**Structural Engineering and Mechanics**, *Vol. 66, No. 2 (2018) 217-227*
DOI: https://doi.org/10.12989/sem.2018.66.2.217


# Effect of soil in controlling the seismic response of three-dimensional PBPD high-rise concrete structures

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*(Received January 18, 2017, Revised November 26, 2017, Accepted January 23, 2018)*

**Abstract.** In the last decades, valuable results have been reported regarding conventional passive, active, semi-active, and hybrid structural control systems on two-dimensional and a few three-dimensional shear buildings. In this research, using a three-dimensional finite element model of high-rise concrete structures, designed by performance based plastic design method, it was attempted to construct a relatively close to reality model of concrete structures equipped with Tuned Mass Damper (TMD) by considering the effect of soil-structure interaction (SSI), torsion effect, hysteresis behavior and cracking effect of concrete. In contrast to previous studies which have focused mainly on linearly designed structures, in this study, using performance-based plastic design (PBPD) design approach, nonlinear behavior of the structures was considered from the beginning of the design stage. Inelastic time history analysis on a detailed model of twenty-story concrete structure was performed under a far-field ground motion record set. The seismic responses of the structure by considering SSI effect are studied by eight main objective functions that are related to the performance of the structure, containing: lateral displacement, acceleration, inter-story drift, plastic energy dissipation, shear force, number of plastic hinges, local plastic energy and rotation of plastic hinges. The tuning problem of TMD based on tuned mass spectra is set by considering five of the eight previously described functions. Results reveal that the structural damage distribution range is retracted and inter-story drift distribution in height of the structure is more uniform. It is strongly suggested to consider the effect of SSI in structural design and analysis.

**Keywords:** performance-based plastic design (PBPD); tuned mass damper (TMD); high rise reinforced concrete structure; soil-structure interaction; inelastic analysis

## 1. Introduction

Many researches and advances in recent years have identified Tuned Mass Damper (TMD) as one of the most widely utilized and approved response control systems for high-rise buildings (Lu and Chen 2011, Lu and Chen 2011, Chung *et al.* 2013). TMD is one of the simplest and most effective passive control systems with low repair and maintenance costs, proposed to enhance the performance of the structures against environmental loads (Pourzeynali *et al.* 2013, Mohebbi *et al.* 2015). TMD is composed of a mass block connected to the structure via a spring and a viscous damper. Previous studies on the structures equipped with TMD can be categorized into three general groups: (1) researches that focused on finding the optimal parameters of TMD (Warburton and Ayorinde 1980, Warburton 1982, Tsai and Lin 1993, Hadi and Arfiadi 1998, Jangid 1999, Bakre and Jangid 2004, Lee *et al.* 2006, Bakre and Jangid 2007, Sgobba and Marano 2010, Bekdaş and Nigdeli 2013, Chung *et al.* 2013, Greco and Marano 2013, Nigdeli and Bekdas 2013, Aguirre and Almzán 2014, Nigdeli and Bekdas 2014, Nigdeli and Bekdaş 2017); (2) efforts to evaluate the efficiency of TMD, multiple TMDs and active/semi-active TMDs in reducing the dynamic response generated by lateral loadings in the elastic domain of structures (McNamara 1977, Kaynia *et al.* 1981, Sladek and Klingner 1983, Xu and Igusa 1992, Yamaguchi and Harnpornchai 1993, Abé and Fujino 1994, Kareem and Kline 1995, Jangid and Datta 1997, Jangid 1999, Li 2000, Wang *et al.* 2001, Li 2002, Li and Liu 2003, Pinkaew *et al.* 2003, Bakre and Jangid 2004, Bakre and Jangid 2007, Lewandowski and Grzymisławska 2009, Shooshtari and Mortezaie 2012, Li and Cao 2015, Mevada 2015) which has been proven since the 1970s; and (3) discussions on aspects such as the non-linearity of seismic behavior of the structures and their effect on efficiency and optimal parameters of TMD (Wong and Johnson 2009, Wong and Harris 2010, Zhang and Balendra 2013, Aguirre and Almzán 2014, Shooshtari and Mortezaie 2017).

Nevertheless, most of these studies did not considered concerns related to their modeling, simulation, effect of soil on structural response and structural design. Consequently, limited researches related to structures with a TMD are available which consider the effect of soil-structure interaction (SSI). SSI is important for both static and dynamic loads and related literatures investigate at least 30 years of analytical approaches for solving SSI problems. SSI is important, especially for stiff and massive structures on relatively soft grounds, which may significantly change the dynamic characteristics of the structural response (Mihailo *et al.* 2001). The common inelastic design procedures excluding SSI are no longer efficient to ensure the structural safety for

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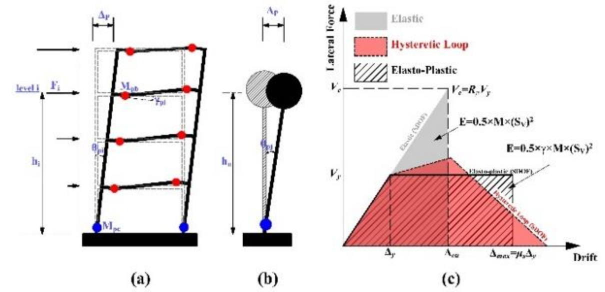



Fig. 1(a) Desired yield mechanism of special moment frame (SMF) (b) Equivalent single degree of freedom (SDOF) structure (c) Energy-work balance concept

