PERFORMANCE-BASED SEISMIC DESIGN OF 3D R/C BUILDINGS USING INELASTIC STATIC AND DYNAMIC ANALYSIS PROCEDURES

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ABSTRACT

A performance-based design procedure for realistic 3D reinforced concrete (R/C) buildings is presented, that involves the use of advanced analytical tools. Depending on the building configuration, use of two alternative tools is suggested, i.e. either time-history analysis for appropriately scaled input motions, or inelastic static (pushover) analysis, both for two different levels of earthquake loading. The critical issues of defining appropriate input for inelastic dynamic analysis, setting up the analytical model that should account for post-yield behaviour of the plastic hinge zones, defining loading in two directions and target displacement for the pushover analysis, and detailing in a way consistent with the deformations derived from the advanced analysis, are discussed. The proposed method is then applied to a regular multistorey reinforced concrete 3D frame building and is found to lead to better seismic performance than the standard code (Eurocode 8) procedure, and in addition leads to a more economic design of transverse reinforcement in the members that develop very little inelastic behaviour even for very strong earthquakes.

KEYWORDS: Performance-Based Design, Seismic Design, Reinforced Concrete Buildings, Inelastic Analysis, 3D Effects

INTRODUCTION

The last decade or so has witnessed a clear trend towards "performance-based" seismic design, which can be thought of as an explicit design for multiple limit states (or performance levels, in US terminology). Analysing structures for various levels of earthquake intensity and checking some local and/or global criteria for each level has been a popular academic exercise for the last couple of decades, but the crucial development that occurred relatively recently was the recognition of the necessity for such procedures by a number of practising engineers influential in code drafting. In the US, following a number of recent earthquakes, particularly the 1994 Northridge earthquake, it was realised that while structures built in industrialised countries aware of the seismic risk are in general adequately safe, the cost of damage inflicted in these structures by earthquakes, as well as the indirect cost resulting from business interruption, need for relocation, etc., can be difficult to tolerate. This points to the need to address the problem of designing a structure for multiple performance levels (limit states), i.e. performance-based design (PBD) (Fajfar and Krawinkler, 1997; Priestley, 2000). Furthermore, the need to explicitly include displacement (or drift) as a seismic design parameter, rather than as a final verification of a structure already designed for a certain force level, is increasingly being recognized (Priestley, 2000).

The selection of the type of analysis to be used within the framework of a PBD procedure is also a topical issue, and inelastic analysis is gaining popularity during the last few years, a reason for this being that appropriate analytical tools are now available for performing both types of inelastic analysis, i.e. static ("pushover") and dynamic (time-history). Proposed design methods using inelastic analysis mainly involved dynamic time-history analysis of "equivalent" single-degree-of-freedom (SDOF) systems and pushover or limit analysis of the entire structure, an idea originally suggested by Saiidi and Sozen (1981) and subsequently integrated into design methods for regular buildings, such as the N2 method (Fajfar and Fischinger, 1988). The idea of using inelastic time-history analysis for design purposes was also explored (e.g. Fintel and Ghosh, 1982), but the suggested methods were of an iterative nature, i.e. a preliminary design was improved by successive time-history analyses that identified the weaknesses in that design. This is not necessarily a major handicap but, for practical design purposes, a procedure to judiciously obtain the initial design of the structure is clearly needed.

A promising procedure that does not involve iterations is the "direct displacement-based approach", initially proposed for bridges (Kowalsky et al., 1995) and recently adapted to building design (Priestley and Kowalsky, 2000). This approach also forms the basis of the alternative design procedure included in Appendix I (Part B) of the SEAOC Blue Book (SEAOC, 1999), which also requires verification of the resulting design using pushover-type inelastic analysis. A number of other displacement-based procedures have also emerged recently, such as that by Chopra and Goel (2001), wherein use of inelastic spectra is made in lieu of the elastic spectra for increased (equivalent) damping used in previous studies (Kowalsky et al., 1995; Priestley and Kowalsky, 2000). A more complete presentation of PBD and/or displacement-based design procedures is given in a recent *fib* document (*fib*, 2003), prepared by its Task Group 7.2, where an interesting comparative study of eight methods applied to five different idealized building types is also included.

In Europe, the new seismic Eurocode, prEN1998 (CEN, 2003), retains the two limit state ("ultimate" and "damage limitation") design of previous versions (CEN, 1995), and recognises that inelastic analysis can be used in the design procedure, but the guidance given is limited, mainly referring to the type of input seismic action to be used, such as the selection of accelerograms (for time-history analysis) and the way they should be scaled to match the design spectrum, and also the definition of target displacement for pushover analysis. A significant step towards the use of inelastic analysis in a practical context was the publication of the ASCE-FEMA Prestandard for Seismic Rehabilitation (i.e. Strengthening) of Buildings (ASCE, 2000); however, these guidelines do not address the issue of designing new buildings.

Carrying out PBD requires a definition of the seismic actions that is more detailed than in "code-type" approaches, i.e. the multiple limit states have to be explicitly checked for distinct levels of earthquake loading, which have to be selected taking into account the importance of the structure to be designed. For instance, the SEAOC Blue Book (SEAOC, 1999) defines four distinct levels of earthquake hazard, as follows:

- Earthquake I (EQ-I), representing a "frequent" event, with an 87% probability of being exceeded in 50 years (mean return period of approximately 25 yr)

- Earthquake II (EQ-II), representing an "occasional" event, with a 50% probability of being exceeded in 50 years (mean return period of approximately 72 yr)

- Earthquake III (EQ-III), representing a "rare" event, with a mean return period between 250 and 800 yr

- Earthquake IV (EQ-IV), representing a "maximum considered" event, with a mean return period between 800 and 2500 yr.

Eurocode 8 (CEN, 2003) associates the serviceability verification with a 95 yr event (this is higher than EQ-II, and also higher than the seismic action specified in most other seismic codes for serviceability checks), and the ultimate limit state verification with a 475 yr event (this is similar to EQ-III in high seismicity zones) in the case of buildings of usual importance.

The main objective of this paper is to present a new PBD procedure for realistic 3D reinforced concrete (R/C) buildings, which involves the use of advanced analytical tools. Two alternatives are suggested, depending on the building configuration, one involving time-history analysis for appropriately scaled input motions, and a simpler one involving inelastic static (pushover) analysis; the latter is only recommended if some conditions are met. The proposed method is then applied to a regular multi-storey R/C frame building and seismic performance of this building is compared with that of a similar building designed to a current seismic code (CEN, 1995).

DESCRIPTION OF THE PROPOSED DESIGN PROCEDURE

The first writer and his co-workers (Kappos, 1997; Kappos and Manafpour, 2001) have been developing a performance-based procedure for the seismic design of R/C buildings, which involves use of inelastic analysis, either dynamic or static. This procedure has so far been restricted to 2D buildings (medium-rise and high-rise R/C frames and dual structures) and has shown some potential advantages both from the conceptual and the economy of design point of view. In the following, the various steps of the methodology, extended to be able to tackle realistic three-dimensional buildings, are described and discussed. Extending the method to 3D structures is far from straightforward and some of the associated problems are identified, and possible solutions suggested. In addition to the previous extension, a more consistent safety format is introduced at the various steps of the procedure and some specific recommendations are made regarding the final (and most critical) step, i.e. performance-based 'detailing' of the members.

Finally, a broader scope is given to the method, by prescribing a number of alternative procedures that can be used depending on the structural configuration and the importance of the structure studied. Hence, what is presented in the following should be viewed as a general methodology that can be tailored to the specific R/C building for which it is used, rather than as a single design method that is applied in (essentially) the same fashion in all cases.

The three key features of the proposed procedure are:

- Two distinct performance objectives, 'serviceability' (or damage limitation), typically associated with an earthquake with 50%/50 yr probability, and 'life safety', typically associated with an earthquake with 10%/50 yr probability, are explicitly considered, and the basic strength level of the structure is not defined on the basis of life safety criteria (as in most existing procedures, code-type or otherwise), but on the basis of serviceability criteria. A third performance objective ('collapse prevention'), typically associated with an earthquake with 2%/50 yr probability, is accounted for in the design for shear and the detailing of members, although no separate analysis is made for this performance objective.

- Explicit consideration of inelasticity is made in the analysis, but is only restricted to those members that are selected as seismic energy dissipating zones (plastic hinge zones) by the designer; the selection of these zones follows the well-established capacity design principles (e.g. Paulay and Priestley, 1992).

- Detailing of critical members for confinement is performance-based, i.e. it is controlled by post-yield deformation requirements predicted from the inelastic analysis.

In addition to the above key features, well-established capacity design principles are behind several steps of the proposed method, such as the design against unfavourable failure mechanisms like shear. However, instead of using fixed values of overstrength factors, design actions of non-yielding (under the considered seismic action) members are designed for action effects derived from analysis involving the aforementioned, partially inelastic, model. Interestingly, the New Zealand Concrete Code (NZS, 1995) hints to this option by stating that as an alternative to using the "standard" capacity design approach (Appendix A of the Code), time-history analyses for appropriate records could be used.

