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Inelastic displacement-based design approach of R/C building structures in seismic regions

Marcelo Rubinstein† and Oscar Möller‡

Instituto de Mecánica Aplicada y Estructuras, Universidad Nacional de Rosario, Riobamba y Berutti, 2000 Rosario, Argentina

Alejandro Giuliano‡†

Instituto Nacional de Prevención Sísmica, Roger Balet 47N, 5400 San Juan, Argentina

Abstract. A two-level displacement-based design procedure is developed. To obtain the displacement demands, elastic spectra for occasional earthquakes and inelastic spectra for rare earthquakes are used. Minimum global stiffness and strength to be supplied to the structure are based on specified maximum permissible drift limits and on the condition that the structure responds within the elastic range for occasional earthquakes. The performance of the structure may be assessed by an inelastic push-over analysis to the required displacement and the evaluation of damage indices. The approach is applied to the design of a five-story reinforced concrete coupled wall structure located in the most hazardous seismic region of Argentina. The inelastic dynamic response of the structure subjected to real and artificially generated acceleration time histories is also analyzed. Finally, advantages and limitations of the proposed procedure from the conceptual point of view and practical application are discussed.

Key words: performance based seismic design; earthquake resistant structures; reinforced concrete buildings; inelastic seismic analysis; damage evaluation.

1. Introduction

The widely accepted philosophy of seismic design establishes that structures should be able to withstand relatively frequent minor intensity earthquakes without structural and non-structural damage, moderate earthquakes without structural damage but with some non-structural damage, and severe rare earthquakes with structural and non-structural damage but without collapse (Park and Paulay 1975).

Accordingly, unless structures are proportioned to possess exceptionally large strength with respect to lateral forces, inelastic deformations during severe rare earthquakes are to be expected, with the strength of the structure at its yield value while maintaining the deformation capacity not less than the deformation demand.

Only one design earthquake corresponding to a severe rare earthquake is embodied in current

† Professor
‡ Research Fellow
‡† Director

seismic codes, the design earthquake being represented by a set of design spectra, normally with a 10% probability of being exceeded in 50 years.

Although the current seismic design procedures are force/strength-based, there has been a growing interest in the development of displacements-based design procedures, due to the recognition that structural and non-structural damage is more related to displacements rather than forces.

The force/strength-based design procedure has important advantages for designers, mainly simplicity and experience in its use, but presents also some drawbacks: (i) the procedure is not transparent because it is not pointed out that the inelastic structural response is governed by displacements rather than forces. In this context designers might be tempted to think that it is possible to exceed the strength demand by increasing the supplied strength, when really both strengths remain always equal; (ii) The actual inelastic response is evaluated by means of a linear elastic analysis through the use of strength reduction factors; (iii) The serviceability limit state can not be addressed explicitly.

Recently the recommendations of the International Workshop on Seismic Design Methodologies for The Next Generation of Codes held at Bled/Slovenia in June 1997 (Fajfar and Krawinkler 1997) stated that: "*Future seismic engineering practice should be based on explicit and quantifiable performance criteria, considering multiple performance and hazard levels.*" Furthermore, it was also stated that: "*The most suitable approach for seismic design to achieve the objective of performance based engineering appears to be deformation controlled design.*"

The displacement-based design approach is conceptually attractive and has been the subject of several publications in the last years (Kowalsky *et al.* 1994, Collins *et al.* 1996, Giuliano *et al.* 1997). Its practical application is straightforward for single-degree-of-freedom systems; for multi-degree-of-freedom systems, however, some approximations need to be made.

In an attempt to improve the current seismic design procedure leading to a gradual implementation of displacement-based design within the framework of performance-based design (Bertero 1996) in future seismic codes, a two-level displacement-based design procedure is developed. The earthquake design levels correspond to occasional earthquakes and rare earthquakes with mean recurrence intervals of 72 and 475 years respectively (Vision 2000 1995). Occasional earthquakes are described in terms of elastic displacement spectra, while rare earthquakes in terms of inelastic displacement spectra. For both earthquake levels, drift limits are established. Elastic response is assured for occasional earthquakes. The designer checks or determines global structure characteristics: (i) stiffness for drift control at both design levels, and (ii) minimum necessary strength for the structure to respond within the elastic range under occasional earthquakes (Collins *et al.* 1996).

As the result of this procedure the demanded strength (base shear) is obtained. If it is used as design strength, the actual strength will be greater. Several sources contribute to the overstrength of the structure, namely: other load combinations not including seismic actions, minimum reinforcement ratios, availability of reinforcement diameters, actual resistance of the materials greater than the nominal ones.

The overstrength reduction factor is defined as the ratio of the actual and the design strength. With the objective to equal the actual and the demanded strengths, the design strength results a ratio between the demanded strength and the overstrength reduction factor (Giuliano *et al.* 1996).

The proposed design procedure is relatively simple and easy to use. It was planned as an alternative to the traditional lateral force procedure. It is recommended, however, to perform a push-over analysis up to the maximum required displacement corresponding to the rare earthquake level, computing local and global damage indices (Fajfar and Gaspersic 1996, Park and Ang 1985, Williams and Sexsmith 1995).

The procedure is applicable to planar structures. It is assumed that they respond predominantly in the fundamental mode in the elastic range and following certain inelastic displacement shapes according to the structure typology in the inelastic range (Fajfar and Gaspersic 1996).

An example of the application of the proposed procedure to coupled structural walls, including the push-over analysis and the evaluation of damage indices is presented. The non-linear dynamic response of the structure subjected to occasional and rare earthquakes is also included.

Finally, the results are discussed and further refinements of the procedure are proposed.

2. List and definition of main symbols

C : Transformation factor from properties of MDOF to the corresponding properties of SDOF system.
D_S : Maximum elastic displacement demand at the structure top level.
D_u : Maximum inelastic displacement demand at the structure top level.
D_y : Yield displacement of the bilinear structure response.
D_e' : Elastic displacement at the structure top level resulting from $\{V_e'\}$.
H : Total height of the structure.
K_G : Global stiffness of the bilinear structure response.
L_1 : The first modal earthquake-excitation factor.
M_1 : The first normal-coordinate generalized mass.
M_{e1} : The first mode effective mass.
T : Period of the structure.
V : Design base shear.
V_0 : Base shear.
V_y : Demanded base shear.
V_0' : Arbitrary base shear.
h_i : Story height at level i.
Δ_S : Maximum drift demand for occasional earthquakes.
Δ_u : Maximum drift demand for rare earthquakes.
Δ_S^c : Drift limit, for occasional earthquakes established in the Code.
Δ_u^c : Drift limit, for rare earthquake established in the Code.
ξ : Damping ratio, assumed.
μ^c : Displacement global ductility factor, established in the Code.
$[K]$: Stiffness matrix of the structure.
$[M]$: Diagonal mass matrix of the structure (masses lumped at each level).
$\{V\}$: Design lateral force vector at ultimate limit state.
$\{V_e\}$: Lateral force vector from V_0.
$\{V_y\}$: Lateral force vector from V_y.
$\{V_e'\}$: Lateral force vector from V_0'.
$\{\psi_1\}$: Shape vector of the first vibration mode or, in the step 3.4.2, the displacement shape vector of the plastic mechanism (Fajfar and Gaspersic 1996).
$\{1\}$: The displacements vector for a rigid body translation of unity value.

3. Design procedure

The design procedure to be described is based on the control of deformations through drift limits specified for occasional and rare design earthquakes and the condition that the structure responds within the elastic range for the former.

Global structure characteristics (stiffness and strength) are verified or derived in terms of top level displacement vs. base shear, assuming a bilinear approximation for the structure response. Finally the equivalent lateral forces and the corresponding internal forces for member design, are obtained.

3.1 Input data

· Δ_u^c , Δ_S^c , μ^c , ξ .
· Preliminary design: global geometry, cross sections, material properties.
· Specified actions: gravity load per story,
inelastic displacement spectrum for rare earthquakes (IDSRE),
elastic displacement spectrum for occasional earthquakes (EDSOE).

