BEST RETROFIT BASED ON ENERGY DISSIPATION FOR A BUILDING SUBJECTED TO VARIOUS STRONG EARTHQUAKES

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Abstract

A Mexico City building built in 1958 located on soft soil is analyzed, which has been damaged by the 1985 Mw 8.1 and 2017 Mw 7.1 earthquakes. Buildings in the same area and similar structural systems collapsed in these earthquakes (210 in 1985 and 38 in 2017). To perform the mathematical model, a structural identification was carried out using a rebar locator and a concrete rebound hammer. The response of the building subjected to these earthquakes was analyzed, finding the changes of the stiffness of the building when plastic hinges were formed, as well as the adjustment in the period of the structure. The analysis performed are nonlinear time-history. The response of the structure shows considerable damage to several elements being at risk of collapse with another earthquake, the reason why the building is uninhabited. Therefore, reinforcements are proposed with energy dissipation which gives the building the capability to resist another earthquake like the experienced in 1985 or 2017. To determine which system is the most convenient in this case, the response of the building is compared using several energy dissipation systems; those that only provide damping to the structure, and those that provide damping and also stiffness. An important point observed was that systems that provide stiffness and damping can lead the period of the structure to coincide with the site period (2 seconds), which would cause a resonance effect, the main reason of why many near-2-second period buildings collapsed in 1985 earthquake. Therefore, it is highly recommended to carry out an analysis of this type on all buildings that have been affected by more than one strong earthquake in Mexico City.

Keywords: structural retrofit; energy dissipation; strong earthquake

1. Introduction

Due to the great disaster in Mexico City caused by 1985 and 2017 earthquakes, where there were 210 and 38 collapsed buildings respectively, this research heads towards finding the resistance degradation of structural elements. Strong earthquakes lead to structural elements to yielding, which will suffer a decrease of stiffness and will decrease their capacity for upcoming seismic events. To study the above, a building located on soft soil in Mexico City and present in 1985 and 2017 earthquakes is analyzed. The building erected in 1958 will difficultly fulfill the actual norm [1]. The main purpose of this research is to propose a rehabilitation to the building in order to pass the regulations of the local norm, said norm was updated after de 2017 earthquake. Worth mentioning that many buildings with similar characteristics in the site collapsed due to de ground site period of 2 s.

A field investigation was performed in order to capture the characteristics of the different structural elements that conform the building. Information collected is used to create a mathematical model the closest to reality as possible. The main characteristics searched for were the overall dimensions of the structure's elements and steel reinforcement, and the concrete compressive strength. Many buildings affected by past earthquakes have been rehabilitated increasing their stiffness, but in this study, a rehabilitation based an energy dissipation devices is proposed. Two types of energy dissipation devices are used: fluid viscous dampers (FVD) and triangular added damping and stiffness (TADAS). The first will add damping to the structure, while the second in addition to increase structural damping also adds stiffness to the structure.

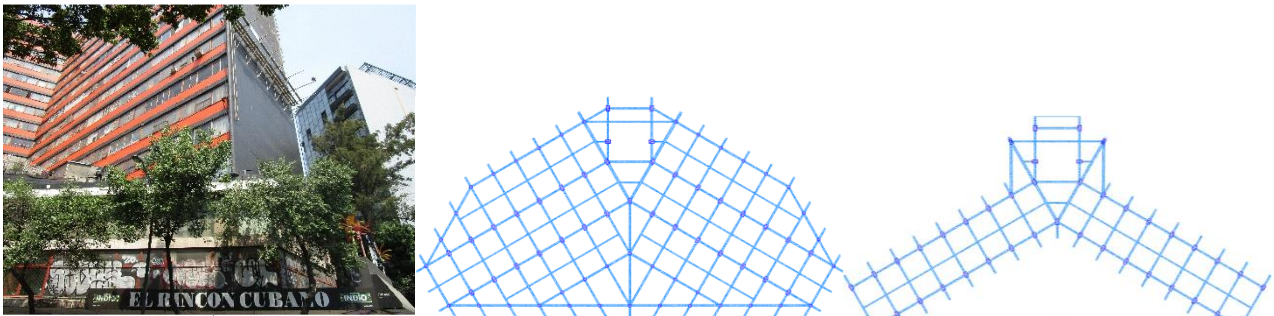
Structural response of both solutions will be compared with the building without rehabilitation, using fast nonlinear analysis (FNA) [2] and subjecting the structure to three ground motions: the two earthquakes that already affected the building and a third earthquake that matches the maximum expected earthquake proposed by the newest norm. It seeks to use the least amount of energy dissipation devices as to not make the rehabilitation more expensive. The proposed analysis is a manner to encounter resistance of damaged buildings and their best rehabilitation, not only for buildings in Mexico City but in all locations with high seismic hazard.

2. Building Description

The studied structure is a 17-story and 2 basements building located at *Avenida Insurgentes Sur* #300, Mexico City, known as *Condominio Insurgentes*, it also has a helipad which is three level above the roof. It was built in 1958 and was one of the tallest buildings of its time. After the 1985 earthquake, the building suffer some damage so several residents left, and after the 2017 earthquake the building is almost uninhabited. The first three stories have a slight shape of a triangle, but what makes the building stands out is its shape in "V" in the remaining 14 levels. Fig. 1a shows a photograph of the south side of the building, while Fig. 1b and Fig. 1c presents the plan layout of the building.

