5th World Congress on Civil, Structural, and Environmental Engineering (CSEE'20)
~~Lisbon, Portugal~~ Virtual Conference – October 2020
Paper No. ICSECT 112
DOI: 10.11159/icsect20.112

Simplified Procedures for a Peruvian Standard of Analysis and Design of Buildings with Energy Dissipation Systems

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***Abstract* -** The application of the simplified procedures of Chapter 18 of ASCE 7-16 is studied together with the seismic Peruvian E.030 standard for the design of new buildings with energy dissipation systems in Peru. An example of design for the seismic force-resisting system of a 5-story reinforced concrete building with fluid viscous dampers located in the city of Lima is developed. The analyses performed show that it is possible to reduce the dimensions of the structural elements of the corresponding undamped original building, while controlling the story drifts and deformations as required by the E.030 standard. The results of the proposed methods were compared with the results of nonlinear time-history analyses and in general conservative predictions of maximum roof displacements, story drifts and base shears were obtained.

***Keywords*:** Seismic protection, energy dissipation, Peru, earthquakes.

1. Introduction

At present many energy dissipation systems are already commercially available in Peru and many new projects as well as retrofitting of existing buildings are being developed using these seismic protection systems. While several countries have already established codes for the analysis and design of buildings with energy dissipation devices, our country does not yet have an own standard on the matter. Peru is a country with high seismic activity, therefore it is very important to do research as a basis for developing a first local standard for the regulation of the use of dampers in civil structures taking into account the main international codes, the characteristics of the Peruvian earthquakes and the local engineering practice. This paper is based on a master thesis which covers more in detail the issue [1].

2. Description of the alternate procedures of Chapter 18 of ASCE 7-16

The simplified methods of analysis of buildings with dampers are the response spectrum procedure and the equivalent lateral force procedure which were originated from the Method 2 of FEMA 274 standard [2]. Both methods are called alternate procedures in Chapter 18 of ASCE 7-16 [3] and are based on the following assumptions [4]:

- Under certain conditions, a structure with damping devices (either velocity dependent or displacement dependent) and with nonlinear behaviour of the seismic force-resisting system can be represented as a structure with equivalent linear stiffness and viscous damping.
- The building must be designed to have a single-degree-of-freedom collapse mechanism with plastic hinges that meet the weak beam/strong column criterion in order to estimate the plastic base shear strength.
- The inelastic response of the building will be represented with an elastoplastic model.
- In each principal direction the building will be analyzed with one degree-of-freedom per floor.

Strictly speaking, a building with dampers is a system with nonclassical damping and such a system has coupled differential equations and cannot be solved with the classical modal analysis. For a building with viscous dampers, the first approximation of the simplified methods is the assumption that the frequencies and mode shapes of the damped system are the same of the undamped system. In this way it is possible to perform the dynamic analysis using mode superposition.

To account for the inelastic behaviour of the structure in the first mode of vibration an effective stiffness related to an effective period and the displacement ductility ratio will be employed. The total damping of the system is called effective damping and is the sum of the structural inherent damping, the viscous damping of the devices added to the structure and the hysteretic damping due to the inelastic deformations of the structure. For each mode of vibration, its corresponding effective damping is related to a damping coefficient which will reduce the demand spectral curve.

The analysis process is iterative because the procedure is based on the capacity spectrum method and it has been documented in a large base report [4] in 2000. In that report were presented many complementary studies which validated the analysis procedures proposed and shed light on the scope of them. The simplified methods are linear and hence can be employed in the commercial analysis programs.

3. Application of the simplified procedures for analysis and design of buildings with dampers with the seismic Peruvian standard E.030

3. 1. Design of the reference building without dampers

The pseudo-acceleration spectrum of the seismic Peruvian standard E.030 *Diseño Sismorresistente* [5] with 5% damping is defined by Eq. (1) and corresponds to a design earthquake with 475-year return event. The parameters which contribute to the spectral acceleration S_a are the seismic factor zone Z, the occupancy factor U, the dynamic amplification factor C, the site factor S which accounts for soil characteristics and the response modification factor R which depends on the structural system (R=8 for a building with reinforced concrete moment frames). The dynamic amplification factor represents the magnification of the accelerations at the building foundation due to the structure itself and is a function of the mode period T and parameters T_P and T_L, which also depend on the soil and delimit the velocity-sensitive region of the spectrum (Fig. 1).