For the application of the methodology, it is assumed that the structure has already been designed to satisfy code requirements under normal (gravity, wind, environmental) loading, and an initial selection of the geometry of the various members has been made. However, no initial selection of reinforcement (to satisfy seismic requirements) is needed for any member; this selection is made in the course of carrying out the seismic design of the structure. This is a distinctive feature of the method, compared to other procedures involving inelastic analysis, which cannot be applied without an initial seismic design that includes member reinforcement.

1. Flexural Design of Plastic Hinge Zones Based on Serviceability Criteria (Step 1)

The purpose of this step is to establish a basic level of strength in the structure, adequate for satisfying the damage control (serviceability) criteria of Step 4. This strength is controlled by the reinforcement of plastic hinge zones that are normally the beam-ends and the base of R/C walls and/or ground storey columns.

To avoid over-conservatism, the structure is analysed for a fraction, v_0, of the earthquake level associated with "serviceability" or "incipient damage" verifications, i.e. the requirement that structural members should remain essentially within the elastic range, with minor yielding tolerated in a limited number of members. This earthquake is defined (ASCE, 2000; CEN, 2003) as the one having a 40% to 50% probability of exceedance in 50 years (the EQ-II in SEAOC), while lower probabilities are appropriate for critical facilities. The v_0 factor is intended to provide, in combination with minimum reinforcement and other requirements, the aforementioned basic strength level. This raises the critical issue of the safety format to be adopted for a PBD method, particularly when it involves inelastic deformation checks. Among the possible formats, the most appropriate is deemed to be the one that results in the most realistic estimation of the inelastic demands; the latter can then be checked against permissible values incorporating a reasonable degree of conservatism. Therefore, the proposed format is to use the most probable values for material strengths (i.e. the 'mean' concrete and steel strengths, f_{cm} and f_{ym}) when estimating inelastic deformations and corresponding drifts, and then introduce appropriate safety factors in defining the permissible values of these deformations and drifts. For member design against flexure (and shear), the standard code format is retained, and the Eurocode material safety factors are used in the case study presented later. A preliminary calibration led to a value of the v_0 factor

between 2/3 and 3/4 but, clearly, more work is required for establishing appropriate values for structures different from those studied. As a somewhat simpler alternative, one might also consider carrying out Step 1 for the SEAOC EQ-I level (87%/50 yr), and checking the serviceability criteria of Step 4 for the EQ-II level (50%/50 yr).

Design moments for plastic hinge zones are calculated from a conventional elastic analysis based either on the fundamental mode in each principal direction (equivalent lateral force analysis) or on multiple modes (modal analysis for the response spectrum corresponding to v_0 times the serviceability earthquake or the EQ-I), depending on the structural system. Stiffness of R/C members is estimated assuming moderate amount of cracking (see Step 3). Detailing of the flexural reinforcement of beam and wall critical regions (potential plastic hinges) is carried out taking into account minimum requirements and convenience of construction.

2. Selection of Seismic Actions for PBD (Step 2)

The reference for defining the earthquake level is the design spectrum corresponding to the selected probability of exceedance. For all building types, earthquake actions can be defined by an appropriate number of input accelerogram sets using techniques similar to those prescribed by modern seismic codes (CEN, 2003; ICBO, 1997); each set should include a longitudinal and a transverse component. Actually recorded motions should preferably be used, and a minimum of three record sets is recommended. Such records should be scaled to the intensity of the design spectrum for the probability of exceedance corresponding to the limit state checked (Kappos and Kyriakakis, 2001).

In the specific case of low-rise and medium-rise, regular (in plan and elevation), buildings, earthquake loading can be defined by a horizontal static force pattern, to be used later for inelastic 'static' (pushover) analysis; specific criteria for regularity in plan and elevation are given in modern codes (CEN, 2003; ICBO, 1997) and can be used within the proposed approach. The recommended loading pattern is that of modal forces accounting for the modes that contribute at least 90% of the total mass in each principal direction, but for low-rise and medium-rise buildings, dominated by the first translational mode, the code-type "triangular" pattern gives almost equally good results.

The issue of modal forces in pushover analysis is a critical one, and different procedures have been suggested. Separate analyses for the forces of each mode can be used and calculated responses be combined using the SRSS rule (Chopra and Goel, 2002); this usually works well with displacements, but could lead to unrealistic local response quantities, such as inelastic deformation demands. In the present study, assuming that the dynamic degrees of freedom of the model coincide with the two horizontal translations and the rotation of the floor mass centres, the procedure used was to calculate the horizontal forces and torques resulting at the floor centres from elastic modal analysis (including a sufficient number of modes) for the corresponding response spectrum applied separately in each direction; in other words, modal spectral forces (calculated from elastic modal analysis) were combined (in an SRSS fashion), rather than combining the resulting response parameters (member forces, deformations).

3. Set-up of the Partial Inelastic Model (PIM) (Step 3)

During this step, a model of the structure is set up, wherein beams and the bases of columns and/or walls are modelled as yielding elements, with their strength based on the reinforcement actually present, including that in the adjacent slab. In the same model, columns, as well as portions of walls (when present) intended to remain elastic, are modelled as elastic members.

The sophistication of the basic analytical model should be compatible with the purpose of the analysis, as well as feasible for practical application. For design purposes, modelling of buildings can, as a rule, be based on member-type models (i.e. one finite element per structural member). The effective rigidity assumed for each member of the structure should be consistent with its intended behaviour and the models used. For lumped plasticity elements, it is recommended (Kappos, 1986; Paulay and Priestley, 1992; Penelis and Kappos, 1997; ASCE, 2000) to use 30% to 50% of the gross flexural rigidity (EI_g) for beams, 40% to 60% EI_g for the walls, and 60% to 80% EI_g for columns (in compression), to account for member cracking, which is different in each type of member; the higher values for columns and walls apply for high axial compression. Simple equations involving the longitudinal reinforcement ratio (as determined in Step 1) are also available (Kappos, 1986) for cracked section properties of beams and columns. Using fully cracked section properties ($EI_{ef} = M_y/\varphi_y$, where M_y and φ_y the moment and curvature at yield) for the entire member, rather than some average value of EI_{ef}, might lead to underestimation of ductility demands, but typically gives higher values for displacements.

4. Verification of Serviceability Criteria (Step 4)

In the general case, this step involves time-history analysis of the model described in the previous step for each of the selected sets of input motions scaled to the intensity of the "serviceability" or "incipient damage" earthquake, normally associated with a probability of exceedance of 50% in 50 years. In the specific case of regular buildings, pushover analysis is recommended, using the modal (or the triangular) load pattern, until a displacement estimated from the elastic response spectrum of the serviceability earthquake is reached in the direction under consideration. It is proposed to apply the loading in such a way that, while a certain portion α of the base shear (as defined in Step 2), say V_{bx}, is applied in one direction (x), a portion, 0.3α, of the base shear in the orthogonal direction V_{by} is concurrently applied.

The following performance criteria should be checked using the envelope of calculated response quantities (from the inelastic dynamic or static analysis):

- Maximum drifts do not exceed the limits corresponding to damage requiring repair in the non-structural elements. Recommended interstorey drift values, to be considered for the serviceability check, range from 0.2% to 0.5% the storey height, depending on the type of partitions used (see Kappos and Manafpour, 2001). If the drift criterion is not satisfied anywhere within a storey, stiffening of the structure is necessary.

- Plastic rotations in beam critical regions do not exceed the value corresponding to "non-tolerable" damage (typically that requiring repair). Recommended plastic hinge rotations of yielding members (beams, and, where applicable, walls) can be taken from FEMA 356 (ASCE, 2000) as a function of geometry, axial stress and shear stress levels. Alternatively, the check can involve rotational ductility factors, with recommended values between 1.0 and 2.0, or strain values for concrete (between 0.35% and 0.4%) and steel (between ε_{sy} and $2\varepsilon_{sy}$, where ε_{sy} is the yield strain). Conceptually, strain values are the most appropriate parameter for checking serviceability, but also the least convenient in practical terms (unless fibre models are used for analysis, which leads to drastic increase in analysis time). If the selected limits are exceeded in some members, the corresponding reinforcement is increased.

It is emphasised that both criteria have to be satisfied, as their role is complementary, the one mainly referring to damage in the "non-structural" elements and the other to damage in R/C members. This dual criterion ensures that although some damage may appear in both structural and non-structural elements, the structure does not require structural repair and can be occupied immediately after the earthquake. It is worth pointing out here that the aforementioned criteria are generally similar to those adopted by Eurocode 8 (CEN, 2003), but stricter than those suggested in other PBD procedures (Priestley, 2000; Priestley and Kowalsky, 2000), which do not appear to tackle the issue of whether repair under the "occasional" earthquake will be required.

