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A displacement-based seismic design method with damage control for RC buildings

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Abstract. This paper presents a displacement-based seismic design method with damage control, in which the targets for the considered performance level are set as displacements and a damage distribution is proposed by the designer. The method is based on concepts of basic structural dynamics and of a reference single degree of freedom system associated to the fundamental mode with a bilinear behaviour. Based on the characteristics of this behaviour curve and on the requirements of modal spectral analysis, the stiffness and strength of the structural elements of the structure satisfying the target design displacement are calculated. The formulation of this method is presented together with the formulations of two other existing methods currently considered of practical interest. To illustrate the application of the proposed method, 5 reinforced concrete plane frames: 8, 17 and 25 storey regular, and 8 and 12 storey irregular in elevation. All frames are designed for a seismic demand defined by single earthquake record in order to compare the performances and damage distributions used as design targets with the corresponding results of the nonlinear step by step analyses of the designed structures subjected to the same seismic demand. The performances and damage distributions calculated with these analyses show a good agreement with those postulated as targets.

Keywords: displacement-based seismic design; damage control; reference single degree of freedom system; modal spectral analysis; nonlinear behaviour

1. Introduction

Recent destructive earthquakes have produced far more damage than expected; this damage has been partly attributed to the failure of current design codes in estimating the magnitudes of the seismic demands and above all the design deficiencies of the design methods based on forces, rather than methods based on displacements and/or other more objective performance indices. Because of this, research groups throughout the world have focused their efforts on obtaining consistent seismic demands and new design methods capable of taking into account the actual performance of the structures once they incursion into the nonlinear range of behaviour. It is within this context that, the concepts of the so-called performance based seismic design (PBSD) has been revived. Documents such as Vision 2000 (SEAOC 1995) define PBSD as "the selection of design criteria, appropriate structural systems, layout, proportioning and detailing for a structure and its nonstructural

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components and contents, and the assurance and control of construction quality and long-term maintenance, such that at specified levels of ground motion and with defined levels of reliability, the structure will not be damaged beyond certain limiting states or other usefulness limits". In accordance with this seismic design philosophy, several seismic design methods have been developed attempting to control, through proper consideration of the design objectives, structural performance, explicitly considering the different factors and parameters that influence the performance of a structure subject to the seismic demands considered in their design. One of these design approaches, the direct displacement-based design method, DDBD, (Priestley *et al.* 2007) is the most widely known reference for the displacement-based seismic design of structures due to its practical approach using displacements as performance indices. Another performance-based design method, known for the accuracy of its results, is the seismic design method based on direct deformation, SDMBDD, proposed by Kappos and Stefanidou (2010). Current literature contains several other displacement based design methods such as the method of displacement-based seismic design with energy factor (Sullivan 2011), the method using inelastic displacement spectra (Chopra and Goel 2001) and the deformation-controlled earthquake-resistant design of RC buildings (Panagiotakos and Fardis 1999), among others.

This paper proposes a displacement-based seismic design method for building structures based on a formulation that explicitly considers nonlinear behavior and damage control. This formulation approximates the calculation of the structural performance of a building, through concepts of basic structural dynamics and modal spectral analysis rendering a transparent and straightforward direct design method that, due to its efficiency and ease of implementation, may be incorporated in the recommendations of future seismic design codes. To show the applicability and accuracy of the method, the design of three regular reinforced concrete frames 8, 17 and 25 storey and two irregular 8 and 12 storey for a particular seismic demand is presented. To validate the results of the design method the target displacements and damage distributions are compared with the corresponding, obtained from non-linear step by step dynamic analyses. Finally, the application potential of the method proposed and the good accuracy of its results are discussed.

2. Previous work

From the evaluation of the current methods for the displacement-based seismic design of building structures it may be concluded that some are essentially variants of the method proposed by Priestley *et al.* (2007), modifying only some parameters, as is done in the method proposed by Chopra and Goel (2001), where inelastic displacement spectra are used instead of elastic spectra reduced by equivalent damping and others do not, strictly speaking, control displacements but deformations as it is the case of the methods proposed by Kappos and Stefanidou (2010) and Panagiotakos and Fardis (1999). Other displacement-based methods, such as the method proposed by Sullivan (2011), simplify, with restrictions to the type of buildings, the original method of Priestley *et al.* (2007) using energy concepts.

In what follows, due to their relative importance, potential of practical application, and quality of results obtained, the formulations of two of the methods mentioned above, Priestley *et al.* (2007) and Kappos and Stefanidou (2010), are discussed further together with the formulation of the method presented in this paper.

2.1 Direct displacement-based design method

In the DDBD method by Priestley *et al.* (2007), the nonlinear multi-degree of freedom system, MDOF, representing the building, is transformed, using the concepts of the equivalent structure proposed by Shibata and Sozen (1976), into a single degree of freedom, SDOF, system with an effective stiffness, associated to the secant to the maximum displacement of the MDOF system, and an equivalent viscous damping to consider the energy dissipated by the structural elements through hysteresis. Fig. 1 illustrates the fundamentals of this design method.

The steps followed to implement this method are shown in the flowchart of Fig. 2. The application of this method involves the calculation of the design displacement, Δ_d, the equivalent mass, M_e, the effective height, H_e, the ductility, μ, and the yield displacement, Δ_y, of an equivalent SDOF system. The effective period of the substitute structure, T_e, is found using the value of Δ_d in the displacement spectrum associated to the equivalent viscous damping ratio, and from it, the effective stiffness, K_e. With this information, the base shear of the simplified system may be calculated. This base shear is then distributed among all floors in proportion to their masses and assumed displacements. Once the force vector is calculated, the design forces of the elements are determined from a conventional linear static analysis of the structure subjected to the force vector. The final design of the structural elements is defined from a capacity design aimed to guarantee that the mechanism, required to occur under design demands (*i.e.*, weak beam-strong column behavior),

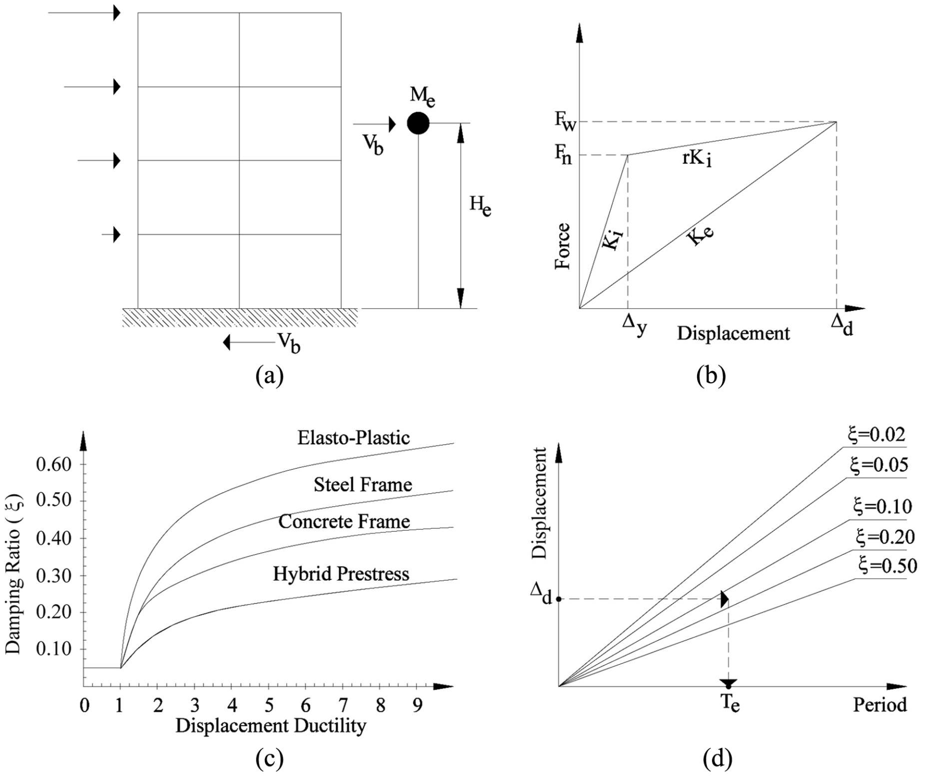


Fig. 1 Fundamentals of the direct displacement-based design after priestley *et al*. (2007): (a) SDOF simulation, (b) effective stiffness *Ke*, (c) equivalent damping vs. ductility and (d) design displacement spectra

