Dynamic Time History Analysis of Plane Frame with Tuned Mass Damper by Considering Soil-Structure Interaction

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Paper Reference Number: ٥١٢-١٩٠
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Abstract

One of the effective way that caused to decrease the response of structures is using of tuned mass damper. The dynamic behavior of these dampers is determined by natural frequency of structures and it is an efficient parameter for the response of structure. So the accurate determination of this frequency is very important. In this paper the effect of soil- structure interaction with cones has been used in dynamic equation of structure with tuned mass damper. This paper presents that the response of structure with modeling of tow frames with different dynamic characteristic and tuned mass damper has been compared by tow method with and without considering of soil-structure interaction. Also response of the frames has been investigated with north-south component of horizontal ground acceleration recorded at the Elcentro during the earthquake of May ١٨, ١٩٤٠. The comparison of results indicates that considering of soil-structure interaction in modeling has more efficiency in response of structure.

Key words: mass damper, Soil-structure interaction, Cones method, Dynamic Time History Analysis.

١. Introduction

There are many passive control systems of reducing the earthquake demand on structural system. One of the suitable passive controls for structures is tuned mass damper [١]. A Tuned Mass Damper is a device consisting of a mass, a spring, and a damper that is attached to a structure in order to reduce the dynamic response of it. The frequency of the damper is tuned to a particular structural frequency so that when the frequency is excited, the damper will resonate out of phase with the structural motion. Energy is dissipated by the damper inertia force acting on the structure. The mass of the damper transmits its inertia force to the building in a direction opposite to the motion of structure itself, thereby reducing the building's oscillations [٢]. The mass of TMD is a small fraction ٠.٢٥ to ٠.٧ of total mass of building, which corresponds to about ١ to ٢% of first modal mass [٣].

The invention of the TMD as an energy dissipative vibration absorber is credited to Frahm, who developed the concept in ١٩٠٩. The theory was later described by Emeritus and J.Den Hartog, in textbook on mechanical Vibrations in ١٩٤٠.The initial theory was applicable for an

undamped SDOF system subjected to sinusoidal force excitation. Significant contributions were made by Randall et al in ١٩٨١, Werburton in ١٩٨٢, and Tsai and Lin in ١٩٩٣ [١].
The estimation of natural periods of structures is the most important phase of the design or retrofit of a structure resisting earthquake motions. The large number of simplified assumptions required to determine natural periods of structures, one of them considering soil flexibility or assuming fixed base. The researches mentioned that natural period of structures considering soil flexibility is greater than structures assuming fixed base [٤]. For example fundamental natural period of two reinforced of the San Onofre power plant in California was computed to be ٠.١٥ sec assuming the base as fixed, and ٠.٥ sec considering soil flexibility. Difference in the period indicates the important effect of soil-structure interaction for structures [٤]. This lack of accuracy can be improved to different method.
About the effect of soil-structure interaction upon the structural response Chopra and Gutierres, ١٩٧٤; Clough and Penzien, ١٩٨٧; Jennings and Bielak, ١٩٧٣; Meek and Veletsos, ١٩٧٢; Singh, ١٩٨٠; Veletsos and Wei, ١٩٧١; Veletsos and Verbic, ١٩٧٤; Wu and Smith, ١٩٩٥. Most research can be divided into two categories. One uses the finite element method to simulate soil deformation effects on the structural response. This method is effective for complicated soil conditions. The other uses a modal combination method, by assuming that the soil is a half-space continuum, and treating the soil as a spring-damper system to resolve structural responses in the frequency domain for excitations. In such an approach, the modal combination method was adopted for linearly elastic systems. In this paper, a modified structural response in the frequency domain approach was adopted [٥].
One of the methods to Modeling of soil-structure interaction is used of springs and dampers at the base of structure, effective stiffness and damping factors that compute from simple equations. These values can be used directly in a computer model without any difficulty. In this paper are used stiffness and damping factors that are independent to frequency. Figure (Fig. ١) describe simplified model for analysis of interaction.

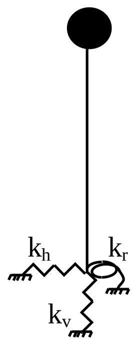

Fig ١: Simplified model for Analysis of Interaction.