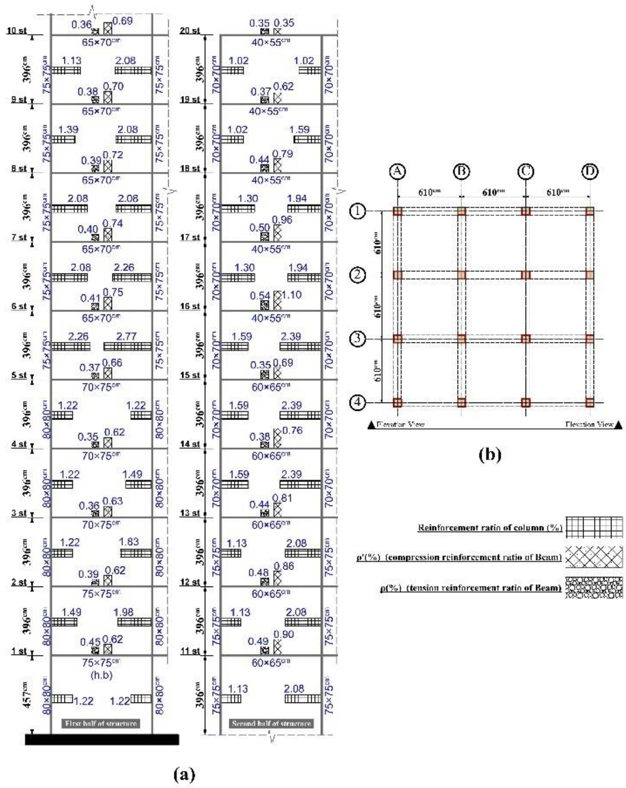



Fig. 2(a) Elevation view and (b) Plan of designed PBPD structure

the building frames placed on soft soil deposits. SSI can significantly amplify the lateral displacements and inter-story drifts that, may change the performance (Fatahi *et al.* 2011, Tabatabaiefar *et al.* 2015).

Takewaki (2000) has presented a systematic method for optimal viscous damper placement in structures with SSI effect by a combination of viscous damper and TMD. Ghosh and Basu by studying the effects of SSI on the performance of TMDs for seismic vibration, showed that, as the soil stiffness decreases, the structural properties show considerable changes in comparison to the fixed base structure (Ghosh and Basu 2004). Dynamic vibration control of irregular buildings, modeled as torsional coupled structures equipped with multiple tuned mass damper (MTMD), has been investigated by Wang and Lin (2005). They suggested that, to avoid overestimating the effectiveness of MTMD for irregular buildings located on soft soils, the SSI effects should be considered in order to determine the optimal MTMD parameters. Another research has indicated the performance of 10-, 15-, and 20-story linear steel shear structures with TMD, under near-field ground motion earthquakes with linear and nonlinear SSI effect (Khoshnoudian *et al.* 2016). In recent years, some studies have been carried out in order to determine optimized parameters of TMD by considering the effect of SSI on two-dimensional 40-story shear building frame with linear behavior (Farshidianfar and Soheili 2013, Khatibinia *et al.* 2016, Bekdaş and Nigdeli 2017, Mortezaie and Rezaie 2018).

On the other hand, regardless of the type of structure, linear or nonlinear behavior of the materials, type of structural analysis and considering issues such as torsion, an issue that has not been covered is the fact that, the actual response of the structure associated with the design philosophy and construction of the structure and unlike the gravity loads, earthquake seismic input energy is a function of structural properties and its design. Therefore, to model what happens in reality, a true three-dimensional model which is closest to the actual behavior of the structure is needed. In this study, nonlinear behavior of the structures has been considered from the beginning of the design stage. In 2008, Goel and Chao presented a method called performance based plastic design (PBPD) for structural steel and then, in 2010 this method was developed for special moment concrete structures (Goel and Chao 2008, Liao 2010). In contrast to the current code design method, the design base shear force in PBPD for a selected risk level is obtained by considering the work required to move the structure toward the target relative displacement, equal to single degree of freedom structure (SDOF). In this method, more predictable structural performance can be obtained for the designed structure based on the performance limit states of drift target and desired yielding mechanism selected from the beginning of the design process. Although the current PBPD procedure works very well for low-rise buildings, but for mid to high-rise buildings, it works for over-size columns (Bayat 2010, Rezaie and Mortezaie 2017).

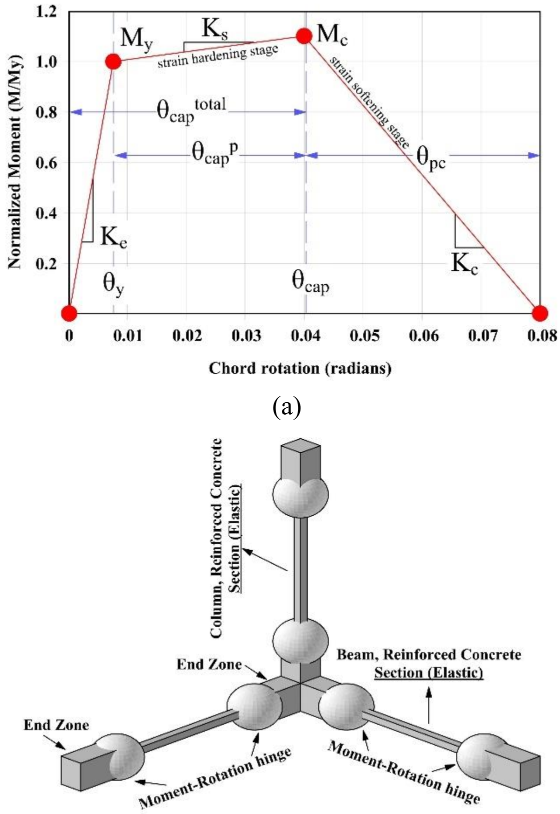
In this study, the efficiency of using TMD to enhance the performance of PBPD structure and reducing loads on structural columns designed by PBPD, with and without TMD, by considering the effect of SSI under 22 pairs of far-field earthquake records was reviewed and analyzed. By making a sophisticated three-dimensional finite element model of a concrete structure, and considering the cone model for the soil, the issues raised were investigated. The aim of this research is to study the aspects that are given less attention in literature as already mentioned.

## 2. Overview of PBPD procedure and properties of designed structure

A design of the step-by-step PBPD method has been provided in the following steps. Details of this design and its procedure can be found in studies carried out by Goel and Chao (2008) and Liao (2010) (Goel and Chao 2008, Liao 2010).

