

## EVALUATION OF SITE EFFECT ON TMD'S PERFORMANCE FOR SEISMIC RESPONSE REDUCTION OF TEHRAN TOWER

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### ABSTRACT

A Tuned Mass Damper (TMD) is a passive energy dissipation device connected to the structure in order to reduce the dynamic vibrations. Evaluation of site effect is also one of the most important problems considered in geotechnical earthquake engineering field which plays an important role in earthquake resistant design. In most of the cases, the effect of soil properties on the performance of control systems is usually neglected. In practice, however, many structures are built on soft soils and strong interaction appears between soil and the structure. Without considering the soil, a control system might be installed when it is considered unnecessary due to overestimation of the responses of uncontrolled structures or a control system may not perform optimally due to erroneous identification of structural properties. The purpose of this study is to evaluate the site effect on TMD's performance for response control of Tehran Tower under seismic excitations. Time history analyses are performed to calculate the response of the structure subjected to a number of earthquake records without considering the soil layers. The same procedure is followed for the model fitted with TMD. The mentioned records are applied to the soil layers and obtained ones from ground surface are used for time history analyses. The results are reported and comparisons are made in terms of defined performance indices to demonstrate the effects of soil layers on the structural response reduction of Tehran Tower. In addition, a parametric study is performed to investigate the effect of soil stiffness on the TMD's performance.

Keywords: Tehran Tower, Site Effect, Tuned Mass Damper, Response Reduction, Passive Control

### INTRODUCTION

A Tuned Mass Damper (TMD) is a passive energy dissipation device, consists of a mass, spring, and a damper, connected to the structure in order to reduce the dynamic vibrations induced by wind or earthquake loads. The frequency of TMD is tuned to one of the dominant frequencies of the structure usually the first natural frequency. The reduction of vibrations is accomplished by transferring some of the structural vibrational energy to the TMD and dissipating the energy at the damper of the TMD (Soong and Dargush, 1997).

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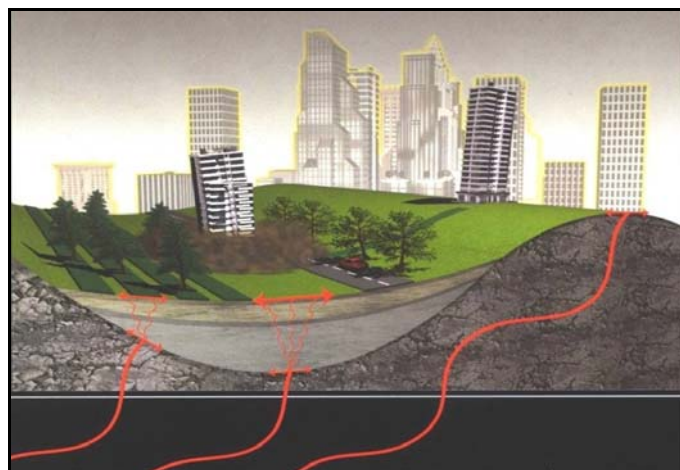
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The concept of TMD was first suggested by Frahm in 1909 (Frahm, 1909) and was used in order to reduce the vertical vibration of ships (Housner et al., 1997). The detailed theory and working principles of undamped and damped tuned mass damper to control the displacement of an undamped single degree-of-freedom system subjected to a harmonic force has been described by Den Hartog in his monograph (1956). He developed simple formulas for proper selection of optimum parameters of TMD. McNamara (1977) demonstrated that TMDs are effective in reducing wind-induced response of structures. Warburton (1982) derived closed-form expressions for optimum damper parameters for undamped single degree-of-freedom main system for both harmonic and white noise random excitations with force and base accelerations as input and minimization of various response parameters. Using a numerical searching procedure, Tsai and Lin (1993) obtained the optimum parameters of TMDs for steady-state response reduction of damped systems subjected to support excitations. Fujino and Abe (1993) summarized a set of design formulas for TMDs based on a perturbation technique under various type of loading conditions. Sadek et al. (1997) determined the optimum parameters of TMDs which result in seismic response reduction of structures.

