



PRELIMINARY DESIGN AND INELASTIC VERIFICATION OF EARTHQUAKE-RESISTANT STRUCTURAL SYSTEMS

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SUMMARY

A displacement-based design methodology for a preliminary seismic design of structures is presented. The approach includes two ground motion input levels: occasional and rare. The occasional earthquake level design is governed by elastic structural response and controlled by limiting inter-story drifts provided by codes for serviceability states. The limiting engineering states associated with the exceptional ground motion level have been adopted as: maximum inter-story drifts given by standards, global ductility and damage index. By using these limiting states within the conceptual design philosophy, and based on a simple hand-made calculations, structural components are sized.

The seismic demand is defined by both the elastic and the inelastic displacement response spectra for the occasional and rare earthquake input level respectively.

The system yield displacement is derived from the components geometry and longitudinal rebar yielding. Thus, the structural stiffness is computed as the ratio of the strength suitable provided to the mentioned yielding deformation.

Preliminary design results are verified by both push-over and dynamic time history analyses, applying a 3D mathematical model with components connected by rigid slabs at each story level. Three degrees of freedom per story are assumed: two horizontal displacements and a twist around the vertical axis. Each component is discretized by nonlinear bar elements. The Newmark algorithm is applied for the step-by-step integration of the equation of motion. The equilibrium at each time-step is achieved by using the Newton-Raphson iterative scheme.

Numerical example of a conventional buildings with asymmetry-plan is presented which can be suitable assessed by the proposed approach since the post-elastic torsion effects are taken into account in the evaluation

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INTRODUCTION

There is a broad consensus that the seismic design be based on displacements, corresponding deformations, ductility and damage indexes [1], [2], [3], [4]. It is also accepted that the structure should satisfy performance objectives for different levels of ground motion input levels. Acceptance criteria are based on limits established for each design level [5], [6].

One desirable condition for the preliminary design of a multistory building structure is that the initial structural layout proposed by the designer according to architectural requirements, be controlled and eventually modified to fulfill its intended earthquake-resistant purpose following a simple and transparent methodology according to the design conditions. It should be clearly stated that the structural system is developed following these principles [1] and that the designer performs the preliminary design stage by simple and, as far as possible, handmade calculations.

These guidelines have been followed in a previous study [3], but they were applied to planar structures. Spatial systems were treated in a later study [7], resulting in a trial and error procedure performed with a computer code. A preliminary design methodology was developed in a subsequent study [8]. The methodology, based on the conceptual design philosophy [8], combined the advantages of the two above, and covered asymmetric spatial systems using simple calculations. Current concepts were used [9], [10], [11]. The strength and stiffness properties of a member were considered interrelated and the yield displacement as an independent parameter based on the geometric characteristics and the yield strain of the reinforcement.

An improved preliminary design methodology is presented in this paper. The initial conditions are clearly established, that is, the intrinsic characteristics of the structural system, i.e. top yield displacement and global ductility, and the acceptance requirements. The top yield displacement of the system is determined at the beginning, based on the corresponding component yield displacements (in this paper the term component is used to indicate a vertical resistance plane of the structure). Thus, the top yield displacement plays the role of an invariant design parameter.

The initial conditions are completed by establishing the acceptance requirements for two performance levels: operational for occasional earthquakes and life-safety for rare earthquakes. Elastic response and an inter-story drift index limit are established for the operational level. Inelastic response and limiting conditions for the structural system such as global ductility, Park and Ang damage index [12] and inter-story drift index, are established for the life safety level. A recurrence period of 475 years (10% probability of exceedance in 50 years) is considered for rare earthquakes, while 72 years (50% probability of exceedance in 50 years) for occasional earthquakes.

An operational improvement is the use of yield point spectra (YPS) [13] combined with the capacity curve (capacity diagram), for the equivalent SDOF system [14]. The YPS is especially suitable for considering the yield displacement as an invariant design parameter. On the other hand, the capacity curve seems attractive to designers familiarized with typical push-over results. Three acceleration time histories were used for each level of ground motion. The time histories were based on a microzonation study of Mendoza city in Argentina [15]. The corresponding YPS for each ground motion level was determined by computing the mean plus one standard deviation of the three spectra. For rare earthquakes, the inelastic response for different ductility levels was computed while for occasional earthquakes only the elastic response is needed.

In what follows, the preliminary design methodology is described, including the corresponding flow diagram. Next, a preliminary design example is presented. The example consists of a five story

asymmetric dual system (frames and walls), located in Mendoza city, Argentina. The preliminary design results are then compared with the results obtained from non linear analyses, i.e. push-over and time history, applied to a model of the structural system with three degrees of freedom per floor. Finally, conclusions are drawn and possible future improvements are suggested.

