Effectiveness of tuned mass dampers for improved seismic performance of dual eccentrically braced steel frames

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Abstract. Tuned mass dampers (TMDs) have been widely used for the control of seismic response of various new or existing buildings, successfully. However, the use of TMDs in the case of dual eccentrically braced steel frame structures has not been found in the technical literature. The paper appraises numerically the efficiency of TMDs in enhancing the seismic response of dual eccentrically braced steel frame structures and investigates the influence of mass of the TMD on the level of the response reduction achieved. To achieve these goals, a typical mid-rise five story steel frame structure with perimeter dual eccentrically braced frames, located in Bucharest, has been considered. Then, bidirectional TMDs with three mass ratios 0.04, 0.06 and 0.08 have been added on the top floor of the building, without redesign the frame members. Comparative nonlinear time history analyses of the unequipped and equipped structure have been performed with accelerograms of Vrancea 1977 type. Numerical results (peak base shears, peak inter-story drifts ratio and peak absolute story accelerations) show the efficiency of TMDs in mitigating the seismic response of dual eccentrically braced steel frames.

1. Introduction
Dual eccentrically braced steel frames are constituted by moment resisting frames (MRFs) and eccentrically braced frames with short links (EBFs). These frames combine the main advantages of MRFs (high energy dissipation capacity) and EBFs (high lateral stiffness), while minimizing their disadvantages (significant inter-story drifts at the upper stories of MRFs and great story shears at the lower stories of EBFs) [1].

A TMD is one of the most reliable passive devices for reducing the response of buildings subjected to wind and earthquakes, due to the advantage of an easy installation, non-interference with vertical and horizontal load paths and low maintenance costs [2, 3].

A TMD consists of a mass, spring and a damper which is attached to a story of the structure with the aim of reducing the structural response. The frequency of the TMD is tuned to the first natural frequency of the structure. When that frequency is excited, the TMD will resonate out of phase with the structural movement. The TMD dissipates energy, when its mass oscillates relative to the structure [4, 5].

The current TMD originates from a vibration absorber device, invented by Frahm in 1909 [6]. Over the last few decades, TMDs have been installed in many buildings such as Centerpoint Tower (Sidney, Australia), Citicorp Center (New York, USA), John Hancock (Boston, USA), Burj Al Arab (Dubai, UAE) etc. – to mitigate wind-induced vibrations and Taipei 101 (Taipei, Taiwan) – to mitigate the seismic response of building [3, 7].

Several papers have derived the optimum parameters of tuned mass dampers that reduce the seismic response of single and multiple degree of freedom structures [7, 8, 9]. The parameters of TMDs
have been developed considering different optimization criteria of the structure: minimum displacement of the structure, minimum velocity of the structure, minimum acceleration of the structure etc. [6].

Studies regarding the optimal placement of the TMDs in the structure recommend locating the damper at the top floor, where the displacement amplitude of the fundamental mode is highest [8].

Several researchers have investigated numerically or by experimental studies the effectiveness of TMDs in mitigating the seismic response of various types of structures (symmetrical or asymmetrical - steel or reinforced concrete moment resisting frames) [2, 3, 8, 9, 10, 11, 12, 13].

To the knowledge of the authors, this paper represents the first study regarding the seismic protection of dual eccentrically braced steel frame structures by using TMDs.

2. Description of the structure

The seismic performance of dual eccentrically braced frames equipped with TMDs has been investigated by numerical simulations on five story office building, located in Bucharest.

The building has a square plan (18x18 m), as shown in figure 1. The structure is formed by two groups of four planar frames oriented along two orthogonal directions. The perimeter moment resisting frames (MRFs) are eccentrically braced in the middle bay, being dual frames (figure 2), while the internal frames are MRFs. The floors are treated as rigid diaphragms, their thickness being 15 cm.

![Figure 1. Plan view of the building.](image)

The structure has been designed according to P100-1 (2013) and EN 1993-1-1 (2005) requirements, considering a dissipative behaviour with ductility class high (DCH) [14, 15].

Two limit states have been checked:
- Ultimate Limit State (ULS) - for an earthquake with a reference return period of 225 years;
- Serviceability Limit State (SLS) – for an earthquake with a reference return period of 40 years, considering the inter-story drift limit as 0.5% of the story height.