5. Flexural Design of Non-Dissipating Zones on the Basis of Life Safety Criteria (Step 5)

In the general case, time-history analysis of the same model (with beam reinforcement revised if required during the previous step) is carried out for each of the selected sets of input motions scaled to the intensity of the "life safety" earthquake. For normal buildings (i.e. not for essential facilities), this event is usually taken as the one corresponding to a probability of exceedance 10% in 50 years; this is the "design" earthquake in most current codes that require explicit verification of a single limit state. In the specific case of regular buildings, pushover analysis of the PIM can be used, in the same fashion as in Step 4, but with target displacements compatible with the spectrum of the earthquake associated with life safety requirements.

The analyses of this step provide the critical moment (M) and axial load (N) combinations for each column and wall critical section, required for design and detailing of longitudinal reinforcement in these structural members. The safety format used in modern seismic codes is preserved (as in Step 1) and design values (f_{cd}, f_{yd}) for strength of materials are used.

In the general case, when time-history analysis is carried out, the biaxial bending combinations of critical moments M_1 and M_2 and axial load N must correspond to simultaneous values, i.e. envelope values of moments recorded in each direction of the column cross-section should not be used. Depending on the software used, it might be difficult to keep track of all values that occur at the same time as a peak moment, and simplified rules might be necessary, as discussed in the case study presented in the next section.

6. Design and Detailing for Shear (Step 6)

Design and detailing of all members for shear is carried out using the shear forces calculated in Step 5, multiplied by a magnification factor γ_v to account for an earthquake intensity higher than that having 10% probability of exceedance (other effects, such as inelastic dynamic amplification, are deemed to have been accounted for, due to the use of the partially inelastic model). In other words, the design shears are (implicitly) associated with a seismic action higher than that used for the life safety check, ideally with the action associated with the "collapse prevention" requirement, normally assigned a 2% probability of exceedance in 50 years (cf. the SEAOC EQ-IV). Based on the calibration made so far, which assumed a ratio of seismic actions corresponding to the 2%/50 yr earthquake to those from the 10%/50 yr earthquake equal to about 2, the recommended values of the magnification factor are $\gamma_v = 1.20$ for plastic hinge zones and $\gamma_v = 1.15$ for all other zones; γ_v values are relatively insensitive to higher ratios of seismic actions for the two reference earthquakes, since shear forces do not change substantially after member yielding. For important buildings, a third set of inelastic analyses can be envisaged to determine the design shears, with seismic loads scaled to the intensity of the 2%/50 yr earthquake, but for normal buildings this extra effort is not deemed necessary.

7. Detailing for Confinement, Anchorages and Lap Splices (Step 7)

Detailing of all members for confinement, anchorages and lap splices, is carried out with due consideration of the level of inelasticity expected in each member. Ideally, all the expressions used should involve as a parameter the rotational (or curvature) ductility of the member, which, for yielding members, is calculated in Step 5; γ-factors should again be used to roughly account for the 2%/50 yr earthquake, assuming a proportional increase in ductility demand across the structure. An example of such expression is the Eurocode 8 (CEN, 1995, 2003) formula for the volumetric ratio of confinement reinforcement in columns (and wall critical regions) that involves as a key parameter the target curvature ductility factor μ_φ. Expected ductility of non-yielding members (columns and upper portions of walls) can be taken as that of limited ductility structures (e.g. the ductility class "Low" in the Eurocode); alternatively, whenever an extra analysis for explicitly checking the collapse prevention limit state is deemed necessary and/or can be afforded, inelastic demands can also be estimated for columns and other "non-dissipating" members, but it was found that minimum requirements usually control this part of the design.

8. Final Remarks

It is worth clarifying here that different design verifications in the proposed method are controlled by different limit states (performance levels), and by different levels of seismic action; more specifically:

- Member dimensions and flexural strength of plastic hinge zones are controlled mainly by the serviceability limit state (Step 4); recall that horizontal member (beam) dimensions may also be controlled by deflection requirements under service (gravity) loading.

- Flexural strength of 'non-dissipating' zones is controlled by the life safety limit state (Step 5) and, of course, by minimum reinforcement requirements (the latter is also true for beams).

- Ultimate deformation capacity of members is controlled by the collapse prevention limit state (i.e. inelastic deformations developed for that earthquake intensity should be within the corresponding capacities). With respect to flexural deformations, the first check is carried out at Step 4, and detailing is tailored to the non-collapse limit state requirements by empirical increase of the demands corresponding to the life safety limit state.

- Shear deformations are not explicitly checked (in line with all other methods), but shear strength is calculated (Step 6) for forces anticipated during the collapse prevention limit state.

Finally, although design of foundation members is not specifically addressed herein, there is no problem in incorporating it in the proposed procedure; the general provision in this respect would be to design foundation members at Step 5, using the action effects determined for columns and walls, at their bases. Since the proposed method currently focuses on the design of the earthquake-resisting system of the building, secondary (or "gravity") frame members and/or infill panels are not addressed at this stage, although again there is no restriction imposed by the method that would preclude including in the method the design of such members.
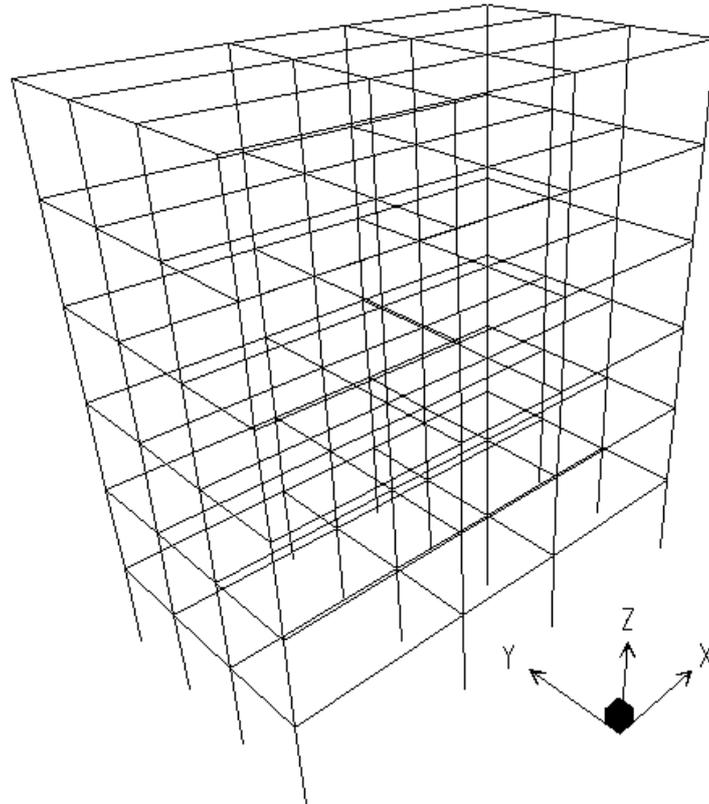
Fig. 1 Geometry of six-storey building

APPLICATION TO A SIX-STOREY BUILDING

The proposed design methodology is applied to a multi-storey R/C building with a lateral load resisting system consisting of a 3D frame. The building is first designed to a standard code procedure, then redesigned to the previously proposed method. Due to its high regularity, the building is designed using both versions of the method (based on either inelastic dynamic, or inelastic static analysis). In addition, several alternative designs to the new method are carried out, depending on the strength level of the plastic hinge zones (Step 1). All designs are subsequently assessed for a number of performance objectives, using both local and global criteria.

The geometry of the six-storey R/C building studied is shown in Figure 1. It is a doubly symmetric structure (three 3 m spans in *y*-direction, three spans of 6-4-6 m in *x*-direction), and was selected as a first test of the proposed procedure to a case where torsional effects are minimal. The procedure has also been tested to a similar 10-storey R/C building (not reported herein) using only the inelastic static version, as the computation time required for inelastic time history analysis was prohibitively long. The choice of a regular and relatively simple structure as a first design example was mainly dictated by the need to identify any problems that may arise in applying the proposed procedure, other than those of the complexity of the structure, and obtain a first idea of the relative performance of the procedure in the case of regular frame buildings, before embarking into the application of the method to the design of more complex structures. Having said this, it has to be stressed at this early stage that it is exactly with respect to the irregular and/or complex structures that the proposed method presents most of its potential advantages; this was already pointed out in the comparative study carried out by the *fib* TG7.2 (*fib*, 2003), wherein the 2D version of the method (Kappos, 1997; Kappos and Manafpour, 2001) was applied to a number of idealized buildings.

1. Code Design

The building was first designed to the provisions of the current Greek Seismic Code, which is very similar to Eurocode 8 (CEN, 1995) – ductility class "M" (medium), for a design ground acceleration of 0.25*g*, assuming class A soil conditions (stiff deposits). Earthquake loading was combined with gravity loading $G + 0.3Q$ (*G* are the permanent actions and *Q* the variable actions, i.e. the live loads), as required by the Eurocodes for dwellings and similar buildings.

The materials used in the structure are C20/25 (characteristic cylinder strength of 20 MPa) concrete, and S400 steel (characteristic yield strength of 400 MPa). Design strengths of materials were derived by dividing characteristic strength by material safety factors, 1.5 for concrete, and 1.15 for steel bars (hence f_{cd} = 13.3 MPa and f_{yd} = 347.8 MPa).

