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SEISMIC PERFORMANCE OF A WIB-ENHANCED PILE FOUNDATION

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SUMMARY

Past strong earthquakes have caused structural collapses and their sever damages. Earthquake engineers have developed seismic design methods and procedures to mitigate them through experiences and introduced innovative ideas. Herein, the authors propose an improvement design method, called Wave Impeding Barriers (WIB), as seismic responses reduction measures, for a pile-supported viaduct foundation. The present WIB consists of a multiple number of soil-cement mixed columns, which are arranged in honeycomb cells shape. Nonlinear behaviors of the Structure-Soil Dynamic Interaction (SSDI) system are of interest so that investigation in time domain by a two-dimensional FEM-BEM technique is carried out. The optimum design is pursued of this measure by parametric studies for better performance of the whole system. Significant responses reduction is demonstrated from the comparison between the whole system without and with the WIB.

INTRODUCTION

Earthquake resisting design has been developed through the seismic damage experiences due to strong motions. Innovative design ideas have been introduced Herein, the authors paid attention to foundation design at soft ground. In most design procedure, seismic force action to superstructure is determined first and then the substructures are designed strong enough to support it. It is of great importance in foundation engineering field to have enhancing design countermeasures for this aim.

Pile foundations are commonly used to support structures at soft sites by transferring axial loads from superstructures to stiffer strata at depth through soft soil. Such deep foundation type should also be suited to resist against horizontal seismic loading. However, a lot of pile foundation damages of highway bridges have been reported from the disastrous earthquakes, notably from 1995 Hansin Earthquake. Those damages are centered at pile heads or tips, and also at presumably liquefied zones in certain depths. They were documented in several reports.

In view of the recent trend of performance design concept, we make use of the idea of controlling the structural behavior during strong input motions. The foundation impedance is an influential factor for the

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natural periods of the whole system. Conventionally, improving horizontal impedance by increasing pile number or increasing individual pile section area is preferred. Those procedures cost much, however. In contrast, installing soil-cement columns can be used to enhance the stiffness of the surrounding soils and eventually for pile foundation. In view of the advantage in cost of soil improvement technique in the neighborhood ground, it is one of the promising methods for increasing horizontal resistance of surface soft layers. Partially improvement of the neighboring soils where the piles are imbedded has been proved to be an effective measure by Takemiya & Shimabuku [1]. In this paper, the honeycomb shaped WIB is proposed for increasing seismic resistance of a viaduct pile foundation. This configured WIB was first developed in the Takemiya Laboratory for traffic-induced vibrations. Takemiya [2] has evaluated the mitigation effect by honeycomb shaped WIB for pile foundations of a high-speed train viaduct. The WIB was also applied for seismic reduction by Takemiya & Chen [3] with linear assumption. However, the countermeasure is expected to get damaged while keeping piles inside honeycomb cells safe in sever earthquake cases. Then nonlinearity is necessary for simulating WIB's behavior to meet the actual performance. This paper improves the simulation of such a SSDI system mainly at the nonlinear analysis of WIB measure.

In this paper, the Taiwan shinkansen viaduct foundations (Figure 1) are taken as an illustrative example to evaluate the effectiveness of the application of honeycomb shaped WIB for seismic response reduction. The proposed countermeasure is easy to be optimized to meet the trend of performance based design requirement.