3.2 Acceptance criteria

· Drift limits:

$$\Delta_u \leq \Delta_u^c \tag{1}$$

$$\Delta_S \leq \Delta_S^c \tag{2}$$

· Structure response within the elastic range for occasional earthquakes:

$$D_S \leq D_y \tag{3}$$

As it will be seen in 3.4, the first two conditions establish a lower bound for the stiffness while the last one defines the minimum strength to be supplied.

3.3 Additional remarks

· As response spectra represent the response of single-degree-of-freedom systems, each structure is characterized by its corresponding equivalent single-degree-of-freedom system.
· Superscript (1) denotes properties of the equivalent single-degree-of-freedom system. These properties are obtained by the transformation

$$P^{(1)} = CP \tag{4}$$

where P represents the quantity in the MDOF system and $P^{(1)}$ the corresponding quantity in the SDOF system. The transformation factor C is:

$$C = M_1/L_1 \tag{5}$$

where:

$$M_1 = \{\psi_1\}^T[M]\{\psi_1\} \tag{6}$$

$$L_1 = \{\psi_1\}^T[M]\{1\} \tag{7}$$

$\{\psi_1\}$ is the shape vector of the first mode of vibration, or in the step 3.4.2 the shape vector of the displacements of the plastic mechanism (Fajfar and Gaspersic 1996). {1} is the displacement vector for a rigid body translation of unity value.

This transformation assumes the same vibration period for the MDOF and the equivalent SDOF system. Then the global stiffness (ratio between the base shear and the top level displacement) of MDOF system is equal to the stiffness of the equivalent SDOF system.

· According to the equivalent lateral force procedure fundamentals (Clough and Penzien 1975), the effective mass of the first mode of vibration is:

$$M_{e1} = L_1^2/M_1 \tag{8}$$

The lateral force vector results:

$$\{V_e\} = ([M]\{\psi_1\}/L_1)V_0 \tag{9}$$

where V_0 is the base shear. Assuming a linear variation for the first mode shape

$$\psi_{1i} = h_i/H \tag{10}$$

where h_i is the story height and H the total height of the structure.

· Adopting "inelastic displacement shapes" for the inelastic response of planar structures (Fajfar and Gaspersic 1996), it is possible to relate the maximum displacement at the top D_u with the maximum story drift Δ_u. For frames:

$$D_u = \frac{\Delta_u H}{2\left(1 - \frac{1}{2\mu^c}\right)} \tag{11}$$

For walls and wall-frame structures:

$$D_u = \Delta_u H \tag{12}$$

In a general form:

$$D_u = D_u(\Delta_u) \quad \text{or} \quad \Delta_u = \Delta_u(D_u) \tag{13}$$

3.4 Design steps

A preliminary design is available. If not see note in the setp 3.4.2.

3.4.1 Derive the global stiffness and the period

$$K_G = V_0'/D_e' \tag{14}$$

where V_0' is an arbitrary base shear, and D_e' is the elastic displacement at the top level resulting from the lateral force vector (9)

$$\{V_e'\} = ([M]\{\psi_1\}/L_1)V_0' \tag{15}$$

To compute the elastic displacement, allowance should be made for reduced member stiffnesses due to cracking (Paulay and Priestley 1992).

The mass of the equivalent single-degree-of-freedom system, from (4), (5) and (8)

$$M^{(1)} = CL_1^2/M_1 = L_1 \tag{16}$$

Then, the period results:

$$T = 2\pi(M^{(1)}/K_G)^{1/2} \tag{17}$$

3.4.2 Check condition (1)

With T, μ^c, ξ obtain the inelastic displacement demand $D_u^{(1)}$ from IDSRE. Then,

$$D_u = \frac{1}{C} D_u^{(1)} \tag{18}$$

Finally Δ_u is calculated by (13).

If (1) is satisfied, go to step 3.4.3 to control performance under occasional earthquakes. If not, the stiffness should be increased. To this aim with (13), (18) and IDSRE inversely used, determine the period for which $\Delta_u = \Delta_u^c$.

Note: if a preliminary is not available the procedure starts determining the necessary period for which $\Delta_u = \Delta_u^c$.

3.4.3 Check condition (2)

With T, ξ obtain the elastic displacement demand $D_S^{(1)}$ from EDSOE. Then,

$$D_S = \frac{1}{C} D_S^{(1)} \tag{19}$$

Assuming a linear variation of displacements with height, that is classical in equivalent lateral force procedure, for elastic structure response under occasional earthquakes, results:

$$\Delta_S = D_S/H \tag{20}$$

If (2) is satisfied, go to step 3.4.4.

If not, stiffness should be increased. To this aim, with (20), (19) and EDSOE inversely used, determine the period for which $\Delta_S = \Delta_S^c$.

With the new value of the period and IDSRE recalculate D_u (18).

3.4.4 Check condition (3) and compute the demanded strength

Compute:

$$D_y = D_u/\mu^c \tag{21}$$

Then check (3). If this condition is not satisfied, define:

$$D_y = D_S \tag{22}$$

It means an increase of the strength.

The strength, as base shear, is

$$V_y = K_G D_y \tag{23}$$

where K_G was evaluated with (14) or, if the period was modified in steps 3.4.2 and/or 3.4.3, it must be computed using (17).

