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Preliminary design and inelastic assessment of earthquake-resistant structural systems

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Abstract. A preliminary performance-based seismic design methodology is proposed. The top yield displacement of the system is computed from these of the components, which are assumed constant. Besides, a simple procedure to evaluate the top yield displacement of frames is developed. Seismic demands are represented in the form of yield point spectra. The methodology is general, conceptually transparent, uses simple calculations based on first principles and is applicable to asymmetric systems. To consider a specific situation two earthquake levels, occasional and rare are considered. The advantage of an arbitrary assignment of strength to the different components to reduce eccentricities and improved the torsional response of the system is addressed. The methodology is applied to an asymmetric five story building, and the results are verified by push-over analysis and non linear dynamic analysis.

Keywords: design; system; earthquake-resistant; displacement; yield.

1. Introduction

There is a broad consensus for the earthquake-resistant design to be based on displacements, ductility and damage indexes (Moehle 1992, Priestley 1993, Collins *et al.* 1996, Rubinstein *et al.* 2001, Paulay 2002). It is also accepted that the structure must satisfied pre-established performance levels for different earthquake levels (VISION 2000; 1995, Bertero 1996).

Usually the structural system selected for a multistory building and the sizes of its elements initially adopted are based on architectural constrains. The preliminary assessment of the system and its elements should be made by means of a transparent and simple methodology, if possible manual based.

The methodology should be applicable to asymmetric spatial systems. Traditionally, in force-based design the invariant parameter is the fundamental period of the structure. Input data is this period

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and the global ductiltity capacity of the system. In displacement-based design, maximum displacements (story drift) are used as input data.

According to latest findings (Priestley 1998, Paulay 2001), the invariant parameter should be the top yield displacement of the system. This top yield displacement and the global ductility capacity in each principal direction of the system should be used as start conditions (Giuliano *et al.* 2003, Rubinstein *et al.* 2004).

Especially suitable when considering the invariance of the top yield displacement is the representation of seismic demands using Yield Point Spectra (YPS) (Aschheim *et al.* 2000). This format is useful for design purposes and attractive for the designer familiarized with the capacity spectrum method (Freeman 1994).

Within the above framework this paper presents a comprehensive, transparent and simple methodology. Although the proposed methodology may be applied to several performance levels, only two performance levels are considered for the sake of clarity: operational for occasional earthquakes and life safety for rare earthquakes.

Rare earthquakes are characterized by a mean recurrence period of 475 years (10% probability of exceedance in 50 years) and occasional earthquakes by a mean recurrence period of 72 years, (50% probability of exceedance in 50 years).

Elastic response and a limit for the story drift are established for the operational level. Inelastic behavior and limits to the global ductility and the story drift for the life safety level are also established. When considering the global ductility, the effects of cumulative damage are taken into account by the Park & Ang damage index.

In what follows, the methodology for the preliminary design is described. A simple procedure to evaluate the top yield displacement of a frame is presented. The possibility of an arbitrary assignment of strengths for two equal geometry structural walls to diminish the strength eccentricity of the system is addressed.

A factor of 2 in the expression of the yield curvature at the base of the walls (Paulay 2001, 2002) based on moment curvature analyses is adopted. Accordingly, a best agreement in the top yield displacement as compared to the one obtained by push over analysis is obtained. This constitutes a sensible improvement related to a previous paper (Rubinstein *et al*. 2005).

An example of a five story asymmetric dual system, (frames and walls), located at Mendoza City, Argentina, is then presented. The preliminary design results are assessed by push-over analysis and non linear dynamic analysis, applied to a model with three degrees of freedom per story. Finally, useful conclusions for the application of the methodology and refinements to improve the accuracy are suggested.

2. Description of the methodology

A flowchart showing the steps of the methodology is depicted in Fig. 1.

2.1 Lay out

According to the architectural constrains and the experience of the designer, the geometry of the structural system, generally in two orthogonal directions, is adopted.

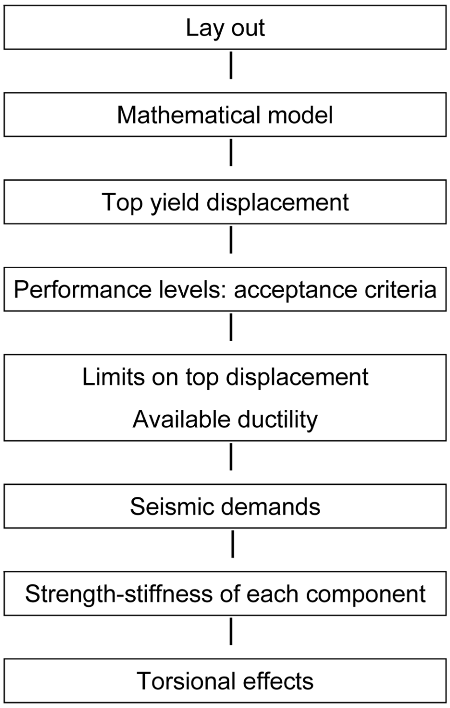


Fig. 1 Methodology: flowchart

2.2 Mathematical model

Elements of the system (frames and walls) connected by rigid floors, with three degrees of freedom per level.

