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Seismic loss assessment of medium-rise reinforced concrete frames in Colombia equipped with TADAS dampers

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ABSTRACT

Colombia, South America's third most populous country, faces high seismic hazards, making reinforced concrete (RC) buildings particularly vulnerable to earthquakes. To mitigate this vulnerability, seismic protection systems have emerged as effective solutions for enhancing the seismic resilience of buildings. This study evaluates the seismic performance of medium-rise RC regular plan buildings equipped with Triangular Added Damping and Stiffness (TADAS) devices, using a performance-based earthquake engineering (PBEE) framework. The analysis focuses on 4-, 7-, and 10-story RC buildings located in Cali, Colombia. Structural models were developed in OpenSeesPy and subjected to non-linear Incremental Dynamic Analysis (IDA) using FEMA P695 seismic records. Fragility curves were derived from the IDA results to estimate the probability of exceeding various damage states, while vulnerability curves quantified expected annual losses (EAL) and lifetime economic losses for buildings with and without TADAS devices. Results demonstrate that TADAS devices significantly enhance seismic performance, reducing inter-story drifts by up to 55.4% under maximum considered earthquake (MCE) conditions. The probability of complete damage in a 4-story building decreased from 20.4% to 6.8%, with similar reductions observed for mid-rise buildings. Economically, the TADAS devices were effective, reducing EAL by up to 71.7%. These findings underline the dual benefits of TADAS devices: improved structural performance and significant economic savings over the lifespan of the building.

1. Introduction

Colombia is South America's third most populous country and has experienced significant seismic events. Notable examples include the 1999 M6.1 Armenia earthquake and the 1983 M5.6 Popayan earthquake, which resulted in 1185 and 287 deaths, respectively, and caused economic losses equivalent to 1.9% and 1.5% of the country's annual gross domestic product [1,2]. The country's high seismic activity is attributed to its location at the convergence of the Nazca, Caribbean, and South American tectonic plates. According to the Colombian Geological Service, approximately 83% of the population resides in areas classified as having intermediate to high seismic hazard [3,4], with over 10 million people living in buildings vulnerable to collapse in the event of a major earthquake [5]. Understanding all the aspects of seismic risk in Colombia requires collaborative initiatives among government institutions, academia, industry, non-profit organizations, and people. Seismic risk studies in Colombia have been ongoing for decades, initially focusing on major cities such as Bogotá [6,7], and later expanding to other urban areas such as Medellin, Cali, Manizales, and the province of Sabana [2,5,7–9]. In parallel, the Colombian Association of Seismic Engineering (AIS) recently created the AIS 400 committee, which is responsible for developing new guidelines for the seismic vulnerability of existing structures in Colombia, including assessment and retrofitting.

The stock of buildings in Colombia includes many reinforced concrete frame (RC-F) buildings constructed informally without adhering to the minimum of current design standards. Additionally, many formal structures were built before the enforcement of the Colombian seismic design codes NSR-10 [5] and NSR-98 [10], rendering between

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60% and 90% of residential buildings vulnerable to seismic events. The seismic provisions of the Colombian Seismic Design Code NSR-10 closely align with those of ACI 318-08, particularly in the requirements for RC-F buildings. Both codes emphasize similar details for ductile elements and the capacity to resist seismic forces [11]. The seismic performance of these buildings has been severely compromised during previous earthquakes, primarily due to poor construction quality and substandard materials [12]. Post-earthquake damage assessments reveal that RCF buildings often suffer significant damage, primarily due to connection failures that severely limit their ability to resist seismic forces [13]. This underscores the need to assess the seismic vulnerability of these structures in earthquake-prone regions [14] as part of a seismic risk mitigation strategy. In addition, recent advances in artificial intelligence, such as the use of machine learning models, offer promising tools for assessing the seismic vulnerability of existing structures more accurately. These methods allow large volumes of structural and seismic data to be analyzed, identifying patterns that improve efficiency in the evaluation and prioritization of building reinforcement [15–17]. Multiple studies in Colombia have quantified the relative economic losses associated with the seismic behavior of RC-F buildings, demonstrating a trend for having significant damage under high seismic intensity levels [18,19]. Such a situation requires precise guidelines to reduce structural vulnerability in the country.

