Assessment of the performance of circular shaped tuned liquid dampers on the behavior of an existing building

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SUMMARY:
The integration of damping by means of including TLDs on existing structures is very appealing in a way that allows in most cases a simple installation without major additional costs. The paper describes some numerical simulations, based on approximate mechanical models, performed over circular shaped TLDs when included in an existing reinforced concrete structure designed in accordance with ancient Portuguese codes. The selection of the building studied was based on its geometric characteristics and in in-situ experimental studies previously developed. The TLDs proposed for the simulated situation will tend to be similar to the circular shaped prototypes studied during an extensive experimental program carried out at LNEC. The main results of the numerical simulations performed (SAP2000) both for the MDOF structure as well as for the MDOF structure + TLDs will be presented and analyzed.

Seismic Protection; Mechanical Models; Mitigation of Vibrations; Tuned Liquid Dampers

1. INTRODUCTION

According to the present earthquake-resistant design criteria on which rely the majority of current regulations, it is desirable that the structures are capable of developing inelastic deformation, thus presenting a non-linear behavior when subjected to intense seismic loading. This capability is based on the degree of redundancy and ductility of the structure (resistance and redistribution of effort). This implies that on the one hand, is being explored the structures inelastic deformation capacity, by imposing to the structure the dissipation of energy transmitted by the earthquake, and on the other, that is being assumed the possibility of occurrence of more or less important damages [Guerreiro 1996].

The economic impact of some intense earthquakes that have occurred recently and of the constraints arising from current design criteria, and significant limitation of increased security requirements for certain structures, led to the emergence of some alternative methods of design of structures resistant to earthquakes. These systems are commonly named seismic protection systems. The main goal to be achieved with the use of such systems is the reduction of internal forces in the structures, depending on alterations of the frequency level or increasing the energy dissipation capacity of the whole protective system + structure [Coelho et al., 2007]. The objective is to reduce the relative displacements of the structure or reducing the accelerations acting on it, or even both.

This paper aims to develop a seismic assessment of a building constructed without seismic design during the 70s of the XXth century in an island of Azores, analyzing the benefits resulting from the introduction of a seismic protection system called Tuned Liquid Damper. To analyze the benefits that such devices might have sunk in the building under study was considered a mechanical model (macro-element) implemented in a commercial finite element program. The efficiency of TLD's in improving the seismic response of a building, based on linear dynamic analyzes with particular emphasis on the evaluation of seismic vulnerability, essential for estimating the seismic performance, will be presented.

2. CASE STUDY: SOLMAR BUILDING

2.1. Structures description

The Solmar building was designed in the beginning of the 70s of the XXth century and constructed in downtown of Ponta Delgada in S. Miguel Island of Azores. Its structure is developed in height and has 22 floors above the reference plane and two in the subsoil, totalling approximately 75m in height (Fig. 2.1.). The building with an approximated y shape comprises three independent compartmentalized towers to each other by reinforced concrete walls, each of them served by lift and steps. From a structural viewpoint, the Solmar building (Fig. 2.2.) corresponds to a reinforced concrete mixed structure of portico and wall, conditioned presence of three wall-resistant cores, cited above, govern almost all the behavior of the system.

Figure 2.1. Implementation of Solmar tower in Ponta Delgada downtown

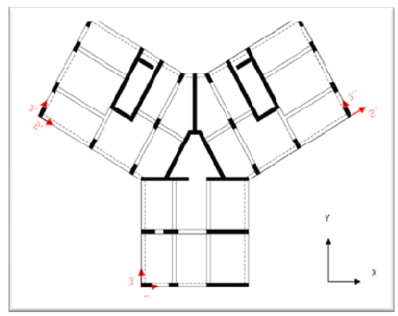

Figure 2.2. Detail of the plant of a typical floor of the Solmar tower

The resistant elements are made in reinforced concrete, with the concrete classified according to EC2 in the class C25/30. The reinforce steel is classified as A400NR. The masonry walls are made of blocks of volcanic cement compounds. Table 2.1 shows the mechanical properties considered to be the most relevant to the modelling.