Modal response spectrum analysis has been performed independently for the ground motion in two horizontal directions. The main characteristics of the design spectrum are:
- peak ground acceleration: a_g=0.30g;
- corner period: T_c=1.6 s;
- behaviour factor: q=6 (dual eccentrically braced steel frames with DCH).
The overstrength of the structural system, $\Omega_T=2.5$, has been used for the design of non-dissipative members. Steel grade S235 has been used for dissipative members (links and beams), while steel grade S355 for the other members. The characteristic value of dead load is 5.5 kN/m², except the roof level where it is 5.5 kN/m², while the characteristic value of live loads is 2 kN/m² for all floors. The structural member sections are tabulated in Tables 1-4.

**Table 1.** Member cross sections for frames from axes 1 and 4.

| Story | Exterior columns | Interior columns | Current beams | Middle bay beams | Braces |
|-------|------------------|------------------|---------------|------------------|--------|
| 4-5   | HEM 160          | HEM 160          | IPE 240       | IPE 400          | HEB 200|
| 1-3   | HEM 160          | HEM 260          | IPE 240       | IPE 600          | HEB 240|

**Table 2.** Member cross sections for frames from axes 2 and 3.

| Story | Exterior columns | Interior columns | Beams | Middle bay beams |
|-------|------------------|------------------|-------|------------------|
| 4-5   | HEM 160          | HEM 220          | IPE 300| IPE 300          |
| 1-3   | HEM 260          | HEM 280          | IPE 330| IPE 330          |

**Table 3.** Member cross sections for frames from axes A and D.

| Story | Current beams | Middle bay beams | Braces |
|-------|---------------|------------------|--------|
| 4-5   | IPE 240       | IPE 400          | HEB 200|
| 1-3   | IPE 240       | IPE 600          | HEB 240|

**Table 4.** Member cross sections for frames from axes B and C.

| Story | Interior columns | Current and middle bay beams |
|-------|------------------|------------------------------|
| 4-5   | HEM 220          | IPE 300                      |
| 1-3   | HEM 280          | IPE 330                      |
Then, the structure has been equipped with bidirectional TMDs, located on the top floor, without redesigning its members.

Mass ratio $\mu$ is defined as [6]:

$$\mu = \frac{m_d}{M_{tot}}$$  \hspace{0.5cm} (1)

where $m_d$ is the damper mass and $M_{tot}$ is the total mass of unequipped structure.

The influence of mass of the TMDs on the seismic response has been studied for three mass ratios: 0.04, 0.06 and 0.08 [16, 17].

The design of the TMD involves specifying the mass, stiffness and damping coefficient. Den Hartog method has been used to compute these parameters [18, 19, 20]:

$$\alpha_{opt} = \frac{1}{1+\mu} \sqrt{\frac{2-\mu}{2}}$$  \hspace{0.5cm} (2)

where $\alpha_{opt}$ is the optimum frequency ratio;

$$\zeta_{opt} = \frac{3\mu}{8(1+\mu)} \sqrt{\frac{2}{2-\mu}}$$  \hspace{0.5cm} (3)

where $\zeta_{opt}$ is the optimum damping ratio of TMD;

$$k_d = 4\pi^2 \mu \alpha_{opt}^2 \frac{m_d}{T^2}$$  \hspace{0.5cm} (4)

where $k_d$ is the stiffness of TMD;

$$c_d = 4\pi \mu \alpha_{opt} \zeta_{opt} \frac{m_d}{T}$$  \hspace{0.5cm} (5)

where $c_d$ is the damping coefficient of TMD.

3. Numerical analyses and discussions

To assess the influence of the TMD mass on the seismic response of the dual eccentrically braced frames, nonlinear time history analyses have been performed by the program SAP 2000. This is a structural analysis program with capabilities for nonlinear dynamic analysis of structures equipped with energy dissipation devices. The researches have shown that results obtained by SAP 2000 for structures equipped with energy dissipation devices are in good agreement with the results obtained by ANSYS and with experimental results [21]. This is the reason why SAP 2000 has been used for the numerical studies.

The seismic loading is represented by seven sets of semi-artificial accelerograms of Vrancea 1977 type (NS and EW components). The seismic performance of the structures has been analyzed for Ultimate Limit States (ULS) and Serviceability Limit States (DLS).

Peak base shears, which represent the maximum lateral forces that appear at the base of the structure during an earthquake; peak inter-story drift ratios, that are related with non-structural drift sensitive components damages; and peak absolute story accelerations that are related with non-structural acceleration sensitive components damages, have been selected as response parameters [22]. Due to space considerations, the response parameters are graphically represented only for the dual eccentrically braced frames along the X direction. Comments refer to dual frames from both directions.

