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A DISPLACEMENT-BASED SEISMIC DESIGN METHOD CONSIDERING SIDESWAY COLLAPSE PREVENTION FOR FRAMED STRUCTURES

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ABSTRACT

This paper presents an innovative displacement-based seismic design method considering sidesway collapse prevention. The method proposed allows the design of a frame structure with P-Delta induced "negative stiffness" for either actual collapse or deformation control. To prove the validity of the method proposed, design applications of 8-, 12- and 16-storey frames using median collapse and constant ductility spectra are shown, along with the comparison of the design's targets with the results obtained from non-linear step-by-step analyses of the designed structures subjected to the corresponding earthquake records.

INTRODUCTION

Collapse of structures due to seismic events is one of the primary concerns of earthquake engineering. For this reason, extensive analytical and experimental studies on the subject have been carried out, especially in recent years, in which several factors that lead to the occurrence of collapse have been identified via rigorous structural modelling and extensive numerical simulations. *e.g.* Ibarra and Krawinkler (2005), Lignos and Krawinkler (2009), Haselton *et al.* (2009).

Two primary types of collapse may be defined: progressive and sidesway (Ibarra and Krawinkler 2005). The first is the collapse of a system triggered by the failure of one or a few structural components that lead to progressive failure of other parts of the structure until it collapses entirely. Sidesway collapse is the global failure of the system due to severe deterioration of lateral storey stiffness-strength due to cyclic degradation of structural components and the destabilizing effect of gravity loads, *i.e.* P-Delta effects. According to Adam and Jäger (2012a) the latter is the predominant type in seismic events, hence, most of the studies relative to seismic- induced collapse have given particular emphasis to this type.

Due to the difficulty of rigorous collapse prediction, research focussed on the development of simplified procedures to assess the vulnerability of structures to sidesway collapse, particularly P-Delta governed, has been carried out by authors such as Bernal (1998) and Adam and Jäger (2012b). However, few studies have focussed on the development of simplified design procedures that consider explicitly sidesway collapse prevention, *e.g.* Asimakopoulos *et al.* (2007). The present paper illustrates a new simplified method that allows the design of a structure with P-Delta induced negative stiffness for either actual sidesway collapse prevention or the achievement of a predefined interstorey drift threshold. The latter is the approach followed by most of the current national and international codes, in which collapse is defined as the exceedance of a prescribed interstorey drift threshold.

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The displacement-based seismic design method described in this paper is an evolution of the original method proposed by Ayala et al. (2012). This method consists of the definition of a bilinear or trilinear, spectral pseudo-acceleration, Sa, *vs.* displacement, Sd, curve of the fundamental mode of a structure, that provides the properties that the structure requires to satisfy a considered performance objective, PO.

The present paper illustrates the general framework of the design method proposed, focusing on the sidesway collapse prevention approach. Design applications for 8-, 12-, 16-storey frames designed using median collapse and constant ductility spectra, built from the FEMA (2009a) far field set of earthquake records, are shown. The validity of the method proposed is assessed by comparing the design median collapse intensities with those obtained from incremental dynamic analyses (Vamvatsikos and Cornell, 2002) of the designed structures for the corresponding records.

P-DELTA EFFECTS IN STRUCTURES

Second order effects, usually referred to as P-Delta effects, are defined as the amplification of the demands of a structure due to the action of vertical loads over the lateral displacements. The most straightforward approach to introduce second order effects in the equilibrium equation of a structure is by adding to the first order stiffness matrix of the system, K_0, the geometric stiffness matrix, K_G, in which P-Delta effects are represented as a decrement of stiffness of the vertical load carrying components of the structure in terms of the column axial load to storey height ratios (Clough and Penzien, 1993). Hence, the effective stiffness matrix, K, *i.e.* second order stiffness matrix, of a given structure is:

$$K = K_0 - K_G \tag{1}$$

The stiffness modification due to gravity loads in structures is usually negligible in the elastic stage of behaviour; however, it may be significant in the inelastic stage since the global stiffness matrix corresponding to such instance is reduced due to damage of structural components. Moreover, this reduction may be of such magnitude that the system is statically unstable, *i.e.* K not positive definite. If a structure presents such condition it is susceptible to exhibit dynamic instability. Thus, dynamic instability potential can be identified if a negative eigenvalue is obtained from eigen- analysis (Bernal 1998); the larger the eigenvalue the more vulnerable the system is to collapse (Miranda and Akkar 2003). However, dynamic instability depends also on the strength of the system and the characteristics of the demand. Hence, non-linear dynamic analysis is required to identify sidesway collapse of a structural system for a particular loading.

