Study on seismic upgrade of 5 storey reinforced concrete building by Tuned Mass Damper

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ABSTRACT: In civil engineering, Tuned Mass Dampers (TMD) are generally used to reduce the vibrations induced by strong winds in tall buildings. The TMD efficiency in the case of seismic actions is still in question. TMD effect on the seismic behaviour of a 5 storey reinforced concrete framed structure is presented in the paper. The structure analysis according to the present Romanian seismic code, P100-2006, shows that the structure presents a high level of vulnerability and strengthening measures are necessary. In order to do not interrupt the functioning of the building, and for other well-known advantages, TMD is chosen as a possible strengthening solution, but the studies show the TMD inefficiency in improving the seismic response of the building.

1 EXISTING STRUCTURE. DESCRIPTION AND ANALYSES RESULTS

The building was constructed between the years 1963 and 1965, in the north-east of Bucharest. The structure was designed according with the Romanian seismic code of the time, P13-1963. Figure 1 shows the floor plan, with 9 equal bays of 4.7 m and 3 unequal spans of 6.175, 3.4 and 7.175 m, respectively. The building transverse cross section is shown in Figure 2. The 5 storeys are 3.8 m high each. The columns are rectangular, with different dimensions on the building height. The floor slab has different thicknesses, of 12 cm in the lateral spans and 10 cm in the middle span. The structure is made of concrete C12/15, with reinforcing steel Fe360. The building has masonry partition walls, that prevented major damages in the structure, but these walls are seriously damaged, having fracture lines at 45^{o}.

1.1 *Structure analysis*

The safety of the building in accordance with the present Romanian codes for new structures has been evaluated through linear static, nonlinear static (pushover) and nonlinear time history analyses. The structure being regular in plan and elevation, only the transverse frame behaviour has been analyzed.

In the lateral force method of analysis, the horizontal seismic forces have been calculated based on the fundamental vibration mode, determined by modal analysis.

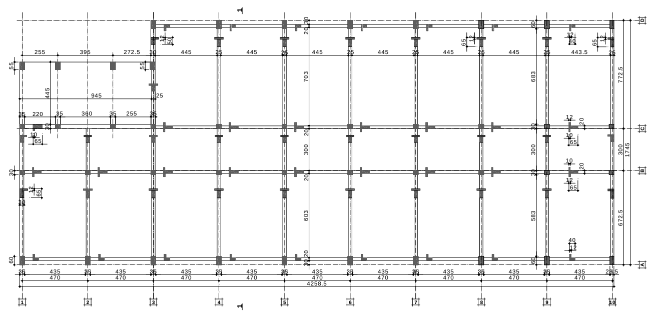


Figure 1. The building floor plan

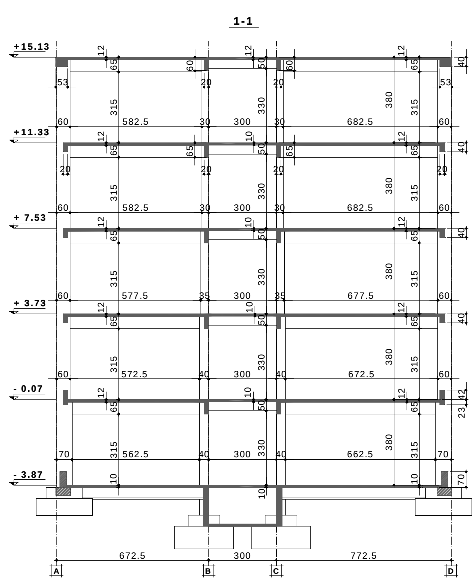


Figure 2. The building transverse cross section

According to the present Romanian seismic code, P100-2006, that is similar with EUROCODE 8, the ground type in the building site is established by the value of T_C, which is of 1.6 sec, while $T_B = 0.16$ sec and $T_D = 2$ sec. The design ground acceleration, a_g is 0.24g for a reference return period of 100 years. The damping correction factor is 1. The building belongs to the class of ductility M and the class of importance II ($\gamma_I = 1.2$).