Square column cross-sections (from 300 to 450 mm) were used, with reinforcement ratios not exceeding about 2% (the minimum reinforcement ratio for columns was 1%). Beam sections varied from 200×400 to 300×650 (mm^2). In line with common practice, it was decided to keep member cross-sections the same in (at least) every two storeys.
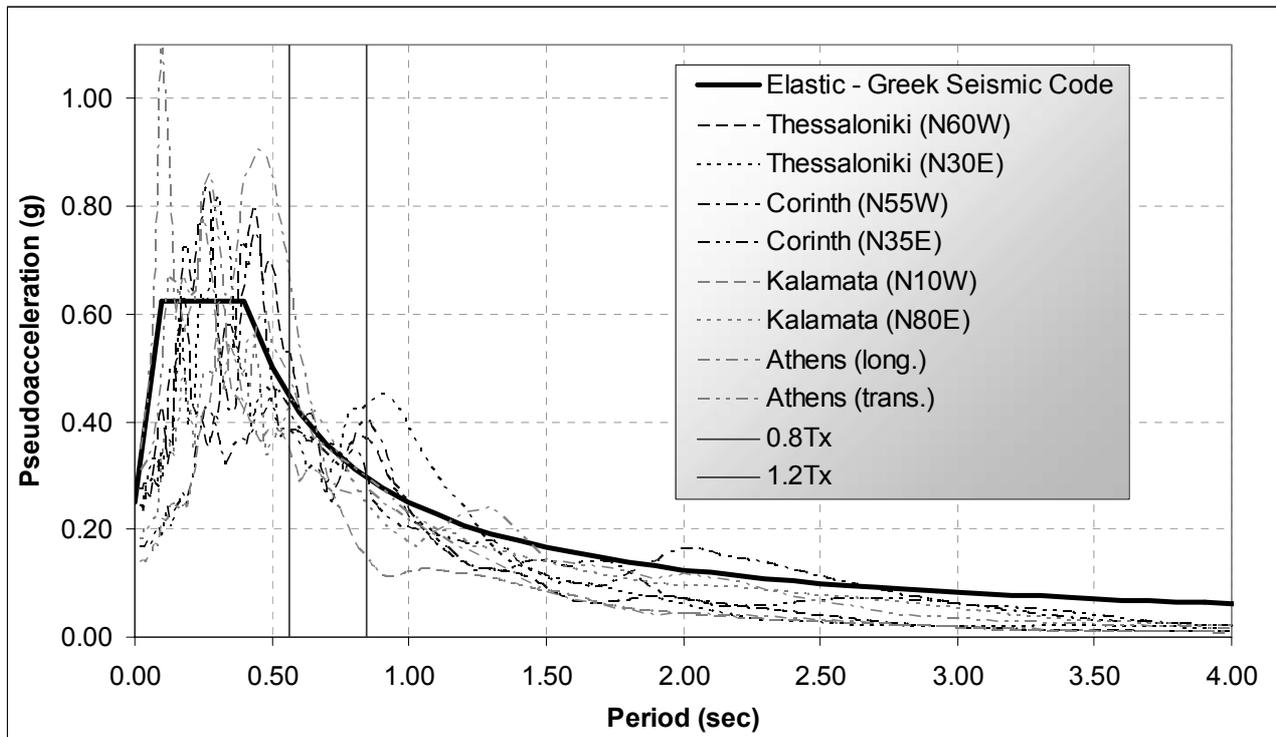

Fig. 2   Response spectra of records used, scaled to the intensity of the design spectrum in the range from 0.8 to 1.2 times the fundamental period T_x

2. Analytical Modelling and Definition of Seismic Loading

Both elastic and inelastic (dynamic and static) analyses of the structure were carried out using "SAP 2000 Nonlinear" (Computers and Structures, 2000), adopting a member-by-member modelling approach. Inelastic beam (and column) members were modelled as elastic elements with inelastic springs (plastic hinges) at their ends; the effective rigidity (EI_{ef}) of T-beams was taken equal to 40% the gross section rigidity (EI_g), while for columns 80% of EI_g was assumed. The moment curvature characteristics of the plastic hinges were estimated from section analysis using appropriate non-linear constitutive laws for concrete and steel (Penelis and Kappos, 1997); member strength and ductility were estimated on the basis of the non-linear section analysis results. Consistent with the proposed format (see Step 1 in the previous section), mean values of strengths of materials were used for calculating the resistance of inelastic members, to be used in non-linear analyses. The effect of slab reinforcement lying within the effective width of the flanged beams (CEN, 2003) was also taken into account in determining the negative moment strength of beams.

For the dynamic time-history analysis, a set of three actually recorded motions was selected at the 'design' stage (additional records were considered for 'assessment'); they were selected among the most damaging earthquakes that struck Greece in the last 25 years (1978 Thessaloniki, 1981 Corinth, and 1986 Kalamata earthquakes). All motions include a longitudinal and a transverse component, and were scaled to the intensity of the design spectrum using a modified Housner technique based on the area under the pseudovelocity spectrum, in the range from $0.8T_0$ to $1.2T_0$ (where T_0 is the fundamental period, in each principal direction); this technique was found (Kappos and Kyriakakis, 2001) to significantly reduce scatter in

the calculated response. The response spectra of all motions used (including the extra ones used for assessment only) are shown in Figure 2; it is pointed out that all records are scaled to the spectral intensity of the design spectrum (also shown) in the range from 0.56 to 0.84 sec, i.e. from 0.8 to 1.2 times the fundamental period T_x (fundamental periods of the building were 0.70 and 0.74 sec, in the *x* and *y* directions, respectively).

For the pushover analysis, the "triangular", code-type, distribution of lateral loading and a 'modal' pattern, defined by the forces acting on the mass centres of each floor when the building is subjected to the response spectrum acting along each main axis, were tried. Modal forces were calculated taking into account the first three modes in each principal direction, whose modal masses contribute about 95% of the total. Due to the regularity of the structure, it was found that the two loading patterns are quite similar; this would not be the case in stiffness asymmetric structures. The "uniform" loading pattern was also used in some analyses, for comparison purposes, but was not considered in the design of the building.

The effect of the accidental eccentricity, taken equal to 5% the corresponding plan dimension according to usual practice (CEN, 2003; ICBO, 1997), was first studied by carrying out analyses of the PIM with and without this eccentricity, and it was found that the effect was very small. Hence, most of the subsequent analyses focussed on the case 'without' the accidental eccentricity, in order to obtain a clear picture of the effect of the other design variables.

The basis for defining the seismic loading corresponding to the various limit states was the Greek Code/EC8 elastic design spectrum for firm soil conditions and a PGA of 0.25g; this is a uniform hazard spectrum for a 10%/50 yr probability of exceedance. Explicit definition of the spectrum for other probabilities generally requires a hazard study for the specific site, which is generally not feasible for buildings of usual importance. In the present case study, a multiplier of 2.0 was selected to scale the EC8 spectrum (10%/50 yr, related to life safety requirement) to the 2%/50 yr spectrum (related to the collapse prevention requirement). The serviceability earthquake was taken to vary from 1/2.5 to 1/2 the design spectrum, along the lines suggested in EC8 (CEN, 1995, 2003) for serviceability checks.
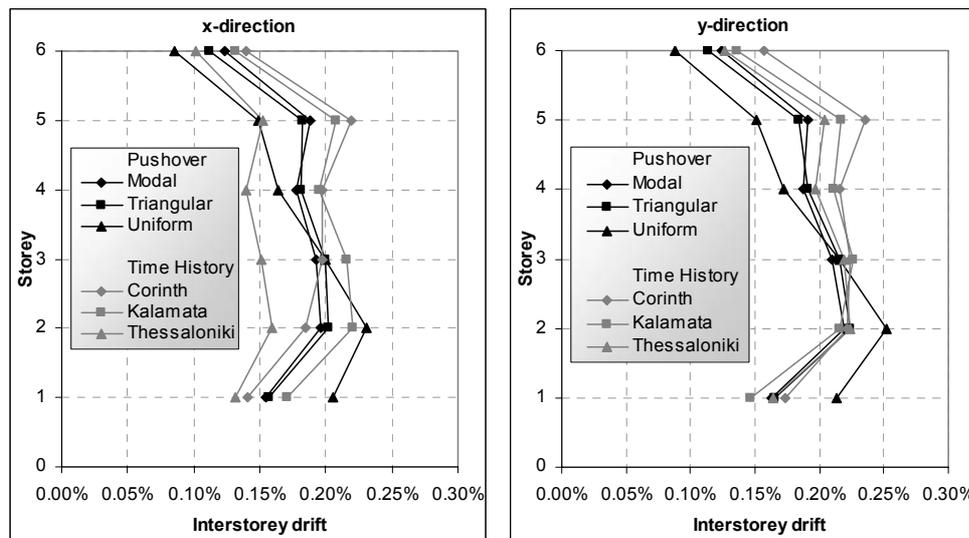

