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Effects of Soil-Foundation-Superstructure Interaction on Seismic Response of Cable-Stayed Bridges Tower with Spread Footing Foundation

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A nonlinear dynamic soil structure interaction analysis is developed to estimate the seismic response characteristics and to predict the earthquake response of cable-stayed bridges tower with spread foundation that is supported on a good bearing layer. Rocking response of a spread foundation under a strong near-field ground motion may result in the uplift in the foundation and it often results in the yield of the underlying soils. So in this analysis both of strain-dependent material nonlinearity and geometrical nonlinearity by base mat uplift are considered. An incremental iterative finite element technique is adopted for a more realistic dynamic analysis under great earthquake ground motion of nonlinear soil-foundation-superstructure interaction system. The numerical results show that the soil foundation interaction in the physical sub-structure stiffness model reduces the tower member forces. The soil yielding below the foundation and uplift at the interface have significantly contribution to foundation rocking response. The soil bearing stress beneath the footing base dramatically increases due to footing base uplift. The predominant contribution to the vertical response at footing base comes from the massive foundation rocking rather than from the vertical excitation.

Key words: Cable-stayed bridges tower, soil-structure interaction, soil yielding, uplift, seismic design, seismic response

1. Introduction

In recent decades, the long span bridges such as cable-stayed and suspension bridges have gained much popularity, due to their aesthetic appearance, efficient utilization of structural materials, increase of the horizontal navigation clearances and the economic trade off of span length cost of deep water foundation. These bridges structural synthesis provide a valuable environment for the nonlinear behavior due to material and geometrical nonlinearities of the structure relatively large deflection on the stresses and forces[1~3]. The Hyogoken-Nanbu earthquake of 17th January 1995, led to an increased awareness concerning the response of highway bridges subjected to earthquake ground motions, the ductility design and dynamic analyses have been reconsidered by Japan Road Association[3~5]. The necessity has arisen to develop more efficient analysis procedures that can lead to a through understanding and a realistic prediction of the precise three-dimensional nonlinear dynamic response of bridge structural systems to improve the bridges seismic performance.

The safe and economic seismic designs of bridge structures depend directly on the understanding level of seismic excitation and the influence of supporting soil on the structural dynamic response. Long span bridges are susceptible to relatively more severe soil structure interaction effect during earthquakes as compared to buildings due to their spatial extent, varying soil condition at different supports and possible incoherence in the seismic input. The consideration of the soil structure interaction effects in analysis requires the system to be extended from the structure to include the total structure foundation soil system. Although soil-structure results in a significant modification of the system properties, which in turn alter its seismic response, most current analysis methods that are either used or recommended in design codes (e.g. Caltrans, Seismic Design References, 1994) do not account for it. This discrepancy can be primarily attributed to numerous difficulties including the problem complexity, scarce pertinent experimental data and the lack of an easy to use design procedure. On the other hand, the necessity of incorporating soil structure interaction in the design of a wide class of bridge structures has been pointed out by several post earthquake investigations, experimental and analytical studies[3, 6~8)], especially that have been constructed on relatively soft ground, which results in a great demand to evaluate the effects of soil-structure interaction on the seismic behavior of bridges and properly reflect it in a seismic design to accurately capture the response, enhance the safety level and reduce design costs.

The dynamic interaction between the pier-foundation and soil has a significant effect on the earthquake response of bridges. Soil-structure interaction can be classified into two effects. The first effect, kinematic interaction, is the modification of the free-field ground motion by the presence of the massless foundation. In this study, kinematic interaction is considered through a simplified approach of effective ground motion at foundation base level[9, 10)]. The second form of interaction, inertial soil-structure interaction, is caused by the deformation of the soil by the time varying inertia-induced forces developed in the footing. The dynamic characteristics of soil structure-structure interaction system change due to materials and geometrical nonlinearity during a severe earthquake. This nonlinearity is sometimes treated by equivalent linear model[11 ~ 13)]. But the dynamic characteristics of soil structure interaction system, such as the shape of the peak of frequency transfer function change from high frequency-low damping type to low frequency-high damping type, are depending on the stress level of the surrounding soil during a severe earthquake[14~16)].

The purpose and objective of this study are to assess the effects of soil-structure interaction on the seismic response and dynamic performance of the cable-stayed bridges tower with spread foundation supported on a good bearing layer. An incremental iterative finite element technique for a more realistic dynamic analysis of nonlinear soil-foundation-superstructure system subjected to earthquake ground motion is developed. Particular attention is directed to the tower structure, which are modeled as beam-column finite element with both material and geometric nonlinearities, while the soil is idealized by nonlinear springs and dashpots uniformly attached along most of the embedded length of the pier. Both material and radiation damping are incorporated in the model. Rocking response of a spread foundation under a strong earthquake ground motion may result in the uplift in the foundation and it often results in the yield of the underlying soil[17)]. So, both of strain-dependent material nonlinearity and geometrical nonlinearity by base mat uplift is considered through nonlinear soil element connected in series with gap element springs system. The results of this study show that a massive rigid foundation activates the high frequency translational motion of the input ground motion and generates foundation-rocking responses. The predominant contribution to the vertical response at footing base comes from the massive foundation rocking rather than from the vertical excitation. The permanent settlement is found to be the less significant. The rocking vibration dominates the lateral bearing stress for the soil along the embedded depth of the tower pier.

2. General Solution Procedure

Based on the total incremental equilibrium equations, finite displacement three-dimensional beam-column element formulation is carried out. The governing nonlinear dynamic equation of the tower response can be derived by the principle of energy that the external work is absorbed by the work of internal, inertial and damping for any small admissible motion that satisfies compatibility and boundary condition. By assembling the element dynamic equilibrium equation for the time $t+\Delta t$ over all the elements, the incremental FEM dynamic equilibrium equation[2, 18)] can be obtained as:

$$[\mathbf{M}]\{\ddot{u}\}^{t+\Delta t}+[\mathbf{C}]\{\dot{u}\}^{t+\Delta t}+[\mathbf{K}]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = \{\mathbf{F}\}^{t+\Delta t}-\{\mathbf{F}\}^{t} \tag{1}$$

where $[\mathbf{M}]$, $[\mathbf{C}]$, and $[\mathbf{K}]^{t+\Delta t}$ are the system mass, damping and tangent stiffness matrices at time $t+\Delta t$, the tangent stiffness considers the material nonlinearities through bilinear stress strain relation for the beam column element, and the geometrical nonlinearities for the case of in-plane, out-plane bending deformations and linear torsional deformations. $\ddot{u}$, $\dot{u}$, and Δu are the accelerations, velocities, and incremental displacements vector at time $t+\Delta t$, respectively, $\{\mathbf{F}\}^{t+\Delta t}$- $\{\mathbf{F}\}^{t}$ is the unbalanced force vector. It can be noticed that the dynamic equilibrium equation of motion takes into consideration the different sources of nonlinearities both geometrical and material nonlinearities, which affect the tangent stiffness and internal forces calculations.

