Evaluation of the Seismic Performance of Structures with Energy Dissipators for the city of Quito in a Seismic Scenario

Cagua, Brian [1,2,4*]; **Aguiar, Roberto** [3,4]

[1] *Universidad de los Andes, Doctorando en Ciencias de la Ingeniería, Santiago, Chile*
[2] *Universidad de las Fuerzas Armadas ESPE, Programa de Maestría de Investigación en Ingeniería Civil, Sangolquí, Ecuador*
[3] *Universidad de las Fuerzas Armadas ESPE, Departamento de Ciencias de la Tierra y la Construcción, Sangolquí, Ecuador*
[4] *Red de Investigación CEINCI LAB, REG-RED-22-0177, Quito, Ecuador*

Abstract: In Ecuador, the seismic-resistant design is based on design philosophy by capacity and energy dissipation mechanisms based on damage. Consequently, after a severe seismic event, significant damage is evident in structural elements that require intervention, which sometimes is not viable and must be discarded. For this reason, the current trend is to design structures that allow functionality and contribute to the resilience of cities. This research article analyzes reinforced concrete and steel frames for 4, 8, and 12 floors with and without TADAS, SLB, and BRB-type energy dissipators. Subsequently, the capacity of the structures is determined by using a non-linear static Pushover type analysis. In addition, a non-linear dynamic analysis of the systems subjected to eleven accelerograms is performed, scaled according to a Quito seismic scenario for intensities of design for an earthquake $TR = 475$ years and a maximum considered at $TR = 2475$ years. Finally, seismic performance is evaluated using an ASCE methodology, and dynamic incremental analyzes are generated to build fragility curves.

Keywords: Performance, structures, steel, reinforced concrete, energy dissipators, NEC-15

Evaluación del Desempeño Sísmico de Estructuras con Disipadores de Energía para el Escenario Sísmico de Quito

Resumen: En Ecuador, el diseño sismo resistente se fundamenta en la filosofía de diseño por capacidad y mediante mecanismos de disipación de energía basados en daño. En consecuencia, después de un evento sísmico severo se evidencia afectaciones importantes en elementos estructurales que requieren de intervenciones, que en algunos casos no son viables y se requiere derrocar. Por esta razón, la tendencia actual es diseñar estructuras que permitan funcionalidad y que contribuyan a la resiliencia de las ciudades. En este trabajo, se presenta el análisis de pórticos de hormigón armado y acero, de 4, 8 y 12 pisos sin y con disipadores de energía tipo TADAS, SLB y BRB. Posteriormente, se determina la capacidad de las estructuras mediante análisis estáticos no lineales tipo Pushover. Además, se realiza análisis dinámicos no lineales de las estructuras sometidas a once acelerogramas, escalados para el escenario sísmico de Quito para las intensidades del sismo de diseño $TR = 475$ años y el máximo considerado $TR = 2475$ años. Finalmente, se evalúa el desempeño sísmico mediante la metodología del ASCE y se generan análisis incrementales dinámicos para construir curvas de fragilidad.

Palabras claves: Desempeño, estructuras, acero, hormigón armado, disipadores de energía, NEC-15

1. INTRODUCTION

Ecuador is a country located in the Pacific Ring of Fire. For this reason, earthquakes constitute one of the leading causes of structural destruction. They generate damage and, in some cases, collapse, as evidenced in the past, as evidenced by the Manabí earthquake in 2016 (Vanegas, 2018). In addition to the subduction of the Nazca and South American plates, there is a system of local faults that can generate significant earthquakes at shallow depths (Parra, 2016). This is Quito's seismic scenario, where earthquakes generated by local faults can cause significant damage to structures and their occupants.

Some structural damage is tolerated when designing against earthquakes. The methodology of the Ecuadorian Construction Standards, in its chapter on seismic resistant design, states that structures should have non-damage behavior from frequent earthquakes. These should maintain the integrity of the systems and aim to avoid the collapse of said structures for earthquakes of greater intensity (MIDUVI, 2015d).

bjcagua@miuandes.cl, bjcagua@espe.edu.ec
Recibido: 15/03/2023
Aceptado: 26/11/2024
Publicado en línea: 28/02/2025
10.33333/rp.vol55n1.09

To create resilient cities, the quandary of damage accepted in the regulations and tolerated by society must be addressed. The global trend emphasizes continuous functionality, especially for essential structures or those critical to the community. In conventional designs, the aim is to minimize damage. According to Soong and Spencer (2002), it is necessary to find energy dissipation, base isolation, and active control techniques to prevent extensive damage to the structure and lower repair costs. Energy dissipation devices such as TADAS (Triangular plate added damping and stiffness), SLB (Shear Link Bozzo), or Restricted Buckling Bars (RBB) are intended to resist the largest portion of seismic forces. These also concentrate damage onto these devices to protect structural elements such as beams or columns. In addition, these systems make it possible to rigidize structures and increase damping; consequently, the response in displacements and accelerations can be reduced.