Table 2.1. Mechanical properties of the materials used

Material	E (GPa)	γ (KN/m3)	ν
Reinforced concrete	30,5	25	0,2
Masonry	2,0	9,07	0,2

The terrain in which the structure is founded is characterized as type B according to the National Annex of the European regulation, EC8 [EN 1998-1, 2004]. This is mainly made up of landfills, alluvial deposits and basaltic terrain (hard rock).

2.2. Analytical modelling

Both the modeling and posterior analysis of the building structure were performed using the program SAP 2000NL [SAP2000NL, 2003], a commercial software based on finite element models. For the study presented in the following sections, which included the quantification of the seismic loading, a linear dynamic method was used.

It was considered relevant to the study account for the influence of the adjacent building to the west and south gables. Their modeling was done by placing springs on the boundary of the confining walls.

The study of these springs was made by distributing an appropriate and estimated stiffness to the various springs. Foundations were also simulated by fixed restrictions with springs. The recommended solution for their modeling was based on the simplified Vogt expressions. For the same was taken into account the maximum admission ground tension [Engil, 2006].

2.3. Dynamic Characterization

Considering the results obtained during the dynamic characterization, it was found that the vibration modes of the structure are coupled together in sets of three, which can be confirmed by the proximity of their frequencies (Table 2.2).

Table 2.2. Vibration modes, frequencies and Periods

Mode	Description	Frequencies	Periods
1	1º Mode X (mainly in X)	1.005	0.995
2	1º Mode Y (mainly in Y)	1.014	0.986
3	1º Mode Torsion	1.094	0.914
4	2º Mode Torsion	3.266	0.306
5	2º Mode X (mainly in X)	3.542	0.282
6	2º Mode Y (mainly in Y)	3.742	0.267

The first mode is identified with a frequency of 1.0054 Hz. The modal configuration shows that this is the first mode towards X, (Fig. 2.3), in which the mass participation in this direction is 52.77%. The second mode was identified with a frequency of 1.0141 Hz and its modal configuration, as shown in Fig. 2.4, reveals that this is the first mode in Y direction in which its mass participation in this direction is 54.13%. The first torsion mode was recorded at 1.0942 Hz, having a well defined configuration as shown in Fig. 2.5.

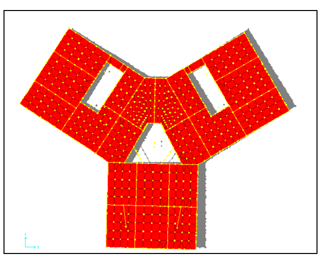

Figure 2.4. First vibration mode

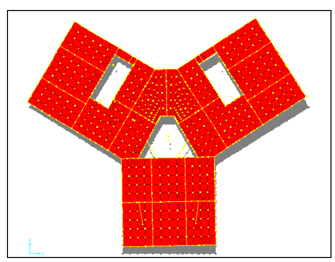

Figure 2.5. Second vibration mode

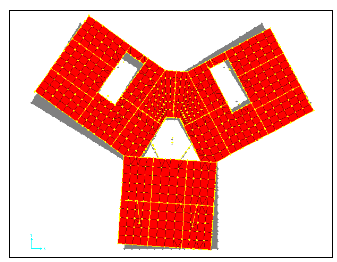

Figure 2.6. Third vibration mode

As stated, this building has already been a subject of some studies [Rocha, 2008] [Azevedo, 1986] [Tecniagra, 1975]. Table 2.3 shows a comparison between the frequencies of the various modes obtained either by experimental analysis of the building, or through reduced models or analytical models. It was found that the final calibration solution approached in terms of frequency of the tests performed on the same.