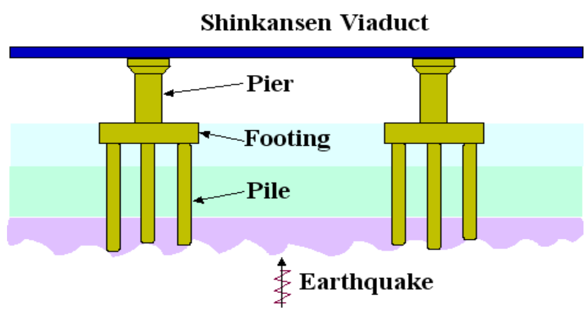


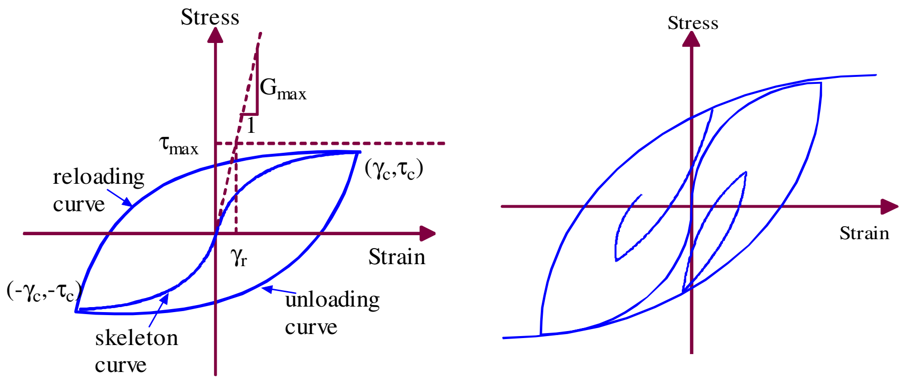
Figure 1. Taiwan shinkansen viaduct foundations

METHODOLOGY OF ANALYSIS

In view of large deformations of structures and soil occurred inevitably in strong earthquake cases, nonlinearity characteristics can not be neglected. In this study, therefore, the nonlinear treatment is conducted to meet the real performance.

Modeling of nonlinearity

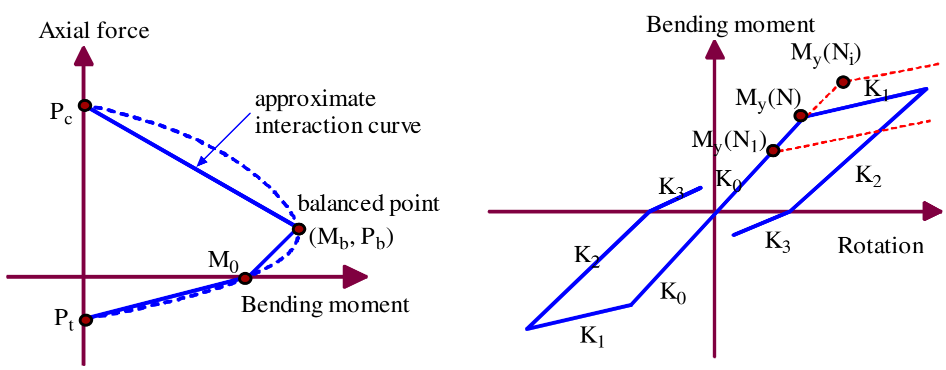
For strong earthquake loading cases, soil behaves heavily nonlinear so that the shear modulus of soil decreases with increasing the shear strain. Such material nonlinear stress-strain relationship of soil is usually obtained from experiments. Hardin and Drnevich [4] proposed an approximate hyperbolic model (H-D) that satisfies the Masing's rule, on basis of the results from a lot of shear tests. The model is now widely accepted for convenience and also used herein to simulate the performance of soils. Further, the technique developed by Takemiya [5] to fulfill the supplementary requirements for irregular earthquake loading case is adopted. Figure 2 characterizes the nonlinear soil model and the extended behavior for earthquake cases.



(a) Nonlinear soil model for symmetric loading (b) Extending Massing's rules for irregular loading

Figure 2. Nonlinear mathematic model for dynamic soil

The inelastic behavior of RC (reinforced concrete) beam elements is represented by the one component model proposed by Giberson [6] by considering both swaying and rotational motions at both ends of each elements. The hysteresis characteristic of RC beams is represented by the Q-hyst model proposed by Saidi [7]. Shimabuku [8] modified the Q-hyst model so as to take account of the relationship between bending moment and axial loading. The yield bending moment is revalued at every calculation step by referring to the bending moment-axial loading diagram, as illustrated in Figure 3.



(a) Yielding Bending moment considering axial load (b) Modified Q-Hyst model

Figure 3. Nonlinear mathematic model for RC beams

The WIB consists of a great number of soil-cement mixed columns and they are arranged in honeycomb shape (see Figure 4). Such columns are expected to work as shear beams according to experiments. In order to simulate the complex configuration in a two dimensional model, the WIB is modeled as several vertical walls connected with each other by diagonal truss elements. The horizontal shear forces of WIB are equivalently replaced by the horizontal internal forces of the crossing truss elements. Therefore, shear forces can be totally substituted by the axial forces of truss elements. Thus, the correspondence relationship between a WIB block and a corresponding truss element is described as

$$A = \frac{S}{2(1+\mu)\sin 2\theta \cos\theta} \tag{1}$$

where A and S denote the section areas of the truss element and the WIB block respectively. θ is the angle of the truss element, and μ is the poisson ratio of WIB.

