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Evaluating the response modification factor (R) for reinforced concrete and steel structures equipped with TADAS devices designed for high seismic hazard in Colombia

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ABSTRACT

Colombia is a country with high seismic activity due to its location at the convergence of the Nazca and South American tectonic plates, making structural safety a key priority in the design, construction, and retrofit of buildings. Currently, one of the most used seismic resistance systems are moment-resisting frames, due to their easy construction and good behavior under vertical loads, however, they are susceptible to structural damage when subjected to moderate to high-intensity earthquakes, which makes the use of seismic protection systems an adequate alternative to prevent damage during seismic events. The Colombian seismic design code NSR-10 proposes a design methodology using a linear structure that represents a non-linear behavior of the building when the expected design earthquake occurs through the response modification factor R. However, the NSR-10 seismic design code does not contemplate the response modification factor for structures with seismic protection systems. Therefore, in this research, a design procedure was configured for buildings with moment-resisting frames of reinforced concrete and steel equipped with TADAS (triangular-plate added damping and stiffness) dampers. The procedure evaluated the developed response modification factor of structures located in high seismic hazard areas in Colombia and considered regular buildings with varying numbers of stories and special energy dissipation capacity. The capacity of the structures was evaluated through nonlinear static analysis using the Pushover method in SAP2000 software. Finally, the response modification factors were calculated for regular buildings with moment-resisting frames and different numbers of stories.

1. Introduction

The use of response modification (R) factors is one of the key aspects that facilitates the design of various types of structural systems for buildings [1], as suggested by international seismic design codes. The underlying idea is to carry out the design process using a linear structure that presents a behavior equivalent to that obtained for the actual nonlinear model of the building when the expected design earthquake occurs. Whittaker et al. [2] indicate that factor R was included as a type of empirical value in the design codes, which considered energy dissipation in buildings due to damping, over-strength, and their inherent ductility when subjected to a seismic event. This factor represents the relationship between the seismic design force considering the linear response of the building and the maximum force obtained in the non-linear building for the design earthquake [3]. Consequently, the design seismic forces obtained in the analysis of the elastic model could be reduced according to this factor in the structural design of conventional buildings [4]. This strategy is also implemented in the Colombian building seismic design code [5], with the R factor depending on the seismic resistance system and the seismic hazard zone. In Colombia, some research has been carried out to calculate the factor R in concrete structures [6] and steel structures [7]. The results of the investigation show that the actual demand for ductility developed by buildings with the same structural system does not necessarily match the value R defined by the code for such a system.

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Current research on the computation of the R-factors is mainly addressed to studying non-conventional structural types. Li et al. [8] evaluated the factors R for buildings built from steel frames equipped with eccentric K-shaped braces, arguing that the factors calculated from a pushover analysis were higher than those added using IDA analysis. Aliakbari et al. [9] evaluated the R factors for steel slit panel frames of 3, 6, 9, 12, and 15 stories, concluding that the R factor decreases with the number of stories and remains constant after ten stories. Furthermore, the influence of specific building configurations on the R factor has been studied. Hussein et al. [10] calculated the R factor for reinforced concrete frames with spans and heights of different sizes throughout the structure, observing that the structure showed a faster collapse mechanism and an irregular distribution of plastic hinges by increasing the widths of the span and the heights of the floors. Shen et al. [11] calculated the response modification factor for high-strength steel frames with D-eccentric brace using an improved pushover analysis method for 8, 12, and 16-story buildings. Ramenazi et al. [12] calculated the R-factor for steel frames with concrete-filled tubular steel columns (CFST) for regular buildings of 10 stories in different soil types. The above results show the importance of understanding the process of simplification of the design process for new structural systems by computing their corresponding R factors.

