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THE USE OF TUNED MASS DAMPERS FOR REDUCING STRUCTURAL VULNERABILITIES DURING THE SIDE MOVEMENTS OF BUILDINGS WITH REINFORCED CONCRETE FRAMES

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Abstract:*The present article aims to point out, with the help of a comparative research, the efficiency of tuned mass dampers, modern variants of consolidation ensuring seismic structural safety, used for buildings with a reinforced concrete structure, designed and produced according to the new codes. Case studies were based on structural computations in the linear elastic field using the ETABS program.*

Keywords: TMD, vulnerability, earthquake, damper

1.INTRODUCTION

The TMD (Tuned Mass Damper) is extremely efficient in monitoring the vibrations of a structure; it is made of a mass, a viscous damper and a spring attached to a specific structure, in order to reduce the wind and earthquake induced vibrations. The frequency and damping of these systems are tuned, so that, when the structure resonates at a certain frequency (period) the TMD oscillates with the same period, but not in phase with the structure. Consequently, the energy is transmitted from the primary system (structure) to the secondary one (TMD) and dissipated into the damper. The main purpose of this paper is to observe the efficiency of TMD for different building structures and rise. In order to obtain responses three shapes of layouts were used: circular, square and rectangular, each one with two type of structures (RC frames and dual RC) for 5, 10, 15, 20, 25, 30, 35 and 40 levels.

2. THE USE OF TUNED MASS DAMPERS AROUND THE WORLD

The first tuned mass dampers were proposed by Fhram in 1909, their purpose being to reduce vibrations produced by monotonous harmonic forces. He noticed that, if a secondary system made of a mass, a damper and a spring is attached to a primary system and its period is tuned in with the basic period of the primary system, the result is a significant decrease of the dynamic response.

The first building equipped with a tuned mass damper is Centerpoint Tower in Sydney, Australia. Its structure was completed in 1981 and it is 309m high.

One Wall Center, also known as the Sheraton Vancouver Wall Center Hotel, has 48 levels and a height of 158m. In order to prevent potential harmonic vibrations caused by the strong wind, the tower was equipped with a tuned water damper. This system was placed on the top level and is made of two tanks with the capacity of 227,000 l. These tanks were designed to ensure that the harmonic frequency of water movement and the harmonic movement are balanced.

Taipei 101, also known as Taipei World Financial Center was the tallest building from 2004 until the erection of Burj Khalifa. It has 101 levels and a height of 508m.

Its civil engineers designed a steel pendulum weighing 660 tons to act as a tuned mass damper. Suspended between the 92^{nd} and the 87^{th}levels, the pendulum was set to move out of phase with the structure, to stop the effect of wind gusts. The pendulum with a length of 5.5m is made of 41 circular steel plates, 125mm high.

Two more tuned mass dampers, weighing 6 tons, are mounted at the top of the spire and they help to prevent the damage caused by high wind loads.

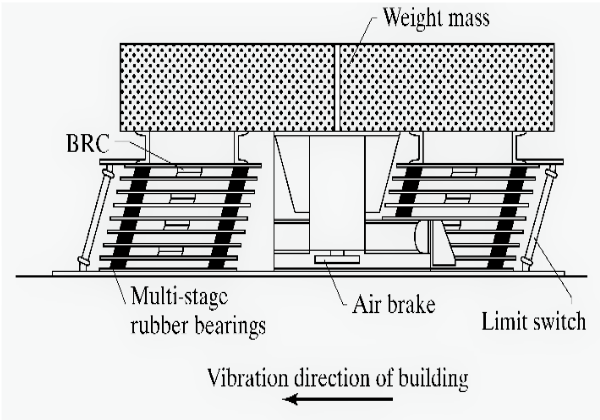
The tuned mass damper was also used in engineering works such as the Akashi Kaikyo bridge. With a total length of 3911m, the bridge has pendulums designed to operate at the resonant frequency of the bridge, thus absorbing the forces given by wind and earthquake loads.

3. THEORETICAL CONCEPTS AND PRINCIPLES

Structural vibrations are reduced with TMD inside a structure, according to the principle of vibration energy transfer into the TMD for dissipation. Thus, the TMD frequency is in tune with a certain frequency of the structure. When this point is reached, the TMD will resonate, out of tune with the movement of the structure.

In comparison with other dissipation methods, the use of TMDs requires the use of a mass and of wide movements. The way in which the TMD is mounted and set is extremely important in the system design. Usually the mass of the damper system is in the range of 0.25…1% of the building mass in the basic vibration mode.

In certain cases, the limited space does not allow the use of a classic tuned mass damper. This limitation led to the creation of alternative systems, such as the multi-stage pendulum, the reversed pendulum, or hydrostatic bearings. The systems are usually tuned with the help of spired springs or pneumatic springs with a variable rigidity.



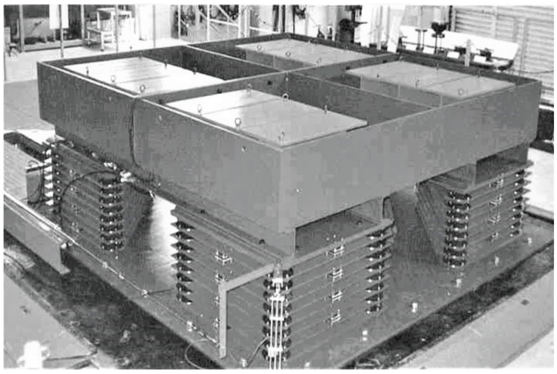


Figure 1. Example of a system with tuned mass (Courtesy of J. Connor)

Problems arising due to space limitation can be solved if a pendulum is used:

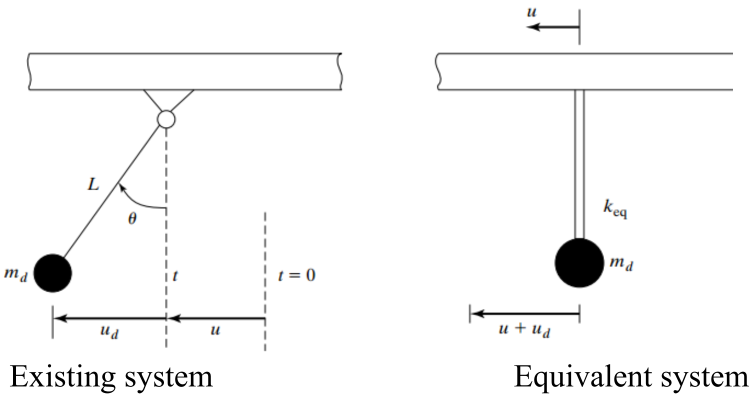


Figure 2. Diagram of a TMD pendulum

The horizontal movement equation is:

$$T \sin\theta + \frac{W_d}{g}(\ddot{u} + \ddot{u}_d) = 0 \tag{1}$$

where T is the cable tension; u(t) – the structure movement; $u_d t$ – the movement of the pendulum mass; W_d – the pendulum weight; m_d – the pendulum mass:

$$u_d = L\sin\theta \cong L\theta; \qquad T \cong W_d \tag{2}$$

Therefore:

$$m_d\ddot{u}_d + \frac{W_d}{L}u_d = -m_d\ddot{u} \tag{3}$$

So that the resulting equivalent shear rigidity is:

$$k_{eq} = \frac{W_d}{L} \tag{4}$$

The pendulum pulsation is:

$$\omega_d^2 = \frac{k_{eq}}{m_d} = \frac{g}{L} \tag{5}$$

Whereby the resulting period is:

$$T_d = 2\pi\sqrt{\frac{L}{g}} \tag{6}$$

The only major limitation related to the use of a pendulum is that the pendulum must be longer than the height of a storey. This issue can be solved using the diagram below:

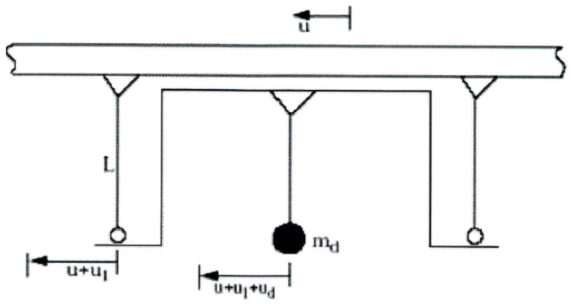


Figure 3. Solving the limitation related to the excessive length of a pendulum

The rigid inner joint amplifies the support movement and the following balance equation is valid for the pendulum:

$$m_d(\ddot{u} + \ddot{u_1} + \ddot{u_d}) + \frac{W_d}{L}u_d = 0 \tag{7}$$

The rigid joint moves in phase with the damper and has the same movement width. If $u_1 = u_d$ we have:

$$m_d\ddot{u}_d + \frac{W_d}{2L}u_d = -\frac{m_d}{2}\ddot{u} \tag{8}$$

The equivalent rigidity shows that the actual length is 2L. Therefore, each additional joint adds to the actual length with the value "L".

4. COMPUTATION OF TMD-TYPE SYSTEMS

` As a rule, the analysis of TMD systems involves the computation of the answer from a system with two degrees of freedom, the first one being the primary freedom degree (structure), while the second belongs to the TMD. For this purpose, we use mathematical procedures to derive the system response with two degrees of freedom, being submitted to harmonic frequency stimuli. The following section presents these responses for different cases.

Several basic steps must be made for the computation of a tuned mass damper-type device:

First, the location (position) of the damper must be set to coincide with the maximum amplitude point of the modal form to be monitored;

The primary system mass will be the mass on the module to be monitored with the help of the damper;

The optimal ratio of frequencies (periods) f is computed and then used to determine the damper rigidity $k_d = m_d\omega_d^2$

The optimal fraction from the critical damping $\zeta_{d,opt}$ *is computed and used to determine the device damping*$c_{d,opt} = 2\zeta_{d,opt}\omega_d m_d$

The parameter used for optimization is the "movement" and it is used to ensure the integrity of the structure and non-structural elements. However, "acceleration" can also be used for optimization in the case of heavy/sensitive equipment at high acceleration rates.

For widely used seismic applications,Villaverde (1985) suggests the use of the following equations with optimal parameters, when the ratio of the masses is based on the modal mass and the existing vector normalized in the point where the device is placed:

$$f_{opt} = 1 \tag{9}$$

$$\zeta_{d,opt} = \zeta + \mu \tag{10}$$

For the same conditions, Fadek (1997) finds that:

$$f_{opt} = \frac{1}{1+\mu}\left[1 - \zeta\sqrt{\frac{\mu}{1+\mu}}\right] \quad (11)$$

$$\zeta_{d,opt} = \frac{\zeta}{1+\zeta} + \sqrt{\frac{\mu}{1+\mu}} \quad (12)$$

Note:
Devices of this type are efficient in low damping seismic applications $\zeta = 0.022$. For structures with $\zeta = 0.05$, damping, they are not too efficient because the ratio of the masses is too big. Also, they are not efficient in the case of highly rigid structures with 0.1s – 0.2s periods.

Table 1. Optimal parameters.

System	**Excitat-ion type**	**Excitat-ion applied**	**Optimized parameter**	**The optimal freque ncy ratio f_{opt}**	**Optimal damping $\zeta_{d,opt}$**	**Maximum optimized response of the structure u_r**
Damped system, TMD damped	Deterministic harmonics	Primary mass and base	Movement of the primary mass	-	-	$\frac{(p_0 + ma_0)/k}{\sqrt{2\zeta(1-\zeta^2)}}$
Undamped system, TMD undamped		Primary mass	Movement of the primary mass	-	-	-
Undamped system, TMD damped		Primary mass	Movement of the primary mass	$\frac{1}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)}}$	$\frac{p_0}{k}\sqrt{1+\frac{2}{\mu}}$

Undamped system, TMD damped		Primary mass	Acceleratio nof the primary mass	$\frac{1}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu/2)}}$	$\frac{p_0}{k}\sqrt{\frac{2}{\mu(1+\mu)}}$
Undamped system, TMD damped		Base	Movement of the primary mass	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)(1}}$	$\frac{ma_0}{k}(1+\mu)\sqrt{\frac{2}{\mu}}$
Undamped system, TMD damped	Random	Base	Acceleration of the primary mass	$\frac{1}{1+\mu}$	$\sqrt{\frac{3\mu}{8(1+\mu)}}$	$\frac{ma_0}{k}\sqrt{1+\frac{2}{\mu}}$
Undamped system, TMD damped		Primary mass	Movement of the primary mass	$\frac{\sqrt{1+\mu/2}}{1+\mu}$	$\sqrt{\frac{\mu(1+\frac{3}{4}}{4(1+\mu)(1}}$	$\frac{p_0}{k}\sqrt{\frac{(1+\frac{3\mu}{4})}{\mu(1+\mu)}}$
Undamped system, TMD damped	Deterministic harmonics	Base	Movement of the primary mass	$\frac{\sqrt{1-\mu/2}}{1+\mu}$	$\sqrt{\frac{\mu(1-\frac{\mu}{4}}{4(1+\mu)(1}}$	$\frac{p_0}{k}\sqrt{\frac{(1+\frac{3\mu}{4})}{\mu(1+\mu)}}$
Damped system, TM damped		Primary mass	Movement of the primary mass	$\frac{1}{1+\mu} - (0.241 + 1.7\mu - 2.6\mu^2)\zeta - (1.0 - 1.9\mu + \mu^2)\zeta^2$	$\sqrt{\frac{3\mu}{8+(1+\mu)}} + (0.13 + 0.12\mu + 0.4\mu^2)\zeta - (0.01 + 0.9\mu + 3\mu^2)\zeta^2$	

