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COMPARISON OF PERFORMANCE OF DAMPED AND CONVENTIONAL STRUCTURES IN ESSENTIAL BUILDINGS IN PERÚ

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Abstract

Peru is located in the Pacific Ring of Fire, which is one of the regions of higher seismic activity in the world, so its history of devastating earthquakes is large, as well as the loss of lives and property damage. Structural engineers can not be oblivious to these losses, which is why it is necessary to raise the challenge of incorporating new technologies such as seismic protection systems in new and existing buildings as a feasible alternative.

The concept of energy dissipation is a design strategy that can be used in new structures and seismic rehabilitation of existing structures. Therefore, researchers and engineers have shown interest in using energy dissipation devices as an efficient way to reduce the response of structures subjected to strong ground motions.

In this research a method for the analysis and design of buildings with passive energy dissipation is proposed following the guidelines set by the American Society of Civil Engineers (ASCE) and Federal Emergency Management Agency (FEMA). The proposed method performs a nonlinear time history analysis (NLTHA), based on the philosophy of performance design proposed by the Structural Engineers Associations of California (SEAOC) across the Vision 2000 Committee.

This method was applied to a building located in the city of Lima, Peru, which will be used as a Health facility. The building has a structural regular pattern in plan and elevation. It is a four-story reinforced concrete rigid frame structure, with stiffness below what is established by the Peruvian Code "*Reglamento Nacional de Edificaciones*" (RNE) [6] but with adequate resistance.

According to the recommended levels of spected preformance for buildings proposed by the SEAOC, the building should have a Operational and Life Safety performance under earthquakes of 475 and 970 years of return period. The targets drift associated with each damage state of according to the *Manual Multi-hazard Loss Estimation Methodology Earthquake Model* (HAZUS-MH) are 0.0035 and 0.006, respectively.

The building was design using the dynamic modal spectral analysis specified in the RNE. After that, a NLTHA was performed iteratively to the building with viscous fluid damper until reduced lateral drifts below the determined targets.

In order to compare the structural behavior and the costs of construction by incorporating the building energy devices, a conventional solution was developed, that consists in increasing the size of the columns and beams until the building reaches similar lateral drifts. Verified that the conventional solution is more expensive that building unconventional.

Keywords: Energy dissipation; earthquake protection; earthquake resistant design.

1 Introduction

Energy dissipation systems are devices designed to reduce deformation and design forces in structural elements (beams, columns, walls and others), thus preventing the energy from being dissipated through inelastic deformations.

In this research a method for the analysis and design of buildings with passive energy dissipation devices is proposed following the guidelines set by ASCE and FEMA. Such a method is presented by a series of structural dynamic analysis based on previous studies and steps. The developed method is applied in a four-story building located in the city of Lima, the building meets resistance but not stiffness code requirents. Finally performance and construction cost of the building with dampers and a conventional solution are compared.

2 Passive energy dissipation systems

According to ASCE7-10 and FEMA 274 specifications, passive energy dissipation systems can be classified as either displacement dependent or velocity dependent devices. Metallic and frictional damper are displacement dependent devices and viscous fluid and viscoelastic damper are velocity dependent devices [3].

The passive energy dissipation devices are connected to the structure with fixed mechanical properties that can not be controlled and generally do not depend on the seismic excitation and the response of the system, and does not require an external power source for operate, developing forces that opposes the motion [2].

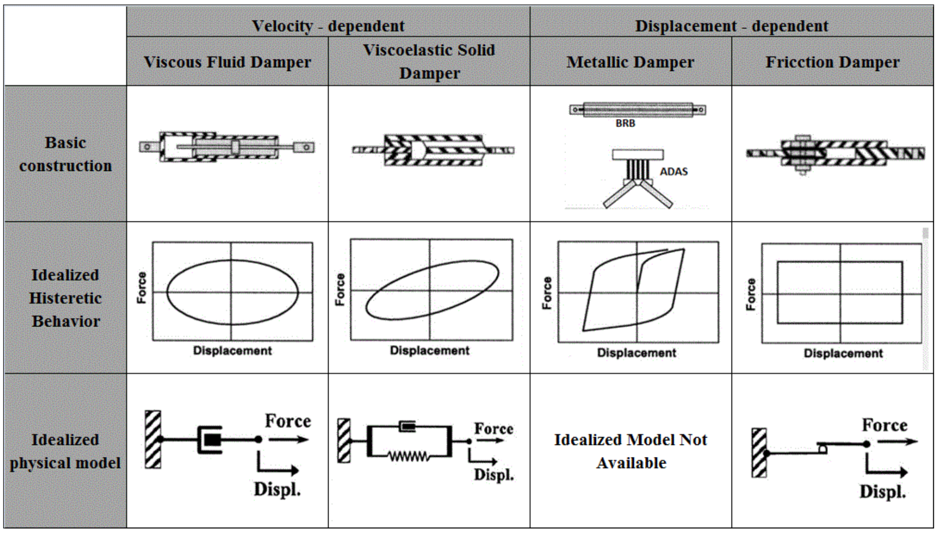


Fig. 1 - Sumary of constructions, hysteretic behaivor and physical model of passive energy dissipation divices. Adapted from [10].