The current Colombian NSR-10 code [20] recommends a range of conventional retrofitting strategies to improve the seismic performance of existing RCF buildings. These strategies include strengthening structural members such as beams and columns and installing traditional steel bracing systems. Currently, the national code does not provide further details regarding the utilization of seismic protection systems (SPS). However, these systems have emerged as a highly effective method for enhancing the seismic resilience of existing reinforced concrete frame (RCF) structures during seismic events [21–25]. SPS effectively absorb and dissipate a significant portion of the seismic energy introduced into the structure, thereby reducing the demands on structural components and minimizing potential damage [26]. By mitigating the effects of seismic forces, SPS improve the overall stability and resilience of structures, increasing occupant safety and reducing post-event repair costs [24,27,28]. Unlike traditional strategies that focus on improving the strength of structural elements, SPS is designed to directly target and dissipate the seismic energy introduced into the structure. ASCE 41-23 [29] is an international document that provides the main requirements for the design of SPS-based retrofitting solutions of existing buildings. It is worth mentioning that the national AIS-400 committee presents a division defining the guidelines for using SPS for building retrofitting.

Among the various seismic energy dissipating systems, those employing triangular added damping and stiffness devices (TADAS) have shown efficacy and cost effectiveness [30,31]. Experimental investigations on TADAS devices have shown their ability to undergo multiple yielding reversals without any significant loss of stiffness or strength, particularly in RCF buildings [32]. TahamouliRoudsari et al. [33] studied the impact of varying the number of TADAS plates on the seismic behavior of RCF buildings and concluded that the shear capacity and ductility of structures equipped with TADAS devices significantly exceeds that of unmodified structures. Furthermore, analytical studies have highlighted the advantages of incorporating TADAS devices in buildings [34]. In a study by Saghafi et al. [35], the seismic performance of three RC moment-resisting frames with four, seven, and ten stories was assessed. The findings revealed that incorporating TADAS enhanced the seismic performance of these structures to levels of immediate occupancy and life safety. Sedaghatnezhad et al. [36,37] conducted incremental dynamic analyses and developed fragility curves for steel buildings reinforced with TADAS devices, evaluating 3-, 6-, and 9-story structures. The study concluded that TADAS devices are highly effective in improving the seismic performance of steel frame buildings. Regarding life cycle cost analysis, Sanati et al. [27] conducted an economic evaluation of 4- and 8-story steel structures equipped with TADAS devices, finding a 28% reduction in life cycle costs for buildings reinforced with TADAS devices. In Colombia, Caballero-Castro et al. [38] developed a methodology for modeling TADAS in buildings with RC and steel moment-resisting frames, utilizing SAP2000 to implement the finite element model. Additionally, Caballero-Castro et al. [39,40] evaluated the response modification factor (R) for the same type of buildings, demonstrating the advantages of including TADAS devices in seismic design.

Prior research has indicated that incorporating TADAS devices augments the structural resilience of reinforced concrete frames (RCF) in the context of seismic loading [40]. However, existing research has predominantly concentrated on computing the enhanced structural capacity and the reduction of inter-story drifts concerning the original frame. To gain a more comprehensive understanding of the efficacy of seismic protection devices in Colombia, it is imperative to broaden the scope of the assessment to encompass the resilience and lifetime losses of existing buildings reinforced with TADAS devices. This would facilitate a more nuanced comprehension of the economic advantages inherent to utilizing these alternatives. In response to this need, the present study evaluates the seismic performance of RCF buildings equipped with TADAS devices, considering variations in building height and seismic hazard levels. The TADAS-based equipped process of 4-, 7-, and 10-story buildings in Cali (Colombia) was carried out. Nonlinear Incremental Dynamic Analysis (IDA) was performed using the 44 seismic records from FEMA P695 [41]. The structural analysis was implemented in the open-source software OpenSeesPy [42], considering a special modeling for the TADAS devices. Then, fragility curves were generated to assess the probability of exceeding specific damage states for the original and equipped structures. To quantify the economic implications, relative economic losses were assigned to each damage state, forming the basis for estimating vulnerability functions. Finally, expected annual losses (EAL) and lifetime economic losses were calculated to provide a more comprehensive evaluation of the economic impact of TADAS devices.