A number of TMDs have been installed in tall buildings, bridges, towers, and smoke stacks for response control against lateral loads. TMD has been successfully used in structures to improve the wind-induced vibrations, such as CN Tower in Canada, John Hancock Building in Boston, Center-Point Tower in Sydney, and the Taipei 101 Tower in Taiwan, the tallest building in the world; but it can be generally said that under earthquake-type loading, TMDs are not as effective as for wind-type loading (Soong and Dargush, 1997).

Moreover, evaluation of site effect is one of the most important problems considered in geotechnical earthquake engineering field. Macmurdo noted that buildings situated on rock were not by any means so much affected as those whose foundations did not reach to the bottom of the soil in the 1819 earthquake in cutch, India (Macmurdo, 1824). In the report on the 1857 neapolitan earthquake, Mallet mentioned the effect of local geology conditions on damages (Mallet, 1862). Wood and Reid showed that the intensity of ground shaking in the 1906 San Francisco earthquake was related to soil and geologic conditions (Wood, 1908; Reid, 1910).

Since these early observations, the effects of site conditions on ground motions have been illustrated in earthquakes around the world. Also, the availability of strong-motion instruments has allowed the site effects to be measured quantitatively in recent years. Site effect plays an important role in earthquake resistant design and must be accounted in each case. Although the local soil effect was considerably evidenced, provisions specifically accounting for site effects did not appear in building codes until the 1970s. Local site conditions can influence the important characteristics such as amplitude, frequency content and duration of the motions. The extent of the influence of the soil depends on the geometry and material properties of the soil layers, site topography, and the characteristics of the input motion. The effect of the local soil and site geometry can be seen in Figure 1.



**Figure 1. Effect of the local soil on earthquakes**

Two of the most important earthquakes in which the local site effect can be clearly seen, were the 1985 Michoacan (Mexico) earthquake (Stone et al., 1987) and the 1989 Loma Prieta (California) earthquake (Seed et al., 1990). These earthquakes produced strong motions records at sites underlain by a variety of different subsurface conditions in Mexico City and San Francisco Bay area. Case histories of ground response in Mexico City, the San Francisco Bay area, and many other locations have clearly shown that local site conditions strongly influence peak acceleration amplitudes and the amplitudes and shapes of response spectra.

Although significant progress has been made in the field of structural control during the past two decades (Housner et al, 1997; Soong and Spencer, 2002; Spencer and Nagarajaiah, 2003), the effect of soil properties on the performance of control systems is usually neglected. In most of the research works, structures are assumed to be located on the rock outcrop. In practice, however, many structures are built on soft soil and strong interaction appears between soil and the superstructure. Without taking this factor into account, a control system might be installed when it is considered unnecessary due to overestimation of the responses of uncontrolled structures or a control system may not perform optimally due to erroneous identification of structural properties. Wu et al. (1999) studied the effect of soil-structure interaction on TMD's performance for seismic applications. They noticed that the damper's effectiveness rapidly decreases as the soil medium gets softer due to the significant contribution to the damping of the soil-structure system from soil material hysteresis and radiation effect.

Previously, Okhovat et al. (2006) investigated the effectiveness of TMD on seismic response reduction of Tehran Tower. The purpose of this study is to evaluate the site effect on TMD's performance for response control of Tehran Tower under seismic excitations. A lumped mass model of the building is provided with 112 translational and 56 rotational degrees of freedom using solid and shell elements. Time history analyses are performed to calculate the response of the structure subjected to a number of earthquake records without considering the soil layers. The same procedure is followed for the model fitted with TMD. The mentioned records are applied to the soil layers and obtained ones from ground surface are used for time history analyses. The results are reported and comparisons are made in terms of defined performance indices to demonstrate the effects of soil layers on the structural response reduction of Tehran Tower. In addition, a parametric study is performed to investigate the effect of soil stiffness on the TMD's performance.