The nonlinearity of the WIB is expressed by the nonlinear relationship between shear stress and shear strain of the transversal columns of WIB. Consequently, the nonlinearity can be represented by the nonlinear relationship between normal stress and strain of the equivalent truss elements in a two dimensional calculation model. Herein, the truss elements are modeled by a bilinear hysteretic model, the behavior of which will be shown in the response depiction later. The yielding shear strain of the model is taken according to Ramberg-Osgood (R-O) model. By comparing the R-O model with the H-D model with respect to the shear modulus ratio G/G_0 [9], we can obtain the yielding strain approximately as

$$\gamma_y = \frac{7}{3}\gamma_r \tag{2}$$

in which γ_r is the reference strain of H-D model. The WIB columns are expected to get damaged when their shear strains exceed the ultimate strain γ_u, which is set as $\gamma_u = 12\gamma_r$ in this paper according to engineering experience.

Nonlinear calculating method

The computer program for analysis was developed on basis of a hybrid technique of the Finite Element Method (FEM) and Boundary Element Method (BEM). The FEM-BEM hybrid technique utilizes respective advantages of the two discretization methods. The FEM covers flexibly the structure and the near field soil with complicated zone of the model, while the BEM fulfills the infinite boundary condition inherently. Therefore, the deeper stiff half space is included in the BE zone and it is considered as super finite elements. The pier and piles of the foundation are modeled by beam elements, and the near field soil is discretized by isoparametric solid elements. Artificial high damping is imposed at side edge elements of the FE zone for absorbing the outgoing waves.

Since the BEM deals with the far field of a homogenous linear material, a relative large time step may be accepted for the time discretization. On the other hand, the FE zone requires smaller time step for its complicated soft soil. Thus, the BEM time step Δt is divided into *N* number of smaller time step Δt_f for FEM, that is $\Delta t_f = \Delta t / N$. By assembling the stiffness matrix of the BE region to the global system matrix, and by adopting the weighted residual technique, the equations for the coupled FE-BE system at each time step *n* can be written as

$$\begin{bmatrix} KF_{OO} & KF_{OI} \\ KF_{IO} & KF_{II} + \Delta t_f{}^2 \alpha K_{BB} \end{bmatrix} \begin{Bmatrix} \Delta U_O \\ \Delta U_I \end{Bmatrix}^n = \Delta t_f{}^2 \left[\begin{Bmatrix} 0 \\ RB \end{Bmatrix}^n - \begin{Bmatrix} 0 \\ RB1 \end{Bmatrix}^{n-1} \right] + \begin{Bmatrix} RFN_O \\ RFN_I \end{Bmatrix}^{n-1} + \begin{Bmatrix} RF_O \\ RF_I \end{Bmatrix}^{n-2} + \{\Delta R_0\}^n \tag{3}$$

$$[KF] = \left[M_F + \gamma \Delta t_f C_F + \beta \Delta t_f{}^2 K_F \right] \tag{4}$$

$$\{RB\}^n = \alpha F_{BB}^n + (1-\alpha) F_{BB}^{n-1} \tag{5}$$

$$\{RB1\}^{n-1} = (1-\alpha) K_{BB} U_I^{n-1} \tag{6}$$

$$\{RFN\}^{n-1} = \left[M_F - (1-\gamma)\Delta t_f C_F \right] U_F^{n-1} + (\beta - \gamma - 1/2)\Delta t_f{}^2 \{RS\}^{n-1} \tag{7}$$

$$\{RF\}^{n-2} = \left[-M_F + (1-\gamma)\Delta t_f C_F \right] U_F^{n-2} + (-\beta + \gamma - 1/2)\Delta t_f{}^2 \{RS\}^{n-2} \tag{8}$$

$$\{RS\}^{n-1} = \sum_{e=1}^{elements} \int_{V_e} [B_e]^T \{\sigma_e\}^{n-1} \partial V_e \tag{9}$$

$$\{RS\}^{n-2} = \sum_{e=1}^{elements} \int_{V_e} [B_e]^T \{\sigma_e\}^{n-2} \partial V_e \tag{10}$$