2. Methods

2.1. Design of the TADAS devices

The TADAS dampers were introduced by Tsai et al. in 1993 [30] as an enhanced solution to previous ADAS devices. Fig. 1 on the left depicts a typical configuration of vertical triangular steel plates of varying cross-sections in height. These plates are rigidly linked to the superior horizontal plate and feature a hinge connection on the bottom side of the damper, facilitating the deformation process. As seen in Fig. 1 on the right, the device's thickness is perpendicular to the beam axis to allow the bending. During seismic events, the device dissipates energy by developing inelastic deformation raised in the relative movement between the superior and bottom side [43]. The varying width of the device is designed with the intention of enhancing its deformation capacity during earthquakes. Dareini et al. [34] demonstrate that integrating TADAS devices into structures modifies the distribution of shear forces at the base, reduces drifts, and alters internal forces, all without impacting the vertical load-bearing performance. It is worth mentioning that metallic hysteretic devices, such as TADAS, are particularly favored due to their stable and predictable hysteretic behavior. Fig. 2 presents a typical representation of the characteristic hysteretic behavior of a TADAS device for designing issues.

The equations in Table 1 are derived from fundamental principles of mechanics and validated through experimental studies by Tsai et al. [30] and Aguiar et al. [44]. These equations effectively describe the hysteretic behavior of TADAS devices, capturing their energy dissipation capacity and cyclic response under seismic loading. They account for key parameters such as shear strength, energy dissipation, and the expected deformations of the TADAS plates. Specifically, V_{T_y}

Table 1
Equations that describe the performance of the TADAS device.

Yield force V_{Ty}	Ultimate force V_{Tu}	Yield displacement Δ_{Ty}	Ultimate displacement Δ_{Tu}	Stiffness of TADAS plates K_{TADAS}
$V_{Ty} = \frac{nf_y bt^2}{6h}$	$V_{Tu} = R_t \frac{nf_y bt^2}{4h}$	$\Delta_{Ty} = \frac{f_y h^2}{Et}$	$\Delta_{Tu} = \mu \frac{f_y h^2}{Et}$	$K_{TADAS} = \frac{nEbt^3}{6h^3}$

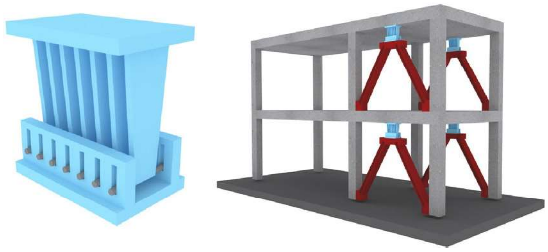

Fig. 1. Typical schematic of TADAS plates in a concrete frame.

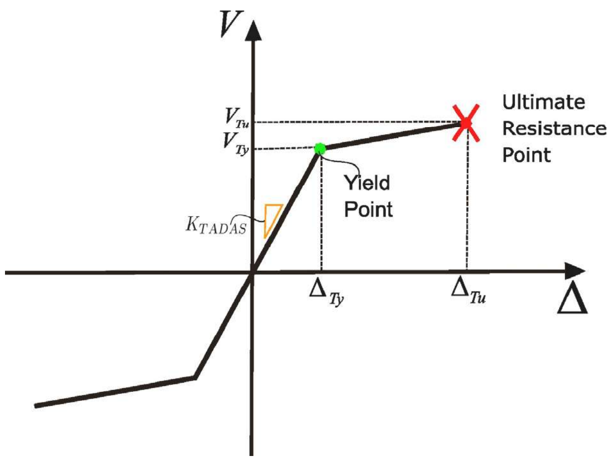

Fig. 2. Characteristic hysteretic behavior of the TADAS device.