2.1 Viscous Fluid Damper

The viscous fluid damper uses displacement of a piston through a viscous fluid, see Fig. 2 The friction between the piston and the liquid generates forces, opposing the movement of the piston, tha are proportional to the reaction rate. The viscous fluid may be oil, silicone, etc. [10].

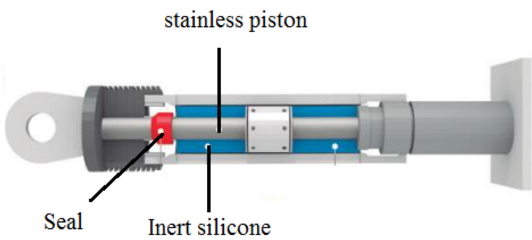


Fig. 2 - Viscous fluid damper type Taylor.

The force in the viscous fluid damper is estimated by:

$$f(d) = C_\alpha sgn(\dot{u})|\dot{u}|^\alpha \quad (1)$$

Where C_α is the damping coefficient, $\dot{u}$ is the damper velocity, sgn(.) is the funcion sign and α is the damping exponent that can have values between 0.2 and 1.0 for seismic aplications [9], [5] y [8]. The exponent α represents the nonlinearity of damper; if α=1 the damper is lineal and for intervals of $0 \leq \alpha < 1$ is nonlinear.

3 Performance design

The performance of a structure is determined by its response to an earthquake. This response is related to the damage to the structure and impact of these damages in post-event activities. This concept is not only applicable to buildings, but can be extended to all types of structures and even to nonstructural components and contents.

Over the years, there have been efforts to develop performance-based seismic design methods, many of them differ in notations and terminologies, but not in the basic concepts. In this study applied the proposal developed by the SEAOC, through the Vision 2000 Committee, in their book *Recommended Lateral Force Requirements and Commentary - Appendix G (1999).*

3.1 Choosing performance objectives

The SEAOC define the minimum acceptable performance for the new buildings. The basic objective for typical buildings as houses, essential/hazardous objective for essential buldings as hospitals and safety critical objectives for security critical buildings as nuclear plants. These three minimum objectives are illustrated in Fig. 3 as diagonal lines in the performance objective matrix [7].

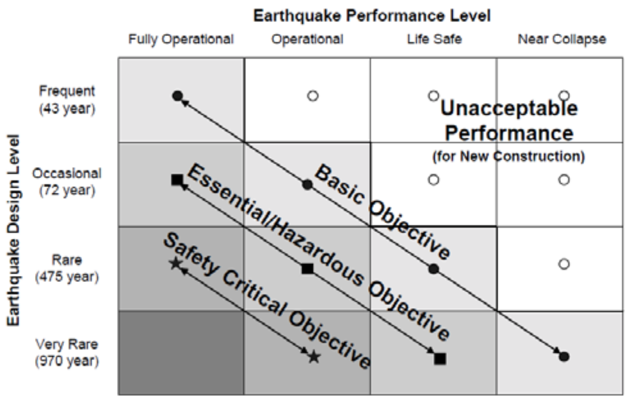


Fig. 3 - Recommended seismic performance objectives for buildings [7].

3.2 Performance levels

The manual Multihazard Loss Estimation Methodology (HAZUS-MH) define 36 types of buildings according to their structural system and subdivided into categories depending on the predominant material. The type C1M is a structure of concrete moment frame of mid rise (4-7 stories), the buildigs analysis in this research is clasified as C1M.

Also the manual HAZUS define four design level: Pre Code, Low, Moderate and High. The Pre Code design level is used for structures that are located in non-seismic zones, the Low design level applies for structures in areas of very low seismicity, the Moderate design level applies for structures in areas of moderate intensity and High design level is used for structures that are in high seismic hazard zones. The code used in Peru is similar to the moderate code level, because its description suits the demands of the current Peruvian standard.

The performance levels are expresed as the maximum degree of damage experienced in a building according to a specific design earthquake level; considering the conditions of the structural elements, nonstructural and contents [7].