The most common soil-structure interaction SSI approach, used for three dimensional soil-structure systems, is based on the "added motion" formulation. This formulation is mathematically simple, theoretically correct, and is easy to automate and use within a general linear structural analysis program. In addition, the formulation is valid for free-field motions caused by earthquake waves generated from all sources. The method requires that the free-field motions at the base of the structure be calculated prior to the soil-structure interactive analysis. In order to develop the fundamental SSI dynamic equilibrium equations consider the three dimensional soil-structure systems shown in figure (Fig. ٢) [٦].

Consider the case where the SSI model is divided into three sets of node points. The common nodes at the interface of the structure and foundation are identified with *"c"*; the other nodes within the structure are *"s"* nodes; and the other nodes within the foundation are *"f"* nodes. From the direct stiffness approach in structural analysis, the dynamic force equilibrium of the system is given in terms of the absolute displacements, **U,** by the following sub-matrix equation (Eq. ١).

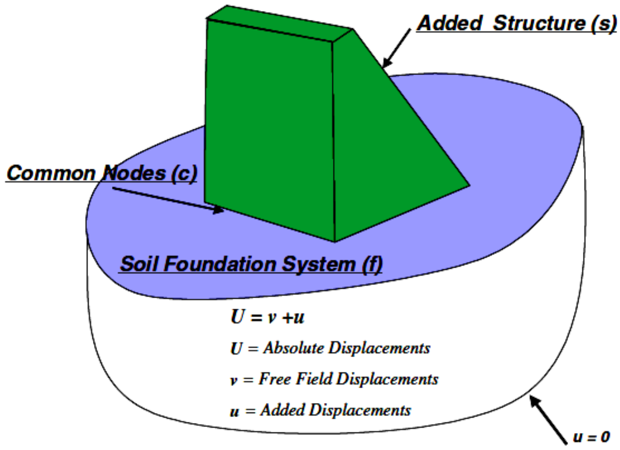

Fig ٢: Soil-Structure Interaction Model[٧].

$$\begin{bmatrix} M_{ss} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix} \begin{bmatrix} \ddot{U}_s \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} + \begin{bmatrix} K_{ss} & K_{cf} & 0 \\ K_{cf} & K_{cc} & K_{fc} \\ 0 & K_{fc} & K_{ff} \end{bmatrix} \begin{bmatrix} U_s \\ U_c \\ U_f \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{١}$$

Where the mass and the stiffness at the contact nodes are the sum of the contribution from the structure *(s)* and foundation *(f)*, and are given by equation (Eq. ٢).

$$M_{cc} = M_{ss}^{(s)} + M_{cc}^{(f)} \quad and \quad K_{cc} = K_{ss}^{(s)} + K_{cc}^{(f)} \tag{٢}$$

In terms of absolute motion, there are no external forces acting on the system. However, the displacements at the boundary of the foundation must be known. In order to avoid solving this SSI problem directly, the dynamic response of the foundation without the structure is calculated. In many cases, this free-field solution can be obtained from a simple one-dimensional site model. The three dimensional free-field solution is designated by the absolute displacements $V$and absolute accelerations, $\ddot{V}$. By a simple change of variables it is now possible to express the absolute displacements U and accelerations $\ddot{U}$ in terms of displacements U relative to the free-field displacements V, and given by equation (Eq. ٣).

$$\begin{bmatrix} U_s \\ U_c \\ U_f \end{bmatrix} = \begin{bmatrix} u_s \\ u_c \\ u_f \end{bmatrix} + \begin{bmatrix} v_s \\ v_c \\ v_f \end{bmatrix} \quad and \quad \begin{bmatrix} \ddot{U}_s \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} = \begin{bmatrix} \ddot{u}_s \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} \tag{٣}$$

Equation (١) can now be written as equation (Eq. ٤).