The implicit Newmark step-by-step integration method is used to directly integrate the equation of motion and then it is solved for the incremental displacement using the Newton Raphson iteration method where the stiffness matrix is updated at each increment to consider the geometrical and material nonlinearities and to speed the convergence rate. In addition, the tower structure damping mechanism is adapted to the viscous damping of Rayleigh's damping type with equivalent damping coefficient[4] to equal to 2% for steel material of tower superstructure elements and 10% for concrete material of embedded pier substructure elements at vibration periods of 2.5 sec and 0.50 sec, to represent a broad range of high participation modes and the softening that takes places as the columns and soil yield. A common design approach of maintaining elastic behavior in the substructure to avoid inelastic behavior below the ground surface is considered, where the damage would be difficult to detect or to repair.

3. Tower Structure Model

The tower of Iwamizawa cable-stayed bridge located in Hokkaido, Japan is considered. Since the cable-stayed bridges are not structurally homogeneous, it is concluded from previous study that the tower, deck and cable stays affect the structural response in a wide range of vibration modes. The tower is taken out of the cable-stayed bridge and modeled as three-dimensional frame structure. A fiber flexural element[19, 20] is developed for the tower characterization, the element incorporates both geometric and material nonlinearities, a cubic displacement field is employed for the transverse displacement and linear displacement field is employed for the axial and torsional displacements. The stress-strain relationship of the beam element is modeled as bilinear type. The yield stress and the modulus of elasticity are equal to 355 MPa (SM490) and 200GPa, respectively; the strain hardening in the plastic region is 0.01.

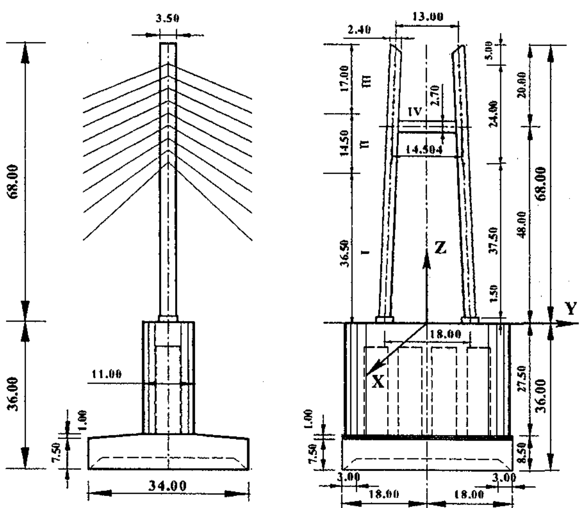
Inelasticity of the flexure element is accounted for by the division of the cross section into a number of fiber zones with uniaxial plasticity defining the normal stress-strain relationship for each zone, the element stress resultants are determined by integration of the fiber zone stresses over the cross section of the element. By tracking the center of the yield region, the evolution of the yield surface is monitored, and a stress update algorithm is implemented to allow accurate integration of the stress-strain constitutive law for strain increments, including full load reversals. To ensure path dependence of the solution, the implementation of the plasticity model for the implicit Newton-Raphson equilibrium iterations employs a stress integration whereby the element stresses are updated from the last fully converged equilibrium state.

The nonlinearity of inclined cable stays is idealized by using the equivalent modulus approach[21]. The nonlinearity of the cable stays originates with an increase in the loading followed by a decrease in the cable sag as a consequence the apparent axial stiffness of the cable increases. In this approach each cable is replaced by a truss element with equivalent tangential modulus of elasticity E_{eq} that is given by Ernst as:

$$E_{eq}=E\,/\,\{1+EA\,(\,wL\,)^2\,/\,12T^3\,\} \tag{2}$$

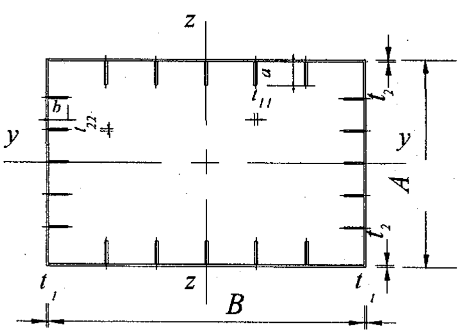
where E is the material modulus of elasticity, L is the horizontal projected length of the cable, w is the weight per unit length of the cable, A is the cross sectional area of the cable and T is the tension force in the cable.

This cable-stayed bridge tower has nine cables in each side of the tower. The stiffening girder dead load is considered to be equivalent to the vertical component of the pretension force of the cables and acted vertically at their joints, and two vertical components at stiffening girder-tower connection at substructure top level. For the numerical analysis, the tower geometry and the structural properties of the steel superstructure and concrete substructure are shown in Fig. 1. The tower steel superstructure has rectangular hollow steel section with internal stiffeners, which has different dimensions along the tower height and its horizontal beam as shown in Table 1.



(a)

