13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2670

A DAMAGE-CONTROLLED FORCE-BASED SEISMIC DESIGN METHOD FOR RC FRAMES

Alireza MANAFPOUR[1]

SUMMARY

In the light of increasing awareness of the importance of controlling the damage in seismic design, it is now well known that a paramount role should be given to deformation quantities in earthquake resistant design. As a result the traditional force-based strategy has been questioned and consequently displacement-based strategy has been suggested during the past decade. While the actual impact of displacement-based design strategy on seismic design practice has yet to be explored, clear advantage can be expected by using current advancement in computational tools to quantify and control the deformation demands within the structure.

The paper presents an effectively force-based seismic design strategy that incorporates inelastic analysis approaches in design process to control the excessive deformations. The method benefits from a performance-based framework and an explicit deformation check at immediate-occupancy and life-safety performance levels. It is based on nonlinear analysis of an analytical model wherein structural members with intended inelastic behavior are modeled as inelastic members while the others are modeled as elastic members. To allow for reasonable modeling of the inelastic members their initial design is carried out using a conventional design. Both nonlinear static and dynamic analysis alternatives are explored.

Practical application of the method is illustrated by a case study involving ten-story reinforced concrete frame structure. The design of the case study is evaluated using nonlinear time history analysis at damage-controlled, life-safety and collapse-prevention levels and a set of seismic ground motions.

The results of the study indicate that a practical control of damage can be achieved by incorporating nonlinear analysis approaches. This is being encouraged by the continuing progress in achieving high speed and capacity for computational tools.

INTRODUCTION

The importance of excessive deformations has been noted since early years of seismic design studies, which then was reflected in the definition of seismic design philosophy in a damage-based form [1]. This recognition did not necessarily result in a design method that gives a leading role to deformation demands. Traditional seismic design methods were developed with excessive emphasis on force quantities and a

[1] Consultant Engineer, Special Structures, Halcrow Group Ltd, Glasgow, UK, manafpoura@halcrow.com

secondary treatment of the deformation demands. Nevertheless in recent years there has been growing recognition of the importance of the damage-control as an explicit design consideration [2]. Therefore there is clearly a need for explicit addressing of inelastic deformation demands in design process.

The direct displacement-based design method [3] has been introduced during the past decade or so as an alternative approach to traditional methods, which explicitly considers the deformation quantities in seismic design. However it has been suggested [4] that the actual impact on seismic design practice would be marginal if the current force-based methods is replaced by a direct displacement-based method. This is mainly due to the similar level of simplification in both methods to reduce the complicated nonlinear seismic design problem to a simplified linear one. Otherwise the overall trends followed in both methods consist of similar actions, though they are carried out in different order in each method. On the other hand displacement-based design methods are relatively young and a unified understanding of the corresponding concepts has yet to be established.

In view of aforementioned status of direct displacement-based methods, the other alternative is to improve the conventional force-based design methods by incorporating more explicit deformation verification. With regard to this alternative the equivalent force procedure for seismic design, embodied in most of the building design codes, has several shortcomings, which restricts the consideration of deformation demands in the design process. One important deficiency of the equivalent force method is that it is not possible to determine the location and the extent of inelastic deformations within the structure by using this method. This restriction can be removed by using more realistic model of the structure that incorporates inelastic behavior during seismic event. With current advancement in analytical capabilities and capacities a clear advantage can be expected in using inelastic analytical methods in design process.

Inelastic analysis approaches for determining the earthquake response of structures have been used as research tools during the past few decades. Although recent codes recognize inelastic static, or time history, analysis as alternative tools in the design procedure, they rarely give sufficient guidance about how the method should be implemented in practice

The main objective of this study is to investigate the possibility of promoting current force-based design methods in terms of control on deformations with the aid of nonlinear analysis procedures. The paper focuses on the requirements for practical incorporation of inelastic analysis methods in the seismic design process. A design method with incorporated nonlinear analysis capability is introduced which benefits from a performance-based framework. To elaborate possible advantages and disadvantages of the method a RC frame is designed using the suggested method as well as EC8 [5] conventional method. An assessment procedure including several criteria is used to evaluate and compare the design outcomes.

THE ESSENCE OF DEFORMATION CONTROL CONCEPT

Current seismic design practice is a result of the gradual awareness and understanding among the engineering community that took place in the course of the last century. The evolution of seismic design practice continues as our understanding of seismic actions and effects are increased. From a conceptual point of view the seismic design is an attempt to provide structures with strength and deformation capacities that exceed, with an adequate margin of safety, the demands imposed by severe earthquakes.

Historical development of seismic design approaches followed the recommendations, in the first decade of the twentieth century, to assume additional lateral loads on the structure equal to a fraction of supported weight. Hence the earthquake effects were treated like the other types of environmental effects such as gravity and wind loads, by prescribing a set of lateral loads. This is how the so-called force-based approach for seismic design started. However, a considerable time past before a general philosophy for earthquake resistant design of buildings was introduced. According to Bertero [6], this was done in the US for the first time in the SEAOC Blue Book in1967 [7] which has changed very little since then. It essentially states that the design should accomplish the following objectives:

- prevent non-structural damage in minor earthquake ground shaking, which may occur frequently during the service life of the structure;
- prevent structural damage and minimise non-structural damage during moderate earthquake ground shakings, which may occasionally occur;
- avoid collapse or serious damage during severe earthquake ground shakings, which may rarely occur.

It is clear that one of the key parameters in the definition of the general philosophy of seismic design is *damage*. The philosophy essentially calls for a controlled damage situation within the structure depending on the severity of the seismic action. Since the main measure of damage is structural *displacement* or *deformation*, therefore a deformation-controlled design procedure would be a desirable one.

It can also be seen that the design philosophy has a multi-level characteristic, i.e. the objectives concern with different levels of acceptable damage at different levels of seismic action. This necessitates a move from traditional one-level earthquake criteria and methodology to multi-level criteria and methodologies, which in commonly known as *Performance-Based Engineering*.

NONLINEAR ANALYSIS FOR DESIGN

Reinforced concrete structures designed to resist intense earthquake ground motions should be capable of withstanding inelastic deformations. Nonlinear modeling and analysis of the structure can provide essential information about expected inelastic demands during a severe earthquake and hence lead to a proper control of deformations.

Generally two alternatives of nonlinear analysis are available. First alternative is the nonlinear dynamic analysis which has long been recognized as the best approach to evaluate the inelastic response under seismic loading. Despite the complexity inherent in using nonlinear dynamic analysis, it has been applied to design and evaluation of many important structures during the past 3 decades. However code drafting committees have been reluctant to adopt this type of analysis [8]. Second alternative is the nonlinear static analysis where the inelastic behavior of the system under seismic loads is considered by incremental application of equivalent static loads. The most frequently used method is pushover analysis, i.e. nonlinear static analysis of structure under monotonically increasing lateral loading. It represents a relatively simple option to estimate nonlinear structural performance. The sequence of component yielding and the history of deformations and shear forces in the structure, as well as potential collapse mechanisms, can be traced, as lateral loads/displacements are increased.

It is noteworthy that the explicit application of nonlinear analysis procedures for the design of new structures is inevitably restricted, because nonlinear modeling normally requires information about the details of the structural elements to such an extent that cannot be established without an initial trial. The implication is that simplified methods are still required at least for initial design of the whole structure or for those parts that would make possible the construction of a reasonable nonlinear modeling of the system.

PREVIOUS STUDIES

During last decade in somewhat different formats, pushover analysis (POA) has been proposed, formulated and evaluated in several research studies [9-12]. Some design and analysis procedures have also been proposed [13-16]. The procedures are primarily directed at the evaluation of a designed structure using pushover analysis. Although the design implications are not complicated, they are not explicitly addressed and no comprehensive design framework involving nonlinear static analysis is proposed.

Developing design methods by means of nonlinear dynamic analysis, applicable in design practice, has also been the focus point of several research studies [17-22]. The proposed methods either are exceptionally complicated with high computational cost, or mainly concerned with redesign, retrofit or optimization of an original design. Hence it has been difficult to adopt these methods in design standards.