Table 2.3. Results of the modal characterization obtained for the different studies developed

	Ensaio sobre edifício		Modelos analíticos				Modelo reduzido em perspex	
Mode	In-situ excitation	Mild earthquakes	2-D (1976)	3-D (1986)	3-D (2008)	3-D (2010)	3-D	Modal configuration
1	1,09-1,1	1,07		0,71	0,88	1,0054	0,63	1X
2	1,07	1,07		0,82	0,92	1,0141	0,7	1Y
3	1,43			0,92	1,46	1,0942	0,73	1T
4	3,6	3,71		2,58	3,28	3,266	2,07	XT
5	3,6	3,71		3,05	3,51	3,5417	2,3	2T
6	4,6	4,3		3,22	4,58	3,7421	2,58	2Y

3. MECHANICAL MODELLING OF THE TUNED LIQUID DAMPERS

It is known from the literature and research projects that were previously developed, that several analytical solutions are possible for tanks of various sizes with a number of complex problems of interaction between the vibrations caused by the fluid and the deformation of the boundaries of the reservoir [Housner, 1963].

For oscillatory motions associated with reduced values of speed, from the hydrodynamic pressures on the walls of the reservoir, can lead to the answer in a simplified form as the sum of responses for each natural way, by multiplying it by cos (ωnt) or by sin (ωnt) where ωn is the natural frequency of vibration associated with the nth mode, and time t. This simplification shows that the fluid can be replaced by a mass coupled to the tank by means of a spring and introducing a determinate damping.
In this type of solution, it should be noted that the first mode corresponds to a circular tank subject to a simple translational movement [Newmark, et al., 1971].

Regarding the proposed made by Housner [Housner, 1957] and later modified by Housner [Housner, 1963] and also adapted by Newmark [Newmark and Rosenblueth, 1971] it is possible to define the main mechanical characteristics of the model adopted in this study:

$$M_0 = \frac{tanh(1{,}7R/h_f)}{1{,}7R/h_f} \times M \tag{3.1}$$

$$M_1 = \frac{0{,}71 \times tanh(1{,}8h_f/R)}{1{,}8h_f/R} \times M \tag{3.2}$$

$$H_0 = 0{,}38h_f \times \left[1 + \alpha\left(\frac{M}{M_0} - 1\right)\right] \tag{3.3}$$

$$H_1 = h_f \times \left[1 - 0{,}21 \times \frac{M}{M_1} \times \left(\frac{R}{h_f}\right)^2 + 0{,}55 \times \beta \times \frac{R}{h_f} \times \sqrt{0{,}15 \times \left(\frac{R \times M}{M_1 \times h_f}\right)^2 - 1}\right] \tag{3.4}$$

$$k = \frac{4{,}75 \times g \times M_1^2 \times h_f}{M \times R^2} \tag{3.5}$$

To calculate the resultant of the forces produced by fluid in the tank, the fluid is replaced by mass M_0 and M_1 rigidly fixed to the tank at a certain height, H_0 and H_1, and with certain stiffness, k, in accordance with the sketched in Fig. 3.1. For the calculation of H_0 and H_1 was considered α=1.33 and β=2.0, admitting hydrodynamic pressure in the walls and base of the TLD. [Newmark and Rosenblueth., 1971] The damping was determined using the empirical expression (3.6).

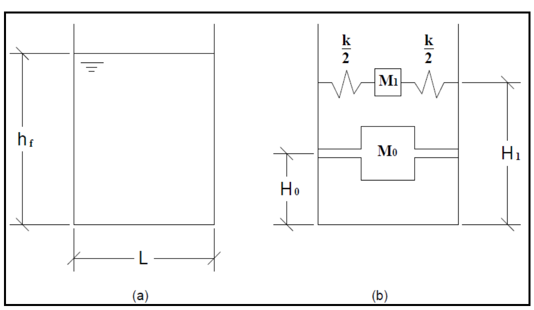

Figure 3.1. Circular TLD: (a) real prototype at rest, (b) equivalent mechanical model [Newmark and Rosenblueth, 1971]

$$\zeta_f = \sqrt{\frac{v_f \times \omega_f}{2}} \times \left[1 + \left(\frac{2 \times h_f}{h}\right) + < s\right] \times \frac{L}{h_f \times \sqrt{g \times h_f}} \tag{3.6}$$

The modeling of the TLDs on the commercial program used for the numerical simulations was made by means of a macro-element (Fig- 3.2) created for that purpose and performed in accordance with the following procedure:

i. Definition of four link elements of damper type, which concentrate all the damping and stiffness of the TLD;
ii. The static mass associated with the TLD was considered at the endpoints of the macro-element, and the dynamic mass considered in the centre joint of the macro-element.