5. CASE STUDIES I

Considering the above theoretical information, the following section presents just several data from all case studies for RC structures with different plane shapes and a height regime for a structure P+39E (40 levels). The height for each floor was set at 3m. Structures are classified as follows:

Function of the structural system:

Structural system with reinforced concrete frames;

Dual structural system (reinforced concrete frames and walls)

Function of the plane form:

Circular

Square

Rectangular

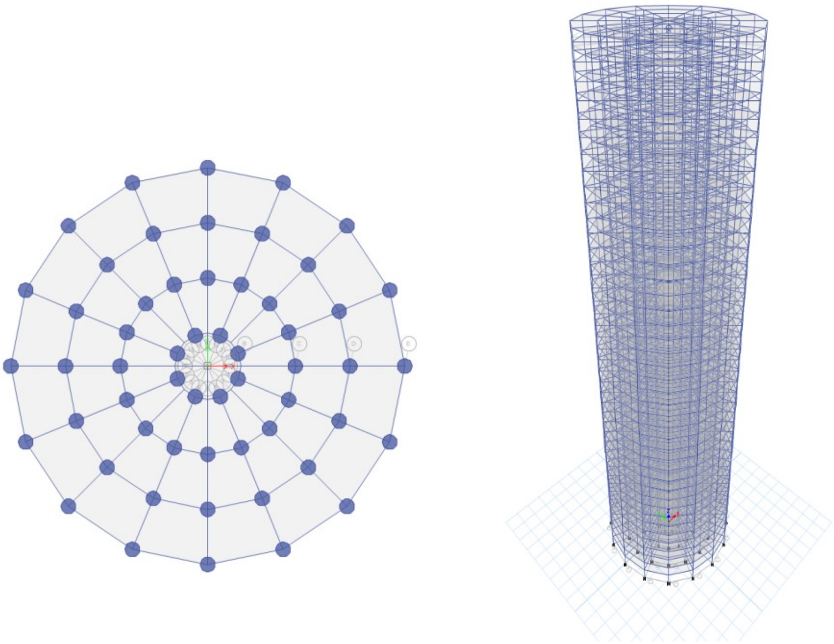


Figure 4. Structure with circular reinforced concrete frames – plane and elevation

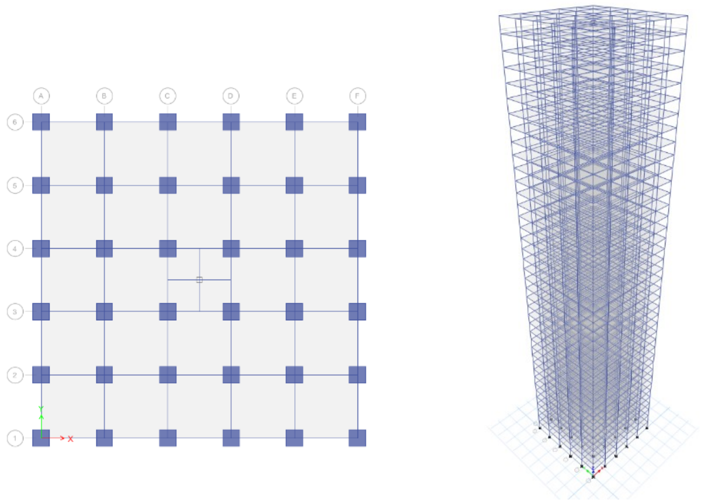


Figure 5. Structure with square reinforced concrete frames – plane and elevation

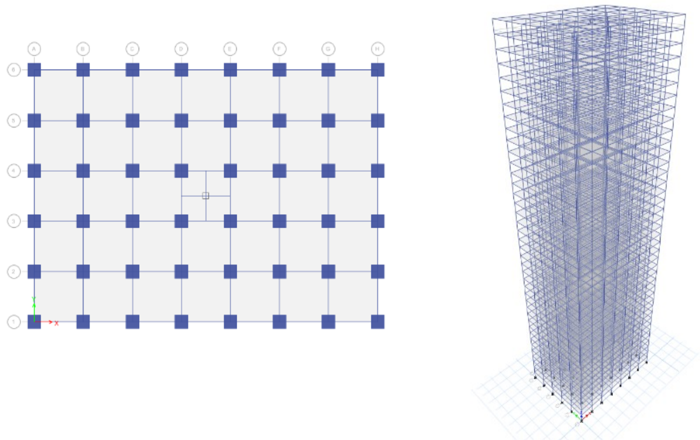


Figure 6. Structure with rectangular reinforced concrete frames – plane and elevation

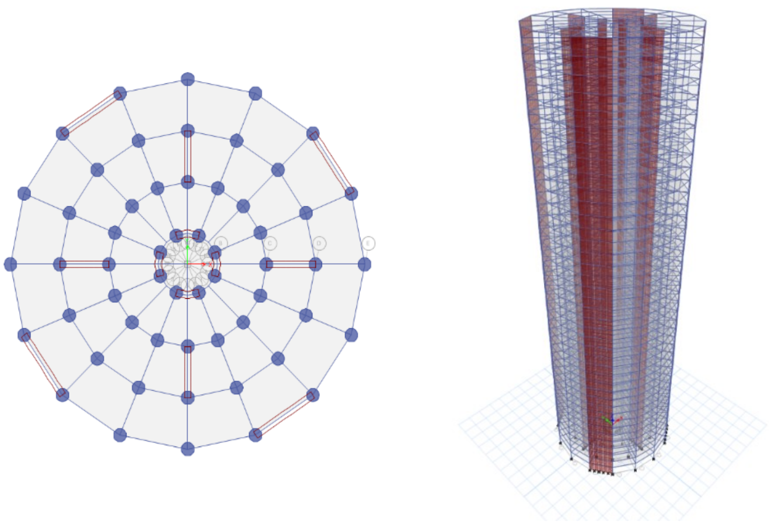


Figure 7. Dual reinforced concrete structure with a circular shape – plane and elevation

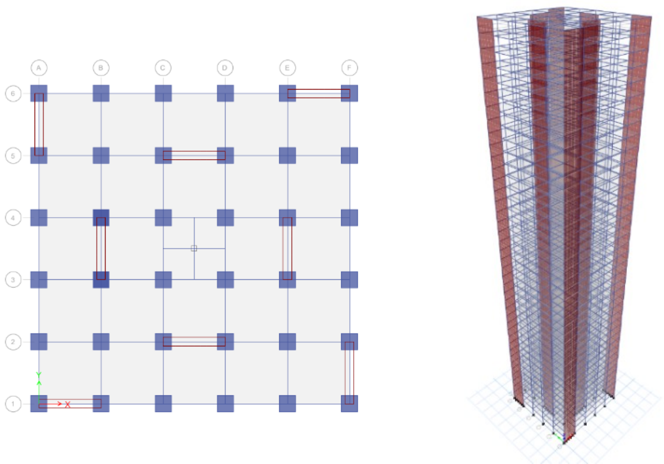


Figure 8. Dual reinforced concrete structure with a square shape – plane and elevation