The subscript I and F indicate the FE-BE interface nodes and the FE region nodes respectively. ΔU_F is the increment of nodal displacement. M_F, C_F and K_F denotes the mass, damping and stiffness matrixes for FEM zone, and K_{BB} represents the stiffness for BEM region. KF is the equivalent stiffness matrix for the FE region, where α, β and γ are coefficients for numerical integration. The vectors $\{RFN\}^{n-1}$ and $\{RF\}^{n-2}$ represent the initial conditions from the previous two step solutions and the vectors $\{RS\}^{n-1}$ and $\{RS\}^{n-2}$ are the restoring forces corresponding to the converged solutions of the previous time steps *(n-1)* and *(n-2)*. The vector $\{\Delta R_0\}^n$ is the out-of balanced load, which accounts for the difference between the actual nonlinear forces and the assumed linear forces, and is obtained by iterative scheme of modified Newton-Raphson method.

DESCRIPTION OF STUDIED CASES

A Taiwan shinkansen viaduct foundation is dealt with. The span length of the girder is 30 meters between adjacent foundations, which is large enough to neglect the interaction among those supporting foundations. Figure 4 depicts the sectional elevation of the viaduct-pile foundation-soil system. The foundation consists of five piles with diameter 1.8 m each, and it is considered imaginably to be surrounded by honeycomb shaped WIB. The WIB depth is determined by the active length $1/\beta$ of an embedded pile. The active length calculated according to the Japan Road Association [10] is about 9.5 m for a pile with diameter 1.8 m embedded in the studied soil strata. The Japan Highway Technical Center [11] proposed the range from $1/\beta$ to $\pi/2\beta$ for soil improvement depth. In this paper, the WIB depth varies in this range for different cases.

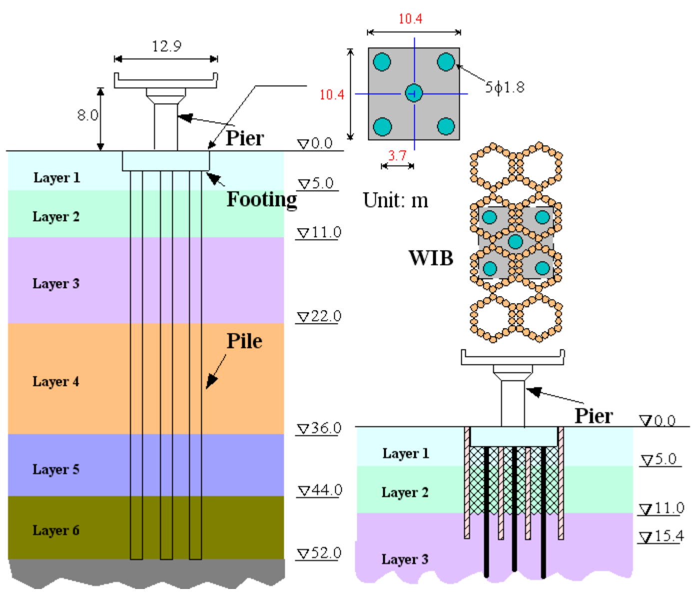
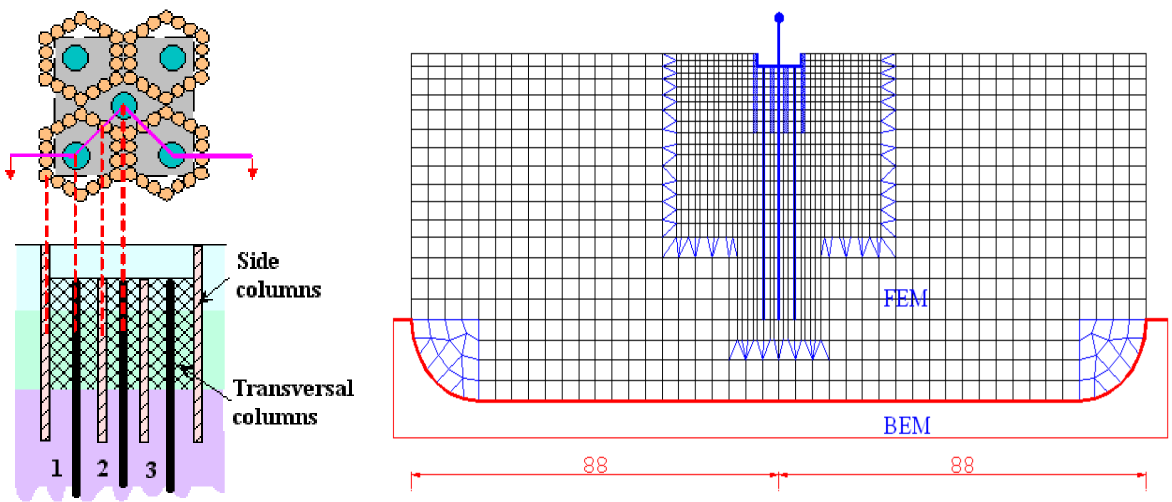


