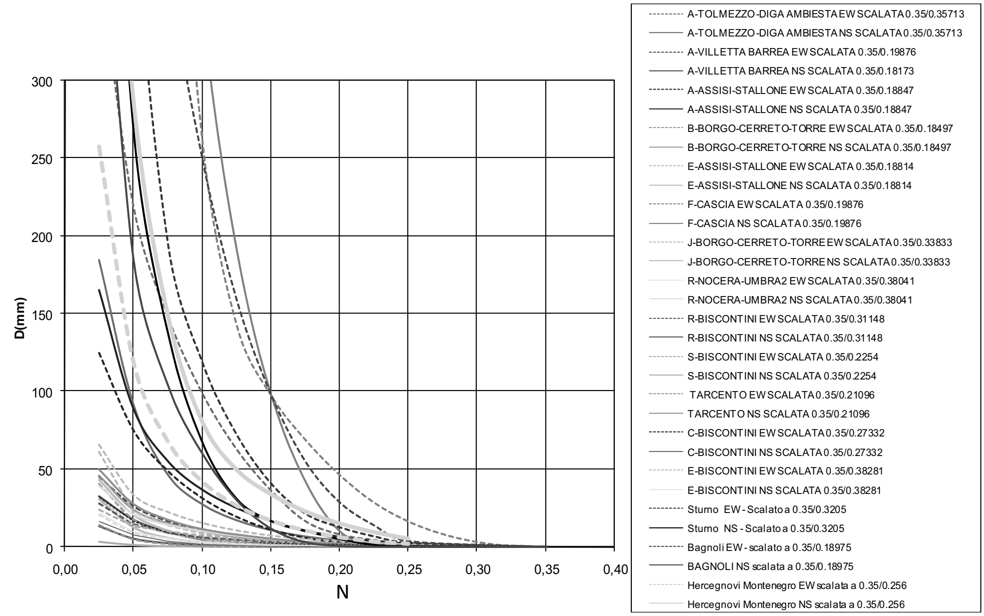

Figure 7. Example of displacement analysis results: displacements induced by accelerograms on subsoil A, scaled at a_{max}=0.35g.

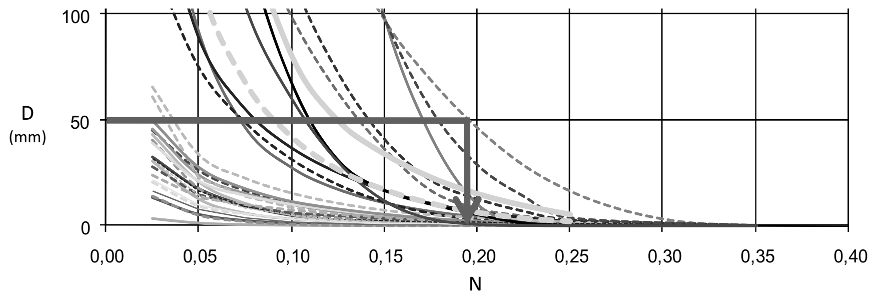

Figure 8. N_{SLI}=0,19 for D=50 mm.

The values of β_m factors in NTC2008 have been determined on the basis of the results of an extensive displacement analysis, having assumed that the critical mechanism under strong earthquakes is the sliding one. All the accelerograms of the Italian SISMA database (Scasserra et al., 2008), integrated with some other international data, have been utilized as input motion. In Figure 7, for example, the displacements produced by the accelerograms recorded on subsoil A and scaled at the 0.35g peak ground value are plotted vs. the wall threshold acceleration coefficient N.

The values of β_m have been calibrated by pseudostatic back-analyses against sliding, in order to produce walls that under ULS conditions suffer displacements of the order of a few cm (5 cm maximum for N=N_{SLI}, see Figure 8). Obviously the further verification of the bearing capacity can produce walls whose displacements could be even lower.

5 SUGGESTIONS FOR EC8

On the basis of the experience on retaining wall design related to NTC2008 Code, a first suggestion for further developments of EC8 would be to find out a better calibration of the seismic coefficient, both in terms of the already proposed r factor, or in terms of β_m factor, in order to avoid that the application of the present EC8 in other European Countries with high seismic hazard could produce large over-conservative and unacceptable design (as happened in Italy after OPCM3274).