Tower geometry



(b) Cross section

Fig. 1 Steel tower of Iwamizawa cable-stayed bridge

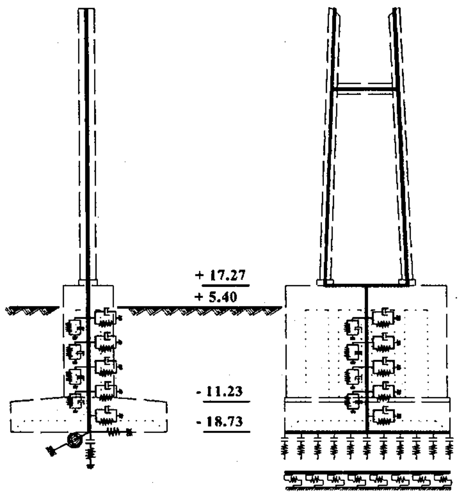


Fig. 2 Mathematical model of soil foundation superstructure system

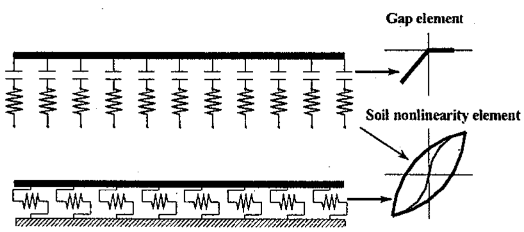


Fig. 3 General concept of uplift and sliding modeling

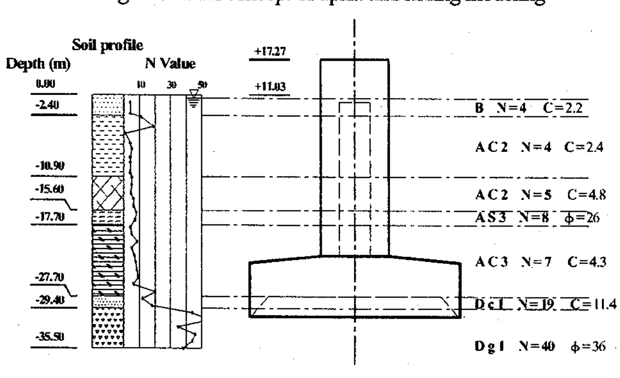


Fig. 4 Soil profile at the bridge tower site

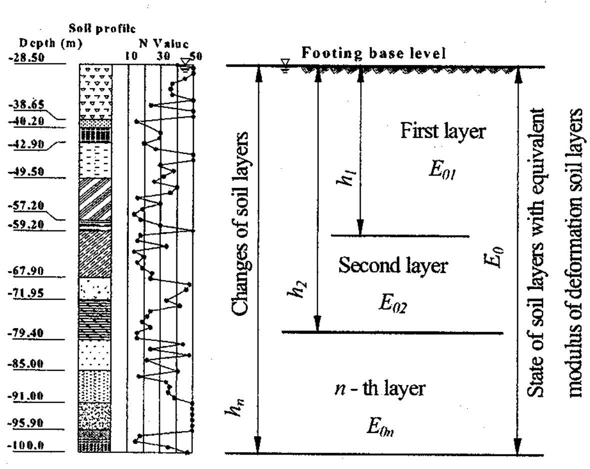


Fig. 5 Soil layers changes in the direction of depth

Table 1 Cross section dimension of different tower region

C. S. Dim. (cm)		Outer dimension				Stiffener dimension			
		A	B	t_1	t_2	a	b	t_{11}	t_{22}
Tower parts	I	240	350	2.2	3.2	25	22	3.6	3.0
	II	240	350	2.2	3.2	22	20	3.2	2.8
	III	240	350	2.2	2.8	20	20	2.8	2.2
	IV	270	350	2.2	2.6	31	22	3.5	2.4

4. Soil Foundation Model Characterization

4.1 Soil Behavior

Three types of soil resistance displacement models could describe the soil characteristics. The first type represents the lateral soil pressure against the pier and the corresponding lateral pier displacement relationship. The second type represents the skin friction and the relative vertical displacement between the soil and the pier relationship. The third type describes the bearing stress beneath the spread footing and settlement relationship. All three types assume the soil behavior to be nonlinear and can be developed from the basic soil parameters.

The interaction between the soil and the structure is simulated with nonlinear springs and dashpots system along the embedded depth of the pier, and both of strain-dependent material nonlinearity and geometrical nonlinearity by base mat uplift are considered through nonlinear soil element connected in series with gap element springs system as shown in Figs. 2 and 3. The spring constants in both bridge axis and right angle directions are calculated based on foundation geometry and soil profile of different layers underneath and along embedded depth of foundation as specified in Japanese Highway Specification[4,5] as illustrated in Figs. 4 and 5. Soil properties from the Standard Penetration Test (SPT) data and logs of borehole at tower site are used to determine the coefficients of vertical and horizontal subgrade reaction that are used in a spread foundation design.

The coefficients of vertical and horizontal shear subgrade reactions at the bottom and of horizontal subgrade reaction in front of the foundation embedded portion are obtained from the ground stiffness corresponding to the deformation caused in the ground during an earthquake. Moreover, an equivalent modulus of deformation (E_0) for which soil layers depth (h_i) and deformation modulus (E_{0i}) below the foundation bottom change in the direction of the depth is considered, as seen in Fig. 5. The soil properties are obtained from field investigations carried out prior to the construction of the bridge. Since a homogeneous half-space is assumed to model the different layers ground, the elastic properties used in the analysis are the averaged values of the first several layers of about 100 m in depth.

4.2 Material Nonlinearity Idealization

One of the most important factors in the analysis of soil-foundation interactive behavior is the nonlinear constitutive laws for the soil. In this study, Hardin-Dmevich model is proposed to represent the soil material nonlinearity that is often used for its capacity to trace the degradation of stiffness[22]. The parameters used to trace the skeleton curve and its hysteresis curves are indicated in Fig. 6. The skeleton curve is expressed as

$$\tau = G_0\,\gamma / (1 + |\gamma/\gamma_r|), \quad \gamma_r = \tau_{max} / G_0 \tag{3}$$

Where G_0 is the initial shear modulus; τ is the generalized soil shear stress, τ_{max} is the shear stress at failure, γ_r is the reference strain, and γ is the generalized strain. The hysteretic curve can be constructed by applying Masing's rule and is given as

$$\tau \pm \tau_m = G_0\,(\gamma \pm \gamma_m) / \{1 + |(\gamma \pm \gamma_m)/2\gamma_r|\} \tag{4}$$

τ_m and γ_m indicate the coordinates of the origin of the curve, that is the point of most recent load reversal. The nonlinear dynamic soil parameters including the dynamic shear moduli and the damping ratios for the employed soil models in this study are modulated based on the shear strain dependent relationships for gravel, sand and clay shown in Fig. 7; soil exhibits nonlinear nature even at small strains. The shear modulus (G) can be described as

$$G / G_0 = 1 / (1 + \gamma_m/\gamma_r) \tag{5}$$

4.3 Geometrical Nonlinearity Idealization

The stiffness of supporting soil is idealized by spring model. The spring coefficients are computed by the method suggested in Specification for Highway Bridges issued by Japan Road Association[4, 5]. Although it has been recognized that the spring coefficients are frequency dependent, the spring coefficients computed using this method are frequency independent for practical use. Each spring consists of a gap element and a soil element. The gap element transmits no tensile stress, which can express the geometrical nonlinearity of base mat uplift[16]. The general concept of this model is shown in Fig.3.

4.4 Hysteretic Damping of Soil

The damping characteristic of the soil resulting from the deformations produced by interaction with the pier is represented by nonlinear viscous dashpots have been employed. The damping ratio of the soil dashpots considering strain-dependent material nonlinearity is described by a simple relationship between the shear modulus and damping[23] as shown in Fig. 8.

$$h = (\Delta w / w)/2\pi = (2/\pi)[(2G_0/G)\{(\gamma_r/\gamma_m) - (\gamma_r/\gamma_m)^2 \log(1 + \gamma_r/\gamma_m)\} - 1] \tag{6}$$

Where Δw is energy dissipated during the stress strain cycle and w is elastic energy of an equivalent linear system shear modulus G. The material-damping ratio is defined as

$$h = C_m / C_r \tag{7}$$

in which C_m is the coefficient of material damping, C_r is the coefficient of material critical damping[14] that can be given as:

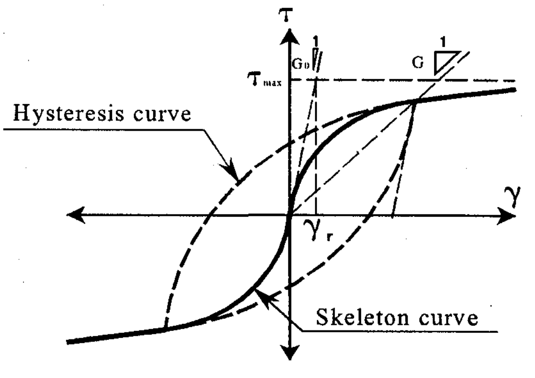
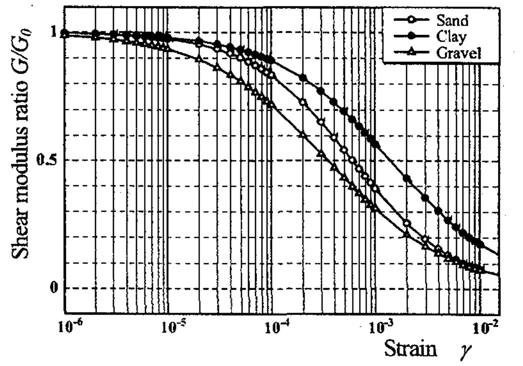
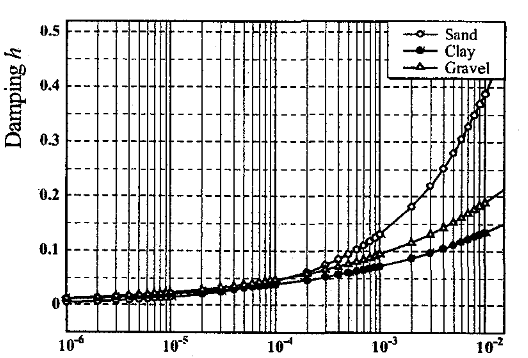