2.3 Top yield displacement

Walls: For an individual wall, the collapse mechanism for the life safety level consists in one plastic hinge at the base, associated to a linear distribution in height of the seismic forces, as shown in Fig. 2(a).

Yield curvature at the base (Priestley 1998, Paulay 2001, 2002)

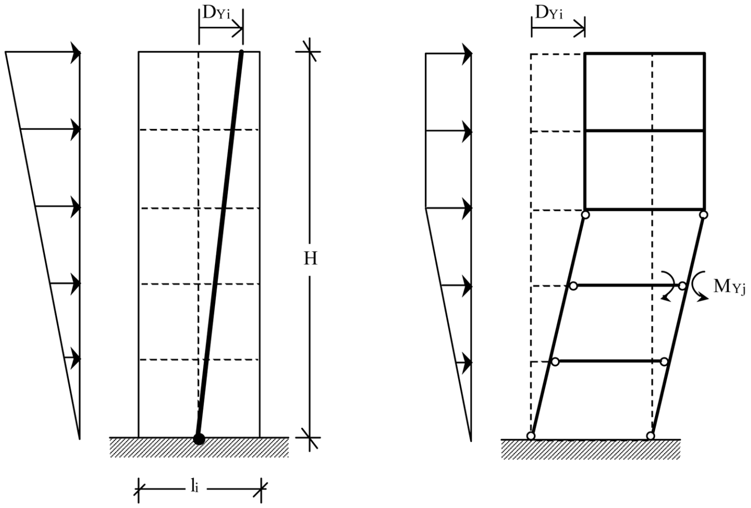
$$\phi_{yi} = \frac{\eta \varepsilon_y}{l_i} \tag{1}$$

ε_y being the yield strain of the longitudinal reinforcement.

Top yield displacement: for the triangular lateral load distribution shown in Fig. 2(a).

$$D_{yi} = \frac{11}{40}\phi_{yi}H^2 \tag{2}$$

Frames: To compute the top yield displacement of frames, some procedures for regular frames (Priestley 1998) and general frames (Rubinstein *et al.* 2004) already exist. A general procedure which is attractive for its simplicity and in line with capacity design is proposed.



(a) Wall (b) Frame

Fig. 2 Mechanisms and lateral loads

It consists of selecting a collapse mechanism for the frame and for the life safety level, i.e. beam sway mechanism, extending approximately two third of the frame high, as shown in Fig. 2(b). The yield bending moment at each plastic hinge j is evaluated as the product of the yield curvature by the flexural stiffness. The yield curvature, ϕ_{yj}, is determined based on the cross section geometry of the member and the yield strain of the longitudinal reinforcement (Priestley 1998). Code values for the moment of inertia (INPRES-CIRSOC 103, 2000) are used to determine the cracked flexural stiffness $(EI_r)_j$, that is

$$M_{yj} = (EI_r)_j \phi_j \tag{3}$$

Where

$$\phi_j = \frac{1.9\varepsilon_y}{h_b} \tag{4}$$

for beams, and

$$\phi_j = \frac{2.12\varepsilon_y}{h_c} \tag{5}$$

for columns. h_b, h_c: depth of beams and columns

The base shear, V_{yi}, is then determined by applying the Principle of Virtual Work to the adopted mechanism. The global stiffness K_i, corresponding to the base shear vs. top displacement relationship is determined loading the frame with a triangular lateral load distribution with unit intensity considering the above mentioned flexural stiffness of the members. Finally, the yield displacement becomes

$$D_{yi} = \frac{V_{yi}}{K_i} \tag{6}$$

System: The system top yield displacement may be obtained as the average yield displacement of the components

$$D_y = \frac{\sum_{i=1}^{n} D_{yi}}{n} \tag{7}$$

Alternative procedure:

Strength proportional to cracked flexural stiffness is assigned at each plastic hinge.

Walls:

Base moment

$$M_{yj} = (EI_r)_i \phi_{yi} \tag{8}$$

Base shear

$$V_{yi} = \frac{M_{yi}}{\frac{2}{3}H} \tag{9}$$

Frames:

The base shear is the same used in Eq. (6).

The strength assignment may be modified, within some rational limits, as the invariance of the yield displacement suggests, with the aim of reducing the eccentricity of the mass center. For rare earthquakes the strength eccentricity is relevant. For occasional earthquakes the stiffness eccentricity is relevant.

System:

Assuming for each component bilinear base shear-top displacement relationship without strain hardening, as shown in Fig. 3, the system top yield displacement gives

$$D_y = \frac{\sum_{i=1}^{n} V_{yi}}{\sum_{i=1}^{n} (V_{yi}/D_{yi})} = \frac{\sum_{i=1}^{n} V_{yi}}{\sum_{i=1}^{n} K_i} \tag{10}$$

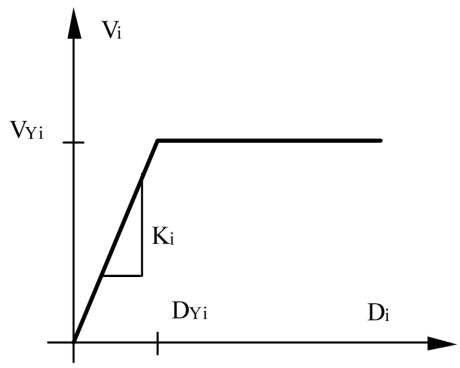


Fig. 3 Base shear-top displacement relationship for a component

Where V_{yi}: base shear of a component, frame or wall, n: number of components of the system in the direction being considered.