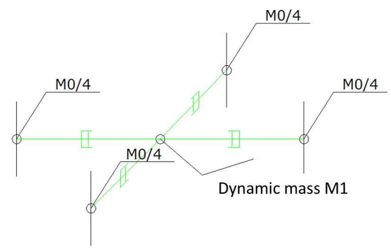

Figure 3.2. Macro-model of a TLD implemented in commercial program SAP2000NL [SAP2000NL, 2003]

4. DESIGN OF TUNED LIQUID DAMPERS FOR VIBRATION MITIGATION

As the modal analyses shown, the first frequencies are coupled. As a matter of optimizing the TLDs performance, they were tuned to a fundamental frequency of operation, in which the ratio of its frequency and the frequencies of the first three modes of the structure was not to be less than 0.9. The objective went through to get the TLD's to be implemented to have a positive effect on all the 3 modes of vibration, which has concentrated a significant percentage of modal mobilized mass. For this reason it was necessary to conceive of a TLD with the capacity to respond in both directions, longitudinal and transverse, when requested by any dynamic load.

Regarding to previous studies [Falcão Silva, 2010] it was identified that a mass ratio of between 1% and 5% represents the best performance of the TLD devices. Knowing that the building under consideration has certain area limitations that prevent the placement of the maximum values between the range indicated above, it was decided to perform simulations that mirror a realistic implementation in the building and as such not to exceed a mass ration of 3%. For the applications carried out in the scope of the work presented, and particularly in the building studied, was necessary to consider the seismic action in longitudinal and transverse directions, and the TLDs were designed to both directions request, following the procedure:

i. Due to the first 3 vibration modes have very similar frequency values, it was considered for the building frequency to be tuned 1.0141 Hz and not the 1.0054 Hz obtained for the fundamental frequency by numerical simulation. With that value it became to configure the TLDs for all three modes, within the range of ideal relationship between frequencies. Equating the frequency of the fluid's movement to the frequency of the building and remembering that:

$$f = \frac{1}{T} = \frac{\omega}{2 \times \pi} \tag{4.1}$$

ii. Using the fluid's movement frequency equation [Falcão Silva, 2010], knowing the relationship between f (Hz) and ω (rad / sec), and considering the ratio between the height of the fluid, h_f, and the radius of the tank R , obtained by an equation system resulting in the identification of h_f and R of 0.08m and 0.25m, respectively.

iii. Determination of the number of TLDs required to form the desired seismic protection system

$$N = \frac{\mu X m_e}{M}(\%) \qquad (4.2)$$

in which μ is the desired mass ratio, me is the building's mass and M is the fluid's mass.

The building studied has a total mass of 19,658 ton, however the mass to be properly used to determine the number of TLD must be the dynamic mass of the 1st vibration mode, 17 104 tons [Coelho, 2010]. Thus, in accordance with equation (4.1) it is possible to determine the number of TLD's necessary to form the seismic protection system, based on the mass ratio (see Table 4.1). The chosen percentages result of the limited space in the structure, but it is perfectly feasible to include 1% to 2%, and a 3% theoretical situation since there is the need to resort to the floor covering indented to be able to place all of the corresponding tanks.

Table 4.1. Number of TLDs needed function of imposed mass relation

μ(%)	Number of TLDs
1.0	11335
2.0	22669
3.0	34004

As each level can take up to 12000 TLD's, the number of TLD's to be applied in order to standardize all tank levels will be the presented in Table 4.2:

Table 4.2. Number of TLDs considered

Number of TLDs	μ(%)
12000	1.06
24000	2.12
36000	3.18

Table 4.3 shows the parameters obtained for a TLD to be considered for the macro-model defined based on the formulations proposed in section 3.