A procedure based on time-history analysis has already been presented by Kappos [17], specifically for the capacity design of columns in RC buildings, and comparisons with some other existing capacity design procedures have been made. This procedure forms a critical part of the method presented in the following, which however constitutes a comprehensive proposal for performance-based design of RC frame buildings. Moreover, a simpler alternative of the method, based on pushover analysis is also presented. Some key features of the proposed method along with a trial application have been discussed in Kappos [8]. Here a summary of the method is presented and some additional features are discussed. The application of the method is further illustrated using a case study with 10-story RC frame structure.

SUMMARY OF PROPOSED DESIGN METHOD

Inelastic dynamic approach

The various steps in the suggested methodology are summarized below; it is assumed that the structure has already been designed to satisfy code requirements under normal (gravity, wind, environmental) loading:

1. Flexural design of the beams of the structure for the seismic action under which the structure is required to remain essentially in the elastic range, combined with the appropriate ("quasi-permanent") gravity loading. For usual buildings this action can be taken as a fraction ν_0 (varying from about 2/3 to 3/4) of the earthquake with a 50% probability of exceedance in 50 years (lower probabilities are appropriate for critical facilities). The ν_0 factor is intended to provide (in combination with the minimum reinforcement requirements) a level of strength to the structure adequate for satisfying the serviceability criteria of step 5.

   Design moments are calculated from an elastic analysis based on the fundamental mode or multiple modes, depending on the structural system, and stiffnesses of members are estimated assuming a moderate amount of cracking. If required, some moment redistribution is carried out with a view to optimising beam design.

2. Detailing of the flexural reinforcement of beams, taking into account minimum requirements and convenience of construction (e.g. use of a limited number of bar diameters). This step establishes a basic strength level for the structure, as the strength of the remaining members depends strongly on that of beams (see following steps).

3. Selection of an appropriate set of input accelerograms, using techniques similar to those described in modern seismic codes. Either artificial, spectrum-compatible, or (preferably) actually recorded, motions, may be used. A minimum of three records is recommended.

4. Construction of a model of the structure wherein beams are modeled as yielding elements, with their strength based on the reinforcement actually present (including that in the adjacent slab), and with due consideration of factors such as stiffness degradation and strength degradation. In the same model, columns, intended to remain elastic, are modeled as elastic members. With regard to initial stiffness assumptions, fixed percentages of the gross section rigidity EI_g for each member type may be used for convenience, but somewhat more sophistication can be involved in the modeling of beams, whose reinforcement is already known. The same model can also be used for the first set of analyses (Step 1), with beams assumed to be of "infinite" strength (i.e. to respond elastically).

5. Time-history analysis of the model described in the previous step for the selected set of input motions scaled to the intensity of the "immediate-occupancy" or "serviceability" earthquake; this earthquake is normally associated with a probability of exceedance of 50% in 50 years.

   The following performance criteria should be checked.

   a) Maximum drifts do not exceed the limits corresponding to damage requiring repair in the nonstructural elements. If the drift criterion is not satisfied at any story, stiffening of the structure is necessary; this can be done by increasing the cross-section dimensions and/or reinforcement.

   b) Plastic rotations in beam critical regions do not exceed the value corresponding to "non-tolerable" cracking (i.e. that requiring repair). If the specified plastic rotation or ductility limits are exceeded in some members, the corresponding reinforcement is increased.

   It is emphasised that both of the foregoing 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 RC members.

6. Time-history analysis of the same model (with beam reinforcement revised if required during the previous step) for the selected set of input motions scaled to the intensity of the "life-safety" or "repairable damage" earthquake. For normal buildings (i.e. not for essential facilities) this event is typically taken as the one corresponding to a probability of exceedance 10% in 50 years. This analysis provides the critical moment (M) and axial load (N) combinations for each column.

7. Design and detailing of longitudinal reinforcement in columns of the structure. For a column subjected to biaxial loading, consideration of the following three combinations will be sufficient for most practical purposes (M_y and M_z are the moments acting along the two main axes of a column):

   - max $|M_y|$, and corresponding M_z and N
   - max $|M_z|$, and corresponding M_y and N
   - min N (maximum compression) or max N (minimum compression), and corresponding M_y and M_z.

   For uniaxial loading two combinations will suffice. The foregoing are valid on condition that the axial load limitations imposed by modern codes are respected; these limitations should be checked using the min N (maximum compression) calculated in the time-history analysis.

8. Design and detailing of all members for shear, using the shear values calculated in Step 6, multiplied by a γ_R factor (of about 1.10) to account for an earthquake intensity higher than that of the 10% probability of exceedance in 50 years.

9. Detailing of all members for confinement, anchorages and lap splices, using design equations involving the level of inelasticity expected in each member. Target ductility of non-yielding members (columns and upper portions of walls) is taken as that of limited ductility structures.

Inelastic static approach

It will be shown that most of the essential features of the inelastic dynamic approach can be maintained if a simpler static pushover analysis is used instead. This is generally appropriate in the case of regular low- and medium-rise buildings with minor contributions from higher modes. Application of pushover analysis, with due consideration given to its limitations, can be considered as lying halfway between existing elastic methods and nonlinear time history analysis. Hence a carefully performed pushover analysis will provide insight into both methods in terms of level of reliability versus complexity.

Design steps

In the pushover analysis-based procedure, step 3 of the suggested inelastic dynamic approach is replaced by the selection of a loading pattern for the analysis (e.g. triangular, uniform, exponential, or modal). Then at steps 5 and 6 a pushover analysis of partial inelastic model is performed up to a target displacement limit value for deformation demands corresponding to "serviceability" or "immediate-occupancy", and "life-safety" or "design" earthquakes, respectively. Generally the selection of a target value for deformation demands can be based on local or global demands (ductility factors, interstory or global drift ratio). Steps 7, 8 and 9 are applied as before, except that in step 7, unlike the inelastic dynamic approach, the appropriate combination of M-N can be found simply by considering the values at the end of the analysis.

Target displacement

Having defined the loading pattern for the pushover analysis (see subsequent section), target values of displacement are needed to establish a link with the various levels of seismic action that pertain to different performance levels. From the practical point of view a simple choice for target displacement parameter, to be used in pushover analysis, is top story displacement or global drift ratio for the structure. Various methods have already been suggested for the estimation of an appropriate top story displacement or drift ratio for the structure [2,11,23-25]. The procedure adopted in the recent NEHRP recommendations for seismic rehabilitation [23] was found to be an appropriate option for the current study. The procedure is based on an equivalent SDOF modeling of the structure and defines target displacement at the top of the structure as:

$$\delta_t = C_0 C_1 C_2 C_3 (T_e^2 / 4\pi^2) S_a \tag{1}$$

Where:

- T_e is the effective fundamental period of the building calculated based on a force-displacement relationship derived from pushover analysis:

  $$T_e = T_i (K_i / K_e)^{1/2} \tag{2}$$

  where K_i is the elastic lateral stiffness of the building (initial stiffness at the origin of the force-displacement curve), K_e is the effective lateral stiffness of the building (secant stiffness at the base shear force equal to 60% of the yield strength, Figure 1) and T_i is the elastic fundamental period calculated by elastic dynamic analysis.

- S_a is the elastic response spectrum acceleration at the effective fundamental period (unit length/s^2).

- C_0, C_1, C_2 and C_3 are modification factors to relate the spectral displacement to building roof displacement, the maximum inelastic displacement to linear elastic one, and also to take into account the effect of hysteresis loop shape and P-Δ effects on displacement response.

It is recalled that the target displacement value is still a rather controversial parameter in the literature related to pushover analysis. Although Equation 1 is generally applicable to different performance levels, it has mostly been calibrated and used for life-safety earthquakes.