Fig. 3 Interstorey drifts for the serviceability level (strength corresponding to "high" serviceability demands case)

3. Design to the Proposed Method

In order to explore the various aspects of the proposed method and test the effect of some key design parameters, it was decided to carry out alternative designs of the same structure, resulting not only from different type of analysis (static or dynamic), but also from different 'strength' of plastic hinge zones. The flexural design of plastic hinge zones was carried out accepting either "usual" or "high" serviceability requirements; in the first case the v_0 factor of Step 1 was taken as 2/3 and the serviceability earthquake as 1/2.5 the code spectrum (the lower value suggested in the previous section), while in the second case, the v_0 factor was taken as 3/4 and the serviceability earthquake as 1/2 the code spectrum (upper value suggested). It is recalled again here that other codes (in the US, New Zealand, and elsewhere) prescribe significantly lower levels of hazard for checking serviceability; however no reduction of the design spectrum due to ductility is allowed (NZS, 1995) when checking serviceability, hence the resulting design actions

are not much lower than those used in EC8 where the same *q*-factor (mainly accounting for ductility) is used for both limit states. A third case was also considered, wherein the longitudinal reinforcement of plastic hinge zones was taken the same as in the code design, to see the difference the rest of the proposed method would make. These three variations of the beam strength level, combined with the two versions of the proposed method (inelastic static or inelastic dynamic analysis) resulted into a total of six different designs to the proposed method.

3.1 Serviceability Verifications

Having established the strength of the plastic hinge zones, the PIMs of the building were then analysed for the time histories of the three selected pairs of records scaled to the intensity corresponding to the serviceability level (1/2.5 the code spectrum for the "usual" serviceability requirement, and 1/2 the code spectrum for all other cases).

According to the inelastic static version of the method, the PIMs were analysed for increasing fractions of the full modal loading in each direction and simultaneously 30% of the modal loading in the perpendicular direction (Step 4), both applied at the floor mass centres. The building was then pushed to the displacement corresponding to the EC8 elastic spectrum scaled to the intensity of the serviceability earthquake (31 mm or 25 mm in the *x*-direction, 33 mm or 26 mm in the *y*-direction). Due to the symmetry of the structure, one analysis in each direction suffices, instead of four that would be required in the general case.

The maximum response (to the serviceability earthquake) of the building in terms of interstorey drift is shown in Figure 2 for the "high" serviceability requirement case. It can be seen that maximum values of interstorey drift ratio do not exceed 0.25%, which is acceptable for most infill types, as discussed in Step 4 of the procedure and also in Kappos and Manafpour (2001). It is recalled that the uniform load pattern was used for comparison purposes, but was generally not considered in the design, since the calculated response of the structure did not indicate any possibility of a storey mechanism forming. The "usual" serviceability case leads to lower drift values, as the analysis was carried out for a lower intensity (1/2.5 instead of 1/2 the EC8 spectrum).

Plastic rotations in beams from the time history analyses were well below the 0.005 rad limit (ASCE, 2000), with a maximum value (recorded for the Corinth earthquake) of 7×10^{-4} rad in the beams of the 5th storey interior *y*-frame. For the pushover analyses under the serviceability-level earthquake, yielding did not occur in any of the members for the modal and the triangular load pattern; for the uniform pattern, some beams at the lower storeys just entered the post-yield region. This was anticipated, since the building was subjected to seismic actions that were 33% to 50% higher than those for which beams were designed (recall the v_0 factor of Step 1), while 'mean' values of material strengths were used for checking drifts and plastic rotations, and some additional overstrength was also present in several members (due to minimum reinforcement and/or rounding of required reinforcement areas, and/or the contribution of slab reinforcement to negative moment capacity). It has to be noted here that due to the limitations of the software used (SAP 2000), symmetric moment-rotation (*M-θ*) curves had to be input for all members (including beams) in the dynamic analysis, which means that the average of the positive and negative strength had to be used in this case, whereas in pushover analysis, the actual unsymmetric *M-θ* curve could be used for beams; this contributed to the somewhat higher rotation values calculated in the case of dynamic analysis.

Table 1: Required Column Reinforcement, According to Different Procedures

|		Exterior Frames (A & D)			Interior Frames (B & C)		
	Storeys	ω_{max}	$\omega_{average}$	$\omega_{N+0.8M}$	ω_{max}	$\omega_{average}$	$\omega_{N+0.8M}$
Exterior	1-2	0.7924	0.6370	0.5650	0.8868	0.7450	0.6360
Columns	3-4	0.4562	0.3775	0.3379	0.7244	0.6229	0.5244
	5-6	0.4467	0.3969	0.3421	0.6279	0.5698	0.4529
	Storeys	ω_{max}	$\omega_{average}$	$\omega_{N+0.8M}$	ω_{max}	$\omega_{average}$	$\omega_{N+0.8M}$
Interior	1-2	0.9910	0.8027	0.7290	1.0111	0.8686	0.7490
Columns	3-4	0.4563	0.4269	0.3448	0.7043	0.6625	0.5201
	5-6	0.5350	0.4835	0.4098	0.7380	0.6607	0.5422
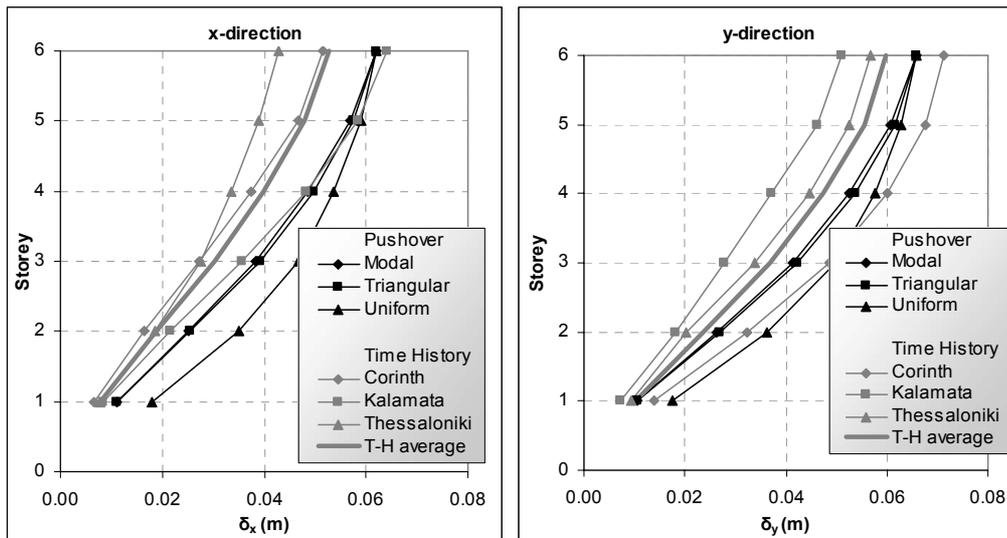
Fig. 4 Storey displacements for life safety level (strength corresponding to "usual" serviceability requirement)

3.2 Life Safety Verifications

Since the serviceability criteria were satisfied, no modification of the original design was deemed necessary and the design proceeded to Step 5 of the proposed method: critical values for flexural design of columns were calculated from time-history analyses of the PIMs for the selected pairs of earthquake records scaled to the intensity corresponding to the unreduced 10%/50 yr spectrum. The time-histories of biaxial moments (M_1, M_2) and the corresponding axial loads (N) were stored (in very large files!) and design of sections was carried out for combinations including $\max |M_1|$, $\max |M_2|$, and $\max N$ (tension or lowest compression), and the simultaneous values of the other actions in each case, using an ad-hoc developed post-processor of the SAP output.

For the inelastic static analyses, the target displacement was estimated to be 62 mm in the *x*-direction and 66 mm in the *y*-direction, using the procedure recommended in ASCE-FEMA 356. A preliminary pushover analysis of the PIM was required in this respect, to establish the secant stiffness at apparent yield of the building (i.e. the slope of the bilinear approximation to the actual pushover curve); such curves are shown later (Figure 6). Flexural design of columns for biaxial bending and axial force was based on the conventional (code) procedure using design values (f_{cd}, f_{yd}) for strength of materials.

Comparison of the column designs, based on the results of static and dynamic analyses, revealed that if the procedure recommended by some codes (ICBO, 1997) is applied, use of the most unfavourable value, calculated in the time-history analysis (if less than 7 records are used), results in significantly larger reinforcement compared to the case of design based on pushover analysis. The reason was that in the case of dynamic time-history analysis, the moment value, M_1, that occurred simultaneously with $\max M_2$ (1, 2 being the axes of the column cross-section) was much higher than the value from pushover analysis (where only 30% of the transverse loading was applied, in line with current practice for combining seismic actions in two directions). Whether this should be interpreted as over-conservatism of the time-history analysis or unconservatism of the $100\%X + 30\%Y$ rule, is not an easy question to answer, but it has to be said that design of a member for the single peak response predicted from time-history analysis is unrealistic since several cycles of displacement are required to cause significant damage, and also design for the most severe of the motions considered is clearly a conservative approach.