Frame structures are particularly vulnerable to P-Delta effects, especially those with long periods, in which the influence of cyclic deterioration on the occurrence of sidesway collapse is relatively not significant. Several studies show that, in general, dynamic instability of frames is governed by the first mode and, thus, the collapse capacity may be approximated using artifices that rely on single degree of freedom, SDOF, systems (Ibarra and Krawinkler 2005). The design method proposed, based on an alternative representation of a multiple degree of freedom, MDOF, structure by an SDOF system, relies on such approach.

FUNDAMENTALS OF THE METHOD PROPOSED

Reference system

The main basis of the method proposed is the assumption that it is possible to approximate the maximum inelastic response of an MDOF structure using an SDOF system whose properties match those of the fundamental mode of the MDOF structure. Such SDOF system is termed as the reference system and, accordingly, the main tool employed to describe the response of a structure is the corresponding curve of strength per unit mass, *in* terms of spectral pseudo-acceleration, Sa, *vs.* spectral displacement, Sd, referred to as the behaviour curve.

For the case of design for a PO involving two limit states, the application of the method proposed consists of the definition of a design bilinear behaviour curve that provides the strength and stiffness required by the structure to satisfy the considered PO (Fig. 1). The characteristic points that define this curve are: origin (0,0); yield (Sd_y,Sa_y) and ultimate (Sd_u,Sa_u). The stiffness of the first branch, λ^E, is limited in such a way that the interstorey drift threshold associated with the serviceability limit state, SLS, is satisfied. The postyield stiffness, λ^D, is defined according to a design damage state, *i.e.*, strong column-weak beam, corresponding to an ultimate limit state, ULS, *e.g.* life safety, LSLS; collapse prevention, CPLS. These stiffnesses are obtained from eigen-analyses of elastic models representative of the stiffness of the structure in both elastic and inelastic stages of behaviour. The yield and ultimate strengths are those required by the structure to comply with the interstorey drift threshold of the ULS.

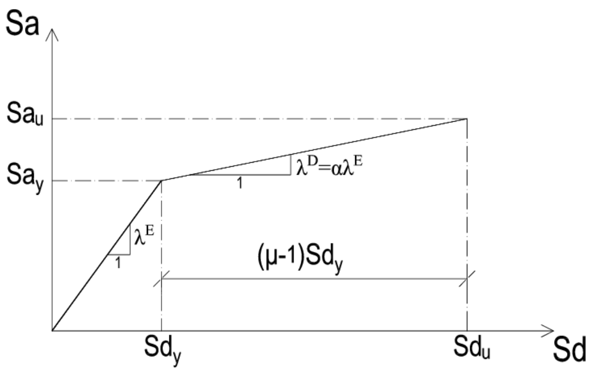


Figure 1. Behaviour curve of the reference system for a two-limit state PO

Damaged model

In accordance with structural dynamics fundamentals regarding the modal superposition approach, it is assumed that the stiffness of the structure corresponding to the maximum inelastic response is a function of its design damage state. Therefore, in the method proposed, the post-yield stiffness of the reference system is obtained from eigen-analysis of the so-called damaged model, a replica of the elastic model in which the plastic hinges corresponding to the design damage state are characterized by rotational springs whose stiffnesses match the post-yield stiffness of the structural elements.