In the Romanian seismic code P100-1992, the safety factor is defined as the ratio

$$R_i = \frac{F_{cap,i}}{F_{d,i}} \tag{1}$$

where $F_{cap,i}$ is the storey shear force associated to the resistance of the critical sections and $F_{d,i}$ is the storey shear force computed for a new structure. For buildings of importance class II, the code requires that $R_i \geq 1$ at each storey. If $R_i < R_{\min} = 0.6$, urgent interventions are necessary. Considering critical sections only at the column ends, the storey shear force may be calculated as

$$F_{cap,i} = \sum_{j=1}^{nc} \left(\frac{M_{cap,j}^{top} + M_{cap,j}^{bot}}{h} \right) \tag{2}$$

where nc is the number of columns that resist the seismic forces at the storey i, of height h. The moments of resistance at the j column ends are determined by taking into account the presence of axial force from vertical loads and the indirect effect of the overturning moment at the storey under consideration. The cross section resistance and the shear force capacity are determined based on the design values of the concrete and reinforcing steel strengths, f_{cd} and f_{yd}. The analysis showed that $R > 1$ only at the third floor.

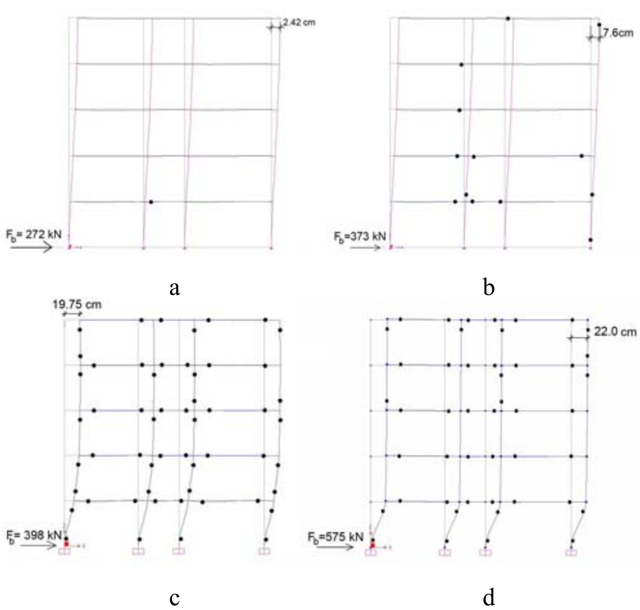


Figure 3. Pushover analysis for design strength values (a, b, c) and for mean strength values $f_{cm} = 1{,}75 f_{cd}$; $f_{ym} = 1{,}35 f_{yd}$ (d)

For the structural members, the safety ratio is defined by

$$r = M_{cap} / M_d \quad \text{and} \quad r = M_{cap}^{M-N} / M_d \tag{3}$$

for beams and columns, respectively. Both fulfill the safety condition if $r \geq 1$. The analysis showed that at the ground floor, 1st and 2nd floor, in the axes B, C and D, the beam and column safety ratio is smaller than 1, which indicates that node mechanisms may develop. Due to large displacements and lack of ductility, brittle failure of the concrete members may occur and storey mechanisms may develop at the same storeys.

The results obtained through linear static analysis have been confirmed by the pushover analysis. Figures 3a, b show the plastic hinge occurrence obtained through nonlinear static analysis for design strength values of the materials. Node and storey mechanisms can be identified in the plastic mechanism shown in Figures 3c, d, for design strength values and mean strength values, respectively.

Figure 4 shows the relation between the lateral displacements at the roof level and the base shear force, in the case of design strength values. The nonlinear static analysis showed the exceeding of the interstorey drift and resistance capacities, both in service limit state (SLS) and ultimate limit state (ULS).

For the nonlinear time history analysis, the north-south component of the accelerogram recorded at INCERC-Bucharest during the severe earthquake from 4th March 1977, with PGA = 0.211g, has been used to determine the dynamic response of the structure.

Figure 5 shows this accelerogram (INCERC) and one of the artificial accelerograms that have been generated in order to calibrate the TMD (ACC1). Raighley structural damping has been considered, the same in all vibration modes, with the concrete structure damping ratio, $\xi = 5\%$.