Two less conservative procedures were explored (and are summarised in Table 1), i.e. using the average of column reinforcement calculated for the three pairs of earthquake records (this would be allowed by the UBC and other codes if seven or more records had been used), or, alternatively, reducing the max M_i by 20%, selected more or less arbitrarily, to reflect the aforementioned concept that peak dynamic moments are applied to the structure for just a very short time interval, and also that both components were scaled to the same intensity (this was conservative for the Corinth records). As shown in Table 1, each option results in significantly different demand for column reinforcement; the finally selected procedure was that based on the 20% reduction in moments M_1 and M_2 (indicated as $\omega_{N+0.8M}$), mainly on the basis that this gave requirements much closer to those

resulting from the 100%*X* + 30%*Y* rule used in the static case. Clearly, the appropriate selection of pairs of biaxial moments from dynamic analysis is an issue requiring further research, particularly since the background work carried out so far is either based on purely elastic analysis (e.g., Rosenblueth and Contreras, 1977) or on capacity design principles (Paulay and Priestley, 1992); the writers are currently working in this direction.

Storey displacements for the 10%/50 yr earthquake (associated with life safety), calculated from both inelastic static and dynamic analyses, are shown in Figure 4 for the case where strength was selected according to the "usual" serviceability requirement. The target displacement for the pushover analyses seems to have been rather appropriately selected, as it lies within the "exact" values calculated from the three time-history analyses (static displacements are very similar to the average dynamic values in one direction, but larger than those in the other direction). Distribution of drifts along the height of the buildings is different in the static and dynamic case, with higher drifts occurring in the lower storeys in the static case, but in the upper storeys in the dynamic case; this is not surprising, if higher mode effects are considered.

Design shears for members resulted from the forces calculated in the previous step multiplied by the γ_v-factor, taken equal to 1.15 for columns and 1.20 for beams (see Step 6). These values of the γ_v-factor were selected based on additional inelastic static analyses, where the structure was pushed to the displacement corresponding to the 2%/50 yr earthquake, described by double the EC8 spectrum. Clearly, more calibration would be necessary for specifying γ_v-factors appropriate for a broader range of structures.

Design of columns to the proposed procedure resulted in an increase in longitudinal reinforcement, which is larger for interior columns than exterior ones, compared to the original EC8 design, but there was no need to revise the cross-section dimensions. The column-to-beam strength ratio ($\Sigma M_{Rc}/\Sigma M_{Rb}$) in the reference case of 'usual' serviceability requirements was between 1.3 and 1.7 for the interior columns and between 1.2 and 1.8 for the interior columns (column strengths were calculated for the axial loads derived from the pushover analysis); recall that seismic codes typically require such ratios to be of the order of 1.4 for ductile frames. On the other hand, application of Step 7 (detailing) of the proposed method resulted in considerable relaxation of transverse reinforcement requirements in columns compared to conventional code-based design. The most critical factor in this respect was the decision to comply only with the EC8 ductility class "L" (low) transverse steel requirements in the columns (with the exception of their base, wherein class "M" provisions were applied). The column transverse reinforcement was significantly lower than in the code procedure in the lower half of the building where axial loading is high and confinement requirements prevail, but differences tended to vanish in the upper part where minimum hoop reinforcement requirements (to avoid early buckling of longitudinal bars) govern.

ASSESSMENT OF THE ALTERNATIVE DESIGNS

1. Comparisons in Economic Terms

The amount of beam longitudinal reinforcement strongly depends on the initial choice regarding the serviceability requirements and the proposed procedure resulted in an increase of about 10% with respect to design to the "conventional" Code for the "high" serviceability case, while for the "usual" case there was a decrease of about 16%. The transverse reinforcement of the beams was usually governed by minimum requirements and is very close to that resulting from the Code in all cases.

As already mentioned, the proposed procedure resulted, as anticipated, in an increase in 'longitudinal' reinforcement of columns, especially at the lower storeys. This increase was more significant (about 20% in the usual serviceability case and 40% in the high serviceability case, compared to Code design) when the design was carried out using time-history analyses of the PIMs; increases of only 8% to 25% were found when inelastic static analysis was used. On the contrary, the 'transverse' reinforcement is significantly reduced (from 17% to 23%). A complete picture of the reinforcement requirements in each alternative design can be obtained from Figure 5 (slab reinforcement is not included in this comparison). For the usual serviceability requirement (which is the recommended option for common buildings), it is seen that the proposed method leads to a small reduction (5%) in the total amount of reinforcement when the pushover-based procedure is utilised, while this amount is essentially the same as in the Code design when the time-history-based procedure is applied.
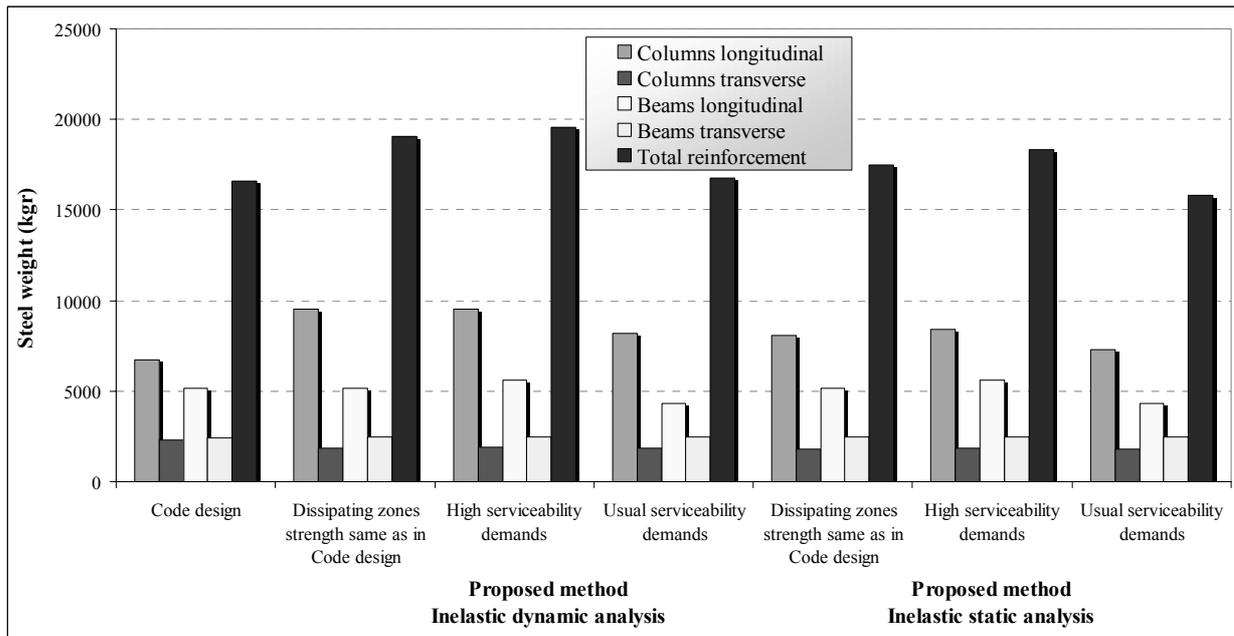
Fig. 5 Required amount of steel in beams and columns, for all designs

2. Assessment of Seismic Performance

To assess the seismic performance of the buildings, designed to the new procedure, as well as the conventional one (European Code), all structures were modelled as full inelastic systems (column yielding permitted) and analysed for different levels of intensity, including that of a "maximum considered" earthquake having a probability of exceedance of about 2% in 50 years (relevant with respect to the "survival" or "collapse prevention" objective); it is recalled that this earthquake was not explicitly considered at the design stage. The assessment was carried out using both inelastic dynamic and static analysis for all buildings designed (one Code design, and six according to the proposed procedure). At this stage, one extra pair of records was used, selected from the 1999 Athens earthquake (Chalandri Station); their spectra are shown in Figure 2.

In pushover analysis, the effect of variation of axial load on the biaxial strength of columns was accounted for, at the assessment stage, by specifying appropriate interaction surfaces (M_x-M_y-N) in SAP2000. These surfaces were constructed on the basis of moment curvature analysis of the columns, accounting for confinement effects in concrete (Kappos, 1986; Penelis and Kappos, 1997), rather than relying on defaults of the program that are based on code (ACI 318) procedures ignoring confinement and failing to distinguish between yield moment and ultimate moment. Ductility of each member was also estimated with due account for confinement, and M-θ curves for the rigid-plastic point hinges in the concentrated plasticity models were constructed by defining the yield moment, the ultimate moment, and the plastic rotation capacity, estimated from moment-curvature analysis and assuming an appropriate plastic hinge length (Penelis and Kappos, 1997; Priestley, 2000). A residual strength flat part of the M-θ curve, at a level of 20% the yield moment, extending up to twice the rotation capacity, was also included. The plastic rotation capacities, estimated from refined analysis, generally exceeded the values recommended in the ASCE-FEMA Prestandard (ASCE, 2000), although the failure criteria used in the refined analysis were rather conservative (the minimum of strain values corresponding to strength drop to 0.85 the unconfined strength along the descending branch of the stress-strain curve for concrete, or first buckling of longitudinal bar, or first hoop fracture).