and V_{Tu} represent the yield and ultimate forces, respectively, while Δ_{Ty} and Δ_{Tu} denote the corresponding yield and ultimate displacements. The stiffness of the TADAS plates is defined as K_{TADAS}, and other geometric parameters include the number of plates (n), plate width (b), thickness (t), and height (h). For steel, critical material properties such as the overstrength coefficient (R_t), ductility (μ), yield strength (f_y), and modulus of elasticity (E) are incorporated. The parameters listed in Table 1 were determined based on the structural demands of the analyzed buildings, including interstory drift limits and expected seismic forces. Finite element simulations and iterative design processes were utilized to ensure that the dimensions and material properties met the required performance criteria. The optimization of these parameters was conducted through an iterative analysis, balancing structural performance with cost efficiency to achieve an effective and economical design.

The authors in Caballero-Castro et al. [40] configured a design procedure for buildings equipped with TADAS devices, which is based on the relationships between stiffness and displacement, as outlined in the parametric study by Oviedo et al. [45]. A summary of the steps carried out for the sizing of the TADAS devices is presented below, detailing the iterative process and considerations used to ensure compliance with the performance objectives established in the design.

1. Nonlinear static (pushover) analyses are conducted to evaluate the seismic performance of the primary structural system. These analyses initially determine the properties of the main frames, such as the building's overall capacity curve. From this, capacity curves for each story are derived, allowing for the calculation of the story's yield shear force (V_{Fy}) and yield displacement (Δ_{Fy}).
2. Based on Ref. [45], the drift ratio (υ) and shear ratio (β) are established. In this study, $\upsilon = 0.40$ and $\beta = 0.35$ are adopted. These parameters are utilized to calculate the design forces (V_{Dy}) and design displacements (Δ_{Dy}) for the damper system, which comprises TADAS devices and braces, using the following equations:

$$V_{Dy} = \frac{\beta}{1-\beta} V_{Fy}$$

$$\Delta_{Dy} = \upsilon \Delta_{Fy}$$

3. The overall stiffness of the damper system (K_d), which comprises both TADAS devices and braces, is calculated for each story based on the methodology outlined in Ref. [30]:

$$K_d = \frac{V_{Dy}}{\Delta_{Dy}}$$

4. The configuration and cross-sectional geometry of the braces are defined, and their total stiffness (K_b) is calculated as:

$$K_b = 2N \frac{A_b E}{L_b (\cos\theta)^2}$$

Here, A_b and L_b represent the cross-sectional area and the length of the brace, respectively; θ denotes the angle formed between the brace and the horizontal plane; and N refers to the total number of bays equipped with a damper system on each story and in each direction of analysis.

5. The required stiffness of the TADAS plates (K_{TADAS}) for each story is determined using the stiffness of the braces (K_b) and the overall stiffness of the damper system (K_d), as follows:

$$K_{TADAS} = \frac{K_d K_b}{K_b - K_d}$$

6. The yield force (V_{Ty}) and the corresponding yield displacement (Δ_{Ty}) of the TADAS devices for each story are calculated as:

$$V_{Ty} = V_{Dy}, \quad \Delta_{Ty} = \frac{V_{Ty}}{K_{TADAS}}$$

7. With the design shear forces V_{Ty} and displacements of the TADAS plates Δ_{Ty} already determined, their geometry (b, t, and h) is calculated using the equations provided in Table 1 through an iterative process. This approach ensures that the plates meet the performance requirements for each story. Finally, the overall structural behavior with the incorporated TADAS devices is analyzed to verify that the system satisfies the performance objectives established in the design.

