Effect of Inertial and Kinematic Interaction on Seismic Behavior of Various Types of Structures

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1. Introduction

It has been clarified that the dynamic behavior of a bridge pier with deep foundations is affected by both inertial force of structure (inertial interaction) and by ground deformation (kinematic interaction). Kinematic interaction, however, has not been necessarily taken into consideration in a design procedure of Japanese design standard [1], if a natural period of a layered surface soil is under 0.5sec. It is partly because deformation of such a stiff soil is regarded as small enough to be neglected for assessing foundations' behavior. Consequently, structures are designed to ensure damage might be suppressed to the small extent by reducing an inertial force through use of base isolations. Nonetheless, the large deformations and momentum responses of foundations constructed on the good conditioned soil might take place, depending on the layered soil properties and its nonlinear behavior. It follows that the severe damage will occur at foundations and structures, if the soil's nonlinear characteristic does not properly taken into consideration. Since the nonlinear behavior of free-field soil is very complex, it would be difficult to estimate kinematic interaction in a good accuracy only by using one condition such as the natural period of the surface soil.

Consequently, this research investigates influence of inertial and kinematic interactions on the behavior of a bridge pier, a viaduct and an isolated bridge in a good soil condition through static and dynamic analyses. It was assumed that those structures had pile foundations and were constructed in a good soil, where deformation could be regarded as negligible according to the design standard. The frame model, representing the behaviors of free-field soil, piles and surrounding soil and superstructures simultaneously, was employed for characterizing assumed structures, by which both inertial and kinematic interactions were taken into consideration. Upon maximum distributions and time histories of the maximum bending moments with regard to superstructures obtained from simulations, structural responses were investigated in view of effects of inertial and kinematic interactions.

2. Evaluation of kinematic interaction by using the displacement response method

In this section, effects of the free-field soil characters on behavior of structures are investigated in a static manner, namely, by a displacement response method. This method has already been widely used in seismic design of railway structures whose dynamic behavior is simple enough to be characterized by a single-degree-of-freedom system.
by introducing nonlinear springs in between. The interaction between surface soil and piles were expressed in a simple single-degree-of-freedom model. It be noted that the obtained response excludes the effect of kinematic interaction. The seismic responses of large section piers and cast-in-place piles were selected for numerical simulation. The seismic responses of the structure were calculated in both longitudinal and transverse directions to the track. A rigid frame viaduct was chosen as another representative structure, and its seismic response in the transverse direction was also examined. The diameter and length of the pile were 1.0m and 19m, respectively. Figure 1 illustrates the geometry of the three assumed structures in this study. As mentioned above, they were designed so that their dynamic behavior could be accurately expressed in a simple single-degree-of-freedom model. Table 1 shows the ground soil layer properties for each structure, with a natural period of 0.48 s. According to the seismic design code for railway structures, this soil was classified as type G3 whose deformation is regarded as negligible. The objective of the simulation is to verify whether such a good soil's displacement (kinematic interaction) affects the response of piles or not.

The structures were represented by space frame, and interaction between surface soil and piles were expressed by introducing nonlinear springs in between.

### 2.1 Target structures

The seismic responses of large section piers and cast-in-place piles were selected for numerical simulation. The seismic responses of the structure were calculated in both longitudinal and transverse directions to the track. A rigid frame viaduct was chosen as another representative structure, and its seismic response in the transverse direction was also examined. The diameter and length of the pile were 1.0m and 19m, respectively. Figure 1 illustrates the geometry of the three assumed structures in this study. As mentioned above, they were designed so that their dynamic behavior could be accurately expressed in a simple single-degree-of-freedom model.