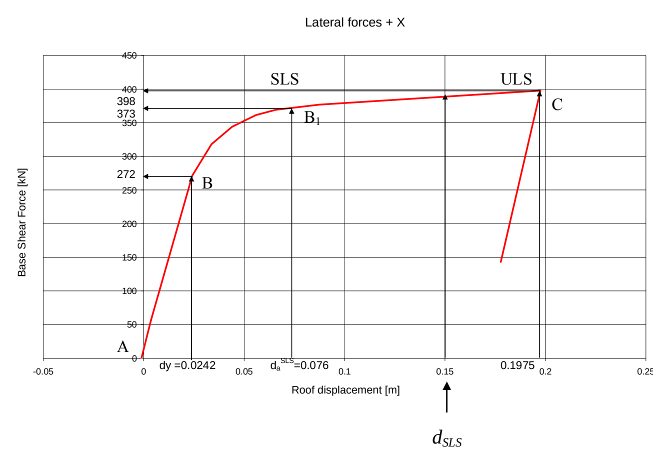
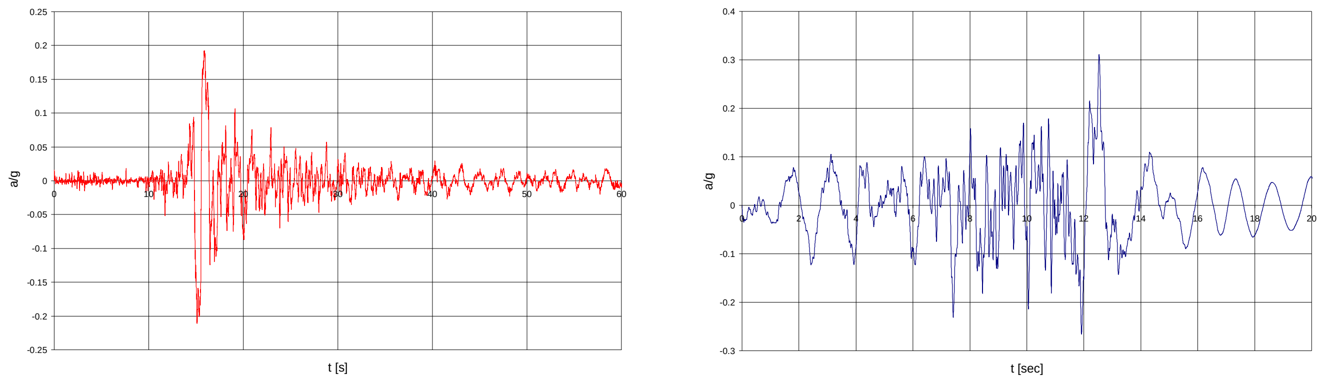


Figure 4. Pushover analysis: base shear force – roof displacement curve for design strength values



a   b

Figure 5. Recorded accelerogram INCERC (a) and generated accelerogram ACC1 (b)

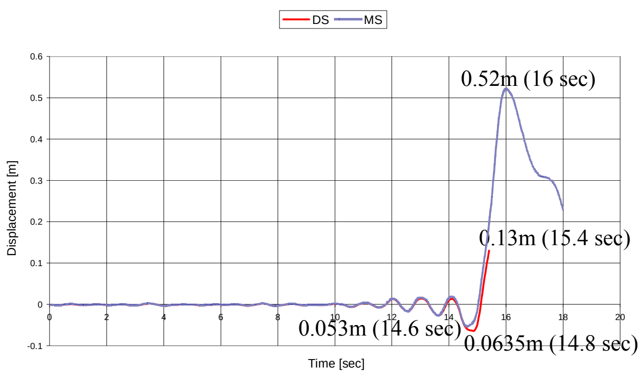


Figure 6. Nonlinear time history analysis: roof lateral displacement time evolution

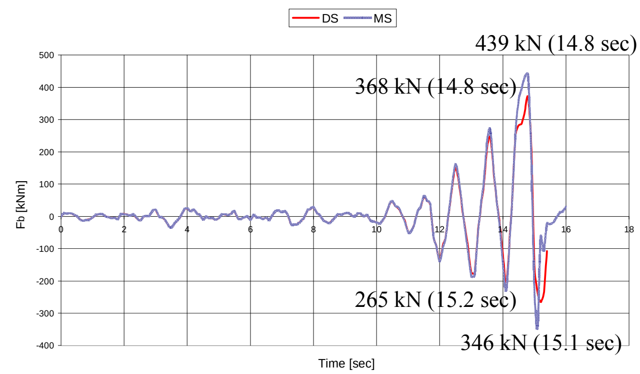


Figure 7. Nonlinear time history analysis: base shear force time evolution

Two models have been done, one using the design strength values (DS), the other using the mean strength values (MS) of the concrete and reinforcing steel. The analysis showed the limited resistance capacities of the structural members, due to the premature forming of the global plastic mechanism and the large deformability that exceeds the deformation capacities.