$$S_a = \frac{ZUCS}{R} g \tag{1}$$

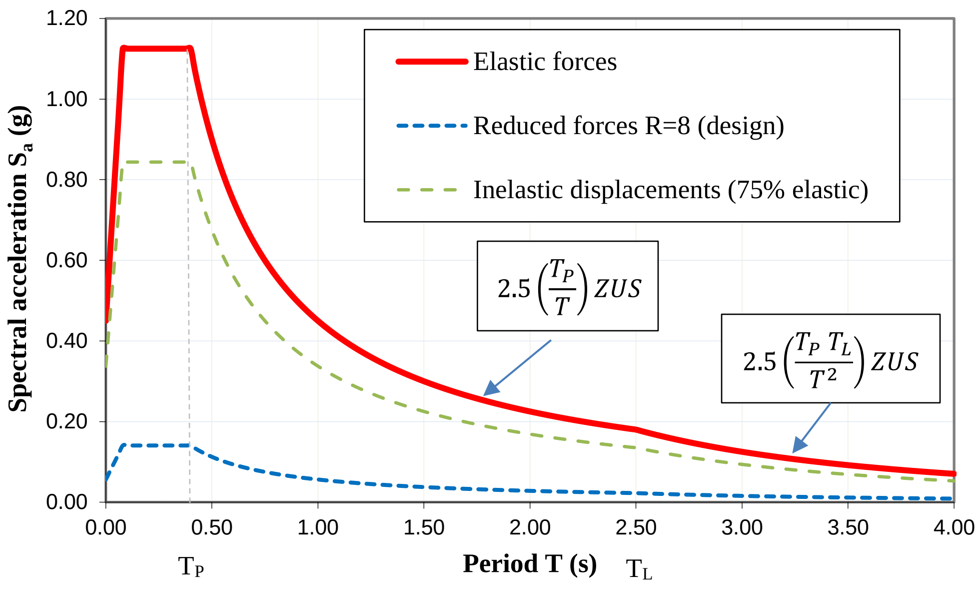


Fig. 1: Seismic Peruvian standard E.030 spectrum – 5% Damping – RC moment frame regular building.

The reference building is a five-story office building located in Lima, on sandy gravel, with a square plan with side 37.5 m, total built area of 7,031 m^2 and is isolated from any other structure. The structural system are six reinforced concrete moment frames in each direction (Fig. 2a) and the centre-to-centre distance between columns is 7.5 m. The height of the first story is 4 m and the typical story height is 3.6 m (total height of the building is 18.4 m). The floor is a 180 mm-thick two-way concrete slab. The parameters which define the spectrum for this example are shown in Table 1. The SAP2000 program was used to model the building and to perform the analyses (gravity loads, modal and response spectrum). The E.030 standard requires that the total seismic weight of the structure is 100% of the dead load plus 25% of the live load for common occupancy. The seismic weight obtained was 61,522 kN and the fundamental period was 0.756 s in both directions due to the symmetry. The mass participation factor of this first translational mode was 85% and thus the structural response is basically in the fundamental mode. The spectral base shear obtained was V = 3,971 kN = 0.065g.

Table 1: Spectrum parameters of E.030 standard – Building located in the coast of Peru on good soil [4].

Seismic factor zone	$Z =$	0.45		Place:	Lima (Zone 4)
Site factor	$S =$	1.00		Soil:	S_1
Start of velocity-sensitive region	$T_P =$	0.40	s	Soil:	S_1
Start of displacement-sensitive region	$T_L =$	2.50	s	Soil:	S_1
Occupancy factor	$U =$	1.0		Category:	C – Common buildings