The new seismic coefficients could be proposed according two alternative strategies:

1. the coefficients could be explicitly provided inside EC8 document, after a calibration performed utilizing a wide European accelerometric database (which should contain SISMA); in the Authors' opinion, the obtained values could result higher but not very different from those proposed in NTC2008;
2. the EC8 coefficients of step 1 could be proposed as bracketed values, and their proper determination could be demanded to the National Annex of each European Country, which could perform calibration based on more representative regional seismic database.

At present the 1st step appears to be the more suitable, and it could be complementary with more specific studies already performed at national level (as for NTC2008). The 2nd step, on the other hand, will probably be more reliable in the future, when more recorded data and representative seismic database will be available in any Country.

A second improvement regards the performance based design philosophy itself, which could effectively include the principle of "capacity design" even for 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 the input motion data set, then should imply pseudostatic analysis for verifying the other collapse mechanisms.

In detail, the steps of the procedure are the following:

1. individuation of the allowable displacement of the wall D_{ALL}(as a function of the restrain conditions and the tolerable displacements for the whole structure);
2. analysis of the displacements induced by the selected accelerometric input motions, and individuation of the threshold acceleration a_t=N_{SLI}·g of the wall corresponding to D_{ALL} (as in Figure 8);
3. among the group of walls having N=N_{SLI}, selection of those walls which verify the following conditions:
   - bearing capacity global safety factor PSF > 1 (say 1.1) (9)
   - overturning global safety factor PSF > 1 (say 1.1) (10)

   for pseudostatic actions corresponding to the seismic coefficient:

$$k_h = N_{\mathrm{SLI}} \tag{11}$$

Equ. (9) and (10), combined with the adoption of equ. (11), guarantee that the potential kinematism of the selected walls under high seismic actions will be the sliding one. In fact, as the maximum acceleration attains the value a_t=N_{SLI}·g, the wall will start sliding, and the thrust will not be able to exceed the corresponding pseudostatic action, for which the bearing capacity and overturning result to be verified (equ. (9) and (10)).

With the suggested procedure, the capacity design principle is effectively implemented, since the sliding kinematisms, which is much more ductile and not catastrophic as the other two, is the preferred mechanism of the wall, that on the other hand will not exceed the maximum tolerable displacement D_{ALL}.

6 CONCLUSIONS

In this paper the performance-based design of retaining walls under seismic actions has been dealt with, first reviewing the present version of Eurocode 8, then examining the indications of the recent Italian Building Code (D.M. 14/01/2008, alias NTC2008).
Both EC8 and NTC2008, which practically derives from the Eurocode, take into account the performance based design criteria. In fact the codes encourage the utilization of dynamic analyses which allow to forecast the behaviour of the wall under real excitations; further they propose simple pseudostatic method, whose seismic coefficients depend on the amount of "displacement" tolerable by the structure. Nevertheless the application of EC8 rules to the Italian territory, coupled with the peak ground accelerations expected for high return period earthquakes (SLU conditions) showed to produce over-conservative design.
On the other hand, in the recent NTC2008 a better calibration of the seismic coefficients for computing pseudostatic actions has been produced, on the basis of parametric displacement analyses performed adopting as input motion the Italian accelerometric database SISMA. These results suggest that in the next future a similar procedure should be implemented in EC8 too, with the aim to improve the effectiveness of the suggested pseudostatic methods.
In this last part of the paper a simplified dynamic design procedure is proposed, still based on the performance evaluation, which more effectively takes into account the principle of the "capacity design" for retaining wall. According to the design procedure the sliding phenomenon, which is the more ductile and not catastrophic kinematism, becomes the potential mechanism under severe seismic motion, preventing the wall from bearing capacity and overturning collapse.

7 ACKNOWLEDGEMENTS

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PERFORMANCE-BASED DESIGN OF EMBEDDED RETAINING WALLS SUBJECTED TO SEISMIC LOADING

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ABSTRACT
In common practice, the seismic design of an embedded retaining wall is carried out using the pseudo-static method. In this approach, constant forces are introduced in a limit equilibrium calculation, and the seismic analysis of a retaining wall is treated similarly to the evaluation of the safety against a collapse mechanism. This paper is aimed to propose a reconsideration of the simple pseudo-static calculation: it shows that the method can be used within the context of the performance-based design to predict the actual seismic performance of the wall, and that concepts employed in the capacity design of structural members can be extended to the design of embedded retaining walls. The paper also points to possible code prescriptions that may provide guidance for the correct application of the pseudo-static method to the design of retaining walls.

KEYWORDS
Seismic design, retaining walls, numerical analysis, nonlinear response, earthquake resistant structures.