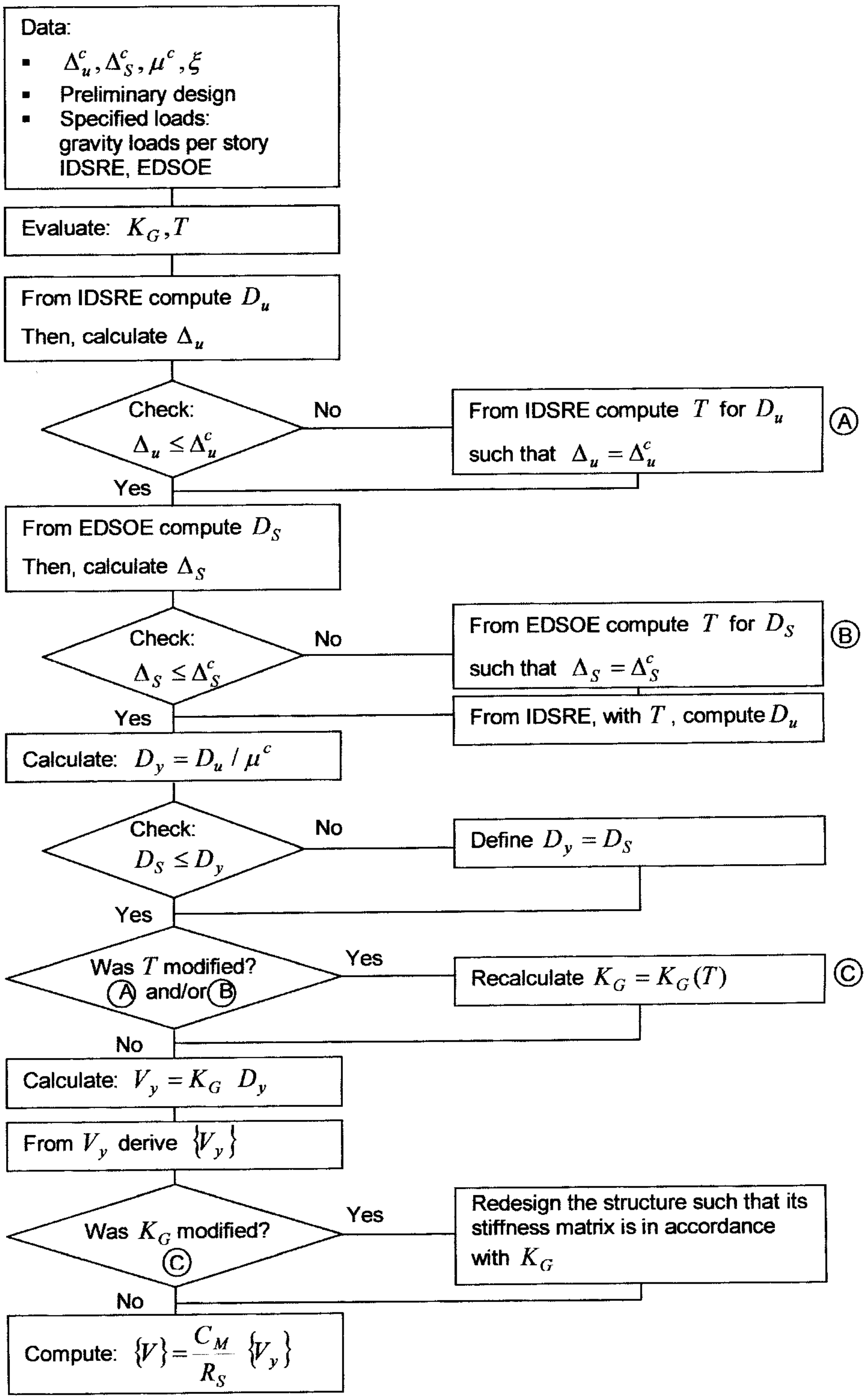

Fig. 1 Flow chart of the proposed design method

3.4.5 Compute $\{V_y\}$ and, if it is necessary, a new stiffness matrix

From V_y the lateral force vector is:

$$\{V_y\} = ([M]\{\psi_1\}/L_1)V_y \tag{24}$$

If the global stiffness (14) was modified, it will be necessary to find a stiffness matrix $[K]$ in order to obtain D_y for the vector $\{V_y\}$.

To update $[K]$ it is proposed:

a) Modify the element section dimensions in order that their flexural stiffness, dominant in the deformations, will be proportional to its former value and the ratio between the last and the initial global stiffness K_G.

b) If the procedure a) produce section dimensions not convenient, it will be necessary, by trial and error, to modify the structural project until get the $[K]$ required.

3.4.6 Compute the design lateral force vector at ultimate limit state

$$\{V\} = \frac{C_M}{R_S}\{V_y\} \tag{25}$$

where
$$C_M = (C\{1\}^T[M]\{1\})/L_1 \tag{26}$$

With this factor the total mass of the structure is considered, instead of the effective mass of the first mode, to be in accordance with the fundamentals of the equivalent lateral force procedure.

R_S is a reduction factor to make allowance of the overstrength of the structure.

Fig. 1 shows a flow chart of the proposed procedure.

4. Nonlinear analysis

The structure designed according to the procedure described above, is subjected to nonlinear static and dynamic analysis in order to evaluate its performance:

a) verify that the assumed design parameters are within acceptable limits.

b) evaluate local and global damage indices for the rare earthquake.

4.1 Analytical modeling for nonlinear analysis

The finite element method formulated in displacements is used for computing the nonlinear static and dynamic response. Step-by-step integration of the equations of motion by the Newmark's method is used to solve the dynamic problem. In each time step, the nonlinear problem is solved by iteration with a variant of the Newton Raphson's scheme (Filippou *et al.* 1992).

The structure is represented as an assembly of finite bar elements. Each element is composed by subelements, as shown in Fig. 2, allowing the representation of the different mechanisms that contribute to the hysteretic behavior of the reinforced concrete members (Möller *et al.* 1997, 1998).

· Elastic plastic subelement: this takes into account the elastic behavior of the bar and the nonlinear behavior at its extremes, with gradual spread of the inelastic deformations due to bending, as a function of the load history.

· Joint subelement: this incorporates the fixed end rotation which occurs at the interface bar-joint due to loss of bond and slip of the reinforcement at its anchorage.

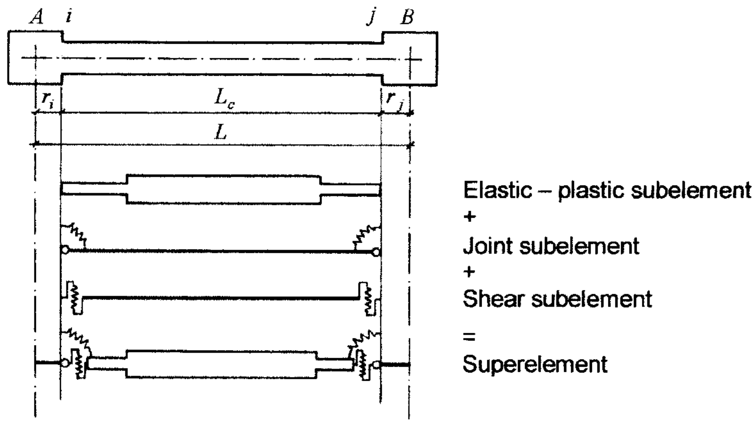

Fig. 2 Components of the structural calculation model

· Shear subelement: this takes into account the shear deformation at critical regions. It is not used in this work.

Rigid extremes are incorporated at the ends of the bar element to take into account the actual dimensions of the nodes.

Parameters of the moment-curvature relationship are obtained from a previous detailed analysis of the cross section with realistic constitutive laws for the materials. Similarly for the spring parameters of the moment-rotation relationship at the member-joint interface.

The ultimate curvature of a section and the ultimate rotation of the connection are defined as that when at the end of a predetermined number of cycles, a limit state is reached (Zahn *et al.* 1986). Values corresponding to one cycle (monotonic) and four cycles representing the dynamic response during an earthquake, were obtained.

4.2 Nonlinear static (push-over) analysis

The nonlinear static analysis is performed as follows:

a) Discretization of the structure, derivation of the moment-curvature relationship at member's end sections, and of the moment-rotation relationship at the member-joint interfaces.

b) Adoption of a displacement shape $\{\psi_1\}$ as a function of the structure typology and of the assumed collapse mechanism (Fajfar and Gaspersic 1996).

c) Derivation of the vertical distribution of lateral forces $\{f\} = [M]\{\psi_1\}/L_1$. This vector has a unit base shear.

d) Perform the nonlinear static analysis described at 4.1, applying the vertical loads and then the vector of lateral forces $\{H\} = \{f\}V$, V being a variable base shear. The maximum displacement at the top, D, is obtained.

e) The *V-D* relationship progresses up to the maximum available displacement $D_{u\ \mathrm{dis.}}$, when in any member's end section the ultimate curvature is reached or when in any connection (member-joint interface) the ultimate rotation is reached.

According to the two criteria to define the ultimate curvature/rotation, two values for $D_{u\ \mathrm{dis.}}$ are obtained. As an alternative, a nonlinear static analysis with four complete cycles up to a maximum displacement at the top, $D_{u\ \mathrm{dis.}}$, defined as when at the end of the process the ultimate curvature of a section or ultimate rotation of a connection is reached, is proposed. In this case the ultimate curvature/rotation is defined for one cycle.

4.2.1 Application

As a result of the application of the nonlinear static analysis, the top displacement-base shear relationship is obtained and a bilinear approximation is used.

For the demanded displacements for both earthquake levels (occasional and rare), the displacement patterns $\{\psi_1\}$ are obtained.

The results of the push-over analysis are used to check the validity of the assumptions made in the proposed procedure and, eventually, redesign the structure if necessary.

4.2.2 Seismic demand and damage index

To evaluate the structural damage, the Park & Ang model is used (Park and Ang 1985). This model combines two effects contributing to damage: the amplitude of the inelastic excursion and the dissipated hysteretic energy.

$$DI = \frac{\theta}{\theta_u} + \beta \frac{E_H}{M_y \theta_u} \tag{27}$$

where θ, θ_u are demand and capacity (ultimate) values of curvature for the member's end section or rotation for the connection between member and joint, E_H the dissipated hysteretic energy, M_y the yield moment of a bilinear approximation of the moment-curvature/rotation of the section/connection, and β a parameter depending on the section detailing. θ and E_H are demands, β, M_y and θ_u are capacities.