Figures 6 and 7 show the time evolution of the roof displacement and base shear force, respectively, for both analyzed models.

2 ANALYSES RESULTS AFTER TMD IMPLEMENTATION

The TMD role is to take over some of the earthquake energy input, thus reducing the seismic response of the existing structure. The classic TMD consists of a mass supported by springs. It is usually installed on the roof of the building (Fig. 8a). The TMD dynamic characteristics have to be calibrated until its oscillations are out of phase with the structure oscillations during the earthquake excitation. The optimal TMD mass and stiffness (m and k) are determined through numerical simulations, until the lateral displacements of the structure at the roof level are minimum.

The TMD that consist of a mass supported by linear elastic springs without damping is known as the Hooke type TMD, for which the relation force-displacement is given by $F=ku$.

In order to increase the tuning band of the dominant frequencies structure – seismic action, dampers can be added to the classic TMD (Fig. 8b).

The Kelvin type TMD is characterized by

$$F = F_k + F_c = ku + c_a(\dot{u})^{\alpha}\, sign(\dot{u}),\ \ u = u_k = u_c,$$
$$0.2 \le \alpha \le 1$$

$$c_a = 2m\omega_a\xi_a,\ \omega_a = 2\pi/T_{TMD},\ T_{TMD} \cong T_1$$

If dampers are added in series to the classic TMD, the Maxwell type TMD is obtained, for which

$$F = F_k = F_c = k_a u_k = c_a(\dot{u}_c)^{\alpha}\, sign(\dot{u}_c)\ \ u = u_k + u_c$$

For $k_a = 1000 c_a\omega_a$, pure damping is obtained. The dampers behave linear elastically for $\alpha = 1$.

The Zener type TMD has linear elastic springs and nonlinear dampers and consists in Hooke and Maxwell TMD types, put in parallel.

2.1 *Existing structure with TMD*

The additional mass installed on the roof has been calculated only for the movement in the building transverse direction.

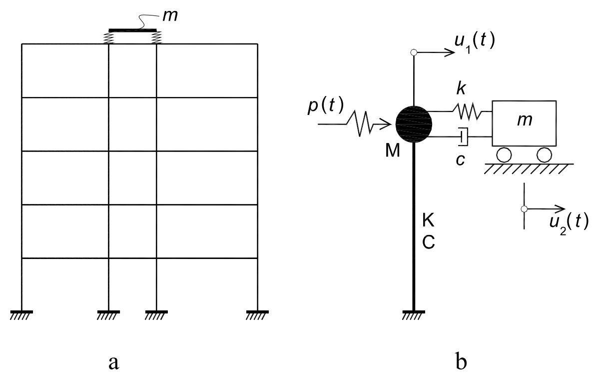


Figure 8. Existing structure with TMD

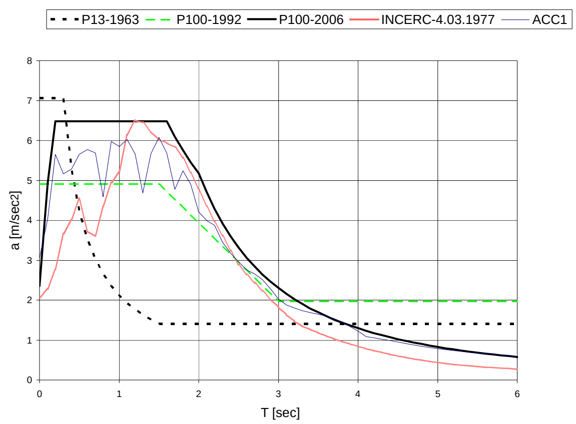


Figure 9. Absolute acceleration elastic spectra

The TMD calibration has been done for the accelerogram INCERC and three generated artificial accelerograms. The elastic response spectra for 5% viscous damping for the accelerograms INCERC and ACC1 are shown in Figure 9, in comparison with the spectra from the Romanian codes P13-1963, P100-1992 and P100-2006. The following situations have been compared: existing structure without TMD; structure with Hooke type TMD; structure with Zener type TMD, having ξ_a = 15% and α = 0.4. Time history analyses have been performed for the recorded accelerogram INCERC and the artificial accelerogram ACC1. From the analyzed cases, the most efficient TMD is the Hooke type TMD, with $m = 0.015M$ and $\omega_a = \omega_s$ (TMD tuned with the structure of mass M).