On the other hand, one of the most widely used structural systems for low- and medium-rise buildings corresponds to moment-resisting frames, due to their ability to withstand lateral loads during an earthquake [13]. This ability can be enhanced by including seismic protection (SP) devices to absorb or dissipate energy, a strategy that has gained worldwide attention in recent decades. In this sense, there exists a class of devices that dissipate energy through their hysteretic behavior [14], which are commonly used due to their relatively affordable price and minimal maintenance requirements [15]. Using non-elastic deformation for energy dissipation is an appropriate strategy to concentrate damage on dissipation devices and avoid damage to beam–column joints [16]. An example of hysteretic devices is Triangular Added Damping and Stiffness (TADAS) dampers, which dissipate energy through the inelastic bending of a set of triangular steel plates [17,18]. It should be noted that these devices must be designed to initiate energy dissipation in moderate earthquakes and short displacements [19,20]. TADAS devices have been implemented as an alternative to seismic reinforcement of existing structures in developing countries. In Ecuador, the Rodriguez Zambrano Hospital in the city of Manta [21] was reinforced with TADAS due to the slight damage suffered by the earthquake that occurred in 2016. The Cardiology Hospital of Siglo XXI in Mexico City was reinforced with an ADAS damper [22]. In the case of Colombia, no building has been equipped with TADAS dampers. In addition, numerical research supports the good performance of TADAS-equipped structures for earthquake loads [23,24].

In practice, building design codes consider factors R for buildings with conventional seismic resistance systems, such as concrete walls or moment-resisting reinforced concrete and steel frames. However, they do not directly consider the R factor for the entire system (main structure plus SP systems). Some research efforts have been conducted to compute the R factor for buildings with SPS. Abou-Elfath et al. [25] calculated the R factors for 3, 6, and 9-story buildings with steel frames with restricted buckling braces. Asgarian et al. [13] calculated the factor R in buildings of 4, 6, 8, 10, 12, and 14 story buildings with steel frames and restricted buckling braces with different configurations (split X, Chevron V, Chevron-inverted V, and diagonal type). Abdi et al. [26] studied the response modification factor for steel frame buildings with 4, 8, 12, 16, and 20-story buildings equipped with viscous dampers. It is worth mentioning that the literature on the subject is relatively scarce despite the increase in real SPS applications. To the best of the author's knowledge, only two researchers have reported the computation of the R factor for TADAS structures [3,16]. Both articles calculated the R factor for a steel frame building equipped with TADAS in Iran, considering between three and fifteen stories. Some of the most important conclusions of these works were that the inclusion of TADAS devices improves the capacity of buildings, which is reflected in the reduction of drifts and damage to buildings. No studies have been reported for reinforced concrete buildings with TADAS devices.

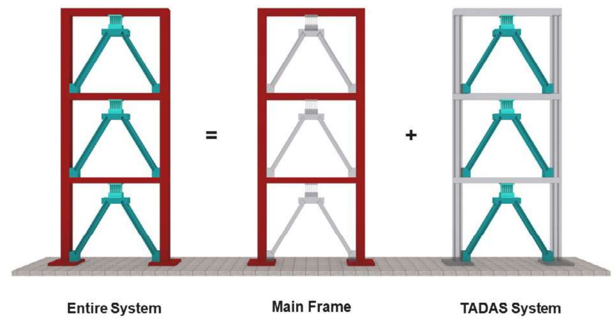


Fig. 1. Conceptual division of a frame with TADAS devices.

This research evaluates the response modification factors for reinforced concrete and steel frame buildings with varying numbers of stories equipped with TADAS devices. The buildings were located in a region of high seismic hazard in Colombia and a life safety design objective was targeted. It proposes a design procedure for buildings with TADAS devices based on the methodology proposed by ASCE 7-16 [27], the Colombian seismic design code NSR-10 [5], the mechanical characterization of TADAS devices proposed by Tsai et al. [17], and the design procedure of Oviedo et al. [28]. A non-linear static analysis (pushover) was selected to follow the process of computations of R-factors by Mahmoudi et al. [3]. This research is important for the following reasons:

• An approximation to compute the response modification factors R for concrete and steel moment-resistant frame structures designed with TADAS devices in Colombia is carried out. These results contribute to the knowledge of R-factors for buildings with seismic protection systems.

• A design proposal for buildings with TADAS devices located in Colombia was outlined in detail in the document. These guidelines are important for promoting the use of TADAS devices not only in Colombia but also in other LMIC (low-and middle-income countries).

2. Methods

2.1. Design of the main structural system

From a conceptual design perspective, it is typically considered that a building equipped with a seismic protection system corresponds to the combination of two components. Fig. 1 shows this division for a 3-story moment-resisting steel frame equipped with TADAS devices, which are supported by steel diagonals. The structure must be provided with a main traditional structural system that in the case of moment-resisting frames will be constituted of beams and columns. The other component consists of the dissipation system, composed of the TADAS device and the supporting Chevron braces. Fig. 2 presents the relative contribution of each component to the structural performance of the entire system, where the level of contribution depends on the design criteria. In this sense, this section is about the design of the main structural system, while Section 2.3 presents the sizing process of the TADAS devices and the braces.