1 INTRODUCTION

In the performance-based design of a retaining wall, different degrees of seismic protection can be prescribed, depending on the limit state being analysed. For instance, the current Italian Construction Code (Decreto Ministeriale 14.1.2008) defines four different limit states, each associated to a different seismic action, characterised by a given probability of exceedance in the structure's lifetime.
For a building, the seismic performance is usually expressed by the maximum instantaneous displacement (e.g. the inter-storey drift) that occurs during the earthquake, and by the ductility demand associated to this displacement. On the other hand, the seismic performance of a retaining wall is more commonly expressed in terms of permanent displacements at the end of the earthquake (Richards & Elms 1979, PIANC 2001). A possible reason of this difference is that, unlike buildings, retaining structures undergo permanent displacements in one direction only, and therefore the displacements increase monotononically, attaining a maximum at the end of the seismic event.
In order for the displacements to be irreversible, they must stem from the instantaneous development of a plastic mechanism; therefore these displacements are associated to quasi-rigid body movements. For an embedded retaining wall, if one admits that only the soil strength can be fully mobilised during the seismic event, while the retaining wall and the

restraining system remain in an elastic state, then a plastic mechanism can form only if the wall is cantilevered or singly restrained (propped or anchored). The present discussion is largely devoted to the design of these wall categories. It is also assumed that, because of the limited height of these wall types, the dynamic motion of the soil interacting with the excavation is essentially synchronous. Some effects of asynchronicity on the design of the retaining structures for deep excavations are discussed by Callisto & Aversa (2008).

2 EVALUATION OF THE SEISMIC PERFORMANCE

Upon instantaneous attainment of the available strength, cantilever and singly-restrained walls can undergo rigid-body movements in a way which is qualitatively similar to the behaviour of gravity retaining walls. This was shown, for instance, by the experiments of Richards & Elms (1992) and Neelankatan et al. (1992), and by the results of some dynamic tests that were recently carried out using the Cambridge Dynamic Centrifuge (Viggiani & Conti 2008): for accelerations larger than a critical value, a progressive development of wall rotations was observed. Figure 1 is taken from the results of a series of dynamic numerical analyses carried out by Callisto et al. (2008): it shows the instantaneous distribution of the contact stresses σ_h exerted by the soil against a pair of mutually propped retaining walls during the strong motion phases of two different seismic events. During the earthquake, these stresses increase both at the rear and in front of the walls, and permanent displacements occur as a consequence of full mobilization of the soil strength. This phenomenon occurs in an alternate fashion to the two facing walls: in Figure 1 it is happening to the left-hand wall. Figure 2 shows, for the two walls, the progressive accumulation of the computed horizontal displacement of the toe relative to the top.

For a given retaining wall, a critical value a_c for the horizontal acceleration can be evaluated by performing iteratively a limit equilibrium analysis, and finding the pseudo-static acceleration for which soil strength is fully mobilised. Permanent displacements can then be assumed to result from a Newmark-type integration of the relative motion, and are bound to decrease as a_c increases. For a given soil and a given excavation height H, the value of a_c depends essentially on the embedded length d. Therefore, the embedded length (or, equivalently, the total length $L = H + d$) should be chosen on the basis of the maximum displacement allowed for the seismic event (and therefore the limit state) under consideration.

Relationships between the permanent displacements u and the ratio a_c/a_{max} were recently derived by Rampello & Callisto (2008) from a parametric integration of a database of Italian Strong Motion Accelerograms (SISMA, Scasserra et al. 2008). Each recording was scaled to different maximum accelerations a_{max}, with a scale factor not exceeding the value of 2. Figure 3 shows the u-a_c/a_{max} relationships obtained for accelerograms recorded on rock and scaled to $a_{max} = 0.35$ g, that yielded the largest displacements. Also shown in the figure are regression lines through the computed data points, of the form:

$$u = B \exp\left(A \frac{a_c}{a_{max}} \right); \tag{1}$$

specifically, the continuous line, computed with $A = -7.4$ and $B = 1.8$, is close to the upper bound of the results and was used to develop a relationship between the expected displacement (that define the requested seismic performance) and the pseudo-static horizontal acceleration that was then adopted by Italian Construction Code (Decreto Ministeriale 14.1.2008) for the seismic design of embedded retaining walls.