## **TEHRAN TOWER STRUCTURAL PROPERTIES**

Tehran residential tower with 571 apartment units in 56 stories will be the tallest residential tower in Iran with 170 m height and more than 200,000 m<sup>2</sup> construction area (Figure 2(a)). Being still under construction, it has three main wings as shown in Figure 2(b). With approximate dimension of 48 × 22 m, these 3 wings coincide in one central point with no construction or expansion joint according to its shop drawings. Lateral strength of this structure is provided by three main shear walls which are located in an angle of 120 degrees to each other in the centerline of the wings. Gravitational forces are tolerated by concrete slabs and secondary shear walls. Ranging from 0.7 m to 2.0 m, the thickness of the main shear wall changes in the height of the building and the thickness range of the secondary shear walls is between 25 cm to 50 cm. The main shear walls have four series of openings which are located in every other story. All the secondary walls, also, have openings next to the main shear wall (Ghalibafian et al., 2005).



(a)



(b)

**Figure 2. Tehran Tower; (a) Elevation, (b) Perspective (During construction)**

### MODELING ASSUMPTIONS

Because of complexity of the structure, one should idealize the model in order to have less time-consuming process. However, the main goal of present research, which is the global behavior of the structure, is not affected by these modifications. The assumptions which are used in each structural element are as follows:

**Main Shear Walls:** The thickness of these shear walls is very close to the other two dimensions so we have used eight-node Solid elements to model them. It is based upon an isoparametric formulation that includes nine optional incompatible bending modes. The incompatible bending modes significantly improve the bending behavior of the element if the element geometry is of a rectangular form. There are about 7045 Solid elements in the model.

**Secondary Shear Walls:** Four-node Shell elements are used to model these shear walls because their thickness is not very high. For each Shell element in the structure, one can choose to model pure membrane, pure plate, or full shell behavior. The membrane behavior uses an isoparametric formulation that includes translational in-plane stiffness components and a rotational stiffness component in the direction normal to the plane of the element. The plate bending behavior includes two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element. There are about 11430 Shell elements in the model.

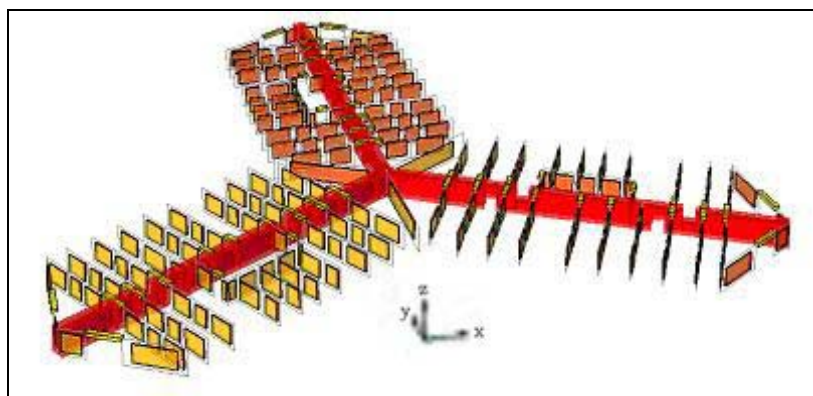
**Coupling Beams:** Coupling beams are those beams which are located at the top of the openings. Although the ratio of the length to the depth of these beams is usually less than 2, Frame elements have been used for modeling. In fact, this idealization does not affect the global behavior of the structure. There are about 6050 Frame elements in the model.

**Floor Slabs:** Since floor slabs are just used to provide the rigidity of the story elevations and to determine the story masses for modal analysis, they are not modeled. Accordingly, dead and live loads and their distribution on the floor are modeled by a translational concentrated mass and a rotational concentrated mass in the center of area in each floor. A rigid Diaphragm is used to constrain all the translational and rotational degrees of freedom of each story nodes to this master joint.