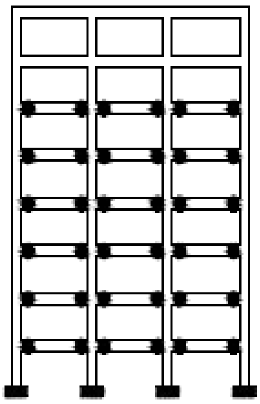


Figure 2. Damaged model with strong column-weak beam damage state

Identification of dynamic instability potential of the structure

Dynamic instability potential of the system is determined in a straightforward manner from inspection of the eigenvalues obtained from the eigen-analysis of the damaged model considering the geometric stiffness matrix formulation. As mentioned in the preceding section, if a negative eigenvalue is attained, which for frame structures will most likely be that of the first mode, the slope of the second branch of the behaviour curve will be negative, indicating that the structure is prone to sidesway collapse under dynamic loading. If such is the case, two approaches may be followed to design the structure.

The first approach is to modify the stiffness of the structure and/or the design damage state so that the post-yield stiffness of the structure is positive. This option is based on the fact that by increasing the stiffness of the structural elements, especially columns, the effect of the gravity loads in the effective lateral strength is minimized, *i.e.* the values of the elements of K_0 are increased such that the effective stiffness matrix, K, of the damaged model is positive definite. The same result may be obtained by preventing inelastic action in all columns including those of the first floor. Evidently, combination of both actions would also prove useful.

Nonetheless, since structures with “negative stiffness” will not necessarily collapse under dynamic loading, the second design approach is to allow a certain degree of negative stiffness due to vertical loads and provide the required strength to avoid collapse under the ULS design demands. Although the first approach seems more rational than the second as any condition of instability is objectionable, the latter may result in a more cost effective design. If this option is chosen, the artifice of the auxiliary system (Ibarra and Krawinkler, 2005) is employed and the design demands are obtained from either collapse capacity spectra or constant ductility spectra for systems with P-Delta induced negative stiffness. The latter approach is the one illustrated in the following.

Auxiliary system

The auxiliary system, developed by Ibarra and Krawinkler (2005), is an artifice employed to approximate the collapse capacity of a structure via an equivalent SDOF system. It consists of the definition of an SDOF system whose properties are such that its second order backbone matches that of the equivalent SDOF system. It is necessary to employ this artifice since the stability coefficients in the elastic and inelastic stages of behaviour in MDOF structures may be different, whereas in SDOF systems the stability coefficient is the same regardless of the damage state. In the proposed method, the stability coefficients and the properties of the auxiliary system are defined from the properties of the reference system via the following equations:

$$\theta^E = \frac{\lambda_0^E - \lambda^E}{\lambda_0^E} \tag{2}$$

$$\theta^D = \frac{\lambda_0^D - \lambda^D}{\lambda_0^D} \tag{3}$$

$$T_{aux} = T_0\sqrt{\frac{1-\alpha}{1-\alpha-\theta^E+\theta^D}} \tag{4}$$

$$\theta_{aux} = \sqrt{\frac{1-\alpha}{1-\alpha-\theta^E+\theta^D}} \tag{5}$$

where θ indicates the stability coefficient, T is the fundamental period, sub-index *aux* indicates the properties of the auxiliary system, α is the first order stiffness of both the reference and auxiliary systems; λ and λ_0 are the eigen-values of the structure with and without considering P-Delta effects, respectively; subindex *0* refers to first order analysis, superindices *E, D*, denote elastic and damaged stages of behaviour, respectively.