For time-history analysis, SAP2000 (Version 7.44) does not offer many possibilities, and the post-yield behaviour was modelled using bi-linear M-θ curves for each principal direction, with no account for M_x-M_y-N interaction. As already mentioned, the average of the positive and negative strength had to be used for beams, since the program can treat only symmetric loops. These are major drawbacks in inelastic time-history analysis and improvements are needed in this direction; some of these improvements, such as the treatment of non-symmetric hysteresis loops, were included in Version 8 of SAP2000.
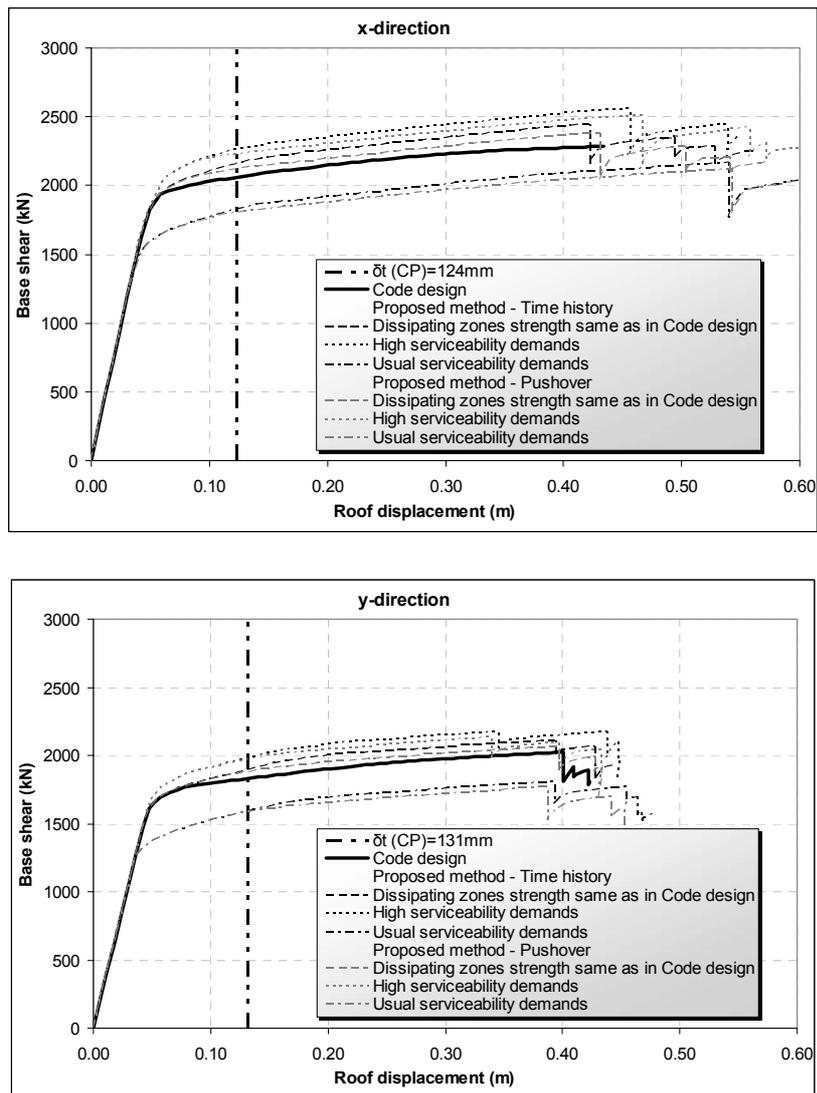
Fig. 6 Pushover curves for building designed to different procedures: Main
loading in the *x*-direction (top); Main loading in the *y*-direction (bottom)

A complete set of inelastic dynamic and static analyses of the fully inelastic models were carried out for the life safety level, while for the collapse prevention level, all static, but very few dynamic, cases were analysed, due to the prohibitively long computational time required for the latter (about 2 weeks for each case, on a standard 1.6 GHz machine); this is, again, a major weakness of the software used.

Figure 6 summarises the pushover curves (up to failure) calculated for all alternative designs of the building. The curves were derived for the fully inelastic model of the structure (columns allowed to yield), applying the aforementioned modal patterns of static loading; pushover curves were very similar for the modal and triangular patterns, a clear indication of the dominance of the fundamental mode in both directions of the structure. Also shown in Figure 6 are the top displacement requirements, estimated on the basis of the ASCE-FEMA 356 procedure for δ_t, for the 2%/50 yr earthquake (i.e. twice the EC8 spectrum; δ_t for the 10%/50 yr earthquake is half the value shown in Figure 6). The top displacements, calculated in the inelastic time-history analyses run, were about 110 mm in the *x*-direction, and 90 mm in the *y*-direction; the former value is very similar to that predicted by the FEMA procedure, while the latter is lower. It is clear from the curves in Figure 6 that the building can safely withstand the maximum considered earthquake; member failures initiate at displacements in excess of 330 mm in the more critical direction (*y*), which corresponds to a top storey drift of 1.8%, while the required drift is only 0.7%. For the 10%/50 yr earthquake, all designs (including the 'usual' serviceability ones, i.e. minimum provided strength) respond just past the (global) yield point. Hence, both the Code and the PIM-based designs appear to provide adequately safe structures, but it is clear that the safety margin (in terms of the ratio of

roof displacement at failure to the corresponding displacement demand), is higher for the proposed method; this is far more the case in the *x*-direction of the building.

Code Design   PIM- High Serviceability Demands   PIM - Usual Serviceability Demands

10%/50 yr earthquake (modal pattern)

2%/50 yr earthquake (modal pattern)

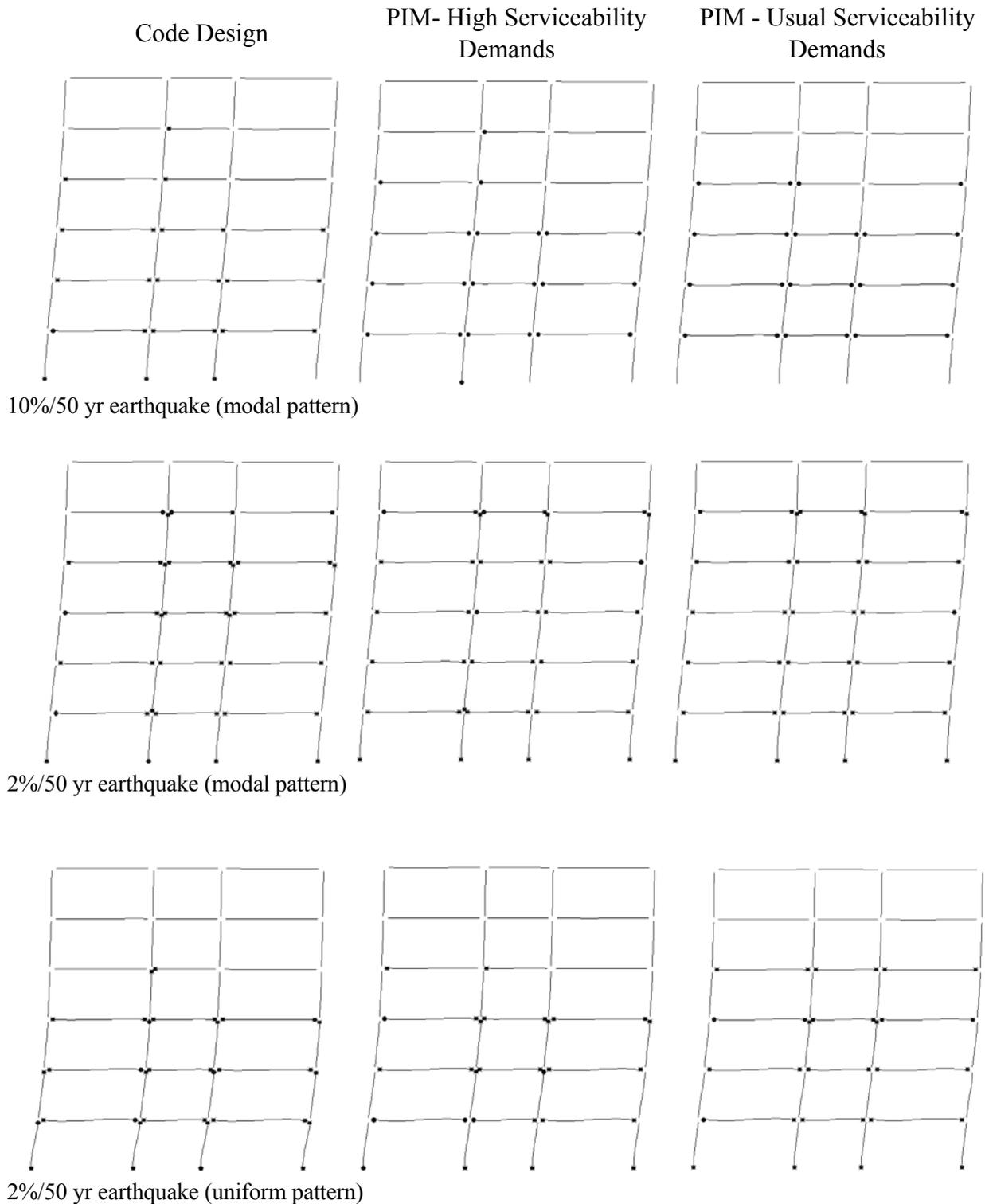2%/50 yr earthquake (uniform pattern)

