NONLINEAR SEISMIC RESPONSE OF SOIL-Foundation-STRUCTURE INTERACTION MODEL OF CABLE-STAYED BRIDGES TOWER

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SUMMARY

The problem of soil structure interaction in the seismic analysis and design of long-span bridges in seismically active regions is potentially important to highway facilities design. A quite general nonlinear dynamic soil structure interaction analysis methodology is developed to estimate seismic response characteristics of cable-stayed bridges tower, in which the effect of the soil flexibility underneath the foundation and its associated geometric and hysteretic damping are included. Due to rocking response of a spread foundation under a strong near-field ground motion that may result in the foundation uplift and the yield of the underlying soils, both of strain-dependent material nonlinearity and geometrical nonlinearity by base mat uplift are considered. The soil parameters values that are considered in the design process are used to observe the effect of soil nonlinearities upon the tower-pier seismic response. Numerical results indicate that force response quantities of the physical sub-structure stiffness tower model can be reduced by considering soil foundation interaction and different soil nonlinearities. 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. The spectrum amplitude at tower top contains only the dominant flexible super-structure frequency and all other frequencies, which present in the massive and rigid sub-structure base level, have been essentially filtered out.

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 (Hayashikawa [1], Ali [2] and Committee of Earthquake Engineering [3]). The Hyogoken-Nanbu earthquake of 17th January 1995, led to an increased awareness concerning the response of highway

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bridges subjected to earthquake ground motions, the ductility design and dynamic analyses have been reconsidered by Japan Road Association [4, 5]. The necessity has arisen to develop more efficient analysis procedures that can lead to a thorough understanding and a realistic prediction of the 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 (Committee of Earthquake Engineering [3], Valssis [6], Trifunac [7] and Megawati [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.

Soil-foundation-structure interaction could be classified into: Kinematic interaction, which 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 (Harada [9] and Ganev [10]); inertial soil-structure interaction, which 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 (Abdel Raheem [11], Chaojin [12] and Spyarakos [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 (Ahn [14], Gazetas [15], Kobayashi [16] and Abdel Raheem [17, 18]).

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 (Kawashima [19]). 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.
FINITE ELEMENT ANALYSIS 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 (Chen [20]) can be obtained as:

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

where $[M]$, $[C]$, and $[K]^{t+\Delta t}$ are the system mass, damping and tangent stiffness matrices at time $t+\Delta t$. $\ddot{u}$, $\dot{u}$ and $\Delta u$ are the accelerations, velocities, and incremental displacements vector at time $t+\Delta t$, respectively, $\{F\}^{t+\Delta t} - \{F\}^{t}$ is the unbalanced force vector. It can be noticed that the dynamic equilibrium equation of motion takes into consideration the different sources of nonlinearities (geometrical and material nonlinearities), which affect the calculation of the tangent stiffness and internal forces. The implicit Newmark’s 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.

The tower structure damping mechanism is adapted to the viscous damping of Rayleigh’s damping type with damping coefficient equal to 2% for steel material of tower superstructure and 10% for concrete material of embedded pier substructure 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 place 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.

In the dynamic response analysis, the seismic motion by an inland direct strike type earthquake that was recorded during Hyogokken-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 (Committee of Earthquake Engineering [3] and Japan Road Association [4,5]) 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). The kinematic interaction can be modeled by a transfer function, which can be used to modify a free-field ground motion so that an effective input motion for a soil-structure system can be obtained. A simple model based on analytical modeling of rigid foundation is adopted, where the effective seismic motion (ESM) is obtained starting from the free-field motion by an approximate analytical solution (Harada [9] and Ganev [10]) for embedded foundation with its base resting on a stiffer layer.

MODELING OF TOWER STRUCTURE

The tower of a 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 is developed for the tower characterization; the element incorporates both geometric and material nonlinearities. 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 plastic region strain hardening is 0.01. The nonlinearity of cable stays is idealized by using the equivalent modulus approach. 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 [21] as:

$$E_{eq} = E \left/ \left(1 + \frac{EA(wL)^2}{12T^3}\right) \right.$$  \hspace{1cm} (2)

where $E$ is the material modulus of elasticity, $L$ is the cable horizontal projected length, $w$ 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 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 of different dimensions along the tower different parts as shown in Table 1.

**MODELING OF SOIL FOUNDATION INTERACTION**

The soil structure interaction is simulated with nonlinear springs and dashpots system along the pier embedded depth, 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 stiffness of supporting soil is idealized by Winkler spring model. The spring constants in both bridge axis directions are calculated based on foundation geometry and soil profile underneath and along embedded depth of foundation as specified in Japanese Highway Specification [4, 5]. Soil properties from the Standard Penetration Test data and logs of borehole at tower site, as shown in Fig. 4, are used to determine the coefficients of vertical and horizontal subgrade reaction. The subgrade reaction coefficients are obtained from the ground stiffness corresponding to the deformation caused in the ground during an earthquake.
Table 1 Cross section dimension of different tower region