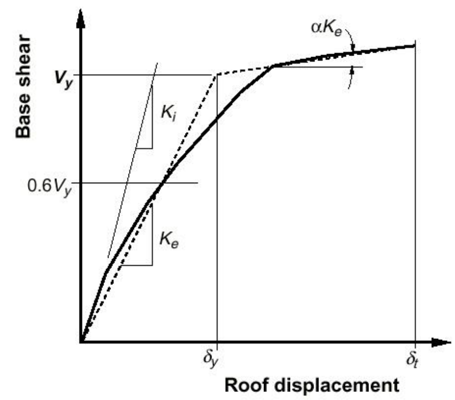


Figure 1. Calculation of effective stiffness K_e [23]

MODELING OF SEISMIC ACTION

The importance of the proper establishment of seismic loads is reflected by the need to know against what we have to design the building. From the analytical point of view the type of analysis greatly affects the process of definition and modeling of these loads in practical applications. While for elastic and inelastic static analysis the distribution of loads along the structure appears to be an essential issue, for inelastic dynamic analysis the selection and scaling of earthquake ground motions are challenging tasks for the analyst.

Loading Patterns for Nonlinear Static Analysis

The selection of an appropriate lateral load/displacement pattern is one of the fundamental issues for POA. First of all one needs to be aware of differences between choosing a force pattern or a displacement pattern for pushover analysis. It is noticed that for some structures, such as those with regular stiffness distribution, it is likely that both patterns result in a similar response. The purpose of the load pattern is to represent the relative inertial forces that a structure may experience during an earthquake. Similarly by applying a displacement pattern it is attempted to model relative displacements of a structure under seismic action. It is clear that both these relative values will change with the extent of nonlinear behavior [9]. However it is noteworthy that the application of a fixed displacement distribution may lead to obscure a soft story mechanism. Probably the application of a force distribution with a displacement control of the incremental solution is the only reasonable solution that is capable of crossing the summit of the resistance-displacement curve [12].

The most frequently used load pattern in the literature is the fixed one, which assumes a constant load pattern during the analysis. For instance: uniform pattern, inverted triangular or standard code-based pattern, generalized power distribution and multimodal pattern. Adaptive pushover analysis has also been proposed that utilizes a self-correcting load distribution based on the internal story resistance of the system [16,12,26,27]. Since the pushover analysis is based on the static application of lateral story forces (or displacements), dynamic strain rate effects and system degradation or deterioration are not captured

For frame structures whose seismic behavior is essentially dominated by their first mode of vibration and have been designed based on the well known weak beam-strong column concept, the inverted triangular pattern is a simple choice which can result in a reasonably acceptable approximation of actual seismic behavior. For other structures using a single pattern is expected to lead to an increased error. NEHRP seismic rehabilitation guidelines require application of at least two vertical distributions of lateral loads, namely uniform and modal patterns [23].

Finally it is noticed that, there is no general agreement on the best choice of the lateral loads/displacement pattern to be used for pushover analysis. In fact due to the inherent approximation involved in pushover analysis procedure, increased complexity of the applied pattern may not guarantee their superiority over more simplified patterns. In the present study a lateral force distribution with a inverted triangular shape will be used with pushover analysis.

Selecting and scaling of earthquake ground motions

Inelastic dynamic response is quite sensitive to characteristics of input ground motions. If no record has been given for the site, the designer should select some records from a database or generate artificial records based on a reasonable method. Spectrum-compatible generated accelerograms are common artificial records for the seismic analysis of structures. However as in most cases they do not represent any physical event, care should be taken in interpreting results from analyses which are done using these records.

Use of actually recorded ground motions is recommended. There are various parameters affecting the selection of a set of suitable records for design and assessment of a specific structure at a certain site. For instance the mechanism, distance and depth of source event; travel path, magnitude, peak ground parameters and duration of shaking. One of the parameters that can provide information on the relative frequency content and duration of ground motions as well as structural response characteristics is the ratio of peak ground acceleration and velocity, a/v, suggested by Zhu [28].

With regard to scaling of the records for the proposed method, whenever elastic response spectra for the site are available for given probabilities of exceedance, the input motion will be scaled to match the spectrum intensities of the corresponding velocity spectra. Otherwise, the design spectrum specified by the applicable code can be used. The suggested method for the scaling of the record is based on narrow-band spectrum intensity calculated in the region of the expected fundamental period of the structure under consideration.

THE OVERSTRENGTH ISSUES

Due to the inevitable sources of uncertainties involving various aspects of the design, such as loads, material properties, applied theoretical and analytical procedures and construction of the structure, it is essential to provide adequate safety margins in satisfying design goals. In developing seismic design strategies these safety margins are considered in a variety of ways by means of different terms as load and overstrength factors or partial safety factors. Qualification and quantification of safety margins, which is rather cumbersome issue, can be found elsewhere [29-31]. Only some important practical considerations with regard to the application of the suggested methodology will be pointed out in the following. These are mainly related to the strength of constituent materials with regard to the calculation of the required strengths and supplied capacities of members.

As material properties are not known precisely but vary between probable limits the choices of these properties in inelastic analysis needs to take into account the purpose of the application of computed strength. Different codes use different definitions of material strength. For instance in design provisions according to ACI 318 a characteristic strength is adopted which corresponds to the 5 percentile limit of measured strength. However in the European code [5] in addition to the characteristic strength (f_{ck} or f_{yk}) a design strength is defined as the characteristic value divided by partial safety factors for materials ($f_{cd} = f_{ck}/\gamma_c$ or $f_{yd} = f_{yk}/\gamma_s$). Clearly above definitions are dependent on the design strategies implied in these codes with regard to safety verifications.

As previously mentioned, considering applications of inelastic analysis most of the available literature on this subject is related to assessment of the designed or preliminarily designed structures rather than a procedure involving explicit inelastic design concerns. The most common practice is to use the mean value of strengths along with relevant stress-strain relationship for the materials (concrete and steel in case of reinforced concrete structures). This is probably because of the better representation of the most probable actual strength of the material, which is a rationally acceptable value for assessment purpose, by its mean value. As the general strategy of Eurocodes for safety verification has been accepted in design example presented in this study the design of sections are based on the design strength of materials. Likewise other design strategies can be applied. In performing inelastic analysis in step 6 of the proposed method, the mean value of the material strength should be employed in modeling of the inelastic members. This will help to provide for the uncertainties regarding the strength hierarchy at beam-column joints, as prescribed by the capacity design concept, which is treated in codes by introducing empirical overstrength factors (γ_{Rd}). This is one of the main advantages of the proposed method. It should be noticed in the aforementioned step as the performance check is aimed for inelastic members, it might be unconservative to use the mean strength values. Notwithstanding this fact, to simplify the application of the method the same model can be used for immediate-occupancy (serviceability) performance level in step 5. This is because limited inelastic behavior is expected in this level and also it is assumed that the acceptance criteria set forward in this step takes into account most probable actual strengths of materials (mean value).

APPLICATION EXAMPLE

A trail application of the method for a six-story frame structure can be found elsewhere [8]. To cover the most dominating range of medium rise building structures, which are potentially attractive for the application of inelastic analysis approaches for design, here another RC building case study with 10-story frame is considered. The building that was first designed to the elastic analysis-based EC8 procedure, was redesigned to the previously described methods, and then assessed by subjecting them to various appropriately scaled input accelerograms. In the following sections various features of the design and assessment of this frame are discussed.

Description of the frames

The ten-story frame studied is part of RC buildings within importance category III according to EC8, assumed to be the central one in a series of frames, equally spaced at a distance of 3m. The geometrical characteristics of the system are shown in Figure 2. Square cross sections are assumed for columns. As it can be seen, the structure is symmetric in terms of stiffness in elevation. The ductility class considered for the design according to EC8 is DC "M". The frame is designed for a design ground acceleration of 0.25g assuming class A soil (rock or stiff deposits) for the site of the buildings and brittle non-structural elements

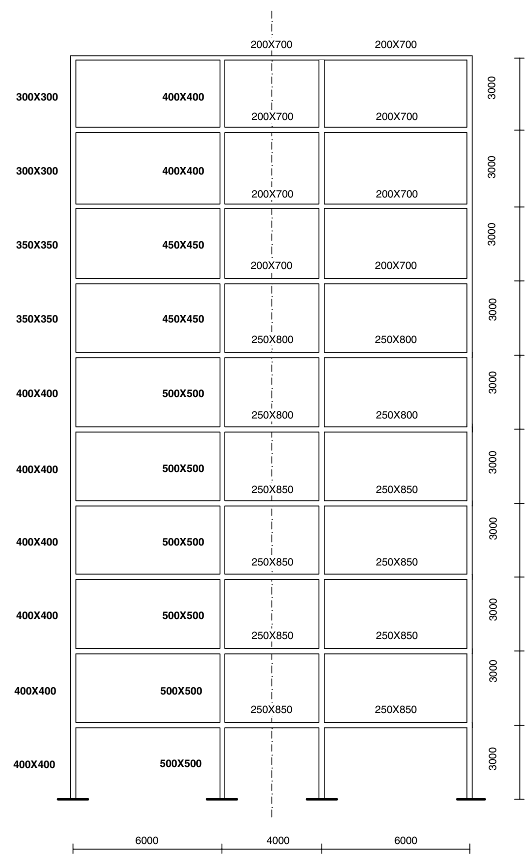


Figure 2, Initial geometry of 10-story frame structure (units: mm)

attached to the system. The materials used in the structure is C20/25 (characteristic cylinder strength of 20 MPa) concrete and S400 steel (characteristic yield strength of 400 MPa). The total dead and live loads on the floor slabs are assumed to be 4.0 and 2.0 KN/m^2, respectively.