2.4 Performance levels: acceptance criteria

Although several performance levels may be included, for the sake of clarity only operational and life safety levels are considered.

Operational performance level: maximum drift $\theta_{op,\,\mathrm{lim}}$ and elastic behavior are established.

Life safety performance level: maximum drift $\theta_{ls,\,\mathrm{lim}}$, global ductility and damage index for each component and for the system are established.

2.5 Top displacement limits: available ductility

Maximum top displacement for each performance level and available ductility are determined base on the above requirements.

To include a priori the effects of induced torsion the drift limits are reduced by factors C_{op} and C_{ls} less than one. These factor are based on engineering judgment according to the lay out of the structural system.

Operational level:

The maximum top displacement based on drift limit is

$$D_{\theta_{op}} = HC_{op}\theta_{op,\mathrm{lim}} \tag{11}$$

Finally, considering the need for elastic behavior, the limit displacement at the top will be

$$D_{op} = \min(D_y, D_{\theta_{op}}) \tag{12}$$

Life safety level:

A reduced value of the global ductility, or equivalent ductility, to take into account the cumulative damage effects (Fajfar 1992) is computed for each component based on the established damage index.

Accordingly, system ductility will be the ratio of the displacement of the component with the least displacement capacity to the yield displacement of the system

$$\mu_{syst} = \frac{\min(D_{yi}\mu_{eqi})}{D_y} \tag{13}$$

Maximum top displacement based on drift limit $\theta_{ls,\,\mathrm{lim}}$: According to suggested deflection shapes (Fajfar *et al.* 1996) for walls (14) and for frames (15)

$$D_{\theta w} = HC_{ls}\theta_{ls,\,\mathrm{lim}} \tag{14}$$

$$D_{\theta f} = \frac{HC_{ls}\theta_{ls,\,\mathrm{lim}}}{2 - 1/\mu_{syst}} \tag{15}$$

The maximum displacement at the top $D_{\theta ls}$ will be a weighted mean according to the relative strength assigned to frames and walls.

The ratio of this displacement to the yield displacement is an upper bound for the ductility. The available ductility will be the least between this limit and the system ductility

$$\mu_{disp} = \min(D_{\theta ls}/D_y, \mu_{syst}) \tag{16}$$

Finally the maximum displacement at the top will be

$$D_{ls} = \mu_{disp} D_y \tag{17}$$

2.6 Seismic demands

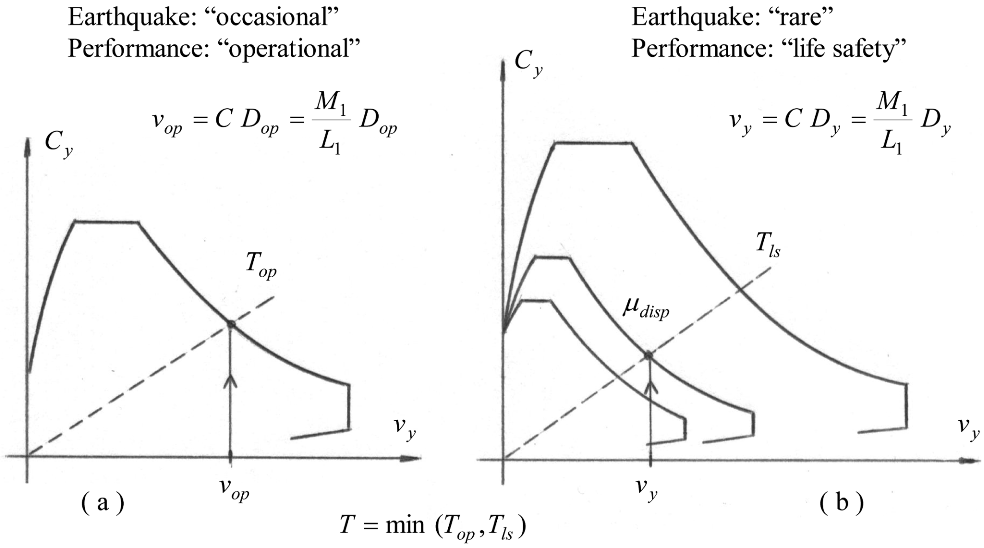
For the operational level, with the limit displacement at the top of the equivalent one degree of freedom system (Fajfar and Gaspersic 1996) and the YPS for occasional earthquakes, an upper bound, namely T_{op}, for the period is determined, see Fig. 4(a).

For the life safety level, with the top yield displacement of the equivalent one degree of freedom system, the available ductility and the YPS for rare earthquakes, another upper bound for the period, named T_{ls}, is determined, see Fig. 4(b).

The value to be adopted will be the least of both, because it leads to a larger required strength.