3.1. Base shears

As an example, base shear response history in the case of equipped structure with mass ratio of 0.08, at ULS, along X direction, for one set of semi-artificial accelerograms of Vrancea 1977 type, compatible with the response spectrum for Bucharest, is depicted in Figure 3. For this set of semi-artificial accelerograms, the peak value of base shear, at ULS, along X direction, is extracted from the graph. Similarly, in the case of equipped structure with mass ratio of 0.08, at ULS, along X direction, the peak base shears are extracted from the corresponding graphs of the other 6 sets of semi-artificial accelerograms. Finally, the mean of the 7 values is computed. The same procedure has been applied for all considered cases (unequipped and equipped), for all response parameters.
Figure 3. Base shear response history in the case of equipped structure with mass ratio of 0.08, at ULS, in X direction, obtained for one set of accelerograms of Vrancea 1977 type.

Figure 4 shows the average value (of the seven-time history analyses) of the maximum values of the base shears at ULS, of the equipped and unequipped structure, along X direction.

It can be observed that by adding TMDs to the structures, the values of the maximum base shears have been significantly decreased in all cases. The reductions are slightly higher in the Y direction.

In the X direction, the peak base shear has been reduced by 29.4% in the case of the equipped structure with mass ratio of 0.04, by 35.72% in the case of the structure with mass ratio of 0.06 and by
39.65% in the case of the structure with mass ratio of 0.08, compared to the unequipped structure. In the direction Y, the reductions have ranged from 31.55% to 41.16%.

3.2. Inter-story drift ratios
Figure 5 shows the distribution of the peak inter-story drift ratios over the height of structure, at SLS, for the perimeter moment dual eccentrically braced frame, along X direction.

By equipping the structure with TMD, the peak inter-story drifts along X direction have been reduced by 27.32%-29.11% for the structure with mass ratio of 0.04, by 34.05%-35.27% for the structure with mass ratio of 0.06, and by 37.90%-39.55% for the structure with mass ratio of 0.08, compared to the unequipped structure. The distribution of inter-story drift ratios over the height of structure is similar along the Y direction, the reductions being slightly higher. Along Y direction, the reductions have ranged from 27.65%-41.11%. It is worth mentioning that on both directions, reductions of inter-story drifts over the height of structure are approximately constant, slightly higher values being registered at the lower floors.

3.3. Absolute story accelerations
Figure 6 illustrates the distribution of the peak absolute story accelerations over the height of structure, at ULS, for the perimeter moment dual eccentrically braced frame, along X direction.

In the case of unequipped structures, it can be observed an increase of lateral accelerations from the lower floors to the upper floors. By equipping the structure with TMDs, the maximum lateral accelerations along X direction have been reduced by 17.45%-20.12% for the structure with mass ratio of 0.04, by 23.58%-25.47% for the structure with mass ratio of 0.06, and by 27.70%-29.58% for the structure with mass ratio of 0.08, compared to the unequipped structure. Slightly higher reductions have been obtained along Y direction. These reductions have ranged from 18.7% to 31.3%. The distribution of the reductions over the frame height is approximately constant at all floors, for all equipped structures.
4. Conclusions
A 3D five story structure with perimeter dual eccentrically braced frames, located in Bucharest, has been chosen for this study. Then, the building has been equipped at the top floor with bidirectional TMDs with three mass ratios 0.04, 0.06 and 0.08, without redesigning the structural members. The influence of mass ratios on the seismic response as well the efficiency of the TMDs in enhancing the seismic response of bare structure has been investigated through nonlinear time history analyses.

The main findings from this research are summarized as follows:
1. The analysed seismic response parameters (peak base shears, peak inter-story drift ratios and peak absolute lateral story accelerations) have been reduced for all equipped structures, in orthogonal directions X and Y, in comparison with the bare structure. The reductions are higher, as the mass ratios are higher.
2. Base shears have been reduced by 29.4%-31.55% for mass ratio 0.04, by 35.72%-37.31% for mass ratio 0.06 and by 39.65%-41.16% for mass ratio 0.08.
3. Inter-story drift ratios have been reduced by 27.32%-29.82.55% for mass ratio 0.04, by 34.05%-36.30% for mass ratio 0.06 and by 37.90%-41.11% for mass ratio 0.08.
4. Lateral absolute story accelerations have been reduced by 17.45%-21.4% for mass ratio 0.04, by 23.58%-26.80% for mass ratio 0.06 and by 27.70%-31.30% for mass ratio 0.08.

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