DESIGN METHODOLOGY

Notation

As appear in the flow diagram

D_{Yi} : top yield displacement of a component	T_{sv} : lower bound period for rare earthquakes (life safety level)
D_Y : top yield displacement of the system	T : minimum value of T_{op} and T_{sv} , required and adopted for the period $D_{op,act}^{(1)}$: current value of $D_{op}^{(1)}$.
μ_{ui} : global displacement ductility of a component	$D_{op,act}$: current value of D_{op} .
μ_u : global displacement ductility of the system	$C_{Y,op}$: required seismic coefficient for occasional earthquakes (operational level)
θ_{op} : limit inter-story drift index for the operational level	g : gravity acceleration
D_{op} : limit displacement at the top of the structure for the operational level	M_{ei} : effective mass of the first mode
θ_{sv} : limit inter-story drift index for the life safety level	V_{op} : required base shear for the operational level
DI : global damage index of the system	μ_{req} : required global ductility
C_{op} : reduction coefficient at the operational level to consider torsional effects	$D_{sv,act}$: current value of D_{sv} .
C_{sv} : reduction coefficient at the life safety level to consider torsional effects	$C_{Y,sv}$: required seismic coefficient for rare earthquakes (life safety level)
H : total height of the system	V_Y : required yield base shear for rare earthquakes.
$D_{op,\theta}$: limit displacement at the top of the system related to θ_{op} and C_{op} .	V_{sv} : required base shear for the life safety level ($V_{sv}=V_Y$)
$D_{sv,\theta}$: limit displacement at the top of the system related to θ_{sv} , C_{sv} and μ_{eq} .	j : index for the operational level and for the life safety level
μ_{eq} : equivalent global ductility to consider cumulated damage effects	M_{tj} : torsional moment at the base for the j performance level
D_{sv} : limit displacement at the top of the system for the life safety level	$e_{d,j}$: design eccentricity for the j performance level
μ_{disp} : available global ductility	ϕ_j : twist angle at the top for the j performance level
$D_{op}^{(1)}$: equivalent SDOF displacement related to D_{op} .	$D_{i,j}^t$: torsional displacement at the top of an component for j performance level
$D_Y^{(1)}$: equivalent SDOF displacement related to D_Y .	$\text{Max } D_j$: maximum value of $D_{i,j}^t$
YPS : yield point spectra for rare earthquakes	$\text{Max } D_j$: maximum displacement at the top of the component corresponding to $\text{max } C_j$
YPS (occas.): yield point spectra for occasional earthquakes	$\text{Máx } \theta_j$: maximum inter-story drift index corresponding to $\text{max } D_j$.
T_{op} : upper bound period for occasional earthquakes (operational level)	

Description of the methodology

In what follows it will be described sequentially the mathematical model, the intrinsic characteristics of the proposed structural system, i.e. top yield displacement and global ductility, the design requirements, the input motions, and the demand results expressed as base shear and displacements at the top which establish minimum global requirements for strength–stiffness for the system and each component. Finally torsional effects, which were a priori estimated, are verified.

- **Mathematical model:** Structural system composed by resistance components (frames and/or walls) connected at floor levels by rigid slabs with three degrees of freedom per floor.
- **Initial Conditions:**
Intrinsic characteristics of the structural system: The geometry of the system is adopted according to functional requirements and engineering judgment. Generally the structure is arranged with components in two orthogonal directions. Before considering torsional effects, the following should be applied independently in each direction.

For walls the top yield displacement can be determined once the wall lengths l_i and the yield strain of the longitudinal reinforcement are known [11], for instance, for rectangular walls with reinforcement concentrated at both ends:

$$D_{yi} = \frac{11}{40} \frac{1,8 \epsilon_y}{l_i} \quad (1)$$

For regular frames the top yield displacement can be determined by the yield inter-story drift index [10]. The following procedure has been used in this paper. Each frame is analyzed independently for gravity loads and for horizontal forces with an inverted triangular distribution in height. The resultant of these forces should be equal to unity. The resulting bending moments and displacements at the top respectively are: M_v, D_v , and M_1, D_1 , . Code values are used for moments of inertia and areas of the cross sections. These values take into account approximately the cracking effects [16]. With the geometry of the cross section of each member and the yield strain of the longitudinal reinforcement, the yield curvature is evaluated [10], and with it and the bending stiffness the yield moment M_y .

Following principles of capacity design, a collapse mechanism for the life safety level is proposed. For instance, a partial beam sway mechanism extending two third in height of the structure. At each plastic hinge:

$$M_{vi} + C_i M_{li} = M_{yi} \quad (2)$$

where: $i = 1, \dots, n_r$, n_r being the number of plastic hinges. The equation is solved for C_i , and the mean value is obtained:

$$C = \frac{\sum C_i}{n_r} \quad (3)$$

Finally, a sufficient accurate value for the top yield displacement at the preliminary design stage results:

$$D_y = D_v + CD_1 \quad (4)$$

Besides, the base shear related to the code stiffnesses is obtained:

$$V_y = CV_1 \quad (5)$$

It is necessary to point out that D_y is an intrinsic property of the frame, while V_y may change by changing the resistance-stiffness characteristics of the members [9], [10], [11]. Similarly, it is possible to determine the yield moment at the base and the base shear for the walls. Then, a bilinear relationship base shear-top displacement without strain hardening is assumed and the yield displacement of the system results:

$$D_y = \frac{\sum_{i=1}^n V_i}{\sum_{i=1}^n (V_i / D_{yi})} \quad (6)$$

where V_i is the base shear of a component, frame or wall, and n the number of components of the system in the direction considered.

Alternatively, the top yield displacement of the system can be determined approximately as the mean value of the top yield displacements of the components:

$$D_y = \frac{\sum D_{yi}}{n} \quad (7)$$

Note that in this approach the base shear is not needed. Anyway, the first approach may be useful for the designer as a guide in a further step of the design process, when the required base shear be distributed among the components of the system.

The global ductility of the system may be evaluated as the mean weighted value of the code ductilities for frames and walls, according to the adopted share of resistance by the designer.