2.1 Field Investigation

A site visit was performed to obtain as-built information: building configuration and conditions, building and structural element dimensions. To have greater reliability about structural elements, a rebar detector and a rebound hammer were used to obtain steel reinforcement arrangement and concrete compressive strength respectively. With the visit it was possible to confirm the structure of the building is based on concrete frames and walls. Concrete walls were only found from stories 4 to 17 in the outer line of the building. It was also possible to investigate that the ground floor is dedicated to commerce, and the remaining 16 floors are residential and office usage.



a) b) c)

Fig. 1 – a) Outside view of the building, b) layout of ground level, c) layout of 5th level

Using the rebar detector (Fig. 2a) is was found that concrete frames have #8 rebar, concrete slabs are 15 cm thick in all floors, and concrete walls are 20 cm thick. Both concrete columns and beams have poor confinement, so they do not fulfill the requirements for ductile members. With the help of the rebound hammer (Fig. 2b), it was possible to know that the concrete compressive strength is 400 kgf/cm^2. Table 1 shows the different section dimensions found in the field investigation.

Table 1 – Section of concrete frames

Levels	**Column 1** cm	**Column 2** cm	**Column 3** cm	**Column 4** cm	**Beam 1** cm	**Beam 2** cm	**Beam 3** cm	**Beam 4** cm
1 - 2	80 X 80	65 X 130	Ø 65	Ø 65	70 X 80	45 X 75	30 X 75	20 X 70
3 - 5	65 X 130	Ø 65	Ø 65	-	70 X 80	45 X 75	30 X 75	20 X 70
6	65 X 130	Ø 65	-	-	70 X 80	45 X 75	30 X 75	20 X 70
7 - 9	65 X 105	-	-	-	20 X 70	30 X 80	45 X 90	-
10 - 13	55 X 95	-	-	-	20 X 70	30 X 80	45 X 90	-
14 - 19	45 X 85	-	-	-	20 X 70	30 X 80	45 X 90	-

a) b)

Fig. 2 – a) Rebar detection and b) Rebound hammer test on a concrete column

3. Mathematical Model

A mathematical model was developed using SAP2000 software [3] using the information collected in the field investigation. Building administrators mentioned that foundation is based on concrete piles and grade beams, which can also be inferred by the height and the fact that it is found in the *lake zone* of Mexico City. For this reason fixed restriction was applied to base of the structure and no soil-structure interaction was considered. A comparative between the building and the mathematical model can be observed in Fig. 3.

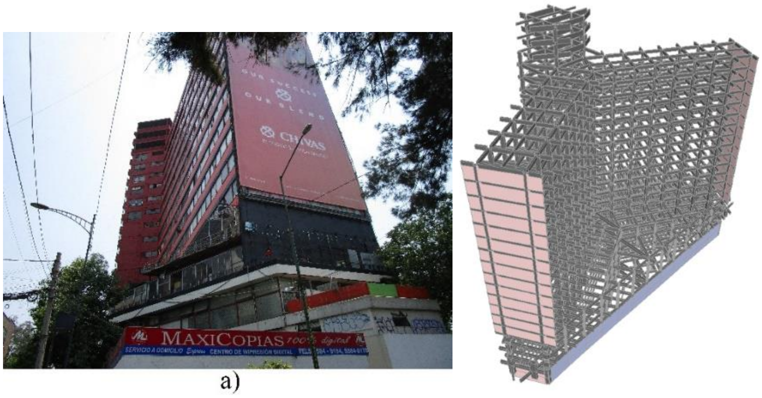


Fig. 3 – a) Building photograph, b) mathematical model

3.1 Permanent and Variable Loads

With the evidence found on the building and the different arrangements that nonstructural elements have at each floor, besides a 425 kg/m^2 due to floor slab and terminations, a 250 kg/m^2 of dead load was applied in the residential and office zone. As mentioned previously, there were 4 different usages of the building, in the mathematical model a total of five different live loads were applied. As specified by the Mexico City standard there is a maximum live load (W_m) for gravity loads and instantaneous live load (W_a) for seismic loads. Both types of loads for each usage are:

1. Garages and parking lots (basement): W_m = 250 kg/m^2, W_a = 100 kg/m^2
2. Commerce (ground level): W_m = 350 kg/m^2, W_a = 315 kg/m^2
3. Office (level 1-16): W_m = 250 kg/m^2, W_a = 180 kg/m^2
4. Passages for free access to the public (all levels): W_m = 350 kg/m^2, W_a = 150 kg/m^2
5. Roof: W_m = 100 kg/m^2, W_a = 70 kg/ m^2

3.2 Material Nonlinearity

As mentioned be the complementary technical standard of concrete structures, material nonlinearity can be considered by modeling plastic hinges. These plastic hinges can be developed by using the moment-rotation diagram (backbone curve) [4]. In the SAP2000 program, plastic hinges were modeled using MultiLinear Plastic links located at the end of beams and columns. In the case of reinforced concrete elements, the behavior can be represented using a degrading hysteretic loop from Takeda or Pivot models [5, 6]. The backbone curve used in this research was developed using FEMA-356 methodology [7], moment rotation plot of a beam considered is shown in Fig. 4.