In order to use the results of the push-over analysis, the following assumptions need to be made:

a) The curvatures or rotations demanded by the earthquake are equal to those obtained from the push-over analysis up to the ultimate displacement D_u.

b) The energy demanded E_H is included implicitly in a γ parameter, resulting the damage index (Fajfar and Gaspersic 1996).

$$DI = \frac{\theta}{\theta_u}\left[1 + \frac{\beta\gamma^2\mu^2}{\mu - 1}\ \frac{\theta - \theta_y}{\theta}\right] \tag{28}$$

$\mu = D_u / D_y$ being the ductility reached. Parametric studies (Fajfar and Vidis 1994) show that γ depends mainly on the characteristics of the earthquake and that some structural parameters (period, ductility, hysteretic behavior) have also some influence. An approximate formula is:

$$\gamma = Z_T Z_\mu Z_g \tag{29}$$

for a stiffness degrading model:

$$\begin{aligned} Z_T &= 1.05 - 0.30\, T/T_C && T \le T_C \\ Z_T &= 0.75 - 0.25 \frac{T - T_C}{T_D - T_C} && T_C \le T \le T_D \\ Z_T &= 0.50 && T \ge T_D \end{aligned} \tag{30}$$

where: T is the period of the structure, T_C and T_D are characteristic soil periods

$$Z_\mu = \frac{(\mu - 1)^{0.58}}{\mu} \tag{31}$$

$$Z_g = \left[\frac{\int a^2 dt}{a_g v_g}\right]^{0.40} \tag{32}$$

$\int a^2\, dt$ being the integral of the square of the ground acceleration, and a_g, v_g, the maximum acceleration and velocity of the ground.

Z_g may be computed as :

$$Z_g = 0.6\left(\frac{a_g}{v_g t_D}\right)^{0.3} \tag{33}$$

t_D being the strong motion duration according to Trifunac & Brady (Fajfar and Vidis 1994).

The damage index (28) is evaluated in each member end section and member-joint connection, to obtain then a global weighted damage index for the structure.

$$DI_G = \sum w_i\, DI_i, \qquad w_i = \frac{DI_i}{\Sigma DI_i} \tag{34}$$

The Park & Ang damage index has been calibrated (Williams and Sexsmith 1995) with observed seismic damage.

$DI < 0.10$	No damage or localized minor cracking.
$0.10 \le DI < 0.25$	Minor damage, light cracking throughout.
$0.25 \le DI < 0.40$	Moderate damage, severe cracking, localized spalling.
$0.40 \le DI < 0.80$	Severe damage, crushing of concrete, reinforcement exposed.
$DI \ge 0.80$	Collapse.

4.3 Nonlinear dynamic analysis

For important structures it may be convenient to perform nonlinear dynamic analysis with real or artificially generated earthquakes records compatible with the seismicity of the site. This allows a more realistic assessment of the response of the structure.

5. Inelastic displacement spectra

5.1 Characterization of the seismic action

The ground motion is represented by an stochastic process, white noise filtered type, with a power spectral density function given by (Clough and Penzien 1975):

$$S_{XX}(f) = S_0 \frac{1 + 4\xi_g^2 (f/f_g)^2}{[1-(f/f_g)^2]^2 + 4\xi_g^2 (f/f_g)^2} \frac{(f/f_f)^4}{[1-(f/f_f)^2]^2 + 4\xi_f^2 (f/f_f)^2} \tag{35}$$

S_0 being the power spectral density function of the white noise, f_g, ξ_g are the characteristic ground motion frequency and its damping ratio, f_f, ξ_f are the high filter parameters that attenuates the very low frequency components.

From (35) artificial accelerograms are generated with:

$$x(t) = I(t)\sum_{i=1}^{n} [4S_{XX}(i\Delta f)\Delta f]^{1/2}\sin(2\pi i\Delta ft + \theta_i) \tag{36}$$

n being the number of frequencies $f_i = i\Delta f$ equally spaced in the interest domain, θ_i are the phase stochastic angles with uniform distribution within 0 and 2π.

The process is not stationary in amplitudes through the modulation function $I(t)$ (Barbat *et al.* 1994).

A base line correction is applied to function (36) to minimize the velocity mean square value, and the same function is scaled to adjust the maximum acceleration.

5.2 Strength and displacement spectra

The single-degree-of-freedom system used to obtain strength and displacement spectra has the following characteristics, see Fig. 3: bilinear force-displacement relationship with yield strength Fy, stiffness K, strain hardening slope $K_h = 0.01K$ and Clough's model for the hysteresis loop.

The oscillator has mass m, damping ratio $\xi = 0.05$, yield displacement $v_y = F_y/K$; and displacement ductility ratio $\mu = v/v_y$. Thus, the natural circular frequency is $\omega = \sqrt{K/m}$ and the period $T = 2\pi/\omega$.

For the elastic range ($\mu \le 1$), the normalized elastic strength, related to the weight of the oscillator:

$$C_e = \frac{F}{W} = \frac{Kv}{mg} = \frac{\omega^2 v}{g} = \frac{S_a}{g} \tag{37}$$

which is the elastic pseudo-acceleration response spectrum expressed as fraction of g.

For the elasto-plastic range ($\mu > 1$):

$$C_y = \frac{F_y}{W} = \frac{Kv_y}{mg} = \frac{\omega^2 v_y}{g} \tag{38}$$

which is the yield strength response spectrum. For design purposes, the inelastic strength design spectrum is obtained in current seismic codes, reducing the elastic design spectrum by a strength reduction factor.

The displacement spectrum is obtained from (38):

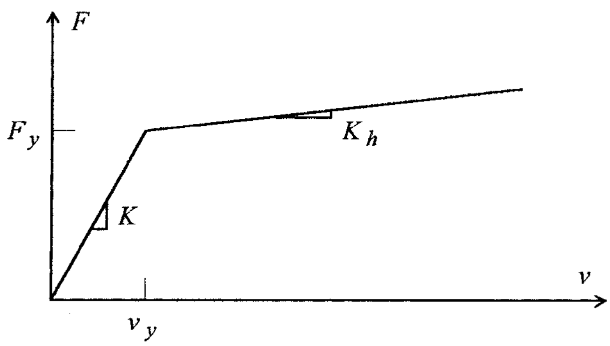

Fig. 3 Characteristic of the single degree of freedom

$$v = \mu v_y = \mu \frac{C_y g}{\omega^2} = \frac{\mu C_y g T^2}{4\pi^2} \tag{39}$$

Spectra are computed for ductilities μ = 1, 2, 4 and 6

5.3 Spectra for Mendoza city

According to Vision 2000 (1995), two earthquake design levels are considered:

a) Occasional earthquakes, with mean recurrence interval $T = 72$ years, corresponding to a probability of exceedance of 50% in 50 years.

b) Rare earthquakes, with mean recurrence interval $T = 475$ years, corresponding to a probability of exceedance of 10% in 50 years.

From "Microzonificación sísmica del Gran Mendoza" (INPRES 1995), the following mean peak ground acceleration values are obtained: 0.2 g for $T = 72$ years and 0.6 g for $T = 475$ years.

The parameters for the power spectra density function (35), for a soil type II, are: $f_g = 2$ Hz, $\xi_g = 0.50$, $f_f = 0.40$, $\xi_f = 0.60$, S_0 is adjusted to obtain the corresponding values of peak ground acceleration.

For occasional earthquakes a total duration of 12 sec., with a strong motion part of 5 sec., is considered. For rare earthquakes, the corresponding figure are 30 sec. and 20 sec. respectively.

Two records, for each design level, were generated, with different phase angle sequences, (see Eq.