Acceptance requirements: Operational performance for occasional earthquakes and life safety performance for rare earthquakes are established. Accordingly, limiting values for the inter-story drift index, 0.7% and 2%, are adopted. Besides, elastic behavior for the operational level and a value of the Park and Ang damage index of 0.6 [12] for the life safety level are also adopted.

- **Top displacement and ductility limits:**

The limiting values of the inter-story drift index for each performance level are judgmentally reduced, according to the arrangement in plan of the components of the system, to make an a priori allowance of the torsional effects, which will be explicitly considered later.

Thus, the top displacement for the operational level corresponding to the limit inter-story drift index results:

$$D_{op,\theta} = HC_{op} \theta_{op} \quad (8)$$

Finally, considered the imposed elastic performance the top limit displacement will be:

$$D_{op} = \min(D_y, D_{\theta}) \quad (9)$$

For the life safety level it is necessary to consider the cumulative damage effects by reducing the global ductility (equivalent ductility) [17]. Thus, the limit inter-story drift index results [14]:

$$D_{sv,\theta} = HC_{sv} \theta_{sv} \quad \text{for walls} \quad (10)$$

$$D_{sv,\theta} = \frac{HC_{sv} \theta_{sv}}{2 - 1/\mu_{\max}} \quad \text{for frames} \quad (11)$$

Finally, the top limit displacement will be the weighted mean of frames and walls.

The top displacement-yield displacement ratio is an upper bound for the ductility. The available ductility will be the less value between this limit and the equivalent ductility.

- **Period and base shear:**

For the operational level, an upper bound for the period is obtained with the aid of the top displacement of the equivalent SDOFS [14] and the YPS[13],

Similarly, for the life safety level with the top yield displacement of the equivalent SDOFS [14], the YPS [13] and the available ductility, another upper bound limit for the period is obtained.

The required and adopted period will be the less of the two above.

Then, the base shear for the operational level can be obtained from the corresponding YPS considering the adopted period and the effective mass of the first mode. If the adopted period is controlled by the rare earthquake, a new updated required displacement is obtained.

The same procedure is followed for the life safety level, entering the corresponding YPS. If the adopted period is controlled by the occasional earthquake, the required ductility is obtained, which will be less than the available ductility, and a new updated required displacement is also obtained by the product of the required ductility and the yield displacement.

- ***Strength and stiffness of each component:***

The global strength of each component is obtained distributing the yield base shear among the components of the system. The assignment of strength to each component is a decision to be taken by the designer based on engineering judgment. The traditional strength assignment based on the elastic stiffnesses, cracked or uncracked, of the elements is just one option. Alternative distributions may, in certain cases, be more convenient.

Then, the global stiffness of each component can be obtained as the ratio of the global strength to the top yield displacement.

- ***Torsional Effects:***

Once completed the above procedure for both principal direction of the system, the torsional effects may be evaluated computing the torsional moment at the base for each performance level. For the operational level:

$$M_{t,op} = V_{op} \cdot e_{d,op} \quad (12)$$

$$\text{where } e_{d,op} = 1,5e_{op} + 0,1l \quad (13)$$

$e_{d,op}$ being the design eccentricity for the operational level, e_{op} the distance between the center of mass and the center of stiffness of the components in the direction considered, and l the length of the plan perpendicular to the direction being analyzed. For the life safety level:

$$M_{t,sv} = V_{sv} \cdot e_{d,sv} \quad (14)$$

$$\text{where } e_{d,sv} = 1,5e_{sv} + 0,1l \quad (15)$$

$e_{d,sv}$ being the design eccentricity for the life safety level, e_{sv} the distance of the center of mass to the center of strength or resistance of the components in the direction considered and l the same as above. If the location of the center of mass is different at each story, the global center of mass of the system should be considered.

The respective angles of twist due to torsion are evaluated as follows. For the operational level:

$$\varphi_{op} = \frac{M_{t,op}}{\sum k_y x^2 + \sum k_x y^2} \quad (16)$$

where the stiffnesses of the components in each direction are included together with the square of the distances to the center of stiffness. For the life safety level:

$$\varphi_{sv} = \frac{M_{t,sv}}{\sum k_p d_r^2} \quad (17)$$

where only the stiffnesses of the components perpendicular to the direction considered are included with their respective square distances to the center of stiffness, because it is assumed that these components remains elastic, thus contributing to resist the torsional moment.

Then, the top displacement of the farthest component from the center of twist (center of stiffness for the operational level or center of strength for the life safety level) can be calculated.

The top displacement due to torsion is added to the top displacement due to translation and the maximum inter-story drift index is determined and compared with the corresponding limit. The fulfillment of this condition completes the preliminary design. If not, the procedure is repeated.

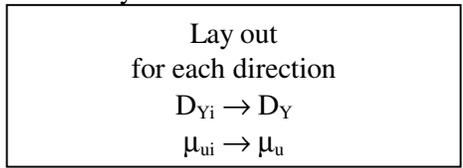
It should be noted that when assigning strength to the different components, the designer should comply with the maximum reinforcement ratios stipulated by the code.

The flow diagram for the proposed design methodology follows.