$$\begin{bmatrix} M_{ss} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix}\begin{bmatrix} \ddot{u}_s \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} K_{ss} & K_{sc} & 0 \\ K_{cs} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix}\begin{bmatrix} u_s \\ u_c \\ u_f \end{bmatrix} = -\begin{bmatrix} M_{ss} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix}\begin{bmatrix} \ddot{v}_s \\ \ddot{u}_c \\ \ddot{v}_f \end{bmatrix} + \begin{bmatrix} K_{ss} & K_{sc} & 0 \\ K_{cs} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix}\begin{bmatrix} v_s \\ v_c \\ v_f \end{bmatrix} = R$$ (٤)

If the free-field displacement V_c is constant over the base of the structure, the term V_s is the rigid body motion of the structure. Therefore, Equation (٤) can be further simplified by the fact that the static rigid body motion of the structure is given by equation (Eq. ٥).

$$\begin{bmatrix} K_{ss} & K_{cf} \\ K_{cf} & K_{cc}{}^{(s)} \end{bmatrix}\begin{bmatrix} v_s \\ v_c \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix}$$ (٥)

Also, the dynamic free-field motion of the foundation requires that equation (Eq. ٦) is written as follow.

$$\begin{bmatrix} M_{cc}{}^{(f)} & 0 \\ 0 & M_{ff} \end{bmatrix}\begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \end{bmatrix} + \begin{bmatrix} K_{cc}{}^{(f)} & K_{cf} \\ K_{cf} & K_{ff} \end{bmatrix}\begin{bmatrix} v_s \\ v_c \end{bmatrix} = \begin{bmatrix} 0 \\ 0 \end{bmatrix}$$ (٦)

Therefore, the right-hand side of Equation (٤) can be written as equation (Eq. ٧).

$$\begin{bmatrix} M_{ss} & 0 & 0 \\ 0 & M_{cc}{}^{(s)} & 0 \\ 0 & 0 & 0 \end{bmatrix}\begin{bmatrix} \ddot{v}_s \\ \ddot{v}_c \\ 0 \end{bmatrix}$$ (٧)

Hence, the right-hand side of the equation (Eq. ٤) does not contain the mass of the foundation. Therefore, three dimensional dynamic equilibrium equations, for the complete soil-structure system with damping added, are of the following equation (Eq. ٨) for a lumped mass system:

$$M\ddot{u} + C\dot{u} + Ku = -m_x\,\ddot{v}_x(t) - m_y\,\ddot{v}_y(t) - m_z\,\ddot{v}_z(t)$$ (٨)

Where, **M**, **C** and **K** are the mass, damping and stiffness matrices, respectively, of the soil-structure model. The added, relative displacements, **u**, exist for the soil-structure system and must be set to zero at the sides and bottom of the foundation. The terms, $\ddot{v}_x(t), \ddot{v}_y(t)$ and $\ddot{v}_z(t)$ are the free-field components of the acceleration if the structure is not present. The column matrices, m_i , are the directional masses for the added structure only.

Most structural analysis computer programs automatically apply the seismic loading to all mass degrees-of-freedom within the computer model and cannot solve the SSI problem. This lack of capability has motivated the development of the mass less foundation model. This allows the correct seismic forces to be applied to the structure; however, the inertia forces within the foundation material are neglected. The results from a mass less foundation analysis converge as the size of the foundation model is increased. However, the converged solutions may have avoidable errors in the mode shapes, frequencies and response of the system.

To activate the soil-structure interaction within a computer program it is only necessary to identify the foundation mass in order that the loading is not applied to that part of the structure. The program then has the required information to form both the total mass and the mass of the added structure. The SAP٢٠٠٠ program has this option and is capable of solving the soil-structure interaction problem correctly [٦].

٢. Data and Material

٢.١. Properties of frames

The first building is a ٧ story steel frame adopted from the verification manual of the SAP٢٠٠٠ program with a typical story mass ٨٥.٥ $KN\ S^{٢}/m$, all columns of this frame are section W۱۴×۲۶۴ and all beam are section W۲۴×۱۳۰, modules of elasticity of steel assumed ٢٠٠GPA, story height is same for all story and is ٤.١١m and the bays length is equal ٩.١٤m. The second building is ١٠ story reinforced concrete frames with story mass of ٨٥.٨ $KN\ S^{٢}/m$, typically load at each story is ٥٧.٧ KN/m, all columns of this frame are section ۵۰×۷۰ cm and all beam are section ۲۵×۷۰ cm, modules of elasticity are assumed ٢٠٠GPA for steel and ٣٥GPA for concrete[٧]. The TMD mass for frames is ١.٥% of total mass of frame and stiffness of TMD compute that natural period of TMD is equal to natural period of first mode of frame with fixed base [١].