Figure 4. A viaduct foundation surrounded by honeycomb-shaped WIB

Side columns of WIB cells are discretized into solid elements, which work as several walls. Such walls are connected by truss elements as mentioned before. In order to simulate the interaction between structural piles and the WIB in a 2-D model, the honeycomb shaped WIB is simplified as shown in Figure 5 (a) by considering the soil around the piles. Piles are marked in order from left to right to denote the left side pile, the center pile and the right side pile. The FEM-BEM computation model is illustrated in Figure 5 (b), where the nearer area to the piles is meshed much thinner for more precise responses requirement there. For simplicity, the viaduct deck is just regarded as a lumped mass in the 2-D model. The properties of the structures and the layered soil are listed in Table 1 and Table 2. Such soil layers are further divided into thinner sublayers for computation requirements. The subdivision should be less than 1/5 of the propagating wavelength which is concerned for a specified maximum frequency of the loading (Roesset [12]). Herein, 10 Hz is adopted for the maximum frequency.



(a) 2-D WIB

(b) FEM-BEM model

Figure 5. Model of analysis

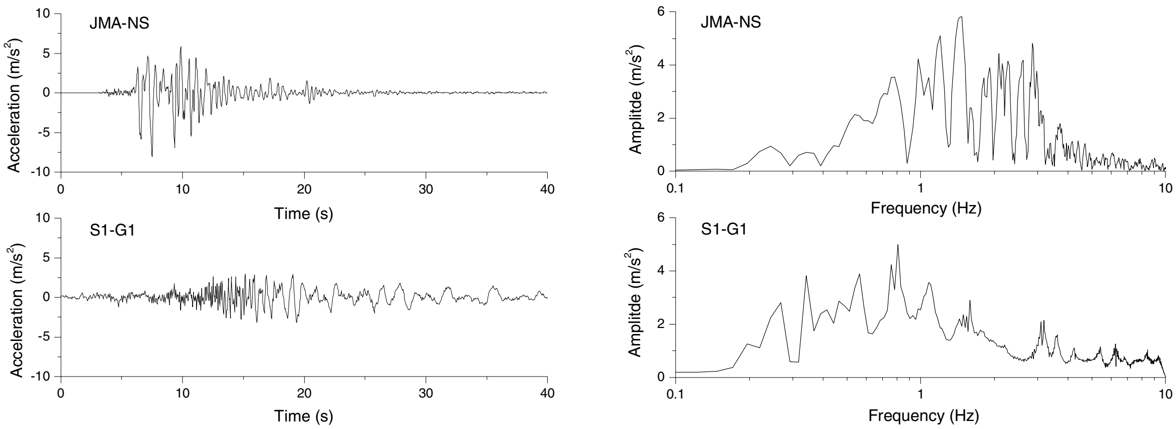
Table 1 Properties of the layered soil

Layer depth (GL-m)	Shear velocity (m/s)	Poisson ratio	Mass density (t/m^3)	Damping ratio (%)
(L1)5.0	122	0.485	2.0	5.0
(L2)11.0	172	0.491	2.0	5.0
(L3)22.0	231	0.489	2.0	5.0
(L4)36.0	279	0.486	2.0	5.0
(L5)44.0	331	0.480	2.0	5.0
(L6)52.0	376	0.474	2.0	5.0
∞	405	0.469	2.0	5.0

Table 2 Properties of the piles and WIB

Pile:	Diameter	1.8 m
	Density	2.4 t/m^3
	Young's modulus	2.54E6 tf/m^2
WIB:	Density	2.0 t/m^3
	Poisson ratio	0.2
	Column	1.0 m
	Shear velocity	1000 m/s

After the devastating Hyogo-ken Nanbu earthquake, the Japanese codes have been revised to take into account of the devastating earthquake motions of so-called Level II. In this study, the North-South component of the Hyogo-Ken Nanbu Earthquake record in Kobe (JMA-NS) is adopted as a representative excitation for such ground motion, and an artificially generated motion called S1-G1 is additionally used for a typical near-source earthquake. The time histories and Fourier spectrums of these two earthquake records are shown in Figure 6. The predominant frequencies are respectively 1.46Hz and 0.8Hz.