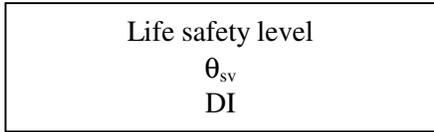
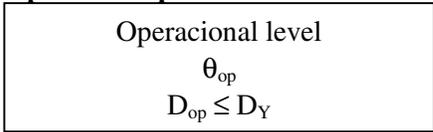
Flow diagram for the proposed design method

Initial Conditions

Intrinsic characteristics of the structural system

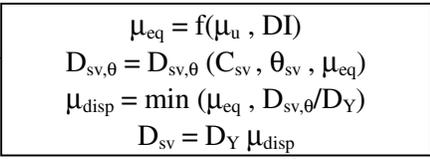
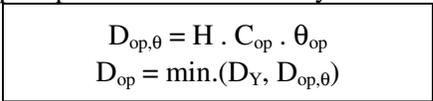


Acceptance Requirements

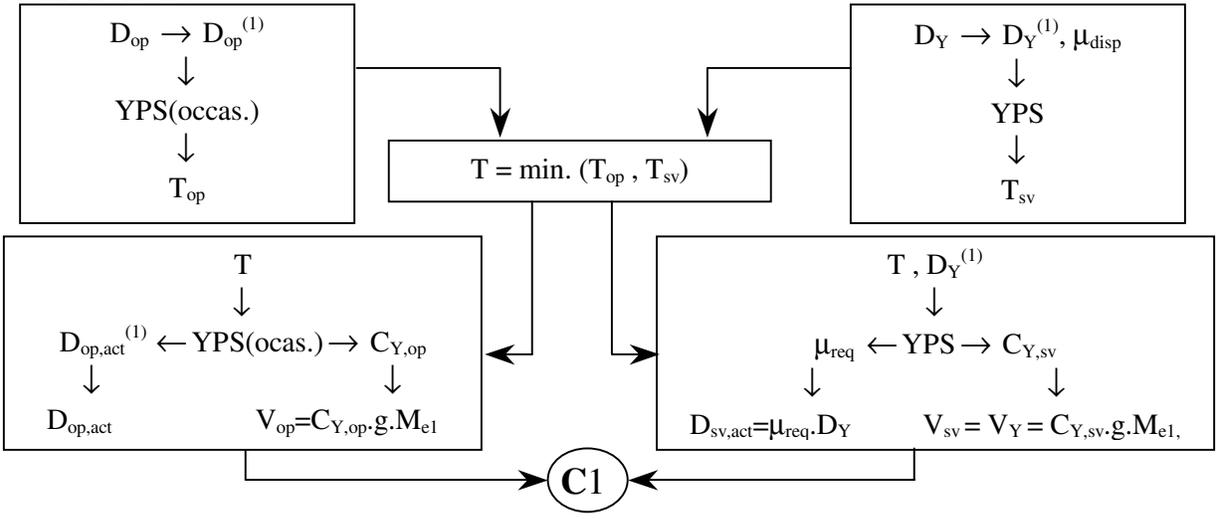


For each direction:

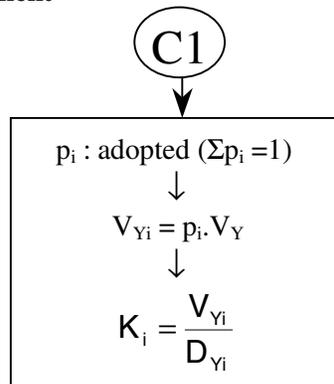
Top displacement and ductility limits



Seismic Demands



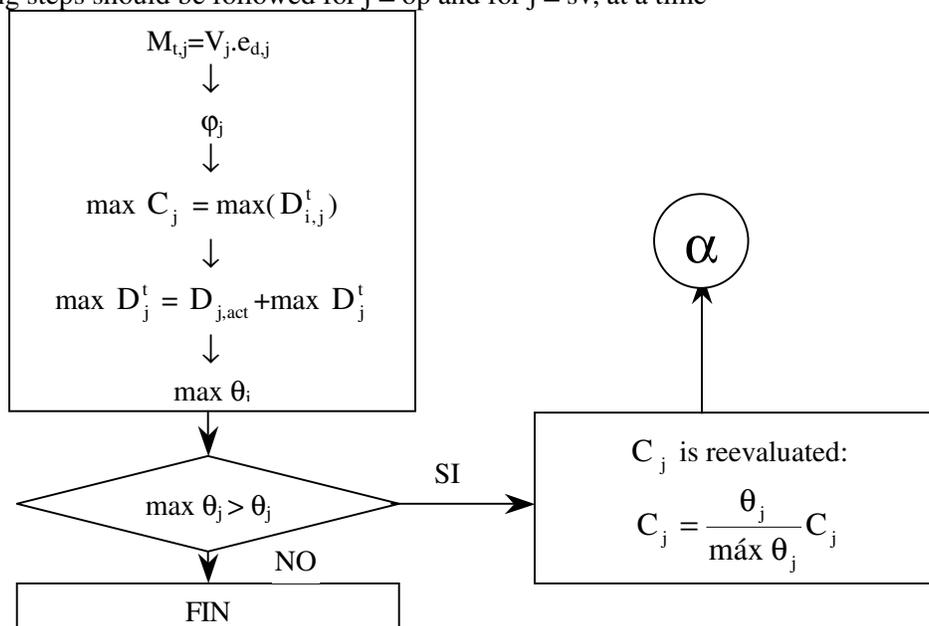
Strength and stiffness of each component



The above steps should be followed in both directions.