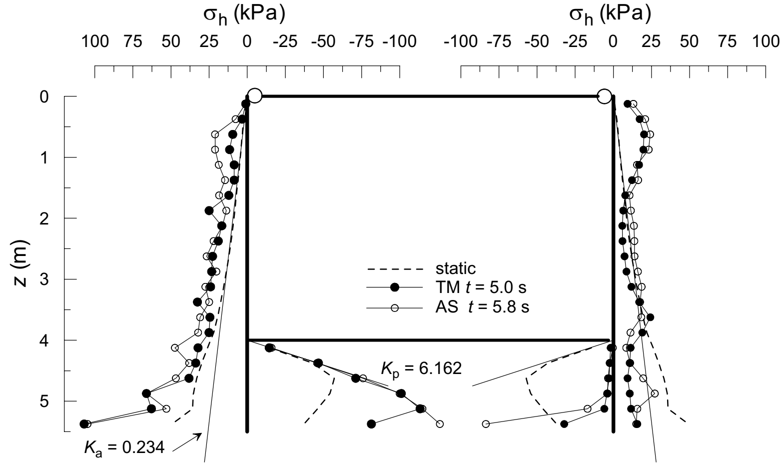

Figure 1. Contact stresses acting against a pair of propped retaining walls (Callisto et a. 2008).

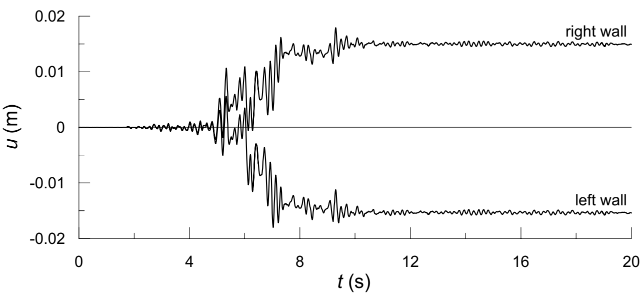

Figure 2. Time histories of the top-toe relative displacements of the retaining walls shown in Figure 1.

Consider the example of Figure 4(a), taken from Soccodato & Callisto (2009), where an excavation with $H = 4$ m in a homogenous coarse-grained soil is retained by a cantilevered wall. The soil has a constant angle of friction $\varphi = 35°$, while the soil-wall angle of friction is $\delta = 20°$. For this retaining wall, the critical acceleration a_c was found using the Blum (1931) limit equilibrium method; the Mononobe-Okabe solution was used for the seismic active pressure, while the coefficient of passive pressure was evaluated with the closed-form solution developed by Lancellotta (2007).

Figure 4(b) shows, for the case at hand, the computed values of the critical acceleration a_c plotted as a function of the total length of the wall. As L varies from 7 to 9 m, a_c increases from 0.15 to 0.4 g. Figure 4(b) also shows the permanent wall displacements u computed using equation (1) and assuming that the maximum horizontal acceleration in the soil interacting with the wall is either $a_{max} = 0.5$ g or $a_{max} = 0.75$ g: as the length of the wall increases, the permanent displacements u decrease rapidly.

Maximum bending moments were computed in the wall using the limit equilibrium method, with a pseudo-static horizontal acceleration equal to a_c: it can be expected that, as the accelerations increase during an earthquake, so do the soil-wall contact stresses, until full mobilisation of soil strength is attained, that is until the critical acceleration a_c is reached. For accelerations larger than a_c, contact stresses cannot vary significantly, since soil strength is

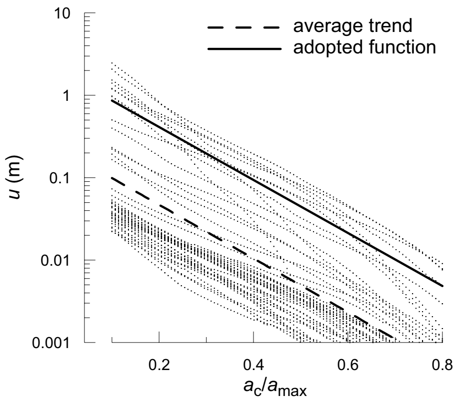

Figure 3. Relationships between permanent displacements and the ratio a_c/a_{max} obtained by Rampello & Callisto (2008) for very stiff soils and a_{max} = 0.35 g.

already fully mobilised, and the acceleration in excess of a_c is spent to produce a movement of the wall under quasi-constant contact stresses. Since the internal forces in the wall are a consequence of the contact stresses, bending moments should increase as the acceleration increases to up the critical value, and then remain constant while relative soil-wall displacements occur.