Another aspect in which the advantage of using energy dissipators is evident is that the seismicity in Ecuador is not yet fully known, and the uncertainty in seismic hazard studies is great. It is, therefore, possible to have a higher seismic energy than expected by the designers. In short, more significant damage could be manifested in conventional structures if the intensity exceeds the design intensity. However, by including energy dissipators, excess energy can be resisted through damage to the dissipators, thus safeguarding the integrity of the structures and protecting their occupants.

This paper presents the design of 4-, 8- and 12-story steel and reinforced concrete portal frames with and without energy dissipaters, considering TADAS, SLB, and RBB devices. Non-linear behavior under static and dynamic loads is studied, structural performance is evaluated, and fragility curves are determined. Damage states corresponding to mild, moderate, severe, and complete are considered. In addition, the design of the structures is verified by considering their structural performance.

2. STRUCTURE DESIGN

Four-, eight-, and twelve-story portal frames with three spans are analyzed and designed. The TADAS and SLB dissipaters are placed on steel diagonals; these elements are located in the central span, similar to the RBB case, see Figure 1.

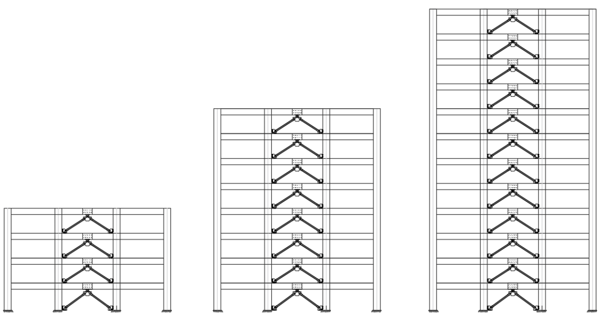


Figure 1. Types of frames under study

The total number of structures analyzed is 24.

The analysis and design of the structures is carried out applying national regulatory guidelines (MIDUVI, 2015b, 2015c, 2015d), and their corresponding international standards (American Concrete Institute, 2019; American Institute of Steel Construction, 2016a, 2016b, 2016c; American Society of Civil Engineers, 2016). The loads are determined by the chapter on Non- seismic Loads (MIDUVI, 2015a). Dead load for slabs is considered at 500 $\frac{kg_f}{m^2}$ and 380 $\frac{kg_f}{m^2}$ for the roof. Furthermore, office occupancy is assumed at a live load of 250 $\frac{kg_f}{m^2}$ and 100 $\frac{kg_f}{m^2}$ for the roof. To define the design earthquake, the spectrum for the city of Quito is used considering type D soil and a seismic response reduction factor of 6. The reactive seismic load is assumed at 100% of the dead load and 15% of the live load (Cagua et al., 2022).

In structures with TADAS dissipators, 20% damping is assumed; for structures with SLB, 15%, and other structures are modeled with 5%. For damping greater than 5%, the design spectrum is divided by value B, which is the function of the damping factor of the structure with dissipators ζ, as indicated in Equation (1).This approach to modifying the design spectrum is proposed in the methodology utilized by Aguiar et al. (2017).

$$B = \left(\frac{\xi}{0.05}\right)^{0.3} \tag{1}$$

The structures are analyzed and designed using "CEINCI LAB Computing System functions" (Aguiar & Cagua, 2022), developed by said authors using the rigidity method. For the design, the linear properties of the structure are considered. However, when analyzing the dissipator, non-linear properties are operated through a bilinear model, and secant stiffness is taken into consideration. This is determined with an iterative analysis (Aguiar et al., 2016).

The models with TADAS and SLB dissipators contemplate these as a short vertical element (Aguiar et al., 2019). For the seismic case, the degrees of freedom consider a rigid floor, and therefore the horizontal degree of freedom is unique in each level; in addition, the dissipators are considered axially rigid in the vertical sense. For the static analysis, 3 degrees of freedom per node are used due to being two-dimensional models.

The sections of the structural elements, beams, columns, diagonals, and steel core of the RBB, taken into account in the models, are presented in Table 1.

The reinforcement of reinforced concrete beams and columns meets the regulatory requirements, and 20mm and 25mm rods are used.

TADAS dissipators with 6 triangular plates are considered. These plates have a geometry of 17cm in base and height, and a thickness of 2.5cm is also assumed. The SLBs used have the properties of an $SLB3\_25\_2$ device as indicated by the manufacturer (Bozzo, 2022).