In Fig.4 shows the comparison of damage description proposed by the Vision 2000 Committee and the manual HAZUS-MH for a building type C1M considering the moderate code level. When comparing of damage descripcion is obtained the relationship between performance levels and story drift.

	Vision 2000 Committee		HAZUS -MH	
Damage state	**Performance levels**	**Damage descripcion**	**Damage descripcion**	**Story Drift (target)**
Negligible	**Fully Operational**	No damage, continuous service. Continuous service, facility operates and functions after earthquake. Negligible structural and nonstructural damage.	-	-
Light	**Operational**	Most operations and functions can resume immediately. Repair is required to restore some nonessential services. Damage is light. Structure is safe for occupancy immediately after earthquake. Essential operations are protected, nonessential operations are disrupted.	It can be seen cracks cutting and bending near the junction of some beams and columns.	0.33%
Moderate	**Life Safe**	Damage is moderate. Selected building systems, features or contents may be protected from damage. Life safety is generally protected. Structure is damaged but remains stable. Falling hazards remain secure.	Several beams and columns cracks. Some elements such as beams have reached creep. Clearly the presncia bending cracks long and in some places the concrete coating has come off.	0.58%
Severe	**Near Collapse**	Structural collapse prevented. Nonstructural elements may fall. Structural damage is severe but collapse is prevented. Nonstructural elements fall.	Some frames have reached their ultimate capacity evident by the presence of long cracks bending loose concrete and transverse reinforcement and main deformed. Results from a partial collapse.	1.56%
Complete	**Collapse**	Portions of primary structural system collapse. Complete structural collapse.	The structure has collapsed or is about to do so because of the brittle failure, ductility exhaustion or loss of system stability.	4.00%

Fig. 4 - comparison of damage description proposed by the Vision 2000 Committee and the manual HAZUS.

4 Design methodology

4.1 Definition of the structural system to be reinforced with dampers.

The structures with energy dissipation devices must have an earthquake resistant system, independent of the dissipation system [7]. In case of Peru the system will be formed according to specified the RNE. This system must have the strength required to withstand the forces defined in section 4.2, but do not necessarily meeting the drift limits requirements of the Peruvian code.

4.2 Calculation of the base shear.

The seismic base shear used for the design of earthquake resistant structure should not be less than Vmin, where Vmin is determined as the highest of the values obtained from Eq. (2) and Eq. (3) [7].

$$V_{min.} = \frac{V}{B_1} \tag{2}$$

$$V_{min.} = 0.75V \tag{3}$$

Where:

V: The seismic base shear structure without damper system, calculated as specified in the RNE.

B1: Damping factor given in Table 1 for a total equivalent viscous damping equal to βm and a period of vibration equal to T1, with m = 1 (fundamental mode).

Table 1 – Damping factor (ASCE-7, 2010).

Effective Damping, β (percentage of critical)	Damping Coefficient (B1)
≤2	0.8
5	1.0
10	1.2
20	1.5
30	1.8
40	2.1
50	2.4
60	2.7
70	3.0
80	3.3
90	3.6
≥100	4.0

4.3 Election of performance and story drift objective.

It is necesary to define the level preformance and story drift objective for the structure to be achieved for the corresponding seismic demands, as discussed in Chapter 3

4.4 Nonlinear time history analysis (NLTHA).

The method of nonlinear dynamic analysis can be used for the design of all structures especialy those with passive energy dissipation systems [2]. This analysis is a sophisticated method that incorporates directly the nonlinear properties of all elements of the seismic system, computing the inelastic demand of its components at any time of a given seismic record by iterative methods integration that update numerical stiffness and damping properties of the elements depending on the strain amplitude, frequency and duration, among others.

The nonlinear dynamic analysis is the best alternative to carry out the design process in systems that are expected to respond in the inelastic range and / or have irregularities either in plan or in elevation.

For each record considered in the nonlinear analysis the maximum value of the internal efforts of the elements of the dissipation system and the maximum value of the interstory drift of the structure has to be computed. In the case of viscous damper, it must also has to be determine the maximum deformation and maximum deformation rate between the ends of each of the devices. For deformation dependent devices, it should be determined only the first of these variables.

Internal stresses, strain and strain rates of each of the elements of the damper system and the interstory drift can be obtained as the average of the maximum values of each seismic excitation, considering at least seven sets of records, each one consisting of a pair of horizontal and perpendicular acceleration. Otherwise the design values should be calculated as the maximum value obtained records[2].

4.5 Parameters of design.

The earthquake resistant structure must meet the strength requirements of RNE using any design analysis described in the code and a seismic base shear, Vmin, indicated in Eq. (2) and Eq. (3).