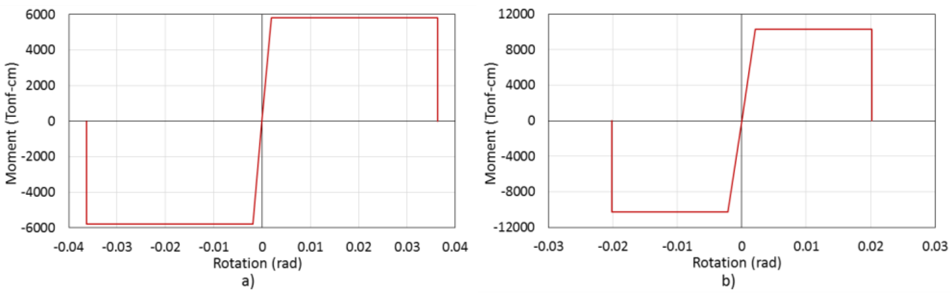


Fig. 4 – Moment-rotation diagram a) beam, c) column

3.3 Modal Analysis

A Ritz-Vector analysis was used to determine the vibration modes of the structure. This type of analysis is suggested when performing a time-history analysis. The results of this analysis show that the first vibration mode is in the short direction (Y) of the building, modes two and three were in X direction and torsional respectively. The vibration modes were computed using an effective inertia moment of beams of 0.5 (as marked by the Mexico City standard) and their period are shown in Fig. 5.

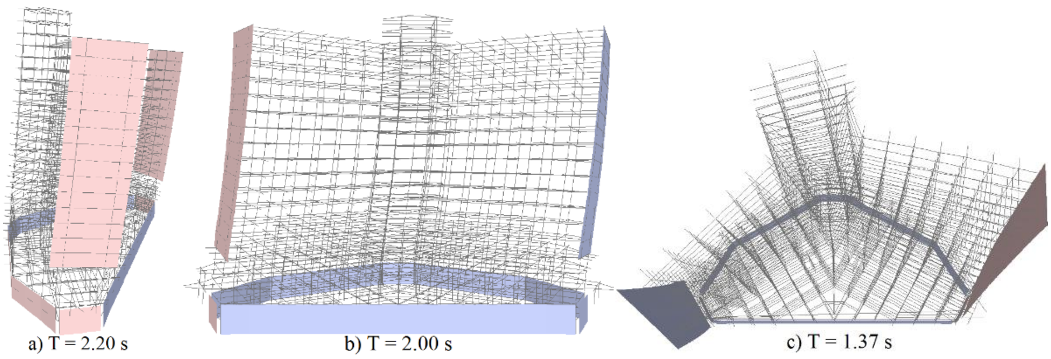


Fig. 5 – Results of modal analysis a) first, b) second, and c) third mode

4. Past Earthquakes Analysis

As mentioned before, major earthquakes have affected this building in 1985 and 2017. Even though there has been many other minor earthquakes, only these two have been used to investigate the damage in the building since these two earthquakes were the ones that caused great destruction in Mexico City. The 1985 earthquake known as *Michoacan* earthquake, occurred on September 19, 1985 off the Pacific Coast of Mexican state Michoacan (320 km from Mexico City) with an 8.0-magnitude. The 2017 earthquake also occurred on September 19 with epicenter in Mexican state Puebla (123 km from Mexico City) with an 7.1-magnitude.

The records used in this research were provided by the Seismic Instrumentation Unit of Engineering Institute of the UNAM [8]. The accelerograms used were registered at a nearby building with the same soil type to ensure the ground motion to be as realistic as possible. The earthquake records and the elastic response spectrum with 5% damping are shown in Fig. 6.

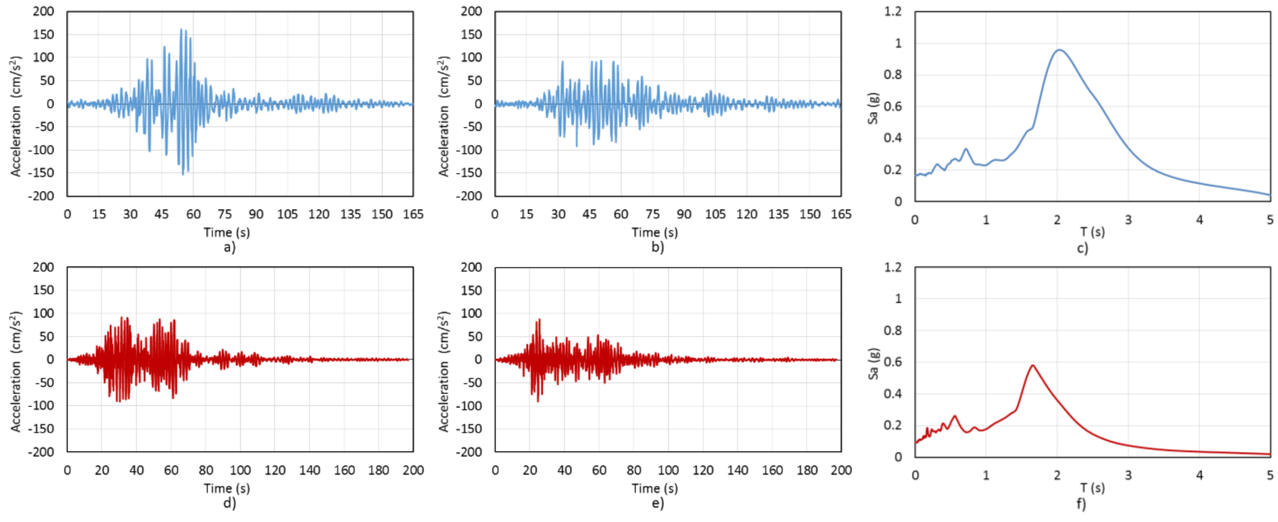


Fig. 6 – Earthquake records a) 1985 N90E, b) 1985 N00E, c) Response spectrum 1985, d) 2017 N90E, e) 2017 N00E, f) Response spectrum 2017

The first response analyzed were the roof displacements. Both earthquakes were set one after another to review how the behavior change step by step throw both earthquakes. As can be seen in Fig. 7a displacements in the X direction were larger in 2017 earthquake, as result of the damaged suffered in the 1985 earthquake. On the other hand, displacements in the Y direction were the same on both earthquakes remembering that this is the most flexible direction, worth mentioning that the 1985 ground motion was stronger than the 2017 one.

