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PERFORMANCE-BASED PRELIMINARY SEISMIC ANALYSIS APPROACH OF NEW REINFORCED CONCRETE FRAME STRUCTURES

Alejandro GIULIANO[1], Marcelo MARTÍNEZ[2], Marcelo RUBINSTEIN[3], Oscar MÖLLER[4]

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SUMMARY

A formal theoretical basis is developed applying concepts of performance-based engineering for a preliminary seismic analysis of new reinforced concrete structures. Rather than advancement in complex analytical techniques, the presentation of simple concepts relevant to the routine seismic design is here advocated.

Within the performance-based seismic engineering framework the procedure is defined in terms of a displacement-based approach. To this end, and according to the performance objective previously selected, the displacement demand assessment takes into account two main aspects. Firstly, the deformation demand derived from the control of both the elastic and the post-elastic torsional twists must not exceed the structural displacement capacity actually provided. Secondly, a partial-sway admissible plastic mechanism is proposed at early stage of the design.

The general formulation expressed in the acceleration-displacement spectral format and defined as a function of constant ductility yield point spectra is presented. As a consequence, and following the current state-of-the-art, the system yielding displacement is assumed as the analysis independent variable. Thus, by means of the structure's yield deformation the required strength is suitable computed according to the spectral region governing the design. The strength reduction factor is established in terms of the ductility level derived from the displacement evaluation previously performed. Finally, a general step-by-step algorithm is described, and a numerical example used to illustrate the implementation of the approach is presented.

[1] Director, Instituto Nacional de Prevención Sísmica – Argentina. email: giuliano@inpres.gov.ar
[2] Researcher, Instituto Nacional de Prevención Sísmica – Argentina. email: mmartinez@inpres.gov.ar
[3] Professor, IMAE, Universidad Nacional de Rosario (UNR) – Argentina
[4] Professor, IMAE. UNR – Argentina. email: moller@fceia.unr.edu.ar

INTRODUCTION

As a starting point for contemporary proposals of seismic analysis-design provisions, the *Performance-Based Seismic Engineering* seeks not only for the control of structural damage in minor to moderate ground motions, but also for the reduction of economic losses without compromising the life safety level during severe earthquakes. With this idea in mind, different authors advocate a change in current seismic analysis-design procedures, encouraging simple yet accurate approaches to tailor the structural performance objectives properly selected. In keeping with this philosophy, some shortcomings should be resolved. The lack of a suitable numerical methodology is perhaps the main weakness of contemporary provisions since current one-level-analysis procedures are incapable of accommodating multiple performance outcomes. Besides, a suitable design approach should be defined enabling the designer to evaluate different structural response targets to select the most advantageous design for a given engineering purpose, avoiding currently available time-consuming computer programmes.

On the other hand, current lines of investigations, particularly related to the redefinition of conventional design parameters and the control of torsion in buildings with ductile response, have complemented the aforementioned efforts to elaborate a precise yet simple earthquake-resistance analysis approach to fit several performance objectives. Consistent with these concepts, the description of a comprehensive step-by-step strategy defined in terms of selected performance objectives represents the main goal of the present paper. Although the concepts involved in the approach proposed in this report are not necessarily innovative, a general application framework is described following the current seismic design tendencies.

STEP-BY-STEP PROCEDURE

The methodology to be described is applied to conventional reinforced concrete framed buildings with an in-floor torsionally restrained mechanism taking into account transverse resistant elements remaining elastic and located in at least two planes. Hence, once the building topology and material properties have been defined, the approach requires a five-phase-strategy synthesised in the procedure described below:

Phase I: Evaluation of parameters depending on the building typology and geometry of elements

I.1. *Assess the structural yielding parameters*

I.1.a. *Estimate the yield rotation for each structural element (frame), in both directions of the analysis*

For typical frame structures [5]:

$$\theta_{yi} \cong 0.25 \frac{f_y}{E_S} \frac{L_s}{D_b} \left[1 + 0.45 \frac{I_{b\,gross}}{I_{c\,gross}} \frac{h_{col}}{L_s} \right]$$

I.1.b. *Define the relative strength V_{ri} assigned on each structure's element* [3].

I.1.c. *Estimate the cracked rigidity of the structural elements* [1], [2], [3].

Compute the cracked stiffness close to the element's yielding point, for each structural element in both directions of analysis.

$$k_i = \frac{V_{ri}}{\theta_{yi} \left(N_s \, h_{col} \right)}$$

I.1.d. *Evaluate the yielding drift on the system Centre of Mass* [1], [2], [3].

$$\theta_{y\,CM} = \frac{1}{N_s \, h_{col}} \frac{\sum V_{ri}}{\sum k_i}$$

I.2. *Postulate the admissible-sway mechanism according to the input ground motion level in analysis* [5]

I.2.a. *For a moderate or a severe EQ*: Adopt a kinematics admissible plastic mechanism for the structure, taking into account that the lowest structural strength is normally achieved in the range of

$$0.45\ N_s < n_{sp} < 0.75\ N_s$$

where n_{sp} *number of storeys involved in the admissible post-elastic mechanism*. Generally, for design purposes a structural plastic mechanism with $n_{sp} \geq 0.60\ N_s$ should be preferred to avoid a concentration of high ductility demands in lower storeys.

I.2.b. *For a frequent EQ*: Since the elastic response dominates the serviceability limit state, the admissible sidesway configuration to be adopted corresponding to $n_{sp} = N_s$, simulating the first mode of vibration.