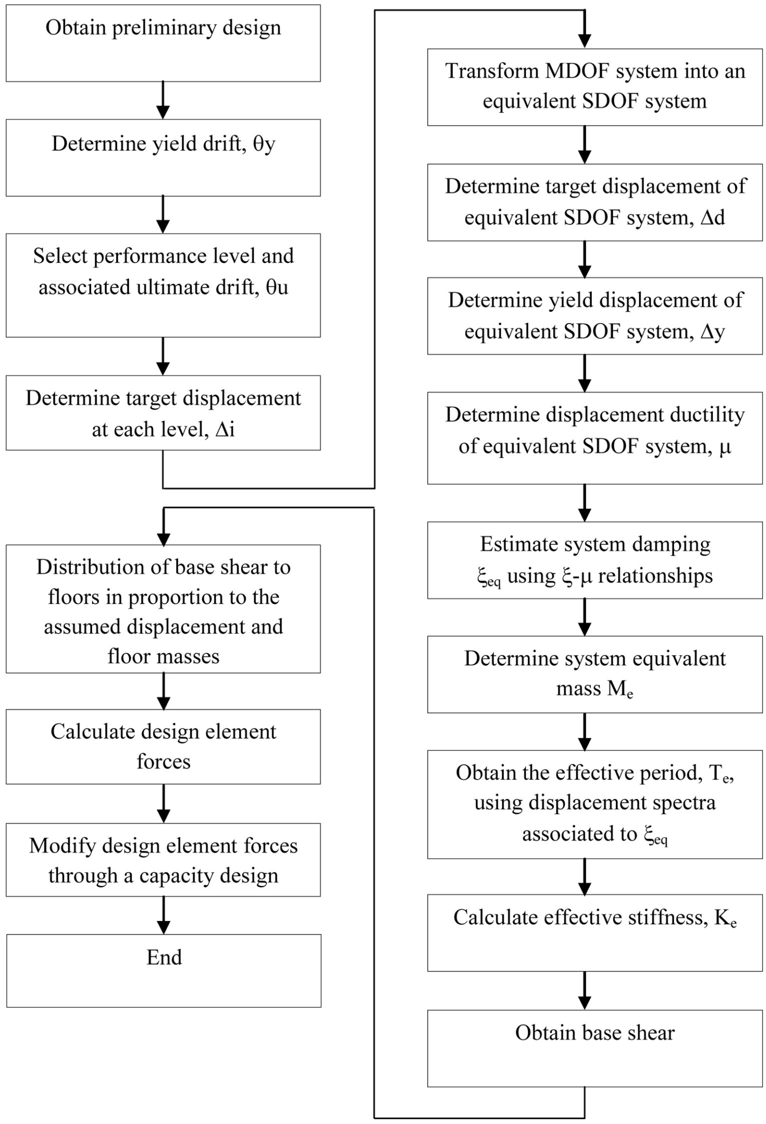


Fig. 2 Flowchart of direct displacement-based design derived from Priestley *et al*. (2007)

is attained. The apparent simplicity of this method has made it very attractive for seismic design in practice, however, its application has some limitations as its formulation is based on the validity of some questionable considerations, e.g. characterizing the behaviour of a nonlinear MDOF structure by the means of an equivalent linear viscous-elastic SDOF system, something which is not always appropriate; defining the equivalent damping ratio as a function only of the ductility demand and of the structural material, which may lead to a poor approximation as it ignores other aspects that intervene, such as structural layout and, most importantly, accepting that the modification of the design forces of the structural elements using capacity design concepts at the end of the displacement-based design process does not modify the essence of the method as the target performance of the designed structure cannot be guaranteed if the design is changed.

2.2 Direct deformation–based approach

In the seismic design method based on direct deformation by Kappos and Stefanidou (2010), the nonlinear behaviour of the structure is approximately and explicitly taken into account through nonlinear step by step dynamic analyses of partially inelastic structural models subject to earthquake records, at least 3 for service conditions and 3 for life safety, scaled in accordance with these two limit states considered in the performance criteria. Considering that nonlinear step by step dynamic analysis is the most reliable analysis tool currently available, as it allows the calculation of the time response and has rigorous mathematical and physical basis, the design method proposed is applicable to irregular buildings with and without shear walls and dual systems with frames and walls. The steps required to implement the method are shown in the flowchart of Fig. 3.

In the application of this method, first a conventional elastic analysis of the structural model

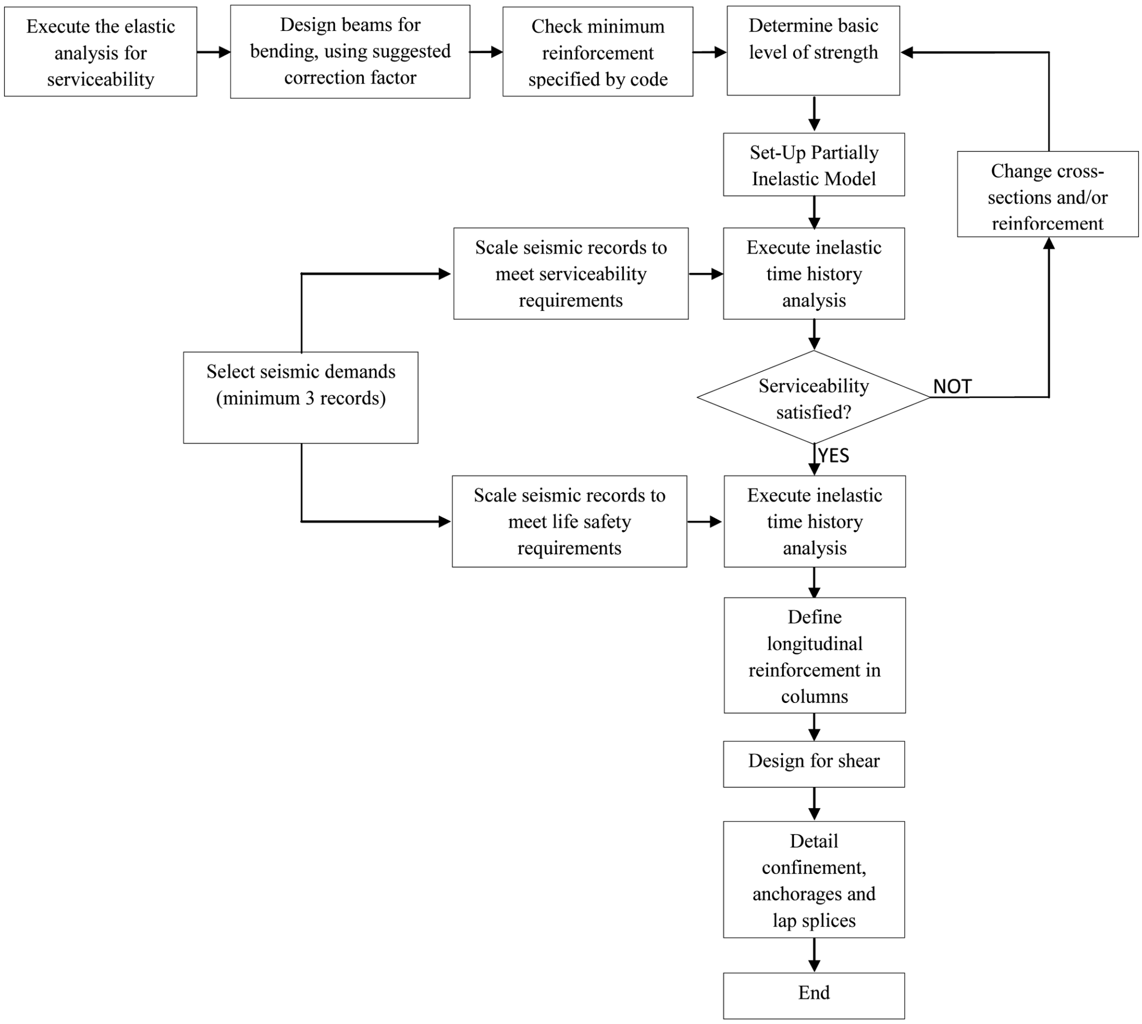


Fig. 3 Flowchart of the seismic design method based on direct deformation derived from Kappos and Stefanidou (2010)