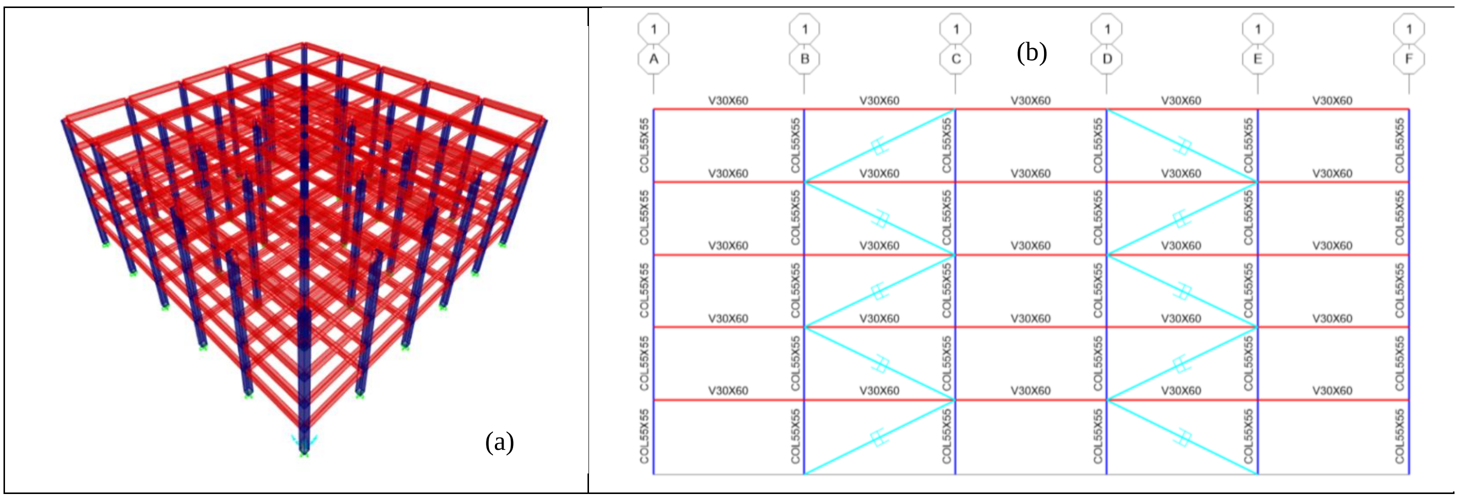


Fig. 2: (a) SAP2000 model of the RC moment frame building and (b) location of fluid viscous dampers.

Beams and columns were designed to meet the strength and serviceability requirements of the structural concrete Peruvian standard E.060 *Concreto Armado* [6] using the seismic base shear V = 0.065g and considering 5% of accidental torsional loads. The specified compressive strength of concrete f'_c was 21 MPa and the specified yield strength of steel reinforcement f_y was 420 MPa (ASTM A615). Were required 600 mm-square columns and beams of width b = 350 mm and height h = 750 mm. Figure 3a shows the reinforcement provided at the base of typical columns and at the section of maximum moment in beams. The design complies with the weak beam/strong column criterion as required by the E.060 standard for this type of structural system.

The E.030 standard requires that lateral displacements shall be 0.75R times the obtained displacements with the reduced seismic forces (Fig. 1) for regular structures. The maximum lateral displacement of the roof was 92 mm and the maximum story drift was 7‰ and occurred in the second floor (it was exactly the maximum allowable story drift for reinforced concrete structures according to the E.030 standard) considering accidental torsional loads. It was verified that there are no torsional irregularities in the structure due to this accidental torsional effect considered in the analysis. The chosen sections of beams and columns meet the strength required with ease, however their dimensions are controlled by the allowable story drift.

3. 2. Design of the building with fluid viscous dampers

A design alternative for the reference building with fluid viscous dampers (FVD) will be presented. The scope of this design will be limited to the seismic force-resisting system as defined in ASCE 7-16 18.2.1.1 and will employ the spectrum of the seismic Peruvian E.030 standard. The requirements of the seismic force-resisting system will be the following:

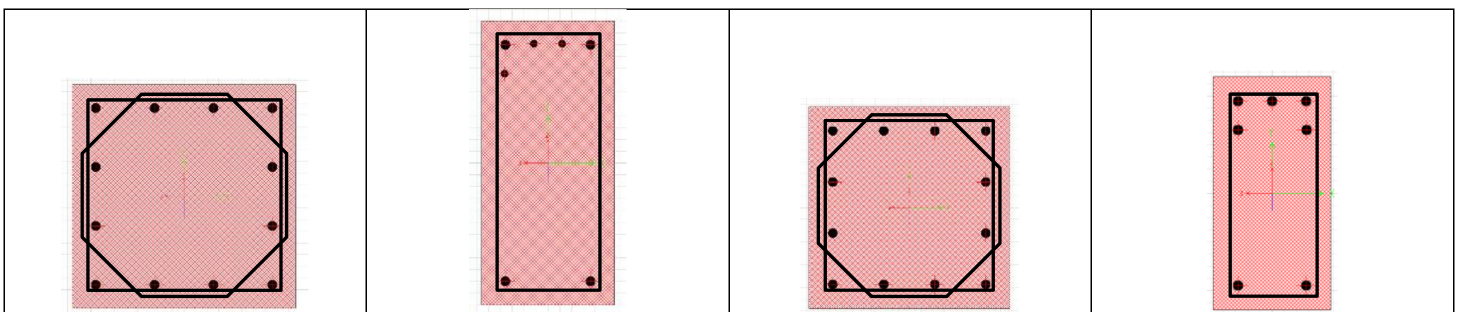
- The response modification factor R of the structural system specified by the E.030 standard and the corresponding overstrength factor Ω_0 from ASCE 7-16 will be taken. The deflection amplification factor C_d will be 0.75R, the corresponding to regular structures in the E.030 standard. For this example with reinforced concrete moment frames: R=8, Ω_0=3 y C_d=6.
- The minimum seismic base shear V_{min} will be the greatest of V/B_{V+I} or 0.75V, where V is the design spectral base shear of the structure without dampers with the E.030 standard and B_{V+I} is a damping coefficient related to the structural inherent damping plus the viscous damping for the fundamental mode under elastic conditions.

The application conditions for the simplified methods with the Peruvian code will be similar to those mentioned in ASCE 7-16 18.2.3 in each principal direction of analysis:

- The damping system will have at least two damping devices per floor in arrangement to resist torsion.
- The maximum effective damping for the fundamental mode will be 35%.
- Depending on the seismic zone in the Peruvian territory and the geotechnical conditions of site, the product of parameters T_P and Z shall not exceed 0.16 [1]. Given that the reference building does not comply this requirement (Z=0.45 and S=1.0 are representative parameters for the buildings in the city of Lima) it will be necessary to confirm the peak responses using additional nonlinear response history analysis.

As a first step it was assumed that V_{min} = 0.75V (this assumption will be checked later) and the required plastic base shear strength $V_{y\,req}$ of the building was calculated (Eq. 2). The sections of the structural elements were then reduced and designed with the weak beam/strong column criterion in such a way to have a minimum plastic base shear of 6,701 kN when the building is push over by static lateral loads on each floor with the shape of the first mode of vibration. Based on a plastic analysis, an approximate pushover curve was constructed using the building with the following reduced sections: 550 mm-square columns and beams of width b = 300 mm and height h = 600 mm. The plastic base shear strength obtained was V_y = 9,678 kN > $V_{y\,req}$ (144%). Figure 3b shows the placed reinforcement on typical sections of columns and beams. The building with reduced sections is more flexible, has a fundamental period T_1 = 1.014 s and has the desired collapsed mechanism. By applying the spectrum of E.030 standard to the reduced building and considering accidental torsional loads, the maximum story drift obtained was 9.69‰ and exceeded the allowable value (7.0‰).

$$V_{y\,req} = V_{min}\frac{\Omega_0 C_d}{R} = 0.75(3{,}971\ kN)\frac{3\times 6}{8} = 6{,}701\ kN \quad (2)$$



(a) Column 600X600 mm Longitudinal reinforcement: 12#8 - Stirrups: 2#4 Beam 350X750 mm Longitudinal reinforcement: 4#8 + 3#6 - Stirrup: 1#3	(b) Column 550X550 mm Longitudinal reinforcement: 12#8 - Stirrups: 2#4 Beam 300X600 mm Longitudinal reinforcement: 7#8 - Stirrup: 1#3

Fig. 3: Typical bars at column base and bars for maximum moment in beams for (a) reference building (b) building with FVD.