Fig. 6 Concept of Hardin-Dmevich model and Masing's Rule hysteresis loop



(a) Rigidity



(b) Damping

Fig. 7 Strain-dependent soil material nonlinearity

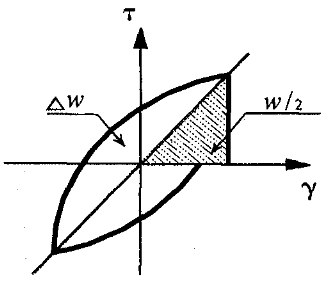


Fig. 8 Hysteretic damping of soil

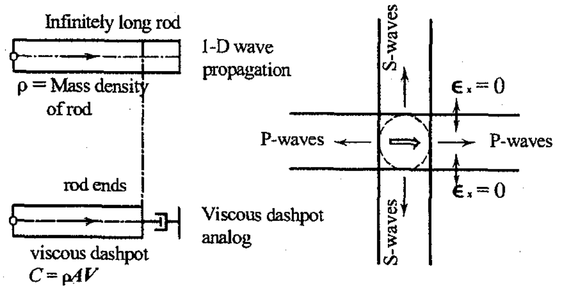


Fig. 9 One-dimensional radiation damping model

$$C_r = 2\sqrt{k/m} \tag{8}$$

in which k and m are the soil spring stiffness and pier mass per unit length, respectively, the coefficient of material damping of the soil is obtained as

$$C_{soil} = 2\, h_{max}(1 - k/k_0)\ \sqrt{k/m} \tag{9}$$

4.5 Radiation Damping of Soil

As an approximation of the para-axial boundary, viscous dampers can be used to represent a suitable transmitting boundary for many applications involving both dilatational waves and shear waves. Since the accuracy is generally acceptable and the procedure is simple and versatile in approximately modeling nonlinear soil behavior, a one-dimensional viscous boundary model is selected for this study. It is assumed that a horizontally moving pier cross section would solely generate one-dimensional P-waves traveling in the direction of shaking and one-dimensional SH-waves in the direction perpendicular to shaking as shown in Fig. 9. The coefficient of viscous dashpot that will absorb the energy of the waves originating at soil-pier interface[14] can be written as

$$C = 2\,\rho A\, V_s(1 + V_p/V_s) \tag{10}$$

$$V_p = V_s\{2\,(1-\nu)/(1-2\nu)\}^{0.5} \tag{11}$$

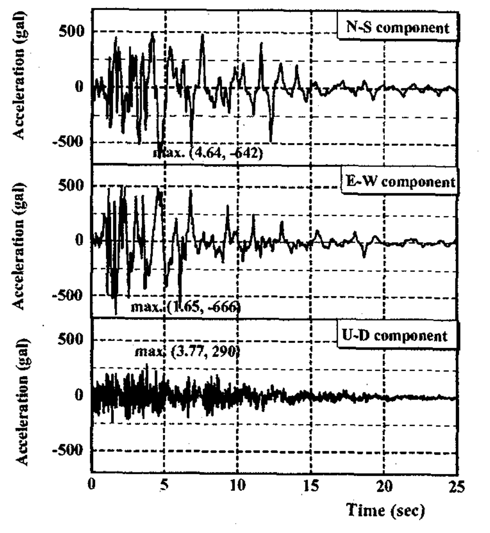


Fig. 10 Strong ground motion measured at JR Takatori observatory

Where V_p and V_s are P-wave and S-wave velocities, respectively. The use of V_p as an approximate wave velocity in the compression-extension zone implies that a perfect constraint is provided in the near field by two lateral boundaries, ($\varepsilon_y = \varepsilon_z = 0$), which is inconsistent with the assumed stress free surrounding soil in the x-direction as shown in Fig. 9. To remove the singularity in V_p as $\nu \to 0.5$, the Lsymer's analog wave velocity can be used instead[12]

$$V_{La} = 3.4\ V_s/\{\pi\,(1-\nu)\} \tag{12}$$

The use of V_{La} recognizes the fact that compression-extension oscillations propagate with some degree of normal straining in the x-direction in the near field. Because the absorption characteristics of this boundary are independent of frequency, the boundary can absorb harmonic and non-harmonic waves.

5. Determination of Input Ground Motion

In the dynamic response analysis, the seismic motion by an inland direct strike type earthquake that was recorded during Hyogoken-Nanbu earthquake 1995 of high intensity but short duration is used as an input ground motion to assure the seismic safety of bridges. The horizontal and the vertical accelerations recorded at the station of JR Takatori observatory[3~5], as shown in Fig. 10, are suggested for dynamic response analysis of the of cable-stayed bridge tower at type II of soil condition. It is considered to be capable of securing the required seismic performance during the bridge service life. The selected ground motion has maximum acceleration intensity of its components equal to 642 gal (N-S), 666 gal (E-W) and 290 gal (U-D).

Table 2 Summary of principal vibration modes for global model

Mode order	Period sec.	Modal mass participation as a fraction of total mass			Mode type
		Global X	Global Y	Global Z	
1	2.1645	0.0000	0.3486	0.0000	H_1
2	0.9208	0.3052	0.0000	0.0002	L_1
3	0.7995	0.0000	0.0000	0.0000	T_1
4	0.5231	0.00001	0.0000	0.0003	V_1
5	0.3805	0.0300	0.0000	0.00008	L_2
6	0.3688	0.0000	0.0001	0.0000	H_2
7	0.3375	0.0000	0.0000	0.0000	T_2
8	0.1857	0.0000	0.0862	0.0000	H_3
9	0.1718	0.0002	0.0000	0.5320	V_2
10	0.1497	0.4132	0.0000	0.0000	L_3
11	0.13315	0.0000	0.0000	0.0002	V_3
12	0.1302	0.0000	0.2123	0.0000	H_4
13	0.0968	0.0000	0.0126	0.0000	H_5
14	0.0946	0.0042	0.0000	0.0000	L_4
15	0.09235	0.0000	0.0000	0.0000	T_3
16	0.07286	0.0000	0.0000	0.00002	V_4
17	0.0568	0.2469	0.0000	0.0000	L_5
18	0.0562	0.0000	0.0000	0.4655	V_5
19	0.0493	0.0000	0.0557	0.0000	H_6
20	0.0475	0.0000	0.2844	0.0000	H_7
Sum	–	0.99971	0.9999	0.9993	–

H: transverse vibration (in-plane), T: torsion vibration
L: longitudinal vibration (out-plane), V: vertical vibration

From Fourier spectrum analysis, the predominant frequencies for N-S and E-W components are 0.83 and 0.81 Hz, respectively, which are relatively low, while the U-D component predominant frequency is 7.96Hz that indicates a high frequency contents and a time lag to horizontal motions.