The earthquake resistant structure should be designed considering the minimum value Vmin, excluding the damper system of mathematical model, which ought to be conservative compared to what indicated in Eq.(2)

and Eq. (3).

The dampers and other elements of the dissipation system must be sized to withstand the forces, displacements and velocities from the nonlinear response analysis considering the maximum possible earthquake [2]. The maximum possible earthquake will be represented by an earthquake with a return period of 975 years.

The effects of gravity and seismic loads acting on the structure and the damper system must be combined as established in RNE.

5 Example

In order to compare the performance of damped and conventional building three types of estructures are defined:

Basic structural system (SEB): This structural system consists of an earthquake-resistant structure as specified in RNE. This structural system may not necessarily meet the requirements of strength and displacement specified in RNE.

Structural system with damper (SED): This structural system is composed of the SEB plus energy dissipation devices.

Conventional structural system (SEC): This structural system is similar to SEB, but structural elements are resized to meet the requirements of strength and displacement specified in RNE.

5.1 Descriptions

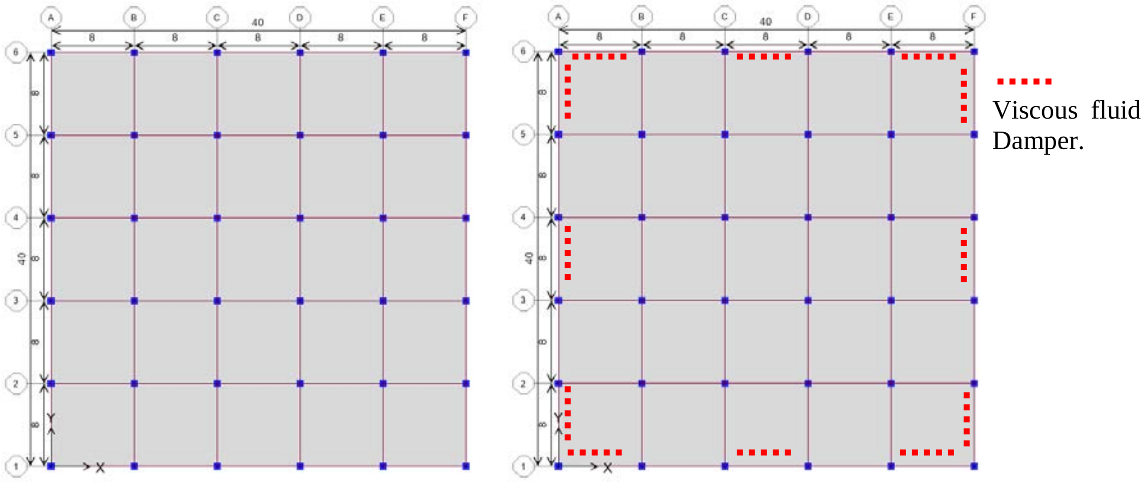
A 4-story building structured of reinforced concrete frames is consider; this building is supposed to be used as a hospital , therefore, it has to be consider as an essential building according to the Peruvian code. The structure is regular both in plan and in elevation and is located in the city of Lima on a stiff soil, which according to RNE is classified as type S1 (equivalent to a B soil type). In plan, the building is has a square shape of 40m x 40m and has five 8m long bays in both directions, as shown in Fig. 5. The floor slabs are 0.20m width, and are consider to act as rigid diaphragms. The story height is 4.5m for all the building.

The SEB has square columns of 0.6m, and rectangular beams with 0.5m width and 0.70m depth.

The SED has the same basic structure of SEB plus system damper (viscous fluid damper).

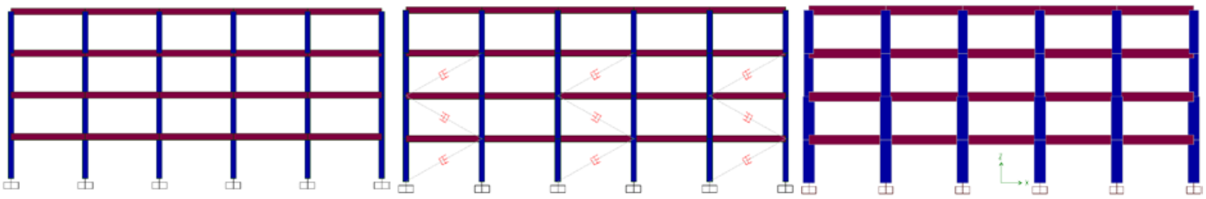
The SEC has square columns of 1.20m in the first and second floor, and 1.0m in the remaining floors, and rectangular beams with 0.5m width and 1.0m depth.

Fig. 5 and Fig. 6 shows plans and elevations of SEB, SEC and SED.