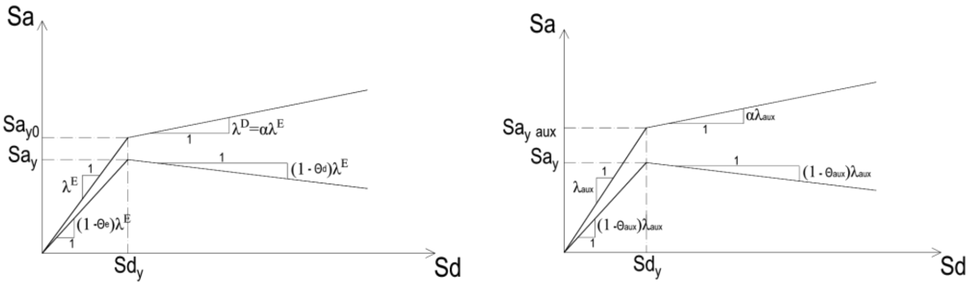


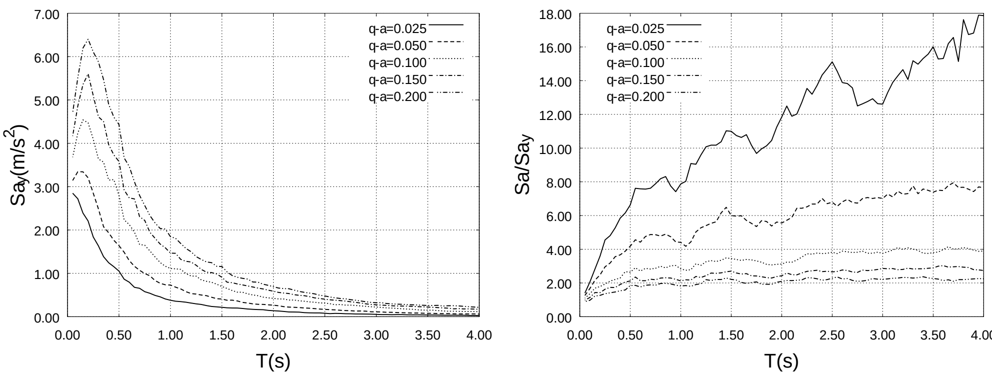
Figure 3. First and second order reference and auxiliary SDOFs.

Collapse capacity and constant ductility spectra for SDOFs with negative stiffness

A collapse capacity spectrum defines the capacity, either in terms of yield strength, Sa_y, or relative intensity, *i.e.* ratio of corresponding elastic system's spectral acceleration to yield strength, Sa/Sa_y, of an SDOF system with a given negative stiffness for which dynamic instability occurs under a particular dynamic loading. The collapse capacity is usually obtained via incremental dynamic analysis, IDA, (Vamvatsikos and Cornell, 2002). The parameters that define this type of spectra for SDOF systems with bilinear backbones are: viscous damping ratio, ζ; effective negative post-yield stiffness ratio defined as the difference of stability coefficient and hardening ratio, $\theta - \alpha$; and a hysteresis rule (Adam and Jäger 2012b). As can be seen in Fig. 4, which depicts median collapse capacity spectra, the steeper the effective stiffness the smaller the collapse capacity.

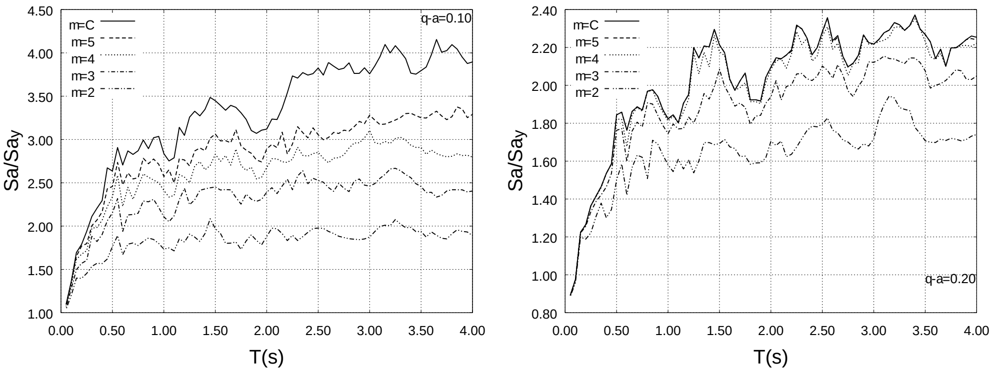
For a design application, the decision on which type of spectra will be used depends on the definition of collapse considered. If the attainment of an interstorey drift threshold is defined as "collapse", such as it is in the CPLS of conventional design codes, then constant ductility design spectra should be used and a congruent design displacement, with respect to a rational value of design ductility, would be defined. On the other hand, collapse capacity spectra should be used if actual dynamic instability is considered. However, information of the post-yield demand is required for an application of the method proposed, *e.g.* collapse displacement spectra would be needed, since the required strength of non-yielding elements, *e.g.* columns, is defined as a function of the ductility of the reference system. Alternatively, design for actual collapse could be achieved by using constant ductility spectra associated with a large value of ductility whose ordinates are similar to the collapse capacity spectra for the considered (θ–α).