Table 1 Summary of design parameters for RC SMF

| Parameter | Range considered |
|---|---|
| Seismic design level | Design Category D |
| Compressive strength concrete for column and beam respectively, | 41.4-34.5 Mpa |
| Design floor dead load | 854.4 kg/m$^2$ |
| Design floor live load | 244.1 kg/m$^2$ |
| Yield drift Ratio | 0.5% |
| Target drift Ratio | 2% |
| Concrete cracking effect in beams | 0.5EI$_g$ (Venture 2000) |
| Concrete cracking effect in columns | 0.7EI$_g$ (Venture 2000) |




(b)

Fig. 3(a) Tri-linear curve, used to model beam-column elements (b) 3D models of beam-column elements

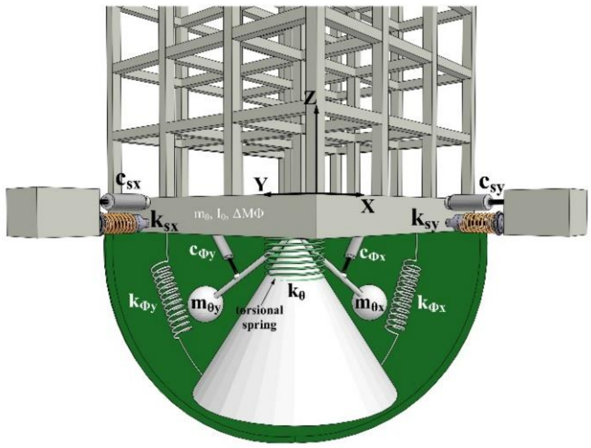



Fig. 4 Cone and spring-dashpot-mass model for foundation on surface of homogeneous half-space

Step 1:

Selection of the desired structural yield mechanism and drift target (Fig. 1(a)) for the design of earthquake event and its damages.

Step 2:

Estimation of structural yielding drift ($\theta_y$) and the fundamental period (T) and selecting an appropriate distribution of lateral force on the height of the structure.

Step 3:

Determination of the elastic spectral acceleration value of the design.

Step 4:

The design base shear calculated on the basis of the principles of design (Fig. 1(b)). If the behavior of structural materials does not follow elasto-plastic behavior, design base shear should be corrected at this step (Fig. 1(c)).

Step 5:

Designing members which are entering the nonlinear behavior in the selected mechanism (such as beams in reinforced concrete moment frame) will be done using a plastic design method; and the members which are remain elastic (such as columns), will be designed by capacity method.

By exactly following the design procedure of PBPD and design parameters described in Table 1, a 20-story reinforced concrete special moment frame was designed. The plan of the structure and details of the members section the PBPD structure are summarized and shown in Fig. 2(a) and 2(b), respectively.

## 3. Modelling of soil-foundation-structure system

### *3.1 Properties of structural model*

Structure nonlinear modeling based on incorrect and unrealistic methods can lead to incorrect and illogical responses. Even though there are many finite element models for reinforced concrete, most of them cannot be utilized to simulate structural collapse. Hysteretic model developed by Ibarra *et al*. (2005) and calibrated by Haselton (Haselton 2007, FEMA 2009) which is capable of capturing severe deterioration that precipitates sideway collapse was employed in this study. Fig. 3(a). illustrates the tri-linear monotonic backbone curve, which provides versatile modeling of cyclic behavior. This model can provide important modes of resistance deterioration which, lead to global sideway collapse. Those parameters include: initial stiffness ($K_e$), stiffness after yield ($K_s$), plastic rotation capacity ($\theta_{cap}{}^{p}$) and post-maximum-resistance rotation capacity ($\theta_{pc}$). The relevant equations have been proposed in FEMA P695 (2009).

One of the important details of this model is the capping point, where monotonic strength loss begins, and the post-capping negative stiffness models the strain-softening behavior associated with concrete crushing, rebar buckling,

Table 2 Specification of soil, foundation and key expressions used in 3D model soil-foundation (Wolf and Deeks 2004, FEMA 2009, ASCE 2010)

| Parameter | Range considered |
|---|---|
| Shallow foundation dimensions (length-width-height) | $1829\times1829\times125^{cm}$ |
| Compressive strength concrete for foundation | 41.4 Mpa |
| Average shear wave velocity | 182 to 365 m/s |
| Building site (Los Angeles, California) | High seismic site |
| Soil class | $S_d$ |
| $S_m$ | 1.5 g |
| $Sm_l$ | 0.9 g |
| Poisson's ratio (ν) | 0.3 |
| Shear wave velocity | 300 m/s |

Key Expressions to model a 3D foundation on a homogeneous soil half-space

| Motion | Horizontal | Rocking | | Torsional |
|---|---|---|---|---|
| Equivalent radius $r_0$ | $\sqrt{\frac{A_0}{\pi}}$ | $\sqrt[4]{\frac{4I_0}{\pi}}$ | | $\sqrt[4]{\frac{2I_0}{\pi}}$ |
| Aspect ratio $\frac{z_0}{r_0}$ | $\frac{\pi}{8}(2-\upsilon)$ | $\frac{9\pi}{32}(1-\upsilon)\left(\frac{c}{c_s}\right)^2$ | | $\frac{9\pi}{32}$ |
| Poisson's ratio $v$ | $all\,\upsilon$ | $\frac{1}{3}\leq\upsilon\leq\frac{1}{2}$ | $\upsilon\leq\frac{1}{3}$ | $\upsilon\leq\frac{1}{3}$ |
| Wave velocity c | $c_s$ | $2c_s$ | $c_p$ | $c_s$ |
| Trapped mass ΔM $\Delta M_\vartheta$ | 0 | $1.2\left(\upsilon-\frac{1}{3}\right)\rho I_0 r_0$ | 0 | 0 |
| Lumped-parameter model | $K=\rho c_s^2\frac{A_0}{z_0}$ | $K=3\rho c^2\frac{I_0}{z_0}$ | | $K=3\rho c_s^2\frac{I_0}{z_0}$ |
| | $C=\rho c_s A_0$ | $C=\rho c I_0$ | $M_\upsilon=\rho I_0 z_0$ | $C=\rho c_s I_0$ |

and fracture. To simulate the cyclic response of reinforced concrete beam-columns, a lumped plasticity approach is employed based on observations that are currently available for fiber element models which are not able to simulate the strain-softening associated with rebar buckling, and thus cannot reliably simulate collapse of the flexural dominated reinforced concrete frames. To simulate the behavior of structural beam-column, the beam-column element formed by an elastic element and two moment plastic hinges focused at two ends with zero length are used. This element was idealized and optimized as illustrated in Fig. 3(b).