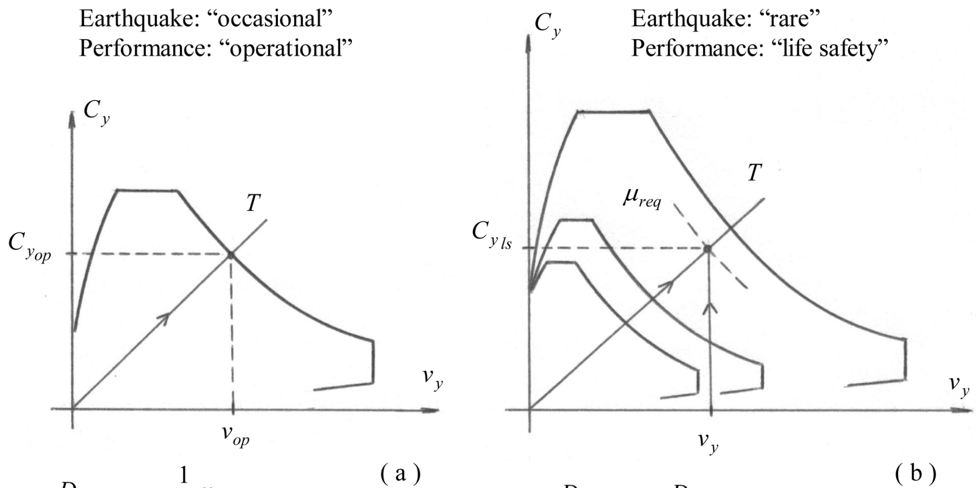
With this value, from the YPS for occasional earthquakes, the base shear for the operational level is determined. If the controlling period comes from rare earthquakes, a new required displacement at the top is obtained, see Fig. 5(a).

In the same way, with the period and the top yield displacement of the equivalent one degree of freedom system, from the YPS for rare earthquakes the base shear for the life safety level is



Note: M_1 : generalized mass of first mode of vibration
L_1 : first modal earthquake-excitation factor

Fig. 4 Period



$$D_{op,actual} = \frac{1}{C} v_{op,actual}$$

$$V_{op} = C_{y,op}\ g\ M_{e1}\ , \quad M_{e1} = \frac{L_1^2}{M_1}$$

$$D_{ls,actual} = D_y\ \mu_{req}$$

$$V_{sv} = V_y = C_{y,ls}\ g\ M_{e1}$$

Fig. 5 Strenghts

obtained. Besides, if the period is controlled by occasional earthquakes, the required ductility is obtained. This ductility will be less than the available ductility. The new top displacement may be obtained by the product of the required ductility by the yield displacement, see Fig. 5(b).

2.7 Strenght-stiffness of each component

The strength assigned to each component for the life safety level is a designer choice (Paulay 2001).

One option is to assign strengths based on cracked flexural stiffnesses. It is also possible to judiciously and arbitrarily assign strengths to improve the eccentricity of the system for each performance level.

If D_y was obtained by Eq. (10) the strength assignment should follow the same criterion. A significant increment of the required strengths would imply a deficit of available strengths and a check on the maximum reinforcement ratios would be necessary.

2.8 Torsional effects

Torsional effects are statically and independently evaluated for each principal direction.

The aim is to verify that story drifts remain within prescribed limits. Dynamic analysis and skew seismic attack should be included in the final design when necessary.

To consider the torsional effects, the procedure described above shall be completed for the two principal directions, because the strength and stiffness of each component are needed.

For each direction and for each performance level the base torsional moment is computed (Paulay 2001)

For the operational level

$$M_{t,op} = V_{op} \cdot e_{d,op} \tag{18}$$

$$e_{d,op} = 1.5e_{op} + 0.1L \tag{19}$$

$e_{d,op}$ being the design eccentricity for the operational level, e_{op} distance from the center of mass to the center of stiffness of the components in the direction being analyzed, and 0.1 L is the accidental eccentricity with L the length of the plan perpendicular to it (INPRES-CIRSOC 103, 2000).

For the life safety level

$$M_{t,ls} = V_{ls} \cdot e_{d,ls} \tag{20}$$

$$e_{d,ls} = e_{ls} + 0.1L \tag{21}$$

$e_{d,ls}$ being the design eccentricity for the life safety level, e_{ls} the eccentricity from the center of mass to the center of strength of the components in the direction being considered, and 0.1 L is the accidental eccentricity with L the length of the plan perpendicular to it (INPRES-CIRSOC 103, 2000). If the location of the center of mass is different in the different stories, the global center of mass shall be determined.

The twist angles at the top will be

For the operational level

$$\varphi_{op} = \frac{M_{t,op}}{\Sigma k_y x^2 + \Sigma k_x y^2} \tag{22}$$

The stiffnesses of the components in both directions are considered.

For the life safety level

$$\varphi_{ls} = \frac{M_{t,ls}}{\Sigma k_p d_r^2} \tag{23}$$

Only the stiffnesses of the components perpendicular to the direction being analyzed are considered because it is assumed that they remain in the elastic range and are the only available to resist the twist induced torsion. This assumption shall be verified. From the twist angles, the top displacement of the component farther from the twist center is computed. The twist center is the center of stiffness for the operational level and the center of strength for the life safety level. This displacement is added to that due to translation and with this total displacement the maximum drift is determined and compared with the corresponding prescribed limit.

If the displacement at the top is less or equal that the established limit the preliminary design is ended, if not, the design process shall be restarted multiplying the top displacement by the ratio of the limiting drift to the maximum drift. It is worth noting the need for the designer to check the reinforcement ratios, and change the cross sections dimensions if the values exceed the maximum permitted.