First the frame was designed and detailed according to EC8 design provisions (EC8 design) using an elastic static analysis. The initial dimensions were taken from a 10-story frame studied by Kappos and co-workers in various research studies concerned with similar design conditions [e.g. 32]. The elastic period for the design was calculated by empirical equations of the code, which amounts to 0.96 second. The behavior (reduction) factor for frame is equal to 3.75 and the design base shear from the code results in 404 KN.

The results concerning the flexural design of beams are shown in Table 1, and those for columns are given in Table 2 together with details of the transverse reinforcement. In line with common practice, it was decided to keep member cross sections the same in every two stories. Elastic analysis based drift calculation, considering the cracked stiffness for beams and columns, indicated that the interstory drifts for the frame amounts to 5.1 mm, which is acceptable in view of the 6.4 mm interstory drift limit from EC8.

With regards to the transverse reinforcement minimum requirement for ductility from the code is controlling the design of the sections inside the critical hinging region of the frame. Therefore 6 mm ties with 80-110 mm (with larger spacing for lower stories) were found to be sufficient in critical regions. Also outside the critical regions 6 mm ties at 300mm (maximum spacing for non-critical regions) are able to resist the required shear demands.

Analytical modeling

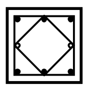
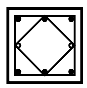
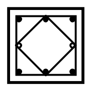
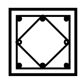
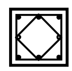
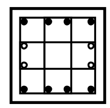
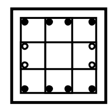
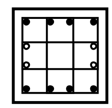
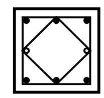
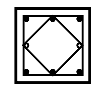
Inelastic analyses of the structure have been carried out using IDARC 4.0 [27], a computer program that adopts a member by member modeling approach. Inelastic beam and column members are modeled as inelastic elements with spread plasticity and yield penetration. The cyclic behavior of end cross-sections is represented by a degrading moment-curvature relationship built on a non-symmetric trilinear envelope curve, which is created using a simple fiber model and cross-sectional and material specifications.

In constructing the model columns are modeled as elastic elements with an effective rigidity EI_{eff} $=0.80EI_g$, where EI_g is the gross section value. The base of the columns is also modeled allowing for inelasticity to occur. Mean values of strengths of the materials are used for calculating the resistance of inelastic members. Also for the reinforcing steel a value of 594 MPa for ultimate strength, 1% for the strain at the initiation of strain hardening, and a strain hardening modulus equal to 1/60 the initial modulus of

Table 1. Results for flexural design of beams based on conventional EC8 method

Story		1 & 2	3 & 4	5 & 6	7&8	9&10
Dimension (b x h, mm)		250 x 850	250 x 850	250 x 800	200 x 700	200 x 700
External supports (cm^2)	TOP	4ϕ16 (8.04)	4ϕ16 (8.04)	3ϕ16+1ϕ14 (7.57)	4ϕ16 (8.04)	4ϕ12 (4.52)
	BOT	3ϕ16 (6.03)	3ϕ16 (6.03)	3ϕ16 (6.03)	2ϕ14+1ϕ12 (4.21)	4ϕ12 (4.52)
Internal supports (cm^2)	TOP	5ϕ16 (10.05)	4ϕ16+1ϕ14 (9.58)	4ϕ16 (8.04)	4ϕ14+1ϕ12 (7.29)	5ϕ12 (5.65)
	BOT	3ϕ16+1ϕ14 (7.57)	3ϕ16+1ϕ12 (7.16)	3ϕ16 (6.03)	2ϕ14+1ϕ12 (4.21)	4ϕ12 (4.52)

Table 2 Results for design of columns designed based on conventional EC8 method

	Exterior Columns					**Interior Columns**				
Story	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10
b (mm)	400	400	400	350	300	500	500	500	450	400
h (mm)	400	400	400	350	300	500	500	500	450	400
Longitu-dinal Re.	3ϕ20	2ϕ20+ 1ϕ16	2ϕ20+ 1ϕ16	2ϕ18+ 1ϕ16	3ϕ16	2ϕ24+ 2ϕ22	4ϕ20	4ϕ20	2ϕ20+ 1ϕ22	2ϕ20+ 1ϕ22
As=A's (cm^2)	9.42	8.29	8.29	7.1	6.03	16.65	12.57	12.57	10.08	10.08
Arrangem-ent of Re.										
Trans. Re.	ϕ10@110	ϕ8@110	ϕ8@110	ϕ8@90	ϕ6@70	ϕ10@140	ϕ8@140	ϕ8@140	ϕ8@120	ϕ8@110

elasticity, are assumed. The effect of slab reinforcement lying within the effective width of the flanged beams is taken into account in the design of columns, assuming 0.20% reinforcement parallel to the beam. For the hysteretic behavior, the nominal stiffness and energy-based strength degradation of IDARC4.0 with no pinching effect were selected, reflecting the good hysteretic behavior of members designed to modern code provisions.

Regarding the modeling of ground motion while the "life-safety" earthquake is described by the unreduced code spectrum, the immediate-occupancy earthquake was taken as 1/2.5 the code spectrum, along the lines suggested in EC8. Hence the intensity used for designing the yielding regions (Step 1 of the method) was taken as 2/3 of the previous value, i.e. the elastic EC8 spectrum divided by 3.75; this conveniently corresponds to the behavior factor (q=3.75) used for designing the structures to EC8 ductility class "M", and allows for more meaningful comparisons between the proposed method and the standard EC8 design.

Natural earthquake ground motions were selected from Imperial College Strong Motion Database (ICSMD) and scaled to EC8 elastic spectra with 0.25g peak ground acceleration using modified spectrum intensity method [33]. The selected records are shown in Table3. More information regarding the selection and scaling criteria for ground motions can be found in Kappos [8].

Table 3. Earthquake ground motions and scaling factors for 10-story frame

Earthquake		Abb. name	Station	Comp.	Mag. (Ms)	ED (km)	PGA (g)	PGV (cm /s)	PGA/ PGV	Duration (sec)	Site Geology	Scaling Factor
1	Imperial Valley, 15/5/1940	**ELC**	El Centro	S00E	7.1	8	0.348	33.5	1.04	26.5	Stiff soil	0.58
2	Northridge, 17/07/1994	**NORE**	Aleta Fire Station	N90E	6.7	6	0.344	40.4	0.85	16	Stiff soil	0.51
3	Friuli, Italy, 6/05/1976	**FRIT**	Tolmezzo-Ambiesta 1	N90E	6.5	27	0.313	32.09	0.96	10	Rock	0.98
4	Kobe, Japan, 17/01/1995	**KOBL**	Kobe University	N90E	7.2	25	0.307	31.69	0.97	13.5	Rock	0.57
5	San Fernando 9/2/1971	**SFERT**	Old Ridge Road	N69W	6.5	24	0.274	26.03	1.05	18.5	Rock	0.85
6	Loma Prieta, 17/10/1989	**LPRL**	Presidio	N90E	7.1	98	0.201	32.42	0.62	19.5	Rock	0.80
7	Erzincan, Turkey, 13/3/1992	**ERZN**	Metrologica l Station	N-S	6.9	13	0.504	76.27	0.66	10.5	Stiff soil	0.40

Abbreviations: Abb.= Abbreviated; Comp.=Component; Mag.=Magnitude; ED= Epicentral distance

Inelastic dynamic approach

Flexural design of beams: Due to the way in which the immediate-occupancy (serviceability) earthquake was defined, the flexural reinforcement in the beams and at the base of the ground story columns was the same as in the EC8 design. A first series of analyses involving the partial inelastic model were then carried out by applying the first three records of Table 3, scaled to the intensity of the immediate-occupancy (serviceability) earthquake. The maximum response in terms of rotational ductility factors, damage index (Park-Ang [27]) and interstory drift are shown in Figure 3 and the corresponding peaks are summarized in Table 4.