(a) Time history (b) Fourier spectrum

Figure 6. Acceleration records of Kobe-Jma-Ns and S1-G1

Figure 7 shows the investigated cases for different arrangement of the WIB around the pile foundation. The original situation without any countermeasure is denoted by Case A. Case B shows that several separate soil-cement walls with the same properties of WIB are installed just parallel beside piles. This case is employed for comparison to reveal the advantages of the special honeycomb shape of WIB. Case C simulates the horizontal shear effects of WIB columns by setting diagonal truss elements between WIB walls. The walls of Case B have the same depth as the WIB of Case C, viz. G.L. 11 m. Case D modifies the countermeasure by extending the side columns of WIB vertically down to G.L. 15.4 m, while keeping the depth of G.L. 11 m for the inside columns.

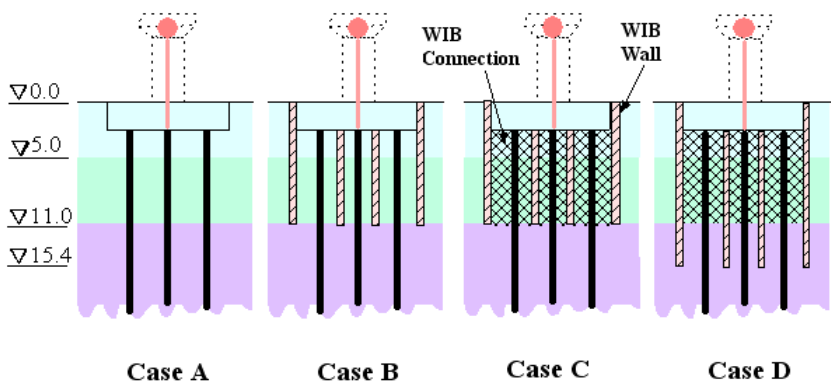
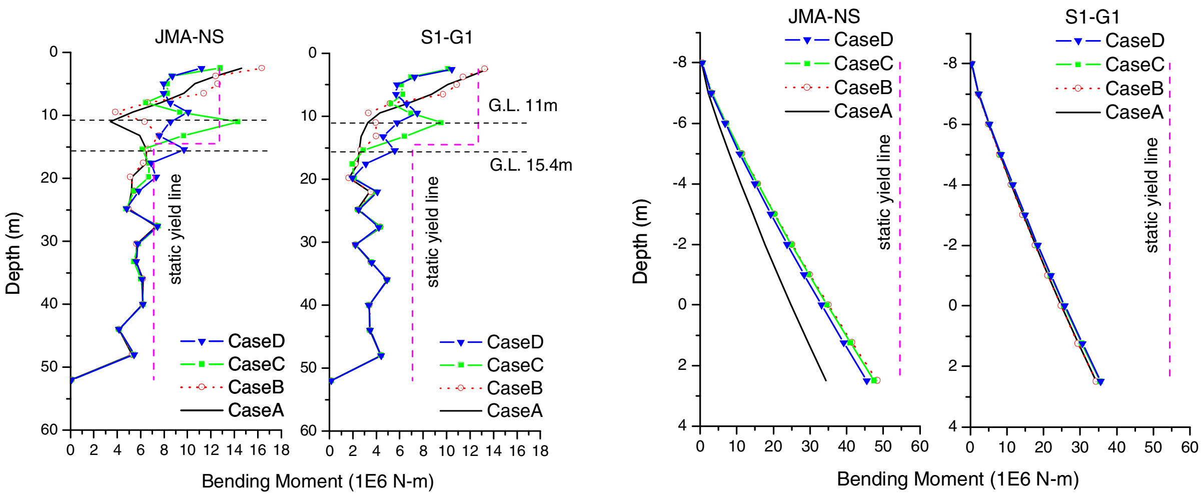