The maximum bending moments computed using the critical acceleration are indicated as $M(a_c)$ and are plotted in Figure 4(b) as a function of the wall length L. Since a_c increases with L, the bending moments increase as well. This means that if the wall is made longer in order to improve its seismic performance (that is, to undergo smaller displacements) it will have to sustain larger bending moments.

Of course, for a retaining wall with $a_c > a_{max}$ the permanent displacements are negligible and the internal forces in the wall are evaluated with a pseudo-static acceleration equal to a_{max}.

3 DESIGN CRITERIA

In its essential terms, the design of an embedded retaining wall with no more than one restrain could be performed through the following steps:

a. for a given limit state, define the required seismic performance by selecting the maximum permanent displacement u;
b. evaluate the maximum horizontal acceleration a_{max} expected for the limit state under consideration;
c. from a relationship of the type shown in Figure 2, evaluate the critical acceleration a_c needed to meet with the desired seismic performance;
d. search iteratively, through the limit equilibrium method, the wall length L that gives the required critical acceleration; if for any reason (e.g. for hydraulic needs) the chosen wall length is larger than L, the critical acceleration a_c must be recalculated using the actual length;
e. compute the internal forces using the contact stresses evaluated with $a = a_c$;
f. design the wall structure (and the eventual restraining system) on the basis of the internal forces evaluated at the previous step.

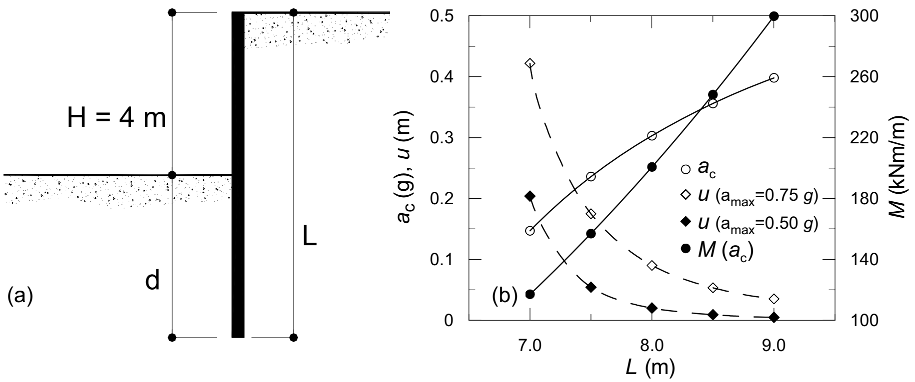

Figure 4. Layout of a cantilevered retaining wall in a coarse-grained soil (a) and plots of the critical acceleration, the permanent displacement and the maximum bending moment as a function of the wall length (b) (Soccodato & Callisto 2009).

Although the above sequence of activities is conceptually clear, some issues are believed to necessitate further investigation.

Firstly, the evaluation of the maximum horizontal acceleration a_{max} at step (b) poses some problems: in principle, a_{max} could be found using the simplified procedures based on amplification coefficients, such as those prescribed by the Eurocode 8 (EN 1998-5) or by the Italian Construction Code (Decreto Ministeriale 14.1.2008). Yet these procedures, being largely based one-dimensional site response analyses, neglect the effect of two-dimensional amplification and may underestimate a_{max}. For instance, a parametric study on cantilevered retaining walls, based on the results of two-dimensional dynamic numerical analyses (Callisto & Soccodato 2009) showed that the maximum acceleration in the soil interacting with the excavation may reach values larger than twice the corresponding maximum acceleration computed in a one-dimensional analysis, and that this effect is not significantly related to the soil or wall stiffness, but rather depends on the two-dimensional nature of wave propagation.

Step (c) implies the availability of relationships similar to the one shown in Figure 2, as specific as possible to the geographic region and to the source mechanisms under consideration.

Step (f) needs further discussion. It may be required that the wall structural strength should be larger than the internal forces evaluated with the pseudo-static method using the critical acceleration a_c. This should be considered equivalent to a common practice used for the capacity design of structural members: energy-dissipating elements of mechanisms are chosen, and other elements are provided with a sufficient reserve strength capacity, to ensure that the chosen energy dissipating mechanisms are maintained at their full strength throughout the deformations that may occur. In the case at hand, a natural choice for the energy dissipating element may be the soil interacting with the retaining wall, also considering that in the initial conditions the strength of significant soil volumes located in the vicinity of the wall is already fully mobilised.