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### 2.2 Procedures for obtaining the maximum response considering inertial and kinematic interactions

#### 2.2.1 Calculation of inertial force (inertial interaction)

Firstly, fundamental features of the selected structures, such as yielding seismic coefficient $k_y$ and corresponding natural period $T$, were obtained from the nonlinear force-displacement curve of the structure. The curve was calculated by nonlinear static loading analysis. In this analysis, the inertial force applied to the structural model was gradually increased, and the corresponding displacement was obtained. The expected response displacement ($\delta_y$) and corresponding seismic coefficient of the structure ($k_y$) due to inertial force were then obtained by applying the given $k_y$ and $T$ to the nonlinear response spectra. The response spectra according to surface ground property were available in the design standard. These successive procedures, commonly used in a seismic design, are summarized in Fig. 2(b). It should be noted that the obtained response excludes the effect of kinematic interaction.

#### 2.2.2 Calculation of soil deformation (kinematic interaction)

Secondly, each surface soil displacement distribution according to depth was calculated by using a nonlinear dynamic analysis. As shown in Fig. 2(c), soil layers were characterized in relation to the soil's mass and spring. The GHE-S model was employed as a nonlinear model of the spring in order to express the behavior of the soil. The L2 earthquake Spectrum II motion assuming the bedrock motion was given to the bottom of the model, and time histories of the soil displacement at every depth were obtained. The motion is defined in the seismic design code for railway structures in Japan, representing the very rare (Level 2) strong inland earthquake (spectrum II) in the bedrock. Although displacement distribution varied over time, the maximum displacement distribution was to be chosen for carrying out the response
analysis in a static manner. In this paper, the distribution of soil layer’s displacement at the moment that relative displacement between pile head and tip takes its maxima is regarded as the inducing displacement to the structures. Figure 2(c) shows the obtained displacement response of the soil deposit. It is found that the soil’s response changes drastically at depths from 7.5m to 12m from bottom of the footing, and maximum displacement at ground surface reached 0.3m.

### 2.2.3 Synthesizing inertial and kinematic interactions

Finally, the overall response of the given structures considering both inertial and kinematic interactions was calculated by combining the response obtained in sections 2.2.1 and 2.2.2. The structure’s total response was obtained by giving an inertial force corresponding to the seismic coefficient \( k_h \) to the superstructure, while applying the soil layer’s displacement distribution to the foundation via nonlinear springs. The schematic explanation of the procedure is shown in Fig. 2(a). In this method, it is assumed that the responses due to inertial force and soil deformation take their maxima simultaneously. Such an event might occur if the structure has a large nonlinear response. Following this, the response due to the soil displacement was simply added to that due to inertial interaction to obtain the total response of the structure.

### 2.3 Results and discussions

Table 2 shows the seismic properties of the analyzed structures obtained from static nonlinear analysis. Table 2 also includes the calculated seismic responses of the superstructures using nonlinear response spectra. In addition, Figures 3, 4 and 5 show the maximum bending moment, shear force and curvature distributions of the piles according to the depth. In these figures, responses due to inertial force (Case 1) are compared with those considering both inertial and kinematic interactions (Case 2). It was found from these figures that two different peaks at depths of around 5m and 12m exist in the

| Table 2 Properties and seismic responses obtained for the assumed structures |
|------------------|---|---|---|
| **Weight of superstructure** (kN) | Unit | Pier (transverse) | Pier (longitudinal) |
| 8030 | 8750 | 1885 |
| **Seismic coefficient** | at yielding | 0.68 | 0.41 | 0.60 |
| at maximum response | 0.78 | 0.46 | 0.75 |
| **Displacement** (mm) | at yielding | 105.0 | 154.0 | 70.0 |
| at maximum response | 322.0 | 491.0 | 187.0 |
| **Equivalent natural period of structure** (s) | | 0.95 | 1.02 | 0.68 |
| **Natural period of surface soil** (s) | | 0.48 |
distribution of the pile’s maximum responses. The causes of these peaks are closely investigated below through separate discussion of the responses for areas down to 10m from the bottom of the footing and those deeper than 10m.

2.3.1 Piles responses (in the 10m to the bottom of the footing)

Figure 3 shows the maximum responses of the piles in the transverse direction to the track. This figure reveals that responses considering both inertial force and soil deformation (Case 2) were almost identical to those only due to inertial force (Case 1). Since the inertial force was obtained by multiplying the weight by the response seismic coefficient of the superstructure, large mass and seismic coefficient shown in Table 2 produced a relatively larger inertial force which dominates the overall momentum and shear force responses. It follows that the soil deformation was less effective against the total response in this case. In case of maximum curvature distribution, however, significantly large response was found in Fig. 3 due to the yielding of the pile.