For clarity and completeness, Fig. 3 provides a simplified flowchart of the seven-step design methodology, which can be used for retrofitting while considering the characteristics of the original structure. The methodology starts with the characterization of the current state of the original structure in steps 1 and 2. Then, the geometry of the TADAS devices is determined through steps 4 to 6; which includes applying the proposal of [45]. Finally, the maximum drifts in the buildings for the MCE and the internal forces in the structural elements are verified. For further details, the reader is suggested to read the Ref. [40].

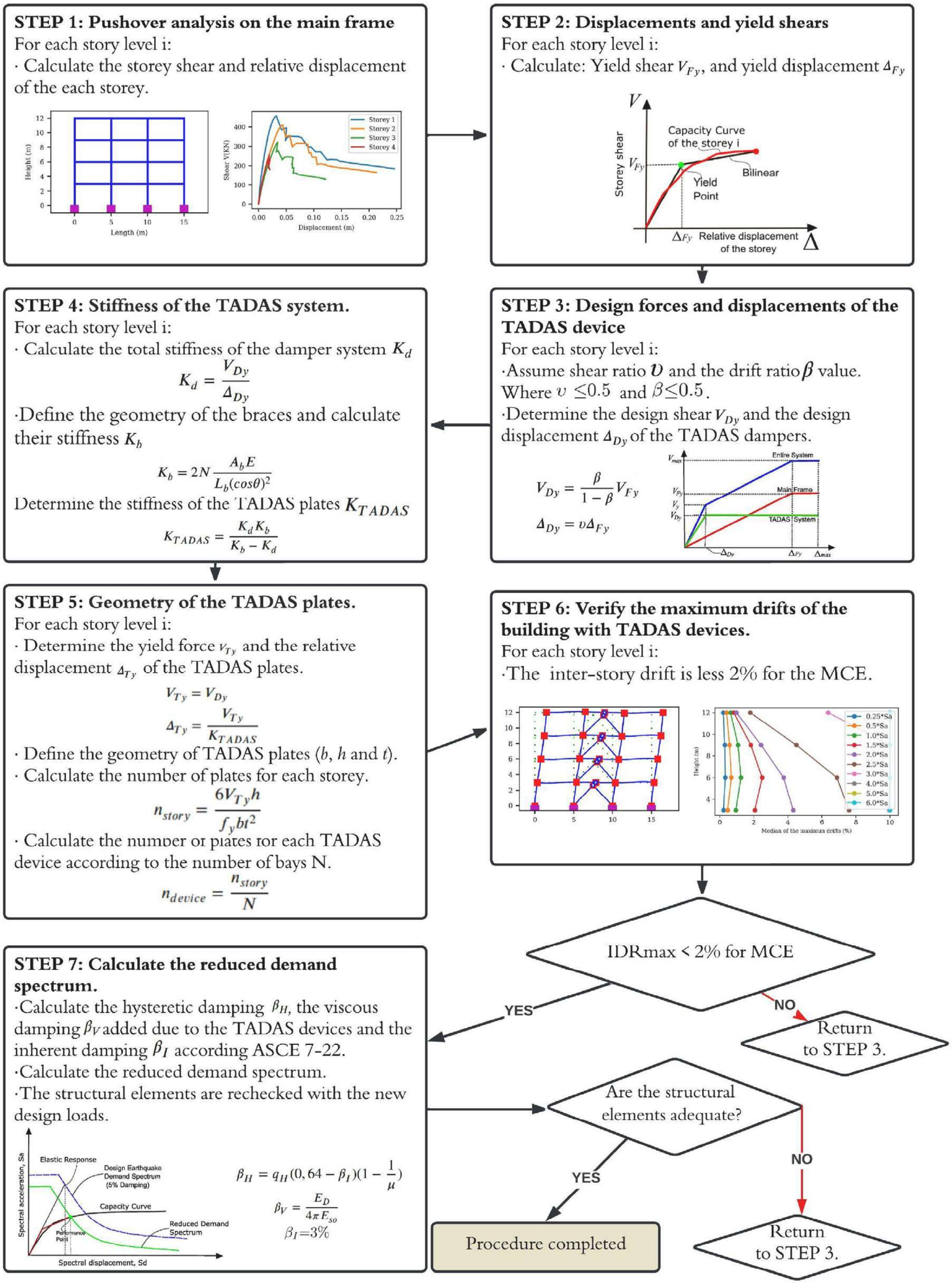

Fig. 3. Flowchart of the structural design procedure of the TADAS-based retrofitting solution according to Caballero-Castro et al. [40].

2.2. Modeling the RC-F building equipped with TADAS in OpenSees

Opensees is one specialized structural analysis software to determine the nonlinear behavior of a structure, which currently presents a version supported in Python language [42]. It is worth mentioning that Several graphical interfaces are being created to facilitate the software's analysis process, such as [46–48]. Opensees software has been used in different studies on the seismic performance of a structure equipped with metallic hysteretic dampers such as Hourglass shape pins [49], hysteretic column daper [50] or honeycomb damper [51]. This research uses Opensees software to compute the response of RC-F buildings with TADAS devices under seismic loads.

Concerning the RC-F, the columns and beams were modeled employing *forceBeamColumn* elements, with five Gauss–Lobatto integration points and the fiber-section approach. The beam–column joints were modeled as rigid connections. This assumption is widely adopted in structural analyses to simplify the computational process and has been shown to provide reliable results for reinforced concrete frames with appropriate detailing [22,52,53]. The material behavior was represented by the *Concrete01* model for concrete and the *HystereticSM* model for reinforcement steel. In this study, concrete was classified as either