Figure 10 shows the time evolution of the base shear force for three situations: existing structure without TMD, structure with Hooke type TMD, with $m = 0.015M$ and structure with Zener type TMD having $m = 0.015M$ and $c_a = 12$. The Hooke type TMD reduces the lateral displacements and the base shear force, but only after the principal seismic attack. After the first peak in the response history, the base shear force decreases with about 30%. The Zener type TMD reduces the structural vibration amplitude after the principal shock with about 37%, but increases the base shear force, although the input energy decreases, as it is shown in Figure 11.

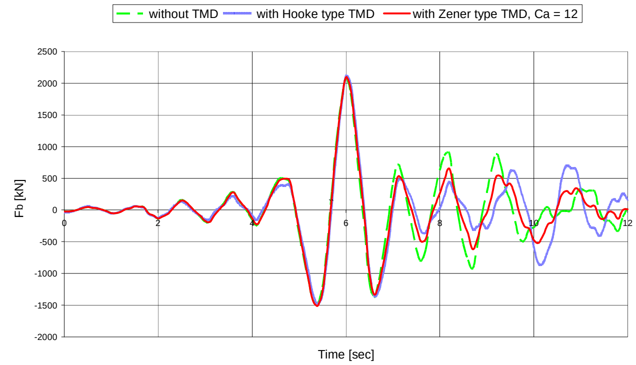


Figure 10. Existing structure. Base shear force time evolution for the accelerogram INCERC

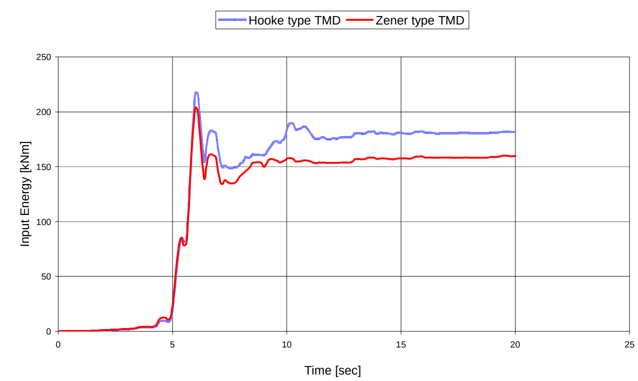


Figure 11. Structure with TMD, $m = 0.015M$. Earthquake energy input time evolution for the accelerogram INCERC

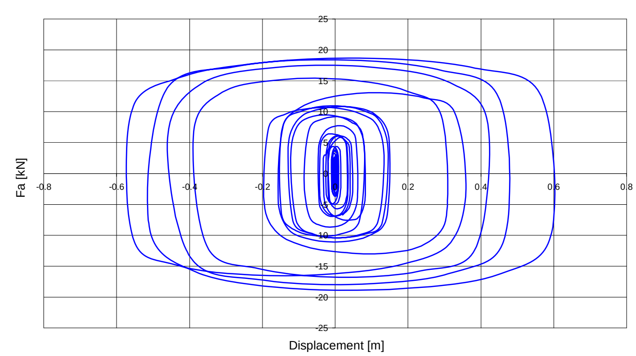


Figure 12. Damping force time evolution for $\alpha = 0.4$

Figure 12 shows the time evolution of the damping force,

$$F_a = c_a |\dot{u}_x|^{\alpha} \, sign(\dot{u}_x)$$

for $\alpha = 0.4$. In practice, the TMD needs some time in order to get tuned with the excitation and the structure vibrations.

Table 1 presents the results obtained for the maximum positive and negative roof lateral displacements d_5, the maximum positive and negative base shear force F_b and earthquake input energy E_g.