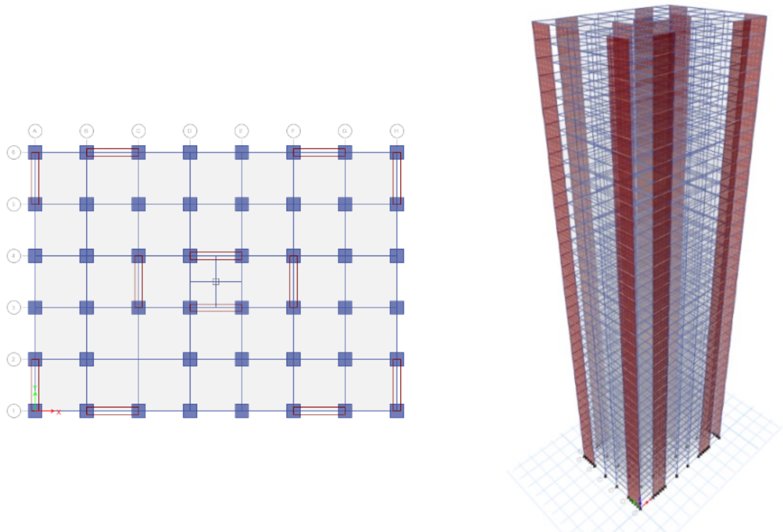


Figure 9. Dual reinforced concrete structure with a rectangular shape – plane and elevation

In order to minimize the number of variables and better emphasize the use of TMDs, the distance between axes is 5m. The concrete is class C30/37 with an elasticity module of $E=33000N/mm^2$. The dimensions of the structural elements under focus are the following:

Table 2. The dimensions of the structural elements.

Structure type	Plane shape	Side poles [cm]	Longitudinal beam [cm]	Cross beam [cm]	Radial beam [cm]	Wall thickness [cm]
Frame structure	Circular	130	-	-	35x80	-
	Square	130	35x80	35x75	-	-
	Rectangular	130	35x80	35x80	-	-
Dual structure	Circular	130	-	-	35x80	70
	Square	130	35x80	35x80	-	70
	Rectangular	130	35x80	35x80	-	70

The following design projection was used for the analysis with response projections:

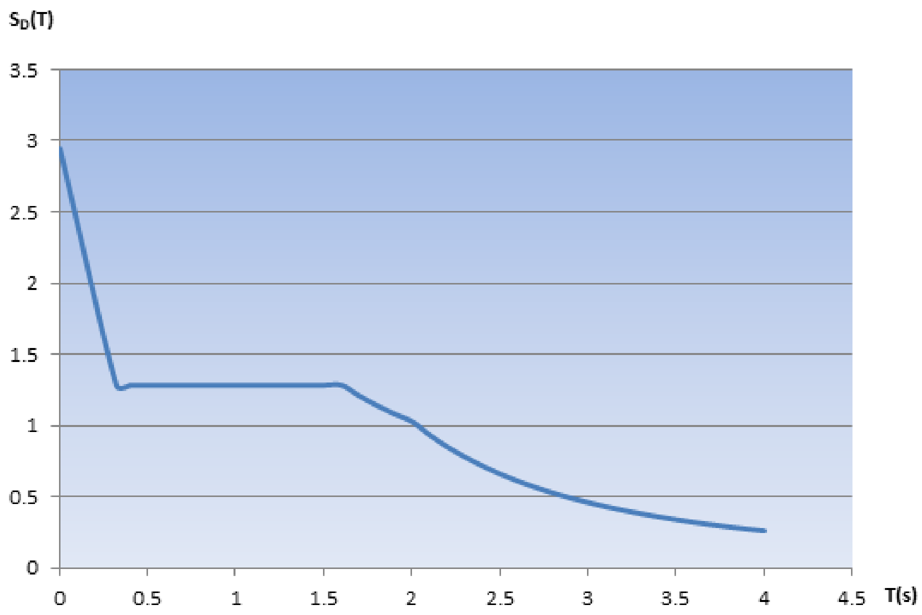


Figure.10. Design spectra for a_g=0.30g

In order to set the size of the tuned mass dampers the following steps were considered:

First, the location (position) of the damper was set to coincide with the maximum amplitude point of the modal shape to be monitored. The mass of the primary system will be the mass applied on the module, to be monitored with the damper. The higher the ratio of the masses μ the lower the response (the damper is less affected by the tuning). Therefore, the highest possible ratio is chosen.

The optimal ratio of frequencies (periods) F is determined, then used to compute the damper rigidity $k_d = m_d \omega_d^2$

The optimal fraction of the critical damping $\zeta_{d,opt}$ is determined, then used to compute the device damping, $c_{d,opt} = 2\zeta_{d,opt}\omega_d m_d$

***System modeling with the tuned mass*:**

As previously described, the Tuned Mass Damper must be positioned in the maximum amplitude point of the mode to be monitored. In the cases under study, this point can be any point on the floor of the last storey.

The TMD system is modeled with the help of a friction pendulum-type insulator, in order to simulate the physical movement of the TMD, while for the damping part a viscous damper-type link was used. Two links with these names are included in the program, with characteristics determined by computation.

The modeling must consider the link design mode (upwards) in order to correctly define the damping movement. For good operation, the point on the floor of the last storey will be set in its center, while the free one will be set on the same vertical axis, at an equal distance to the pendulum length.

The free point must also support masses on the main directions; these masses are determined through TMD computation.

μ – ratio of the masses;
ζ – fraction of the critical damping;
$m_{mod.1}$ – mass moved on the vibration mode 1;
$m_{mod.2}$ – mass moved on the vibration mode 2.

$$m_d = \mu \times m_{mod,1} \tag{13}$$

$$f_{opt} = \frac{1}{1+\mu} - (0.241 + 1.7\mu - 2.6\mu^2)\zeta - (1.0 - 1.9\mu + \mu^2) \tag{14}$$

$$\zeta_{d,opt} = \sqrt{\frac{3\mu}{8+(1+\mu)}} + (0.13 + 0.12\mu + 0.4\mu^2)\zeta - (0.01 + 0.9\mu + 3\mu^2)\zeta^2 \tag{15}$$

$$f_d = f_{mod,1} \times f_{opt} \tag{16}$$

$$T_d = \frac{1}{f_d} \tag{17}$$

$$\omega^2 = 2\pi f_d \tag{18}$$

$$k_d = m_d \omega_d^2 \tag{19}$$

$$c_{d,opt} = 2\zeta_{d,opt}\omega_d m_d \tag{20}$$

$$L_{pendul} = \frac{{T_d}^2 \times g}{4\pi^2} \tag{21}$$

For the structures included in the analysis, the TMD characteristics are the following:

Table 3.TMD characteristics.