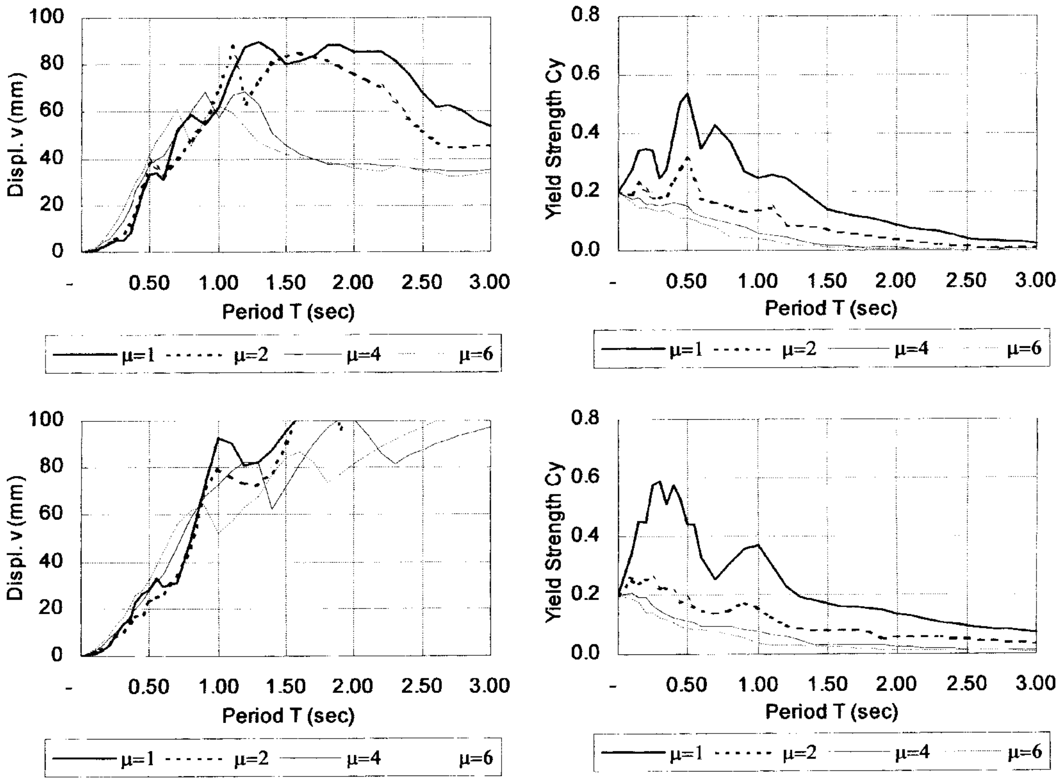

Fig. 4 Displacement and yielding spectra for occasional earthquake

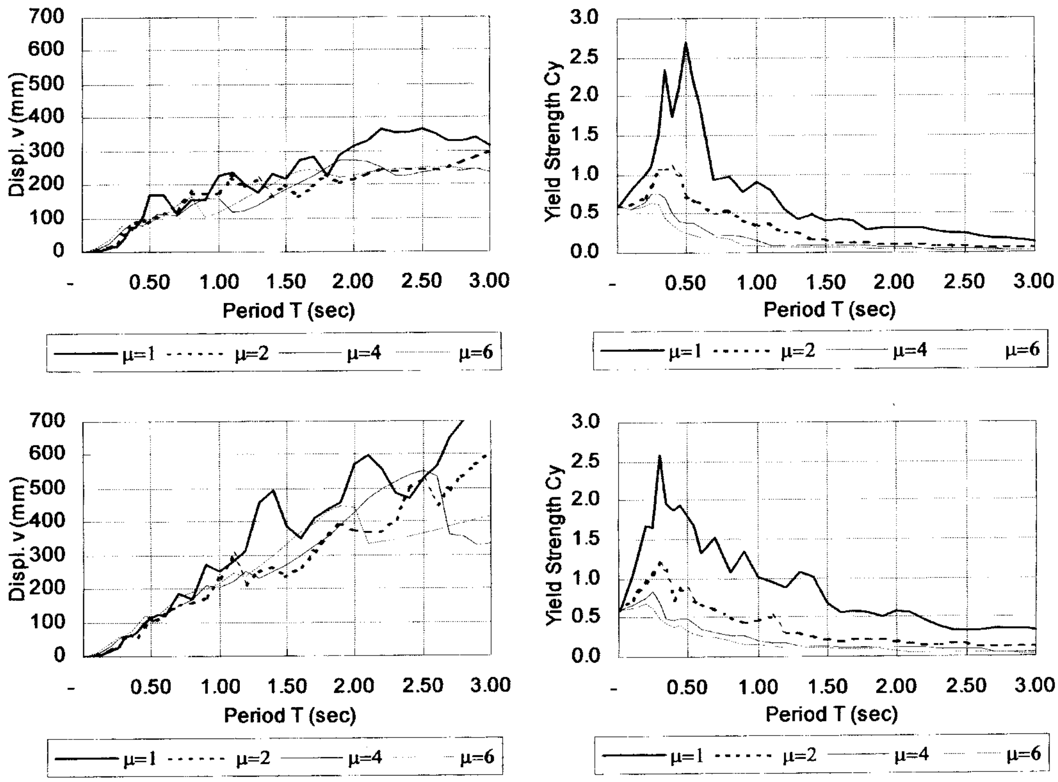

Fig. 5 Displacement and yielding spectra for rare earthquake

36). For each of them, displacement and normalized strength spectra were computed. These spectra are shown in Figs. 4 and 5.

6. Design example of a five-story coupled wall system

6.1 Design procedure

The proposed procedure is applied to the design of a five-story coupled wall system with slender beams, as shown in Fig. 6.

According to the steps described in section 3, the following results are obtained.

1. Derive the global stiffness and the period.

From data of Fig. 6

$$[M] = \begin{bmatrix} 200 & & & & \\ & 200 & & & \\ & & 200 & & \\ & & & 200 & \\ & & & & 240 \end{bmatrix} \frac{KN}{g}$$

From (10) $\{\psi_1\}^T = \{0.250; 0.4375; 0.625; 0.8125; 1.00\}$

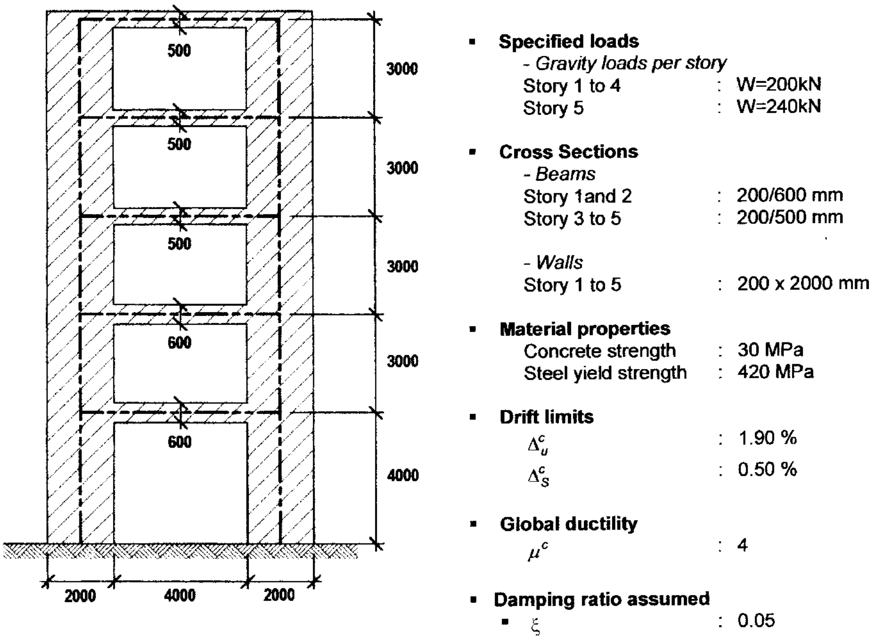

Fig. 6 Five story coupled wall example structure

From (7) $L_1 = \{\psi_1\}^T[M]\{1\} = \dfrac{665.0}{g}$ KN

From (15) and an arbitrary value of $V_0' = 1$ KN, results

$$\{V_e'\} = ([M]\{\psi_1\}/L_1) \times V_0' = \{0.756;\ 1.306;\ 1.890;\ 2.440;\ 3.601\}^T \times 1 \text{ KN}$$

With this unit lateral force vector the displacement at the top level results $D_e' = 0.961$ mm.