I.3. *Compute the transformation factors according to the admissible mechanism previously selected* [5].

a) Acceleration transformation factor

$$\alpha_A = \frac{m_i}{\sum_{i=1}^{N_s} m_i} \frac{\left[\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} \frac{n_i}{n_{sp}} \right]^2}{\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} \left(\frac{n_i}{n_{sp}} \right)^2}$$

b) Displacement transformation factor

$$\alpha_D = \frac{\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} \frac{n_i}{n_{sp}}}{\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} \left(\frac{n_i}{n_{sp}} \right)^2}$$

c) Equivalent height of the SDOF system

$$h_{eq}^{*} = h_{col} \left[\frac{4N_S^3 - n_{sp}^3}{12N_S - 6n_{sp}} \right]^{0.50}$$

I.4. *Select the performance objectives to be evaluated*

I.4.a. *Structure's Seismic Group*
According to the importance of the new facility within the community, the *Use Seismic Group* should be selected at this stage (**G1**, **G2** or **G3** in Table 2. [6]).

I.4.b. *Input ground motion level*
Assess the seismic severity of the site based on the earthquake design level corresponding to *475 years*, and for a selected period of recurrence, R_p [5].

$$\left(Sa_{el1}\ F_i\right)_{Rp} = 0.131\ \lambda_{CI}\ R_P^{0.33} \left(Sa_{el1}\ F_i\right)_{475\ y}$$

The demand amplification parameter λ_{CI} has been calibrated in terms of the *seismic factor of safety* (*SFS*) to provide the confidence interval selected by the designer, as shown in Table 1.

Table 1

Design safety level adopted **SFS**	Amplification factor to be used λ_{CI}	Confidence Interval obtained **CI**
1.50 *	**1.28**	**77 %**
1.57	**1.30**	**80 %**
1.70	**1.45**	**84 %** **
2.00 ***	**1.70**	**90 %**

** This value corresponds to the seismic security margin adopted by NEHRP*
*** Strictly the median plus one standard deviation.*
**** Proposed by Martínez & Mander (2002)*

I.4.c. *Expected minimum performance objectives*

At this stage, the *minimum performance objectives* governing the design should be defined by using the Table 2. This selection is based on both the *Input Earthquake Level* $(Sa_{el1}\ F_i)_{Rp}$ and the expected *Damage State* sustained by the structure, according to the *Seismic Group* previously selected. Thus, the building performance expectations define the maximum extent of damage *DS* and the limiting plastic rotation θ_p^{DS} governing the analysis [5].

Table 2 - Minimum performance objectives for the seismic analysis-design of new structures
Seismically Designed Reinforced Concrete Framed Structures. (Martínez & Mander – 2002)

Damage State		**1**	**2**	**3**	**4**	**5**
Performance Level		Fully Operational	Immediate Occupancy	Restricted Operation	Life Safety	Collapse Prevention
Post-EQ structure utility		No Damage	Slight Damage	Repairable Damage	Non-Repair. Damage	Incipient Collapse
Required Repairs		None	Inspect, patch, make up, etc.	Repair Components	Rebuild Components	Rebuild Structure
Service disruption time		None	< 3 days	< 3 weeks	< 3 months	> 3 months
Plastic Rotation, θ_p (%)		**0.00**	**1.00**	**1.50**	**2.50**	**3.50**
FEQ	$R_P \approx 30$ y	**G1 – G2 – G3**				
OEQ	$R_P \approx 150$ y	**G2 – G3**	**G1 – G2**			
DEQ	$R_P \approx 500$ y	**G3**	**G2 – G3**	**G1 – G2**	**G1**	
MCEQ	$R_P \approx 2500$ y	**G3**	**G2 – G3**	**G2 – G3**	**G1 – G2**	**G1**

Phase II: Analysis corresponding to the Serviceability Limit State

II.1. Define the *serviceability requirements* in terms of the elastic displacement target as the lesser of:

i) Maximum elastic deformation derived from the limiting drift required by codes θ_{el}^{code} for the serviceability limit state.

$$D_{el}^{code} = \theta_{el}^{code}\left(N_s\ h_{col}\right)$$

ii) The upper limit deformation imposed by the yielding displacement of each structural element by using:

$$D_{yi} = \theta_{yi}\left(N_s\ h_{col}\right)$$

II.2. *Compute the elastic displacement demand of the system (D_{elCM})*

II.2.a. Based on the structure's geometry and the rigidity of the elements (k_i) defined in Step I.1.c., compute the *stiffness eccentricity* and the *torque related to a frequent event*

$$e_{CR} = \left(\frac{\sum k_i\, d_i}{\sum k_i} \right) - d_{CM} \qquad \text{and} \qquad M_{T\,el} = e_{CR} \sum V_{ri}$$

II.2.b. Evaluate the *torsional stiffness* (K_{Tel}) associated with the serviceability limit state.

$$K_{T\,el} = \sum k_{yi}\, x_{CRi}^2 + \sum k_{xi}\, y_{CRi}^2$$

II.2.c. Compute the *in-floor angle of twist* (φ_{Tel}) sustained by the building during an expected frequent event.

$$\varphi_{T\,el} = \frac{M_{T\,el}}{K_{T\,el}} = \frac{e_{CR} \sum V_{ri}}{\sum k_{yi}\, x_{CRi}^2 + \sum k_{xi}\, y_{CRi}^2}$$

II.2.d. Compute both the yielding and the elastic displacement of the system (D_{yCM} and D_{elCM})

i) The yielding displacement can be estimated from Step (I.1.d.) by

$$D_{y\,CM} = \theta_{y\,CM} \left(N_s\, h_{col} \right)$$

ii) According to the critical elastic displacement governing the design, and the displacement distribution profile generated by translational and torsional effects, compute the *elastic displacement demand* at the system centre of mass by

$$D_{el\,CM} = D_{y\,crit} \pm \varphi_{Tel} \left(d_{crit} - d_{CM} \right)$$

with $D_{y\,crit}$ = maximum elastic displacement capacity of the critical element, derived from the lesser of those defined in Step II.1.

II.3. *Compute the required elastic strength*

II.3.a. Obtain both the yielding and the elastic displacements of the system at the centre of mass, corresponding to the *SDOF* equivalent system

$$D_y^* = \frac{D_{yCM}}{\alpha_D} \qquad \text{and} \qquad D_{elCM}^* = \frac{D_{elCM}}{\alpha_D}$$

with α_D defined in Step I.3.b. for $n_{sp} = N_s$

II.3.b. Compute the upper bounds for each region of the elastic seismic demand according to the selected input *FEQ* level.