The total mass of the structure used in modal analysis is about 302,400 tons. Another role of floor slabs is to transfer the gravitational loads to secondary shear walls that does not affect the goal of this research. According to these assumptions, a full scale three dimensional model have been made in SAP2000 program using a repeated typical module which is demonstrated in Figure 3 for each story.

**Tuned Mass Damper:** In order to model a TMD, a Viscous Damper has been used. It is a nonlinear link and for each deformational degree of freedom, independent damping properties may be specified.

If nonlinear properties are not specified for a degree of freedom, the degree of freedom will be linear using the effective stiffness, which may be zero.



**Figure 3. Typical repeated module for each story**

### ANALYSIS APPROACH

In order to have an exact modeling of the structure, linear time history analysis is used. Three earthquake records have been applied to the structure along the x-direction. All the earthquake time histories have been recorded on the rock outcrop and are scaled to 0.35g. More details about these records can be found in Table 1.

**Table 1. Properties of the earthquakes used**

Name of Record	Maximum Acceleration (g)	Magnitude ( $M_b$ )	Duration (sec)
El Centro (1940)	0.35	7.1	31.06
Manjil (1991)	0.35	6.4	58.16
Northridge (1994)	0.35	6.8	50.00

In modal analysis, 10 modes are considered so that the modal mass participation ratio would at least reach to 90 percent of the total mass of the building. Table 2 shows the results of modal analysis of the building such as period of the modes and the modal participating mass ratios.

**Table 2. Modal analysis results**

No. of Mode	Period (sec)	Cumulative Mass Ratio for $U_x$	Cumulative Mass Ratio for $U_y$	Cumulative Mass Ratio for $U_z$	Cumulative Mass Ratio for $R_x$	Cumulative Mass Ratio for $R_y$	Cumulative Mass Ratio for $R_z$
1	3.465	0.00	0.00	0.00	0.00	0.00	0.64
2	1.397	0.00	0.70	0.00	0.99	0.00	0.64
3	1.397	0.70	0.70	0.00	0.99	0.99	0.64
4	0.677	0.70	0.70	0.00	0.99	0.99	0.83
5	0.387	0.70	0.87	0.00	1.00	0.99	0.83
6	0.387	0.87	0.87	0.00	1.00	1.00	0.83
7	0.279	0.87	0.87	0.00	1.00	1.00	0.90
8	0.192	0.87	0.92	0.00	1.00	1.00	0.90
9	0.192	0.92	0.92	0.00	1.00	1.00	0.90
10	0.163	0.92	0.92	0.00	1.00	1.00	0.93

As it can be seen in Table 2, the first mode of the structure is torsional mode and does not participate in structural translation. The second and third modes which are translation along  $y$  and  $x$  directions, respectively, have the same period due to the symmetric geometry of the building.

## EVALUATION CRITERIA

In order to evaluate the performance of the control method, both peak- and normed-based evaluation criteria are used. Small values of the evaluation criteria are generally more desirable. The first criterion is based on peak building interstory drift ratios (Othori et al., 2004),

$$J_1 = \max \left\{ \frac{\max_{t,i} \frac{|d_i(t)|}{h_i}}{\delta^{\max}} \right\} \quad (1)$$

over the range  $i = [1, 56]$  for this building, where  $d_i(t)$  = interstory drift over the time history of each earthquake;  $h_i$  = height of each of the associated stories;  $\delta^{\max}$  = maximum interstory drift ratio of the uncontrolled structure calculated by equation of  $\max_{t,i} |d_i(t)/h_i|$ . Second criterion is based on normed interstory drift and defined as

$$J_2 = \max \left\{ \frac{\max_i \frac{\|d_i(t)\|}{h_i}}{\|\delta^{\max}\|} \right\} \quad (2)$$

where the norm,  $\|\cdot\|$ , is computed using the following equation

$$\|\cdot\| \equiv \sqrt{\frac{1}{t_f} \int_0^{t_f} [\cdot]^2 dt} \quad (3)$$

that  $t_f$  is equal to sufficiently large time to allow the response of the structure to attenuate. In this study, the duration of 100 sec is adopted for those mentioned earthquakes. Accordingly, the criteria which are related to acceleration response of the building are defined as