The maximum responses of the same pier in the longitudinal direction to the track (Fig. 4), on the contrary, moment distribution of Case 1 was relatively smaller than that of Case 2. Although the properties of the surface ground and piles in Fig. 4 were the same as those of Fig. 3, the response seismic coefficient in the longitudinal direction to the track was much smaller than that in the transverse direction (see Table 2). Since the section of the pier was rectangular as illustrated in Fig. 1(a) and Fig. 1(b), the section areas, moment of inertias and resulting seismic coefficients were different according to the direction of motion. In fact, the inertial force of superstructure in the longitudinal direction was reduced by 40% compared to that in the transverse direction. It consequently follows that, unlike the structure discussed in Fig. 3, both inertial and kinematic interactions affected the total response of the pile in Fig. 4.

Figure 5 shows the pile’s maximum responses for rigid frame viaduct in transverse direction to the track. The figure illustrates that the maximum moment distribution in Case 2 was totally different from that in Case 1, as kinematic interaction is relatively dominant in the total response. It is quite similar to Fig. 4 because weight of the rigid frame viaduct was 80% less than that of the pier structure (see Table 2), and inertial force was smaller accordingly.

It is concluded from the discussions above that even for the same surface ground property and earthquake cause different response of foundation in accordance with the characteristics of the superstructure and direction of motion. It consequently follows that consideration of both inertial and kinematic interactions is indispensable for obtaining the foundations’ response in a good accuracy.

2.3.2 Piles responses (at positions deeper than 10m)

Figures 3, 4 and 5 show that the effect of inertial force (Case 1) is negligible at depths beyond 10m, unlike for shallow area discussed in section 2.3.1. These results agree with the past research that response of piles is affected by an inertial interaction only in areas close to the ground surface, down to approximately 10m in this simulation. However, in Case 2, significantly large moments and shear forces were observed at depths of around 10 to 12 m in all simulations. Since the inertial force was less effective against the response of pile in deeper areas, those responses were caused by the deformation of the surface soil. In fact, locations of these peaks correspond to the depths where large discontinuity in surface soil’s property was observed (see Fig. 2(c)). It should be recalled that the deformation of the assumed free-field soil is regarded as negligible, according to the design standard. Nevertheless, its displacement may cause significant foundation response depending on the distribution of shear velocity or impedance of layered soil. It consequently follows that not only the effect of inertial force but also kinematic interaction should be properly characterized and evaluated so as to avoid the unexpected damage to the foundations.
3. Evaluation of kinematic interaction by dynamic analysis

In the previous chapter, simple structures whose vibration characteristics could be modeled as a single-degree-of-freedom system were investigated by means of static-based analyses. These simulations revealed the both inertial and kinematic interactions affect the response of the foundation even if the soil condition is regarded as good. In case of more complicated structures such as isolated bridges, however, dynamic analyses should be conducted in order to comprehend the seismic response including the coupling of multiple vibration modes or nonlinear behaviors. In this section, another simulation was therefore conducted for an isolated bridge, in order to verify whether the results obtained from the static-based analysis could be applicable to more complicated dynamic systems.

3.1 Target structure with isolation bearings

The PRC girder (span=29.2m) supported by the RC column and cast-in-place piles (diameter = 1000mm, length=19m) was adopted as the bridge model for this dynamic analysis. The geometry of the structure is shown in Fig. 6. The pile head were rigidly connected to the footing. In the following simulations, two different types of supports installed between the girder and the pier were assumed: a fixed support and an isolation bearing. The fixed support bearing directly transmits the inertial force of the girder to the pier and the foundation, whereas the isolation bearing moderates it. That is to say, soil deformation rather than inertial force might dominate the pile’s response of isolated structure.