However, a pseudo-static calculation of the bending moments in the walls with $a = a_c$ typically assumes a linear distribution of the contact stresses, while the actual distribution of the contact stresses may deviate from this simple distribution; this is visible in Figure 1, and is further substantiated by Figure 5, that shows the instantaneous distribution of σ_h computed for the cantilevered wall of Figure 4(a) during the development of a plastic mechanism (Soccodato & Callisto 2009), together with the corresponding bending moments and

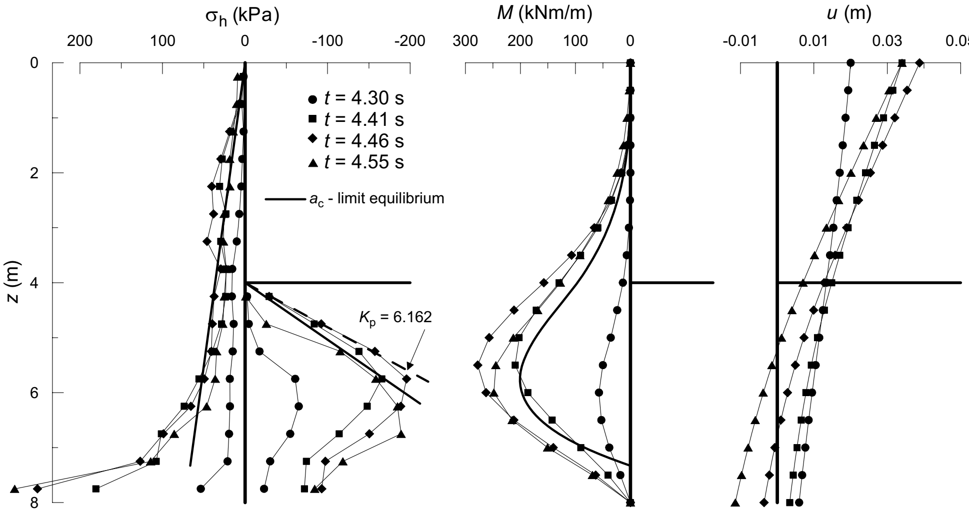

Figure 5. Contact stresses, bending moments and horizontal displacements of a cantilevered embedded retaining wall at several instants during a severe seismic event, compared with results of pseudo-static calculation in critical condition (Soccodato & Callisto 2009).

horizontal displacements. The thick lines in Figure 5 show the limit equilibrium computations with $a = a_c$. It can be seen that the actual instantaneous distribution of the contact stresses can be somewhat different from the simplified distribution, and this makes the maximum bending moments M_{max} larger than $M(a_c)$.

For the same cantilevered retaining wall in a coarse-grained soil, Callisto & Soccodato (2009) showed that the ratio of M_{max} to $M(a_c)$ increases with the stiffness of the wall, but is bounded by a maximum of about 1.6. Therefore, this figure could be adopted as a multiplier of $M(a_c)$ to account for the difference between the simplified and the actual distribution of contact stresses, at least for the particular case examined. In order to protect the retaining wall from bending yielding, a further over-strength factor might be required.

Soccodato & Callisto (2009) explored a different approach, in which no multipliers or over-strength factors were used, and the walls were given a bending strength M_y about equal to $M(a_c)$, allowing both the soil and the retaining walls to undergo plastic yielding during the earthquake. Hence, in this approach yielding of the wall is called to compensate for the inaccuracies of the assumed distribution of the contact stresses. The Authors carried out a series of dynamic numerical analyses, in which two different seismic records were applied to pairs of cantilevered retaining walls with $H = 4$ m and three different embedment depths. The mechanical behaviour of the walls was described with a linearly elastic-perfectly plastic moment-curvature relationship, in which, for the three different walls, the curvature at yielding ψ_y was about constant. The soil behaviour was described by a non-linear hysteretic constitutive model coupled with a Mohr-Coulomb failure criterion (Callisto & Soccodato 2007). Figure 6, taken from Soccodato & Callisto (2009), shows for the two seismic inputs the maximum displacements and bending moments plotted as a function of the overall wall length L. Results are also concisely reported in Table 1.

It can be seen from Figure 6 that the decrease of the displacements with L is qualitatively similar to the results shown in Figure 4(b). The elastic-plastic walls undergo displacements u_y that are larger than those computed for the elastic walls (u_{el}), but the difference becomes very small with increasing L (see Table 1). Bending moments increase with L, as it was expected

Table 1. Main results obtained from the numerical analysis presented by Soccodato & Callisto (2009).