Table 1. Sections of structural elements

	Floor	Steel BEAM IPE	Steel COL HBE	RC BEAM $(cm \times cm)$	RC COL $(cm \times cm)$	DIAG Bxt (mm)	RBB Asc (cm^2)
4-story	1	450	400	35x50	45x45	150x10	25
	2	450	400	35x50	45x45	150x10	25
	3	450	400	35x50	45x45	100x10	20
	4	450	400	35x50	45x45	100x10	20
8-story	1	500	500	35x60	60x60	150x10	50
	2	500	500	35x60	60x60	150x10	50
	3	500	500	35x60	60x60	150x10	50
	4	500	500	35x60	60x60	150x10	50
	5	450	400	35x50	45x45	100x10	40
	6	450	400	35x50	45x45	100x10	40
	7	450	400	35x50	45x45	100x10	40
	8	450	400	35x50	45x45	100x10	40
12-story	1	500	650	35x60	60x60	150x10	62.5
	2	500	650	35x60	60x60	150x10	62.5
	3	500	650	35x60	60x60	150x10	62.5
	4	500	650	35x60	60x60	150x10	62.5
	5	500	650	35x60	60x60	150x10	62.5
	6	500	650	35x60	60x60	150x10	62.5
	7	450	400	35x50	45x45	100x10	50
	8	450	400	35x50	45x45	100x10	50
	9	450	400	35x50	45x45	100x10	50
	10	450	400	35x50	45x45	100x10	50
	11	450	400	35x50	45x45	100x10	50
	12	450	400	35x50	45x45	100x10	50

In all cases, compliance with a maximum inelastic drift of 2% is verified, which is the permissible limit of the MIDUVI (2015d). An equivalent spectral and static modal analysis is performed. Figure 2 shows the result of the seismic analysis for a 4-story steel structure without including dissipators (Cagua et al., 2021).

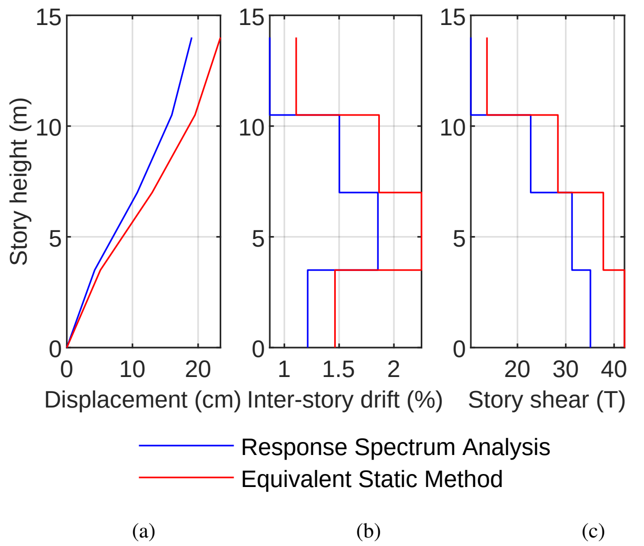


Figure 2. Results - 4 floors of steel: a) Inelastic displacement; b) Inelastic drift; c) Story shear

In steel structures, it is verified that the axial capacity of the columns is greater than the demand before the effects envelope, including the seismic over resistance factor, as indicated in ASCE 7. It is also shown that the demand relationship versus axial bending and shear capacity is less than 1 for all structural elements. These results can be corroborated graphically in the generated programs, as indicated in Figure 3.

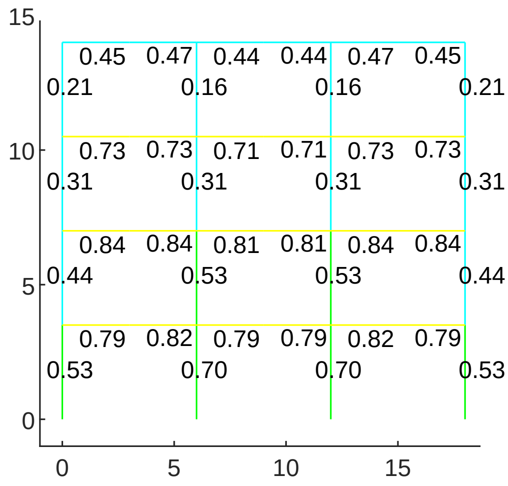


Figure 3. Results of Demand vs. Flexo-Axial Capacity for 4 floors in steel

In reinforced concrete frames, the analysis and design processes are similar. Still, the reinforcement of the elements is taken into consideration for verification. This reinforcement is evaluated at the beginning, middle, and end of each component. The demand vs. capacity relationship in the beam is considered for upper and lower steel due to the moment envelope. An interaction diagram is used for columns. In all cases, "Strong Column and Weak Beam" is checked to verify the connection.