These structures and foundations were embedded in the layered soil deposit with properties as listed in Table 3. It is estimated from the table that large deformation of free-filed soils and piles might take place due to the shear velocity discontinuity at depths of around 12m from the bottom of the foundation. According to the 1/4 wavelength law designated in the Japanese seismic design standard, however, this soil condition was classified as a good soil condition, in which the effect of soil deformation on piles is regarded as negligible.

3.2 Overview of the dynamic analysis

3.2.1 Numerical model considering soil deformation

The comprehensive frame model shown in Fig. 7 was employed to evaluate the response of the structure, foundation and free-field soil. In the model, the superstructure, its foundation and soil behavior in the vicinity of piles were characterized by a combination of lumped mass and nonlinear beams-springs. The key feature of the model is the bunch of layered lumped masses and nonlinear shear springs, representing the behavior of a free-filed soil deposit. This free-field soil model was attached to the piles by nonlinear horizontal springs, representing the behavior of the soil’s horizontal subgrade reaction around piles.

This modeling is analogous to the Penzien-type model [2] except that no preliminary simulation is needed to obtain the deformation, velocity and acceleration of free-field soil. Figure 7 shows that the earthquake motion induced in the seismic bedrock deforms the free-field layered soil, and also excites the bridge and its foundation. This model is both precise and economical from the viewpoint that the inertial and kinematic interactions as well as behavior of free-field soils can be evaluated simultaneously. Details of the modeling are mentioned in the following sections.
**3.2.2 Models for piers, piles and bearings**

The nonlinear behavior of the pier and piles was represented by the moment-curvature relationship according to the geometry and material property of each column section. The nonlinear characters for piers and piles are shown in Table 4. The skeleton curve was represented by a tri-linear curve, including the cracking of cover concrete (Mc), yielding of reinforced bars (Mf) and the maximum resistance moment (Mmax). In modelling the piles, termination of the reinforcement was not taken into consideration, and behavior of multiple piles in the transverse direction to the track was summarized as a single beam model. A modified Takeda’s rule was then employed for both the pier and piles to represent their hysteretic behavior.

Table 5 shows the geometry and material properties of the isolation bearings installed between the girder and the pier. It is assumed in the simulation that the existence of the isolation bearings installed between the girder and the pier is characterized by an elasto-plastic behavior of the free-field’s shear springs were represented by the following Ramberg-Osgood model and Masing’s rule [3], such that

\[
\gamma = \left( \frac{\tau}{G_{\text{max}}} \right)^{1+\alpha} \left( \frac{\tau}{\tau_f} \right)^{\beta}
\]

(1)

\[
G_{\text{max}} = \rho \cdot V_s^2, \quad \alpha = 2\beta - 1, \quad \beta = \frac{2 \pi h_{\text{max}}}{2 \pi h_{\text{max}} - \tau_f}, \quad \tau_f = G_{\text{max}} \cdot \gamma_f
\]

(2)

where \( \tau \) is shear stress, \( \gamma \) is shear strain, \( \rho \) is soil density, \( V_s \) is shear velocity (m/s), \( h_{\text{max}} \) is maximum damping ratio and \( \gamma_f \) is reference strain. Since the soil deposit assumed for the simulation consists of bunch of clay (See Table 3), \( h_{\text{max}} \) and \( \gamma_f \) were given as 0.20 and 0.001 respectively.

**3.2.3 Models for ground resistance and the layered free-field soil deposit**

The horizontal ground resistance property in the vicinity of the piles was characterized by an elasto-plastic model. The hysteretic model was classified according to the depth and the characteristic length of the piles: \( \beta = \frac{2h_{\text{max}}}{4EJ} \). In addition, skin frictions on the pile and vertical subgrade reaction at the pile tip were characterized by a bi-linear model. The stiffness and resisting force for these models were determined by the design standard of foundation for railway structures, which considers the group pile effect.