The modal analysis of the reduced building without dampers was done in MATLAB. In Table 2 are shown the modal shapes and derived properties from the modal analysis which were employed in the simplified procedures with the 5 translational modes for the response spectrum procedure. The equivalent lateral force procedure uses just the fundamental mode and a theoretically defined residual mode [4]. From the theory of linear spectral response, the definition of the modal participation factor Γ_m and the effective seismic weight $\overline{W}_m$ of the *m*th mode are the following, where m_i is the mass of story *i* and $\phi_{i,m}$ is the *m*th mode shape of story *i* (for this example n = 5):

$$\Gamma_m = \frac{\sum_{i=1}^{n} m_i \phi_{i,m}}{\sum_{i=1}^{n} m_i \phi_{i,m}^2} \tag{3}$$

$$\overline{W}_m = \left(\sum_{i=1}^{n} m_i \phi_{i,m} \right) \Gamma_m g \tag{4}$$

Table 2: Modal properties of the RC building with reduced sections.

	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Residual mode
T_m (s)	1.014	0.313	0.166	0.105	0.077	0.406
$\{\boldsymbol{\phi}\}_m$	0.205 0.478 0.721 0.899 1.000	-0.622 -1.009 -0.649 0.219 1.000	1.141 0.624 -0.977 -0.791 1.000	-1.855 1.063 0.880 -1.827 1.000	2.648 -3.735 3.734 -2.598 1.000	-2.591 -1.356 -0.259 0.542 1.000
$\overline{W}_m$ (kN)	45822	5681	2112	883	236	8912
Γ_m	1.284	-0.429	0.213	-0.091	0.022	-0.284

The FVD devices were dimensioned to reduce the story drifts of the reduced building to the allowable value. Four devices were considered per floor in each direction of analysis and were located at the frames of the periphery in diagonal arrangement (Fig. 2b). The elastic damping coefficient was determined simply as $B_{V+I} = B_{1E} = 9.69/7 = 1.38$ considering that the structure responds on the fundamental mode. This damping coefficient confirmed the initial assumed value of the minimum seismic base shear. Subsequently, with the Newmark & Hall formula [7] for spectrum amplification factors in the velocity-sensitive region it was calculated an elastic damping $\beta_{V+I} = 15.3\%$ in the fundamental mode which is necessary to meet the objective story drift. The damper coefficients C of the devices were dimensioned (Table 3) using the calculated elastic damping and employing the existing formulas for supplemental viscous damping ratio for FVD devices [8].

Just some illustrative calculations will be shown on the first mode for the case of linear FVD. With a value of the displacement ductility ratio $\mu_D = 1.48$, it was obtained an effective period T_{1D} which represents the structure inelastic action:

$$T_{1D} = T_1\sqrt{\mu_D} = 1.014\sqrt{1.48} = 1.235\ s \tag{5}$$

The spectrum employed for calculating the design forces in the structure (Eqs. 6-7) is adjusted with factor $R/(\Omega_0\, C_d)$ to match the level of performance at the formation of the first plastic hinge in the building and also is reduced by the factor B_{1D} related to the total effective damping. The calculated value of the spectral acceleration for the fundamental mode S_{a1} = 0.094g corresponds to point A in Fig. 4 where the first yielding in the structure will occur.

$$S_{a1} = \frac{2.5\, ZUS}{\Omega_o B_{1D}} \left(\frac{R}{C_d} \right) g \qquad T_{1D} < T_P \tag{6}$$

$$S_{a1} = \frac{2.5\, T_P ZUS}{T_{1D} \Omega_o B_{1D}} \left(\frac{R}{C_d} \right) g = \frac{2.5 \times 0.4 \times 0.45}{1.235 \times 3 \times 1.73} \left(\frac{8}{6} \right) g = 0.094g \qquad T_{1D} \geq T_P \tag{7}$$

The spectrum employed for calculating displacements (Eqs. 8-9) is reduced by factor B_{1D}. However, the calculated inelastic displacement of the roof D_{1D} = 103 mm (Eq. 10) cannot be lower than the corresponding elastic displacement (105 mm). The respective spectral displacements were obtained dividing both displacements by the participation factor of the fundamental mode Γ_1 = 1.284. The spectral displacements are shown in Fig. 4 as point B (80 mm) and point C (82 mm).