3. Representation of ground motion

Synthetic ground motions are generated by a filtered white noise stochastic process. The spectral density function is given by Clough (1975)

$$S_{XX}(f) = S_0 \frac{1 + 4\xi_g^2 (f/f_g)^2}{[1 - (f/f_g)^2]^2 + 4\xi_g^2 (f/f_g)^2} \frac{(f/f_f)^4}{[1 - (f/f_f)^2]^2 + 4\xi_f^2 (f/f_f)^2} \tag{24}$$

Where

S_0 : spectral density function of the white noise

f_g, ξ_g : characteristic frequency and damping ratio of the soil

f_f, ξ_f : filter parameters to attenuate the very low frequency components.

From Eq. (24) an artificial accelerogram is given by

$$x(t) = I(t) \sum_{n=1}^{NFR} \{4 S_{XX}(n\Delta f)[1 + \delta_S R_N]\Delta f\}^{1/2} \operatorname{sen}(2\pi n \Delta f t + \theta_n) \tag{25}$$

Where

NFR: the number of frequencies from 0 to $f_{max.}$. NFR $\geq f_{max}\, T_0$, T_0 being the duration of the record to be generated.

δ_S : variation coefficient to consider the uncertainties in the ordinates S_{XX}

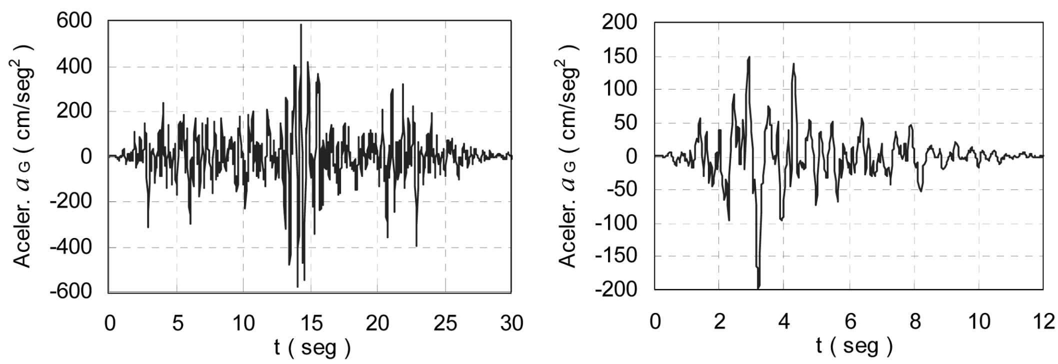
R_N : normal standard variable

θ_n : random phase angles with uniform distribution from 0 to 2π

$I(t)$: module function to consider non stationary amplitudes.

The synthetic accelerogram is subjected to a base line correction to minimize the mean square value of the velocity. Then it is scaled to the maximum ground acceleration a_G.

Based on a seismic microzonation study for the Mendoza City, Argentina (INPRES 1995), the maximum ground acceleration becomes: $a_G = 0.6\, g$ for rare earthquakes with a mean recurrence period of a 475 years, and $a_G = 0.2\, g$ for occasional earthquakes with a mean recurrence period of 72 years. Soil characteristic gives f_g = 2 Hz. The ground motion duration is assumed to be T_0 = 12



Rare earthquake Occasional earthquake

Fig. 6 Acceleration time histories

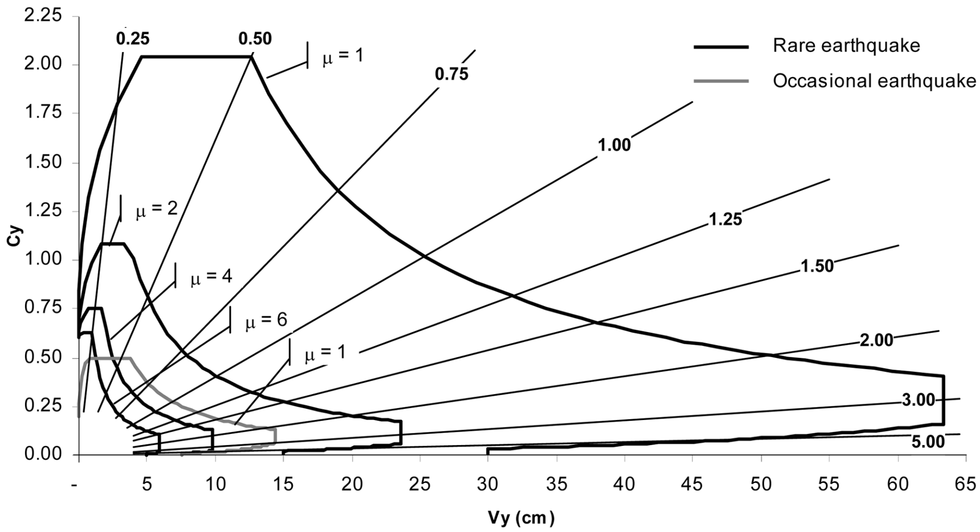


Fig. 7 Yield point spectra

sec for occasional earthquakes and T_0 = 30 *sec* for rare earthquakes. Accelerograms for both earthquakes are shown in Fig. 6.