considering reduced inertias in beams due to cracking and gross inertias in columns is carried out to get the design forces of the element sections that will exhibit damage under the serviceability limit state. All beams are designed for bending only, considering a reduction factor to account for a more general definition of the M-θ diagram, (Kappos and Stefanidou 2010), and the requirements of minimum reinforcement specified in the design code. Subsequently, based on the available information of the properties of the structural elements (stiffness and strength), a partially inelastic model is constructed, in which inelastic deformations are accepted for all beams and columns at the base whereas the rest of columns are considered to behave elastically. With this information, nonlinear dynamic analyses of the structural model are carried out, considering as demands a set of no less than three earthquake records representative of the site of the structure and the service limit state considered for the design, checking that the drifts and the ductility demands obtained are within the range of allowed values. If this is not the case, the design proposed must be modified until acceptable values for these parameters are in accordance with those considered as target in the limit state. Once this condition is satisfied, a new set of nonlinear dynamic analyses of the designed model are carried out, using as demands a set of records consistent with the life safety limit state. With the results obtained, the columns are designed for bending and all the structural elements are designed for shear. Finally, the design of all elements is detailed so that the structural system as a whole can develop the inelastic levels considered in the design. Due to the fact that the application of the method involves the use of results of nonlinear dynamic analyses of the structure, its precision is acceptable and, in general better than that of other existing methods. However, it has as drawbacks that, to apply it, the designer must have enough knowledge and experience in the execution and interpretation of these nonlinear analyses and in the selection of the earthquake records required by these analyses, neither of which easy to carry out.

3. Basis of proposed method

The proposed method is based on the assumption that an approximation to the performance of a nonlinear MDOF structure may be obtained from the performance of a simplified nonlinear SDOF reference system, generally associated to the fundamental mode of the buildings (Ayala 2001). The principle of this design method is that the nonlinear capacity curve of a MDOF structure can be approximated by a bilinear curve using the equivalence of deformation energies corresponding to the real and the bilinear capacity curves (see Fig. 4(a)), and, that, in accordance with basic principles of structural dynamics, the bilinear behaviour curve of a reference SDOF system normally associated to the fundamental mode of the structure may be extracted from this capacity curve following the procedure proposed by Freeman *et al.* (1984). This behaviour curve may be conveniently represented in the acceleration-displacement response spectrum (ADRS) format as shown in Fig. 4(b). In this method the capacity curve and consequently the behaviour curve are obtained from the results produced by two conventional modal spectral analyses, one for the elastic phase of behaviour, structure without damage, and the other for the inelastic phase, structure with damage. In these figures, V is the base shear, d is the roof displacement of the MDOF system, Sa is the pseudo-spectral acceleration equivalent to the strength per unit mass, R, Sd is the spectral displacement and K is the stiffness. The subindices u and y indicate the ultimate and yield conditions respectively.

The slope of the first branch of the capacity curve in Fig. 4(a) represents the stiffness properties

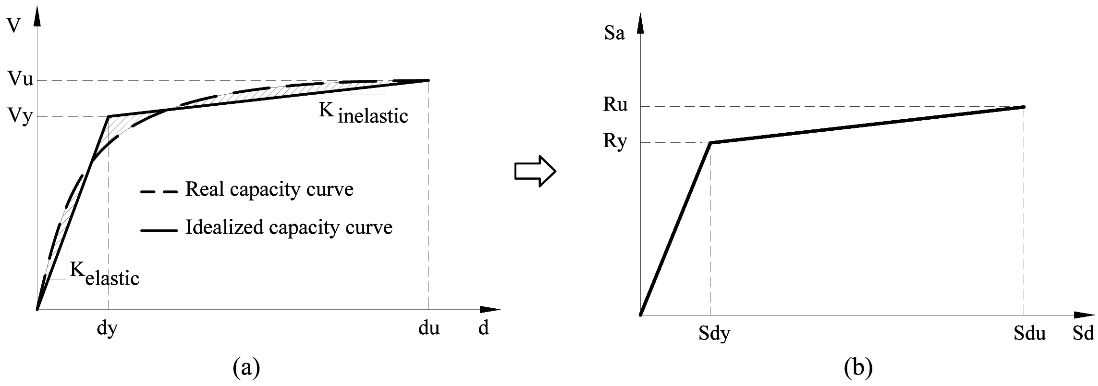


Fig. 4 Transformation of the capacity curve to the spectral space: (a) bilinearization of the capacity curve and (b) behaviour curve in the ADRS format

of the structure in the elastic range associated to the roof displacement and the slope of the second branch, corresponding to the inelastic range. The characteristics of this last branch are defined by the assumed damage distribution associated to the proposed maximum displacement of the given performance level. The yield strength per unit mass, R_y, is the demand level to be met by the structural elements that are assumed to be damaged under design conditions and, the strength per unit mass, R_u, is the demand level for the elements that behave elastically under design conditions.

In the design method proposed, damage on the elements where inelastic incursions are allowed to occur, is assigned by including hinges in the elements of the structural model used in the second analysis phase. Preliminary results of recent work on this topic show that, for the investigated structures, stiffness values corresponding to approximately half the undamaged stiffnesses are acceptable, Mendoza (2011). This paper does not address this problem as its original objective was to propose and validate a practical design method something that could be done regardless of the limitation of using hinges to represent damaged sections.

In practice, the seismic demand is defined by a uniform hazard spectrum for the site of the structure using a procedure consistent with the performance based seismic design philosophy where seismic design levels are defined for rates of exceedance of the index defining the performance level rather than rates of exceedance of earthquake intensity, Niño and Ayala (2012). Fig. 5 illustrates as example uniform hazard strength spectra for a 1/125 rate of exceedance of a ductility of 4 and different values of the post-yielding stiffness ratio, α, calculated for a site in Mexico City.

As mentioned before, the behaviour curve of the reference SDOF system represents the stiffness properties associated to the fundamental mode of a MDOF structure. To consider the participation of higher modes to performance, modal spectral analyses are carried out on the simplified linear models (Ayala 2001). The basis of this approach is the assumption that the modal properties of a MDOF system with bilinear behavior are directly related to the corresponding stiffness in each stage of behavior. This implies that, up to the yield point of the simplified bilinear capacity curve of a MDOF structure, its response is defined the combination of modal contributions associated to its initial (elastic) stiffness (Fig. 6(a)), e.g. using the CQC modal combination rule. Analogously, once the system has reached its damaged state, the combination of modal contributions associated to the post-yielding stage defines the corresponding maximum response (Fig. 6(b)).

Even though the application of modal spectral analysis with conventional modal combination rules

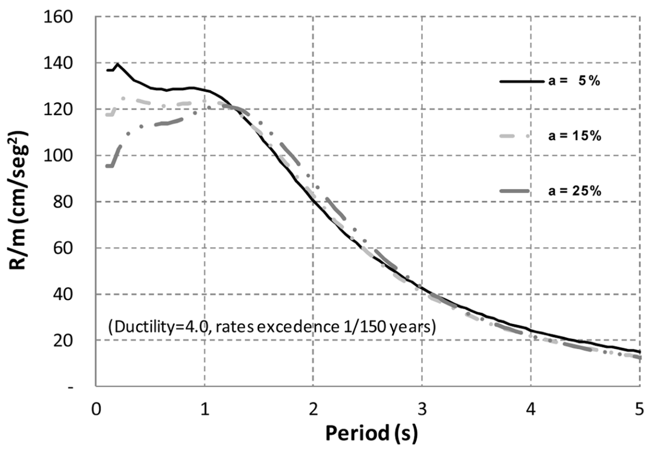


Fig. 5 Uniform hazard spectra for different α

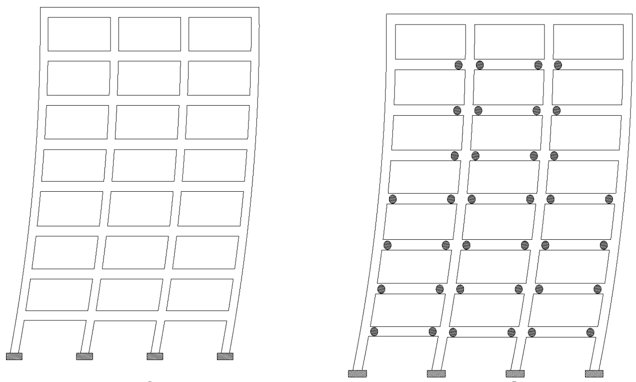


Fig. 6 Dynamic properties of the two stages of behaviour: (a) undamaged model and (b) damaged model

are only valid for elastic systems, the maximum responses obtained with the simplifying consideration that the ultimate and yield base shears and displacements of the system may also be approximately calculated using modal spectral analyses of the elastic models associated to the two stages of behaviour. Furthermore, the uniform hazard spectra to be used in the application of the method proposed are chosen according to the α, and the μ of the design behavior curves, thus, it is implicitly considered that these properties are equal for all modes, an assumption not necessarily true but consistent with the approach of the design procedures that use design spectra in which the seismic performance of the structures is primarily defined by the properties of their fundamental modes.