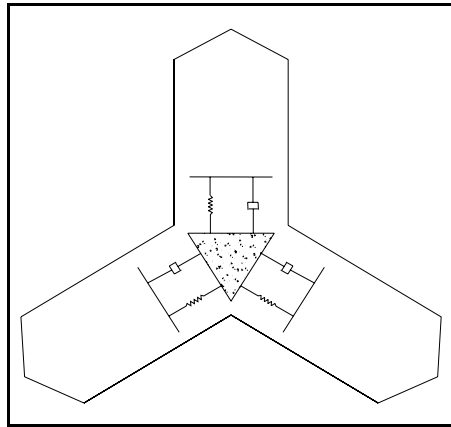
$$J_3 = \max \left\{ \frac{\max_{t,i} |\ddot{x}_{ai}(t)|}{|\ddot{x}_a^{\max}|} \right\} \quad (4)$$

$$J_4 = \max \left\{ \frac{\max_i \|\ddot{x}_{ai}(t)\|}{\|\ddot{x}_a^{\max}\|} \right\} \quad (5)$$

where  $\ddot{x}_{ai}(t)$  and  $\ddot{x}_a^{\max}$  are absolute accelerations of the  $i^{\text{th}}$  level with and without control devices respectively.

## TMD PROPERTIES

Since the second and third modes have the main role in the structural translation of the building, the TMD used is tuned to these modes. As it is shown in Figure 4, the location of the TMD mass which could be a 400 tons concrete block, is assumed in the roof elevation at the central point of the building where three wings coincide. Accordingly, the mass ratio of the TMD ( $\mu$ ) is about 0.1 percent of the total mass of the building and the stiffness of its springs is calculated by tuning the TMD to the second and third natural frequencies. Three springs and dashpots which are considered to be located along the wings direction, are attached to this mass. Since these three springs have the same stiffness of  $k$  and are located in an angle of 120 degrees to each other, it could be proven that their stiffness in every direction equals to  $3k/2$ . This point is also applicable to the damping of the dashpots (Ankireddi and Yang, 2000).



**Figure 4. The schematic location of the assumed TMD on the roof of the building**

## SOIL MODELING

When a fault ruptures below the earth's surface, body waves travel away from the source in all directions. By the time the rays reach the ground surface, multiple refractions have often bent them to a nearly vertical direction. One dimensional dynamic analysis is based on the assumption that all boundaries are horizontal and that the response of a soil deposit is predominantly caused by SH-waves propagating vertically from the underlying bedrock. For one-dimensional ground response analysis, the soil and bedrock surface are assumed to extend infinitely in the horizontal direction. Procedures based on this assumption have been shown to predict ground response that is in reasonable agreement with measured response in many cases (Kramer, 1996).

For one dimensional analysis, SHAKE software is employed. In one dimensional analysis, the nonlinear behavior of soils can be considered by equivalent linear method. The equivalent linear method has been used for many years to calculate the wave propagation and response of the acceleration in soil and rock layers at the sites subjected to seismic excitation. This method does not capture directly any nonlinear effects because it assumes linearity during the solution process. In this method, strain dependent modulus and damping curves are only taken into account in an average sense, in order to approximate some effects of nonlinearity. The equivalent linear shear modulus is generally taken as a secant shear modulus and the equivalent linear damping ratio as the damping ratio that produces the same energy loss in a single cycle as the actual hysteresis loop. Since the linear approach requires that  $G$  and  $\xi$  be constant for each soil layer, in the problem of determining the values that are consistent with the level of strain induced in each layer is required. To solve this problem, an objective definition of strain level is needed. The modulus reduction and damping ratio curves, obtained from laboratory tests, and simple harmonic loading are used to characterize the strain level by the peak shear strain amplitude.