A simple model[9, 10] based on analytical modeling of rigid foundation is adopted for the kinematic interaction by the free-field ground motion modification, so that an effective seismic motion (ESM) for a soil-structure system can be obtained at foundation footing base level as follow:

(1) For rotational acceleration components

$$\frac{\ddot{\varphi}_i' \, a}{\ddot{u}_i} = \begin{cases} [(0.2-0.4G_s/G)(d/a-1)/3+0.4\,G_s/G][1-\cos kd] & 0 \le kd \le \pi/2 \\ [(0.2-0.4G_s/G)(d/a-1)/3+0.4\,G_s/G] & kd \ge \pi/2 \\ 0.2 \qquad d/a \ge 4 & kd \ge \pi/2 \end{cases} \qquad i = X, Y, Z \tag{13a}$$

(2) For translational acceleration components

$$\frac{\ddot{u}_i'}{\ddot{u}_i} = \begin{cases} |\sin kd / kd| & 0 \le kd \le \pi/2 \\ 0.63 & kd \ge \pi/2 \end{cases} \tag{13b}$$

in which $\ddot{u}_i$ are the free-field ground motion components, $\ddot{u}_i'$ and $\ddot{\varphi}_i'$ are the corresponding effective seismic motion components, G_s is the shear modulus of the surface stratum, G is the shear modulus of the soil beneath the foundation base, d is the embedment depth and the coefficient k is evaluated as:

$$k = \omega / V_s \tag{14}$$

where ω is the frequency of the wave, V_s is the shear wave velocity in the soil medium, a is the equivalent radius for rocking vibration to rectangular foundation of dimensions $2b \times 2\,l$ and its value can obtained as follow:

$$a = [8bl\,(b^2 + l^2)/3\pi]^{0.25} \tag{15}$$

The earthquake force of E-W wave component is put into the bridge axis direction. This simplified approach of the effective seismic motion at the foundation base is used to consider the soil structure kinematic interaction due to seismic wave effects.

6. Numerical Analysis and Discussion

6.1 Natural Vibration Analysis

The fundamental period of soil-foundation-superstructure system is one of the most crucial design factors, in order to understand the soil structure interaction effects on the fundamental period of tower structure. A three-dimensional frame model with a consistent mass model has been adopted for eigenvalue and modal analyses. The natural periods and the participation factors for different modes in three global directions obtained from the analysis are shown in Table 2. The lowest vibration mode, with a 2.1645 sec period, involves transverse vibration of the entire tower structure (right angle to the bridge axis). The second mode (0.9208 sec) is the longitudinal vibration (bridge axial direction). The vertical vibration modes have periods of 0.523 sec to 0.056 sec, including modes 4, 9,11,16 and 18. It is apparent that the participation factors for the first and second modes are not only of larger values. But, there are other modes of vibration with large values of participation factors showing a very complicated dynamic behavior.

The results of fundamental period analysis have demonstrated that soil structure interaction prolongs the fundamental period of tower-foundation-soil system. As it is well known, the seismic response of structure depends on the relationship between transfer function of structure and the predominant frequency of seismic motion. The variation of fundamental period certainly affects the tower seismic response.

6.2 Nonlinear Dynamic and Time History Analyses

To study the effects of soil-foundation-superstructure interaction on the seismic behavior of the cable stayed bridges tower, soil-foundation-superstructure model with Winkler hypothesis for soil idealization is employed and dynamic response simulation is conducted by applying acceleration at the base of the soil deposit: the following three different cases of soil idealization are analyzed

Case I: Seismic response of tower model with linear and elastic soil model and equivalent viscous damping of soil.

Case II: Time history nonlinear response with strain-dependant material nonlinearity through Harden-Drnevich soil model.

Case III: Nonlinear dynamic analysis with both soil material nonlinearity and geometrical nonlinearity by base mat uplift.

The acceleration time history at the tower top clarifies that the soil nonlinearity effects in reducing the acceleration response due to degradation of soil stiffness and attenuation of the tower response due to energy dissipation through soil hysteresis. For the studied case, the reduction of acceleration response related to material nonlinearity (case II) of soil approaches about 1 ~ 4 % of that of linear elastic soil (case I), and the geometrical nonlinearity by base mat uplift has slightly effect due to rigid and massive foundation and stiff footing soil, as illustrated in Fig. 11. The moment and vertical force time histories at tower superstructure base show the nature of the response to strong ground motion. The large pulse in the ground motion produces a two or three cycles of large force response, with the amplitude of moment or force decaying rapidly after the peak excursions as shown in Figs. 12 and 13. The predominant contribution to the vertical force response at tower superstructure base comes from rocking vibration rather than from the vertical excitation, dominated by long period in-plane vibration, and slightly affected by high frequency vertical excitation and vertical vibration. The axial force response time history is characterized by high frequency spike related to high frequency content of vertical excitation and symmetric dynamic tension-compression peaks response around the dead load response from static analysis of 12.15 MN value as illustrated in Fig. 13. The dynamic compliance of the soil-foundation on nonlinear soil exhibited greater variation than the same foundation on an elastic soil that can be noted in forces response reduction, even slight variation for the studied case due to massive and rigid foundation and stiff footing soil beneath and in front of the spread foundation. In general, the nonlinearities at the soil-foundation interface as well as the uplift at the interface have the effect of significantly reducing the tower member forces. This idealization, at least, is more conservative that an analysis with fixed base idealization.