Torsional effects

The following steps should be followed for $j \equiv op$ and for $j \equiv sv$, at a time



INELASTIC VERIFICATION

The preliminary design code-comply structure is to be subjected to inelastic static and dynamic analyses to verify the response parameters are within the design limits.

The spatial system is composed by components connected at each floor by an assumed infinitely rigid slab in its own plane and infinitely flexible out of its plane. Thus, the model comprises three degrees of freedom per floor: two horizontal translations and one rotation through the vertical axes. The global coordinate system is arbitrary in plan.

The non linear static and dynamic analyses are performed by the finite element method formulated in displacements. The dynamic problem is solved by step-by-step direct integration of the equations of motion using the Newmark algorithm. The non linear problem at each step is solved by iteration using a variant of the Newton-Raphson scheme.

The “finite elements” of the system are the components of the structure. Displacement increments at the degrees of freedom are obtained by iteration within each step. With them and the rigid slab assumption the displacement increments at each floor in the direction of each component are evaluated. Then, the internal forces at each component are obtained. These forces are horizontal forces in the plane of each component at each floor and should comply with the global equilibrium.

To solve each component, they are discretized in bar elements which allow the different mechanisms contributing to the hysteretic behavior of the critical regions of reinforced concrete members [18] to be considered. Gravity loads are introduced at nodes. The solution at each increment constitutes by itself a non linear problem which is solved by the same iterative Newton-Raphson scheme.

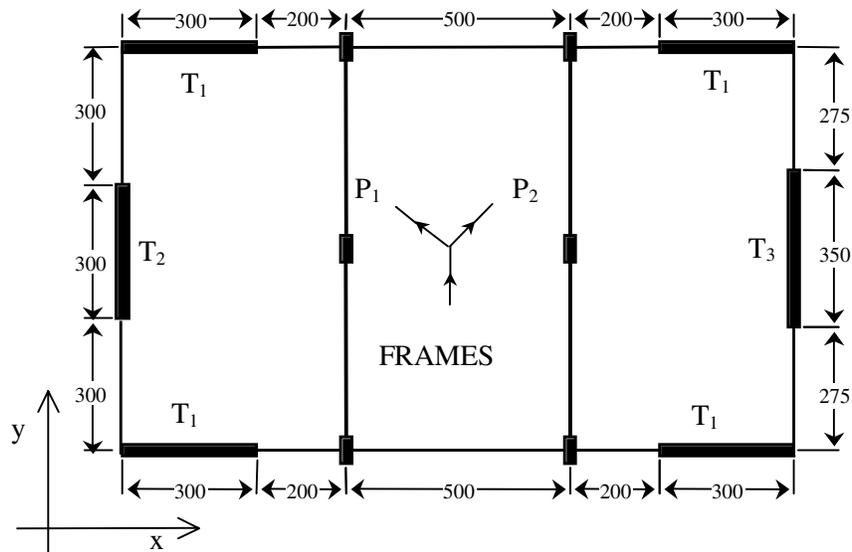
The horizontal forces for the push-over analysis, or the input acceleration time history for the dynamic analysis, are applied to the global system. Masses are assumed to be lumped at each floor level, considering the rotary inertia through the vertical axes of the system. Accidental torsion is taking into account moving the center of mass $\pm 0,1 L$, L being the plan length perpendicular to the direction considered.

As results, global response parameters are obtained (related to the center of mass) like displacements, base shear, inter-story drift indexes and damage index, also for each component. The Park and Ang model is used to evaluate the damage index of the system [12].

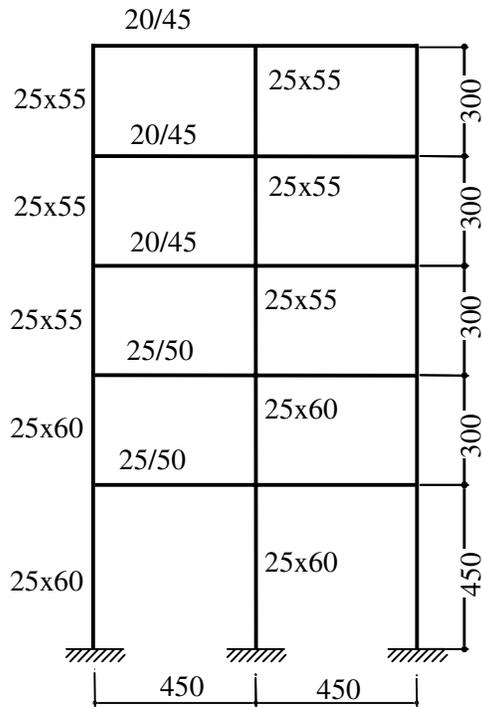
EXAMPLE

The proposed preliminary design method is applied to the structural system whose characteristics are shown in fig. 1.

The YPS [13] obtained from the seismic microzonation of Mendoza city, Argentina [15] are depicted in fig.2 .



PLANT



Notes

1. With of all walls: 25 cm.
Both frames are equal.
2. Mass at each level
 $m = 117 \text{ KN seg}^2/\text{m}^2$.
3. Gravity load at each level.
permanent: 8 KN/m^2
accidental: 2 KN/m^2
participation factor: 0,25
4. Materials:
Concrete H – 21.
Steel: ADN 420.
5. Units in cm.