(a) (b)

Fig. 5 – (a) Typical plan of SEB and SEC. (b) Typical plant of SED.



(a) (b) (c)

Fig. 6 – (a) y (c) Typical elevations of SEB y SEC.

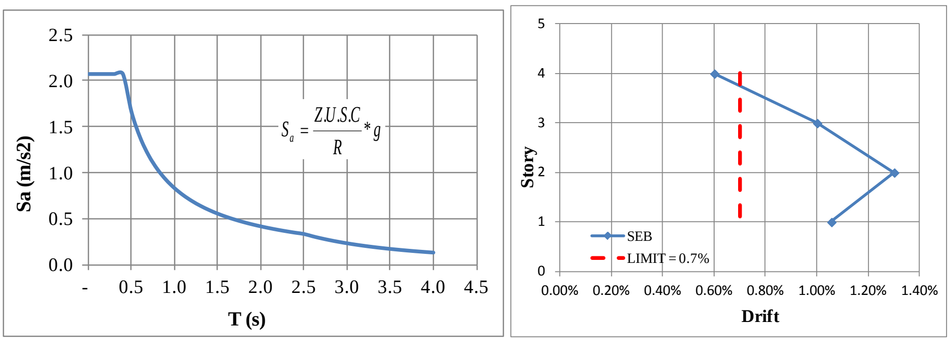
(b) Elevations of axis 1, 6, A and F of SED.

Permanent loads used in the analysis where: Self weight, plaster (1.0 kN / m2), partition walls (1.5 kN / m2) and live loads as specified by the RNE (4.0 kN / m2).

The mechanical properties of the concrete used are: Compressive strength (fc = 21MPa), Modulus of elasticity (Ec = 21737 MPa) Specific gravity (γ = 240 Mpa) and elastic modulus (η = 0.2). Rebar in columns and beams are made of corrugated steel ASTM A615-Grade 60.

5.2 SEB Structural Analysis and Design

The design of SEB was based on a modal analysis as indicated in RNE. Fig 7 (a) shows the design spectrum obtained using Factor zone (Z = 0.45), a coefficient of use (U = 1.5), a soil amplification factor (S = 1), which are values according to the location and future use of the building; the reduction coefficient of seismic force (R = 8) is related to the expected ductity (non linear response) of the building and has to be determined by the designer. In the case of the Peruvian code, reference values are proposed according to the structure type. Other parameters are the seismic amplification factor (C = 0.91) and gravity (g = 9.81m / s2). Fig. 7 (b) shows the interstory drift of the building and the limit indicate in the RNE (0.7%). It is clearly seen that in all the stories but the last one, there is a lack or stiffness in the structure.



(a) (b)

Fig. 7 – (a) Reduced inelastic spectrum of pseudo-acceleration, according RNE.

(b) Story drift

The design of all the structural elements (beams and columns) was made following the considerations established in RNE and ACI 318S-14 [1].

5.3 Nonlinear time history analysis (NLTHA) of SEB, SEC and SED.

Nonlinear analysis is made considering that the plastic behaviour is concentrated at the ends of the structural elements (plastic hinges of beams and columns). This kind of analysis can be made in many of the available software for structural design.

For this analysis three sets of records were used. Each set of registers consists of a pair of horizontal acceleration components, as shown in Table 5.7. These records were selected and scaled to a PGA of 0.45g and 0.55g for the Design Earthquake (SD) and Maximum Probable Earthquake (SMP) respectively, according to seismic hazard studies conducted by the Geophysical Institute of Peru (IGP-2014), for the city of Lima.

Table 2 – Earthquakes considered for the analysis.

Station	Date	Componet	Code	Epicenter			PGA		Magnitud (Mw)
				Latitude -	Longitude	Depth (km)	(cm/s2)	(g)	
Parque de la Reserva	17-oct-66	N82W N08E	**7035** **7036**	-10.7	-78.7	24	180 270	0.18 0.27	7.5
Parque de la Reserva	31-may-70	N82W N08E	**7038** **7039**	-9.4	-78.9	64	105 98	0.11 0.10	7.9
Parque de la Reserva	03-oct-74	N08E N82W	**7050** **7051**	-12.5	-78	13	179 193	0.18 0.20	8.1

In Fig. 8 shows pseudo-acceleration spectra obtained for the records used in the analysis, scaled to a PGA of 0.45g considering a 5% of damping.