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{2.5\,ZUS\,T_{1D}^2}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{2.5\,ZUS\,T_1^2}{B_{1E}} \qquad T_{1D} < T_P \tag{8}$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{2.5\,T_P ZUS\,T_{1D}}{B_{1D}} \geq \left(\frac{g}{4\pi^2}\right)\Gamma_1 \frac{2.5\,T_P ZUS\,T_1}{B_{1E}} \qquad T_{1D} \geq T_P \tag{9}$$

$$D_{1D} = \left(\frac{g}{4\pi^2}\right)(1.284)\frac{2.5 \times 0.4 \times 0.45 \times 1.235}{1.73} = 103\,mm \geq \left(\frac{g}{4\pi^2}\right)(1.284)\frac{2.5 \times 0.4 \times 0.45 \times 1.014}{1.38} = 105\,mm \tag{10}$$

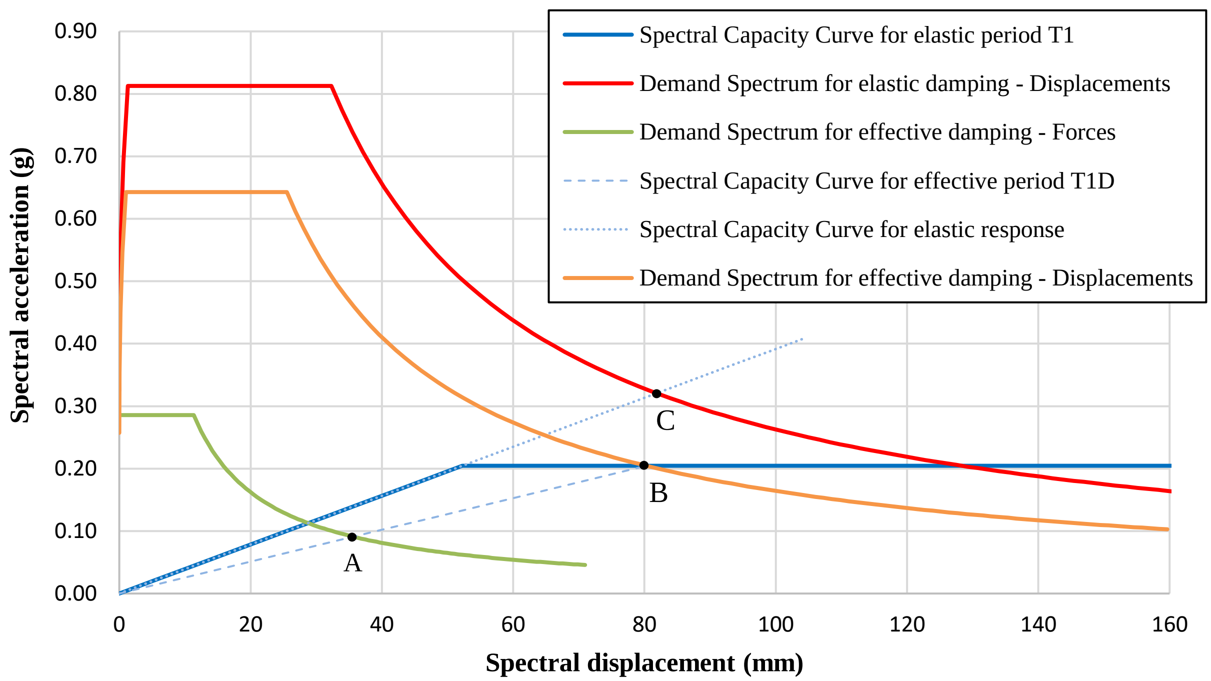


Fig. 4: Response on the fundamental mode – Building with linear FVD – Spectrum of E.030 standard.