Inelastic response spectra are computed for each accelerogram. The Clough model is used for an elasto-plastic oscillator with 2% strain hardening. Varying the yield strength of the oscillators at each period, ductility spectra for $\mu = 1$ (elastic), 2, 4 and 6, are obtained.

For each recurrence period and each ductility value, the mean plus one standard deviation ($X_m + 1\sigma_X$) is computed. The resultant curves are then smoothed to obtain the design spectra shown in Fig. 7. In this figure, $C_y = F_y/m$ is the yield force per unit mass, and v_y the yield displacement.

4. Inelastic assessment

The resultant structure from the preliminary design, proportioned and detailed according to the Argentine code, INPRES-CIRSOC 103 2000, is subjected to inelastic static and dynamic analyses to verify that the response yields within the established design limits.

To model the system, the structure is discretized in vertical resistant planes (components) connected at each level by rigid slabs in its own plane and flexible out of it, see Fig. 8. The model has three degrees of freedom per level, two horizontal displacements and a twist around the vertical axis.

Each resistant plane is also discretized with bar elements to consider the different mechanisms that contribute to the hysteretic behavior of the critical zones of R/C members, Möller (2001), Möller and Foschi (2003). Each element is composed of sub elements: (i) Elasto-plastic sub element: Represent the elastic behavior of the member and the nonlinear response at the ends with variable length according to the load history. (ii) Connection sub element: to characterize the rotation at the interface member-joint due to bond degradation of the longitudinal bars passing through the joint. Rigid ends to consider joints with large dimensions.

The stiffness of the system is computed assembling the stiffness of each resistant plane. The latter

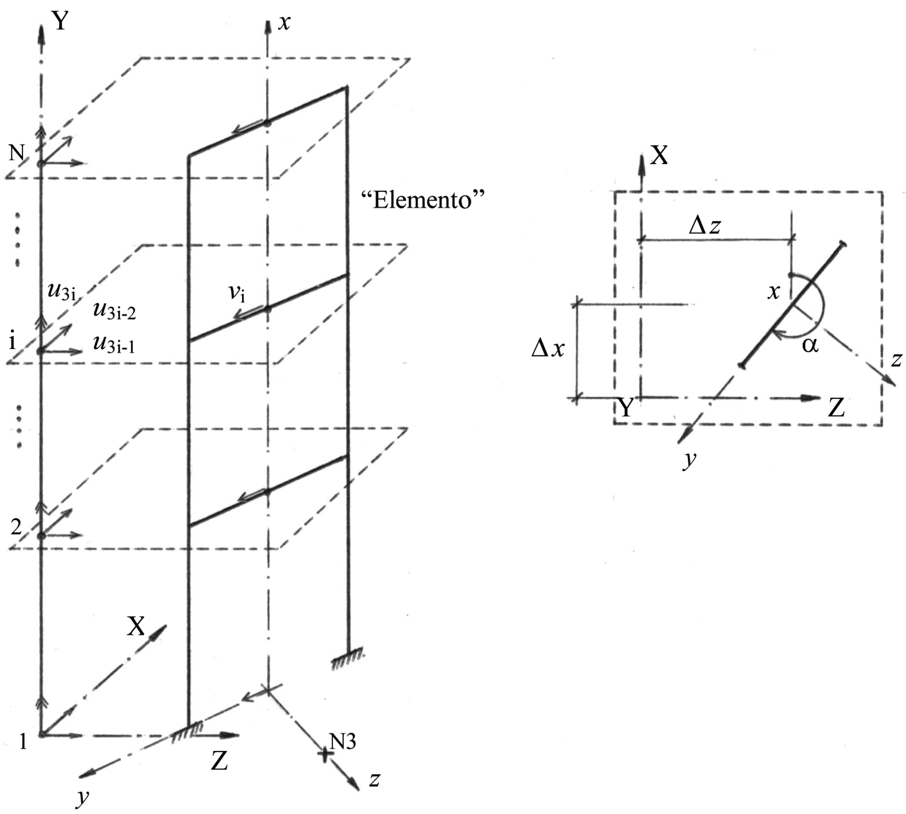


Fig. 8 Structural system – Generic element

also obtained from the stiffness of the bar elements. Masses are lumped at each level. The rotary inertia and the eccentricity from the center of mass to the Y axis are also considered. Rayleigh proportional damping is assumed with a linear combination of mass and initial stiffness.

Gravity loading is included in each plane. This allows considering the influence on the internal forces, the plastic hinging of the critical zones and the evaluation of another response parameters.

The system of non linear equations is solved by direct step by step integration using the Newmark method and iterations according to the Newton-Raphson scheme to reach equilibrium between external loads and internal forces. The DINLI program is used, Möller (2001).

This model combines sufficient accurate results and necessary simplicity to analyze spatial systems of practical interest in earthquake engineering. The lateral loads for push-over analysis or the accelerogram for dynamic analysis are applied at the global system. Accidental torsion is considered, according to the Argentine code, by a displacement of the center of mass of $\pm$ 0.1 L. L being the plan dimension perpendicular to the direction being analyzed. Nominal values for the strength and stiffness parameters are used to compare the results with the preliminary design.

Global response parameters as displacement of the center of mass at the top level, base shear and Park & Ang damage index are obtained. Maximum story drift and damage index are computed for each resistant plane.