Designs with similar inelastic drift responses are performed to simulate comparable steel and reinforced concrete structures. For example, in systems without the inclusion of dissipators, there are drifts of 2%. In the case of a dissipator, these drifts are close to 1%.

3. SEISMIC HAZARD

A seismic hazard analysis is performed for the site of interest, where the structures will be implemented, using the CRISIS program (Ordaz et al., 2014). The ground motion prediction models are incorporated into the calculation using a logic tree composed by Akkar et al. (2014), Chiou and Youngs (2014), and Zhao et al. (2006), all with a weight equal to 33%. The source zones of Beauval et al. (2018) are considered.

From the disaggregation of a seismic hazard, it is determined that for a design earthquake (EQ),i.e., with $Tr = 475$ years, the earthquakes contributing the most danger correspond to events with magnitudes of 6.25 on faults with focal distances of up to 40 km. In this case, eleven accelerograms are chosen from the "PEER Ground Motion Database" (n.d.), corresponding to this seismic scenario, as shown in Table 2.

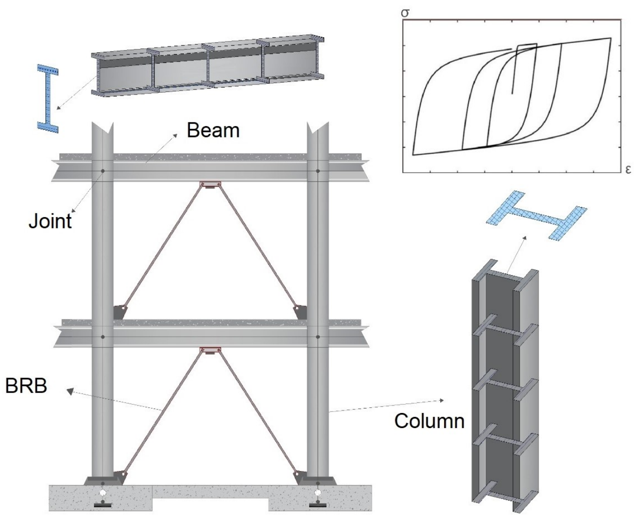
Earthquake scaling is performed using the weighted factors procedure. Amplitude scaling aims to reduce the error, defined as the sum of the squares of the difference between the spectral acceleration of the scaled component and the target spectral acceleration in a range of periods. This range for the analysis structures corresponds to $0.2T$ to $1.50T$.

Table 2. Selected earthquakes

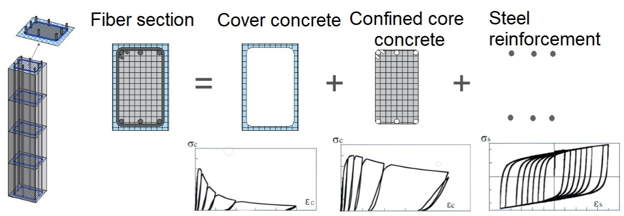
EQ	Event	Year	Mag	Mechanism	$R_{rup}(km)$
1	Loma Prieta	1989	6.93	Reverse Oblique	20.34
2	Landers	1992	7.28	Strike Slip	11.03
3	Manjil, Iran	1990	7.37	Strike Slip	12.55
4	Gazli, USSR	1976	6.80	Reverse	5.46
5	Parkfield-02, CA	2004	6.00	Strike Slip	3.01
6	Northridge-01	1994	6.69	Reverse	7.46
7	Christchurch	2011	6.2	Reverse Oblique	5.55
8	Chalfant Valley-02	1986	6.19	Strike Slip	7.58
9	Westmorland	1981	5.90	Strike Slip	6.5
10	San Salvador	1986	5.80	Strike Slip	6.99
11	Mammoth Lakes-06	1980	5.94	Strike Slip	16.03

4. NON-LINEAR BEHAVIOR AND SEISMIC PERFORMANCE EVALUATION

Non-linear analyses are carried out in OpenSees (Zhu et al., 2018), ForceBeamColumn models are considered for columns and beams, "Two Node Link" type models are used for the dissipators. Truss models are used for the diagonals. "Steel02" and concrete, confined and unconfined, are used with "Concrete02". These models are shown in Figure 4.



(a)



(b)

Figure 4. Distributed plasticity models: a) Steel; b) Reinforced concrete

The capacity curve of the structures is determined by non-linear static analysis. Figure 5 shows the investigation results. It can be seen that the curves for both steel structures (SS) and reinforced concrete frames (RC) are similar when considering the same typology (floor number and type of dissipator).