Analyses for many velocity exponents α of the nonlinear FVD were performed (Table 3) for a unique level of provided elastic damping β_{V+I} = 15.3% in the fundamental mode and thus, keeping the same elastic damping coefficient B_{V+I} = 1.38 in order to obtain the same objective story drift of 7.0‰. All the analyses maintained the same plastic base shear strength of the building. The modal combination method employed was SRSS. From Table 3 it is inferred that, for a similar building performance with reduced sections, as the velocity exponent α of the devices decreases:

- The damper coefficient C of the devices diminishes, which means that it will be required smaller sizes for dampers with a low value of the velocity exponent α.
- The displacement ductility ratio μ_D diminishes slightly and therefore, the effective period T_{1D} and the hysteretic damping β_H also diminish (Eqs. 5 and 11). The hysteretic damping also depends on a hysteretic loop adjustment factor q_H = 0.5 and the structural inherent damping β_I (5%); both quantities remain constant during the analyses.

- The total effective damping β_{1D} increases slightly in the first mode because the viscous damping β_{V1} is multiplied by a power of the displacement ductility ratio with the velocity exponent α (Eq. 12).

$$\beta_H = q_H(0.64 - \beta_I)\left(1 - \frac{1}{\mu_D}\right) \tag{11}$$

$$\beta_{1D} = \beta_I + \beta_{V1}(\mu_D)^{1-\frac{\alpha}{2}} + \beta_H \tag{12}$$

Table 3: Provided damping on the fundamental mode.

Analysis case	Elastic damping β_{V+I}	Damper coefficient C for each device		Hysteretic damping β_H	Displacement ductility ratio μ_D	First mode effective period T_{1D}	Effective damping β_{1D}
	(%)	kN.(s/mm)$^\alpha$	ton.(s/m)$^\alpha$	(%)		(s)	(%)
FVD α = 1.0	15.3	5.1	510	9.6	1.484	1.235	27.1
FVD α = 0.9	15.3	8.3	414	9.5	1.473	1.231	27.4
FVD α = 0.8	15.3	13.2	331	9.4	1.467	1.228	27.5
FVD α = 0.7	15.3	21	264	9.3	1.461	1.226	27.6
FVD α = 0.6	15.3	33.4	211	9.2	1.455	1.223	27.8
FVD α = 0.5	15.3	53	168	9.1	1.449	1.221	27.9
FVD α = 0.4	15.3	84	133	9.1	1.443	1.218	28.0
FVD α = 0.3	15.3	133	106	9.0	1.437	1.216	28.2