The design of the main structural system was carried out using the response of the building for an elastic dynamic analysis as specified in the Colombian seismic design code NSR-10 [5]. This method is valid for the buildings described in Section 3, as they do not have irregularities and are less than 20 stories high. The mathematical model

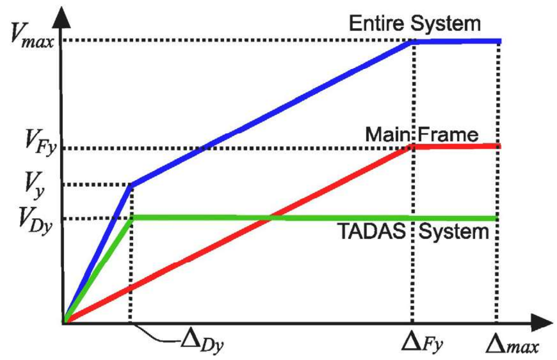


Fig. 2. Typical schematic hysteretic behavior of structure with a TADAS device.

used corresponds to a three-dimensional model with a rigid diaphragm, where the mass of each diaphragm is considered concentrated at its center of mass. The representation of seismic events will be reflected through the seismic design spectrum. For the sake of completeness, the design procedure is described as follows:

1. The vibration modes of the structure must be obtained such that there is a minimum 90% participation of the mass of the structure, considering an accidental eccentricity of 5%. The maximum response for each mode is obtained using the response spectrum shown in Fig. 14 for each vibration period of the mode. The total response in terms of shears, base shears, and forces in the elements must be calculated as a combination of the influence of all the modes considered in the analysis.

2. In the Colombian seismic design code, the design of structures with dissipation systems is not considered. Therefore, the ASCE 7-16 code is used to size the main structure. ASCE 7-16 states that the main structure should be preliminarily sized with 75% (see Eq. (1)) of the base shear when two requirements are met: (1) It has no irregularities and (2) two dampers must be provided in each direction per floor to avoid generating irregularity. Eq. (1) presents the computation of V_d is the design base shear:

$$V_d = 0.75 S_a g M \tag{1}$$

where S_a is the spectral acceleration taken from the design spectrum in Fig. 14, g is gravity and M is the mass of the building. The seismic design forces for the initial main structure are determined by dividing the total internal forces of the elements F_s by the energy dissipation coefficient R defined for the main structure, $F = F_s/R$.

3. TADAS devices are designed using the methodology described in Section 2.3 and are included in the structural model.

4. After the inclusion of TADAS devices in the structural model, the structure must be again checked with 100% of the base shear. The results must be adjusted if the base shear value calculated using the elastic dynamic method is not at least 80% for regular structures and 90% for irregular structures of the base shear calculated using the equivalent lateral force method.

5. Following the base shear adjustments, it is necessary to verify the total drifts obtained, which must not exceed 1% for steel and concrete frame structures for the design-based earthquake (DBE) and 3% for the maximum considered earthquake (MCE).

6. After the design of the TADAS devices, the structural elements are rechecked with the new design loads, this time using 100% of the base shear.

2.2. Assessment of the non-linear static response of moment-resisting frames

The procedure to size the TADAS devices in Section 2.3 requires determining the non-linear response of the structure. In this case, a traditional Pushover was selected to generate the building capacity curve as seen in (Fig. 3). The method consists of applying a pattern of lateral loads, increasing its magnitude until the building reaches a specified target displacement. This point represents the maximum horizontal movement under ground excitation at design level [16]. However, it is necessary to follow the gradual yield of the structural elements. For the structures analyzed in this paper, it is considered that the Pushover analysis adequately describes the non-linear behavior of the frames. This procedure was carried out according to the guidelines of the Colombian seismic design code NSR-10 and FEMA-440 [29].

For this research, the target displacement is calculated using the ATC-40 capacity spectrum method. The capacity spectrum method involves transforming both the capacity curve and the demand response into a capacity spectrum (Sa vs. Sd), which is a representation of the capacity curve presented in the form of an acceleration-displacement response spectrum (ADRS) [30]. The performance point results in the intersection of the capacity spectrum and the reduced demand spectrum (see Fig. 4). The spectral acceleration, S_a is, and the spectral displacement, S_d, are given by

$$S_a = \frac{V_s}{W_1} \tag{2}$$

$$S_d = \frac{D_{1D}}{\tau_1} \tag{3}$$

where V_s is the base shear, W_1 represents the effective weight of the fundamental mode in the analysis direction, D_{1D} is the displacement of the roof in the center mass of the building, and τ_1 is the participation factor of the fundamental mode of the structure in the direction of interest.