In accordance with the precedent assumptions, the capacity curve of the MDOF system, in terms of the basal shear, V, and the roof displacement, d, may be directly obtained from the results of the modal spectral analyses of both linear elastic models (Fig. 4). The results obtained from the case studies dealt in this research have shown that using modal spectral analysis and accepted modal combination rules such as the CQC, in the framework of the proposed method, provides a good approximation for the contribution of higher modes in structures which respond primarily in their fundamental mode when subjected to a seismic event. Research concerning the shift of modal dominance and the change in modal shapes during the time history of the response of inelastic structures and its influence on its maximum values is currently underway Ayala and Escamilla (2011).

The validity of the results obtained from the application of the design method proposed, as well as from all current design methods, is limited to structures maintaining an acceptable level of modal regularity, (Ayala and Escamilla 2011) during their evolution to design performance in the inelastic range of behaviour. Modal regularity depends on the structural configuration, the characteristics of the seismic demand corresponding to the limit state considered in design and, the structural models used to represent, in an equivalent manner, structural damage at design conditions. Assigning damage through hinges, such as those used in the application examples, may not be, in some cases, the best option as this idealization neglects the equivalent stiffness that a damaged section has when responding in the inelastic range with a hysteretic behaviour.

4. Design method

Based on the principles of the method proposed, a bilinear behaviour curve of the fundamental mode of a preliminarily designed structure is constructed, assuming a damage distribution consistent with the philosophy of strong column-weak beam and defining the ultimate displacement according to the required design performance level. Thus, the design forces of the elements are calculated considering the control of the target displacement and a damage distribution which ensures an adequate structural performance. The application of the design method involves the following steps:

1. A preliminary design of the structure is obtained using gravity loads and lateral forces in a rough design method based on forces and engineering judgment.

2. A modal analysis of the model of the undamaged structure, constructed using the properties obtained in the previous step, is performed. From the results of this analysis the fundamental period of the structure, T_1, is obtained and from it the elastic stiffness of the reference SDOF system, i.e., the slope of the initial branch of the idealized bilinear behaviour curve of this system.

3. For a given performance level, a rational damage distribution is defined in accordance with the characteristics of the structure, the design demands and the location of areas of greatest demand. The authors suggest to consider the demand-capacity ratios of the structural elements of the preliminary design to define a desired damage distribution under design conditions structural damage at design seismic demands is introduced in the structural model with hinges or rotational springs with reduced bending stiffness, at the ends of the elements where the inelastic behaviour is expected to occur. With this modified structural model, referred as damaged model, a second modal analysis is performed to obtain the corresponding fundamental period, T_2, and thus, the slope of the second branch of the idealized bilinear behaviour curve of the reference system. It may be shown that the slopes (stiffnesses) of the initial and post-yielding branches of the behavior curve are defined in terms of the modal mass associated to the undamaged structure and the corresponding periods.

4. Based on the prescribed story drift for the required performance level, the target roof displacement, du, is defined by the means of the displaced shape of the damaged model.

5. An approximation of the yield roof displacement, d_y, is calculated using the deformed shape

and the properties of the elements obtained from the preliminary design through the following equations (López 2010)

$$d_y = \frac{\delta_n}{\psi_n} \tag{1}$$

where

$$\delta_n = \frac{0.3\,\varepsilon_y L_1\left(\frac{I_{v1}}{L_1} + \frac{I_{v2}}{L_2} + \frac{I_{cn}}{H_n} + \frac{I_{cn+1}}{H_{n+1}}\right)}{h_{v1}\left(\frac{I_{cn}}{H_n^2} + \gamma_o\frac{I_{cn+1}}{H_{n+1}^2}\right)} \tag{2}$$

$$\gamma_o = \frac{\psi_{n+1}}{\psi_n} \tag{3}$$

where: δ_n is the yield interstorey drift at the floor where maximum drift occurs; ψ_n is the drift obtained from a modal spectral analysis of the undamaged structure at the storey where maximum drift occurs, normalized by the maximum roof displacement; ε_y is the yield strain of the reinforcing steel; L_1 is the length of the span to the left of the node nearest to the centre of the storey where maximum drift occurs; L_2 is the length of the span to the right of such node; H_n is the height of the storey where maximum drift occurs; H_{n+1} is the height of the storey above the storey where maximum drift occurs; I_{v1} and I_{v2} are the moments of inertia of the beams in the spans 1 and 2, respectively; I_{cn} and I_{cn+1} are the moments of inertia of the columns of the storeys n and n+1, respectively; and h_{v1} is the beam depth at span 1. Eq. 2 is derived from the equilibrium of a central node of the storey under consideration, assuming that the rotations at all nodes of this and adjacent storeys are approximately equal.

6. Using the results of modal spectral analysis, the target yield and ultimate spectral displacement of the reference SDOF system corresponding to the fundamental mode are calculated, as well as its ductility, μ, defined by Eq. 4

$$\mu = \frac{S_{du}}{S_{dy}} \tag{4}$$

7. From the design displacement spectrum for given μ and α, the ultimate spectral displacement associated to T_1, is obtained. Finally this spectral displacement and the target spectral displacement of the frame, Sd_u, are compared. If the last value obtained is equal or approximately equal to the target, the design is considered satisfactory; otherwise, the initial period of the structure, T_1, and/or the damage distribution needs to be *ad hoc* modified. Alternatively, the required period to satisfy the target displacement can be directly obtained from the displacement spectra. The initial stiffness of the structure is adjusted to match the required period, in such a way that the distribution of stiffness along the height is not significantly modified, thus, the considered displacement shape of the damaged mode in the previous step is not affected.

8. Once the target displacement of the structure is guaranteed, the yield strength, R_y, for the period that satisfies the target displacement is obtained from the inelastic strength spectrum, ISS, corresponding to the values of μ and α previously calculated.

9. The ultimate strength, R_u, of the reference system is calculated using Eq. 5.

$$R_u = R_y[1 + \alpha(\mu - 1)] \tag{5}$$

10. Once the characteristic points of the behaviour curve are defined, the behaviour curve of the reference SDOF system may be drafted, Fig. 7.

11. To obtain the design forces of the elements, three different analyses need to be carried out: a gravity load analysis of the undamaged structure, a modal spectral analysis of the undamaged structure using the elastic design spectrum scaled by the ratio of the strength per unit mass at the yield point of the behaviour curve and the elastic pseudo-acceleration for the initial period, λ_1 (see Fig. 8(a)), and a modal spectral analysis of the damaged structure using the elastic spectrum scaled by the ratio of the difference of ultimate and yield strengths per unit mass and the pseudo-acceleration for the period of the damaged structure, λ_2 (see Fig. 8(b)). The design forces for the