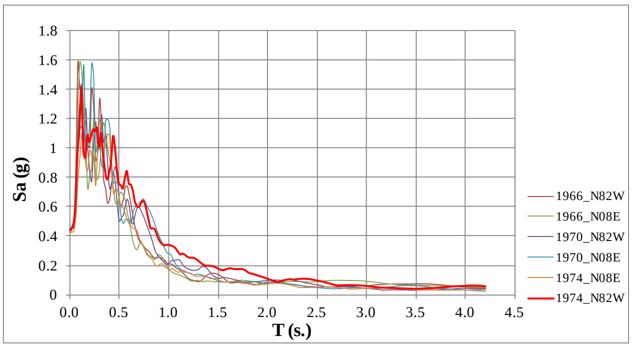


Fig. 8 – Spectral acceleration for a damping of 5%.

For the building in study, if an Operational Performance for the SD, and a Protection of Life performance for a SMP are to be achieved, story drift should not be greater than 0.33% and 0.58% respectively, as shown in Fig. 4. These values are rounded to 0.35% and 0.55%.

To optimize the amount of viscous fluid devices installed in the SED an iterative analysis is conducted changing its configuration and location as well as its parameters such as the damping coefficient (C), and the coefficient of nonlinearity (α). The final configuration and location of the divices was shown in Fig. 6 (b) and parameters were determined to be C = 3000 KN-s / m, α = 0.6 and K = 295000 KN / m.

5.4 Results of SEB

Fig. 9 shows that story drift limit is exceeded in almost all records. The 1974 earthquake record shows the maximum value, which is 0.61%. It is clear that SEB does not meet the drift limits established in the code.

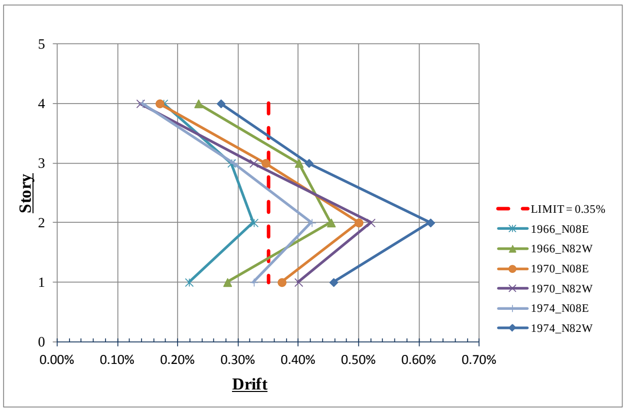
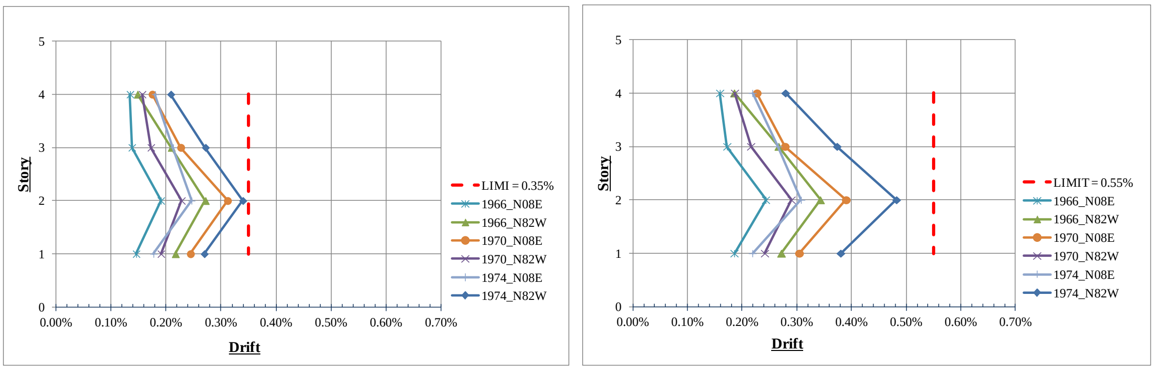


Fig. 9 – Maximum story drift corrisponding to the records of 1966, 1970 y 1974, escaled to PGA of 0.45g.

5.5 Results of SED

Fig 10 (a) shows that story drift are fairly below the distortion targer (0.35%) in all records scaled to a PGA of 0.45g. The building reaches Operational Performance for the SD. Fig 10 (b) shows that the story drift for records scaled to a PGA of 0.55g, are clearly below the distortion target (0.55%). It is concluded that the building reaches a Life Safe Performance for the SMP.