Table 4.3. Simplified parameters for the mechanical model

M_0 [kg]	M_1 [kg]	H_0 [m]	H_1 [m]	K [kN/m]	ζ_f
2,9	9,6	0.2004	0.2564	0.06352	0.01

With respect to its modulation in the numerical model and given the geometrical arrangement of the building, was decided to put a TLD for each tower. Each TLD represents in each position, a set of TLD's positioned in this structure for the different mass imposed (μ). In his position was taken into account the center mass of each large set that each one represents.

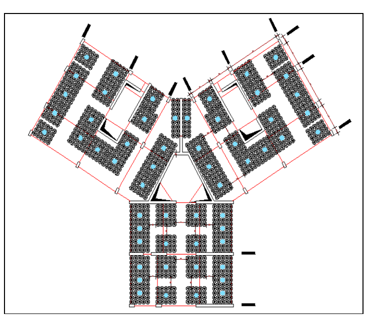

Figure 4.1. Proposed disposal of the TLD's in plant

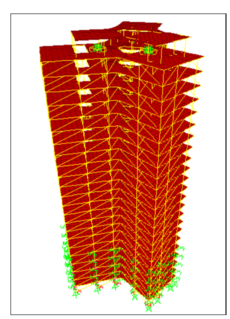

Figure 4.2. Disposal of TLD's in the mechanical model

5. SEISMIC VULNERABILITY ASSESSMENT

It was performed a seismic vulnerability analysis of the building under study with and without tuned liquid dampers devices. It corresponds to a linear dynamic analysis, where at first level are checked the maximum relative displacement between floors, based on a comparison of the obtained values with the displacement limits defined in VISION 2000, [ATC 58-2, 2002] for different seismic levels. However, if the first security requirements described are met, will also be necessary to check if columns, walls and cores of the structure resist to efforts to which will be submitted through a comparison between actuating and resistant efforts – 2nd security requirements. On the other hand, if the maximum displacement analysis achieved in the structure does not meet the safety requirements in drifts, there is no need to develop an analysis in terms of efforts, so it will be necessary to go immediately to a reinforcement solution. The definition of the seismic levels adopted for this study was based on a hazard curve for S. Miguel, Azores, provided by LNEC [Carvalho, 2008]. According to the recommendations of VISION 2000 [ATC 58-2, 2002] the different levels of seismic action are define as a function of average return period. Fig. 5.2 shows the five levels of seismic action adopted.

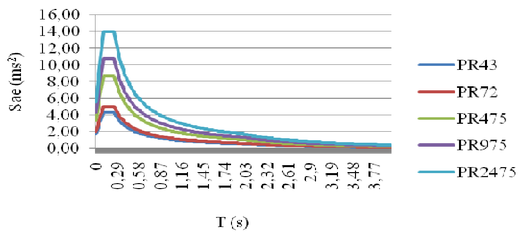

Figure 5.2. Vulnerability assessment levels

Defined the seismic action for this study was then carried out a linear dynamic analysis. First were found the maximum displacement between floors affected by the structure, and then a comparison of these values with the displacement limits defined in VISION 2000 [ATC 58-2, 2002] for different levels of seismic action was made. These define the performance levels and performance targets for buildings and are defined in what concerns to:

i. Structural damages;
ii. Damages in non-structural elements;
iii. Consequences to human lives;
iv. Function intended in building.

For the characterization of the performance objectives must be, firstly, specified objectives according to the class of importance of the building:

i. Basic objective for current buildings with normal occupation;
ii. Essential objective for important buildings, either because they serve to support relief and emergency operations, such as hospitals, whether they are offices of public bodies;
iii. Critical security objective, for facilities in which the consequences of damages may condition the safety and lives of people, such as nuclear power plants.

Finally, it is necessary to develop criteria for acceptability consisting in selecting the intensity levels of the maximum seismic action considered as acceptable for each performance level and specific purpose defined above. Still following the recommendations of VISION 2000 [ATC 58-2, 2002], these intensity levels are defined in accordance with the average return period of the seismic action, obtained through a seismic hazard study conducted for the geographical coordinates of the building analyzed. Thus the return periods adopted in this study are:

i. Frequent earthquakes where there is a probability of exceedance of 50% in 30 years, or an average return period of 43 years;

ii. Occasional earthquakes where there is a probability of exceedance of 50% in 50 years, or an average return period of 72 years;
iii. Rare earthquakes where there is a probability of exceedance of 10% in 50 years, or an average return period of 475 years;
iv. Very rare earthquakes where there is a probability of exceedance of 10% in 90 years, or an average return period of 975 years.