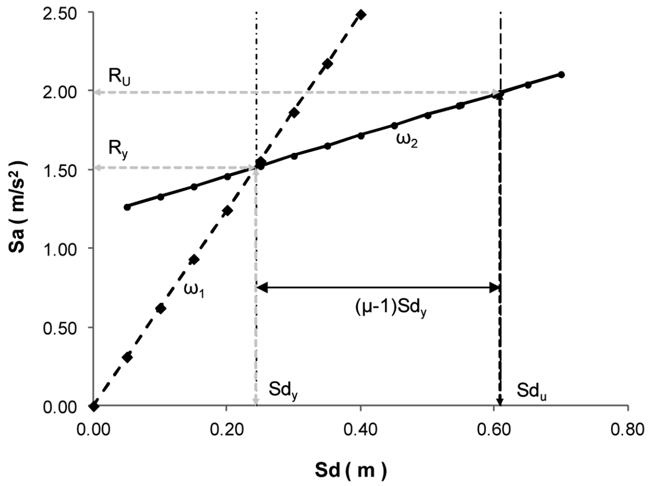


Fig. 7 Behaviour curve of reference SDOF system

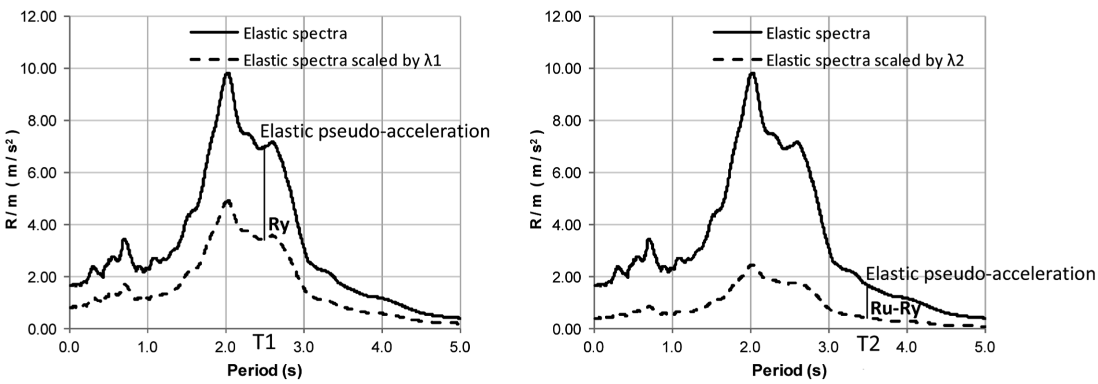


Fig. 8 Strength spectra used for the modal spectral analyses of the (a) undamaged model and (b) damage model

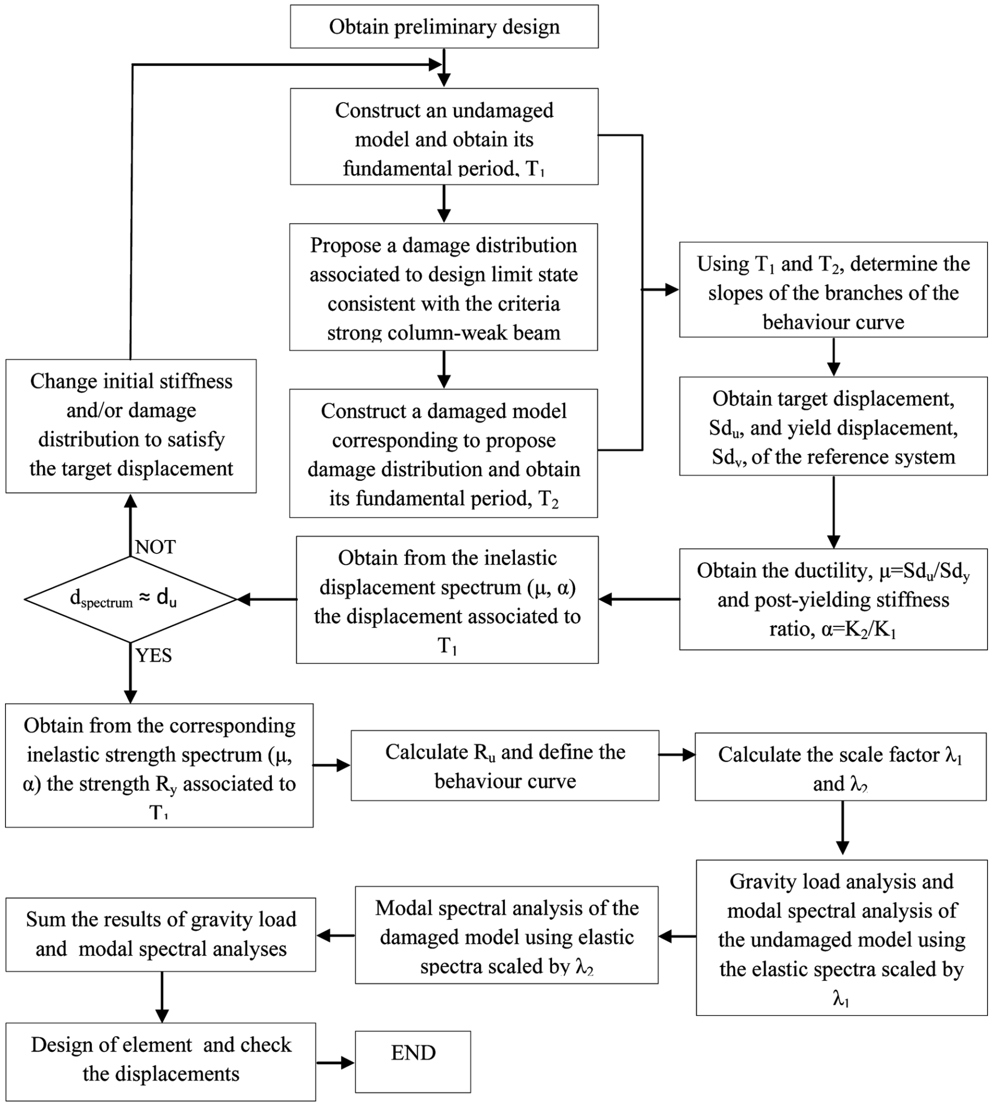


Fig. 9 Flowchart of the a displacement-based seismic design method with damage control for RC buildings

elements that accept and do not accept damage are obtained by adding the forces due to gravity loads and the forces of the modal spectral analyses of the undamaged and damaged structure.

12. Design of structural elements in accordance with the forces obtained from the analysis of the simplified models using the applicable design rules. This design must be performed in such a way that the code design criteria do not alter significantly the expected performance.

5. Application examples

To illustrate the application of the design method proposed, five reinforced concrete frames, three regular and two irregular in elevation (see Figs. 10(a) to 10(e)), are designed. The structural layout