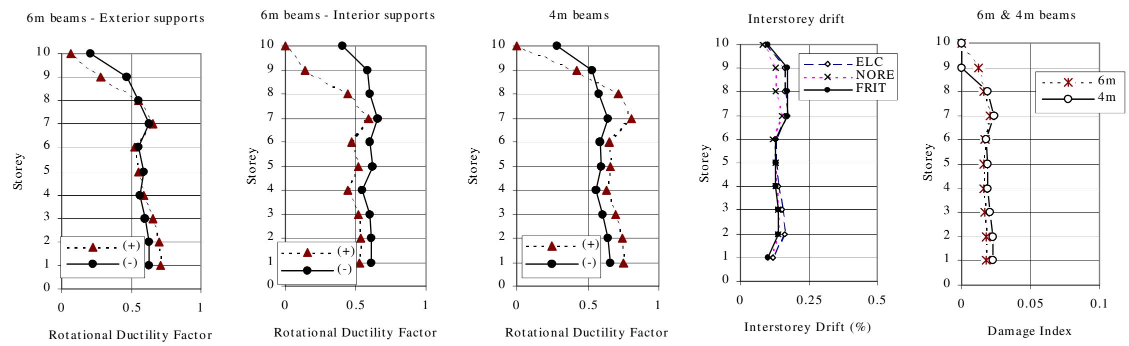


Figure 3. Maximum response under 3 immediate-occupancy (serviceability) level earthquakes for partial inelastic modeling of 10-story frame, (Drift values are shown for each earthquake).

It can be seen from the Figure 3 and Table 4 that the maximum value for interstory drift ratio does not exceed 0.17%, which is acceptable for most infill types. Moreover, maximum rotational ductility factors in beams and the base of the columns are less than 0.85, which means that under immediate-occupancy level earthquakes yielding will not occur in the members. Hence, no modification of the original design was necessary and the design process continued into step 6 of the proposed method.

Table 4. Maximum responses of partial inelastic model of 10-story frame under 3 immediate-occupancy (serviceability) earthquakes

Earthquake	Max. Interstory Drift (%)	Top displace-ment (mm)	Global Drift (%)	$\mu_{\theta max}$ of beams		$\mu_{\theta max}$ of columns*	Damage index		
				(+)	(-)		beams	Columns	Global
ELC	0.17	40.26	0.13	0.77	-0.66	0.49	0.023	0	0.019
NORE	0.15	37.22	0.12	0.71	-0.6	0.42	0.022	0	0.019
FRIT	0.17	38.62	0.13	0.81	-0.66	0.41	0.024	0	0.019

* For the base of columns

Flexural design of column: Critical values for the flexural design of columns (M-N combinations) are calculated, as described in step 7, by means of a post-processing program that derives the critical values from the time history outputs produced by IDARC4.0 for columns. Flexural design of columns is based on the conventional design procedure using design values for the strength of materials (concrete strength $f_{cd}=f_{ck}/1.5=13.33$ MPa and steel strength $f_{yd}=f_{yk}/1.15= 347.8$ MPa). It is noted that, since the behavior of the structural system is deemed to have been modeled in a realistic way by introducing realistic material specifications for beams (mean values for strengths, strain hardening for steel, and hysteretic behavior), no further overstrength (γ_{Rd} factor) was introduced in the design of columns.

Transverse reinforcement design: Critical shear demands from the life-safety level earthquake set, resulting from Step 6 of the procedure, are shown in Table 5 for columns and critical regions of beams (these were then multiplied by the γ_{Rd} =1.10 factor).

Table 5. Maximum shear forces for partial inelastic model from time history analysis under 3 life-safety earthquakes

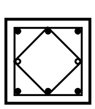
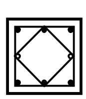
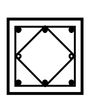
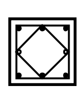
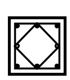
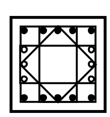
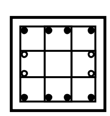
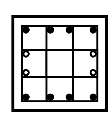
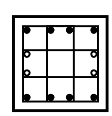
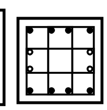
	Columns		Beams	
Story	Exterior	Interior	Exterior	Interior
1 & 2	122.1	252.5	159.9	204.1
3 & 4	122.5	244.2	158.6	196.3
5 & 6	117.6	223.3	143.05	165.4
7 & 8	99.16	196.7	116.12	128.59
9 & 10	54.59	121.6	94.4	99.06

Units: KN

Shear demands for beams were generally larger than corresponding values from the code procedure (note that mean beam strengths are directly considered in the inelastic analysis, as opposed to characteristic strengths in the EC8 procedure). Only small changes in design of exterior beams were needed at the 6 lower stories (90mm spacing instead of 110mm in the first four stories and 100mm instead of 110mm in the 5th and 6th stories), however for interior beams the required increase in shear capacity was about 20% up to 40% of the original EC8 design at the 6 lower stories. The latter resulted in a reduction of tie spacing from 110mm to 70mm for four lower stories and from 110mm to 80mm for the 5th and 6th stories (all ties are 6mm in size). It should be recalled that increased shear demands are expected for beams in the suggested methodology compared with the standard design procedure in EC8, due to using mean strength of materials, the contribution of slab reinforcement as well as the effect of strain hardening in beam reinforcement. In fact these effects are more pronounced wherever excessive inelastic deformations occur. On the other hand it is clear that these effects cannot be captured in elastic analyses and are usually treated by overstrength factors.

Longitudinal and transverse reinforcement design for columns is summarized in Table 6. As it can be seen from Table 2 and Table 6, the cross-section dimensions of exterior columns at the first and second stories and interior columns at the 4 upper stories were increased by 50 mm. This was because of an attempt to

Table 6. Results for design of columns after dynamic analysis of partial inelastic model

	Exterior Columns					Interior Columns				
Story	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10
b (mm)	450	400	400	350	300	500	500	500	500	450
h (mm)	450	400	400	350	300	500	500	500	500	450
Longitudinal Re.	3ϕ20	3ϕ22	2ϕ20+ 1ϕ22	3ϕ22	2ϕ18+ 1ϕ20	5ϕ22	4ϕ22	4ϕ24	4ϕ24	2ϕ22+ 2ϕ20
As=A's (cm^2)	9.42	11.4	10.08	11.4	8.23	19.01	15.21	18.1	18.1	13.88
Arrangement of Re.										
Trans. Re.	ϕ8@130*	ϕ8@150	ϕ6@130	ϕ6@140	ϕ6@110	ϕ8@160*	ϕ8@190	ϕ6@200	ϕ6@200	ϕ6@190

* Except at the base of the columns

keep the reinforcement ratios similar in all designs, hence a change in column dimension was made whenever the required ratio for main longitudinal reinforcement exceeded about 2%. Following a change of dimensions, ideally a second analysis is needed to take into account the effect of this change on the force distribution of columns and also other response parameters in beams. However, the resulting difference in column demands from small cross-section changes is typically not significant and is often disregarded in practice. In addition to the above-mentioned changes in cross sectional dimensions an increase of longitudinal reinforcement of columns also appears in most of the columns, which is again larger for interior columns than exterior ones, compared with the original EC8 design.

On the other hand, application of steps 8 and 9 of the proposed method results in considerable relaxation of transverse reinforcement in columns compared with conventional code-based design (compare Table 2 with Table 6). 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).