This result was obtained assuming reduced stiffness for the members to allow for the effects of cracking: beams $EI = 0.3(EI)_0$ and walls $EI = 0.4(EI)_0$, (Paulay and Priestley 1992), where $(EI)_0$ denote initial stiffness. Then

$$K_G = \frac{V_0'}{D_e'} = 10.40 \text{ KN/mm}$$

From (6) $M_1 = \{\psi_1\}^T[M]\{\psi_1\} = \dfrac{500.94}{g}$ KN

From (5) $C = \dfrac{M_1}{L_1} = 0.7533$

From (16) $M^{(1)} = \dfrac{CL_1^2}{M_1} = \dfrac{665}{g}$ KN

Finally, from (17) $T = 2\pi\sqrt{\dfrac{M^{(1)}}{K_G}} = 0.507$ sec.

2. Check (1)

For $\mu^c = 4$, $\xi = 0.05$

With the period *T*, averaging the two spectra for rare earthquakes

$$D_u^{(1)} = \frac{1}{2}(91.4 + 119.1) = 105 \text{ mm}$$

$$D_u = \frac{1}{C} D_u^{(1)} = 139.4 \text{ mm}$$

From (12) $\Delta_u = \dfrac{D_u}{H} = \dfrac{139.4}{16000} = 0.0087$

The story drift limit is $\Delta_u^c = 0.019$
Then $\Delta_u \le \Delta_u^c \qquad (0.0087 \le 0.019)$

3. Check (2)
With the period *T*, elastic response $\mu = 1$, $\xi = 0.05$ and averaging the two spectra for occasional earthquakes

$$D_S^{(1)} = \frac{1}{2}(27.5 + 33.1) = 30.3 \text{ mm}$$

$$D_S = \frac{1}{C} D_S^{(1)} = 40.2 \text{ mm}$$

$$\Delta_S = \frac{D_S}{H} = \frac{40.2}{16000} = 0.0025$$

The story drift limit is $\Delta_S^c = 0.005$
Then $\Delta_S \le \Delta_S^c \qquad (0.0025 \le 0.005)$

4. Check the condition (3) and compute the demanded strength

$$D_y = \frac{D_u}{\mu^c} = \frac{139.4}{4} = 34.8 \text{ mm}$$

Then as $D_S \le D_y$ is not satisfied, the strength shall be increased.
From (22) $D_y = D_S = 40.2$ mm
Finally from (23) $V_y = K_G D_y = 420$ KN

5. Compute $\{V_y\}$
From (24) $\{V_y\} = ([M]\{\psi_1\}/L_1)V_y = \{31.75, 54.85, 79.38, 102.48, 151.54\}^T$

6. Compute the design lateral force vector at ultimate limit state

From (26) $C_M = (C\{1\}^T[M]\{1\})/L_1 = \dfrac{0.7533 \times 1040}{665} = 1.178$

Strength reduction factor to allow for the overstrength of the structure is adopted (Giuliano *et al.* 1996) as: $R_S = 1.70$.

Finally from (25) $\{V\} = \dfrac{C_M}{R_S}\{V_y\} = 0.693\{V_y\} = \{22; 38; 55; 71; 105\}^T$

Then, the design base shear results $V = 291$ KN.
With these lateral forces and the load combinations specified (INPRES-CIRSOC 103 1991),

Table 1 Reinforcement of structural members

Member	Section	Dim (mm.)	Longitudinal Reinforcement			Transverse Reinforcement	
			Top	Bottom	Lateral	Perim.	Supplem.
Beams Story 1,2	Support	200/600	4ϕ16+1ϕ12	3ϕ16	1ϕ10	ϕ 8@115	–
	Span	200/600	2ϕ16	3ϕ16	1ϕ10	ϕ 8@150	–
Beams Story 3,4,5	Support	200/500	4ϕ16	2ϕ16+1ϕ12	1ϕ10	ϕ 8@115	–
	Span	200/500	2ϕ16	2ϕ16+1ϕ12	1ϕ10	ϕ 8@125	–
Wall Story 1	Any	200/2000	12ϕ12	12ϕ12	ϕ 8@200	ϕ 8@200	ϕ 6@120
Wall Story 2	Any	200/2000	12ϕ12	12ϕ12	ϕ 8@200	ϕ 8@200	–
Wall Story 3,4,5	Any	200/2000	4ϕ12	4ϕ12	ϕ 8@200	ϕ 8@200	–

Note: For walls, top and bottom mean side longitudinal reinforcement

section design is performed.

Results are summarized in Table 1.

6.2 Nonlinear static (push-over) analysis

Nonlinear static analysis (push-over) to the designed structure is performed, as described in 4.2.

Base shear-top displacement relationship is shown in Fig. 7. Bilinear approximation of the actual response, assumed design values, occasional and rare earthquake demands and ultimate displacement obtained by the different criteria already mentioned, are also shown in the figure.

Available ductilities, for the different criteria, are summarized in Table 2.

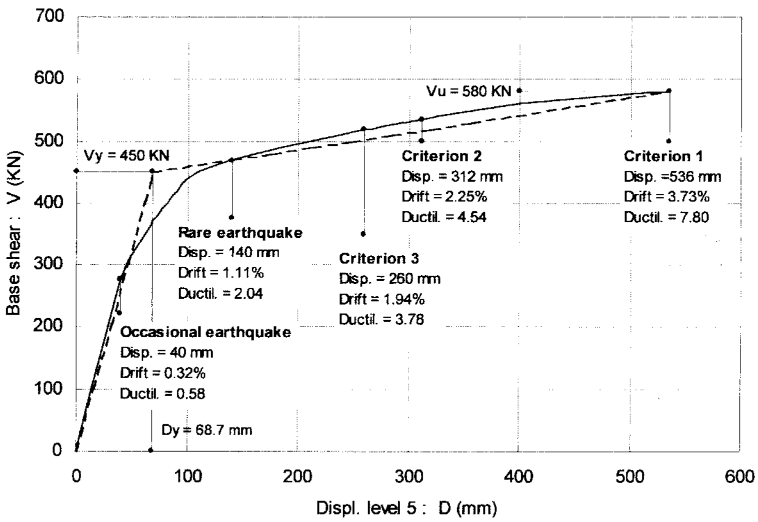

Fig. 7 Results of the push-over analysis

Table 2 Available ductilities

Push-over	Ultimate curvature/rotation	$D_{u\ \mathrm{dis}}$ (mm)	D_y (mm)	μ_{dis}
Monotonic	1 cycle	536.0	68.7	7.80
Monotonic	4 cycles	312.0	68.7	4.54
4 cycles	1 cycle	260.0	68.7	3.78

Table 3 Structural parameters

Parameter	Design procedure	Pushover
Demanded base shear V_y (KN)	495	450
Yield displacement D_y (mm)	47.6	68.7
Stiffness K_G (KN/mm)	10.40	6.55
Period T (sec.)	0.507	0.639
Ductility capacity μ	4.00	4.16
Overstrength V_y/V	1.70	450/291=1.55

Criteria 2 and 3 make allowance of the cyclic effect in section failure. They are more realistic than 1, and the average available ductility is $\mu_{\mathrm{dis}} = 4.16$.

Table 3 compares parameters adopted in design with those obtained from the push-over analysis.

The most significant deviation corresponds to the stiffness, the lower value being obtained from push-over. Allowance for the effect of member-joint connection flexibility and a reduced section stiffness, *EI*, obtained by the previous processing of the moment-curvature relationship, seems to be the reasons.