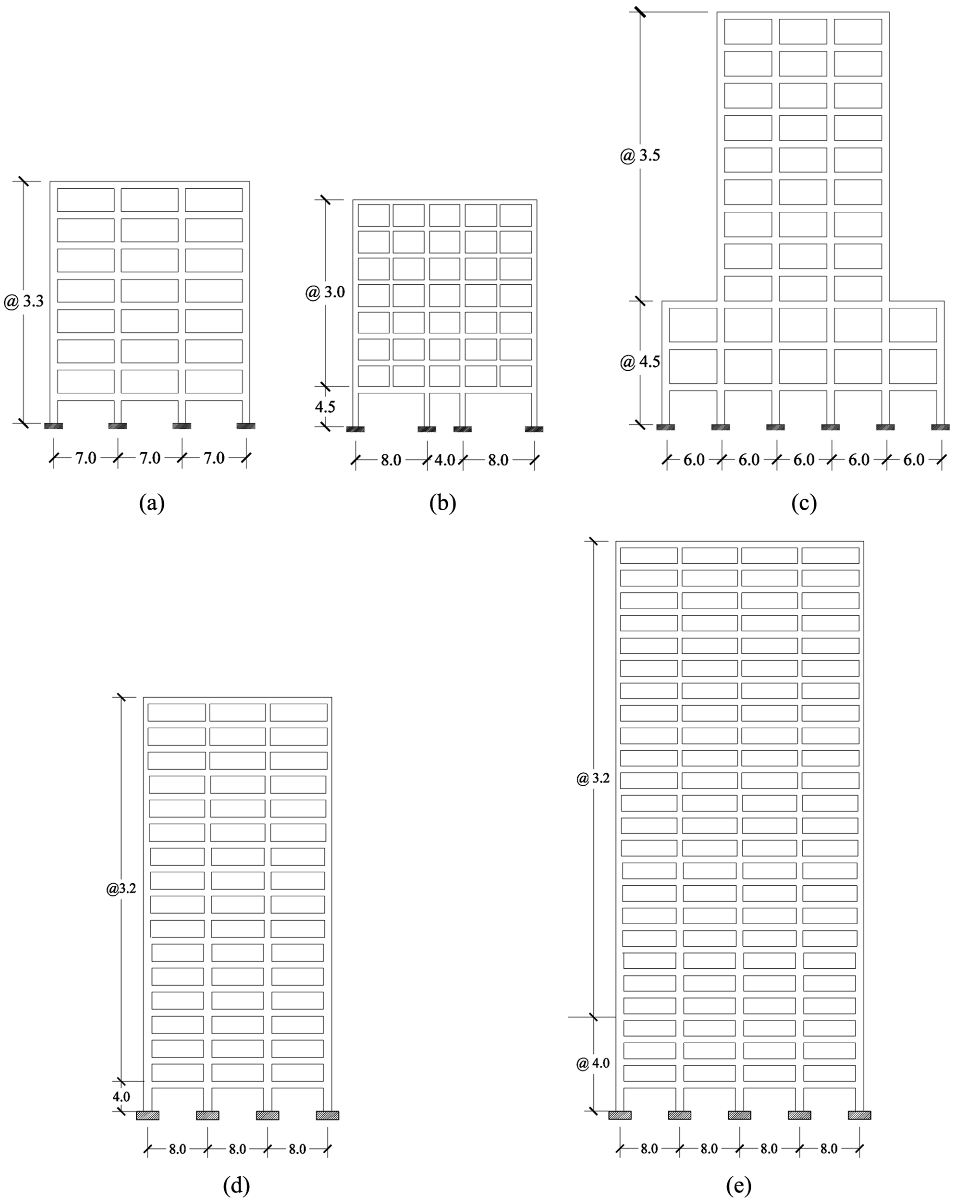


Fig. 10 Geometry of the example frames: (a) 8 storey frame, (b) 8 storey irregular frame, (c) 12 storey irregular frame, (d) 17 storey frame and (e) 25 storey frame

and height of the examples are chosen to evaluate the effect of the participation of higher modes in the approximation, and to assess the accuracy of the method when used to design structures with stiffness changes in elevation.

The preliminary designs of these frames are done using the Building Code for the Federal District, RCDF, (GDF 2004). The nominal properties of the materials used in the design are: for the concrete a compressive strength $f'c = 2.45\times10^4$ kN/m^2, modulus of elasticity $Ec = 21.7\times10^6$ kN/m^2, weight density $\gamma = 23.53$ kN/m^3, and for steel reinforcement a yield stress $fy = 4.11\times10^5$ kN/m^2 and

Table 1 Geometry of the element sections and fundamental periods for each frame

Frame description	Element type	Cross section [m] (preliminary design)	T_1 [s]	Cross section [m] (final design)	T_1 [s]	Level
8 storey regular	Beams	0.45 × 0.75	1.36	0.45 × 0.80	1.29	1-8
	Column	0.60 × 0.60		0.60 × 0.60		1-8
8 storey irregular	Beams	0.40 × 0.80	1.16	0.40 × 0.75	1.37	1
	Beams	0.30 × 0.70		0.30 × 0.65		2-4
	Beams	0.30 × 0.60		0.30 × 0.55		5-8
	Column	0.60 × 0.60		0.55 × 0.55		1-2
	Column	0.50 × 0.50		0.45 × 0.45		3-8
	Column*	0.35 × 0.50		0.35 × 0.40		2-8
12 storey irregular	Beams	0.40 × 0.80	2.01	0.40 × 0.90	1.70	1-5
	Beams	0.35 × 0.60		0.40 × 0.70		6-12
	Column	0.70 × 0.70		0.70 × 0.70		1-3
	Column	0.60 × 0.60		0.60 × 0.60		4-12
17 storey regular	Beams	0.45 × 1.00	1.78	0.45 × 0.90	2.20	1-17
	Column	1.10 × 1.10		0.90 × 0.90		1-7
	Column	1.00 × 1.00		0.70 × 0.70		8-11
	Column	0.90 × 0.90		0.60 × 0.60		12-14
	Column	0.80 × 0.80		0.50 × 0.50		15-17
25 storey regular	Beams	0.40 × 1.10	3.07	0.45 × 1.20	2.52	1-25
	Column	1.20 × 1.20		1.40 × 1.40		1-9
	Column	1.10 × 1.10		1.30 × 1.30		10-18
	Column	1.00 × 1.00		1.20 × 1.20		19-25

*Secondary column (see Fig. 10(b))

modulus of elasticity $Es = 1.96\times10^8$ kN/m^2. Table 1 shows the geometries of the beams and column sections used for each frame together with their corresponding fundamental periods. As for all examples the original geometry of some element sections were modified to give a period, for which the spectral displacement equaled that corresponding to the target displacement; these modified sections are also shown in this table. The moments of inertia of columns were calculated using gross sections, whereas, for beams they were half of those calculated with the gross sections.

For the purpose of validating the effectiveness of the method, the seismic demand considered in the case studies was a set of response spectra corresponding to the record of the SCT E-W component of the 1985 Michoacán earthquake, to evaluate directly the performance of the design procedure, and thus, prove the robust framework of its formulation.

To validate the results of the method proposed, the displacements of the frames were calculated using non linear step by step analyses under the same seismic demand for which they were designed, *i.e.*, E-W component of the SCT record of the 1985 Michoacán earthquake. These analyses were carried out with the program DRAIN 2D-X (Prakash *et al.* 1993) using the following considerations:

- Nearly elastoplastic bilinear stable hysteretic model used in beams and columns.
- Proportional damping matrix.

- Axial load – Moment interaction considered.
- P-Δ effects no considered.
- Yield moments for beam and columns are those obtained from the design method without any standardization.

Using the preliminary design data, the elastic models are constructed and, from their modal analyses, their dynamic properties, obtained. For each model, a damage distribution consistent with the seismic design philosophy of strong column-weak beam is proposed.

With the results of the spectral modal analyses of the damaged models, the configurations of lateral displacements of the frames are obtained. Taking δ as the limit interstorey drift, Table 2, the maximum roof displacements are calculated. To obtain the target displacements, the maximum roof displacements associated to the fundamental modes are divided by the participation factors. Using Eqs. 1 to 3, the yield displacements and the αs are defined, and with them, the ductilities of the reference systems, calculated.

For each of the models, the inelastic displacement spectra, IDS, for their corresponding α and μ, are built. From these spectra the spectral displacement associated to the corresponding periods, T_1, are read. In all cases the demanded maximum displacements are considerably less than those required for the given performance level, therefore, the initial structures are modified so that the structural demands approach the displacement requirements, *i.e.*, a true displacement-based design for the frames is used.

Table 2 Behaviour curve data

Example	T_1	T_2	α	μ	S_{du}	S_{dy}	R_y	R_u	λ_1	λ_2	δ
8 sto.	1.29	3.10	0.17	2.50	0.24	0.096	1.79	2.25	0.63	0.18	0.020
8 sto	1.37	2.49	0.30	2.32	0.22	0.095	1.92	2.66	0.62	0.11	0.015
12 sto.	1.70	3.69	0.21	2.50	0.47	0.188	2.05	2.71	0.38	0.47	0.020
17 sto.	2.20	4.59	0.23	2.50	0.72	0.288	1.80	2.42	0.24	0.89	0.020
25 sto	2.52	5.52	0.21	2.50	0.61	0.244	1.44	1.89	0.21	0.99	0.015

Table 3 Modal participating mass ratios for the undamaged and damaged models

		Modal participating mass ratios				
Frame description	Model	MODE 1	MODE 2	MODE 3	MODE 4	MODE 5
8 storey regular	Undamaged	0.81	0.10	0.04	0.02	0.01
	Damage	0.83	0.08	0.04	0.02	0.01
8 storey irregular	Undamaged	0.84	0.11	0.03	0.01	0.00
	Damage	0.87	0.07	0.03	0.01	0.01
12 storey irregular	Undamaged	0.68	0.18	0.06	0.03	0.02
	Damage	0.77	0.11	0.04	0.02	0.02
17 storey regular	Undamaged	0.78	0.11	0.04	0.02	0.01
	Damage	0.73	0.13	0.04	0.03	0.02
25 storey regular	Undamaged	0.79	0.10	0.03	0.02	0.01
	Damage	0.80	0.07	0.04	0.02	0.02

For each example, the ISS is built for the corresponding values of α and μ, from which the strength value associated with T_1, R_y, is read. Using Eq. 5, the value of the ultimate strength R_u is obtained. With these data the modal behaviour curves for each example are constructed.