L (m)	u_y/u_{el}	M_{fin}/M_y	ψ_{max}/ψ_y
7	1.59	0.96	43
8	1.39	0.87	20
9	1.13	0.71	2.7

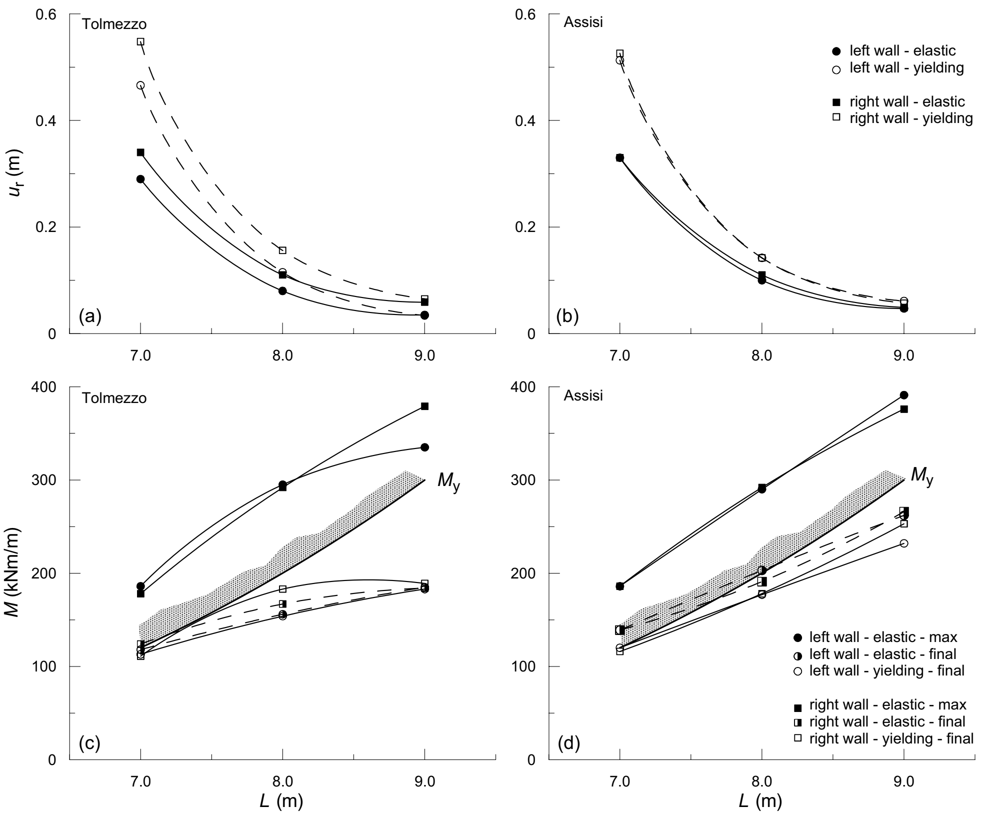

Figure 6. Permanent displacements and maximum bending moments in cantilevered embedded retaining walls of different lengths (Soccodato & Callisto 2009).

(see Figure 4(b)). The final values M_{fin} to range from 70 % (L = 9 m) to more than 95 % (L = 7 m) of M_y.

The mobilised strength in the soil can be quantified by the ratio τ/τ_{lim} of the maximum tangential stress acting at a point to the corresponding available strength. Figure 7 shows, for the cantilevered wall, the contours of the mobilised strength computed by Callisto & Soccodato (2007) before (a) and after (b) a severe earthquake. Before the earthquake, a significant mobilisation of the shear strength is obtained behind the wall, where the soil is in an active limit state, and right below the bottom of the excavation, where the soil is in a passive limit state. At the end of the earthquake the stress state in most of the soil interacting with the walls is quite far from a plastic limit state and the corresponding distribution of the contact stresses produces the large post-seismic bending moments of Figure 6. However, Callisto & Soccodato (2009) showed that a further small excavation causes the soil behind the wall to reach once more a limit active state and therefore produces a decrease of the contact stresses, with an ensuing reduction of the bending moments. Hence, the relatively high internal forces that remain locked into the wall after the earthquake are not deemed capable to endanger the overall safety of the system.

In order to judge the performance of a wall that undergoes plastic yielding during the seismic event, it is of interest to quantify the curvature ductility demand, that is the ductility required for the wall sections to undergo the computed plastic curvatures without a significant strength degradation. Table 1 reports, for the cases considered, the values of the curvature ductility factor, defined as the ratio of the maximum curvature ψ_{max} to the curvature at yield ψ_y. It appears that for the walls with L = 8 and 9 m the curvature ductility demand can easily be satisfied by a properly detailed r.c. section.