The overstrength reduction factor, V_y/V, was estimated 1.70 and a slight less value, 1.55, was obtained.

The overstrength is depicted in Fig. 7. For $V = 300$ KN, very close to the design base shear ($V = 291$ KN), the first plastic hinges appear at beam ends, causing the first slope change in *V-D curve.* Hinges at the base of the walls appears at $V = 450$ KN where the second slope change occurs.

Another source of overstrength is the increment in steel strength due to strain hardening, causing the increase in the slope of the last branch of the base shear-top displacement curve. If this effect is taken into account, the overstrength reaches $V_u/V = 525/291 = 1.80$.

In summary, the value adopted in design, $R_S = 1.70$, seems to be quite reasonable.

Fig. 8 shows seismic demands for occasional and rare earthquakes. For both earthquake levels, the displacement pattern aparts from the linear variation assumed in design, particularly at lower levels of the structure. A refinement of $\{\psi_1\}$ may be considered if appropriate. Although the story drifts are larger than those resulting from the proposed procedure, they are still below the design limits.

A few sections are hinged for occasional earthquake, with reduced curvature ductility demands. According to Vision 2000 (1995), operational limit state objectives is fulfilled.

For rare earthquakes, the hinge pattern shows a beam-sway mechanism, with moderate curvature ductility demands, similar for every beam end. More reduced values are observed at the base section of the walls. According to Vision 2000 (1995), the limit state termed life safety is fulfilled.

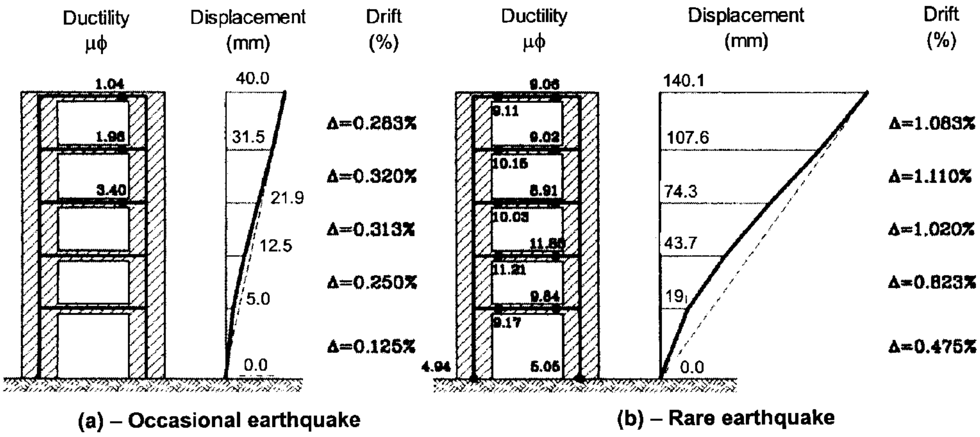

Fig. 8 Earthquake demands

Due to differences in stiffness, explained above, the global ductility demand, $\mu = 2.04$, is less than that assumed in design.

With the push-over results, it is possible to evaluate the damage index as explained in 4.2. First, the parameter γ, for $\mu = 2.04$, $a_g = 0.6$ g, $v_g = 790$ mm/s and $t_D = 20$ sec., is derived:

$$\gamma = Z_T Z_\mu Z_g = 0.51 \times 2.693 \times 0.501 = 0.689$$

Finally the maximum local damage index $DI = 0.59$, and the global damage index $DI_G = 0.44$ are evaluated. The last damage index corresponds to severe damage, in agreement with the life safety limit state (Vision 2000 1995).

6.3 Nonlinear dynamic analysis

The same earthquake records from which displacement spectra used in the design procedure were derived, are used as input motion.

Table 4 summarizes main results and compares them with those obtained from the push-over analysis.

The displacement D obtained from push-over, matches the proposed design procedure demands.

The good agreement between both results (push-over and dynamic analysis) confirms the push-

Table 4 Results of nonlinear dynamic analysis

Parameter	Occasional earthquake				Rare earthquake			
	Push-Over	Dynamic analysis			Push-Over	Dynamic analysis		
		Rec. 1	Rec. 2	Aver.		Rec.1	Rec. 2	Aver.
Level 5 displ.: D (mm.)	40.2	40.6	37.7	39.1	139.4	148.0	136.0	142.0
Base shear: V (KN)	275	303	247	275	470	473	559	516
Max story drift: Δ	0.0032	0.0033	0.0031	0.0032	0.0111	0.0118	0.0115	0.0116
Max.local dam.index: DI	–	0.115	0.090	0.103	0.590	0.520	0.600	0.560
Global dam. index: DI_G	–	0.060	0.035	0.048	0.440	0.330	0.450	0.390

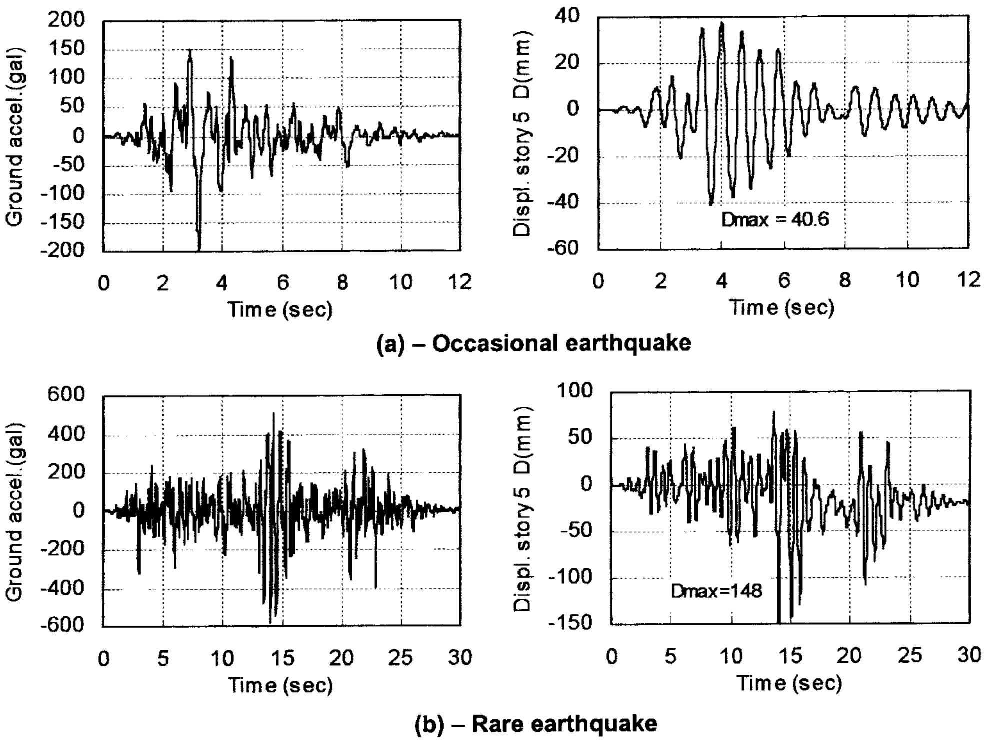

Fig. 9 Nonlinear dynamic response

over as a powerful tool to evaluate the response of structures, when the influence of higher modes is negligible.

Fig. 9 shows two accelerograms and the corresponding time response, for both earthquake levels (occasional and rare).

Finally, the structure subjected to a destructive earthquake (Kobe 1995) is analyzed. Fig. 10 shows the input ground motion and the structure response.