### 3.2 Soil properties

Owing to the indefinite soil environment, its modeling is more complex than structural modeling. Rigorous methods exist to calculate the effect of SSI, including the sophisticated finite element methods like the thin-layer method (the consistent boundary method), the scaled boundary finite-element method and the Dirichlet-to-Neuman method. Nevertheless, these methods require significant computational time and experience. In fact, the use of cone model leads to decreased precision when compared to applying the rigorous methods of elasto-dynamics (Wolf and Deeks 2004).

Nevertheless, it is more compensated by many advantages like; physical insight with conceptual clarity, simplicity in usage and solving, sufficient generality (layered site, embedment, all frequencies) and acceptable engineering precision. Today there is an awareness that anyway, the accuracy of any analysis is limited owing to many uncertainties in determining the dynamic properties, some of which can never be neglected (i.e., soil properties and the definition of the dynamic loads). It is assumed that the structure is constructed on a rigid concrete shallow foundation. The parameters utilized for modeling soil-foundation are listed in Table 2. Fig. 4 shows a three-dimensional view of the 20-story structure with a cone model.

Table 3 The frequencies, and their corresponding periods, of the structural modes

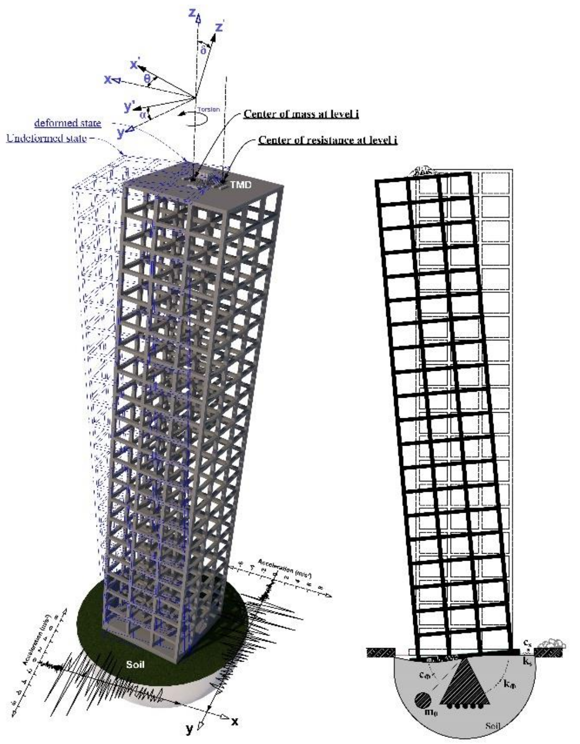
| Vibration modes | periods of vibration (second) | frequency of vibration (Hz) |
|---|---|---|
| first | 1.94 | 3.24 |
| second | 0.72 | 8.77 |
| third | 0.42 | 14.96 |
| fourth | 0.28 | 22.44 |
| fifth | 0.21 | 29.92 |

Table 4 Far-field ground motion record set used in this study (FEMA 2009)

| ID | Name | The dominant frequencies (Hz) | |
|---|---|---|---|
| | | X-direction | Y-direction |
| 01 | Cape Mendocino 1992 | 0.71 | 2.30 |
| 02 | San Fernando 1971 | 0.71 | 8.04 |
| 03 | Friuli Italy 1976 | 2.00 | 1.49 |
| 04 | Imperial Valley Delta 1979 | 2.53 | 0.60 |
| 05 | Imperial Valley El Centro 1979 | 4.13 | 3.78 |
| 06 | Superstition Hills El Centro 1987 | 0.83 | 0.39 |
| 07 | Superstition Hills Poe Road 1987 | 2.17 | 2.27 |
| 08 | Loma Prieta Capitola 1989 | 1.37 | 1.56 |
| 09 | Loma Prieta Gilroy 1989 | 1.73 | 0.51 |
| 10 | Landers Coolwater 1992 | 1.91 | 1.38 |
| 11 | Landers Yermo Fire Station 1992 | 0.72 | 0.71 |
| 12 | Northridge Beverly Hills 1994 | 1.17 | 1.90 |
| 13 | Northridge Canyon Country 1994 | 1.76 | 1.42 |
| 14 | Kobe Nishi Akashi 1995 | 2.08 | 1.37 |
| 15 | Kobe Shin-Osaka 1995 | 1.44 | 0.81 |
| 16 | Kocaeli Arcelik 1999 | 6.05 | 0.20 |
| 17 | Kocaeli Duzce 1999 | 0.27 | 0.54 |
| 18 | Chi Chi CHY101 1999 | 0.34 | 0.23 |
| 19 | Chi Chi TCU045 1999 | 1.56 | 1.32 |
| 20 | Duzce Bolu 1999 | 1.84 | 1.31 |
| 21 | Manjil Abbar 1990 | 2.93 | 4.58 |
| 22 | Hector Mine 1999 | 0.79 | 1.82 |

### 3.3 Ground motions and nonlinear analyses structure with TMD

Dynamic time history analysis has been generally



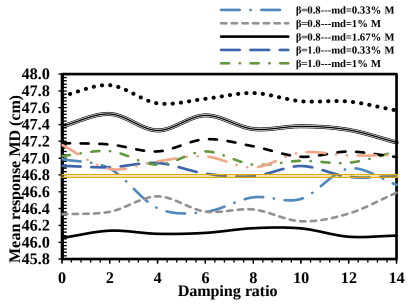

(a) (b)