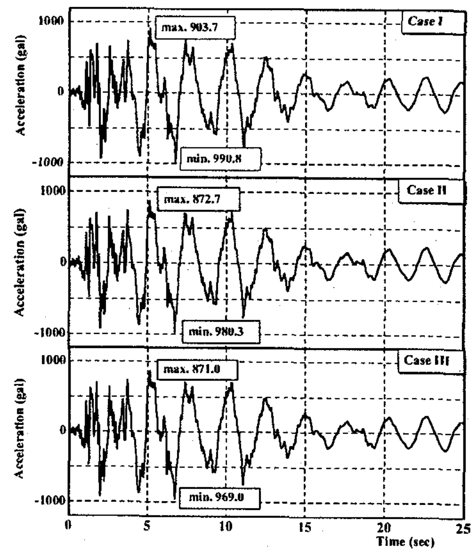


Fig. 11 Acceleration time history at tower top

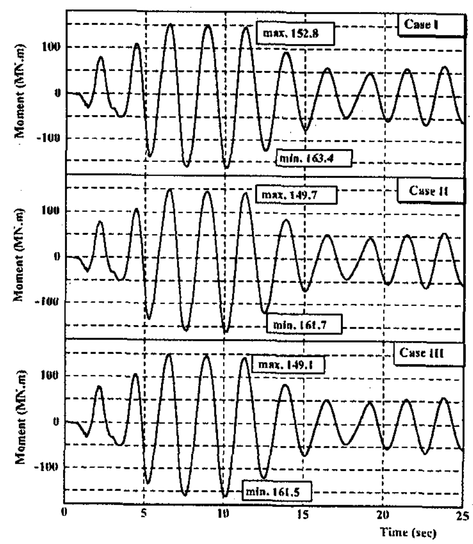


Fig. 12 In-plane moment time history at tower superstructure base

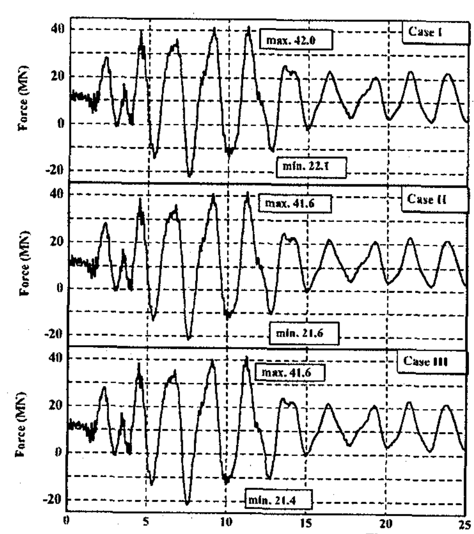


Fig. 13 Vertical force time history at tower superstructure base

Since the foundation (structure) is very rigid compared to the underlying foundation soil, the rocking motion of the foundation is significantly appeared, which affects the characteristics of the foundation motion. The limited shear strength of soil also causes sliding of the foundation block, which could alter the characteristics of the foundation motion. In addition to the inertial soil foundation interaction due to the existence of foundation mass, the size and stiffness of the foundation could also affect the foundation motion characteristics. From superstructure base displacement time history for the different three cases of soil modeling, as shown in Fig. 14, it can be clarifies that the soil effects on substructure response. Since the displacement could reach around 3.0 cm for case III of consideration both material and geometrical nonlinearities, even for elastic and linear soil case I, reach about 2.4 cm, which could have the effect of differential settlement on the redistribution of forces in the superstructure members. This effect is totally ignored for fixed base assumption.

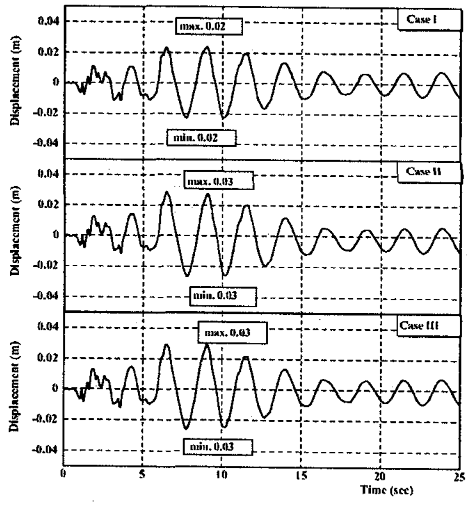


Fig. 14 In-plane displacement time history at superstructure base

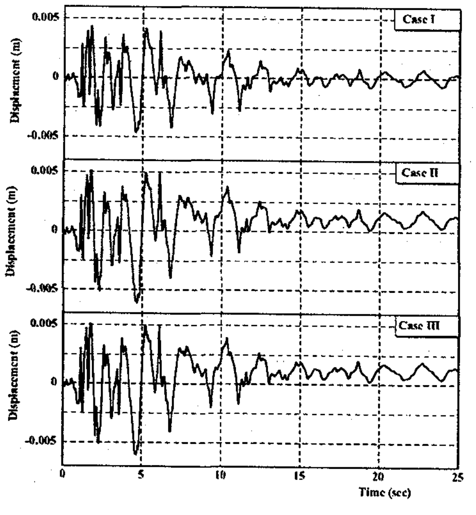


Fig. 15 Foundation sliding time history at footing base level

The limitation shear strength of basement soil causes sliding of the foundation, which could be underestimated by elastic and linear soil assumption as illustrated in Fig. 15. The sliding deformation could lead increase of hysteresis deformation around the footing and along the embedded depth of the tower pier as shown in Figs. 16 and 17. The hysteresis deformation of soil is significantly affected by the foundation rocking, this phenomenon can be clarified by studying the hysteresis along the embedded depth, which presents small hysteresis at the lower layer while the response of the upper layer is characterized by cycles of large shear strain and very small shear stress. The geometrical soil nonlinearity has slight effect in the formation of hysteresis along the depth. The energy dissipated through the soil hysteresis could lead to the tower seismic response attenuation.

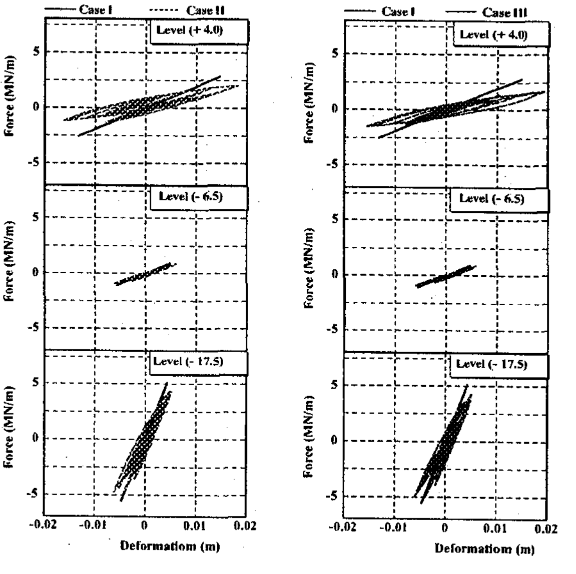


Fig. 16 Soil in-plane hysteresis along the embedded depth

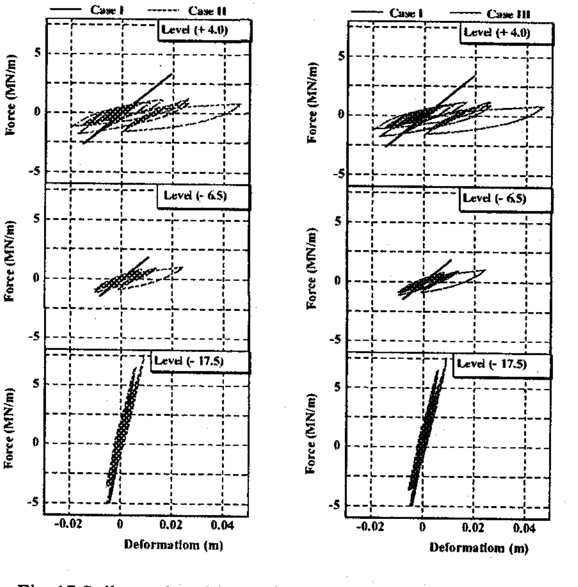


Fig. 17 Soil out-plane hysteresis along the embedded depth

The effects of foundation uplift of the tower are studied by comparing the responses obtained by either allowing or constraining uplift. The uplift of the tower footing is investigated through vertical deformation and force resistance of the basement soil at the extreme right and left of footing base for three different cases of study. It can be concluded that the contribution to the vertical response at footing base comes from the massive foundation rocking rather than the vertical excitation. Moreover, Two distinct aspects of foundation response to seismic excitation, rocking deformation and the accumulation of the permanent settlement can distinguished. The permanent settlement was found to be the less significant in the total vertical displacement at footing base level, as shown in Figs. 18 and 19.