The free-field soil deposits were modeled as a soil pillar, with the ground equally divided into the vertical ground depths of 1m in the vertical direction. The section area of the each pillar was 100 times as large as that of the footing, i.e. large enough to prevent the influence of structure-foundation behavior on free-filed response. This section area size was determined through preliminary numerical simulations. Each soil pillar was then characterized by a group of lumped masses and nonlinear shear springs, and each mass was connected to the piles by nonlinear soil springs. The hysteretic behavior of the free-field’s shear springs were represented by the following Ramberg-Osgood model and Masing’s rule [3], such that

\[
\gamma = \left( \frac{\tau}{G_{\text{max}}} \right)^{1+\alpha} \left( \frac{\tau}{\tau_f} \right)^{\beta}
\]

(1)

\[
G_{\text{max}} = \rho \cdot V_s^2, \quad \alpha = 2\beta - 1, \quad \beta = \frac{2 \pi h_{\text{max}}}{2 \pi h_{\text{max}} - \tau_f}, \quad \tau_f = G_{\text{max}} \cdot \gamma_f
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**3.2.4 Input motion and analysis method**

For input motion, the Level 2 spectrum II motion assumed in the seismic bedrock was applied to the bottom of the model. Since the motion is determined on a free surface, a dashpot depicted in Fig. 7 was introduced to dissipate the downward wave.

Numerical simulations were carried out by employing the Newmark’s β method (\( \beta = 1/4 \)). Rayleigh’s proportional damping was used in the simulations. The damping factors were determined by the strain energy proportional damping method and elements’ damping factors. Damping factors of the structural elements assumed were as follows: 0% for isolation bearings and footing, 2% for bridge pier, 3% for piles, 2% for free-filed soil layers, and 20% for soils in the vicinity of piles. The numerical simulations were carried out in parallel to and in the longitudinal direction to the track. Liquefaction was not taken into consideration in the simulations.
3.2.5 Evaluation of kinematic and inertial interactions

The kinematic as well as inertial interactions affecting the resulting dynamic responses of the pier and piles cannot be evaluated independently due to the aforementioned nonlinearity. However, responses due to inertial interaction should be distinguished from kinematic influences to assess the effect of introducing isolation bearings. In order to overcome the inconsistency, following three simulation models were prepared, by which the contribution of kinematic with inertial interactions on overall response of pier and piles were approximately distinguished.

Case 1: Model with fixed supports (base model)
Case 2: Model with isolation bearings (isolation model)
Case 3: Model without considering weight of the pier and girders (soil model)

The responses with regard to the pier and piles including both inertial and kinematic interactions were evaluated in the Case 1 and Case 2 simulations. The Case 3 model was used to calculate the response excluding inertial interaction. The effects of kinematic and inertial interactions were then approximately distinguished by combining the results of the Case 1, 2 and 3. That is to say, the moment response of the pile due to the inertial interaction at time=$t$ and depth=$z$, or $M_i(t, z)$, was estimated as follows [4].

$$M_i(t, z) = M_k(t, z) - M_k(t, z),$$

(3)

where, $M_k(t, z)$ is an overall moment obtained from Case 1 or 2 models, and $M_i(t, z)$ is a moment due to soil deformation in Case 3.

3.3 Results of simulation and discussion

3.3.1 Pier and girder responses

According to preliminary modal analyses, natural periods of the base model and isolation model were 0.76s and 1.16s respectively. Yielding stiffness was used to compose the total stiffness matrices for the modal analysis. These periods correspond to the primary modes in which the swaying motion of the superstructure and foundation is dominant. Figure 8 shows the absolute response spectra under 5% damping with respect to the bedrock and surface accelerations. The surface acceleration time history was calculated using the Case 3 model. It is estimated from the natural periods and the bedrock spectrum that the maximum pier acceleration response can be reduced by approximately 37% by using isolation bearings. However, its reduction effect might be different in a dynamic analysis due to the nonlinear behaviors of pier and piles.

Figure 9 illustrates the moment versus curvature responses at the bottom of the pier with and without isolation bearings. The figure shows that maximum curvature responses reduced from 0.040 (1/m) to 0.0025 (1/m) due to these isolators. Figure 10 compares the displacement of girders relative to the ground with and without isolators. It is found that the girder’s maximum

Fig. 8 Absolute acceleration response spectra at bedrock and surface (5% damping)

Fig. 9 Moment-curvature responses at bottom of pier with and without isolation bearings

Fig. 10 Displacements of girder relative to the ground

Fig. 11 Total hysteresis of three isolators
responses. Bearing contributed to the reduction of pier and girder responses.