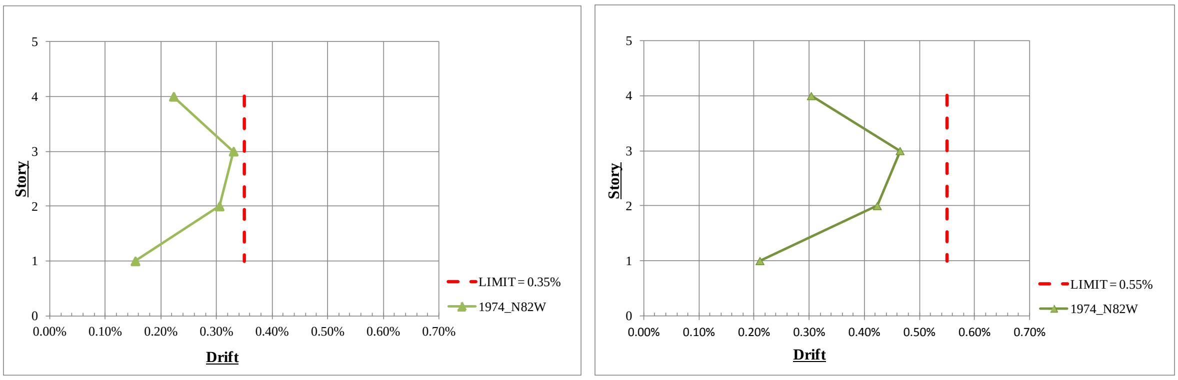
(a) (b)

Fig. 10 - (a) and (b) Maximum story drift corresponding to the records of 1966, 1970 and 1974, scaled to a PGA of 0.45g and 0.55g respectively.

5.6 Results of SEC

The SEC is analyzed only for the record 1974-N82W, because the maximum results are obtained for this record. Fig. 11 (a) shows that the maximum story drift obtained for the record scaled to PGA=0.45g is lower than the story drift target (0.35%). It is concluded that the building reaches operational performance for SD.

Fig. 11 (b) shows that the maximum story drift obtained for recording 1974-N82W scaled to PGA=0.55g is lower than the story drift target (0.55%). It is concluded that the building reaches live safe performance for SMP.



(a) (b)

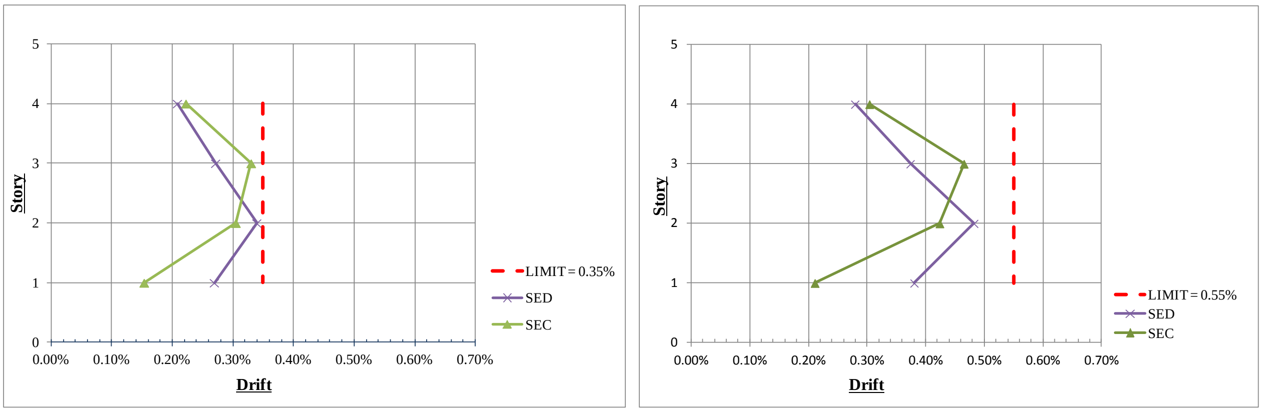
Fig. 11 - (a) and (b) Maximum story drift corresponding to the records of 1966, 1970 and 1974, scaled to a PGA of 0.45g and 0.55g respectively.

6 Comparison of results

To compare the performance achieved by the proposed structural systems (SED and SEC), the results of ANLTH is analyzed using the 974-N82W scaled to a PGA of 0.45g, which represents the design earthquake.

Fig. 12 indicates that both SEC and SED structures have story drifts below 0.35% and 0.55% for SD and SMP respectively. Therefore, the SEC and SED satisfactorily meet the expected performance.