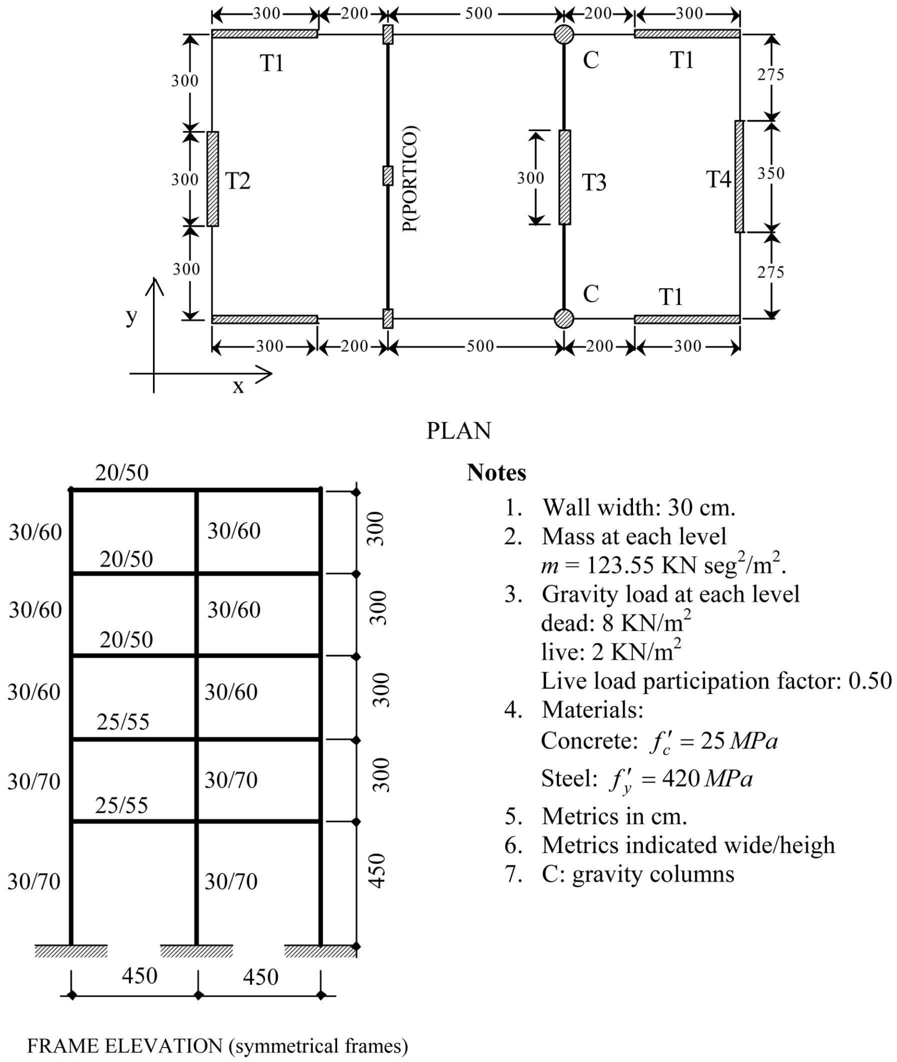


Fig. 9 Structural system data

5. Example

The structural system to be analyzed according to the proposed method is shown in Fig. 9. The design ground motions were obtained from the data of a microzonation study of Mendoza City, Argentina, and are represented in Fig. 7 by YPS.

Acceptance Criteria

For occasional earthquakes, operational performance level: $\theta_{op,\,\lim}$ = 0.70% and elastic response.

For rare earthquakes, life safety performance level : $\theta_{ls,\,\lim}$ = 2.00%, DM : 0.60 (system), 0.75 (comp).

For both directions: C_{op} = 0.80, C_{sv} = 0.80.

This choice is related with the decrease of the asymmetry in the Y direction, due to the somewhat

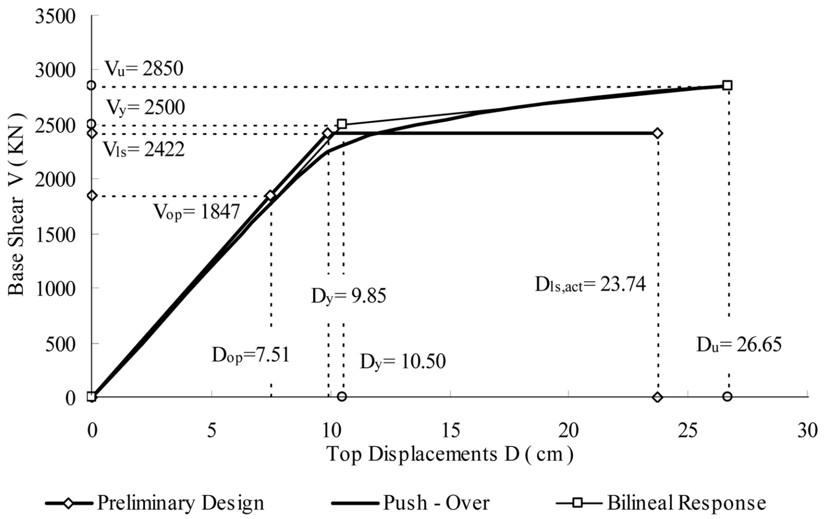


Fig. 10 Preliminary design and push-over

Table 1 Results: preliminary design and push-over

Structural Parameter	Preliminary Design	Push-Over
V_{ls} (KN)	2422	-
V_{op} (KN)	1847	-
V_Y (KN)	-	2500
V_u (KN)	-	2850
D_Y (cm)	9.85	10.50
$D_{ls,act.}$, D_u (cm)	23.74	26.65
μ_{req}	2.61	2.54
P_{T2}	0.240	0.243
P_{T3}	0.220	0.220
P_{T4}	0.270	0.242
$P_{Frame.}$	0.270	0.296
DM	0.600	0.343
θ_{T2} (%)	2.000	1.952
θ_{T3} (%)	1.710	1.932
θ_{T4} (%)	1.940	1.921
$\theta_{Frame.}$ (%)	1.740	1.942