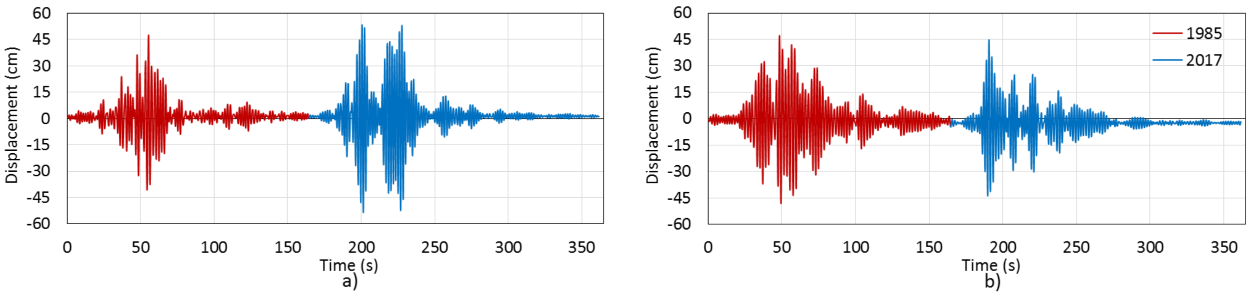


Fig. 7 – Roof displacement a) N90E, b) N00E

To investigate the behavior of the rehabilitated building, it has to start from the actual damage. The structure was subjected to a FNA with the previous accelerograms [2]. The structural damage was monitored using the plastic hinges modeled. Fig. 8 shows the behavior of the most damaged elements, where it can be seen that yielding with stiffness degradation is present in the elements.. After this case the structure will be subject to a third seismic record. In order to consider the previous damage of these two records, a new backbone curve (dotted line obtained at the end of the analyses) is considered for the elements, having as new yielding points the absolute maximum rotation of the elements [4].

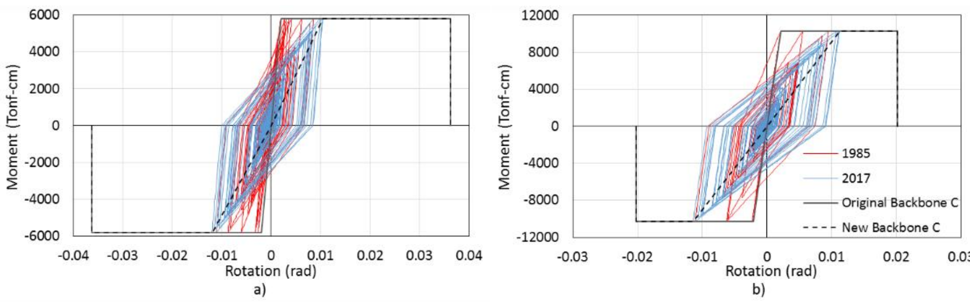


Fig. 8 – Hysteretic loop after 1985 and 2017 earthquakes a) beam, b) column

5. Seismic Demand for a Possible New Earthquake

A renovated norm for Mexico City was published after the 2017 earthquake. The considered seismic demand for the a new possible earthquake is based on the new code [1]. This standard uses SASID software [9], which has a seismic regionalization specific for Mexico City. Entering the coordinates of the building, importance factor B, irregularity factor 0.8, Seismic behavior factor 2, and hyperstaticity factor 1.25; the program gives back the design response spectrum. The specific design response spectrum and soil acceleration variation in Mexico City is shown in Fig. 9. For the building retrofit performance, a time-history analysis is performed. The 2017 earthquake record is calibrated to match the reduced design response spectrum that is consistent to the new standard. The accelerograms are shown in Fig. 10.

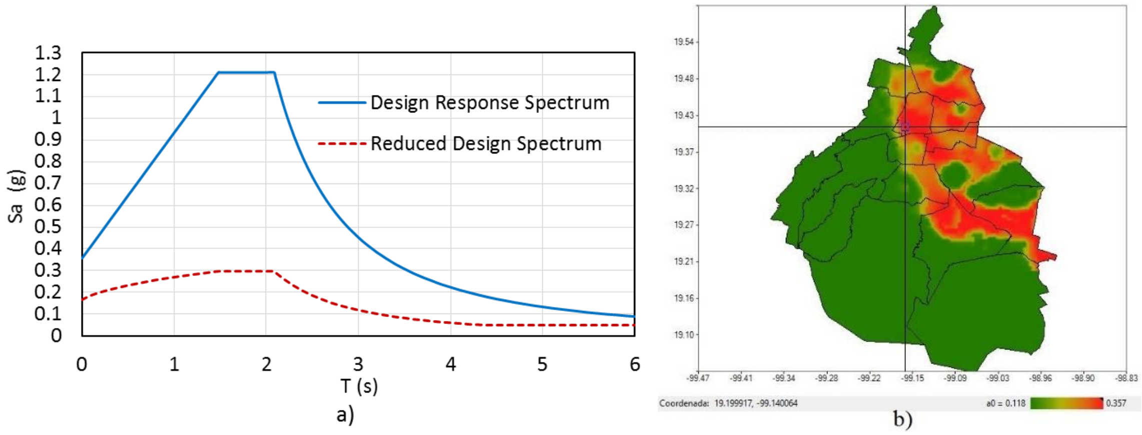


Fig. 9 – a) Design Response Spectrum and b) seismic regionalization [9]