Fig. 5(a) Three-dimensional finite element models of the structure under earthquake excitations, (b) elevation of soil-structure interaction model

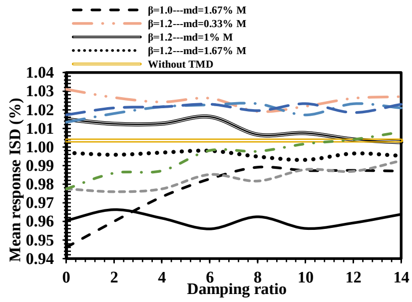
Fig. 6 TMD installed on the roof floor

considered as the most accurate method of analysis which provides accurate models. To ensure the range of possible responses is accurately captured, properly scaled earthquake records must be used for dynamic analysis. In this study, nonlinear dynamic time history analyses were conducted under a set of far-field ground motions records. This set which includes 44 ground motion records, contained 22 horizontal earthquake motions along with x and y direction which were selected from the FEMA P695 (2009). Periods of vibrations and their corresponding frequencies of the first five modes of structure, are presented in Table 3.

The details of this ground motion records set, can be seen in FEMA P695 (2009) (FEMA 2009). The summary of some sets specifications is shown in Table 4. Fig. 5(a) and 5(b) shows the 20-story building, equipped with a TMD on the roof floor in the deformed and undeformed state with applied earthquake records in perpendicular directions, and elevation of soil-structure interaction model, respectively.




(a)

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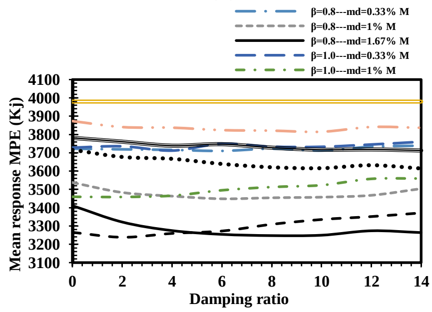


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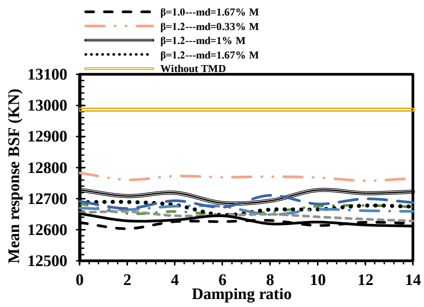


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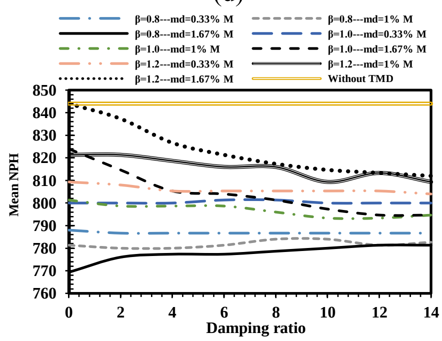


(e)

Fig. 7 Mean of spectral responses (a) displacement, (b) inter-story drift, (c) plastic energy dissipation, (d) base shear force, and (e) number of plastic hinges, of structure under earthquake excitations

Table 5 The range variety of TMD parameters

| TMD parameters | Considered range variety |
|---|---|
| $m_d$ | (0.33%, 1% and 1.67%) |
| $\xi_d$ | (0%, 2%, 4%, 6%, 8%, 10%,12% and 14%) |
| $\beta$ factor | (0.8, 1 and 1.2) |

Table 6 Optimal parameters of TMD for each function under three studied earthquakes and the mean of the results

| Objective function | Cape Mendocino 1992 | | | Superstition Hills Poe Road 1987 | | | Manjil 1990 | | | Mean | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $\beta$ | $m_d$(%) | $\xi_d$(%) | $\beta$ | $m_d$(%) | $\xi_d$(%) | $\beta$ | $m_d$(%) | $\xi_d$(%) | $\beta$ | $m_d$(%) | $\xi_d$(%) |
| MD | 0.8 | 1.67 | 14 | 1 | 1.67 | 0 | 0.8 | 0.33 | 6 | 0.8 | 1.67 | 0 |
| PED | 0.8 | 1.67 | 2 | 1 | 1.67 | 0 | 1 | 1.67 | 2 | 1 | 1.67 | 2 |
| ISD | 0.8 | 1.67 | 14 | 1 | 1.67 | 0 | 0.8 | 1.67 | 10 | 1 | 1.67 | 0 |
| BSF | 0.8 | 1 | 0 | 1 | 1.67 | 0 | 1.2 | 1.67 | 6 | 1 | 1.67 | 2 |
| NPH | 1 | 1.67 | 2 | 0.8 | 1 | 4 | 0.8 | 1.67 | 0 | 0.8 | 1.67 | 0 |

## 4. Tuning of TMD based on response spectrum

Given that no study has been done on the optimum parameters of TMD for a three-dimensional high rise concrete structure with SSI effect, torsion effect, and nonlinear behavior of concrete, in this section, the optimum parameters of TMD are obtained. A TMD was installed on the roof of the structure as illustrated in Fig. 6.

The TMD has three parameters. The first parameter is the ratio of the TMD mass ($m_d$) to the total mass of the structure (M), called the mass ratio. The second parameter is the damping ratio of TMD ($\xi_d$), and the third is the $\beta$ factor, called the frequency ratio ($K_{TMD} = m_d \times \omega^2 \times \beta$) ($\omega$ is the fundamental frequency of vibration). Tuned mass spectra provides the best method for showing the influence and efficiency of TMDs on the structural dynamic responses due to a specific earthquake (Wong and Johnson 2009, Shooshtari and Mortezaie 2017). For this purpose, 216 times nonlinear time history analysis of the 20-story structure with SSI effect was performed under three earthquake records including: Cape Mendocino 1992, Superstition Hills Poe Road 1987 and Manjil Abbar 1990. The range of TMD parameters includes $m_d$, $\xi_d$ and $\beta$ as shown in Table 5.