Table 1. Existing structure. Analysis results before and after TMD implementation

Analysis	Excitation	d_5		F_b		E_g
		cm		kN		kNm
		max +	min –	max +	min –	
no TMD	INCERC	13.0	19.6	2082	1473	–
	ACC1	21.8	20.9	2416	2248	–
Hooke TMD	INCERC	14.4	20.2	2107	1485	216.8
	ACC1	18.7	20.1	2229	1918	484.3
Zener TMD	INCERC	13.6	19.9	2095	1511	202.4
	ACC1	19.8	20.4	2293	2041	429.9

Table 2. Strengthened structure. Analysis results before and after TMD implementation

Analysis	Excitation	d_5		F_b		E_g
		mm		kN		kNm
		max +	min –	max +	min –	
no TMD	INCERC	7.15	6.4	1057	1152	–
	ACC1	12.0	10.9	1432	1505	–
Hooke TMD	INCERC	6.61	8.5	1225	1088	15.21
	ACC1	8.47	9.2	1404	1357	32.10
Zener TMD	INCERC	7.54	7.2	1084	1192	12.84
	ACC1	11.6	10.6	1564	1492	36.68

If the interaction soil-structure effect is considered, the TMD should be tuned to the natural frequency of the soil-structure system instead of the structure with fixed base, in the case of moderate to stiff soils. For soft soils, even properly tuned TMD are ineffective.

If inelastic incursions take place, the frequency of the main structure will decrease, causing the TMD detuning and thus reducing its efficiency. Hence, a structure should behave elastically so that the TMD is effective.

2.2 *Combined strengthening solution, with r. c. walls and TMD*

A natural strengthening solution is to replace the masonry walls with reinforced concrete walls (Fig. 13a). In order to ensure that the existing structure works together with these r. c. walls, chemical anchors are used. The TMD effect on the dynamic response of this new structure is analyzed (Fig. 13b). The structural wall is 0.15 m thick and it is made of reinforced concrete of class C22/25 with E = 30000 N/mm^2. This classical strengthening solution drastically reduces the interstory drift.

The new structure, considered in three situations (without TMD, with Hooke type TMD and Zener type TMD) has been analysed under the seismic actions represented by the accelerograms INCERC and ACC1.

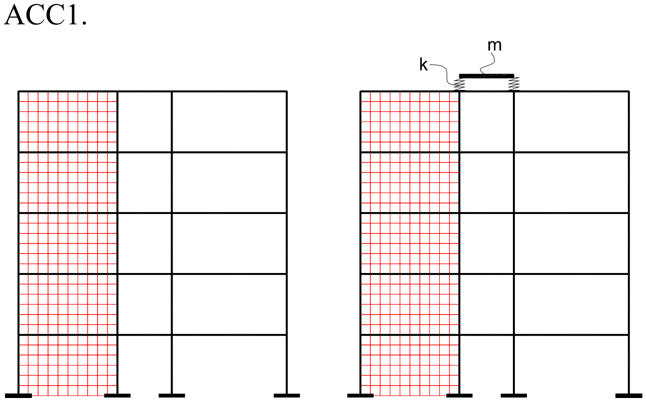


Figure 13. Strengthened structure without TMD (a) and with TMD (b)

Table 2 summarizes the results obtained for the maximum lateral displacements d_5, base shear force F_b and earthquake input energy E_g, in all situations.

Figures 14 and 15 show the time evolution of the base shear force and earthquake input energy in the case of the recorded accelerogram INCERC. As in the case of the existing structure without classical strengthening, the TMD has insignificant effects on the new structure. The displacements and the base shear force are slightly and randomly reduced.

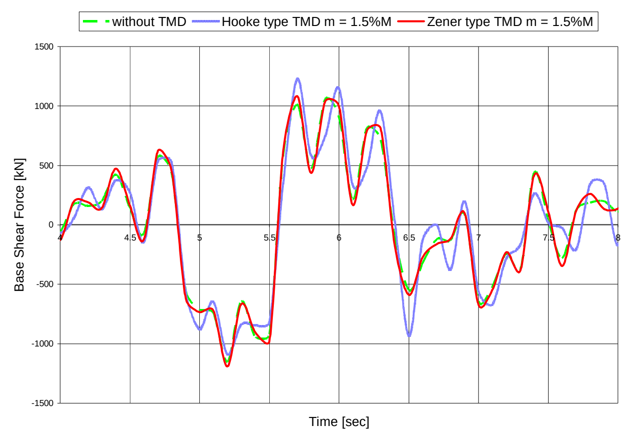


Figure 14. Strengthened structure. Base shear force time evolution for the recorded accelerogram INCERC