Structure type	**Plane shape**	**Pendulum length [m]**	**Rigidity k_d [kN/m]**	**Device dampingc_d [kN/s]**	**Mode 1 mass [kN]**	**Mode 2 mass [kN]**
Frame structure	Circular	2.35	22153	4143	5275	5275
	Square	2.3	15451	2881	3657	3657
	Rectangular	2.2	22325	4030	5019	4953
Dual structure	Circular	1.9	28238	4740	5418	5418
	Square	1.65	21450	3383	3634	3634
	Rectangular	1.7	31128	4934	5325	5390

6. CASE STUDIES II

Further on, in order to apply the notions described before, take a reinforced concrete framed structure with different height regimes: P+40E, P+30E, P+20E, P+10E. The height of each level is 3m. The plane shape of the structure is rectangular.

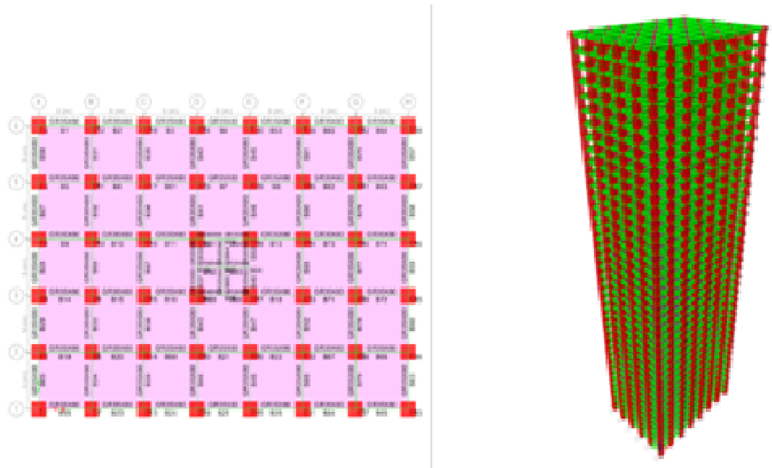


Figure 11. Reinforced concrete framed structure with a rectangular shape40 levels – plane and elevation

In order to minimize the number of variables and to better emphasize the use of TMDs, the distance between axes is 5m. Apart from the loads representing the load of the structure, the following ones were considered:

On existing levels: *Floor and building services – 2kN/m^2; Separating walls – 0.5 kN/m^2; Useful load – 2kN/m^2*

For the top story: *Useful load 2kN/m^2 ; Snow load 2kN/m^2; Heat insulation 3kN/m^2; Building services 0.5 kN/m^2 ; Attic – on the border beams – 5 kN/m^2.*

The concrete used is C20/25 class with an elasticity module E=33000 N/mm^2. Here are the dimensions of the components:

Table 4. Dimensions of the components.

Structure type	**Height regime**	**Column side (cm)**	**Longitudinal beam (cm)**	**Cross beam (cm)**
Framed structure	40 levels	130	35x80	35x80
	30 levels	130	35x80	35x80
	20 levels	100	35x65	35x65
	10 levels	80	30x60	30x60

The following steps were considered for setting the dimensions of tuned mass dampers:

In the first phase, the location (position) of the damper is set, in order to coincide with the maximum amplitude point of the modal shape to be monitored.

The primary system mass will be the participant mass on the module, to be monitored with the damper

The higher the ratio of the masses μ the lower the response (the damper is less affected by the tuning). Therefore, the highest possible ratio is chosen.

The optimal ratio of frequencies (periods) F *is determined, then used to compute the damper rigidity* $k_d = m_d\omega_d^2$

The optimal fraction of the critical damping $\zeta_{d,opt}$ *is determined, then used to compute the device damping,* $c_{d,opt} = 2\zeta_{d,opt}\omega_d m_d$.

Modeling of a tuned mass system:

As previously described, the Tuned Mass Damper must be positioned in the maximum amplitude point of the mode to be monitored. In the cases under study, this point can be any point on the floor of the last storey.

The TMD system is modeled with the help of a friction pendulum-type insulator, in order to simulate the physical movement of the TMD, while for the damping part a viscous damper-type link was used. Two links with these names are included in the program, with characteristics determined by computation.

The modeling must consider the link design mode (upwards) in order to correctly define the damping movement. For good operation, the point on the floor of the last storey will be set in its center, while the free one will be set on the same vertical axis, at an equal distance to the pendulum length.

The free point must also support masses on the main directions; these masses are determined through TMD computation.

μ – ratio of the masses; ζ – fraction of the critical damping; mode.1 – mass moved on the vibration mode 1; mode.2 – mass moved on the vibration mode 2.

$$m_d = \mu \times m_{mod,1} \quad (22)$$

$$f_{opt} = \frac{1}{1+\mu} - (0.241 + 1.7\mu - 2.6\mu^2)\zeta - (1.0 - 1.9\mu + \mu^2) \quad (23)$$

$$\zeta_{d,opt} = \sqrt{\frac{3\mu}{8+(1+\mu)}} + (0.13 + 0.12\mu + 0.4\mu^2)\zeta - (0.01 + 0.9\mu + 3\mu^2)\zeta^2 \quad (24)$$

$$f_d = f_{mod,1} \times f_{opt} \tag{25}$$

$$T_d = \frac{1}{f_d} \tag{26}$$

$$\omega^2 = 2\pi f_d \tag{27}$$

$$k_d = m_d \omega_d^2 \tag{28}$$

$$c_{d,opt} = 2\zeta_{d,opt}\omega_d m_d \tag{29}$$

$$L_{pendul} = \frac{{T_d}^2 \times g}{4\pi^2} \tag{30}$$

For the structures included in the analysis, the TMD characteristics are the following:

Table 5.TMD characteristics.

Structure type	Plane shape	Pendulum length	Rigidityk_d [kN/m]	Device damping c_d [kN/s]	Mode 1 mass[kN]	Mode 2 mass [kN]
Structure in frames	40 levels	2.4	21248	4032	5209	5275
	30 levels	1.1	33252	4265	3276	3757
	20 levels	0.7	28376	2872	1979	1987
	10 levels	0.15	49765	2540	883	885
	5 levels	1.7	31128	4934	5325	5390

The following is a comparative presentation of the structures' behavior with and without the TMD. Details are presented for a 40levels building, along with the other structural responses, in short.