unconfined or confined to account for the effects of confinement, as described in the theory of Mander et al. [54]. A regularization technique based on constant fracture energy was applied to the concrete columns and beams, as outlined by Spacone et al. (2001). The fracture energy was defined as $G_f^c = 28$ N/mm for unconfined concrete and $G_f^c = 36.4$ N/mm for confined concrete, respectively. The reinforcing steel was modeled to account for buckling behavior following the scheme proposed by Dhakal et al. [55], considering the material properties of standard reinforcing steel in Colombia, as detailed by Carrillo et al. [56]. It is considered that the column base are fully restrained, avoiding displacements and rotations.

The TADAS devices were modeled using the *TwoNodeLink* element, which provided lateral stiffness only. This is because the devices do not contribute to vertical load resistance. The connections between the TADAS dampers and the frame were modeled as rigid links to ensure an accurate transfer of forces between the dampers and the structural elements. The lateral stiffness and yield strength of the TADAS devices were calculated using the equations provided in Table 1. Additionally, the fracture behavior of the TADAS devices was simulated using the *MinMax* material available in OpenSees. When the TADAS device reaches its maximum displacement, the material model causes an abrupt loss of stiffness, effectively simulating the failure of the device (see Fig. 2). The steel braces that support the TADAS devices were designed and modeled as elastic elements using *elasticBeamColumn* elements, considering hinge connection to the structural elements. It is essential that braces remain elastic to ensure that the functioning of the dampers is not affected. A comprehensive graphic summary of the OpenSees model is presented in Fig. 4. Table 2 summarizing the OpenSees parameters for all materials used, including concrete, steel reinforcement, and TADAS dampers, has been included. The parameters were selected based on experimental data and validated to ensure accurate material behavior representation under seismic loading.