Analyses were done and five different spectra responses (as objective functions) including: maximum of displacement (MD), inter-story drift (ISD), plastic energy dissipation (PED), base shear force (BSF), and a number of plastic hinges (NPH) were obtained under mentioned three earthquake records. The mean of these results has been calculated and corresponding five spectra responses are presented in Fig. 6.

Optimal parameters of TMD for each function of the five objective functions, under three studied earthquakes are presented in Table 6. In the last column of Table 6, the values for optimal parameters of TMD, for the mean of the results of three earthquakes are presented. It is evident that due to high dispersion of the obtained values for optimal parameters of TMD, capturing the best responses in all of the five studied objective functions with a fixed value for md, $\xi_d$ and $\beta$ factor from a practical point of view, is impossible. However, by looking closely at the results, it is obvious that importance of md and $\beta$ factor is higher than $\xi_d$.

In most previous studies, setting the TMD according to the natural frequency of structure has been raised as a principle, but the current study results reveal that this principle by entering the structure into the non-linear field is no longer true. Considering the results and the importance of each of the functions, the optimal values of TMD, can be reported as md=0.0167×M, $\xi$d=3%, and $\beta$=0.8.

## 5. Analysis results

Three-dimensional numerical analyses were conducted on a 20-story model of a building, under 22 earthquake acceleration records. The structure is equipped with a TMD tuned with optimized parameters as described in the previous section. Analyses were performed in different states of PBPD structure: with and without TMD, by considering the effect of SSI and without it. Due to the large volume of information obtained from the analyses of 22 earthquake records and in order to indicate the dispersion and variation of the results, the mean and standard deviation (SD) for obtained results were calculated. SD predicts the variation and dispersion of the data obtain in comparison to the mean. On the other hand, to create a better understanding, results were classified into two categories: response of each story (Fig. 7) and the maximum response of the entire structure (Fig. 8).

From Fig. 7(a) and 7(c), it is evident that by considering the effect of SSI and without TMD, lateral displacement and inter-story drift are increased. On the other hand, plastic energy, and base shear are reduced (Fig. 7(d) and 7(f)), while Fig. 7(b) and 7(e) shows that the acceleration and NPH do not change significantly due to the effect of SSI. However, when the structure was equipped with an optimized TMD, it leads to an improvement in structural response for all studied parameters. This improved performance is such that, even when compared with a case in which the effect of SSI has been neglected, it leads to a better response except lateral displacement.

Reviewing the local structural response, particularly in Fig. 8(c), Fig. 8(m) and Fig. 8(n) that are important in review of local damage, it becomes evident that by considering the effect of SSI, the mean maximum plastic energy that accumulated under 22 earthquake acceleration in a plastic hinge in the structure was reduced and by equipping the structure with TMD, it can be further reduced.

Fig. 8(n) shows that the mean maximum rotation in a hinge does not change due to the effect of SSI but by using the TMD, there is still 8.6% reduction. In Fig. 8(c), the appropriate influence of TMD is obvious. Therefore, when compared to the case where the effects of soil and TMD are neglected, there is 12.7% decline in inter-story drift. Reduction in acceleration in the fifteen upper floors (Fig. 7(b)) led to reduced non-structural damage that is very useful and vital for reducing losses due to earthquake.