The reduced demand should be calculated due to the hysteretic damping β_H, the viscous damping added due to the TADAS devices β_V, and the inherent damping β_I. The inherent damping β_I is assumed to be 3% following the recommendations of the ASCE 7-16 method. β_H and β_V are computed as:

$$\beta_H = q_H(0,64 - \beta_I)(1 - \frac{1}{\mu}) \tag{4}$$

$$\beta_V = \frac{E_D}{4\pi E_{so}} \tag{5}$$

where E_D is the energy dissipated in one hysteresis loop and E_{so} is the maximum strain energy. q_H represents the adjustment factor by the throat of hysteresis and μ is the ductility of the building.

The concentrated plasticity methodology was used through a force–deformation curve, following FEMA-356 [31], to represent the non-linear behavior of the structural elements as shown in Fig. 5. The curve starts linearly from an unloaded state (A) to an effective yield (B). Then, the stiffness decreases to point C, where the resistance matches the nominal strength. Beyond C, there is a sudden resistance drop, leading to reduced resistance (D) and eventual loss (E). CD represents the initial failure, while DE is the residual strength. FEMA specifies these points to assess hinge rotation in reinforced concrete and steel elements, establishing criteria for hinge performance between B and C, including immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The fiber model was used to define the plastic hinges (see Fig. 6) of the sections of reinforced concrete and steel beams and columns. The properties obtained are assigned to the sections of the beam and column at a distance of H/2 from the end of the element, where H represents the height of the beam or column. For the main structure analysis, plastic hinges and the exclusive involvement of PRM (Moment Resisting Frames) in each analysis direction are considered.

The deformation limits were defined using ATC-40 [30] and FEMA-356 [31]. Performance limits can be divided into two main categories: (1) global or structural limits and (2) local or element limits. Global limits encompass requirements related to the structure's ability to handle gravity loads, resist lateral loads, and limit lateral deformations. In cases where an element loses its capacity to support gravity loads, the structure must possess the ability to redistribute those loads to

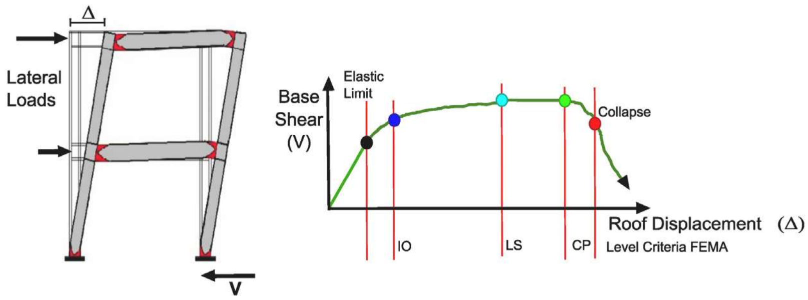


Fig. 3. Typical pushover curve.

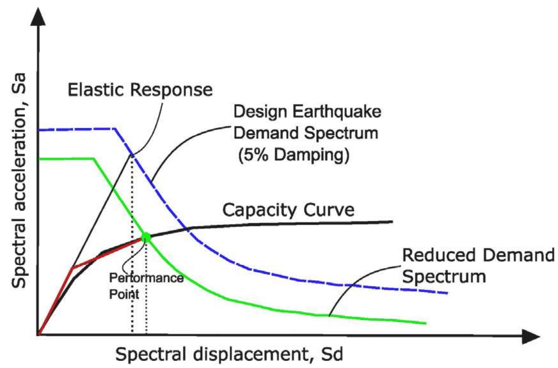


Fig. 4. Capacity spectrum method.