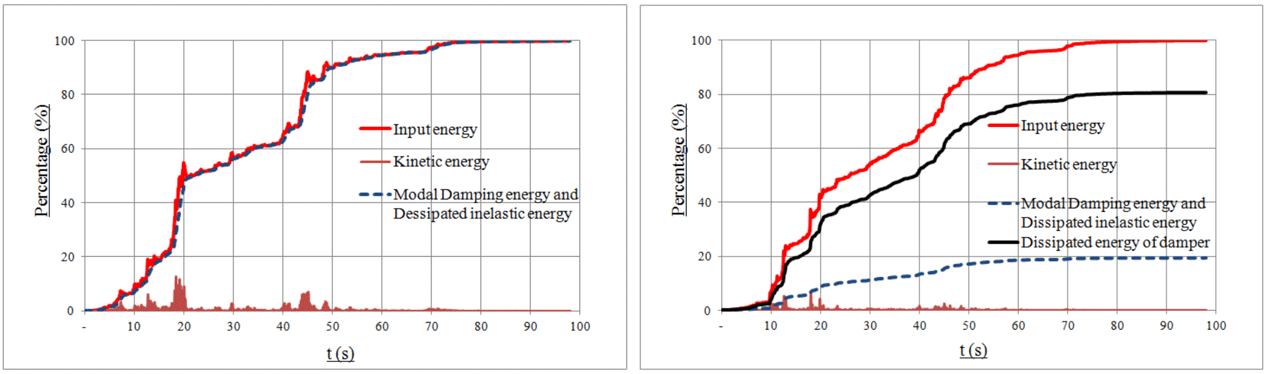
Both structural systems can be a solution (SED and SEC), meeting the requirements specified deformation and resistance in RNE and also meeting the performance objectives proposed in this study. Under these considerations both systems are structurally comparable.



(a) (b)

Fig. 12 – (a) and (b) maximum story drift of SED and SEC, obtained of NLTHA using the records of 1974-N82W escaled to a PGA of 0.45g and 0.55g.

The earthquake input energy of the structure without damper is dissipated by the inelastic behaviour of columns and beams and by internal damping of the structure with dampers that deformation response and damage of structure can be reduced. In the Fig. 13a it show that the structure without damper (SEC) about 100% the input energy be dissipated by the beam and column members for tha inelastic and elastic deformation. In the Fig. 13b it show that the structure with damper (SED) about 80% the input total energy were dissipated by dampers and about 20% the input energy be dissipated by the beam and column members for the elastic and inelastic deformation.



(a) SEC (b) SED

Fig. 13 – (a) y (b) Energy dissipation of structure with and without damper, obtained for the record of 1974-N82W escaled to a PGA of 0.45g

The total cost of implementing the proposed reinforce altenatives are presented in Fig. 14. It can be seen that the total dirct cost of SED is less than the total direct cost of the SEC. Cost per square meter of the SED is $ 177.0 / m2 while for the SEC is $ 181.73 / m2. For this particular case the use of energy-dissipating devices turned out to be the best economic alternative.

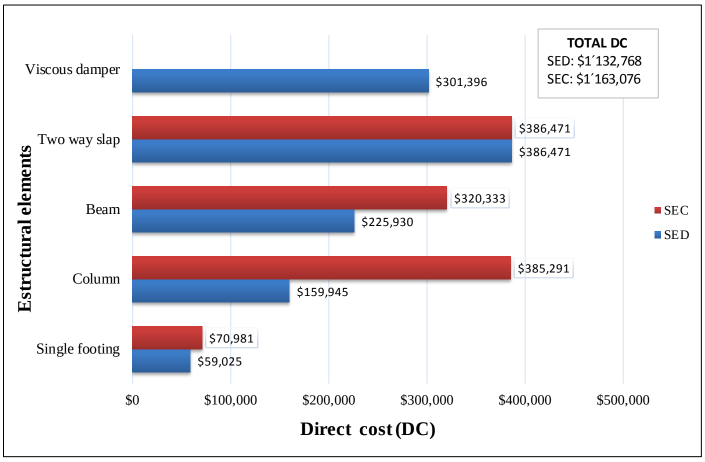


Fig. 14 – Comparison of the direct cost of construction for each type of structural element of SEC and SED.

7 Conclusion

The developed method is exemplified by the design of a building of reinforced concrete, proposing two solutions a conventional solution (SEC) and other unconventional (SED) with both solutions reaching Operational Performance and Life Safety Performance for the design earthquake (PGA = 0.45g) and the maximum possible earthquake (PGA = 0.55g) respectively.

The SEC is a conventional solution commonly adopted by engineers to meet the requirements of resistance and displacement. This solution is to increase stiffness by having more robust structural elements. This leads to a reduction in the usable space, which in the case of some projects may not be acceptable.

It has been verified that it is cheaper to implement this building with energy dissipative viscous fluid type than increasing sizes of columns and beams. The direct cost of the structural system of the building with energy dissipation devices is $ 177.0 per m2 and the building with conventional reinforcement is $ 181.73 per m2, achieving a savings of $ 4.73 per m2.

8 Reference

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