٢.٢. Properties of ground motion

To investigate response of structure all of frames are influenced to ground motion. For this purpose the dynamic time history analysis is performed and use earthquake records north-south component of horizontal ground acceleration recorded at the Elcentro during the earthquake of May ١٨, ١٩٤٠.

٢.٣. Properties of soil

In this paper site consists of a surface foundation on a layered site, as illustrated in figure (Fig.٣) [٨].To evaluate dynamic-stiffness coefficients are used Cone method and CONAN program. CONAN is an executable program that can be used to compute the dynamic-stiffness coefficients foundations in frequency domain. The site may be horizontally layered, and the foundation may be partially or fully embedded. An input text file is first prepared describing the site and the foundation. The CONAN program is then executed, and the input text file describing the problem is processed. A simple menu is used to direct the program, indicating which coefficients to calculate, which range of frequencies to process, and whether to normalize the results or not. The program stores the results into an output text file. The results are output in dimensional form, tabulated against excitation frequency ω specified in rad/sec. typical form of this coefficient is similar to equation (Eq. ٩).

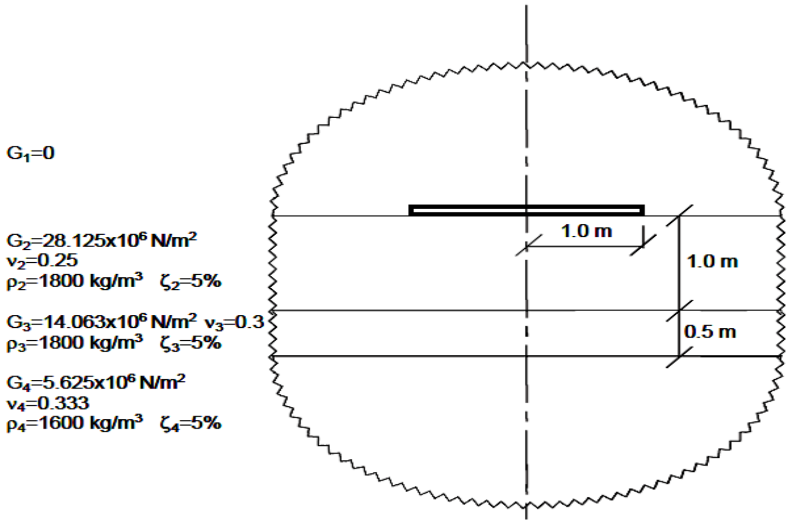

Fig ٣: Surface foundation on layered site overlying homogeneous half-space [٨].

$$S(\omega) = k(\omega) + i\,c(\omega) \tag{١}$$

The result of dynamics stiffness and damping against frequency computed and as shown in figure (Figs. ٤~٥). The frequency range is ٠ to ٥٠ HZ.

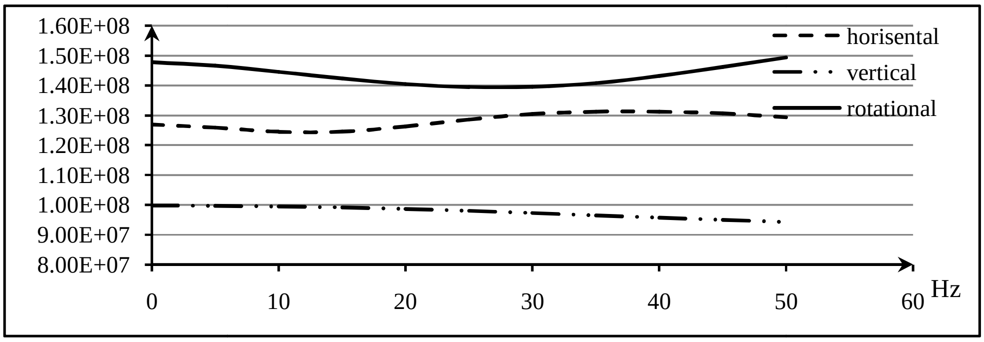

Fig ٤: dynamics stiffness against frequency.