Inelastic static approach

Target displacement: For the 10-story frame considered here the inelastic static procedure was applied using the same analytical model as described above. The calculated base shear versus global drift relation for the partially inelastic model of the frame is depicted in Figure 4 together with its bilinear approximation. For 10-story structure, having an initial elastic period T_i = 0.97 sec, the effective fundamental period (T_e) was found to be 1.04 sec, which results in a target displacement value of about 106 mm (corresponding to a global drift ratio of 0.35%) for the life-safety earthquake.

As it is seen the effective period from Equation 1 is very close to the initial elastic period. In applying the aforementioned equation, while using base shear top displacement curve resulting from pushover analysis of the partial inelastic model of the structure, it was found difficult to differentiate between K_i and K_e as defined in Figure 1. This is primarily because the analytical model does not take into account the inelastic behavior (mainly due to crack development) of the columns and therefore the initial stiffness is not accurately accommodated. However, a trial of pushover analysis of the frames with full inelastic modeling for all members showed that the calculation of the effective period using equation 1 still could be problematic. To be consistent with utilizing the curve derived from pushover analysis of the partial inelastic model, rather than a full inelastic model as intended in FEMA 273 [23] proposal of equation 1, the elastic fundamental period is also calculated using the same model by an initial eigenvalue analysis capability of IDARC program. This results in a conservative target displacement.

Seismic demands: Following the calculation of the target displacement, the frame was then pushed to the calculated top drift value (0.35%) and the force demands in the columns were calculated. The proportioning of the column reinforcement and the detailing of the members were carried out in a similar fashion as in the dynamic inelastic method, though the calculation of critical M-N combinations is carried out in a much simpler way than in the dynamic method. Shear demands in beams from pushover analysis were in good agreement with those from inelastic dynamic analysis Therefore the discussion presented in previous section regarding shear demands in beams from dynamic approach applies for the static approach too.

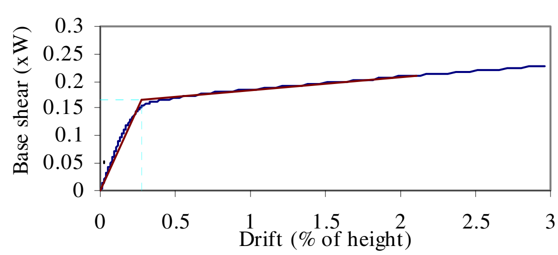
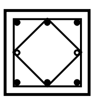
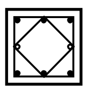
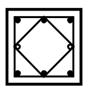
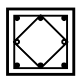
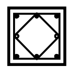
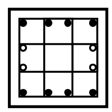
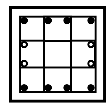
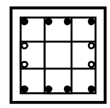
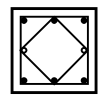
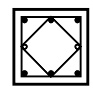


Figure 4. Base shear vs. global drift for partial inelastic model of 10-story frame

Design of columns: Design of columns for flexure and axial loads resulted in a change of dimensions only in the first and second stories with respect to the corresponding EC8 design. Accordingly cross section dimensions of all columns increased by 50 mm. In terms of longitudinal reinforcement generally an increase in the amount of reinforcement can be seen compared with EC8 design. However a reduced demand appears for exterior columns of the four upper stories. The summary of design results is reported in Table 7.

Table 7. Results for design of columns after pushover analysis of partial inelastic model

	Exterior Columns					Interior Columns				
Story	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10
b (mm)	450	400	400	350	300	550	500	500	450	400
h (mm)	450	400	400	350	300	550	500	500	450	400
Longitudinal Re.	2ϕ20+ 1ϕ22	3ϕ20	2ϕ18+ 1ϕ20	2ϕ16+ 1ϕ18	2ϕ14+ 1ϕ16	4ϕ24	2ϕ24+ 2ϕ22	2ϕ22+ 2ϕ20	2ϕ22+ 1ϕ20	2ϕ20+ 1ϕ22
As=A's (cm^2)	10.08	9.42	8.23	6.56	5.08	18.1	16.65	13.88	10.74	10.08
Arrangement of Re.										
Trans. Re.	ϕ8@130*	ϕ8@160	ϕ6@150	ϕ6@140	ϕ6@110	ϕ8@200*	ϕ8@200	ϕ6@190	ϕ6@190	ϕ6@160

* Except at the base of the columns

Transverse reinforcement for columns in this method is more or less similar to that from the dynamic analysis-based method and again shows considerable relaxation compared with the conventional code-based method. The pushover analysis alternative of the proposed method has produced higher demands in the columns of the lower stories than the upper ones, compared with the dynamic analysis. This should be attributed to higher mode effects, which are not usually captured in simple pushover analysis.

Assessment of designed structures

General

To assess the seismic performance of the frames designed to the new procedure, as well as the conventional one (EC8), all the frames were modeled as full inelastic systems and analyzed for the seven earthquake ground motions of Table 3 with different levels of intensity. To check the reliability of the proposed procedure, an additional stronger seismic intensity was also considered, associated with a less frequent earthquake having a probability of exceedance of about 2% in 50 years. This may be referred to as “collapse-prevention” or “survival” seismic level and the simple practice of doubling the life-safety earthquake intensity was selected. The effect of variation of axial load on the strength and stiffness of the columns is neglected in IDARC4.0 and the moment curvature relationship of the columns is evaluated at the average axial loading corresponding to gravity loads. The effect of confinement on concrete strength and ductility in columns is taken into account using the confined concrete model suggested by Kappos [34]. This model was introduced into the IDARC4.0 program. Strength values based on mean material properties are assumed for all members, as typically done in deterministic assessment studies. All materials are modeled with mean strength values.

Immediate-occupancy performance
The results for immediate-occupancy (serviceability) assessment indicate that for the 10-story structure maximum interstory drift varies from 0.17% to 0.20% between three design alternatives. The maximum global drift in this case amounts to about 0.15%. No yielding occurred in the beams or columns and the maximum ductility factor for beams was 0.85.

Life-safety and collapse-prevention performance
Interstory drift: Interstory drift values are compared in Figure 5 for 3 different design alternatives under the life-safety and collapse-prevention events (7 records considered). There exists generally more similarity between the results of the conventional and the proposed static-based method (PSM). In terms of average values the proposed dynamic-based method (PDM) indicates an improved distribution of interstory drift throughout the height of the structure, whilst the overall maximum value has also been reduced from 1.50% for the conventional and static-based design alternative to 1.27% for the dynamic-based one. Nonetheless all frames can be regarded as acceptable in terms of drift values.

Damage indices: Figure 6 summarizes story damage indices (Park-Ang [27]) for beams and columns in the design alternatives, for two levels of earthquake. It is noticed that in the frames designed using the proposed dynamic procedure slightly more damage has occurred in the beams compared with the EC8 conventional procedure. On the other hand, average damage values in columns designed to the proposed dynamic method are almost half the corresponding values for the conventional method (with the exception of the first and last stories). This performance is in agreement with the capacity design concept for frame structures and demonstrates the relative advantage of the proposed method.