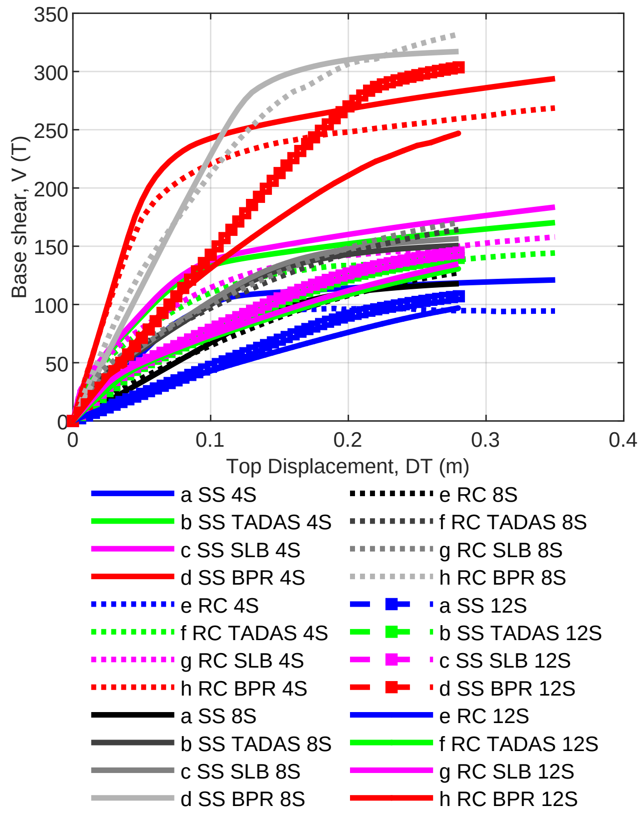


Figure 5. Structure capacity curves

Objective target displacement δ_t for an intensity level defined by *Sa* is estimated by using ASCE 7 methodology, where coefficients C_0, C_1, and C_2 are used to modify the response of a one-degree-of-freedom system (considered with an effective fundamental T_e period); this is summarized in Equation (2).

$$\delta_t = C_0 \cdot C_1 \cdot C_2 \cdot S_a \cdot \frac{T_e^2}{4\pi^2} \cdot g \qquad (2)$$

On the other hand, effective yield displacement $\delta_{y_{eff}}$ is estimated using *Quantification of Building Seismic Performance Factors (FEMA-695)* (2009), expressed in Equation (3). This displacement corresponds to the beginning of the non-linear behavior of the structure, i.e., the first plastic hinge is formed.

$$\delta_{y_{eff}} = \frac{\Gamma_1 \cdot V_{max}}{W} \cdot \frac{g}{4\pi^2} \cdot T_1^2 \qquad (3)$$

For this equation, Γ_1 is the elastic modal participation factor of the first vibration mode. T_1 is the period of the fundamental vibration mode; V_{max} is the maximum shear of the capacity curve; W is the total reactive load.

For the non-linear dynamic analyses, simplified models of one degree of freedom are used, with a response based on a bilinear model. These simulations are calibrated with Pushover capacity curves of the structures, and the damping assumed in each type of analysis. The models do not consider strain hardening (the post-yield line has zero slopes) or the effect of crack closure. They do not contemplate deterioration of rigidity and resistance, either. However, they are applicable because the structures are two-dimensional frames with a response controlled by the first vibration mode.

The performance limit states are defined by the displacement on the roof. The fully occupational limit (O) is considered the effective yield displacement. The collapse limit (C) to the ultimate displacement, for functional (F) 30% is assumed, and for life safety (LS) 60%. In collapse prevention (CP), 80% of inelastic displacement Δ_P is measured from yield displacement (SEAOC, 1995). Figure 6 shows a capacity curve of the structure, identifying the performance limits and the maximum displacements in the roof for the non-linear dynamic response of each earthquake (EQ) scaled to $TR = 475$ years.