Before beginning the construction of the tower, geotechnical investigations in the area have been performed including seven boreholes and six test pits. From these investigations the unified classification,  $N$  values of SPT, plasticity indexes, unit weights of the soil layers and water content of them were determined. Since the soil layers in different boreholes were approximately the same, one of the boreholes is chosen for dynamic analyses. The properties of the soil layers of this borehole can be seen in Figure 5.

A parametric study on the soil stiffness is also performed to find the TMD's performance for different types of the soils. In this research, four types of soil with different shear wave velocities from different types of soil profiles in UBC97 code are chosen. The height of the soil layers is 30 meters. Soil characteristics of these cases for parametric study can be seen in Figure 6. In this research, the model presented by Vucetic and Dobry (1987) is used for dynamic analyses of the soil profiles.

The maximum accelerations are considered as representatives of ground motion characteristics. For the bedrock of the local soil profile, the shear wave velocity of 700 m/s and unit weight of  $24 \text{ kN/m}^3$  are assumed. Also shear wave velocity of 1000 m/s and unit weight of  $24 \text{ kN/m}^3$  are considered as the bedrock characteristics in parametric study.

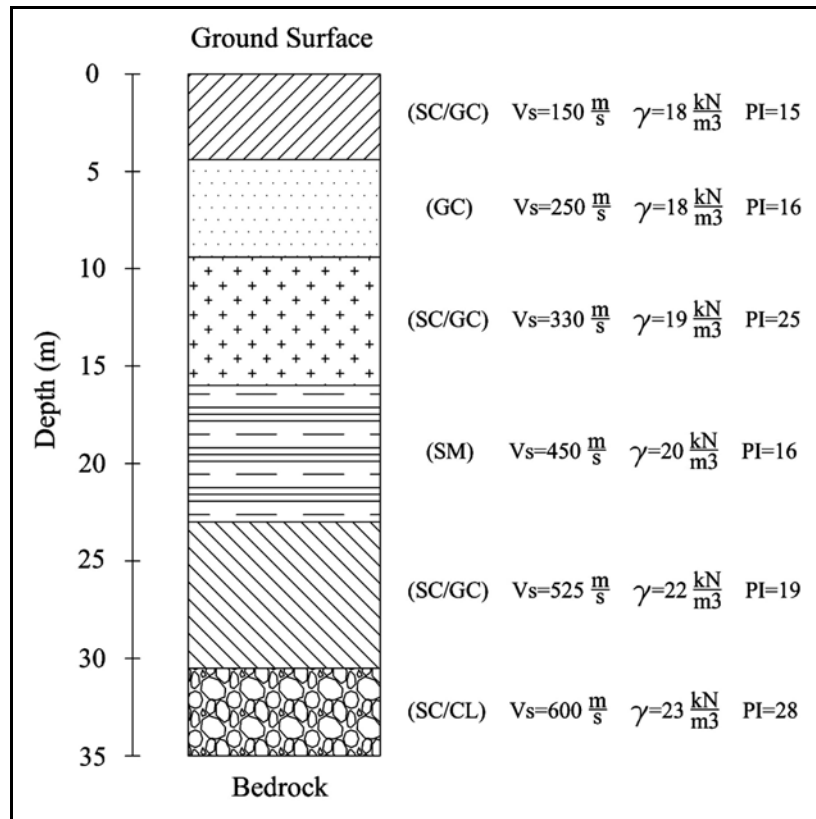


Figure 5. Geotechnical soil profile and corresponding values of the parameters