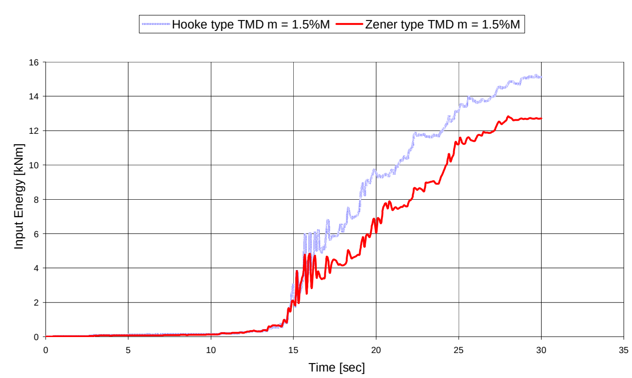


Figure 15. Strengthened structure. Earthquake input energy time evolution for the accelerogram INCERC

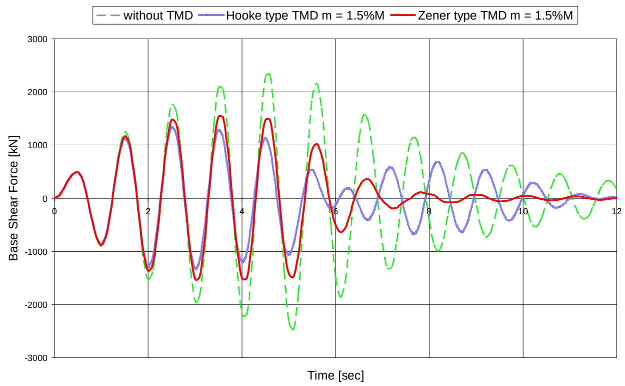


Figure 16. Base shear force time evolution for sinusoidal accelerogram

2.3 *Initial structure with TMD, subjected to harmonic excitation*

When the initial structure (without r. c. walls) is subjected to the harmonic accelerogram

$$\ddot{u}_g(t) = a_o \sin \omega t$$

with a_o = 1 m/sec^2, $\omega = 2\pi$ rad/sec ($T \approx T_1$ = 1.059 sec), $0 \le t \le 5$ sec, the Hooke type TMD reduces the maximum lateral roof displacement with about 60% and the base shear force with about 51%. The Zener type TMD leads to smaller reductions, of about 40% both for roof displacements and base shear force.

Figure 16 shows the time evolution of the base shear force. The advantage of the Zener type TMD with respect to the Hooke TMD is that it reduces more rapidly the vibrations after the ceasing of the exciting action (for $t > 5$ sec).

3 CONCLUSIONS

Seismic rehabilitation of buildings that do not satisfy the safety conditions imposed by the present seismic code can be done by different strengthening solutions. The traditional solution consists in controlling the plastic hinge occurrence, increasing the deformation capacity of the structural members and the limitation of the lateral displacements.

Modern solutions use devices for the active, semi active or passive control of the structural response at seismic actions. Passive control consists either in structure base isolation, either in introduction in the structure of dissipative devices, like tuned mass dampers or viscous dampers. The decision for the most appropriate rehabilitation solution has to be taken in accordance with the structural dynamic characteristics, the structure deformation and resistance capacities, and depending on the ground motion type, the earthquake frequency content and the site conditions.

Passive control devices like TMD are not capable to improve the performance of the structure analysed in the paper. Comparative with the use of TMD, the classic strengthening solution is more appropriate to ensure the safety of the reinforced concrete frame structure.

The study shows the TMD inefficiency in reduction the displacements and forces produced by seismic actions. These devices may oscillate in phase with the structure and increase its response. On the other hand, the TMD requires that the structure behaves elastically, which is a condition quite difficult to be satisfied. The TMD efficiency under harmonic excitations tuned to the fundamental oscillation of the building (at resonance) recommends this device for reducing the oscillations under wind actions.

Analyses done on simple models may prevent possible failure in the tendency of introducing modern rehabilitation solutions.

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