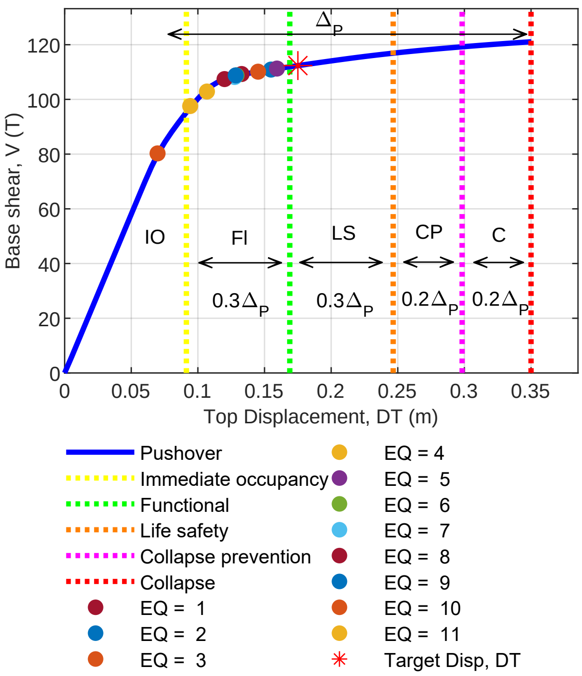


Figure 6. Pushover Curve with performance limits results for design earthquake

The performances of the structures are presented in Table 3, evaluated with the ASCE 7 methodology for the design and maximum earthquake considered. The performances evaluated with the responses of the non-linear dynamic analyzes estimate a level of damage lower than those presented in Table 3 by the ASCE 7 methodology. Dynamic incremental analyses are carried out to obtain the fragility curves for each structural typology (see Figure 7). Four levels of damage are defined, correlated to the described performance levels, corresponding to mild (occupational), moderate (functional), severe (life safety), and complete (collapse prevention). A compendium of the individual results for each structure and of all the analyses is presented in Cagua (2022).

Table 3. Performance using ASCE 7

TYPE	DESCRIPTION	FLOOR	PERFORMANCE $TR = 475$ years	PERFORMANCE $TR = 2475$ years
Steel SMF	a_4_SS	4	LS	CP
	a_8_SS	8	LS	CP
	a_12_SS	12	LS	CP
Steel TADAS	b_4_SS_TADAS	4	O	F
	b_8_SS_TADAS	8	F	LS
	b_12_SS_TADAS	12	F	LS
Steel SLB	c_4_SS_SLB	4	O	F
	c_8_SS_SLB	8	F	LS
	c_12_SS_SLB	12	F	LS
Steel BRB	d_4_SS_BRB	4	O	F
	d_8_SS_BRB	8	F	LS
	d_12_SS_BRB	12	F	LS
HA SMF	e_4_RC	4	LS	CP
	e_8_RC	8	LS	C
	e_12_RC	12	LS	C
HA TADAS	f_4_RC_TADAS	4	O	F
	f_8_RC_TADAS	8	F	LS
	f_12_RC_TADAS	12	F	LS
HA SLB	g_4_RC_SLB	4	F	F
	g_8_RC_SLB	8	F	LS
	g_12_RC_SLB	12	F	CP
HA BRB	h_4_RC_BRB	4	O	F
	h_8_RC_BRB	8	F	LS
	h_12_RC_BRB	12	F	LS

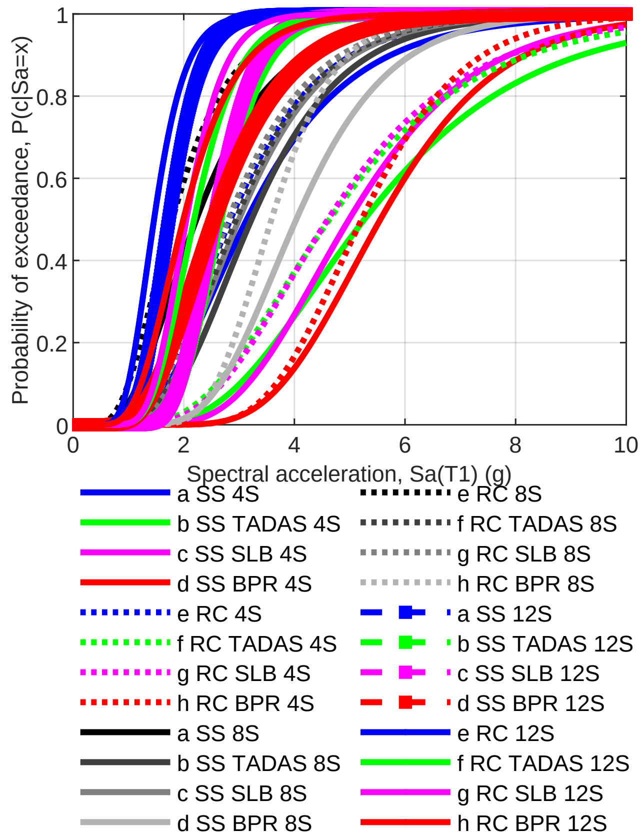


Figure 7. Structure fragility curves: Full damage

5. CONCLUSIONS

Structures designed without energy dissipators present drifts close to 2%. However, by including dissipators, these decrease to almost 1%, which means an improvement in the expected structural response. In addition, the stress on the portal frame elements, beams, or columns is substantially reduced. Consequently, the probable damages from a seismic event will be concentrated on the energy dissipators.

The Pushover curve shows that, at the first moment of analysis, the elastic stiffness of the structures is practically the same for each type of structure regardless of the kind of dissipator. However, an advantage is observed in the inelastic range when this device is included.