In a second level of verification, a comparison was made between the actuating and the resistant efforts in all structural elements. The efforts were determined by the outputs of the numerical model [Coelho, 2010]. On the other hand the resistant efforts were determined using multiple MathCAD programs taking into account previous studies, which disclose a method for computing the interaction surface of the reinforced concrete sections subject to bending composed deflected [Fafitis, 2001].

6. ANALYSIS AND INTERPRETATION OF THE RESULTS

As an example, in Fig. 6.1 and 6.2, are presented the different lateral displacement envelopes and the drift profiles for 475 years return period and for the conditioning alignment. The effectiveness of the solution in order to improve the structural behavior of the building studied can be analyzed based on the results shown in Table 6.1 and in the graphs of Fig. 6.3 and Fig. 6.4. These graphs represent in the structure (original and with TLD's), as an example, the maximum displacement or drift for each return period, at a particular the level (fifth floor).

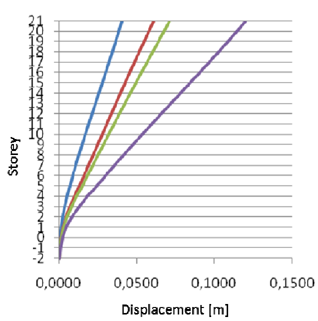

Figure 6.1. Lateral displacement envelope

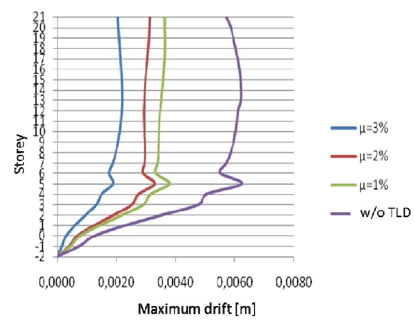

Figure 6.2. Drift profile envelope

Table 6.1 – Reduce deformation requirements at the 5th floor level

RP [year]	Seismic action	μ=1%	μ=2%	μ=3%
43	Type 2	38,67%	46,46%	71,76%
72	Type 2	38,67%	46,45%	71,76%
475	Type 2	38,67%	46,46%	71,7%
975	Type 2	38,67%	46,45%	71,76%

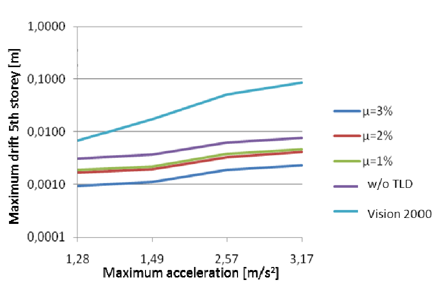

Figure 6.3. Maximum Lateral Displacement for each structure and return period (5^{th} floor)

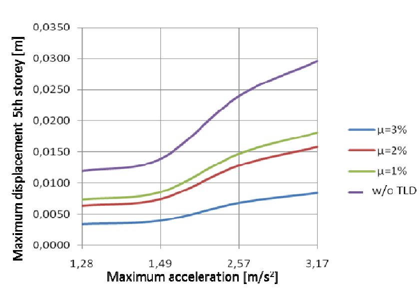

Figure 6.4. Maximum drift requirement for each structure and return period (5^{th} floor)

From the analysis of the results presented in Table 6.1 and in the graphs previously presented, it is possible to withdraw the following conclusions:

i. For any value of the imposed mass ratio (the mass ratio between the TLD mass and the mass of the building, μ), there is always a gain in terms of requiring deformation (drift).
ii. For example, at the level of the fifth floor and for a required mass ratio, either of μ = 1%, μ = 2% or μ = 3% there is a reduction of the displacement which varies between 38% and 72%.
iii. The estimated response is essentially linear (maximum drift is less than 1%), and the level of the estimated deformation requirements remains below the drift limit values indicated in the VISION 2000 [ATC 58-2, 1998].