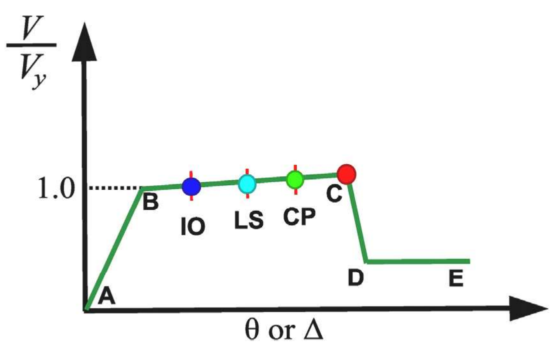


Fig. 5. Typical force deformation curve.

Table 1
Deformation limits for each performance levels (ATC-40).

Immediate occupancy	Damage control	Life safety	Structural stability
0.01	0.01–0.02	0.02	0.33 Vi/pi

other elements. In addition, it is necessary to check lateral deformations against the limits specified in Table 1, where the maximum drift is defined as the drift between floors at the displacement of the performance point. On the other hand, element limits are often influenced by non-structural factors and potential component damage. These limits are related to the response of structural elements, such as beams and columns, and are determined by the capacity for plastic hinge rotations.

2.3. Sizing of TADAS yielding dampers

A TADAS device [17] corresponds to a triangular steel plate, as observed in Fig. 7. The damper works in bending because its thickness is positioned parallel to the beam axis and it has the possibility of rotation on its bottom side. The development of inelastic strains for the movement of an earthquake leads to energy dissipation during repeated cycles of load and discharge. Variation of the width in the devices was defined to produce a higher level of deformation during an earthquake event. Fig. 8 shows a typical installation scheme of TADAS devices on a concrete frame, which is composed of a set of plates to guarantee the energy dissipation necessary for the design earthquake. Dareini et al. [23] show from analytical examples that adding damping and stiffness through TADAS devices allows modification of the shear force at the base, the drifts, and the internal forces in the main structural elements. Likewise, its installation mechanism does not cause an effect from vertical loads.

Table 2
Equations that describe the performance of the TADAS device.

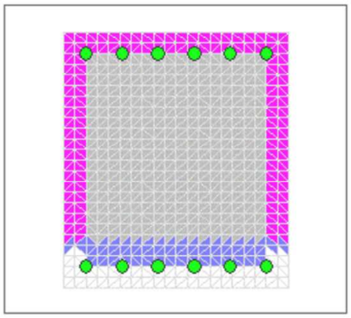
Parameter	Equation
Yield moment M_y	$M_y = nf_y(\frac{bt^2}{6})$
Yield force V_{Ty}	$V_{Ty} = (\frac{nf_y bt^2}{6h})$
Yield rotation θ_y	$\theta_y = \frac{2f_y h}{Et}$
Yield displacement Δ_y	$\Delta_y = \frac{f_y h^2}{Et}$
Ultimate moment M_u	$M_u = nR_t f_y(\frac{bt^2}{4})$
Ultimate force V_{Tu}	$V_{Tu} = R_t \frac{nf_y bt^2}{4h}$
Ultimate rotation θ_u	$\theta_u = \mu \frac{2f_y h}{Et}$
Ultimate displacement Δ_u	$\Delta_u = \mu \frac{f_y h^2}{Et}$

Metallic hysteretic devices have become very popular because their hysteretic behavior is very stable and easy to characterize because the material is quite reliable. In Fig. 9 you can see a bilinear curve that represents the elastoplastic behavior of the TADAS device. After reaching the yield point, the device undergoes strain hardening, obtaining large strains and developing a large capacity for energy dissipation. According to Tsai et al. [17], the equations that describe the hysteretic behavior of a TADAS device can be summarized by the equations shown in Table 2, where n is the number of plates of the TADAS device, b is the width of the TADAS device, and t and h refer to the thickness and height of the plate, respectively. Concerning steel, the following are considered: the coefficient of overstrength R_t, the ductility μ, the yield strength f_y, and the modulus of elasticity E. The variables that intervene in the behavior of TADAS devices are related to the geometry and properties of the material. Thus, it is possible to simplify the manufacturing process without complex technology.

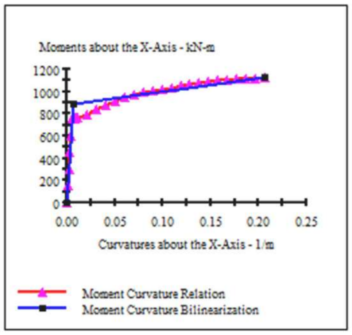
In the following, a procedure for the design of steel and concrete buildings with TADAS devices is described, which is based on the Colombian Seismic design code NSR-10 [5], the American Standard code ASCE 7-16 [27], and the previous work of Oviedo et al. [28] and Tsai et al. [17]. This procedure was first reported by Cano [32] and Caballero [33], as follows:

1. Seismic design requirements for the main structural system are established according to the Colombian design code NSR-10 in Section 2.1.