3.3.2 Contribution of kinematic and inertial interactions to pile moments

Figure 12(a) shows the maximum strain distribution of the free-field soil deposit obtained in Case 3. In the figure, the depth=0m corresponds to the bottom level of the footing. It was found that the large deformation of piles might take place at depths at around 12m below the footing, where the discontinuity in shear strain is relatively larger.

Figure 12(b) illustrates the maximum distributions of pile’s moments with and without isolation bearings. Figure 12(c) and Figure 12(d) are inertial and kinematic interactions affecting the total moment responses shown in Fig. 12(b). The total moments, Fig. 12(b), were obtained by simply plotting the maximum moment of pile at all depths from Case 1 and Case 2 simulations. In the same way, the pile response distribution due to soil deformation shown in Fig. 12(d), was calculated from Case 3. Based on these distributions, moments due to inertial interaction were evaluated as illustrated in Fig. 12(c) by incorporating the equation (3). It should be noted that the times at which maximum pile node moments take place differ in these figures.

It is observed from Fig. 12(c) that the pile moments due to inertial interaction were reduced if the depth was shallower than 10m. It is estimated that the use of an isolation bearing reduced the inertial force of the superstructure transmitting to the foundation, by softly supporting the girder. However, large soil strain observed in Fig. 12(d) significantly contributed to the pile’s response where the depth was 10m or more, regardless of with and without isolation bearings (See Fig. 12(b) and Fig. 12(d)). Inertial interaction had less effect on pile’s response in such a deep zone as demonstrated in both the static analysis and dynamic simulations, say, Case 1 and Case 2 in Fig. 12(b). These results imply that the reduction of inertial force by means of isolation bearings does not necessarily contribute to the reduction of the total momentum response of a pile.

It consequently follows that the deformation of free-field soil and subsequent kinematic interaction should be properly characterized and evaluated in order to avoid underestimating the response of foundations, depending on the distribution of shear velocity or impedance of layered soil. In addition, the natural period of the free-field soil deposit, widely used for seismic design, is not necessarily an appropriate index to estimate whether the kinematic interaction will be dominant or not in the structure. The comprehensive characterization of structures considering superstructure-foundation-soil interactions, such as the frame model used for this study or FEM analysis, are required in order to estimate the response of foundation in a good accuracy, particularly if the shear velocity and/or impedance between soil layers change drastically.

4. Conclusions

In this research, contributions of inertial and kinematic interactions to the behavior of a bridge pier, a viaduct and an isolated bridge in a good soil condition were studied through static and dynamic analyses. It was assumed that those structures have pile foundation and were constructed in a good soil, whose deformation could be regarded as negligible according to the design standard. The results presented in the paper are as follows.

1. The effect of free-field soil deformation (kinematic interaction) cannot necessarily be regarded as neg-
ligible for the evaluation of the foundation response, particularly if superstructure’s inertial force is relatively small compared to the soil deformation.

(2) The response of a pile far from the ground surface, where little inertial force is transmitted, is strongly affected by soil deformation if the given soil property has a large discontinuity in shear velocity distribution. This means that a structure such as an isolated bridge will not necessarily intact after strong motion in view of damage of foundation, even if its inertial force is suppressed to a small extent.

(3) Free-field soil deformation and subsequent kinematic interaction should be properly characterized and evaluated in order to avoid underestimation of the foundation response, depending on the distribution of shear velocity or impedance of layered soil. In addition, the natural period of the free-field soil deposit, widely used for seismic design, is not necessarily an appropriate index to estimate whether the kinematic interaction will be dominant or not to the structure. The comprehensive characterization of structures considering superstructure-foundation-soil interactions, such as the frame model used for this study or FEM analysis, are required in order to accurately estimate the response of foundation, particularly if the impedance between soil layers changes drastically.

References