FRAMES ELEVATION (symetrical frames)

Figure 1. Structural system data

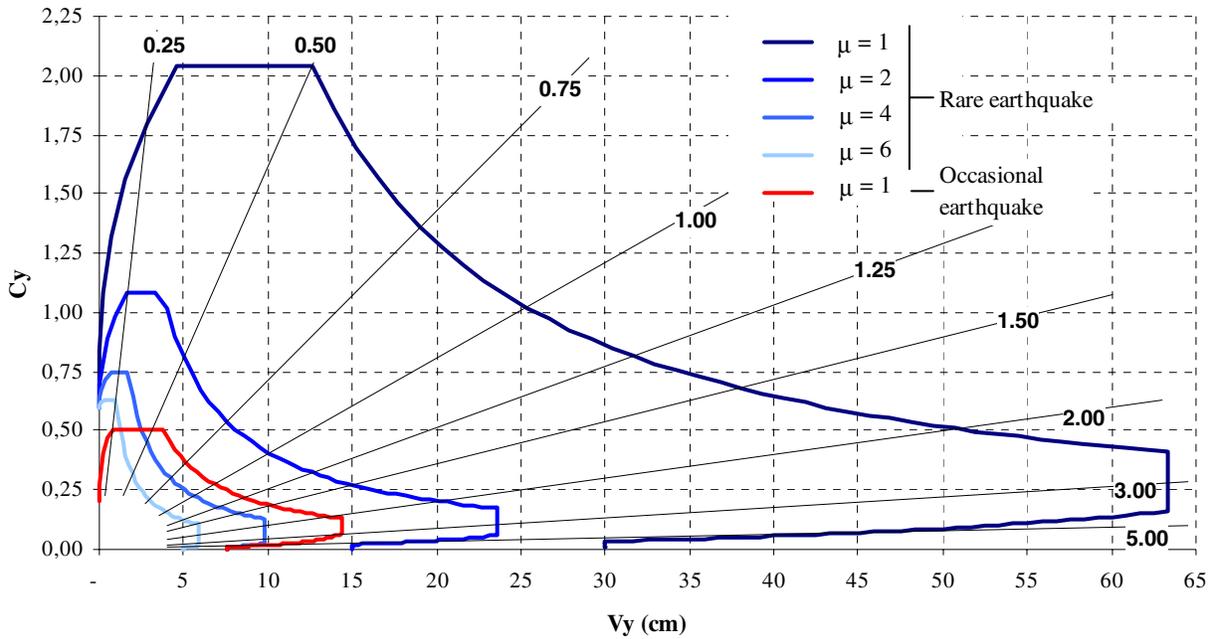


Figure 2. Yield point spectra

Following the flow diagram, data and relevant results are shown in what follows.

- Top yield displacement

Direction x: $D_{Y1} = 8,98 \text{ cm}$, $D_Y = 8,98 \text{ cm}$
 Direction y: $D_{Y2} = 8,98 \text{ cm}$, $D_{Y3} = 7,70 \text{ cm}$, $D_{p\acute{o}rt.} = 8,88 \text{ cm}$
 Base shears computed using code stiffnesses [16]:
 $V_{Y2} = 504 \text{ KN}$, $V_{Y3} = 677 \text{ KN}$, $V_{port.} = 426 \text{ KN}$
 Then, applying eq. (6):
 $D_Y = 8,47 \text{ cm}$

- Global ductility

According to [16]:

Direction x : $\mu_{u1} = 5$, $\mu_u = 5$
 Direction y: $\mu_{u2} = 5$, $\mu_{u3} = 5$, $\mu_{p\acute{o}rt.} = 6$, $\mu_u = 5,20$

- Acceptance requirements.

It is adopted: $\theta_{op} = 0,70 \%$, $\theta_{sv} = 2,00 \%$, $DI = 0,60$

- Top displacement and ductility limits. According to [17]:

$$\mu_{eq} = \frac{\sqrt{1 + 4DI\beta\gamma^2\mu_u} - 1}{2\beta\gamma^2}$$

Assuming: $DI = 0,60$ and adopting $\beta = 0,05$ and $\gamma = 1$, for the x direction : $\mu_{eq.} = 2,65$, for the y direction: $\mu_{eq.} = 2,74$

Adopting:

Direction x: $C_{op} = 0,90$, $C_{sv} = 0,90$
 Direction y: $C_{op} = 0,80$, $C_{sv} = 0,80$

Results:

Direction x: $D_{op,\theta} = 10,4 \text{ cm}$, $D_{op} = 8,98 \text{ cm}$
 $D_{sv,\theta} = 29,7 \text{ cm}$, $\mu_{disp} = \mu_{eq} = 2,65$, $D_{sv} = 23,8 \text{ cm}$

Direction y: $D_{op,\theta} = 9,2 \text{ cm}$, $D_{op} = 8,47 \text{ cm}$
 $D_{sv,\theta} \text{ (walls)} = 26,4 \text{ cm}$, $D_{sv,\theta} \text{ (frames)} = 16,1 \text{ cm}$
 Weighted mean estimated:
 $D_{sv,\theta} = 0,6 \cdot 26,4 \text{ cm} + 0,4 \cdot 16,1 \text{ cm} = 22,3 \text{ cm}$

$$\mu_{disp} = \frac{D_{sv,\theta}}{D_Y} = 2,63 \quad , \quad D_{sv} = 22,3 \text{ cm}$$

- Demands

According to [14]:

Direction x: $D_{op}^{(1)} = 0,74 \cdot D_{op} = 6,65 \text{ cm}$
 $D_Y^{(1)} = 0,74 \cdot D_Y = 6,65 \text{ cm}$
 Direction y: $D_{op}^{(1)} = 0,74 \cdot D_{op} = 6,27 \text{ cm}$
 $D_Y^{(1)} = 0,77 \cdot D_Y = 6,52 \text{ cm}$

From the YPS:

Direction x: $T_{op} = 0,98 \text{ seg}$, $T_{sv} = 0,77 \text{ seg}$
 Direction y: $T_{op} = 0,92 \text{ seg}$, $T_{sv} = 0,75 \text{ seg}$

Then:

Direction x: $T = 0,77 \text{ seg}$, $D_{op,act} = 7,10 \text{ cm}$

$$V_{op} = 1755 \text{ KN} , (M_{e1} = 503 \frac{\text{KN seg}^2}{\text{m}})$$

$$D_{sv,act} = 23,8 \text{ cm} , V_{sv} = V_Y = 2220 \text{ KN}$$

Direction y: $T = 0,75 \text{ seg} , D_{op,act} = 6,99 \text{ cm}$

$$V_{op} = 1825 \text{ KN} , (M_{e1} = 503 \frac{\text{KN seg}^2}{\text{m}})$$

$$D_{sv,act} = 22,3 \text{ cm} , V_{sv} = V_Y = 2318 \text{ KN}$$

▪ Strength and stiffness of each component

Direction x: by symmetry $p_1 = 0,25$

$$V_{Y1} = 555 \text{ KN} , K_1 = 61790 \frac{\text{N}}{\text{cm}}$$

Direction y: adopting $p_2 = 0,25 , p_3 = 0,35 , p_{p\acute{o}rt.} = 0,20$

$$V_{Y2} = 580 \text{ KN} , K_2 = 64540 \frac{\text{N}}{\text{cm}}$$

$$V_{Y3} = 811 \text{ KN} , K_3 = 105370 \frac{\text{N}}{\text{cm}}$$

$$V_{p\acute{o}rt.} = 464 \text{ KN} , K_{p\acute{o}rt.} = 52210 \frac{\text{N}}{\text{cm}}$$

▪ Torsional effects:

Direction x: máx. $\theta_{op} = 0,46 \% < \theta_{op} = 0,70 \%$

máx. $\theta_{sv} = 1,50 \% < \theta_{sv} = 2,00 \%$

Direction y: máx. $\theta_{op} = 0,63 \% < \theta_{op} = 0,70 \%$

máx. $\theta_{sv} = 2,10 \% \approx \theta_{sv} = 2,00 \%$

To verify the obtained results, a push over analysis and a step by step non linear dynamic analysis are performed for the y direction. The reinforcements were obtained applying the Argentine Code INPRES-CIRSOC 103.

Push-over results are shown in figure 3, and in Table N°1 they are compared with those from the preliminary design.

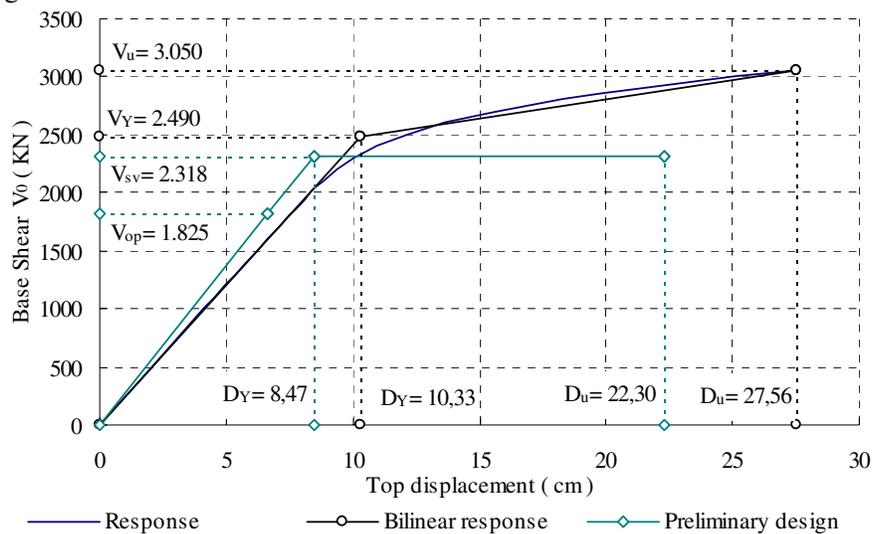


Figure 3. Push - over

Table N° 1:
Push – over and preliminary design results

Structural parameter	Preliminary design	Push – Over
V_{sv} (KN)	2318	-
V_{op} (KN)	1825	-
V_{γ} (KN)	-	2490
V_u (KN)	-	3050
D_{γ} (cm)	8,47	10,33
K (KN/cm)	273,67	241,05
$D_{sv,act.}, D_u$ (cm)	22,30	27,56
μ_{req}	2,63	2,67
P_{P1}	0,200	0,130
P_{P2}	0,200	0,132
P_{T2}	0,250	0,272
P_{T3}	0,350	0,466
DM	0,60	0,453
$\theta_{(P1)}$ (%)	1,80	2,03
$\theta_{(P2)}$ (%)	1,64	1,90
$\theta_{(T2)}$ (%)	2,10	2,16
$\theta_{(T3)}$ (%)	1,74	1,77