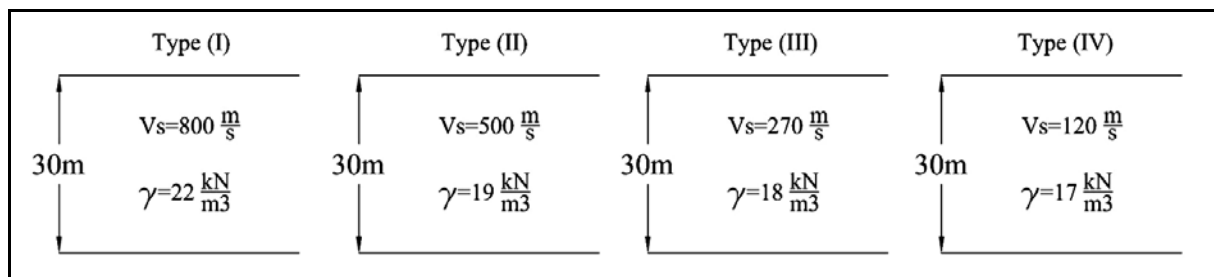


Figure 6. Characteristics of the soil types for parametric study



## NUMERICAL RESULTS

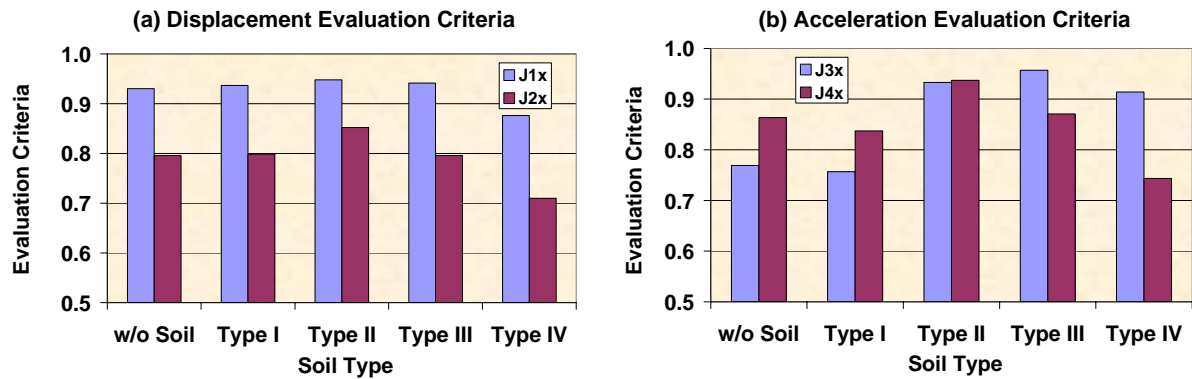
Numerical analyses are performed and the results of the evaluation criteria for the  $x$ -direction, obtained by undertaking three mentioned earthquake records as the external excitations and the soil layers, are shown in Table 4. As it is shown in this Table, TMD is more efficient in structural response improvement when the soil is not included. This indicates the importance of considering the soil layers. Table 5 shows the maximum surface accelerations for different types of soil. Moreover, Figures 7, 8, and 9 demonstrate the results for the parametric study on the types of the soil. It could be understood that the efficiency of the TMD generally conforms to the maximum surface accelerations which are provided by each soil type.

**Table 4. Evaluation criteria with and without considering the soil layers**

Earthquake	Without Considering Soil Layers				Considering Soil Layers			
	$J_{1x}$	$J_{2x}$	$J_{3x}$	$J_{4x}$	$J_{1x}$	$J_{2x}$	$J_{3x}$	$J_{4x}$
El Centro	0.93	0.79	0.77	0.86	0.95	0.89	0.84	0.91
Manjil	0.90	0.74	0.88	0.90	0.94	0.87	0.89	0.95
Northridge	0.79	0.74	0.62	0.67	0.82	0.83	0.64	0.72

**Table 5. Maximum surface acceleration of different soil types**

Earthquake	Maximum Acceleration (g)			
	Type I	Type II	Type III	Type IV
El Centro	0.366	0.558	0.410	0.314
Manjil	0.415	0.482	0.313	0.264
Northridge	0.383	0.486	0.270	0.284