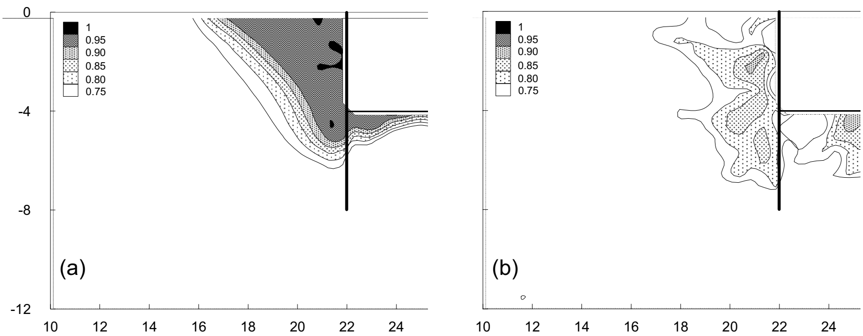

Figure 7. Contour plots of the mobilised strength before (a) and after (b) a seismic event (Callisto & Soccodato 2007).

4 DEVELOPMENT OF TECHNICAL CODES

The Eurocode 8 (EN 1998-5) prescribes that the pseudo-static design of flexible retaining walls be carried out using the maximum expected acceleration a_{max}. From the above discussion, it follows that this prescription is equivalent to the requirement that even under a severe earthquake no permanent displacements should be tolerated.

On the contrary, the Italian Construction Code provides a relationship, obtained directly from Figure 3, between the maximum permanent displacement (that is, the seismic performance) and a factor $\beta = a_h/a_{max} < 1$. The factor β multiplies the expected maximum acceleration a_{max} to yiled the horizontal acceleration a_h that the code requires for use in a pseudo-static calculation. For a cantilevered or singly restrained wall, a_h coincides with the critical acceleration a_c if the wall is designed on the basis of a pseudo-static limit equilibrium calculation performed with unit partial or global safety factors. In this case, if the relationship of Figure 3 holds true, and if the maximum acceleration is evaluated accurately, the wall will not move more than required, and the maximum bending moments will be evaluated as $M(a_c)$, consistently with the approach discussed in the previous section; this value might be amplified by an over-strength factor to protect the structural elements from yielding.

It must be stressed that the seismic internal forces in the retaining wall are proportional to the critical acceleration, and therefore to the soil strength available for the specific plastic mechanism considered. If for any reason this strength is larger than anticipated, the retaining wall will be subjected to larger internal forces. For instance, if the strength parameters of the soil are underestimated, the computation does not err on the safe side: the pseudo-static analysis will lead to an underestimation of the critical acceleration and therefore the maximum bending moments evaluated as $M(a_c)$ will be too low.

Currently, the Italian Construction Code requires that the pseudo-static design of a retaining wall be carried out applying partial coefficients to the strength properties of the soil. This leads to the design of walls with $a_c > a_h$; hence, the embedment depths are larger than those strictly required for the desired performance, and the seismic displacements are likely to be smaller than expected. But these same walls will have to sustain bending moments proportional to a_c, larger than those computed with $a_h < a_c$. A possible, concise way to overcome this difficulty would be to require unit partial coefficients for the seismic pseudo-static calculations. This would be consistent with the real nature of the seismic pseudo-static calculations, than only apparently deal with the safety with respect to a collapse mechanism, but rather serve the purpose to asses the actual seismic performance of a structure.

However, since the length of a retaining wall may be dictated by requirements other than its seismic performance, it may well be that the actual length of a wall is larger than what would be strictly needed for the desired seismic performance: a technical code should therefore explicitly prescribe that the internal forces in the wall be always computed in the hypothesis that the available soil strength is fully mobilised.

The above concepts are yet to be extended to the seismic design of retaining walls with multiple restrains. In these cases, mobilisation of soil strength may not be sufficient for the development of a plastic mechanism, and several different mechanisms may be possible, depending of the choice of the energy-dissipating elements. For this wall types, significant efforts are still needed in order to identify the more convenient plastic mechanisms and to compute the corresponding values for the critical acceleration. In spite of this, it is believed that the concepts exposed herein still hold their validity: relationships like the one shown in Figure 3 may be used to evaluate the seismic displacements, while the maximum internal forces must be calculated considering the full strength of the energy-dissipating elements of the chosen plastic mechanism.

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