The OpenSees model was validated by comparing its results with experimental data from Ghaffary et al. [57], as shown in Fig. 5. The validation focused on reproducing the hysteretic behavior of the TADAS devices. The experimental setup described in Ghaffary et al. [57] involves a full-scale one-story, one-bay steel frame equipped with TADAS dampers. The frame, 3 m in height with a span of 4 m, includes IPE180 columns and a beam composed of welded plates. Chevron braces, made of double-angle steel sections, are used to connect the TADAS damper. The damper consists of six triangular steel plates (34.5 cm high, 3 cm thick), welded to a 60 × 25 cm top base plate and a 48 × 11 cm bottom base plate. Pins (2 cm in diameter, 20 cm long) connect the plates to the base. The experimental tests applied a quasi-static cyclic displacement history using two hydraulic actuators, following the ASCE 41 standard. The setup was designed to evaluate the hysteretic behavior and energy dissipation capacity of the TADAS system. The agreement between numerical and experimental results confirms the accuracy of the modeling approach.

2.3. Incremental dynamics analysis (IDA)

Incremental Dynamic Analysis (IDA) is a widely recognized parametric approach for evaluating structural performance under various ground motions [60]. In this study, IDA was performed using OpenSeesPy software [42] in conjunction with the opseestools library [48]. The buildings under examination were subjected to IDA utilizing the far-field suite of seismic records as outlined in the FEMA P-695 standard. Table 3 details the 44 seismic records used in this study, accompanied by the corresponding peak ground acceleration (PGA). The ground motion records were initially scaled so that the spectral ordinate at the fundamental period of the structures matched the spectral ordinate of the code-based design spectrum. Ten scale factors were applied (0.25, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0), resulting in 440 time–history analyses for each building. A scale factor of 1.0 corresponds to the design-basis earthquake (DBE), while 1.5 represents the maximum-considered earthquake (MCE). Values exceeding the MCE are employed to gain further insight into the seismic behavior of the buildings. The seismic performance of the buildings was assessed by evaluating the maximum inter-story drift ratio (IDRmax), which served as the engineering demand parameter (EDP). Such information is used in the construction of fragility curves as defined in the following section.

2.4. Fragility curves

A fragility curve represents the probability of exceeding a specific structural damage state (DS) due to an earthquake. In this study, fragility curves were developed using the spectral acceleration (Sa) as the intensity measure (IM) and the maximum inter-story drift ratio (IDRmax) as the engineering demand parameter (EDP). The curves were derived from the incremental dynamic analysis (IDA) results. The IDA results were processed to generate fragility functions for each building, with the data binned based on the corresponding damage limits. The probability of occurrence for each DS was calculated for each building, and the results were fitted to a log-normal distribution using the methodology proposed by Baker et al. [61]. The damage state limits employed in this study were based on the recommendations from the Hazus 4.2 SP3 Earthquake Model Technical Manual (2020) [62]. These limits consider various building types, accounting for both height and the quality of the initial seismic design. For this research, the DSs defined for medium-rise buildings with a high-code seismic design level were applied. The corresponding limits, expressed in terms of IDR, are presented in Table 4.

2.5. Loss assessment methodology

The methodology employed for loss assessment in this study is based on the second-generation Performance-Based Earthquake Engineering (PBEE) framework, which was developed by the Pacific Earthquake Engineering Research Center (PEER) [63]. This probabilistic approach evaluates seismic performance by addressing uncertainties inherent to seismic intensity, structural response, component damage, and economic consequences. The methodology entails the calculation of engineering demand parameters (EDPs), such as inter-story drifts, to evaluate the potential damage to both structural and non-structural components. By employing fragility functions and consequence models, the framework estimates repair costs and expected annual losses (EAL). This comprehensive approach allows for a more profound comprehension of seismic risks, enhancing decision-making processes for retrofitting or designing structures in seismically active regions [64]. Furthermore, it facilitates a reduction in repair costs and downtime.

As outlined in the FEMA P-58 guidelines, the PBEE framework utilizes the total probability theorem to calculate the likelihood of exceeding specific performance measures (PM), including casualties, economic losses, downtime, and environmental impacts (see Eq. (1)). The methodology integrates several components, including the consequence function $G(PM|DS)$, which represents the occurrence probability of a performance measure given a particular damage state (DS), such as cracking or buckling. Furthermore, EDPs, such as inter-story drift ratio, peak floor acceleration, and residual drift ratio, are