The capacity of the structures, in terms of shear force, when including BRB, is higher than with TADAS or SLB for this particular case. Although all the dissipators increase the rigidity of the system, this increase is higher in the case of the BRB. In cases with TADAS or SLB dissipators, the increase in damping is its main advantage. The global drifts of structures, the displacement in ceiling concerning its total height, is close to 0.5% for the effective creep displacement and 2.5% for the ultimate removal. Although in reinforced concrete structures of 8 and 12 floors, this decreases to 2%.

The ASCE 7 accelerogram scaling methodology, applied over a range of periods, using weighted averages, allows maintaining the variability of the seismic hazard in the analysis for the types of structures based on the number of floors. This approach facilitates generation of seismic scenarios consistent with the analysis site, in this case corresponding to the city of Quito.

The fragility curves of the structures show that the probability of reaching and exceeding a specific damage state decreases when including energy dissipators. In addition, for 12- story facilities, lower levels of intensity are required to achieve the same probability of damage as for 8- or 4-story structures.

In the fragility curves for slight damage, it can be seen that the damage probabilities are similar regardless of the type of dissipators because in this range of intensities, the structure responds elastically, and the dissipators do not yet contribute predominantly. On the other hand, there are differences for higher damage levels depending on the dissipator type. In addition, a more significant reduction of damage probabilities is observed when dissipators are included compared to only the portal frame structure for severe or complete damage states. This is explained because the inclusion of the design in the inelastic range is more significant, and the additional damping, stiffness, and resistance of the dissipators contribute significantly to the structure.

Choosing one dissipation system over another implies a precise analysis of each project. This requires studying other variables in finances, construction, and device availability, among other aspects not considered in this article. Therefore, it is impossible to infer that one energy dissipation system is better than another because they are different structures. However, it can be concluded that the performance of the networks improve significantly by including these devices. For this reason, implementing energy dissipators in formats is recommended to contribute to the facilities' resilience.

6. ACKNOWLEDGMENTS

Brian Cagua is grateful for the support of ANID-Subdirección de Capital Humano/Doctorado/2022-folio 21220089 for the scholarship to pursue graduate studies in Chile.

REFERENCIAS

Aguiar, R., & Cagua, B. (2022). Sistema computacional "CEINCI LAB" como una herramienta para la enseñanza de Ingeniería Sísmica-Estructural. *DYNA*, *89*(222), 74–82. https://doi.org/10.15446/dyna.v89n222.101814.

Aguiar, R., Cagua, B., Romero, J., & Pilatasig, J. (2019). Dos modelos numéricos de análisis sísmico de estructuras con disipadores TADAS. *Revista Internacional de Ingeniería de Estructuras*, *24*(4), 405–426. https://doi.org/10.24133/riie.v24i4.1575.

Aguiar, R., Mora, D., & Rodríguez, M. (2017). Análisis de una estructura con disipadores sometidos a espectros de diseño y de control. *Revista Internacional de Métodos Numéricos para Cálculo y Diseño en Ingeniería*, *33*(3), 171–178. https://doi.org/10.1016/j.rimni.2016.04.008.

Aguiar, R., Mora, E., & Rodríguez, M. (2016). *Análisis sísmico de estructuras con disipadores de energía ADAS o TADAS*. Centro Internacional de Métodos Numéricos en Ingeniería. https://www.scipedia.com/public/Aguiar_et_al_2019a.

Akkar, S., Sandıkkaya, M. A., & Bommer, J. J. (2014). Empirical ground-motion models for point-and extended-source crustal earthquake scenarios in Europe and the Middle East. *Bulletin of Earthquake Engineering*, *12*(1), 359–387. https://doi.org/10.1007/s10518-013-9461-4.

American Concrete Institute. (2019). *Building Code Requirements for Structural Concrete (ACI 318-19).*

American Institute of Steel Construction. (2016a). *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications (ANSI/AISC 358-16).*

American Institute of Steel Construction. (2016b). *Seismic Provisions for Structural Steel Buildings (ANSI/AISC 341-16).*

American Institute of Steel Construction. (2016c). *Specification for Structural Steel Buildings (AISC/ANSI 360-16).*

American Society of Civil Engineers. (2016). *Minimum Design Loads and Associated Criteria for Buildings and other Structures (ASCE 7-16).*

Beauval, C., Marinière, J., Yepes, H., Audin, L., Nocquet, J. M., Alvarado, A., Baize, S., Aguilar, S., Singaucho, J., & Jomard, H. (2018). A new seismic hazard model for Ecuador. *Bulletin of the Seismological Society of America*, *108*(3A), 1443–1464. https://doi.org/10.1785/0120170259.

Bozzo, L. (2022). Disipadores. Retrieved September 11, 2024, from https://luisbozzo.com/disipadores/.

Cagua, B., Aguiar, R., & Pilatasig, J. (2021). Nuevas funciones de CEINCI-LAB para el análisis y diseño de pórticos de acero acorde a la NEC-15. *Revista Internacional de Ingeniería de Estructuras*, *26*(1), 1–60. https://doi.org/10.24133/riie.v26i1.1958.