However, it should be noted that the first option might be better for a design application, since the design demands of non-yielding elements would be considerably large if the design was carried out for actual collapse. Moreover, since a negative stiffness in a structural system could be a consequence of either P-Delta effects and/or in-cycle degradation of structural components (FEMA 2009b), it may be a good design approach to limit the plastic rotation or interstorey drift of the structure so that its components do not exhibit in-cycle degradation, allowing only P-Delta induced negative stiffness.



a) b)

Figure 4. Median collapse capacity spectra of FEMA (2009a) far field set of records for several θ-α values: a) strength; b) relative intensity



a) b)

Figure 5. Median collapse capacity and constant ductility spectra of the FEMA (2009a) far field set of records for several μ values: a) θ-α = 0.10; b) θ-α = 0.20

Definition of design displacements

It is well recognized that, in general, the displacement response of framed structures is governed by the first mode. Therefore, the design displacement shapes associated with the yield and ultimate characteristic points of the reference system may be estimated, at least in an initial stage of the procedure, by equating the corresponding interstorey drift threshold to the largest difference of modal coordinates of the first mode, which defines the critical storey of the frame. The yield interstorey drift threshold may be estimated via simplified expressions such as those proposed by Priestley (2007) or Ayala *et al.* (2012). The modal shape considered to calculate the serviceability and yield displacement shapes is the eigen-vector obtained from the elastic model. The ultimate displacement shape is calculated using a linear combination of the eigen-vectors of the elastic model and damaged model corresponding to the effective stiffness, assuming that the displacement response under dynamic loading is analogous to that under monotonic adaptive loading. Accordingly, the design spectral displacements of the auxiliary system are calculated via the following equations:

$$Sd_s = \frac{IDR_s H}{\Gamma_1^E (\phi_{k1}^E - \phi_{k-11}^E)} \tag{6}$$

$$Sd_y = \frac{IDR_y H}{\Gamma_1^E (\phi_{k1}^E - \phi_{k-11}^E)} \tag{7}$$

$$Sd_u = \frac{IDR_u H}{\Gamma_1^{D*} (\phi_{k1}^{D*} - \phi_{k-11}^{D*})} \tag{8}$$

$$\phi_{i1}^{D*} = \frac{1}{\mu}\left[\frac{\Gamma_1^E}{\Gamma_1^D}\phi_{i1}^E + (\mu - 1)\phi_{i1}^D\right] \tag{9}$$

where IDR is the interstorey drift ratio threshold; H is the height of the critical storey k; Γ_1 denotes the modal participation factor of the first mode; Φ is the modal shape, sub-indices *s*, *y* and *u* refer to serviceability, yield and ultimate, respectively; super-indices E and D denote the elastic and damaged model, respectively; super-index * identifies the design modal shape.

Definition of design demands of higher modes and modal combination

For the sake of simplicity, it is considered that the design spectrum for the ULS, corresponding to the properties of the auxiliary system, is representative of the contribution of higher modes up to the yield point of the structure. Furthermore, it is assumed that the ductilities are the same as that of the reference system and that the inelastic response is uncoupled. Hence, the design contribution of each mode *j* is given by the following equation.

$$Sa_{u\,j} = Sa_{y\,j}\left[1+\alpha_j(\mu-1)\right] \tag{10}$$

The aforementioned assumptions are employed for the sake of simplicity and may not be consistent with actual behaviour; studies such as Sasaki *et al.* (1998) and Sullivan *et al.* (2008) show that higher modes do not develop uniform strength reduction factors nor ductilities. Nonetheless, the criteria employed allows straightforwardness in the design procedure and the results of validations via non-linear dynamic analysis of designed frames show that acceptable approximations to the design displacements are attained.