6.1. FRAMED STRUCTURE 40 levels

Following the modal analysis with a response range, the following responses resulted for the first four vibration modes:

Table 6. Responses resulted for the first four vibration modes.

Mode	Structure without TMD				Structure with TMD			
	Period	Ux	Uy	Rz	Period	Ux	Uy	Rz
1	2.769	0	0.8001	0	3.398	0	0.4197	0
2	2.62	0.8102	0	0	3.312	0.3573	0	0
3	2.333	0	0	0.8274	2.457	0	0.3916	0
4	0.895	0	0.1167	0	2.384	0.4633	0	0

Note that, when the TMD is mounted, the periods increase, and the placement of masses is fundamentally changed. Thus:

In the first case (without TMD) on modules 1 and 2 the mass participation factor was 80%, the result being high seismic forces; After mounting the TMD, these factors of mass participation are divided into two and distributed on 2 modules each, on each main direction. The general torsion phenomenon "decreases" beyond module 4.

If the movements are analyzed, significant drift decreases can be noted – an average of 20% along the axes X and Y, both for the SLS and the ULS, as shown in the diagrams below:

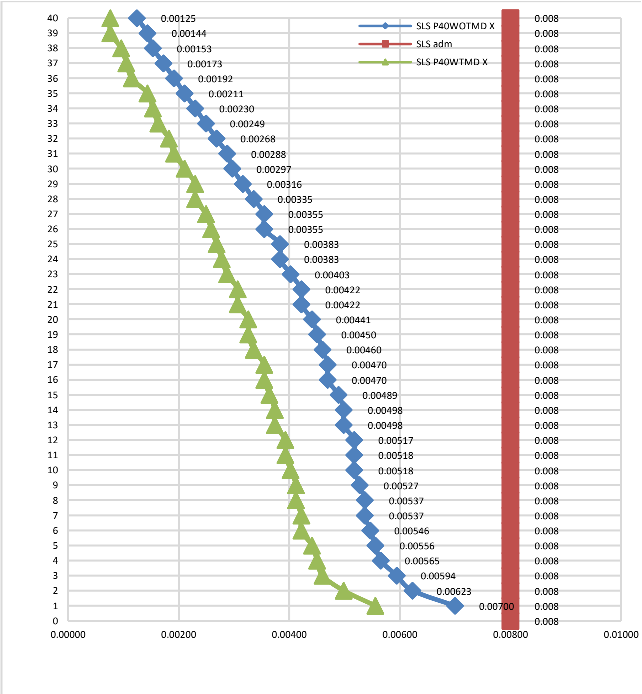


Figure 12 Relative displacements in direction X to SLS

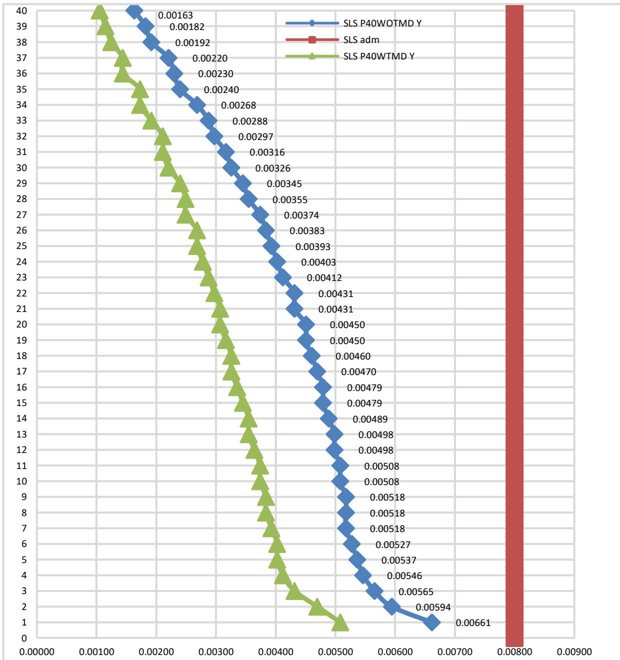


Figure 13.Relative displacements in direction Y to SLS

In order to point out the TMD behavior and its effect in reducing stresses in the components, the following were selected:
A marginal column – named C19; An inner column – named C15; A beam – named B2.
A decrease of the stresses in components can be noted:
In the marginal column C19 the axial stress decreased on average by 5%, and the bending moment decreased on average by 25%;

In the inner column C15 the axial stress decreased on average by 8% and the bending moment decreased on average by 25%
In the left-hand support of the beam B2 the bending moments decreased on average by 25%, while in the right-hand support they decreased on average by 30%.