Finally, with the information of the behaviour curves of the reference systems, given in Table 2, the design forces for each structure are obtained by modal spectral analysis using scaled spectra.

Table 3 shows the modal participating mass ratios for frames used as examples.

The results of lateral displacements and interstorey drifts obtained with this method and those from the nonlinear dynamic analyses are shown in Figs. 11 and 12 respectively. Figs. 6(a) to (e)

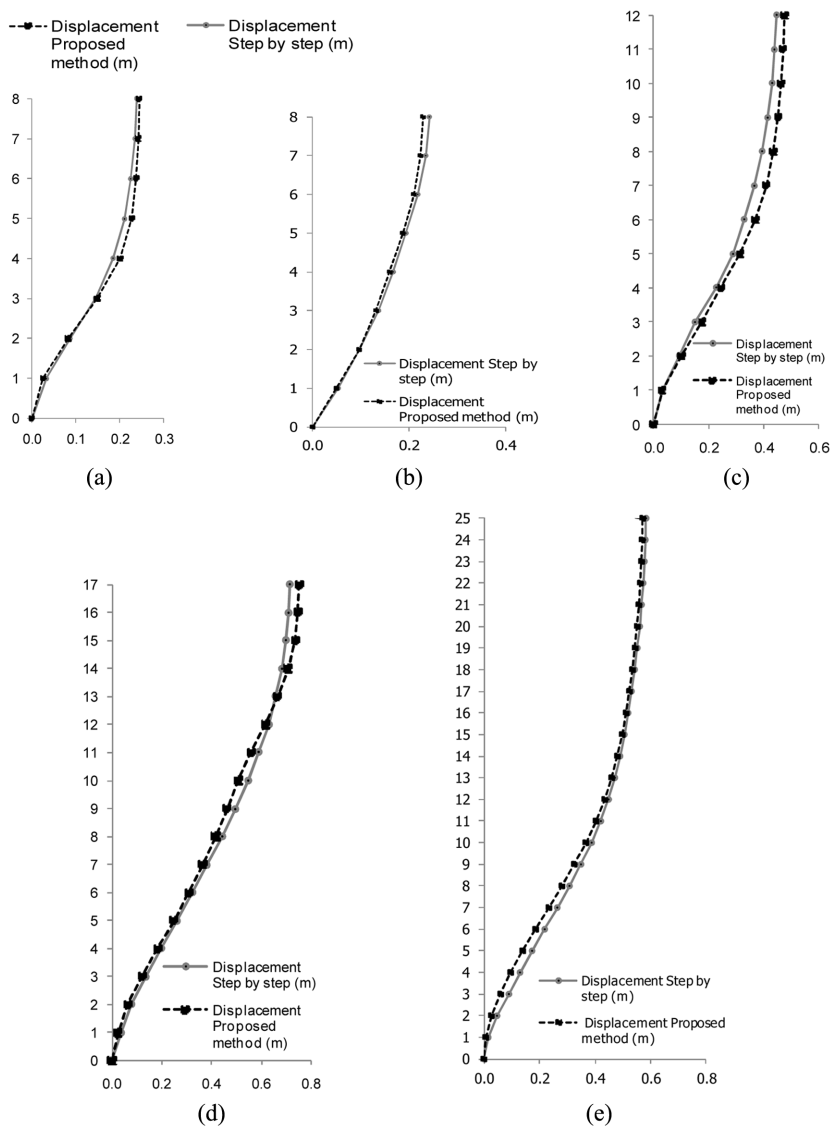


Fig. 11 Comparison of the design and calculated lateral displacements for the example frames: (a) 8 storey frame, (b) storey irregular frame, (c) 12 storey irregular frame, (d) 17 storey frame and (e) 25 storey frame

show an acceptable correspondence between the expected and calculated deformed configurations. The variations in the lateral displacements of the intermediate floors of the structures are not significant, since the design objective is to restrict the maximum interstorey drift ratio to the δ value of Table 1 (G.D.F. 2004), and this value is not exceeded as shown in Fig. 7.

The damage distributions obtained are approximately equal to those proposed as targets (see Figs. 13 to 17). In general, the proposed damaged distributions are maintained, keeping the columns in elastic state.

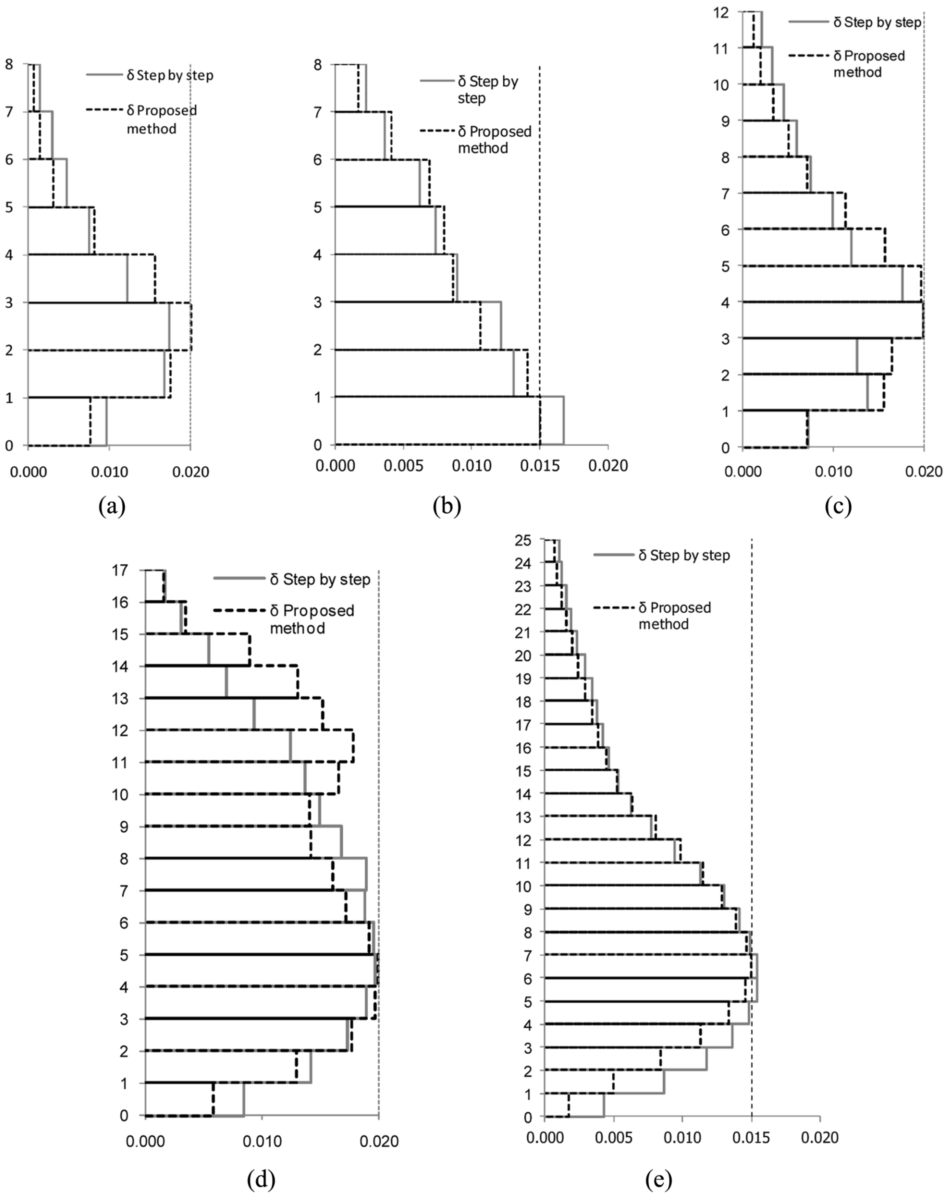
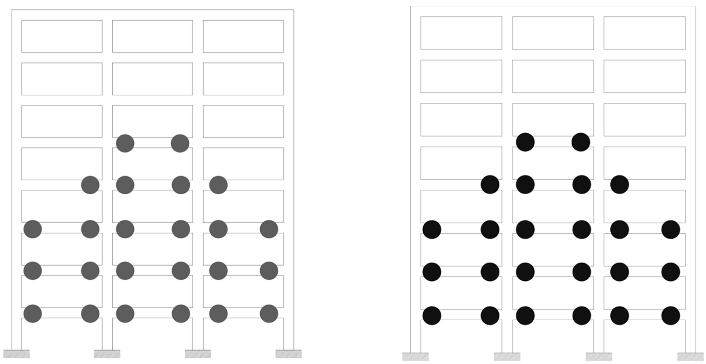
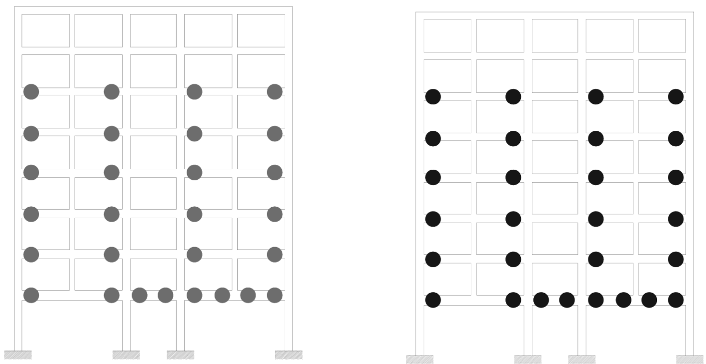