<table>
<thead>
<tr>
<th>Tower parts</th>
<th>C. S. Dim. (Cm)</th>
<th>Outer dimension</th>
<th>Stiffener dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>t₁</td>
</tr>
<tr>
<td>I</td>
<td>240</td>
<td>350</td>
<td>2.2</td>
</tr>
<tr>
<td>II</td>
<td>240</td>
<td>350</td>
<td>2.2</td>
</tr>
<tr>
<td>III</td>
<td>240</td>
<td>350</td>
<td>2.2</td>
</tr>
<tr>
<td>IV</td>
<td>270</td>
<td>350</td>
<td>2.2</td>
</tr>
</tbody>
</table>

**Soil nonlinearity idealization**

One of the most important factors in the analysis of soil-foundation interactive behavior is the nonlinear constitutive laws for the soil (material nonlinearity). In this study, Hardin-Drnevich model is proposed to represent the soil material nonlinearity that is often used for its capacity to trace the degradation of stiffness (Hardin [22]). The parameters used to a skeleton curve and family of hysteresis stress-strain curves are shown in Fig. 5. The skeleton curve is expressed as
\[ \tau = G_0 \frac{\gamma}{1 + |\gamma|} , \quad \gamma = \frac{\tau_{\text{max}}}{G_0} \]  \hspace{1cm} (3)

Where \( G_0 \) is the initial shear modulus; \( \tau \) is the generalized soil shear stress, \( \tau_{\text{max}} \) is the shear stress at failure, \( \gamma \) is the reference strain, and \( \gamma \) is the generalized strain. The hysteretic curve can be constructed by applying Masing’s rule and is given as

\[ \tau \pm \tau_m = G_0 \left( \frac{\gamma \pm \gamma_m}{1 + \left| \frac{\gamma \pm \gamma_m}{2 \gamma_r} \right|} \right) \]  \hspace{1cm} (4)

\( \tau_m \) and \( \gamma_m \) indicate the coordinates of the origin of the curve, that is, the point of most recent load reversal. The hysteresis curve is the same in shape as the skeleton curve but enlarged twice. 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. 6; soil exhibits nonlinear nature even at small strains. The shear modulus \((G)\) can be described as

\[ \frac{G}{G_0} = 1 \left( 1 + \frac{\gamma_m}{\gamma_r} \right) \]  \hspace{1cm} (5)

The spring coefficients are computed by the method suggested in Specification for Highway Bridges issued by Japan Road Association. 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. The general concept of this model is shown in Fig. 3.

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. 7. 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.

**Soil damping idealization**

The hysteretic damping characteristic of the soil resulting from the deformations produced by interaction with the pier is represented by nonlinear viscous dashpots. The damping ratio of the soil dashpots strain–dependent material nonlinearity is described by a simple relationship between the shear modulus and damping as shown in Fig. 8.
The material-damping ratio is defined as

\[ h = \frac{C_m}{C_r} \quad , \quad C_r = 2 \left( \frac{k}{m} \right)^{0.5} \]  

in which \( C_m \) is the coefficient of material damping, \( C_r \) is the coefficient of material critical damping, \( k \) 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

\[ C_{soil} = 2 h_{\max} \left( 1 - \frac{k}{k_0} \right) \left( \frac{k}{m} \right)^{0.5} \]  

The soil radiation-damping characteristic is represented through an approximation of a para-axial boundary, where viscous dampers can be used to represent a suitable transmitting boundary for many applications involving both dilatational waves and shear waves. 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. Based on this assumption, the coefficient of viscous dashpot that will absorb the energy of the waves originating at soil-pier interface is evaluated.

![Fig. 7 Soil layers changes in the direction of depth](image)

![Fig. 8 Hysteretic damping of soil](image)

![Fig. 9 One-dimensional radiation damping model](image)
NUMERICAL ANALYSIS AND DISCUSSION

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 effective ground motion acceleration at the base of the soil deposit: the following three cases of soil idealization are analyzed:

Case I: Linear and elastic soil model with equivalent viscous damping of soil.
Case II: Strain-dependant material nonlinearity through Harden-Drnevich soil model.
Case III: 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. 10. The moment and axial 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. 11 and 12. 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. 12. 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 is more conservative than an analysis with fixed base idealization.

![Fig. 10 Acceleration time history at tower top](image1)

![Fig. 11 Moment time history at tower superstructure base](image2)
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 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. 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. 13. The superstructure base displacement could clarify the soil effects on substructure response, 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 model.

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. 14 and 15. 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. 12 Vertical force time history at tower superstructure base
Fig. 13 Foundation sliding time history at footing base level
Fig. 14 Soil in-plane hysteresis along the embedded depth
Fig. 15 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. 16 and 17. 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 times 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.

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. 18 and 19. 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.
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.

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:

The 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. 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. The soil bearing stress beneath the footing base is significantly increases due to footing base uplift. The rocking vibration dominates the lateral bearing stress for the soil along the embedded depth of the tower pier. 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. The tower foundation almost has
sufficient structural loading to minimize the uplift. The footing layer has sufficient bearing capacity to prevent foundation excessive settlement. For more general and versatile conclusions different input excitations and different ground conditions at bridge construction site should be considered.

REFERENCES