**Figure 7. Effect of soil stiffness on structural responses for El Centro earthquake**

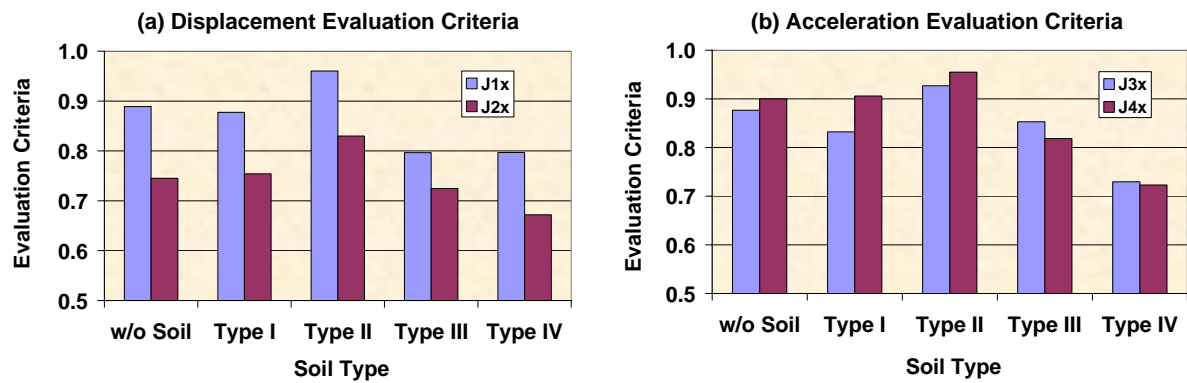


Figure 8. Effect of soil stiffness on structural responses for Manjil earthquake

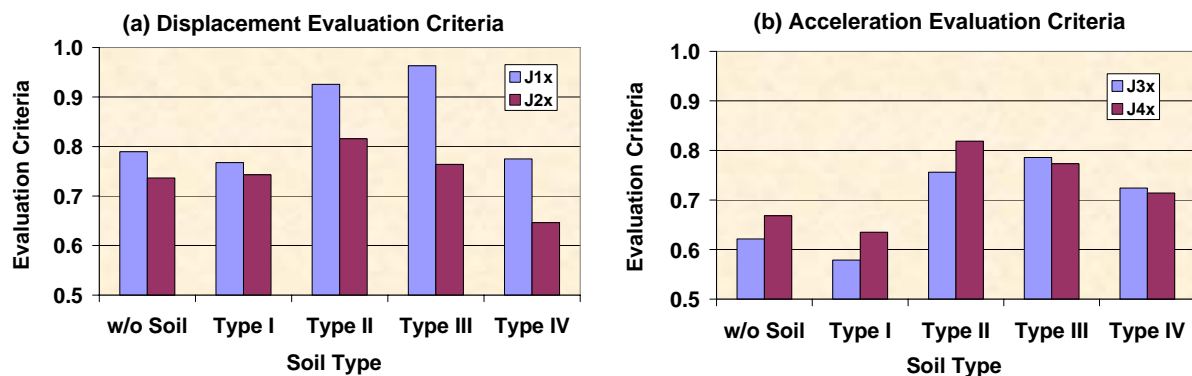


Figure 9. Effect of soil stiffness on structural responses for Northridge earthquake

## CONCLUSIONS

In this research the effect of the soil on TMD's performance for response control of Tehran Tower under seismic excitations was investigated. The results show that considering the soil layers in designing the TMDs for Tehran Tower is of great importance. Without taking site soil into account, the designed TMD may not reduce the structural response as it is anticipated. Furthermore, a parametric study was performed to show the effect of soil stiffness on TMD's performance. It could be inferred that the efficiency of the TMD generally conforms to the maximum surface accelerations which are provided by each soil type. Further investigations should be performed on Tehran Tower considering soil-structure interaction.

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