Fig. 12 Comparison of the design and calculated interstorey drifts for the example frames: (a) 8 storey frame, (b) 8 storey irregular frame, (c) 12 storey irregular frame, (d) 17 storey frame and (e) 25 storey frame



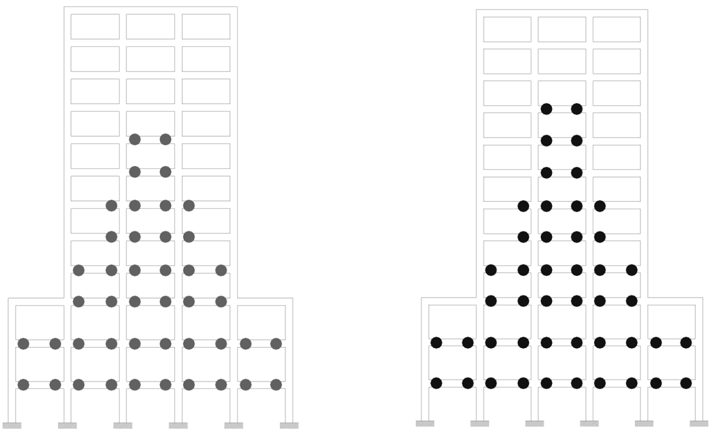
(a) (b)

Fig. 13 Comparison of damage distributions of the 8 storey frame: (a) nonlinear step by step analysis and (b) proposed distribution



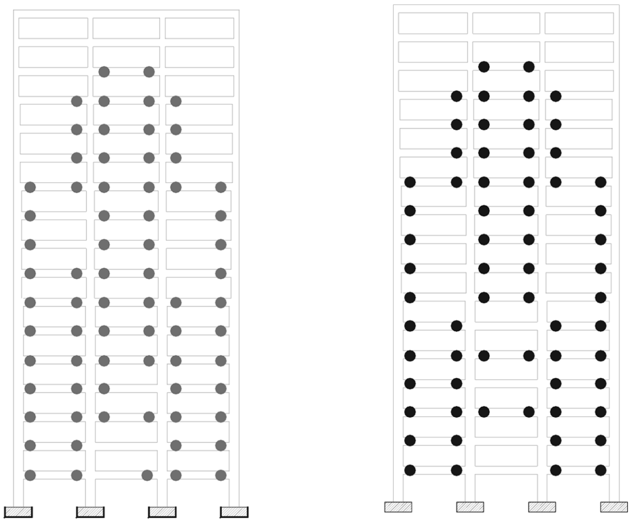
(a) (b)

Fig. 14 Comparison of damage distributions of the 8 storey irregular frame: (a) nonlinear step by step analysis and (b) proposed distribution



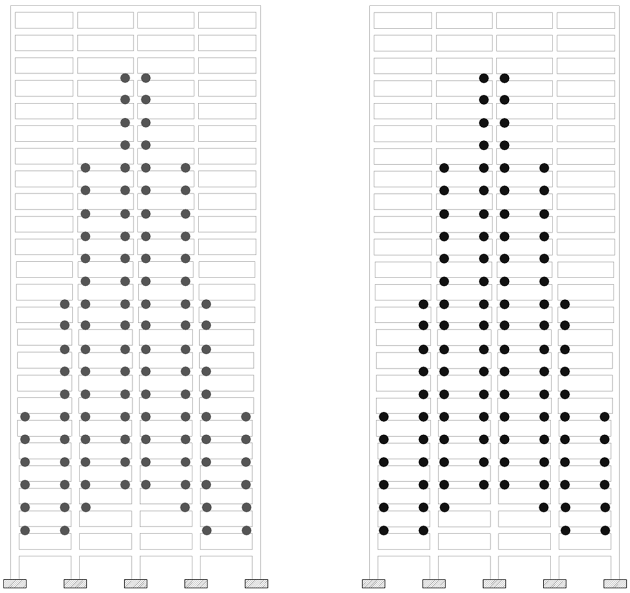
(a) (b)

Fig. 15 Comparison of damage distributions of the 12 storey irregular frame: (a) nonlinear step by step analysis and (b) proposed distribution



(a) (b)

Fig. 16 Comparison of damage distributions for the 17 storey frame: (a) nonlinear step by step analysis and (b) target distribution



(a) (b)

Fig. 17 Comparison of damage distributions for the 25 storey frame (a) nonlinear step by step analysis and (b) target distribution

In most columns, the element forces are larger than those obtained with the nonlinear analysis, with average errors of 15%, 1% and 8% for axial force, shear and bending moment, respectively. For the small number of columns in which the forces calculated with the method are smaller than those of the nonlinear analyses, the maximum errors are 19% for shear and 12% for bending moment; in this case the axial forces obtained with the method are larger. In beams the average errors are 4% for shear and 5% and bending moment, the positive maximum error is 14% for shear and 1% for moment.

6. Conclusions

This paper presents an approximate displacement-based seismic design method of structures, with explicit consideration of nonlinear behaviour and control of structural damage. The method is formulated from basic approximations to concepts of structural dynamics, using the same analysis tools used in design practice, e.g. SAP2000 (CSI 2006). Due to its basis, this method may be applied to structures with different layouts, e.g. regular and irregular frames, dual systems, among others. From the analysis of the results obtained for the five illustrative examples considered, the following conclusions may be drawn:

1. The design performances have a satisfactory correspondence with the maximum displacements of the nonlinear step by step analyses, reproducing the location of the design interstorey drift and acceptable values for other storeys.
2. The differences in damage distributions assumed as target in the application of this method and those obtained from nonlinear step by step analysis are not significant. Even though inelastic behaviour is also present at few unexpected locations, the desired damage distributions are reproduced by the nonlinear dynamic analyses. In all cases, the method manages to keep the strong column weak beam desired behaviour.
3. In general the design forces in the elements shows a good approximation when compared, with those obtained from the nonlinear step by step analyses. However, to obtain better approximations, more realistic damage distributions and more rigorous relationships between stiffnesses of the structural elements and displacements above the elastic range must be considered.
4. For all the examples considered, the attained damage control may be considered satisfactory, even though inelastic behavior was also present at some unexpected locations, the damage distributions of the nonlinear analyses remained, in general, approximately the same as those proposed. In all cases, the method managed to kep the strong column weak beam desired behavior.
5. The comparison of the effort involved in the application of this method using computational tools available in most design offices, e.g. SAP2000 (CSI 2006) and the quality of results obtained with those of other design methods, place it as an excellent design tool which can be conveniently included in future seismic design codes.

Research in progress by the same authors shows that this method also produce designs with guaranteed performances when subjected to other seismic demands. Recent results obtained (Mendoza 2011) show that this method can also be successfully applied to irregular 3D frames with asymmetrical plans and/or other irregularities in elevation.

In summary, with this design method it is possible to guarantee target performances in reinforced concrete buildings subjected to design demands reproducing predefined damage distributions. The results obtained validate the assumption that the capacity curve of a structure and the behaviour curve of the reference SDOF system, derived from it and corresponding to the fundamental mode are structural properties closely related to seismic performance.

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