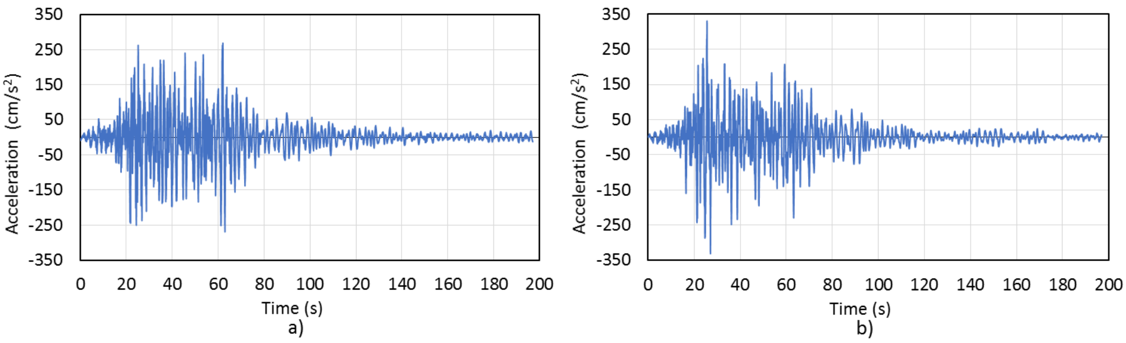


Fig. 10 – Accelerograms a) N00E direction, b) N90E direction

6. Retrofit Based on Energy Dissipation

As mentioned before, two energy dissipation devices will be studied as a retrofit measure. Both proposals are explained below.

6.1 Fluid Viscous Dampers (FVD)

The first energy dissipation device studied is the well-known FVD, which only provides damping to the structure. These devices had been used in the past to rehabilitate damaged buildings, having excellent results [10, 11]. The location in plan was selected to have the least interference with the existing usage of the building. To distributethe dampers along the height of the building a sequential analysis was performed based on story drifts, placing FVDs in the story with the highest drift in each iteration. Different configurations were proposed, after some iterations the proposed configuration is to place the FVD in double diagonal. Fig. 11 shows a level of the mathematical model with FVD. the devices were modeled using Damper Link property in SAP2000. For the nonlinear cases the damping coefficient used is 80 tonf-s/cm, and a damping exponent of 0.3.

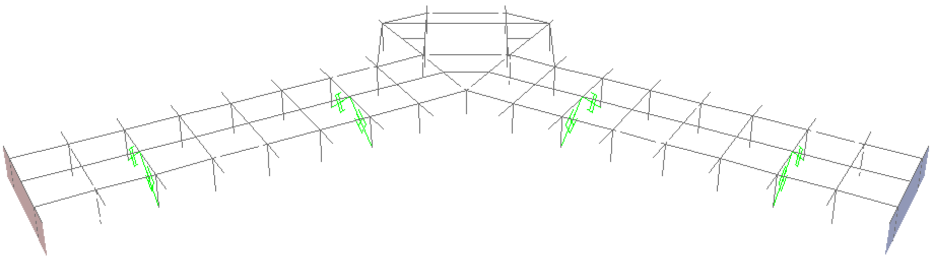


Fig. 11 – FVD in mathematical model

6.2 Triangular Added Damping and Stiffness (TADAS)

Unlike FVDs, TADAS devices are not a usual method for structural retrofit [12]. However, for Mexican earthquakes the stiffening of a 2-second period structure can be as effective as add damping since a reduction of the period will imply lower spectral accelerations. Thus, it will be studied if energy dissipation that in addition of increasing damping also adds stiffness is or not a good method for structural retrofit. TADAS devices were modeled using Plastic Wen Link property available in SAP2000 [13]. To determine the properties equations (1) - (4) were used [14].

$$K_d = \frac{N\ E\ b\ t^3}{6\ h^3} \tag{1}$$

$$P_y = \frac{F_y\ N\ b\ t^2}{6\ h} \tag{2}$$

$$P_p = \frac{F_y\ N\ b\ t^2}{4\ h} \tag{3}$$

$$\Delta_y = \frac{F_y\ h^2}{E\ t} \tag{4}$$

Where K_d is the lateral elastic stiffness, N is the number of triangular plates, E is the modulus of elasticity, t is the plate thickness, b and h are the base width and height of the triangular plate, P_y is the yielding strength, P_p is the plastic strength, and Δ_y is the yield displacement. Hence, a TADAS device with 8 1½ in plates, 25 cm wide and 30 cm high made of ASTM A36 steel is proposed. Properties used to define the TADAS device are: effective stiffness 13.9 tonf/cm, stiffness 139 tonf/cm, yield strength 40 tonf, post yielding stiffness ratio 0.01, and yielding exponent 1. The TADAS devices were placed in a chevron configuration, distributing them along the height using the same approach that FVD, with a sequential analysis. Fig. 12 shows the proposed configuration of TADAS devices at one level, the image shows six TADAS per level, only this level has six TADAS the remaining levels only have 4 TADAS per level.