Fig. 7 Plastic mechanisms (from pushover analysis) for interior *x*-frames, under various levels of seismic action and different loading patterns

Since the proposed design procedure resulted in significantly reduced transverse reinforcement in the columns with respect to the code-based procedure (see Figure 5), it is clear that the generally larger displacements at failure in the designs to the proposed procedure are not due to increased column ductility but rather to a more favourable plastic mechanism. This can best be seen in the plastic hinge patterns at the collapse-prevention earthquake level. It is clear from Figure 7 that in the frames designed according to

the proposed method, column hinging hardly occurs even at the displacement corresponding to this 2%/50 yr earthquake, while some column yielding is present in the lower part of the building designed to the code procedure. When the earthquake level is further increased, these yielding columns are the ones that eventually fail. The same differences in the plastic mechanisms were detected in the *y*-frames.

Table 2: Required and Available Plastic Hinge Rotations in Beams (2%/50 yr Earthquake)

Storey	x-direction			y-direction		
	$\theta_{p,req}$	$\theta_{p,av}$	req/av (%)	$\theta_{p,req}$	$\theta_{p,av}$	req/av (%)
6	-	-	-	-	-	-
5	2.48E-03	6.15E-02	4.03%	3.55E-03	5.17E-02	6.88%
4	5.86E-03	6.46E-02	9.07%	6.41E-03	5.47E-02	11.70%
3	7.87E-03	6.46E-02	12.18%	8.49E-03	5.47E-02	15.51%
2	8.07E-03	6.21E-02	12.99%	8.72E-03	6.13E-02	14.23%
1	6.54E-03	6.31E-02	10.37%	6.44E-03	6.13E-02	10.50%

Having established the superior performance resulting from the proposed design method with respect to the collapse mechanism, a further point worth investigating is the magnitude of plastic rotations in the beams of the building (i.e. the dissipating zones). Table 2 depicts the maximum plastic rotations recorded in the beams of each frame (*x* or *y*) of the building, at the 2%/50 yr earthquake level; it is seen that the values are very moderate (generally less than about 0.009 rad) and indeed well below the corresponding capacities of the beams (no more than 15% the corresponding capacities). Calculated plastic rotation demands in columns are even less, i.e. not exceeding about 0.003 rad, when the modal loading pattern is applied; these values generally do not exceed about 10% of the corresponding capacities.

Finally, Figure 8 shows the interstorey drifts calculated from inelastic dynamic analysis for the Athens earthquake scaled to the 10%/50 yr intensity (0.25*g*); it is recalled that this earthquake was not considered at the design stage. It is clear that the peak values do not exceed about 0.6%, an indication of a very favourable behaviour. Even under the 2%/50 yr earthquake, peak drifts were of the order of 1%, again mainly due to the favourable plastic mechanisms that all structures (particularly the ones designed to the proposed method) formed.
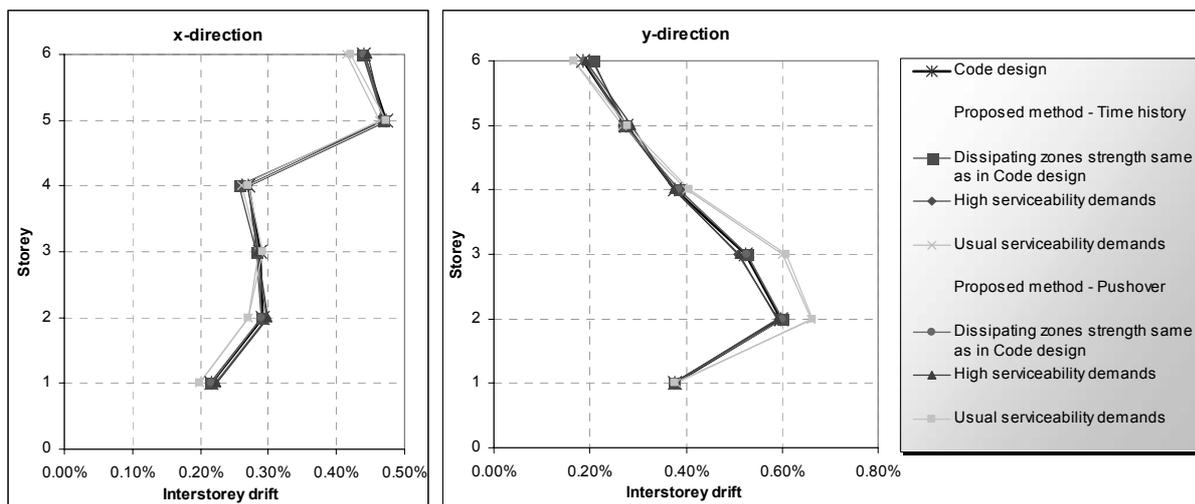

Fig. 8 Interstorey drifts calculated for Athens earthquake scaled to design intensity (0.25*g*)

CONCLUSIONS

A new framework of seismic design for realistic (3D) R/C buildings has been presented, based on the explicit consideration of performance criteria for two distinct limit states and a feasible partial inelastic model of the structure. At least for the buildings studied, a careful selection of the necessary explicit verifications required (for the serviceability and the life safety limit states) leads to an implicit satisfaction of performance criteria for the collapse prevention state as well.

The time-history-based version of the method requires reliable and efficient software, and it appears that ample room for improvement exists in this direction; it is recalled that unrealistic amounts of computation time were found to be needed in the presented case study (involving a simple 3D building), wherein a well-known commercial package was used.

In applying the proposed method, the choice of biaxial moment pairs from inelastic dynamic analysis, to be used for column design, is a matter worth further investigation, since over-conservatism should be avoided.

The suggested new procedure was found to lead to a very satisfactory seismic performance, at least for the type of regular multi-storey frame buildings to which it has been applied herein. Performance under the earthquake related to serviceability, life safety and collapse-prevention objectives was assessed using a full inelastic model of the structure and found to be quite satisfactory. The suggested procedure was compared with that prescribed by a modern design code, compatible with current European practice, and was found to lead to generally better seismic performance than the standard (elastic analysis-based) code procedure, which, nonetheless, results also in acceptable performance.

The better control of the plastic mechanism achieved by applying the proposed method permitted a substantial relaxation of the confinement requirements in columns, particularly those at the lower half of the building where axial loading governs these requirements. Despite the reduced amount of confinement in many columns, the building designed to the proposed procedure was able to develop a larger horizontal displacement than the corresponding code-designed structure.

Although the proposed design framework can readily be applied to realistic 3D buildings, the validity of the procedure has currently been verified only for the case of regular buildings. Further calibration is needed of some empirical factors used, particularly those for estimating shears and inelastic demands for earthquakes higher than that related to life safety. Alternatively, the designer might choose to explicitly check the collapse-prevention limit state, since the strength hierarchy imposed by the method allows a realistic assessment of even this stage; the additional computational effort is moderate, in the light of currently available analysis tools. It is noted though, that even if this additional effort is spent, ductility demands in columns are found to be very low and the designer has to decide on appropriate minimum requirements, which should be preferably based on probabilistic criteria.

Finally, compared to other recent proposals, in particular those based on the displacement-based design (DBD) approach, the method suggested herein is generally more involved in the case that time-history analysis is utilized (*fib,* 2003), but almost equally demanding in the case of pushover analysis. Direct displacement based procedures are based on the ability to have a reasonably accurate estimate of the displacement profile of the structure responding to different fractions of the design earthquake; whenever this is the case, pushover analysis can generally be used, and the effort required by the two approaches becomes similar. The time-history version of the proposed method is clearly the most appropriate option for complex and/or higher-mode dominated structures, to which it is currently being applied by the writers. The seismic performance of buildings designed to the proposed and to other DBD methods is deemed to be similar if the selected performance criteria are similar. It is particularly noted in this respect that the explicit verification of two limit states suggested in this paper is not addressing drift control only, but also tackles issues like degree of damage and need for repair in structural members (which also affects the 'function' of the building) after an "occasional" earthquake, which unavoidably leads to higher strength requirements than in DBD procedures. As a closing remark, one can say that there is certainly room for further improvement in all the aforementioned methods, and time will show which the right option is for each particular class of structures.

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