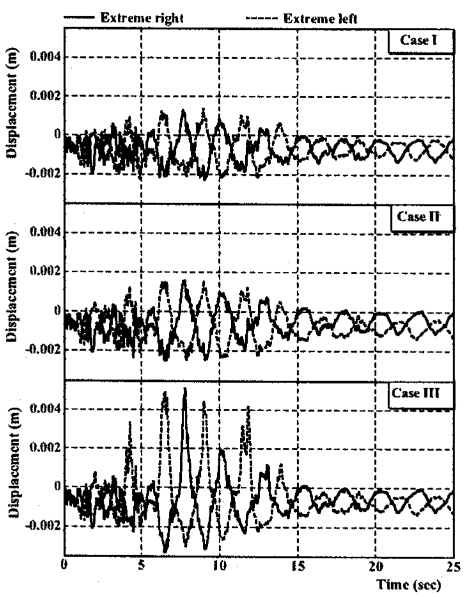


Fig. 18 Vertical displacement time history at footing base level

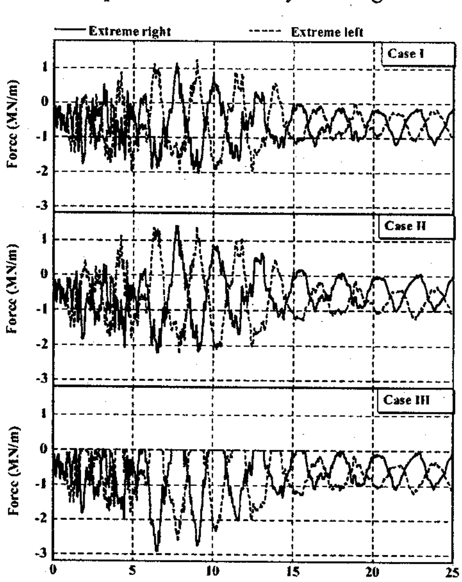


Fig. 19 Vertical force time history at footing base level

It is found that separation of the soil from the structure occurs under large dynamic loads, leading to changes in the predominant vibration of the system. As a result of the decreasing of the soil support at the sidewalls of the foundation by degradation of stiffness due to hysteresis, the stress caused by the structural weight on the bottom soil increases during earthquakes. The bottom soil stresses increase due to soil material nonlinearity consideration (case II) up to 1.15 times that of linear elastic soil (case I), the geometrical nonlinearity by base mat uplift significantly (case III) affects the bottom soil bearing stress that reaches more than 1.5 time of case I. The reduction of the foundation width in contact during uplift induces an increase of the stresses under the foundation. This leads to a larger soil yielding, which itself modifies the uplift behavior of the foundation. The maximum uplift of no more 5 mm clarifies that the tower foundation almost has sufficient structural loading to minimize the uplift. The footing layer has sufficient bearing capacity to prevent foundation excessive settlement.

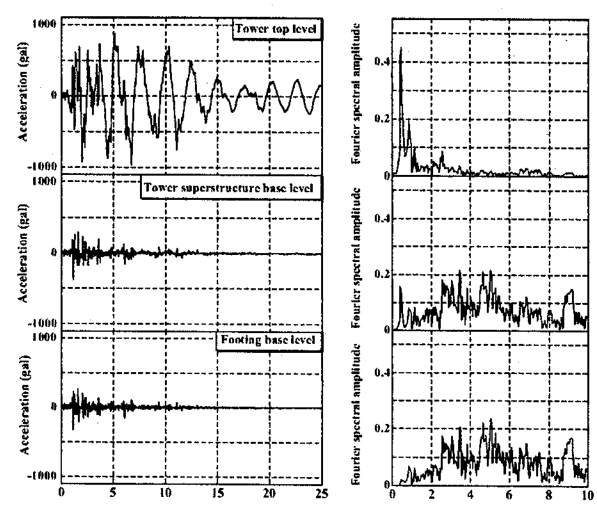


Fig. 20 In-plane acceleration time history and response spectra

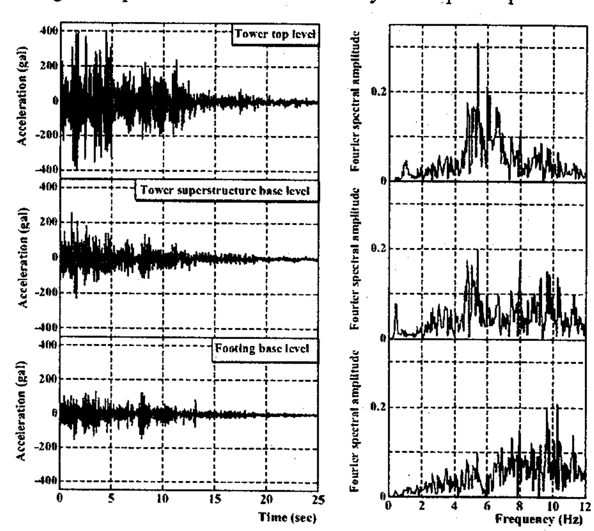


Fig. 21 Vertical acceleration time history and response spectra

From the Fourier spectra study of acceleration response at different level for case III of soil-foundation-superstructure nonlinear interaction, and considering the flexible superstructure relative to substructure, the high rigidity and massive foundation system and stiff footing soil, it can be seen the amplification of different modes through wide frequency range as illustrated in Figs. 20 and 21. The in-plane superstructure base response spectrum is larger than that at footing base under 4.0 Hz of spectral frequency because of amplification induced by flexible superstructure and massive rigid substructure interaction, while at high frequency above 4.0 Hz, the response spectra is slightly attenuated due to inertial interaction. The tower top response spectra is significantly amplified at low frequency range and is almost totally attenuated at high frequency range due to tower superstructure flexibility as seen in Fig. 20. The massive foundation has the effect of amplifying response at more than one frequency. It can be observed the short apparent duration and the relative high frequency (low period) of the vertical motion at the footing base level, the response spectra within the frequency range below 6.0 Hz is slightly amplified at superstructure base level, and dramatically amplified at tower top. On the other hand, It can be observed that in-plane spectrum at tower top contains only the dominant super-structure frequency and all other frequencies, which present in the sub-structure base level or top level have been essentially filtered out, since the response spectrum at high frequency range is attenuated by superstructure flexibility filter of the motion.