Table 3

Spectral Region	*Upper bound of the elastic displacement*
A Region	$D_{yA}^* = 0.625 \frac{g}{\pi^2} \left(Sa_{el1}\, F_A \right)_{FEQ}\ T_{0.4}^2$
V Region	$D_{yV}^* = \frac{g}{4\pi^2} \left(Sa_{el1}\, F_V \right)_{FEQ}\ T_1\ T_{2.5}$

II.3.c. Compute the elastic strength demand

Table 4

Spectral Region	*Range*	*Elastic strength capacity to be provided*
A Region	$0 < D^*_y \leq D^*_{yA}$	$\phi_f \; C^*_{e\,corr} \; = \; 2.50 \; \left(Sa_{el\,1} \; F_A\right)_{FEQ} \; \dfrac{D^*_y}{D^*_{el\,CM}}$
V Region	$D^*_{yA} < D^*_y \leq D^*_{yV}$	$\phi_f \; C^*_{e\,corr} \; = \; \dfrac{g}{4\pi^2} \; \dfrac{D^*_y}{\phi_d} \left[\dfrac{\left(Sa_{el\,1} \; F_V\right)_{FEQ} \; T_1}{D^*_{el\,CM}} \right]^2$
D Region	$D^*_{yV} \; < \; D^*_y$	$\phi_f \; C^*_{P\Delta}$

II.3.d. Verify that the minimum required strength condition for controlling *P-Delta* effects is accomplished:

$$C^*_{e\,corr} \; \geq \; C^*_{P\Delta} \; = \; \frac{4 \, D^*_y}{h^*_{eq}}$$

Phase III: Analysis corresponding to the ultimate limit state (moderate to severe ground motions)

III.1. Define the *ultimate limit requirements* in terms of the displacement targets, as the lesser of:

i) The maximum displacement capacity at the level of plastic substructure n_{sp} [5]:

$$D_{i\theta} = \left[\theta_{yi} \; + \; \theta^{DS}_{pi} \; \frac{L_b}{L} \right] \left(n_{sp} \; h_{col} \right)$$

ii) The element's displacement capacity at the level n_{sp} , based on the ductility level provided by Standards (μ_{max}^{code}):

$$D_{i\mu} \; = \; \left[\theta_{yi} \left(n_{sp} \; h_{col} \right) \right] \mu^{code}_{max}$$

III.2. Define the *system ductility* by an effective control of *inelastic torsion* [1], [3].

III.2.a. Derived the *strength eccentricity* and the *torque related to the ultimate limit state*

$$e_{CV} \; = \left(\frac{\sum V_{ri} \; d_i}{\sum V_{ri}} \right) - \; d_{CM} \qquad \text{and} \qquad M_{T\,ult} \; = \; e_{CV} \; \sum V_{ri}$$

III.2.b. Evaluate the *torsional stiffness* based on the effective stiffness of those transverse structural elements remaining in the elastic domain of response.

$$K_{T\,ult} \; = \; \sum k_{i\,transv} \;\; d^2_{CRi}$$

III.2.c. Compute the *in-floor angle of twist* sustained by the building at ultimate limit state during an expected severe event.

$$\varphi_{T\,ult} = \frac{M_{T\,ult}}{K_{T\,ult}} \; = \; \frac{e_{CV} \; \sum V_{ri}}{\sum k_{i\,transv} \;\; d^2_{CRi}}$$

III.2.d. Based on the critic displacement capacity which controls the design and the displacement distribution pattern generated by the combination of translational and torsional effects, compute the *displacement at the system centre of mass* (D_{CM}) by

$$D_{CM} \; = \; D_{max\;crit} \; \pm \; \varphi_{Tult} \left(d_{crit} - d_{CM} \right)$$

with $D_{max\;crit}$ = maximum displacement capacity of the critical element, derived from the lesser of those defined in Step III.1.

III.2.e. Define the *system design ductility* at the *centre of mass* by

$$\mu_s = \frac{D_{CM}}{D_{y\,CM}} = \frac{D_{CM}}{\theta_{y\,CM}\left(n_{sp}\,h_{col}\right)}$$

III.2.f. If any translatory element remains in the elastic range, then the process should be reassessed from the Step III.2.b. by using the expression shown in Step II.2.b.

II.3. *Determine the required seismic strength*

III.3.a. Obtain the yielding displacement corresponding to the *SDOF* equivalent system from the yielding rotation defined in Step I.1.d.

$$D_y^* = \frac{D_{y\,CM}}{\alpha_D} = \frac{\theta_{y\,CM}\left(n_{sp}\,h_{col}\right)}{\alpha_D}$$

with α_D defined for the partial-sway plastic mechanism adopted (Step I.3.b.)

III.3.b. Compute the upper bounds for each spectral region of the inelastic seismic demand

Table 5

Spectral region	*Upper displacement bound*
A Region	$D_{yA}^* = 0.625\,\frac{g}{\pi^2}\,\frac{\left(Sa_{el1}\,F_A\right)_{Rp}\,T_{0.4}^2}{\mu_s}$
V Region	$D_{yV}^* = \frac{g}{4\pi^2}\,\frac{\left(Sa_{el1}\,F_V\right)_{Rp}\,T_1\,T_{2.5}}{\mu_s}$

III.3.c. Based on the yielding displacement of the *SDOF* equivalent system (D^*_y), define the spectral region controlling the design, and compute the level of required strength to be provided.