The following results are obtained:

$D_{max.} = 287$ mm; $V_{max.} = 665$ KN; $\Delta_{max.} = 0.0215$; $DI_{max} = 0.66$; $DI_G = 0.55$. From Fig. 7 (push-over results) for $D = 287$ mm, the base shear is $V = 525$ KN, less than $V_{max.} = 665$ KN. This means that

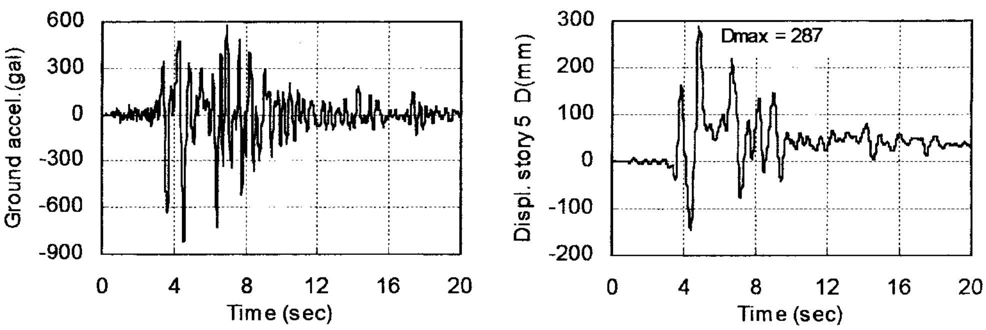

Fig. 10 Response to Hyogo-ken Nanbu (Kobe) earthquake

the distribution of equivalent lateral forces is slightly different to that adopted in the proposed design procedure and in the push-over.

Considering the value of the maximum ground acceleration, the damage index should have been closer to 0.8 (collapse). However, the short duration of the strong motion (approximately 7 seconds), and the excellent mechanism of energy dissipation, make the damage smaller than expected.

7. Conclusions

A two-level displacement-based seismic design procedure for reinforced concrete structures has been presented.

For both earthquake design levels maximum story drifts, to control the stiffness of the structure, are specified together with the condition that the structure responds in the elastic range for the occasional earthquakes, which controls the minimum strength to be supplied.

Accordingly, the designer deals with data related to structural characteristics like stiffness and strength. Thus, the proposed procedure constitutes a step forward in relation to the traditional procedure, being more transparent in the sense that the objectives of the seismic design embodied in current codes, are explicitly considered.

Firstly, the traditional procedure force/strength-based with only one earthquake design level, hides the importance of deformations as measures of structural response, particularly when the response is developed in the inelastic range. Consequently their computations are left to the end of the design process. On the contrary, in the proposed procedure deformations are considered from the beginning of the design process, forces being the final product.

Secondly, the control of the story drift in the traditional procedure, tries to cover the performance of the structure under occasional, moderate earthquakes (serviceability limit state), but being carried out with external forces corresponding to the ultimate limit state, neither succeed controlling the serviceability limit state, nor warning the designer on the relevance this control has.

As in the traditional design procedure, the proposed procedure is limited to planar structures, whose elastic response is governed by the first mode and the inelastic response follows a characteristic shape according to the structure typology.

From the point of view of its practical application, the proposed procedure does not represent a big increase in the design activities, in relation to the current procedure.

To verify the performance of the structure, it is recommended to perform a static nonlinear analysis (push-over) together with the evaluation of damage indices.

In the design example presented, the preliminary design was satisfactory in relation to the stiffness, but it was necessary to increase the strength to allow an elastic response during occasional earthquakes.

The results obtained from the proposed procedure are in good agreement with those obtained from the push-over analysis, with some differences in the global stiffness and in the displacement patterns.

The performance of the structure, evaluated through non-linear analysis procedures (push-over and dynamic) and the computation of damage indices, shows that the structure fulfils the operational and life safety limit states. The proposed procedure may be improved if used in combination with capacity design strategies developed in New Zealand and adopted in some other countries (i.e., Europe, Japan, etc.) to assure that the desired plastic mechanism to develop and to take into account, at least roughly, the influence of the higher vibration modes.

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References

Barbat, A.H., Orosco, L., Hurtado, J.E., and Galindo, M. (1994). "Definición de la acción sísmica", Monografías de Ingeniería Sísmica, CIMNE IS-10.

Bertero, R.D. (1996), "Diseño sismorresistente basado en la performance", *Memorias X Jornadas Argentinas del Hormigón Pretensado y XV Jornadas Argentinas de Ingeniería Estructural*, Tomo I, 252-284.

Clough, R.W. and Penzien, J. (1975), "Dynamics of structures", Mc. Graw Hill Kogakusha, Ltd.

Collins, K.R., Wen, Y.K., and Foutch, D.A. (1996), "Dual - level seismic design: a reliability- based methodology", *Earthq. Eng. & Struct. Dyn.*, **25**, 1433-1467.

Fajfar, P., and Vidis, T. (1994), "Consistent inelastic design spectra: Hysteretic and input energy", *Earthq. Eng. & Struct. Dyn.*, **23**, 523-537.

Fajfar, P., and Gaspersic, P. (1996), "The n2 method for the seismis damage analysis of RC buildings", *Earthq. Eng. and Struct. Dyn.*, **25**, 31-46.

Fajfar, P., and Krawinkler, H., Editors (1997), "Seismic design methodologies for the next generation of codes", *Proc. of the Int. Workshop on Seismic Design Methodologies for the Next Generation of Codes*, Bled/Slovenia, 24-27 June.

Filippou, F.C., D'Ambrisi, A., and Issa, A. (1992), "Non-linear static and dynamic analysis of reinforced concrete subassemblages", Earthquake Engineering Research Center, University of California, EERC 92/08, Berkeley.

Giuliano, A.P., Möller, O., and Rubinstein, M. (1996), "Factores de reducción para pórticos de hormigón armado sismorresistente: ductilidad y sobrerresistencia", *Memorias X Jornadas Argentinas del Hormigón Pretensado y XV Jornadas Argentinas de Ingeniería Estructural, Tomo I*, 285-312.

Giuliano, A.P., Möller, O., and Rubinstein, M. (1997), "Hacia un nuevo reglamento para construcciones sismorresistentes en la Argentina", *Diseño y Construcción Sismorresistente de Estructuras de Hormigón Armado y Acero*, EIPAC 97. Facultad de Ingeniería, Universidad Nacional de Cuyo.

INPRES-CIRSOC 103 (1991), Instituto Nacional de Prevención Sísmica - *Centro de Investigación de los Reglamentos Nacionales de Seguridad para las Obras Civiles.*

INPRES (1995), "Microzonificación Sísmica del Gran Mendoza", *Publicación Técnica Nº 19.*

Kowalsky, M.J., Priestley, M.J.N., and Mac Rae, G.A. (1994), "Displacement based design of RC bridges columns", Seminario Internacional "Últimas Tendencias En El Diseño Sismorresistente", Mendoza Argentina.

Möller, O., Etse, G., and Rubinstein, M. (1997), "Análisis computacional no lineal para pórticos sismorresistentes", *Mecánica computacional*, Vol XVIII; 405-414, AMCA.

Möller, O., Etse, G., Giuliano, A., and Rubinstein, M. (1998), "Computational analysis of non-linear dynamic behavior of reinforced concrete frames", Computational Mechanics: New Trends and Applications (Ed. S.Idelsohn, E.Oñate and E.Dvorkin) Part II, Section 5, Title 9, CIMNE - IACM (CD-ROM).

Park, R., and Paulay, T. (1975), "Reinforced concrete structures", John Wiley & Sons.

Park, Y.J. and Ang, A.H.S. (1985), "Mechanistic seismic damage model for reinforced concrete", *J. Struct. Eng.*, **111**(4), 722-739.

Paulay, T., and Priestley, M.J.N. (1992), "Seismic design of reinforced concrete and masonry buildings", John Wiley & Sons, Inc.

VISION 2000 (1995), "Performance based seismic engineering of buildings", *Structural Engineers Association of California.*

Williams, M.S., and Sexsmith, R.G. (1995), "Seismic damage indices for concrete structures: a state of the art review", *Earthquake Spectra*, **111**(2), 319-349.

Zahn, F.A., Park, R., and Priestley, M.N.J. (1986), "Design of reinforced concrete bridges columns for strenght and ductility", Research Report 86-7, Department of Civil Engineering, University of Canterbury, New Zealand.