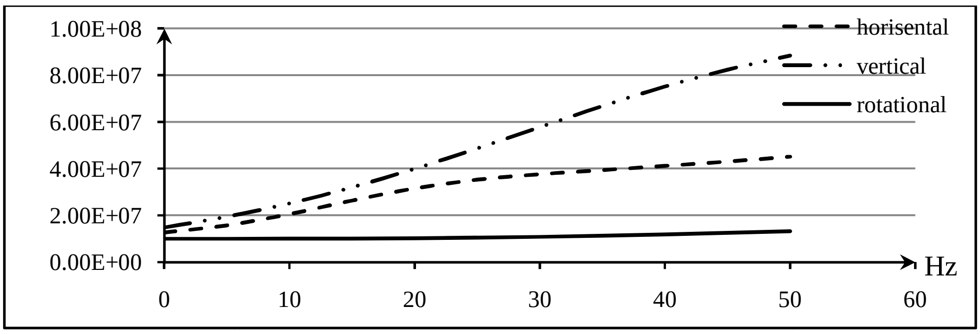

Fig ٥: dynamics damping against frequency.

٣. Research Methodology

Rigorous procedures to calculate the dynamic stiffness and the effective foundation input motion for seismic excitation exist. These include the boundary element method, sophisticated finite element methods such as the thin-layer method (the consistent boundary method), the scaled boundary finite-element method and the Dirichlet-to-Neuman method. These rigorous methods require a formidable theoretical background. A considerable amount of expertise in idealizing the dynamic system is necessary. Significant data preparation and interpretation of the results must be performed [٨].

The vast majority of foundation vibration analyses will thus not be performed using rigorous methods, but with strength-of-materials approaches. Three different types of models are available: lumped-parameter models, consisting of a few degrees of freedom connected by springs, dashpots and masses with constant frequency-independent coefficients; representations based on prescribed wave patterns in the horizontal plane of one-dimensional body and surface waves and cylindrical waves; and cone models [٨].

Dynamic time history is used for analysis. For this purpose analytical modeling is achieved by SAP٢٠٠٠ package program. To compare of behavior of TMD in structures, for frame without and with considering soil-structure interaction, response of the frames has been investigated with north-south component of horizontal ground acceleration recorded at the Elcentro during the earthquake ١٩٤٠. In order to solve dynamic time history analysis, it has been used by direct integration. Time history responses including horizontal displacements, base shears in all degrees of freedom have been computed. In all frames inherent critical structural damping ratio of ٥% is assumed. In order to modeling springs and dampers that its stiffness and damping depend to frequency has been used frequency dependent link element in SAP٢٠٠ package that is suitable for this purpose [٦].

٤. Results and Analysis

Fundamental natural period of frames without and with considering soil-structure interaction, are shown in Table ١. As mentioned that natural period of structures considering soil flexibility is greater than structures assuming fixed base.

Frames	natural period with fixed base (Sec)	natural period with flexibility base (Sec)
٧ storey steel frame	١.٢٧	١.۶۵
١٠ storey concrete frame	١.۶۴	٢.٠۵

Table ١. Fundamental natural period of frames.

Top storey response has been shown in four states with and without TMD and soil-structure interaction. Figure (Figs. ٦) shows that for steel frames. As it is shown in figure (Figs. ٧), with assumption of fixed base structure is effective to reduce response of structure. In consideration of soil-structure interaction, response of structure will in some cases be greater than structure without TMD and fixed base. So this study shows that when TMD is designed base on structure with fixed base and not considering soil-structure interaction, response of structure may be greater than structure without TMD and considering soil-structure interaction.