In accordance with such assumptions, the design demands of the simplified models are obtained from modal spectral analysis of the elastic and damaged models. In both analyses the considered inelastic design spectrum is used, however, for the analysis of the damaged model the modal demands are scaled by the factor given by Eq. 11.

$$Sa_{u\,j} = \alpha_j(\mu-1) \tag{11}$$

Consequently, the ULS design forces are calculated via the modal combination of the maximum inelastic responses of all modes. If the SRSS is used, the design forces, F, are calculated with the following equation:

$$F = \sqrt{\sum_{j}^{n}(F_j^E + F_j^D)^2} \tag{12}$$

Moreover, although the contribution of higher modes in framed structures is more relevant to forces than to displacements (Sullivan *et al.* 2008), for congruence sake with the aforementioned assumptions, a correction of the ULS design displacement shape and drift profiles may be carried out in a later stage of the design procedure by modal combination of the inelastic spectral displacements.

Definition of design demands of higher modes and modal combination

The application of the method proposed to design framed-structures with a negative post-yield stiffness intended to satisfy a two-limit state PO can be summarized in the following steps:

1. Pre-dimensioning of the structural elements based on engineering judgment and/or designer experience. Alternatively, a rough design of the structure using a force-based method may be performed. The purpose of this initial step is to define a realistic stiffness distribution of structural elements throughout the structure, thus, allowing the definition of design displacement shapes that are consistent with actual structures.

2. Modal analysis of the elastic model without considering P-Delta effects, from which the displacement shape of the first mode, *i.e.* displacement shape of the reference SDOF system, is obtained. From this displacement shape, the target spectral displacement of the reference SDOF system, Sd_s, for the SLS is calculated with Eq. (6).

3. The period T_s for which Sd_s is satisfied is obtained from the SLS displacement spectrum. The stiffness of the structural elements is modified in such a way that the period of the elastic model matches T_s, and, if possible, that the modal shapes are not significantly altered.

4. Gravity load and modal analyses of the modified elastic model considering P-Delta effects are performed using the geometric matrix formulation, from which the second order eigenvalue is obtained. Subsequently, the stability coefficient θ^E is calculated with Eq. (2).

5. Definition of a design damage state for the ULS in accordance with strong column-weak beam behaviour and definition of the damaged model corresponding to the structure with fundamental period T_s.

6. Modal analysis of the damaged model with and without considering P-Delta effects from which the eigenvalues are obtained and the inelastic stability coefficient is calculated via Eq. (3). The properties of the auxiliary SDOF system, T_{aux}, θ_{aux}, are defined by Eqs. (4) to (5).

8. If the ULS design is oriented towards deformation control, the yield and target spectral displacements of the reference SDOF system, d_y and d_u, are calculated with Eqs. (7) to (9), using the fundamental mode shape obtained from the second order modal analyses. Subsequently, the period T_u for which Sd_u is satisfied is obtained from the ULS displacement spectrum associated with ductility μ and negative post-yield stiffness ratio, θ_{aux} - α. On the other hand, if the design is focused on actual sidesway collapse, T_u is obtained from the corresponding collapse capacity spectra for a design target intensity Sa_{des}.

9. Definition of the final design period, T_{des}, as the smaller value of T_s and T_u. If T_s governs the design, go directly to the following step. Otherwise, the stiffness of the structural elements is modified proportionally such that the period of the elastic model matches T_u. If this is the case, for the sake of simplicity, the designer is allowed to continue the design process considering the post-yield stiffness ratio $\alpha - \theta_{aux}$ defined up to this step if this parameter does not change significantly. However, if the opposite holds true, modal analyses of both simplified models and the redefinition of the parameters of the reference and auxiliary systems should be carried out. Moreover, it is recommended that in this step the ULS design displacement shape is calculated via modal spectral analysis considering the assumed inelastic contributions of all modes.

10. Definition of the yield strength Sa_y of the design auxiliary SDOF system of period T_{aux} from the ULS collapse capacity or constant ductility, as applicable, for the resulting design parameters defined in the preceding step.