Table 6

Spectral Region	*Yield Displ. Range*	*Inelastic strength capacity to be provided*
A Region	$0 < D^*_y \le D^*_{yA}$	$\phi_f\,C_y^* = \frac{2.50\,\left(Sa_{el1}\,F_A\right)_{Rp}}{1 + \left(\mu_s - 1\right)\left(T_{ei\,A} / T_{0.4}\right)}$
V Region	$D^*_{yA} < D^*_y \le D^*_{yV}$	$\phi_f\,C_y^* = \frac{g}{4\pi^2}\,\frac{1}{\phi_d\,D_y^*}\left[\frac{\left(Sa_{el1}\,F_V\right)_{Rp}\,T_1}{\mu_s}\right]^2$
D Region	$D^*_{yV} < D^*_y$	$\phi_f\,C_{P\Delta}^*$

Note: The cracked elastic period of the structure, for the acceleration sensitive region, can be computed as:

$$T_{ei\,A} = \frac{q}{2} + \left[\frac{q^2}{4} + p\right]^{0.50}$$

with

$$p = 1.60\,\frac{\pi^2}{g}\,\frac{D_y^*}{\left(Sa_{el1}\,F_A\right)_{Rp}} \quad ; \quad q = p\,\frac{\mu_s - 1}{T_{0.4}}$$

III.3.d. Verify the minimum required strength condition for controlling *P-Delta* effects:

$$C_y^* \ge C_{P\Delta}^* = \frac{g}{\pi^2}\,\frac{\left(Sa_{el1}\,F_V\right)_{Rp}\,T_1\,T_{2.5}}{h_{eq}^*}$$

IV. Based on the design strength selected, compute the structure's period and ductility level

IV.1. *Estimation of the cracked elastic period of the structure*

$$T_i = 2\pi \sqrt{\frac{D_y^*}{g \; C_{Design}^*}}$$

IV.2. *Definition of the structural ductility level*

IV.2.a. Structures designed with *serviceability requirements*:

Define the ductility developed by the building designed with serviceability provisions for a frequent earthquake, when sustaining a moderate or severe ground motion:

Table 7

		Period Range	*Ductility level for Serviceability State*
Ductility demand derived from the control of the elastic torsion	*A Region*	$0 < T_i \leq T_{0.4}$	$\mu_f^A = 1 + \frac{T_{0.4}}{T_i}\left(\frac{(Sa_{el1}\,F_A)_{Rp}}{(Sa_{el1}\,F_A)_{FEQ}^{corr}} - 1\right)$
	V Region	$T_{0.4} < T_i \leq T_{2.5}$	$\mu_f^V = \frac{(Sa_{el1}\,F_V)_{Rp}}{(Sa_{el1}\,F_V)_{FEQ}^{corr}}$
Design governs by minimum strength demand for controlling P-Delta effect	*A Region*	$0 < T_i \leq T_{0.4}$	$\mu_{f\,P\Delta}^A = 1 + \frac{T_{0.4}}{T_i}\left[0.625\,\frac{h_{eq}^*}{D_y^*}\,(Sa_{el1}\,F_A)_{Rp} - 1\right]$
	V Region	$T_{0.4} < T_i \leq T_{2.5}$	$\mu_{f\,P\Delta}^V = \frac{h_{eq}^*}{4\,D_y^*}\,\frac{(Sa_{el1}\,F_V)_{Rp}\,T_1}{T_i}$
	D Region	$T_i > T_{2.5}$	$\mu_{f\,P\Delta}^D = \frac{g}{4\pi^2}\,\frac{(Sa_{el1}\,F_V)_{Rp}\;T_1\;T_{2.5}}{D_y^*}$

with

$$(Sa_{el1}\,F_A)_{FEQ}^{corr} = 0.40\;C_{e\,corr}^* \qquad \textit{and} \qquad (Sa_{el1}\,F_V)_{FEQ}^{corr} = C_{e\,corr}^*\,\frac{T_i}{T_1}$$

IV.2.b. Buildings designed to sustain *Moderate or Severe Ground Motions*

Table 8

		Period Range	*Ductility level for Moderate-Severe EQ*
Ductility demand derived from the control of the inelastic torsion		$0 < T_i \leq T_{2.5}$	$\mu = \mu_s$
Design governs by minimum strength demand for controlling P-Delta effect	*A Region*	$0 < T_i \leq T_{0.4}$	$\mu_{s\,P\Delta}^A = 1 + \frac{T_{0.4}}{T_i}\left[0.625\,\frac{g}{\pi^2}\,\frac{(Sa_{el1}\,F_A)_{Rp}\,T_i^2}{D_y^*} - 1\right]$
	V Region	$T_{0.4} < T_i \leq T_{2.5}$	$\mu_{s\,P\Delta}^V = \frac{g}{4\pi^2}\,\frac{(Sa_{el1}\,F_V)_{Rp}\,T_1\;T_i}{D_y^*}$
	D Region	$T_i > T_{2.5}$	$\mu_{s\,P\Delta}^D = \frac{g}{4\pi^2}\,\frac{(Sa_{el1}\,F_V)_{Rp}\;T_1\;T_{2.5}}{D_y^*}$

Phase V: Assessment of the performance objective controlling the design, the associated base shear force and the seismic safety level.

V.1. *Define the performance objective which controls the design*

The critical strength level obtained from the different performance couples $(Sa_{el1}\ F_i)_{Rp}$ - DS_i should be now selected to define the structural final design. An appropriate structural configuration, compatible with the expected building response should thus be conceived based on the level of performance arrived at by consensus agreement among owners, professionals, authorities and Standards.

V.2. *Define the base shear force for the earthquake level which controls the design*

$$V_b = \left(C^*_{Design}\ \alpha_A \right) W_s$$

The base shear force acting upon the building must now be distributed proportionally according to the relative strength assigned on each structural element in Step I.1.b., so that:

$$V_b = \sum V_{ri}$$

V.3. *Compute the equivalent lateral force pattern for each structural element*

Since design quantities acting on each structural member are based on those effects resulting from the application of *equivalent static horizontal seismic forces*, the base shear force previously computed should be distributed vertically by the equations provided in Table 9 [5].