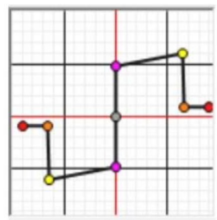
2. Plastic hinges are assigned at the extreme of each structural element that makes up the moment-resisting frames. The building



SECTION	TYPE
V_DIS-1.5%	BEAM
WIDTH (m)	0.5
HEIGTH (m)	0.6
COVER (m)	0.04
Bottom reinforcement	6 # 9
Top reinforcement	6 #9
Yielding moment (+) (kN-m)	883.7
Yielding moment (-) (kN-m)	883.7
Ultimate moment (+) (kN-m)	1122.3
Ultimate moment (-) (kN-m)	1122.3



a)



	Moment / Yielding Moment	Rotation (Rad)
E-	-0.20	-0.0867
D-	-0.20	-0.0640
C-	-1.27	-0.0620
B-	-1.00	0
A	0	0
B	1.00	0
C	1.27	0.0620
D	0.20	0.0640
E	0.20	0.0867

b)

Fig. 6. A typical plastic hinge for reinforced concrete building of 12-story in (a) Fiber model and (b) SAP2000 software.

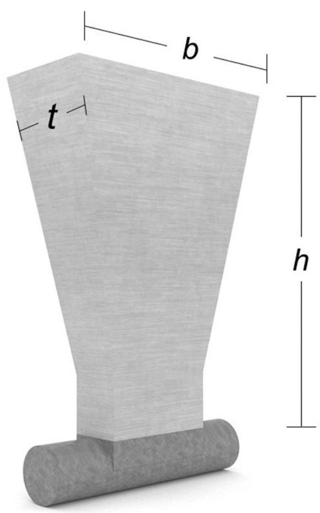


Fig. 7. Typical schematic of a TADAS plate.

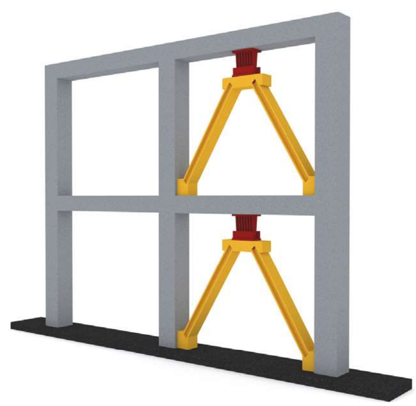


Fig. 8. Typical TADAS devices in a frame.

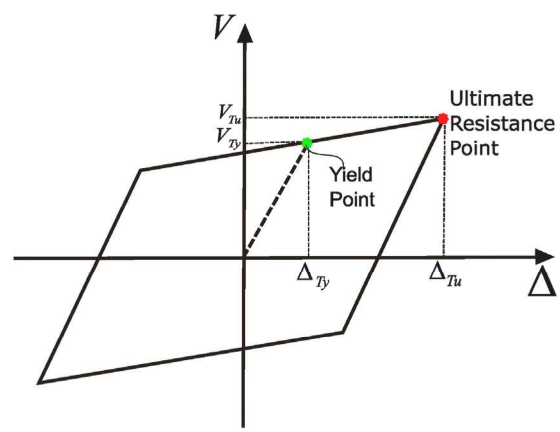


Fig. 9. A typical hysteretic loop for a TADAS device.

capacity curve is determined through a pushover analysis. The capacity curve for each of the stories of the building is built to obtain the shear of the yield (V_{Fy}) and the displacement of the yield of the main frames (Δ_{Fy}) of the story.

3. Using Ref. [28], the drift ratio(υ) and shear ratio (β) factors are defined. In this paper $\upsilon = 0.40$ and $\beta = 0.35$ is used. Such values will be used in the computation of the design forces (V_{Dy}) and the design displacement (Δ_{Dy}) of the damper system, composed of TADAS devices and the braces, as given by:

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

$$\Delta_{Dy} = \upsilon \Delta_{Fy} \quad (6)$$

4. The total stiffness of the damper system (K_d), composed of TADAS devices and the braces, is determined in each story using