11. Modal spectral analysis of the simplified models using the corresponding strength per unit mass spectra and the factor given by Eq. 11. The design demands are obtained via Eq. 12.

12. The design of structural elements is carried out using accepted criteria regarding the behaviour of corresponding materials and structural elements.

EXAMPLE APPLICATIONS

To show the effectiveness of the method, the design of an 8- and 12- storey generic frames for sidesway collapse is presented in this section. Since the present paper is focused on the collapse design approach, these example applications were carried out without considering the SLS. The demands used in the design of these frames were median spectra calculated for the FEMA (2009a) far field set of records, 44 in total, considering aleatory uncertainty only, *i.e.* record to record variability. The validation of the design was carried out via statistical analysis of the results obtained from IDA of the designed structures employing the corresponding set of records.

Characteristics of the example frame

The storey frames considered have a first storey height of 4.50 m and 4.00 m in the other storeys, with 3 spans 10.0 m wide. The mass in every floor is m=78.50 KN s^2/m. The column to beam stiffness ratio is 1.5 in all storeys. Elasto-plastic behaviour in all structural components was considered. No flexure-axial interaction in column elements was considered. The vertical loads in all storey nodes is congruent with a $\theta_{aux} - \alpha$ values considered in their design. The stiffnesses of the structural elements of the 8-storey frame are constant along the height; the 12-storey frame presents uniform stiffness reductions of 85% at every 4 floors.

Design of frame

Both frames were designed for a target median collapse intensity of 3.00 m/s^2 in terms of the spectral acceleration of the first mode of the corresponding linear system of the structure. The post-yield stiffnesses considered were θ_{aux} - α= 0.05 and 0.075, for the 8- and 12-storey frame, respectively. The design demand employed was a median constant ductility spectra associated with such θ_{aux}-α values and a ductility of 12.00, since the ordinates of such spectra are approximately the same as those of the corresponding collapse capacity spectra for the period range of the designed frames, as can be observed in Fig. 6 and 7.

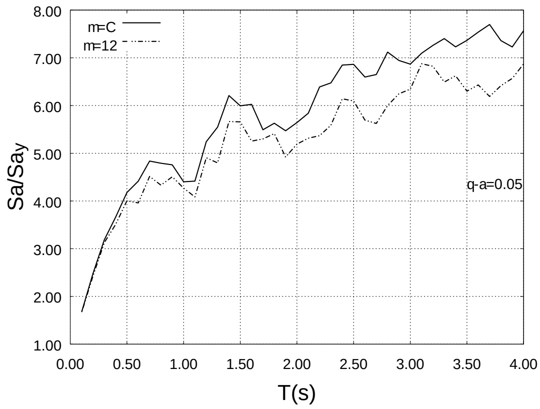


Figure 6. Median collapse capacity and constant ductility spectra (μ=12) of the FEMA (2009b) far field set of records used in the design of the 8-storey frame corresponding to θ-α = 0.05.

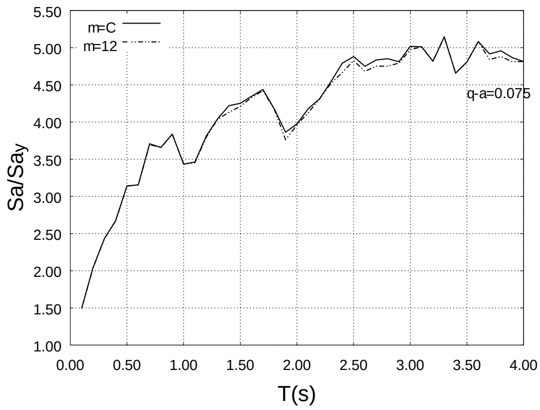


Figure 6. Median collapse capacity and constant ductility spectra (μ=12) of the FEMA (2009b) far field set of records used in in the design of the 12-storey frame corresponding to θ-α = 0.075.