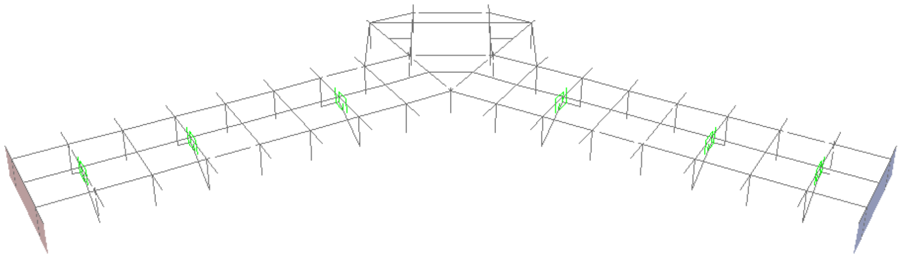


Fig. 12 – Mathematical model with TADAS

7. Analysis Results

Hereafter, the analysis results between the structure with and without rehabilitation will be compared. Mainly the results of moment-rotation diagrams, displacements, story drifts, and reactions will be shown. So all results will compare the three cases: no retrofit (NR), retrofit using FVD (FVD), and retrofit using TADAS (TADAS). First of all, roof displacement is compared. As seen in Fig. 13 both retrofits reducing roof displacements by 40% compared to the building with No Retrofit.

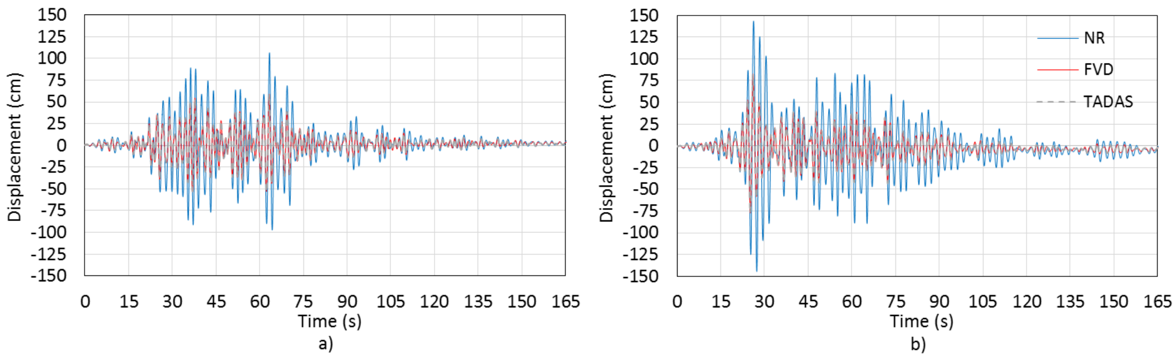


Fig. 13 – Roof displacement a) X direction, b) Y direction

As mentioned before the location of energy dissipation devices was based on story drifts. Both types of retrofit required energy dissipation devices in levels 7, 8 and 9, the difference is that in the retrofit with TADAS extra devices were placed on the 8th level. The allowed drift is 15‰ when designing with linear elastic models, but in Mexico City code it is permitted to increase the allowable drift to 18.75‰ [1] when using a nonlinear time-history analysis. Fig. 14 shows the story drifts of the building, as it can be observed the structure without rehabilitation overpasses the allowed drift by a 45%. After the reinforcement, both retrofit measures meet the standard.

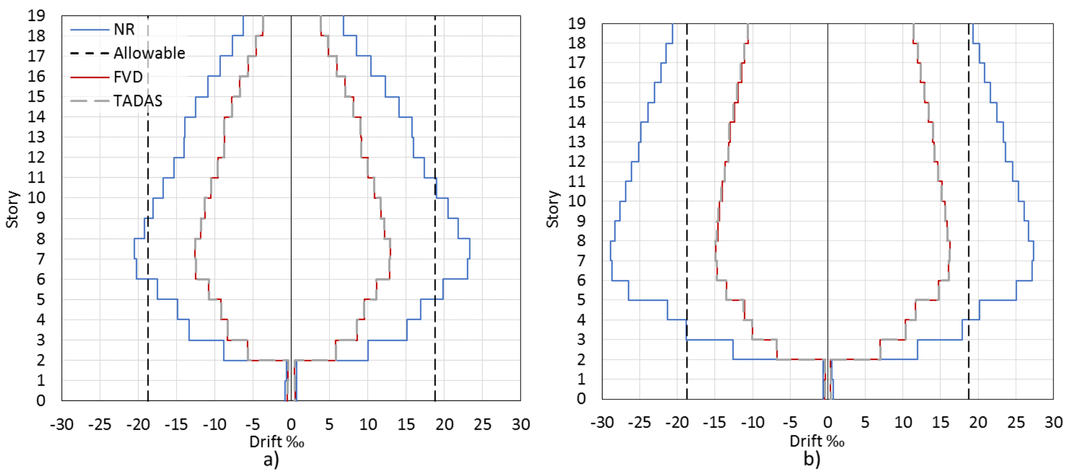


Fig. 14 – Story drifts a) X direction, b) Y direction

Another benefit of the retrofit with energy dissipation is the reduction of the stresses in foundations. As it can be noticed in Fig. 15, all reactions decreased with the use of FVD and TADAS. In this case were foundation details are unknown, transmitting lower loads to the foundation will decrease the probability of settlements, especially in this zone with soft soil. Taking a look at plastic hinges in Fig. 16 shows that the element will reach collapse without retrofit, but incorporating energy dissipation ensures a better behavior of the element, that will not be near collapse.