7. Conclusions

A finite element model capable of capturing the essential feature of tower seismic response is presented, to study the effects of soil-foundation-superstructure interaction on the dynamic response of a cable-stayed bridge tower supported on spread footing foundation. The model takes into account tower geometry, pier flexibility, the presence of a massive foundation and soil nonlinearity including material nonlinearity of strain dependant (rigidity and damping) and geometrical nonlinearity by base mat uplift. Soil yielding under the foundation and along the embedded depth is modeled through Hardin-Drnevich model to express nonlinear soil characteristics. The contact nonlinearity induced by the uplift of the foundation is integrated in gap model. Radiation damping associated with wave propagation is accounted for implicitly through viscous damping, while the energy dissipation through soil material nonlinearity is explicitly modeled. The interaction effects generated by the normal and tangential resistance of the soil against all active sides of the footing are taken into account. The following conclusions, in the light of the studied case adopted, can be drawn as follows:

(1) The fundamental natural period of the steel tower system is slightly increased for the soil foundation model due to high pier and foundation rigidity and stiff soil.

(2) The massive rigid foundation activates the high frequency translational motion of the input ground motion and generates foundation-rocking responses.

(3) The predominant contribution to the vertical response at footing base comes from the massive foundation rocking rather than from the vertical excitation. The permanent settlement is found to be the less significant.

(4) Through the nonlinear earthquake response analysis by using the proposed method, it is concluded that the foundation uplift slightly alters tower displacement seismic response and reduces the structural member forces.

(5) The soil bearing stress beneath the footing base is significantly increases due to footing base uplift.

(6) The rocking vibration dominates the lateral bearing stress for the soil along the embedded depth of the tower pier.

(7) The bearing stress under the foundation should be checked considering both soil material and geometrical nonlinearities with an appropriate factor of safety to avoid catastrophic foundation failure.

(8) The tower foundation almost has sufficient structural loading to minimize the uplift. The footing layer has sufficient bearing capacity to prevent foundation excessive settlement.

(9) For more general and versatile conclusions different input excitations and different ground conditions at bridge construction site should be considered.

References

1) Hayashikawa T. and Abdel Raheem S. E., Nonlinear dynamic behavior of steel tower of cable-stayed bridges with passive energy dissipation system, *Journal of Structural Engineering, JSCE*, Vol. 48A, pp. 863-874, 2002.
2) Ali, H. M. and Abdel-Ghaffar, A. M., Modeling the nonlinear seismic behavior of cable-stayed bridges with passive control bearings, *Computers & Structures,* Vol. 54, No. 3, pp. 461-492, 1995.
3) Committee of Earthquake Engineering, *The 1995 Hyogoken-Nanbu Earthquake, Investigation into Damage to Civil Engineering Structures, Japan Society of Civil Engineers*, June, 1996.
4) Japan Road Association, *Specification for Highway Bridges - Part V Seismic Design*, chapter 1-8, 1996.

5) Japan Road Association, *Reference for Highway Bridge Design, Specification for Highway Bridges-Part IV Substructures*, chapter 7-9, 1996.
6) Vlassis A. G. and Spyrakos C. C., Seismically isolated bridge piers on shallow soil stratum with soil structure interaction, *Computers & Structures,* Vol. 79, pp. 2847-2861, 2001.
7) Trifunac, M. D. and Todorovska, M. I., Nonlinear soil response – 1994 Northridge, California, earthquake, *Journal of Geotechnical Engineering,* Vol. 122, No. 9, pp. 725 - 735, 1996.
8) Megawati, K., Higashihara, H. and Koketsu, K., Derivation of near-source ground motions of the 1995 Kobe (Hyogo-ken Nanbu) earthquake from vibration records of the Akashi Kaikyo bridge and its implications, *Engineering Structures,* Vol. 23, pp. 1256 - 1268, 2001.
9) Harada T., Kubo K. and Katayama T., Dynamic soil-structure interaction analysis by continuum formulation method, *Report of the Institute of Industrial Science, University of Tokyo,* Vol. 29, No. 5, pp. 139-194, 1981.
10) Ganev T., Yamazaki F. and Katayama T., Observation and numerical analysis of soil-structure interaction of reinforced concrete tower, *Earthquake Engineering and Structural Dynamics,* Vol. 24, pp. 491-503, 1995.
11) Abdel Raheem S. E., Hayashikawa T. and Hashimoto I., Study on foundation flexibility effects on steel tower seismic response of cable-stayed bridges under great earthquake ground motion, *Journal of Construction Steel,* Japan Society of Steel Construction, Vol. 10, pp. 349-354, 2002.
12) Chaojin, X. and Spyrakos C. C., Seismic analysis of towers including foundation uplift, *Engineering Structures*, Vol. 18, No. 4, pp. 271-278, 1996.
13) Spyrakos C. C., Soil-structure-water interaction of intake-outlet towers allowed to uplift, *Soil Dynamics and Earthquake Engineering*, Vol. 16, pp. 151-159, 1997.
14) Ahn, K., and Gould, P.L., Interactive base-isolation foundation system: I. Formulation, *Journal of Engineering Mechanics, ASCE*, Vol. 118, No. 10, pp. 2048-2058, 1992.
15) Gazetas, G., and Dobry, R., Horizontal response of piles in layered soils, *Journal of the Geotechnical Engineering Division, ASCE,* Vol. 110, No. 1, pp. 20-40, 1984.
16) Kobayashi, T., Yoshikawa, K., Takaoka, E., Nakazawa, M. and Shikama, Y., Time history nonlinear earthquake response analysis considering materials and geometrical nonlinearity, *Nuclear Engineering and Design,* Vol. 212, pp. 145-154, 2002.
17) Kawashima, K. and Hosoiri K., Effect of nonlinear rocking response of direct foundations on the hysteretic behavior of bridges, *Journal of Structural Mechanics and Earthquake Engineering, JSCE,* No. 703/I-59, pp. 97 – 111, April 2002.
18) Chen, C. N., Efficient and reliable solutions of static and dynamic nonlinear structural mechanics problems by an integrated numerical approach using DQFEM and direct time integration with accelerated equilibrium iteration schemes, *Applied Mathematical Modelling,* Vol. 24 pp. 637-655, 2000.
19) Abdel Raheem S. E. and Hayashikawa T., Construction imperfections effects on steel tower dynamic response of cable-stayed bridge under great earthquake, *Journal of Construction Steel, JSSC,* Japan, Vol. 10, pp. 355-362, 2002.
20) Nonaka, T. and Ali, A., Dynamic response of half-through steel arch bridge using fiber model, *Journal of Bridge Engineering, ASCE,* Vol. 6, No. 6, pp. 482-488, 2001.
21) Karoumi, R., Some modeling aspects in the nonlinear finite element analysis of cable supported bridges, *Computers & Structures,* Vol. 71, pp. 397-412, 1999.
22) Hardin, B. O. and Drnevich, V. P., Shear modulus and damping in soils: Design equations and curves, *Journal of the Soil Mechanics and Foundation Division, ASCE,* Vol. 98, No. 7, pp. 667-692, 1972.
23) DINAS (Dynamic INteraction problem Analysis System), *user's manual-version 7.0,* CRC Solutions, Japan.

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