Analysis performed

The design was carried out using an implementation of the method proposed in the OpenSees computer program (McKenna *et al.* 2004). Hinges used to represent damage were modelled as zero length spring elements. P-Delta effects were considered via the geometric matrix formulation. The non-linear dynamic analyses were carried out using Newmark's Beta method with parameters γ=0.5 and β=0.25 along with the Newton-Raphson method. Rayleigh damping considering ζ=0.05 for the first and second modes was employed. The IDA was performed using an initial intensity of 0.10 m/s^2 with increments of 0.10 m/s^2 up to collapse of the structure.

Results and evaluation

Tables 1 and 2 show the properties of the reference and auxiliary systems, respectively, associated with the target median intensities. Figs. 7 and 8 depict the individual IDA curves obtained from the analysis along with the resultant median collapse intensity and the target design intensity. Confidence intervals of the individual collapse capacities, associated with a 95% confidence level, calculated by assuming that the probability of the median lying within such interval is given by a normal distribution, are also depicted. As can be readily observed in such figure, the value of the median collapse intensity of the individual responses is very close to that of the target intensity and the latter lies within the confidence interval of the individual collapse capacities. Therefore, the design of both frame structures was satisfactory.

Table 1. Properties of design reference systems

FRAME	T (s)	$Sa_{y0}(m/s^2)$	α	θ^E	θ^D
8-storey	1.00	0.703	0.002	0.052	0.052
12-storey	1.50	0.705	0.010	0.057	0.087

Table 2. Properties of design auxiliary systems

FRAME	T (s)	$Sa_{y0}(m/s^2)$	α	θ_{aux}	μ
8-storey	1.00	0.703	0.002	0.052	12.00
12-storey	1.47	0.716	0.010	0.075	12.00

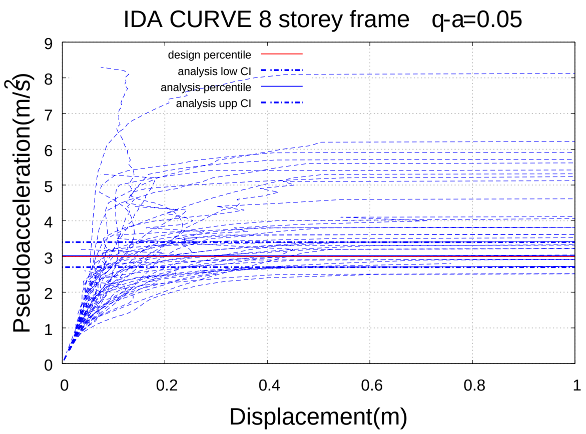


Figure 7. IDA curves of the 8-storey frame with the response and target median intensities for θ_{aux}-α=0.05

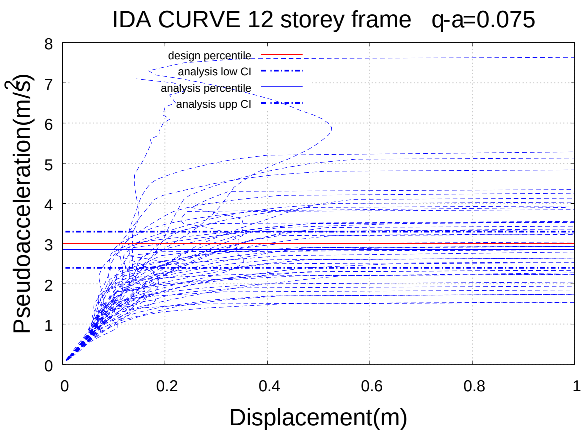


Figure 8. IDA curves of the 12-storey frame with the response and target median intensities for θ_{aux}-α=0.075

CONCLUSIONS

A displacement-based seismic design method considering prevention of sidesway collapse was presented. As can be seen from the results obtained from example applications, some of which are shown in this paper, the method allows the design of a structure with P-Delta induced "negative stiffness", *i.e.* instability prone, for a given collapse target intensity. Since the method proposed relies on the use of elastic analyses and collapse or constant ductility spectra, the procedure is straightforward and easy to apply, thus, it may prove useful for practical design applications. For this reason, further efforts regarding the development of this design approach are being carried out by the authors.

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