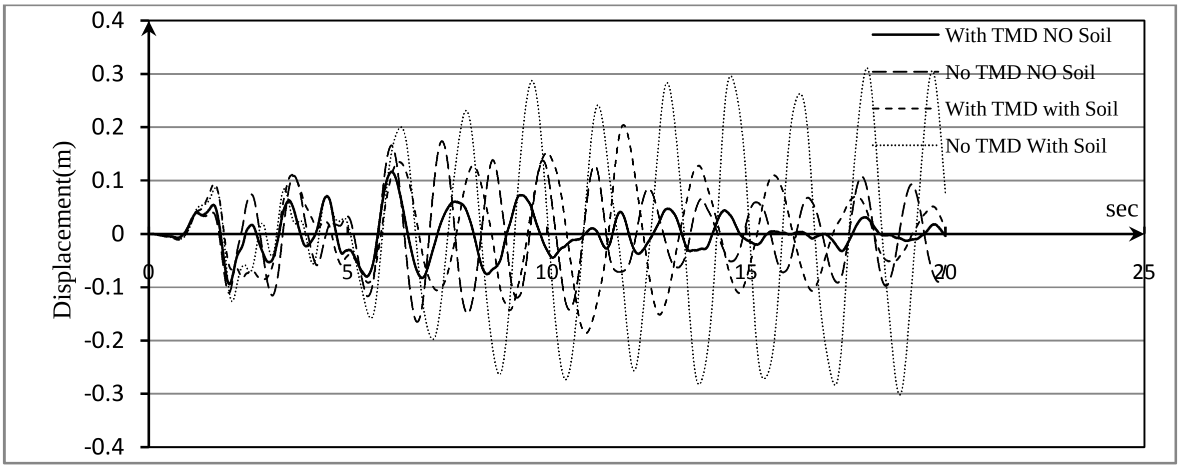

Fig ٦: Top storey relative horizontal displacement (steel frame).

Also Figure (Figs. ٧) shows top storey response for concrete frame. In this figure the results are similar to pervious results.

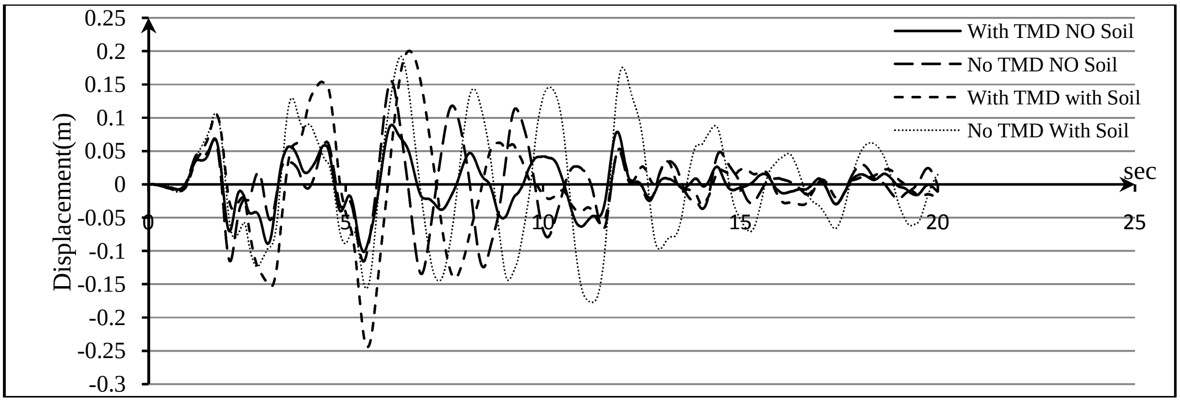

Fig ٧: Top storey relative horizontal displacement (concrete frame).

٥. Conclusions

١- The numerical result indicates that considering soil-structure interaction causes of increasing natural period of structures. For instance natural period is increased about ٣٠% in ٧storey steel building.

٢- In order to the best performance of TMD, in designing process should be considered natural period of structure with considering soil-structure interaction.

٣-However using of TMD reduces response of structures, when TMD is designed base on structure with fixed base, response of structure may be greater than structure without TMD and considering soil-structure interaction.

٤- The computer package SAP٢٠٠٠ is suitable for modeling TMD and soil-structure interaction.

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