Table 9

Height Range	*Lateral force at each level - F_i*
Base level $< n_i \le n_{sp}$	$V_{bi} \dfrac{n_i / n_{sp}}{\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} n_i / n_{sp}}$
$n_{sp} < n_i \le N_S$	$V_{bi} \dfrac{1}{\left(N_S - n_{sp}\right) + \sum_{i=1}^{n_{sp}} n_i / n_{sp}}$

V.4. *Assessment of the Seismic Safety Level and the Confidence Interval*

V.4.i. In order to verify the earthquake level of safety selected in Step I.4.b., the median estimate of the seismic event [5] sustained by the selected design strength should be computed by:

Table 10

	Period range	*Expected seismic event (50% percentile level)*
A Region	$0 < T_i \le T_{0.4}$	$\left(Sa_{el1}\ F_A\right)^C_{50\%} = 0.40\ C^*_{Design} \left[1 + (\mu - 1) \dfrac{T_i}{T_{0.4}} \right]$
V Region	$T_{0.4} < T_i \le T_{2.5}$	$\left(Sa_{el1}\ F_V\right)^C_{50\%} = \dfrac{2\pi}{T_1} \mu \sqrt{\dfrac{C^*_{Design}\ D^*_y}{g}}$
D Region	$T_i > T_{2.5}$	$\left(Sa_{el1}\ F_V\right)^C_{50\%} = \dfrac{4\pi^2}{g} \dfrac{D^*_y}{T_1\ T_{2.5}} \mu$

Note: For the assessment of the serviceability elastic state the ductility level to be adopted must be $\mu = 1$.

V.4.ii. Determine the seismic factor of safety and the confidence interval associated with the proposed design [5].

$$SFS = \frac{\left(Sa_{el1}\ F_i\right)^{C}_{50\%}}{\left(Sa_{el1}\ F_i\right)^{D}_{Rp}}\lambda_{CI} \qquad \text{and} \qquad CI_{\%} = \frac{100}{1 + \left[SFS\right]^{-\left(1.85/\beta_c\right)}}$$

- Proceed as described for both directions of analysis, assessing the clockwise and anticlockwise in-plant-twist. Adopt the most critical situation. If the building presents a given degree of asymmetry the approach may require a major level of refinement by implementing an iteration process.
- Once the proposed procedure has been applied, the external force profile obtained in Step V.3. should be taken into account for the determination of seismic actions. Furthermore, the structural design must be performed in accordance with the *Capacity Design* principles (NZS 3101:1995; INPRES-CIRSOC 103:2000) ensuring the accomplishment of the plastic mechanism previously postulated in Step I.2.

Example:

As a numerical application of the methodology previously described, an example of a moderately flexible building is presented. A 10-storey ($N_s = 10$) reinforced concrete moment resisting frame building is adopted. This is an office building belonging to the seismic group use **G2** (significant public hazard due to high occupancy). The model adopted belongs to the "*Example of a reinforced concrete framed building design*" [7], conceived with *Capacity Design* principles. The plant is presented in Figure 1. In order to compute the element yield rotation (Step I.1.a.), the average dimension value of beams, columns and bay lengths has been taken into account.

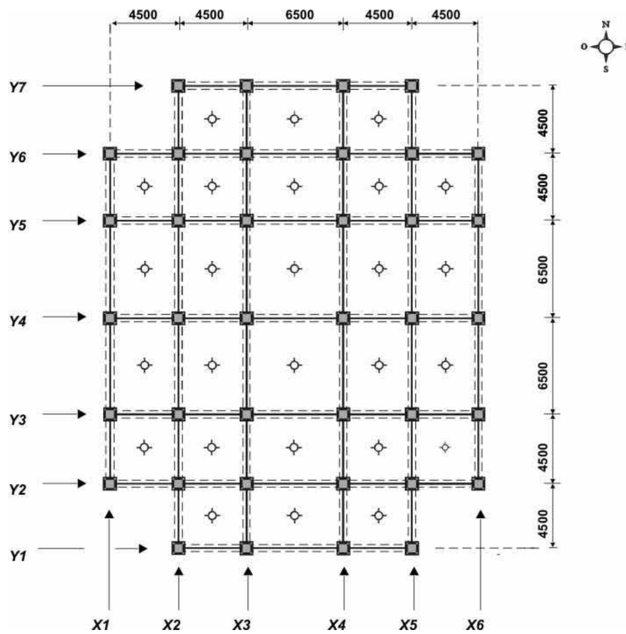

Figure 1

For the assessment of the ultimate limit state, the selected number of storeys involved in the partial sway mechanism is $n_{sp} = 6$. The design earthquake level adopted for the site in analysis, corresponding to *475 year* recurrence interval (strictly 10% probability in 50 years) is $(Sa_{el1}\ F_i)_{475y} = 0.35g$. Besides, an accidental eccentricity is adopted as approximately ±10% of the longest dimension.

Table 11. Parameters of the TRANSLATORY structural elements

Translatory Frames	θ_{yi} *rad*	V_{ri} *kN / kN*	k_{iy} *V / m*	*Coord X m*	*CRx m*	*CVx m*
1y	0.0037	0.10	0.82	0.00	*15.50*	*15.50*
2y	0.0036	0.16	1.38	4.50		
3y	0.0035	0.16	1.42	9.00		
4y	0.0035	0.16	1.42	15.5		
5y	0.0035	0.16	1.42	22.0		
6y	0.0036	0.16	1.38	26.5		
7y	0.0037	0.10	0.82	31.00		

Table 12. Control parameters of inelastic torsion – Ultimate limit state

CV_x	$e_{CV\,x}$	$M_{T\,ult}$	$K_{T\,ult}$	$\varphi_{T\,ult}$
15.50 m	3.0 m	3.0 Vm	627.9 Vm	**0.0048 rad**