Using TMD leads to the reduction of story shear force

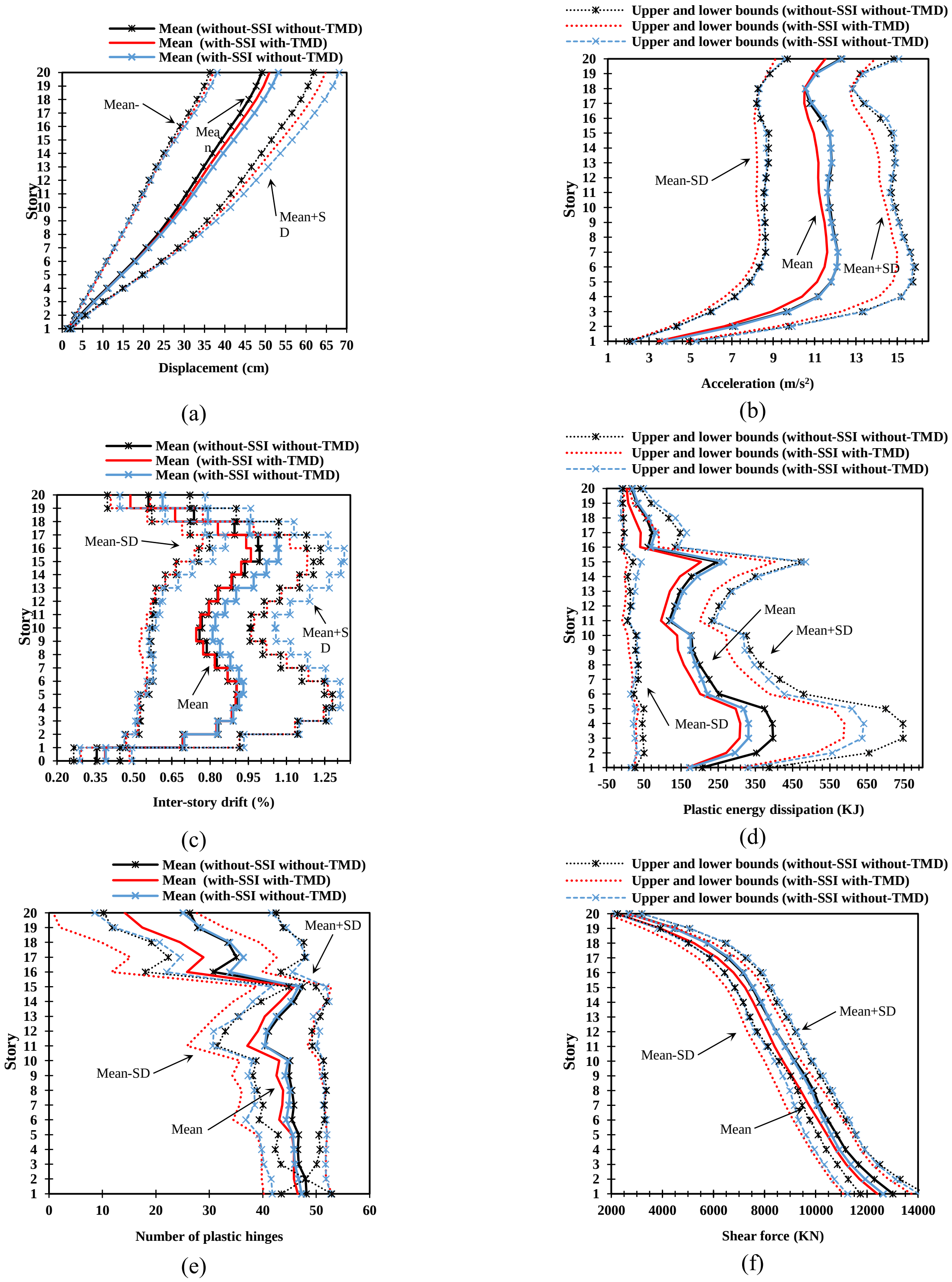


Fig. 8 Mean and one standard deviation of the mean ($\mu$ ± SD, where $\mu$ is the mean) of (a) displacement, (b) acceleration, (c) inter-story drift, (d) plastic energy dissipation, (e) number of plastic hinges, and (f) shear force of structure under earthquake excitations

and destructive forces in lower stories, but this decline is not adequate to overcome the over-size problem of the columns in high-rise buildings. It is evident from the results that by using the TMD, bad performance of the structure can be improved in most assessed parameters due to the effect of SSI. Details of decrease or increase in mean maximum responses for the studied parameters are presented in Table 7.

One of the objectives of inelastic seismic analysis is to directly compute the magnitude of inelastic deformations in order to assess the level of structural performance. Performance levels represent the state of structures after being subjected to a certain hazard level and they are

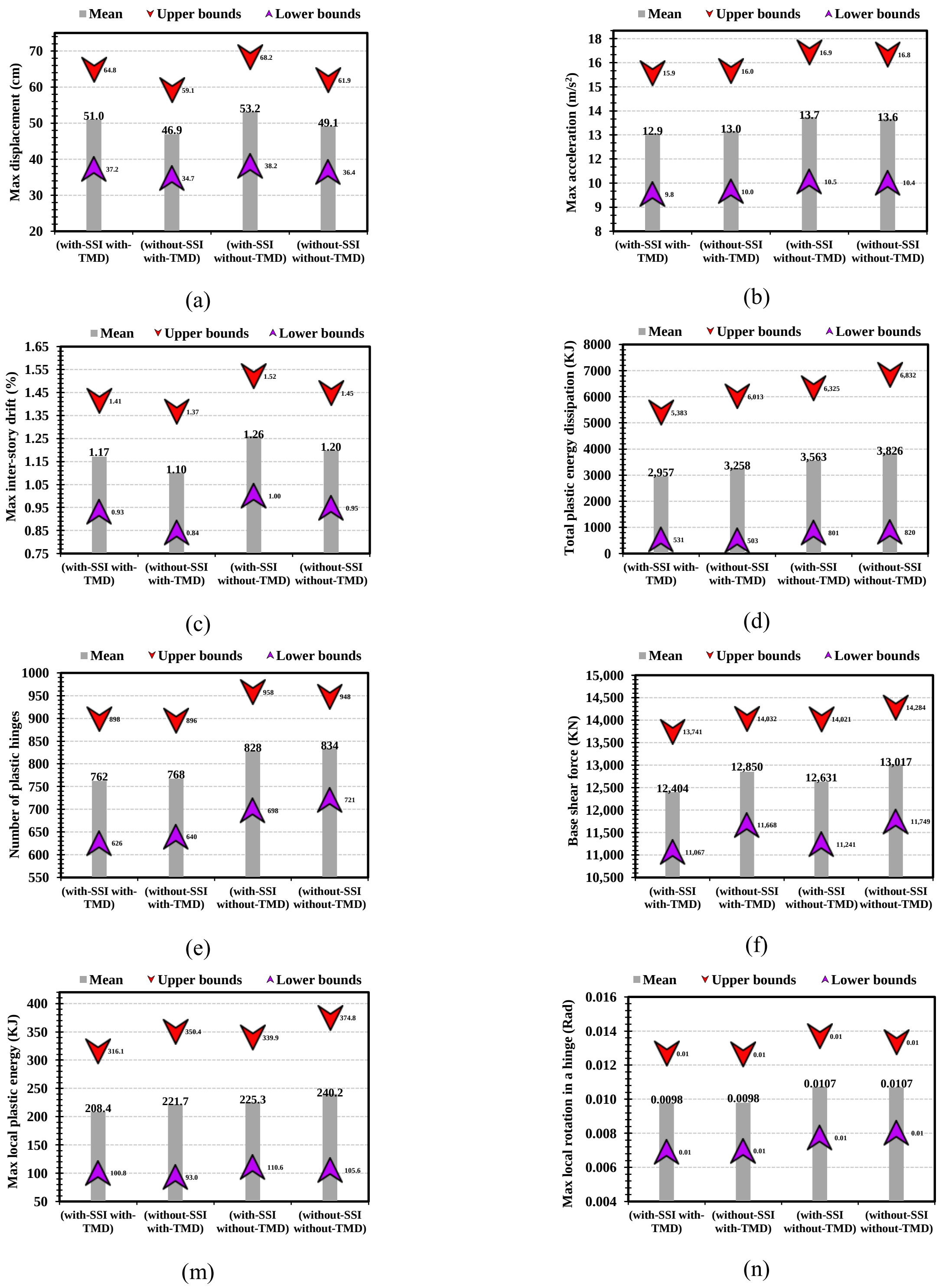


Fig. 9 Mean maximum responses and one SD of the mean of (a) displacement, (b) acceleration, (c) inter-story drift, (d) total plastic energy dissipation, (e) number of plastic hinges, (f) base shear force of structure, (m) local plastic energy and (n) local rotation in a plastic hinge in structure