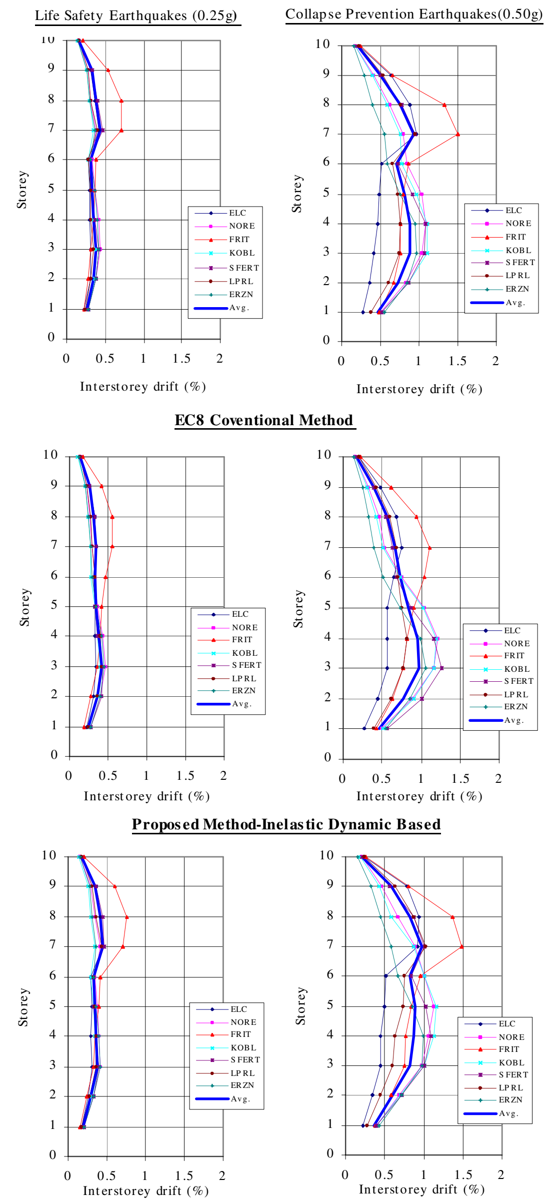


Figure 5. Interstory drift ratio for 10-story frames designed based on conventional and proposed methods under seven life-safety and collapse-prevention earthquakes

Of particular importance are the very low requirements in the columns under twice the design earthquake, indicating the feasibility of drastically reducing the confinement reinforcement above the base of the structure. Once more the aforementioned similarities between different design alternatives can also be seen in terms of story damage indices. The static alternative of the proposed method produces roughly the same story damage distribution in the frame as the conventional method.

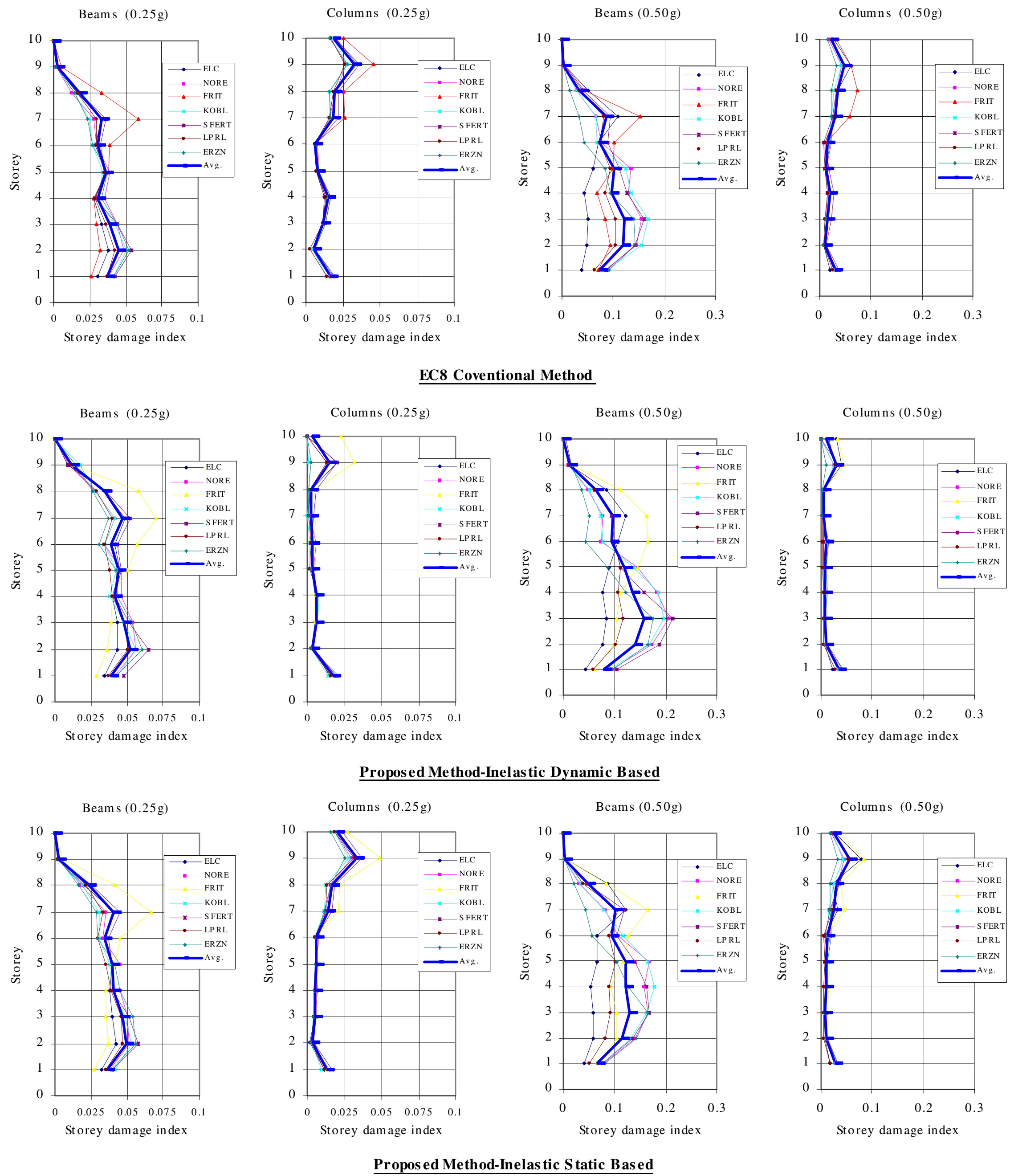


Figure 6. Story damage indices for beams and columns in 10-story frames designed based on conventional and proposed methods under seven life-safety (0.25g) and collapse-prevention (0.50g) earthquakes).

Shear performance

Regarding the shear assessment of beams under the collapse-prevention event the minimum safety factor (=capacity/demand ratio) for EC8 design is found to be 0.96, in the interior beams at the 2 lower stories. In view of large rotational ductilities (more than 4) in aforementioned locations it is likely that the concrete contribution to the shear capacity becomes negligible [32]. Taking this effect into account results in even lower safety margins for shear in EC8 frames (0.91 instead on 0.96 above). This indicates a possible local shear failure for the EC8 frame, which extends up to the 6^{th} story in interior beams. For frames designed according to the proposed method (static or dynamic alternatives) the shear demands were well contained with the provided capacity, using step 8 of the method. For the proposed dynamic method the minimum safety factor in the most critical location, without considering concrete contribution is 1.21. It can be seen that the proposed methods have greatly improved the shear performance of the beams.

Plastic hinge distribution

With regard to the plastic hinge distribution as it is shown in Figure 7, the static alternative of the proposed method as well as the conventional method have not been able to prevent plastic hinge formation in columns under the most critical earthquake (Friuli, see Table 3) at life-safety level. This is clearly undesirable behavior based on the capacity design concept. However, the proposed dynamic method has successfully removed the possibility of column hinging under the life-safety earthquakes. It should be recalled that the Friuli earthquake is one of the 3 earthquakes used in the design of the frames based on the proposed dynamic method.

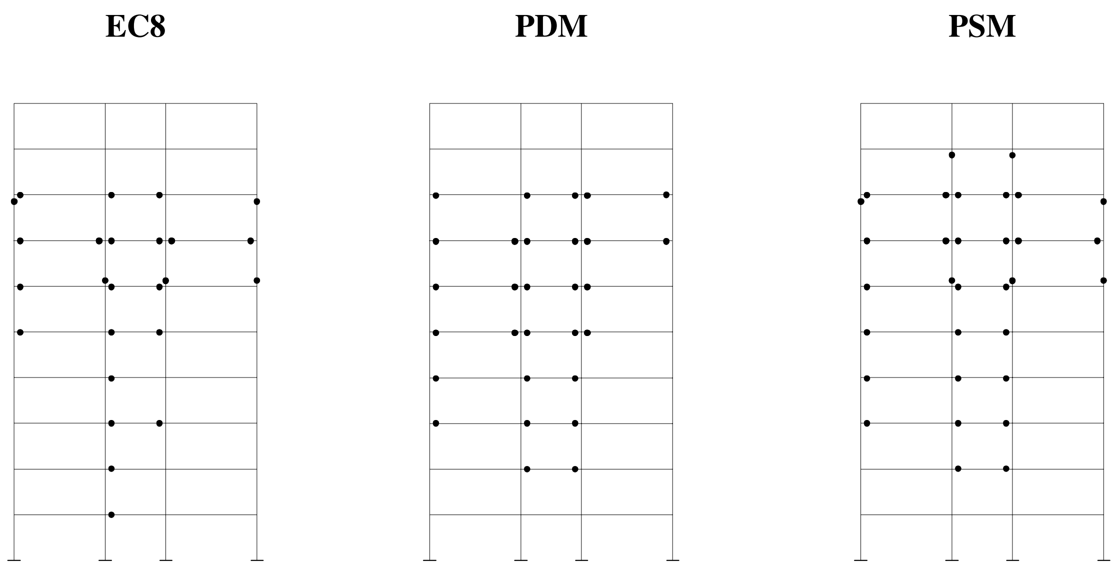


Figure 7. Plastic Hinge Distribution in 10-story frames designed to EC8 and Proposed Dynamic and Static Methods (PDM, PSM) for the most critical earthquake (FRIT) at life-safety level (0.25g).