As verified, the structure not only fulfills (in linear regime) with the seismic safety requirements, but also has improved them substantially. However it is not a guarantee that the structure is effectively safe. Therefore was necessary to proceed to the 2nd level of verification of safety requirements (verification of the seismic efforts generated in the structure). The verifications presented correspond to the ones made with 2% of mass for all sections of main structural elements in zones of change on the detailing of the cross sections.

It is presented for 475 years return period, the calculation of the situations that go beyond the rupture surface for the pillars, walls and core of the transition section floors and considering for each element and an upper section a lower section. Therefore there were analyzed 312 sections which represent a total of 156 elements. The graphs show the analyzed sections for the building without TLD's (Fig. 6.5) and for the building with a TLD mass ratio of 2% (Fig. 6.6).

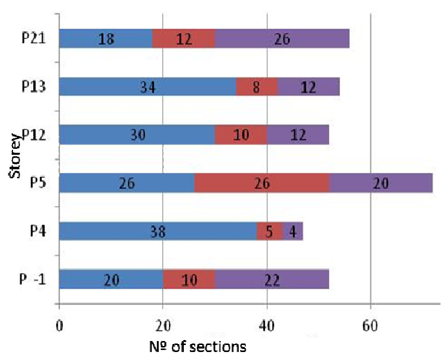

Figure 18. Ratio of sections that match the limits of rupture and the conditions of safety for seismic loads (without TLD)

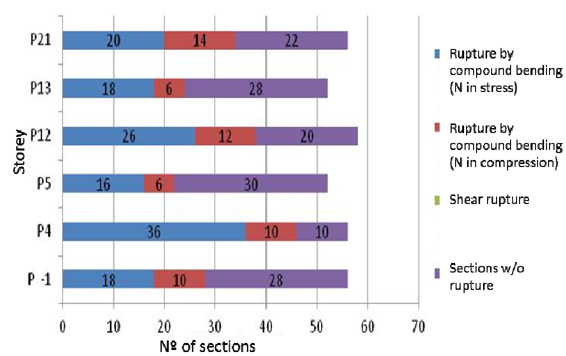

Figure 19. Ratio of sections that match the limits of rupture and the conditions of safety for seismic loads (with TLD)

It appears that there is an increase in the number of sections that match the yield limits of bending composite deflected. The sections that remain in rupture remain mostly in mechanical strength diverted compound with axial traction, though occasionally, there were sections which come into mechanical strength diverted to axial compression.

7. CONCLUSIONS

By its characteristics and modal analysis it can be stated that the three cores of the Solmar building have a very important influence on the bending and torsion behavior of the structure.

After defining the criteria for evaluating the performance proceeded to the seismic vulnerability analysis of the building considering two distinct levels. A first linear dynamic analysis of maximum displacement achieve by the structure was made, based on a comparison of the relative displacements and the maximum displacement limit set out in VISION 2000, considering different seismic action levels. It was later realized a second analysis by security verification of efforts by comparing resistant and actuating ones.

After analyzing the displacement carried out in the structure without TLD included, it was found that the structure meet the requirements of safety seismic internationally accepted for increasing levels of seismic action intensity, just considering the behavior of the structure approximately linear for all levels. However, after efforts analysis was found that the structure presented insufficient seismic resistance for code values of seismic intensity. It was also found that with the increase of seismic intensity the number of sections that collapse increases.

Vulnerability analysis in terms of displacement and stress resistant to the structure was repeated with different TLD weight percentages included. However, the structure continued to fulfill seismic safety requirements for displacements. Upon further analysis of efforts was found that the structure still presented insufficient seismic resistance for code values of seismic intensity, despite the increase of the number of sections without rupture.

After all, taking into account the analysis performed, it can be stated that the TLD's work well as a seismic protection devices for vibration mitigation. As shown these devices have a large capacity for reducing displacement and even efforts, however taking into account the special case of this study, this system is not the most suitable, and there is a need to study other options, from one or another energy dissipation system or a traditional reinforcement process.

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