Table 13. Control parameters of elastic torsion – Service limit state

CR_x	$e_{CR\,x}$	$M_{T\,el}$	$K_{T\,el}$	$\varphi_{T\,el}$
15.50 m	3.0 m	3.0 m	1478 Vm	**0.0020 rad**

Table 14. Transformation factors and system's equivalent height

Transformation factor	Moderate-to-severe EQ n_{sp} = *6 storeys*	Frequent EQ $n_{sp} = N_s$ = *10 storeys*
α_A	0.862	0.786
α_D	1.149	1.429
h^*_{eq}	21.8 m	22.9 m

Table 15. Results obtained for the entire range of performance couples

	Serviceability	Moderate to Severe Earthquakes		
Earthquake Designation	*FEQ*	*OEQ*	*DEQ*	*MCEQ*
Period of Recurrence, R_p	30 years	150 years	500 years	2500 years
Expected Confidence Interval	84% / λ_{CI} = 1.46	90% / λ_{CI} = 1.73		
Seismic Demand $(Sa_{el1}\,F_i)_{Rp}$	0.21 g	0.41 g	0.62 g	1.05 g
Performance Level	Fully Operational	Immediate Occupancy	Restricted Operation	Life Safety Level
Expected Damage State	*DS1*	*DS2*	*DS3*	*DS4*
Plastic Rotation, θ_p	0.000	0.010	0.015	0.020
Displacement Capacity of the Structure's Critical Element	$D_{el\ crit\ 7}$ = 0.120 m	$D_{\theta 7}$ = 0.241 m	$D_{\theta 7}$ = 0.326 m	$D_{\theta 7}$ = 0.410 m
Yielding Displacement (MDOF System)	D_y^{Ns} = 0.084 m	D_y^{nsp} = 0.069 m		
Displacement of the Centre of Mass (MDOF System)	0.084 m	0.153 m	0.237 m	0.321 m
Ductility demand, μ_s	---	2.21	3.43	4.65
Region governing the design	*V Region*	*V Region*	*V Region*	*V Region*
Strength Demand, $\phi_f\ C^*_i$	**0.198 g**	0.145 g	0.134 g	**0.211 g**

Table 16. Summary of the critical parameters

Limit State =	Serviceability	Ultimate
Critical performance couples =	FEQ-DS1	MCEQ-DS4
Design strength level, C^*_{Design} =	0.233 g	0.291 g
Cracked elastic period, T_i =	1.43 sec	0.90 sec
Ductility level to be provided, μ =	3.7	4.7
50% percentile seismic capacity =	0.28g	1.23g
Seismic safety level achieved =	≈ 1.95	≈ 2.00
Confidence Interval, $CI_{\%}$ =	88%	90%
Base Shear, V_b =	10565 kN	15650 kN