classified into five categories: fully operational, operational, life safe, near collapse, or collapsed (FEMA 1997). The most widely utilized damage parameters are, total lateral deflection, ductility demand, and inter-story drifts. The above mentioned five qualitative performance levels are associated with the corresponding quantitative maximum inter-story drifts of <0.2, <0.5, <1.5, <2.5, and >2.5%, respectively.

Majority of the force-based design codes use an additional test in terms of limiting the inter-story drifts to ensure that, particular deformation-based criteria are met. For instance, ASCE7-10 (2010) allowed story drift for structures regarding their type and risk category of the structure and the Australian Earthquake Code indicates the

Table 7 Compare the maximum response of the structure

| Parameter | without SSI and without TMD (1) | with SSI and without TMD (2) | TMD effect | | SSI effect | |
|---|---|---|---|---|---|---|
| | | | with SSI & with TMD (3) | Decrease percentage (3) to (2) | Increase percentage (2) to (1) | Decrease percentage (2) to (1) |
| displacement (cm) | 49.1 | 53.2 | 51.0 | 4.3 | 8.4 | - |
| Acceleration (m/s2) | 13.6 | 13.7 | 12.9 | 5.8 | 0.7 | - |
| Inter-story drift (%) | 1.20 | 1.26 | 1.17 | 7.1 | 5.0 | - |
| TPED (KJ) | 3,826 | 3,563 | 2,957 | 17.0 | - | 6.9 |
| NPH | 834 | 828 | 762 | 7.97 | - | 0.7 |
| Shear force (KN) | 13017 | 12,631 | 12,404 | 1.8 | - | 3.0 |
| Plastic energy (KJ) (in a hinge) | 240 | 225 | 208 | 7.6 | - | 6.3 |
| Rotation (Radians) (in a hinge) | 0.0107 | 0.0107 | 0.0098 | 8.6 | - | - |

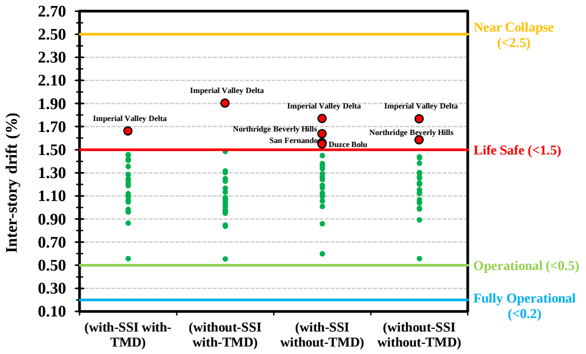


Fig. 10 Cases exceed the Life Safe limit

Table 8 Evaluation structural risk in four state analyses of structure

| SSI | TMD | Number of cases exceed the life safe limit | Percentage (%) |
|---|---|---|---|
| ✓ | ✓ | 1 of 22 | 4.5 |
| × | ✓ | 1 of 22 | 4.5 |
| ✓ | × | 4 of 22 | 18.2 |
| × | × | 2 of 22 | 9.1 |

maximum allowable story drift of 1.5% (AS1170.4 2007, ASCE 2010). The analyses of the structure in four state and underground motion records were performed, and the records that lead to crossing inter-story drift from the life safe limit has been determined, and the name and the number of earthquakes (among 22 earthquakes) is presented in Fig. 9. In one case analysis where the effect of SSI and TMD are considered, only 1 of 22 (4.5%) records exceeds the life safe limit, while this number reaches to 4 of 22 (18.2%) with SSI effect and without TMD. This rate in other cases that has not considered the effect of SSI is specified in Table 8.

Reduced range of inter-story drift variation is clearly defined in Fig. 9. An optimized TMD is suggested so as to prevent the adverse effects of soil type D for which the building is designed. Design engineers need to precisely consider the effects of dynamic SSI in their design. Although in the current design approach, the effect of PBPD and soil-structure interaction was not considered in the design process, however, when compared with previous studies conducted on force-based design of structures and the results reported, PBPD is recommended for structural design.

All previous studies indicated that by entering the structure into the field of nonlinear behavior and increasing structure height, TMD does not affect the linear behavior of structure but the current study optimized parameters for TMD show a good performance for the structure. This performance can be attributed to three reasons: first, the nonlinear behavior of the structure has been considered from the beginning of the design process. Second, the large number of tuning analyses conducted on high-rise concrete structures and lastly, the same level of soil class that was considered at the design stage has been affected in the analyses. To summarize in one sentence, it should be noted that the actual behavior of the structure is considered.

## 6. Conclusions

In this study, the effects of SSI on a 20-story concrete structure designed by PBPD method were investigated. Nonlinear time history analyses were carried out on a three-dimensional 20-story model of concrete structure for obtaining optimal parameters of TMD. Then by arming the structural system with an optimized TMD, an attempt has been made to study the adverse effects of SSI effects like increase in inter-story drift and displacement reduction. In contrast to previous studies, by conducting a comprehensive and detailed study with the main parameters affecting the structure performance, it is clear from the obtained results that TMD could well lead to acceptable results for the main studied parameters.

It was shown that optimized parameters of TMD are very sensitive to both seismic vibration characteristics and soil conditions, therefore it is suggested that the use of active mass dampers (AMD) can be more appropriate for considering the uncertainties in earthquake and soil vibration characteristics. It is also suggested that the structural designer should address the effects of SSI precisely in PBPD design procedure.

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