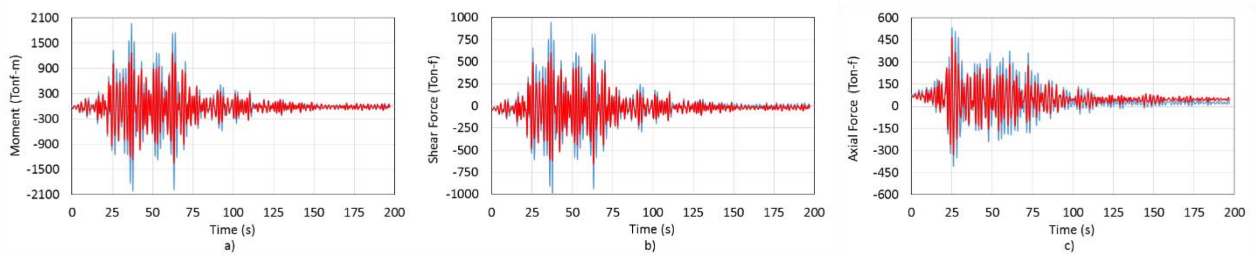


Fig. 15 – Reactions a) Moment, b) Shear, c) Axial force

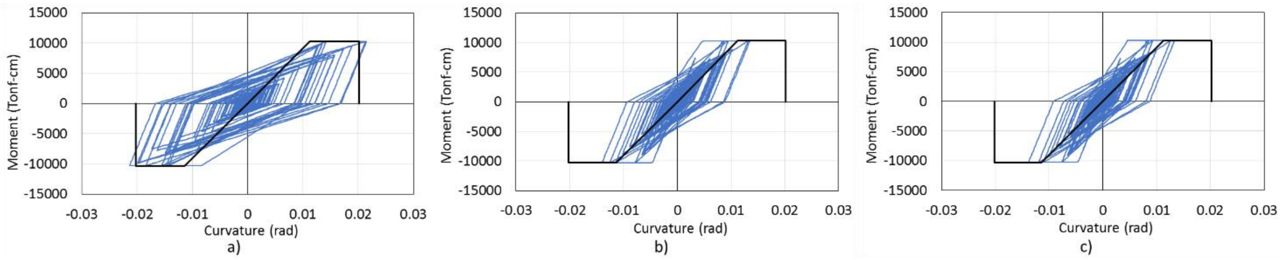


Fig. 16 – Moment rotation diagrams a) No retrofit, b) FVD, c) TADAS

Finally, about the energy dissipation devices, a total of 24 FVD and 14 TADAS were used for each retrofit respectively. The greater amount of FVD is due to that they were placed in double diagonal. FVD had a maximum of 175 tonf axial force, 3.8 cm deformation and 110 cm/s velocity. both plots are shown in Fig. 17. While TADAS had a maximum force of 46 tonf and also 3.8 cm of deformation, hysteretic loop of TADAS are shown at Fig. 18. Fig. 19 shows a detail of how to perform the retrofit for the building, due to the poor confinement found in the structure reinforcement, the use of carbon fiber is proposed [15].