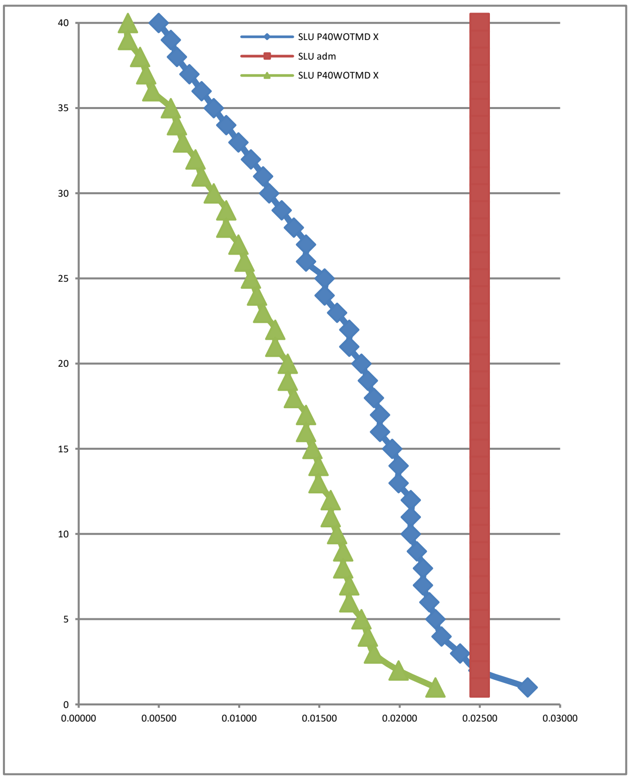


Figure 14 Relative displacements in directionX to ULS

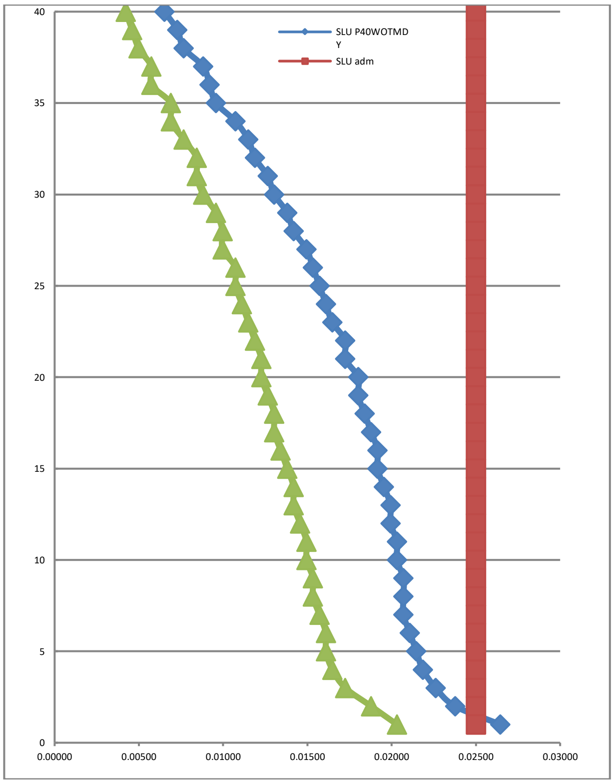


Figure 15 Relative displacements on Y direction to ULS

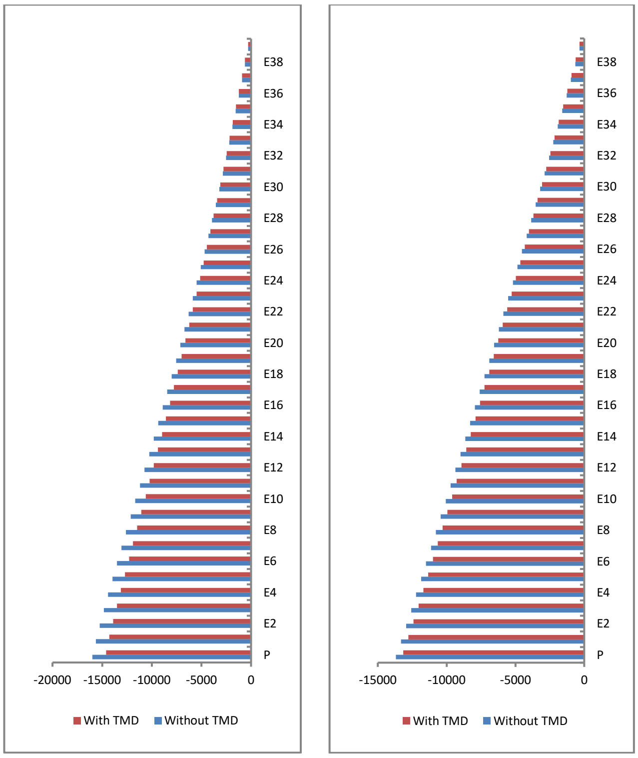


Figure 16 Comparison of axial forces for different columns

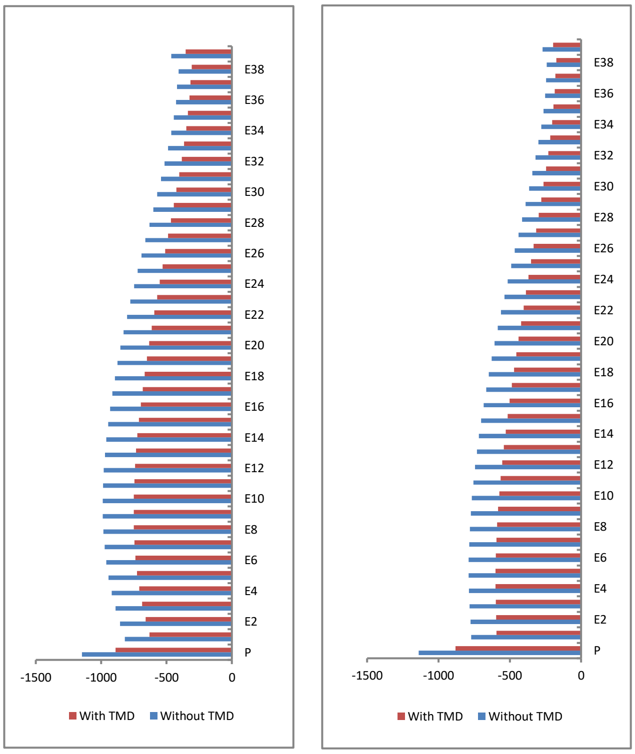


Figure 17. Comparison of bending moments for different columns

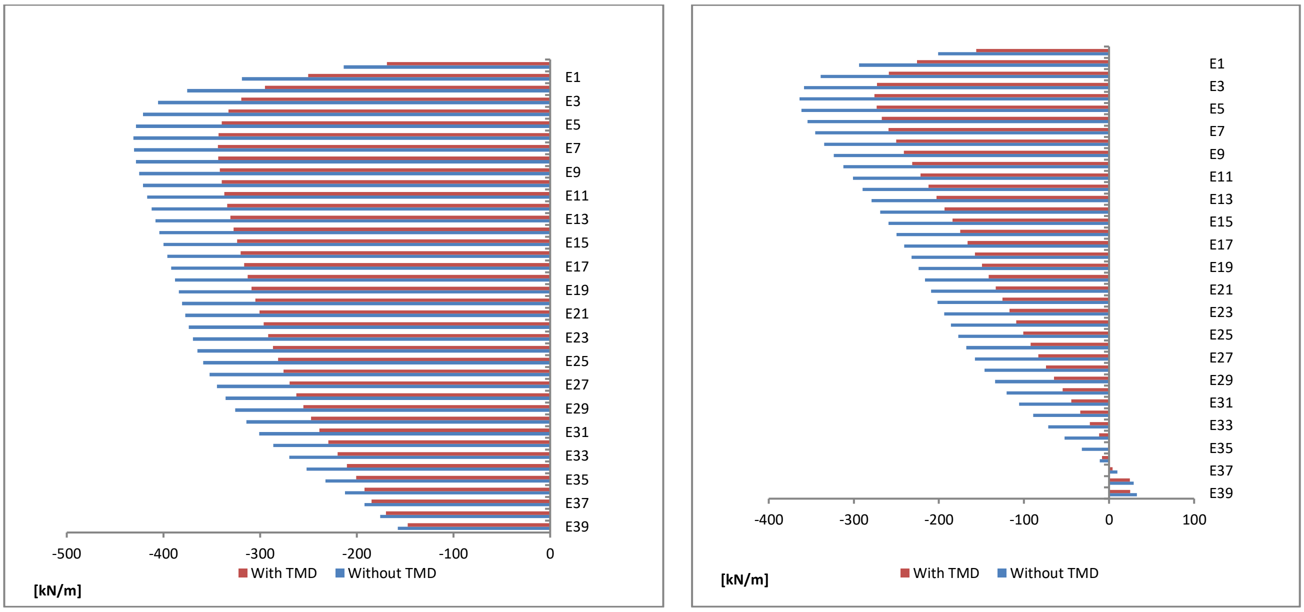


Figure 18 Comparison of bending moment for one beam

6.2. FRAMED STRUCTURE 30 LEVELS

Following the modal analysis with a response range the following periods resulted for the first four vibration modes:

Table 7. Modal analysis with a response range resulted for the first four vibration modes

Mode	Structure without TMD				Structure with TMD			
	Period	Ux	Uy	Rz	Period	Ux	Uy	Rz
1	1.872	0	0.7629	0	2.589	0	0.4573	0
2	1.793	0.7693	0	0	2.531	0.4279	0	0
3	1.623	0	0	0.7789	1.623	0	0	0.7789
4	0.597	0	0.1077	0	1.58	0	0.3334	0

Note that, when the TMD is mounted, the periods increase, and the placement of masses is fundamentally changed. Thus:

In the first case (without TMD) on module 1 the mass participation factor was 76.29%, while on module 2 the mass participation factor was 76.93%;

After mounting the TMD, these factors of mass participation are divided into two and distributed on 2 modules each, on each main direction. The general torsion phenomenon "decreases" beyond module 4.

If the movements are analyzed, significant drift decreases can be noted – an average of 20% along the axes X and Y, both for the SLS and the ULS.

In order to point out the TMD behavior and its effect in reducing stresses in the components, the following were selected: *A corner column – named C32; A marginal column – named C19; An inner column – named C15; A beam – named B20.*

A decrease of the stresses in components can be noted:

In the corner column C32 the axial force decreased on average by 8%, and the bending moment decreased on average by 20%;

In the marginal column C19 the axial stress decreased on average by 5%, and the bending moment decreased on average by 25%;

In the inner column C15 the axial stress decreased on average by 5% and the bending moment decreased on average by 18%

In the supports of beam B20 the bending moments decreased on average by 25%.

6.3. FRAMED STRUCTURE 20 levels

Following the modal analysis with a response range the following periods resulted for the first four vibration modes:

Table 8. Modal analysis with a response range resulted for the first four vibration modes.

Mode	Structure without TMD				Structure with TMD			
	Period	Ux	Uy	Rz	Period	Ux	Uy	Rz
1	1.477	0	0.7694	0	2.05	0	0.4768	0
2	1.434	0.7726	0	0	2.018	0.457	0	0
3	1.323	0	0	0.7761	1.323	0	0	0.7761
4	0.469	0	0.1009	0	1.237	0	0.3217	0

Note that, when the TMD is mounted, the periods increase, and the placement of masses is fundamentally changed. Thus:

In the first case (without TMD) on modules 1 and 2 the mass participation factors were 76.47% and 77.26% resulting in high seismic forces

After mounting the TMD, these factors of mass participation are divided into two and distributed on 2 modules each, on each main direction. The general torsion phenomenon "decreases" beyond module 4.

If the movements are analyzed, significant drift decreases can be noted – an average of 20% along the axes X and Y, both for the SLS and the ULS.

In order to point out the TMD behavior and its effect in reducing stresses in the components, the following were selected: *A corner column – named C32; A marginal column – named C19; An inner column – named C12; A beam – named B20.*

A decrease of the stresses in components can be noted:

In the corner column C32 the axial force decreased on average by 7%, and the bending moment decreased on average by 10%;

In the marginal column C19 the axial stress decreased on average by 6%, and the bending moment decreased on average by 20%;

In the inner column C12 the axial stress decreased on average by 4% and the bending moment decreased on average by 15%

In the supports of beam B20 the bending moments decreased on average by 10%.

6.4. FRAMED STRUCTURE 10 LEVELS

Following the modal analysis with a response range the following periods resulted for the first four vibration modes:

Table 9. Modal analysis with a response range resulted for the first four vibration modes:

Mode	Structure without TMD				Structure with TMD			
	Period	Ux	Uy	Rz	Period	Ux	Uy	Rz
1	0.745	0	0.7748	0	1.033	0	0.5006	0
2	0.728	0.7769	0	0	1.021	0.4859	0	0
3	0.678	0	0	71.0368	0.678	0	0	0
4	0.23	0	0.1036	0	0.615	0	0.3046	0

Note that, when the TMD is mounted, the periods increase, and the placement of masses is fundamentally changed. Thus:

In the first case (without TMD) on modules 1 and 2 the mass participation factors were 77.48% and 77.69% resulting in high seismic forces.

After mounting the TMD, these factors of mass participation are divided into two and distributed on 2 modules each, on each main direction. The general torsion phenomenon "decreases" beyond module 4.

If the movements are analyzed, significant drift decreases can be noted – an average of 2% along the axes X and Y for the SLS and 1.5% for the ULS.

From the point of view of movement reduction, the use of TMD for the P+10E is not justified, the solution is not applicable.

In order to point out the TMD behavior and its effect in reducing stresses in the components, the following were selected: *A corner column – named C32; A marginal column – named C19; An inner column – named C12 ;A beam – named B20.*

A decrease of the stresses in components can be noted:

In the corner column C32 the axial force decreased on average by 1%, and the bending moment decreased on average by 2%;

In the marginal column C19 the axial stress decreased on average by 1%, and the bending moment decreased on average by 2%;

In the inner column C12 the axial stress decreased on average by 2% and the bending moment decreased on average by 4%

In the supports of beam B20 the bending moments decreased on average by 10%.

7. CONCLUSIONS

We noted that by adding TMDs the initial participation factors of around 75% for translation on x (mode 1), translation on y (mode 2) and torsion (mode 3) are significantly modified: both module 1 and 2 become translations on x, with mass participation factors of 37.5%; modules 2 and 4 become translations on y with mass participation factors of 37.5% (all being pure translations, without torsion), while modules 5 and 6 become torsion modes with mass participation factors of 37.5%.

We also noted an increase by 1.4 times of the vibration periods, for the basic vibration modes.

Following the dynamic linear analysis with a response range for all the structures in the two cases mentioned, the following general conclusions were drawn. They are divided below as advantages and disadvantages:

Advantages:

A relatively simple mechanism and easy to make;
Higher reliability resulting from its simplicity;
Relative movements per level decrease on average by around 25%;
Axial stresses in columns and/or structural reinforced concrete walls are reduced by about 5%;
The bending moments in columns decreased by about 20%;
The bending moments in structural reinforced concrete walls decreased by about 10%;
The bending moments in beam supports decreased by around 25%;
Increased damping of structural system oscillations;
Easy computation of TMD characteristics;
In general, they are efficient for buildings with a medium and great height regime.

Disadvantages:

Local load of structural elements with significant gravitational forces;
The linking system to the structure must be designed in order to distribute the load on as many structural elements as possible, so as not to determine major, concentrated local effects;
Problems arise when the pendulum is longer than the level height and when enough empty space must be provided for its oscillation;
They are not efficient for low-rise buildings.

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