Cagua, B., Tola, A., Pilatasig, J., & Aguilar, P. (2022). Influencia sobre el cortante basal y derivas de edificaciones al incluir una fracción de la carga viva en la carga sísmica reactiva. *Investigación y Desarrollo*, *16*(1). https://doi.org/10.31243/id.v16.2022.1667.

Cagua, B. (2022). *Comportamiento no lineal de pórticos de hormigón armado y pórticos de acero con y sin disipadores de energía tipo TADAS, Shear Link Bozzo y Barras de Pandeo Restringido para el escenario sísmico de quito* [Tesis de Maestría]. Universidad de las Fuerzas Armadas ESPE.

Chiou, B. S. J., & Youngs, R. R. (2014). Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra. *Earthquake Spectra*, *30*(3), 1117–1153. https://doi.org/10.1193/072813EQS219M.

MIDUVI. (2015a). *Norma Ecuatoriana de la Construcción: Cargas (no sísmicas) (NEC-SE-CG).*

MIDUVI. (2015b). *Norma Ecuatoriana de la Construcción: Estructuras de acero (NEC-SE-AC).*

MIDUVI. (2015c). *Norma Ecuatoriana de la Construcción: Estructuras de hormigón armado (NEC-SE-HM).*

MIDUVI. (2015d). *Norma Ecuatoriana de la Construcción: Peligro sísmico y diseño sismo resistente (NEC-SE-DS).*

Ordaz, M., Martinelli, F., Aguilar, A., Arboleda, J., Meletti, C., & D'Amico, V. (2014). CRISIS 2014 v1.2. Program for computing seismic hazard. http://www.r-crisis.com/download/binaries/.

Parra, H. (2016). *Desarrollos metodológicos y aplicaciones hacia el cálculo de la Peligrosidad Sísmica en el Ecuador continental y estudio de riesgo sísmico en la ciudad de Quito* [Tesis doctoral]. Universidad Politécnica de Madrid.

Peer Ground Motion Database. (n.d.). Retrieved January 2, 2021, from www.ngawest2.berkeley.edu.

Quantification of Building Seismic Performance Factors (FEMA-695). (2009). Federal Emergency Management Agency (FEMA). Washington, D.C.

SEAOC. (1995). *Vision 2000. A framework for performance based structural enginneering*. California. https://www.seaoc.org/store/viewproduct.aspx?id=9180222.

Soong, T., & Spencer, B. (2002). Supplemental energy dissipation: State of the art and state of the practice. *Engineering Structures*, *24*(3), 243–259. https://doi.org/10.1016/S0141-0296(01)00092-X.

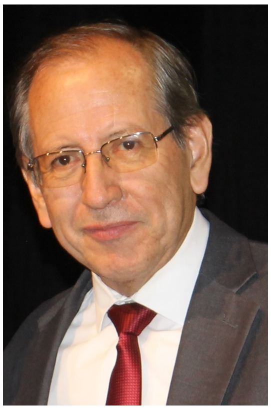
Vanegas, J. P. (2018). *Estudios de las causas de demolición de edificios afectados por el sismos del 16 de abril en Chone* [Tesis de Licenciatura]. Universidad Católica de Santiago de Guayaquil.

Zhao, J. X., Zhang, J., Asano, A., Ohno, Y., Oouchi, T., Takahashi, T., & Fukushima, Y. (2006). Attenuation relations of strong ground motion in Japan using site classification based on predominant period. *Bulletin of the Seismological Society of America*, *96*(3), 898–913. https://doi.org/10.1785/0120050122.

Zhu, M., McKenna, F., & Scott, M. H. (2018). OpenSeesPy: Python library for the OpenSees finite element framework. *SoftwareX*, *7*, 6–11. https://doi.org/10.1016/j.softx.2017.10.009.

BIOGRAPHIES

Brian Jordano Cagua Gómez. Civil Engineer, graduated in 2017 from the Escuela Politécnica Nacional, Master's in Operations Management and Industrial Safety at the Universidad de las Americas in 2020, Master's in Science in Civil Engineering with a focus in Structures at the Universidad de las Fuerzas Armadas ESPE, a Ph.D. student in Engineering Sciences at the Universidad de los Andes in Chile.



Roberto Rodrigo Aguiar Falconí. Civil Engineer, graduated in 1978 from the Escuela Politécnica Nacional; Master's of Science in Earthquake Engineering graduated from the Universidad Central de Venezuela in 1982; Ph.D. in Engineering graduated from the Universidad Politécnica de Cataluña in 1987. He is a Fulltime Professor at the Universidad de las Fuerzas Armadas ESPE and a Structural Consultant.