Table 2 Results of the non linear dynamic analyses

Structural Parameter	Occasional earthquake $\bar{x} + 1\sigma_x$	Rare earthquake $\bar{x} + 1\sigma_x$
$V_{\text{máx}}$ (KN)	1875	3543
$D_{\text{máx}}$ (cm)	7.10	22.596
$Rot_{\text{máx}}$ (rad)	0.00637	0.01812
μ_{req}	0.676	2.152
DM	0.042	0.404
DM_{T2}	0.022	0.465
DM_{T3}	0.021	0.479
DM_{T4}	0.032	0.518
$DM_{Frame.}$	0.070	0.326
θ_{T2} (%)	0.893	2.599
θ_{T3} (%)	0.672	1.932
θ_{T4} (%)	0.842	2.554
$\theta_{Frame.}$ (%)	0.681	1.944

Glossary:

V_{ls}: required strength for life safety performance level
V_{op}: required strength for operational performance level
V_y: base shear at yield
V_u: Ultimate base shear adopted for push-over
D_y: Top yield displacement
$D_{ls,act}$: Top displacement for life safety performance level
D_u: Top displacement at V_u
μ_{req}: required ductility
P_i: relative base shear strength of each component
DM: damage index
θ_i: maximum story drift for each component
Rot: floor twist

arbitrary strength assignment to the different components.

To verify the obtained results, a push over and a step by step non linear dynamic analysis were performed for the *Y* direction. The reinforcement contents were obtained following the prescriptions of the new Argentine Seismic Code, INPRES-CIRSOC (2000).

Results of the preliminary design and those of push-over analysis are shown in Fig. 10 and in Table 1. For inelastic assessment nine synthetic accelerograms were generated according to Eq. (25). The model described above was used to obtain the response parameters. Similarly to the criteria used to develop design spectra, $R_m + 1\ \sigma_R$ values were computed. The results are shown in Table 2.

6. Analysis of results

A good agreement between the results of the preliminary design and those of the push-over analysis is noted. The agreement is related to strength (V_{ls} vs V_y), displacement (D_y, $D_{ls,act}$ vs D_u), strength assignment among the components and story drift. Differences exist in the damage index.

From the comparison between the dynamic results and the limit states adopted in design, it follows

Operational performance for occasional earthquakes:

- Global elastic behavior with $\mu_{req} < 1$ was obtained ($D_{\max} < D_y$).
- A good agreement between the maximum top displacements $D_{\max}$ y D_{op} exists.
- The maximum story drifts at walls T2 and T4 are larger than the established limit of 0.7%. These two walls are the most affected by torsion.

Life safety performance for rare earthquakes:

- Ductility demand is similar to that estimated by the preliminary design.
- The damage index for the system and each component is less than the established limit.
- The story drifts are around the limits: 2% for the wall T3 and the frame; approximately 2.6% for T2 and 2.5% for T4 located at the perimeter of the plan.

The results would show the convenience to obtain preliminary design story drift results more conservative.

7. Conclusions

A preliminary design methodology for earthquake-resistant structural systems based on displacements was presented. The methodology is within the currently accepted framework of performance based design and presents a step forward over past studies on the subject. An improvement in the computation of the top yield displacement for frames was introduced. The invariance of the yield displacement and the dependence of strength and stiffness to diminish the eccentricity for different levels of performance were highlighted.

According to the initial lay-out, the global ductility and the invariant top yield displacement of the system in each principal direction are established as input data

Although the methodology is suitable for considering several performance levels, only two have been included for the sake of clarity. The performance levels are operational for occasional earthquakes and life safety for rare earthquakes. The mean recurrence period for occasional earthquakes is 72 years (50% probability of exceedance in 50 years). The mean recurrence period for rare earthquakes is 475 (10% probability of exccedance in 50 years). For each performance level, requirements for displacements and deformations are established. Design spectra in yield point spectra format, which are especially suitable for the introduction of the yield displacement as the invariant parameter in the design process, have been used.

From these input data, a preliminary design methodology has been developed. The methodology is conceptually transparent; it uses simple calculations and the strength-stiffness dependence. It is applicable to asymmetric systems as the torsion is treated explicitly.

A five story asymmetric building with frames and walls (dual system) is presented as an example of application of the methodology. The building is located in Mendoza City, Argentina, placed in the most hazardous seismic zone of the country. The example confirms the accuracy of the methodology. Non linear static and dynamic analyses were performed for performance assessment. The obtained results are useful for the practical application and for future improvements of the methodology, basically on the treatment of torsion.

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