Probably the most notable differences can be seen in the distribution of plastic hinges throughout the frames under collapse-prevention earthquake level. It is clear from Figure 8 that the frame designed according to the proposed dynamic method has a larger safety margin against the collapse limit state than its simpler pushover alternative, while both of them show improved performance compared with the EC8-based design. For the EC8-based design frame, the majority of column ends are hinged at the 7^{th} and 8^{th} floor under all earthquakes and a story failure mechanism is imminent. This situation is improved in the frames designed using the proposed method, more specifically for dynamic-based design. In Figure 8 the plastic hinge patterns are shown for 3 most critical earthquakes one of which, the Loma Prieta record (see Table 3), had not been considered in the design of the structure.

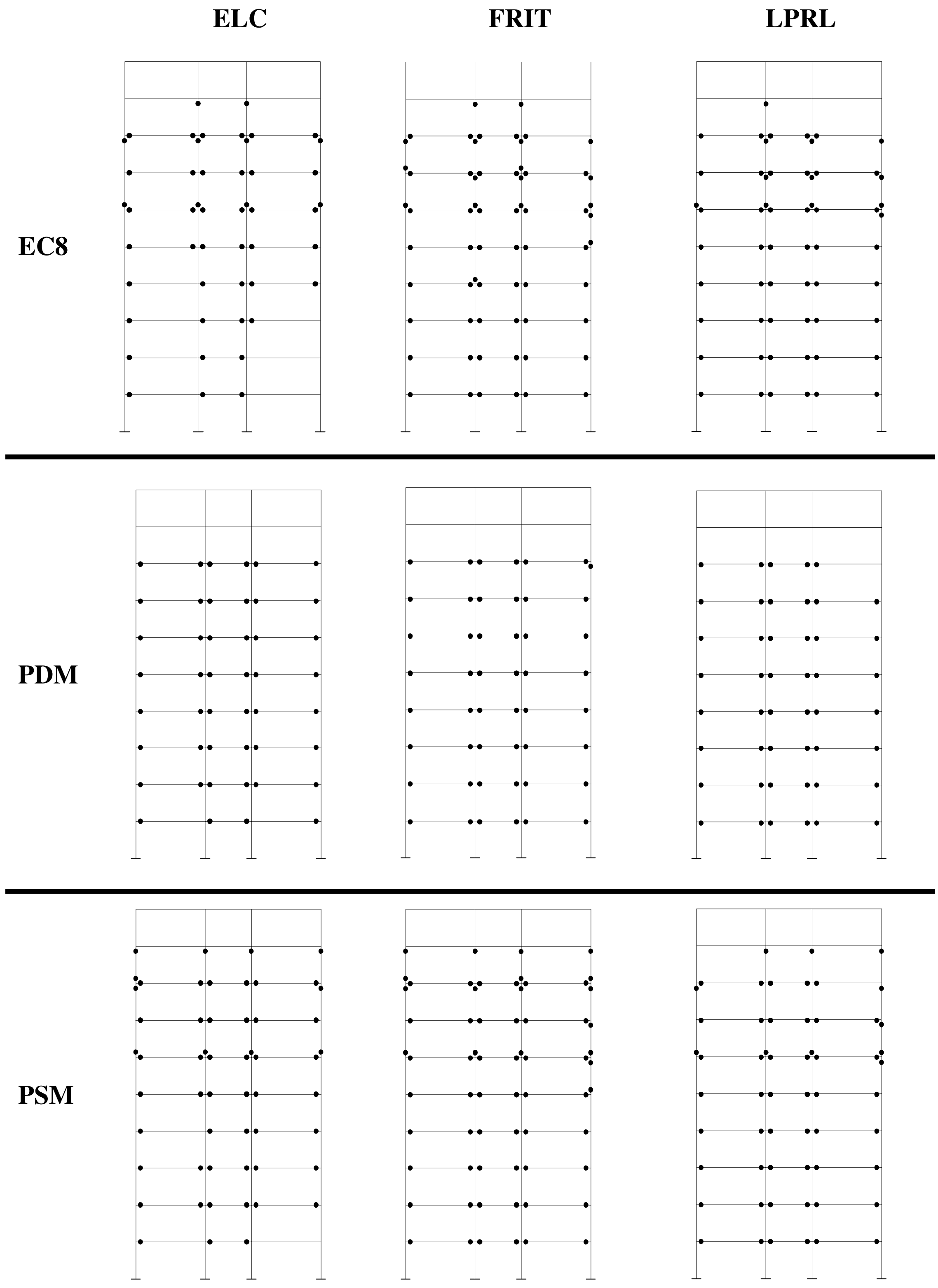


Figure 8. Plastic hinge distribution in 10-story frames designed to EC8 and Proposed Dynamic and Static Methods (PDM, PSM) for the 3 most critical earthquakes at collapse-prevention level.

CONCLUSION

A design methodology based on inelastic analysis of a partial inelastic model of the structure was proposed in this study. Two alternative forms of the method were considered, i.e. nonlinear dynamic analysis-based and nonlinear static analysis-based procedures. The method provides for many of the uncertainties regarding the strength hierarchy at beam-column joints, such as actual material strengths, strain hardening
and the effect of slab reinforcement, and hence allows for effective application of the capacity design concept. These uncertainties are usually treated in conventional methods by introducing empirical overstrength factors and using rather complicated combination rules. The proposed method also makes possible quantification of damage under seismic action and therefore presents an effective approach for control of the deformation within the structure.

A trial application of the method has been presented in previous studies. Here the method applied to a 10-storey frame and the application and assessment of the procedure were discussed in greater detail. Based on the results of these case studies it can be concluded that the proposed method provides a practical tool in achieving deformation-control objectives of the seismic design philosophy for frame structures. It was shown that the dynamic-based method results in better performance than conventional code-based method. Moreover while inelastic static-based method gives better performance than the conventional method in case of the 6-story frame, with almost similar performance to dynamic-based method, for 10-story frame the similarity between the conventional and inelastic static-based method is more than any of them to inelastic dynamic–based method. This can be attributed to higher mode effects, which are more significant in the 10-story frame than the 6-story one.

In general the dynamic-based method is more complicated and its applications requires more information about input motion as well as the structural specifications. Moreover, sensitivity of force values (i.e. accelerations) in dynamic analysis to variations of input values and structural specifications during the analysis implies that designing of structural components based on maximum force values is debatable. This suggests that direct use of deformation values, which are readily produced within the method, as design parameters might be more appropriate than force values. Hence further development of method to establish valid design rules is recommended.

In contrast, the inelastic static-based method, considering its approximations, appears less demanding and less sensitive than the dynamic one and therefore more attractive as a substitute method to conventional methods. The main advantage of the static version is that it does not involve selection and scaling of earthquake loads for time-history analysis. Improvement of the method to allow for higher mode effects is a key issue for further development.

ACKNOWLEDGEMENT

The research reported in this paper was supported by Ministry of Science, Technology and Research, MSRT, Iran. The author is grateful for the financial support of Halcrow Group Ltd, UK. Any opinions expressed are those of the author and do not necessarily reflect the views of the supporters. The author is also wish to acknowledge the support and advice received from professor A. J. Kappos in initial development of this research study.

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