Table N° 2:
Non linear dynamic analysis results

Structural parameter	Occasional earthquake $\bar{x} + 1\sigma_x$	Rare earthquake $\bar{x} + 1\sigma_x$
$V_{m\acute{a}x}$ (KN)	2.039	3.484
$D_{m\acute{a}x}$ (cm)	7,732	19,790
$Rot_{m\acute{a}x}$ (rad)	0,00619	0,01940
$\mu_{m\acute{a}x}$	0,75	1,91
DM	0,082	0,328
$\theta_{(P1)}$ (%)	0,64	1,74
$\theta_{(P2)}$ (%)	0,67	1,75
$\theta_{(T2)}$ (%)	0,86	2,48
$\theta_{(T3)}$ (%)	0,76	2,26
DM _(P1)	0,098	0,418
DM _(P2)	0,107	0,405
DM _(T2)	0,024	0,392
DM _(T3)	0,032	0,360

Where :

V_u : maximum base shear from push - over

D_u : top displacement related to V_u

Good agreement is shown for displacements and deformations. The damage index obtained from the push-over is less than the limit adopted in the preliminary design. Some differences exist in the sharing of the components to the base shear.

Three accelerograms were generated and used as input motions for the rare and occasional earthquakes respectively, following the results of the seismic microzonation of Mendoza City, Argentina. Fig 4 shows one of them for each performance level.

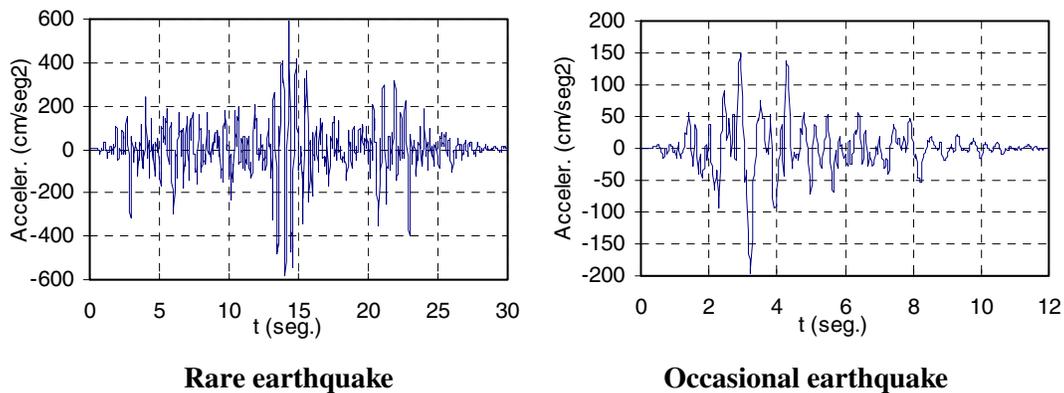


Figure 4. Accelerograms

The system was analyzed with gravity loads and seismic actions. The center of mass was moved a distance $\pm 0,1 L$. The relevant mean plus one standard deviation results (envelop of this two cases) are presented in Table N°2. From comparison of the performance levels it is concluded:

- Operational level for occasional earthquakes:
Global elastic behavior is observed with $\mu_{\text{máx}} < 1$.
The inter-story drift index in each component presents values less than the 0.7% limit for the frames and somewhat greater than 0.7% for the walls T2 and T3 located at the perimeter.
- Life safety level for rare earthquakes:
The global and component damage index is less than the limit, $DM < 0,50$.
The inter-story drift indexes present a similar pattern to the operational level, that is, less than the 2% limit for the frames and greater than 2% for the perimeter walls.

The torsional effects yield inter story drift indexes greater than those from the preliminary design in the perimeters walls, even though the system is “torsionally restrained” by components perpendicular to the analyzed direction which behave within the elastic range. This is due to the fact that the preliminary design method is static.

As a general conclusion from the analysis of the obtained results, it may be concluded that the proposed preliminary design methodology yields sufficiently accurate results.

CONCLUSIONS

A displacement-based method for preliminary design of earthquake-resistant structural systems has been presented. The method is within the frame of performance based design and constitutes an improvement to former studies on the subject.

Once adopted the structural lay-out, the intrinsic characteristics of the system for each direction, i.e global ductility and top yield displacement, are established, top yield displacement being the invariant design parameter.

Two performance levels: operational for occasional earthquakes, with mean recurrence period $T= 72$ years (50% probability of exceedance in 50 years), and life safety for rare earthquakes, with mean recurrence period of 475 years (10% probability of exceedance in 50 years), have been considered. For each level displacement and deformations acceptance requirements are established.

From these conditions, a preliminary design methodology has been developed. The methodology is conceptually transparent and based only on fundamental principles; strength and stiffness are interrelated and it is applicable to asymmetric structural systems because of the explicit treatment of torsional effects. Seismic demands are characterized by yield point spectra which are especially suitable when yield displacement is the invariant design parameter.

The methodology has been applied to a building with an asymmetric dual system (frames and walls) assume to be located in Mendoza City, Argentina. To verify the adequacy of the proposed methodology nonlinear static (push-over) and dynamic analyses have been performed. It has been shown the methodology yields sufficiently accurate results.

Further improvements like a more simple determination of the top yield displacement for frames and the influence of higher modes are currently underway.

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