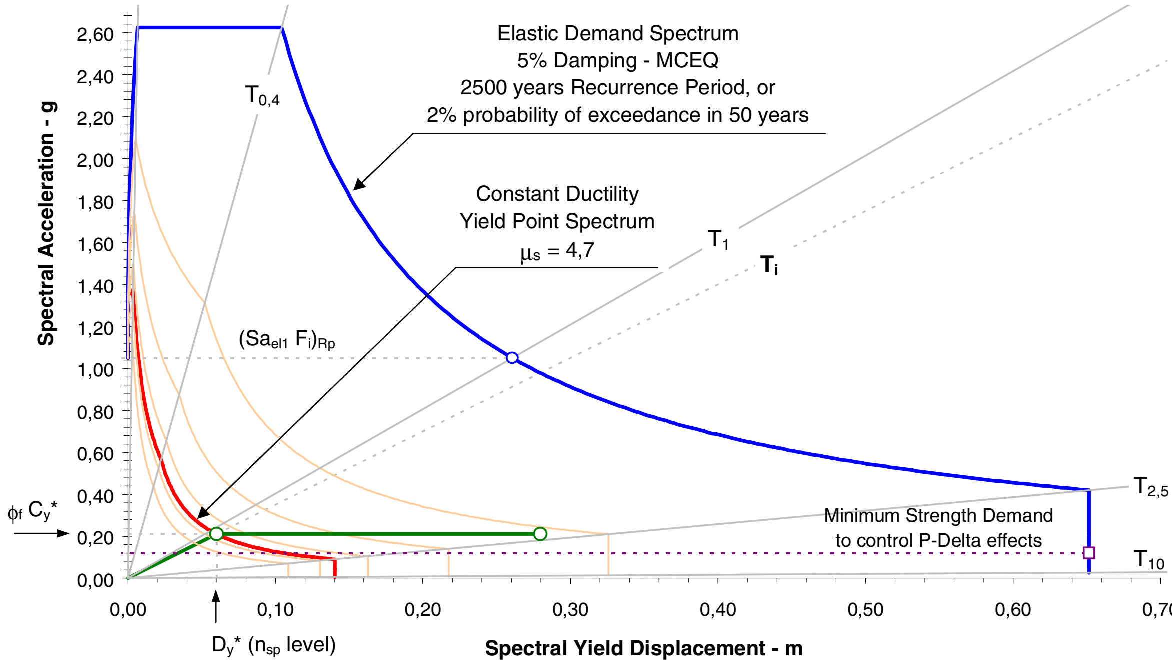

Figure 2. Ultimate Limit State analysis – Performance couple *MCEQ-DS4*

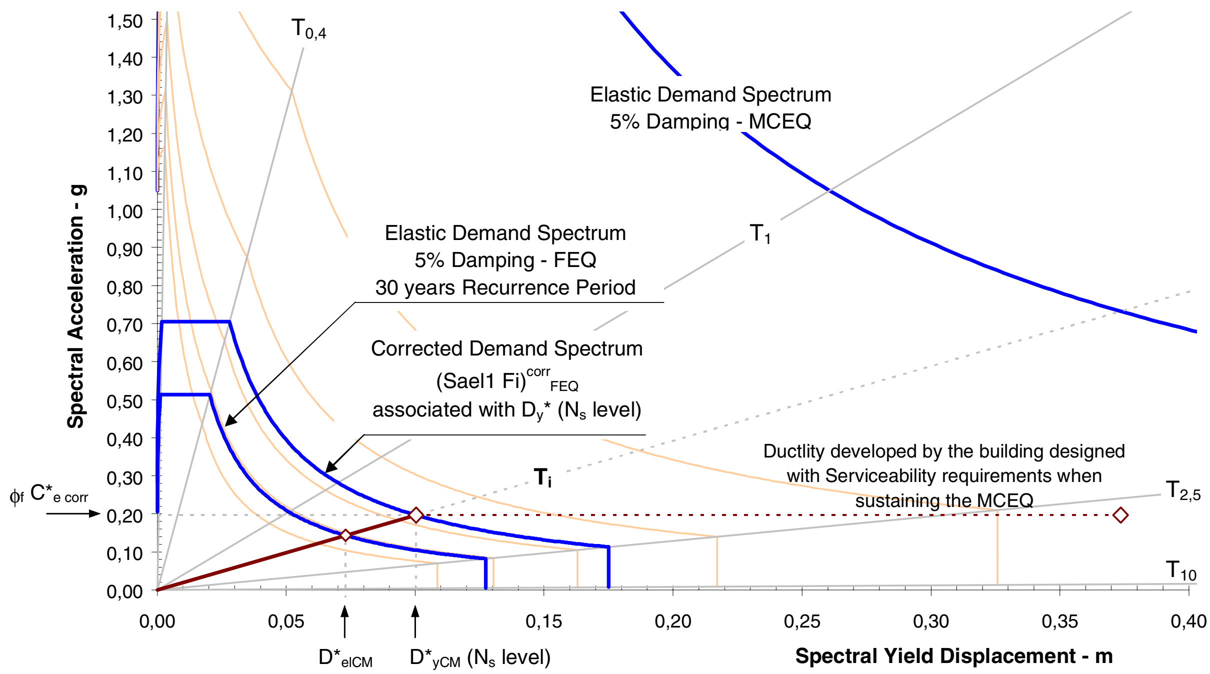

Figure 3. Serviceability Limit State analysis – Performance couple *FEQ-DS1*

CONCLUSIONS

As for the requirements of the emerging *performance-based seismic engineering*, the present paper developed a formal theoretical basis for a preliminary seismic analysis of new reinforced concrete frame structures. The main conclusions of this proposal can be detailed in what follows:

- In keeping with the performance-based seismic engineering philosophy a single procedure enables the designer to assess different minimum performance levels. This approach is not a time-consuming computer process since the procedure can be performed by hand or by a simple spreadsheet in a transparent and straightforward fashion.
- Comprehensive evaluations of torsional effects based on the control of earthquake-induced displacement demand are applied (Paulay) [1]. However, further investigations are needed to account for torsionally unrestrained systems developed under skew displacements imposed by an earthquake.
- Remarkable concepts recently developed and mainly related to strength-stiffness relationship, building yielding displacement, freedom in the strength assignment on structural elements, etc., are involved (Priestley & Kowalsky [2]; Paulay [3]).
- The methodology is described in terms of the constant ductility-yield point spectra (Ascheim & Black [4]) presented as *Acceleration-Yield Displacement* (*A-YD*) spectral smoothed curves. These are based on an elastic spectrum defined in terms of a probabilistic model. (Martínez & Mander [5]).
- Considerations about partial-sway admissible plastic mechanisms have been taken into account. For the evaluation of ultimate state (Peckan, Mander & Chen-1997 and Martínez & Mander [5]).
- Discrete values of *damage states* are proposed and quantified for the entire range of intermediate levels, from elastic serviceability (*DS1*) through collapse (*DS5*), in order to define the performance objectives (Martínez & Mander-2002 [5]).
- Minimum strength demand requirements to control *P-Delta* effects are also contemplated (Martínez & Mander-2002 [5]).
- A probabilistic assessment of the Confidence Interval suitably selected by the designer at early stages of the procedure is also taken into account (Martínez & Mander-2002 [5]).
- The procedure to be described is mostly intended to be a displacement-based design approach, even though the method can also be implemented as a force-based design.
- Rather than advancement in complex analytical techniques, a transparent procedure based on simple concepts relevant to the usual seismic analysis-design is here advocated.