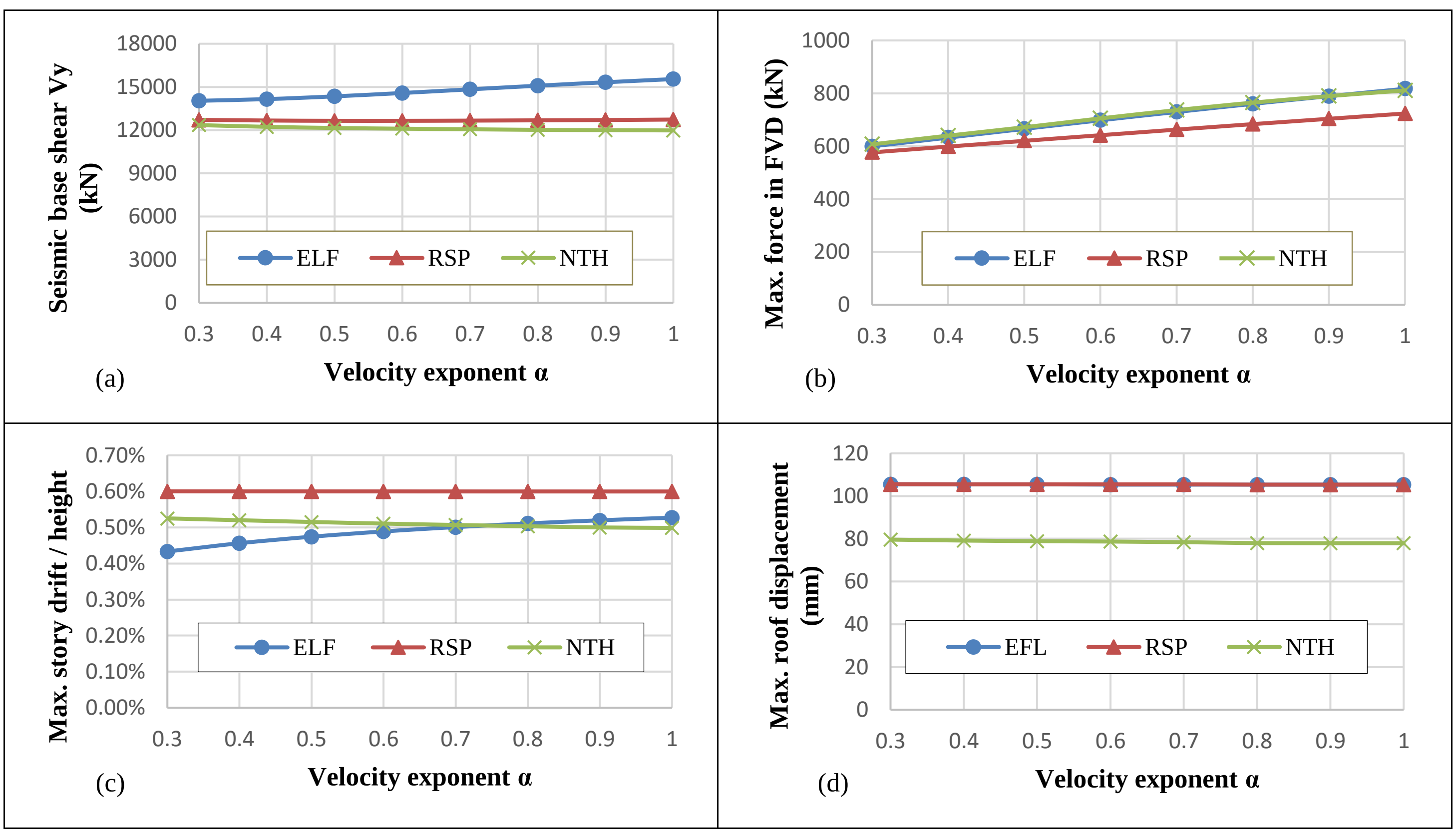


Fig. 5: Comparison of results of the simplified methods with nonlinear time history analysis.

Table 4 shows that with the addition of FVDs to the RC building studied, a lighter seismic force-resisting system was obtained while keeping the same story drifts and lateral displacements of the undamped reference structure. Figure 5 shows the comparison of structural responses of equivalent lateral force procedure (ELF), response spectrum procedure (RSP) and nonlinear time history analysis (NTH) with plastic hinges in the structure, this last analysis was performed in SAP2000. Conservative predictions were obtained for maximum roof displacements (34%), story drifts (18% on average for RSP) and base shears (5% for RSP and 22% for ELF, both on average). The ground motion employed for nonlinear time history analysis was the Ancash earthquake (1970) [9] which was spectrally matched to the design spectrum of E.030 standard using the SeismoMatch software. Fig. 5b shows that the maximum damper force diminishes as the velocity exponent α of the devices decreases and this is the expected trend because nonlinear FVD produce smaller forces than linear FVD while providing the same effective damping. Nevertheless, the simplified methods calculate damper forces based on the pseudo-velocities, that is, the velocities derived from the spectral displacements and in the literature it has already been shown that the approximation of relative velocity with the pseudo-velocity introduces an error [10]. The RSP underestimated the maximum forces of FVDs at the 2nd floor (up to -11%) in comparison with the results of NTH analyses.

Table 4: Results – Response spectrum procedure – Without accidental torsional loads – Displacements.

Analysis case	Elastic first mode period T	Roof displacement without dampers	Roof displacement with dampers	Max. story drift / height without dampers	Max. story drift / height with dampers
	(s)	(mm)	(mm)		
Reference	0.756	108		0.0062	
With FVD	1.014	148	105	0.0084	0.0060

4. Conclusion

Simplified procedures for analysis and design of buildings with energy dissipation devices were implemented using the seismic Peruvian E.030 standard. With these methods, for the RC building application performed in this work, were obtained in general conservative predictions of the inelastic seismic responses of the structure. Based on these results, it is concluded that the simplified methods should be included in a future Peruvian standard for structures with dampers.

Acknowledgements

Financial support for this work was provided by Pontificia Universidad Católica del Perú – Programa de Apoyo a la Investigación para Estudiantes de Posgrado – PAIP 2018.

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