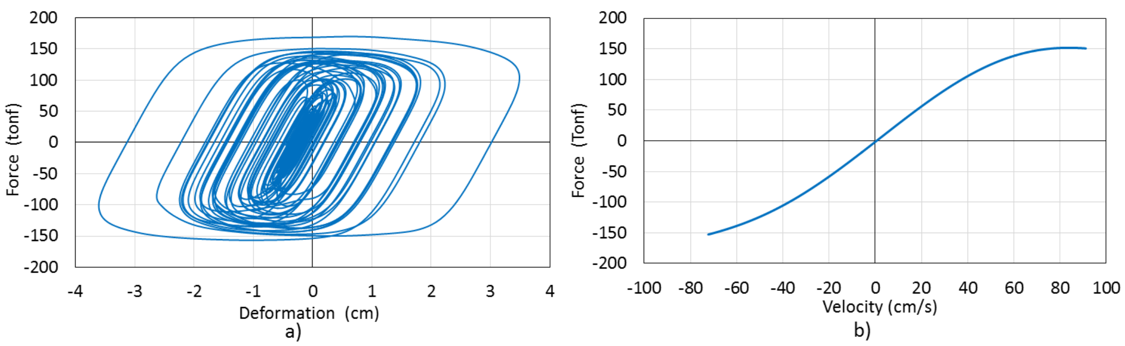


Fig. 17 – FVD a) hysteretic loop, b) force-velocity

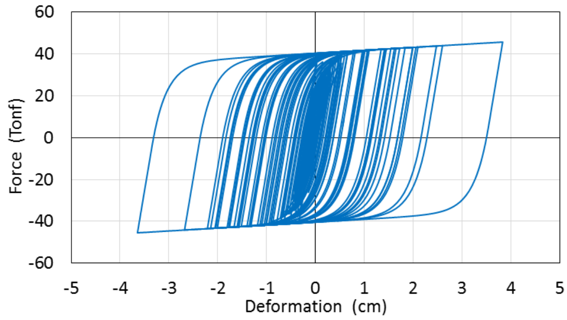


Fig. 18 – TADAS hysteretic loop

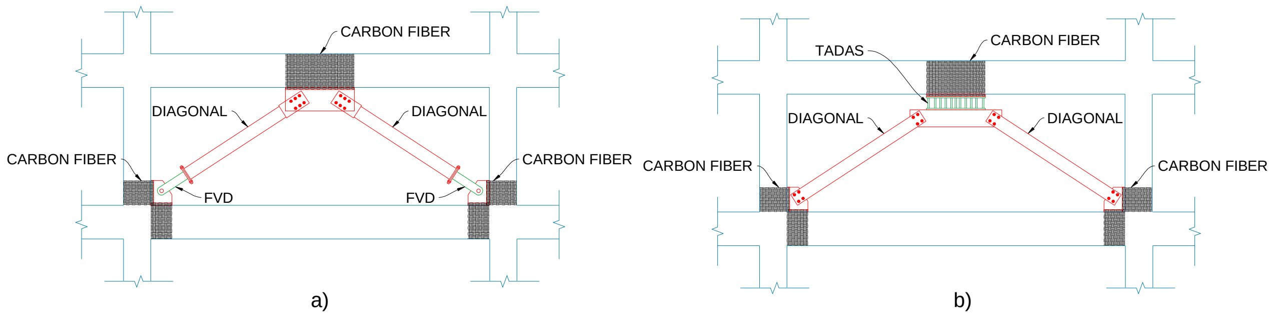


Fig. 19 – Installation detail a) FVD, b) TADAS

8. Conclusions

The main purpose of this paper was to assess the seismic retrofit of a building using two different types of energy dissipators, FVD and TADAS. As the first incorporates damping to the structure, the second in addition to increase damping also adds stiffness to the structure. Damage inflicted on the structure by past earthquakes was monitored though plastic hinges located at each end of elements. FNA analyses were performed for both past and “future” seismic events.

As noticed in the results, a possible earthquake with the spectral characteristics of the new standard code will probably lead the building to collapse, so a seismic retrofit has to be conducted to improve the structure´s safety. A building with these characteristics: age, structural configuration, irregularity, and especially the zone where it is located, make of this an ideal building to incorporate energy dissipation in order to reduce the probability of collapse. As seen in this research, with a total of 24 FVD or 14 TADAS retrofit, this building passes from not achieving the standards story drift to be 20% below the allowed drift. Furthermore, loads transmitted to the foundation have a 30% decrease that is important in this case due to the uncertainty of foundation conditions.

All energy dissipation devices were placed in the short direction of the building, where displacements are larger. Due to the irregularity of the building, the contribution of FVD and TADAS were not only in the Y direction but also in X direction, as it is shown by the story drifts presented above. By carrying out a sequential analysis, 8 FVD and 4 TADAS per level were used, with exception of 8th-level with TADAS that had larger story drift so 6 TADAS were placed instead.

The cost of the retrofit can be higher when using FVD, due to the cost of the devices, because the number of diagonals and connections is greater in the retrofit with TADAS. One point in pro of TADAS devices is that lower forces are transmitted to the joint, in a structure with the confinement found in site may have to be more robustly reinforced. Despite de probable lower cost and forces transmitted to the structure there is an important point that makes FVD the best retrofit for this building. Because the ground on which the structure is built is the lake bed, a high site period of about 2 s makes this structure with the incorrect stiffness enter in resonance which will be catastrophic for this building, for that reason the least amount of TADAS was proportioned to the structure in order to not lower much is period and avoid resonance.

9. Acknowledgments

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10. References

[1] Reglamento de Construcciones para el Distrito Federal (2017). Ciudad de México.

[2] Wilson EL (2000): *Three-Dimensional Static and Dynamic Analysis of Structures. A Physical Approach with Emphasis on Earthquake Engineering.* Berkeley: Computers and Structures, Inc.

[3] CSI (2014): *SAP2000. Integrated Finite Element Analysis and Design of Structures.* Berkeley: Computers and Structures, Inc.

[4] Haselton CB, Liel AB, Lange ST, Deierlien GG (2008): *Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings.* University of California, Berkeley.

[5] Takeda T, Sozen MA, Nielsen NN (1970): Reinforced Concrete Response to Simulated Earthquakes. *Journal of the Structural Division,* **96** (12), 2557-2573.

[6] Dowell RK, Seible F, Wilson EL (1998): Pivot Hysteresis Model for Reinforced Concrete Members. *ACI Structural Journal,* **95** (5), 607-617.

[7] FEMA (2000): *FEMA 356 Prestandard and Commentary for Seismic Rehabilitation of Buildings.* Reston: Federal Emergency Managment Agency.

[8] Instituto de Ingeneria UNAM (2014): *Base de Datos de Registros Acelerográficos de la RAII-UNAM*. Retrieved June 24, 2019, from https://aplicaciones.iingen.unam.mx/AcelerogramasRSM/Inicio.aspx

[9] Sistema de Acciones Sismicas de Diseño. (2017). Mexico City.

[10] Seamer B, & Wang D: *Seismic Retrofit of the Tower of HOPE - Preservation of a Masterwork of Mid-Century Modernism.* Irvine, CA: LPA, Inc.

[11] Miyamoto K, Gilani AS, & Determan L: *Seismic retrofit and FEMA P-58 risk assessment of mid-rise soft-story concrete tower.* Los Angeles, CA: Miyamoto Internacional.

[12] Oinam RM, Sahoo DR (2017): Seismic rehabilitation of damaged reinforced concrete frames using combined metallic yielding passive devices. *Structure and Infrastructure Engineering,* **13** (16), 816-830.

[13] Wen YK (1976): Method for Random Vibration of Hysteretic Systems. *Journal of the Engineering Mechanics Division,* **102** (2), 249-263.

[14] Tsai KC, Chen HW, Hong CP, Su YF (1993): Design of Steel Triangular Plate Energy Absorbers for Seismic-Resistant Construction. *Earthquake Spectra,* **9** (3), 505-528.

[15] American Concrete Institute Committee 440 (2017). *Guide for the Design and Bonded FRP Systems for Strengthening Concrete Structures.* Farmington Hills: ACI 440.2R-17.