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NOMENCLATURE

C^*_{Design}	Seismic strength demand of the *SDOF* equivalent system defined in terms of the normalised base shear provided by Table 4 or Table 6 multiplied by ϕ_f^{-1}; as a fraction of g.
$C^*_{e\,corr}$	Corrected elastic resistance demand coupled with D^*_{yCM} (Table 4); as a fraction of g.
CI	Confidence Interval.
CM	System Centre of Mass.
$C^*_{P\Delta}$	Minimum strength demand required to control *P-Delta* effects; as a fraction of g.
CR	System Centre of Rigidity.
CV	System Centre of Resistance.
C^*_y	Seismic strength demand of the *SDOF* system to be provided (Table 6); as a fraction of g.
D_b	Overall depth of beams; in *mm*.
D_{el}^{code}	Maximum elastic displacement derived by codes for the serviceability limit state; in *mm*.
$D_{el\,CM}$	Elastic displacement demand at the Centre of Mass of the system; in *mm*.
$D^*_{el\,CM}$	Elastic displacement demand of the *SDOF* system derived from D_{elCM}; in *mm*.
d_{CM}	Coordinate at the Centre of Mass of the system; in *mm*.
d_{CRi}	Coordinate of the transverse structural component i^{th} with respect to the Centre of Rigidity.
d_{crit}	Coordinate of the critical element or the first element in reaching the displacement capacity target for either serviceability or ultimate limit state; in *mm*.
d_i	Coordinate of each structural component i^{th}; in *mm*.
$D_{i\mu}$	Available displacement of the structural element i^{th} derived from the level of ductility μ_{max}^{code} gives by code provisions; in *mm*.
$D_{i\theta}$	Available limiting displacement of the structural element i^{th} obtained from a selected performance objective, according to the plastic rotation θ_p adopted; in *mm*.
D_{CM}	Displacement measured at the system Centre of Mass of the *MDOF* structure; in *mm*.
D_{yi}	Yielding displacement of the element i^{th} belonging to the *MDOF* structure; in *mm*.
D^*_y	Yielding displacement of the *SDOF* equivalent system derived from D_{yCM} ; in *mm*.
D^*_{yA}	Limiting yield spectral displacement for the acceleration controlled region; in *mm*.
$D^*_{y\,CM}$	Yielding displacement of the *SDOF* system measured at the Centre of Mass; in *mm*.
D^*_{yV}	Limiting yield spectral displacement for the velocity controlled region; in *mm*.
DS_i	Structural performance parameter corresponding to *Damage State* i^{th}.
e_{CR}	Eccentricity of the Centre of Rigidity respect to the System Centre of Mass; in *mm*.
e_{CV}	Eccentricity of the Centre of Resistance respect to the System Centre of Mass; in *mm*.
E_S	Elastic modulus of reinforcing steel; in *MPa*.
F_A	Coefficient that allows for site conditions in the constant spectral acceleration segment.
F_i	Coefficient that allows for site conditions according to the spectral portion governing the design, either F_A or F_V.
F_V	Coefficient that allows for site conditions in the constant spectral velocity region.
f_y	Longitudinal reinforcement yield strength; in *MPa*.
$G1$	Seismic Use Group I: Buildings of normal occupancy (NEHRP 1997).
$G2$	Seismic Use Group II: Facilities with substantial public hazard (NEHRP 1997).
$G3$	Seismic Use Group III: Essential buildings or emergency response facilities (NEHRP 97).
h^*_{eq}	Equivalent height of the *SDOF* system; in *mm*.
h_{col}	Average storey height; in *mm*.
I_{gross}	Gross moment of inertia of a structural element cross-section; in mm^4.
k_i	Cracked elastic stiffness defined close to the yield point of the element i^{th}; in *N/mm.*

$k_{i\ transv}$ In-plane stiffness of the transverse structural element $i^{\underline{th}}$; in *N/mm*.
K_T In-floor-torsional rigidity; in *Nmm/rad*.
L Bay length or beam span between centrelines of columns; in *mm*.
L_b Beam length between plastic hinges; in *mm*.
L_s Beam span between the internal face of columns; in *mm*.
m_i Mass at level $i^{\underline{th}}$; in *Kg*.
$M_{T\ el}$ Torsional moment associated with the serviceability limit state; in *Nmm*.
$M_{T\ ult}$ Torsional moment associated with the ultimate limit state; in *Nmm*.
n_i Generic $i^{\underline{th}}$ storey belonging to the plastic substructure ($i = 1, 2, \ldots n_{sp}$)
n_{sp} Number of storeys involved in the structure's deformed portion. For the serviceability state $n_{sp} = N_s$, simulating the deflection profile corresponding to the first mode of vibration. For ultimate limit state level, n_{sp} is the number of stroreys involved in the plastic mechanism.
N_s Total number of storeys.
R_P Expected return period of a seismic event, expressed in years.
Sa_{el1} Elastic spectral (pseudo) acceleration for 1 sec period amplitude; as a fraction of *g*.
T_i Structural period in analysis; in *sec*.
$T_{0.4}$ Period amplitude corresponding to 0.40 sec; in *sec*.
T_1 Period amplitude corresponding to 1.00 sec; in *sec*.
$T_{2.5}$ Period amplitude corresponding to 2.50 sec; in *sec*.
T_{10} Period amplitude corresponding to 10.00 sec; in *sec*.
V_b Base shear force; in *N*.
V_{ri} Nominal relative strength provided to each structural element $i^{\underline{th}}$; ($V_b = \Sigma V_{ri}$)
W_S Total tributary seismic weight acting on the structure in analysis; in *N*.
x_{CRi} Coordinates of the structural element $i^{\underline{th}}$ in the *x* direction, respect the centre of rigidity *CR*.
y_{CRi} Coordinates of the structural element $i^{\underline{th}}$ in the y direction, respect the centre of rigidity *CR*.
α_A Acceleration transformation factor from a *MDOF* to a *SDOF* equivalent system.
α_D Displacement transformation factor at level n_{sp} or roof level.
β_c Lognormal composite dispersion associated with the seismic assessment ($\beta_c \approx 0.60$).
ϕ_d Structural displacement under-capacity factor ($\phi_d \approx 0.85$).
ϕ_f Structural strength reduction factor ($\phi_f \approx 0.85$).
$\varphi_{T\ el}$ Angle of rotation related to the serviceability limit state; in *rad*.
$\varphi_{T\ ult}$ Angle of rotation related to the ultimate limit state; in *rad*.
λ_{CI} Demand amplification factor to provide the expected confidence interval.
μ Displacement ductility ratio; $\mu = D_{max}/D_y$.
μ_f Ductility developed by the structure designed with serviceability requirements, when sustaining a rare or severe ground motion.
μ_{max}^{code} Dependable ductility capacity provided by codes.
μ_s Displacement ductility ratio developed under severe ground motion conditions.
$\mu_{P\Delta}$ Ductility associated with the minimum resistant structure's capacity
θ_{el}^{code} Limiting drift required by Standards for serviceability state; in *rad*.
θ_p Plastic rotation of the structural element cross section; in *rad*.
θ_{pi}^{DS} Limiting plastic drift associated with the selected damage state of the element $i^{\underline{th}}$; in *rad*.
θ_{yi} Yielding rotation of the structural element $i^{\underline{th}}$; in *rad*.
θ_{yCM} Yielding rotation of the system Centre of Mass; in *rad*.