Figure 7. Studied cases

CALCULATION RESULTS AND ANALYSIS

The anti-seismic countermeasure WIB is designed on basis of the performance based design concept of structures. The computation results from the hybrid FEM-BEM program are interpreted in what follows. Figure 8 (a) shows the maximum bending moments of piles along depth, which are picked up from the maximum values of all piles responses. Figure 8 (b) depicts the maximum bending moments of the pier along depth. The static yield bending moment lines of the pile and the pier are also provided for reference (vertical dashed line). The static yield bending moments along depth are obtained according to the relationship between the bending moment and the axial force of RC beams (Figure 3 (a)). The yield line of the pile segments around the depth of G.L 14 m, which indicates the reinforcement assignment of piles changes at that point. Actually, the pile portion along the surface depth 12 m is assigned with appropriate reinforcement (56 steel bars), and the rest is reinforced only by a half of it for its relatively less internal forces. The response of bending moment of Case B is quite close to Case A, which means only walls with WIB properties can not enhance the behavior of the viaduct pile foundation system against earthquake loadings. Case C uses honeycomb shaped WIB instead of WIB parallel walls. It seems much improvement is gained so that the bending moments of about 7 meters of the top portion are reduced significantly. This means that the configuration of honeycomb cells can increase local horizontal resistance obviously.



(a) Bending moments of the piles

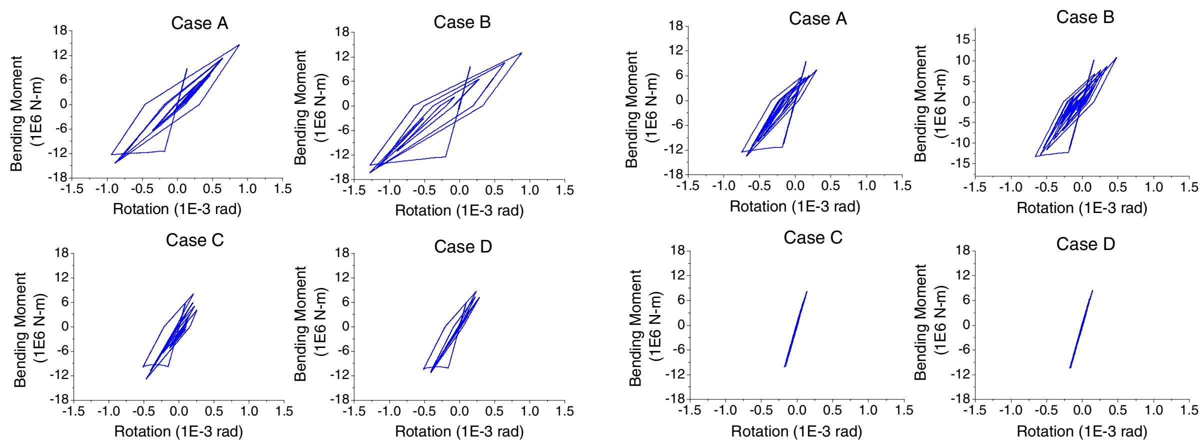
(b) Bending moments of the pier

Figure 8. Maximum bending moments of piles along depth

However, Case C results in an undesirable peak value at the bottom of the WIB, which even exceeds the original peak value at pile head of Case A. This detrimental effect is caused by the sudden stiffness change at the interface of soil improvement measures and soil. An effective way to reduce it is to make a smooth variation of the pile deformation along depth. According to this idea, an optimized scheme is implemented in Case D by extending the side columns of WIB cells while keeping the other columns same as Case C. The response of Case D shows a remarkable improvement of the modified WIB. The value at pile top does not differ much from that of Case C, but the peak value at G.L 11 m of Case C disappeared in this new case. In fact, another two peak values occur around G.L. 10 m and G.L. 15.4 m, which correspond to the two interfaces of bigger stiffness changes along depth. Since the two peaks are much smaller than that of Case C and allowable, the WIB of Case D may successfully be accepted.

The responses of the pier show that introducing the WIB increase maximum bending moments, especially for JMA-NS earthquake input, but the internal forces are still far below the yielding values. The honeycomb shaped WIB modifies the seismic resistance ratio between piles and pier, shifting some burden from the piles to the pier to balance the behavior better. In view of this, the anti-seismic countermeasure should be designed on basis of the performance of the whole system to meet a satisfying result.

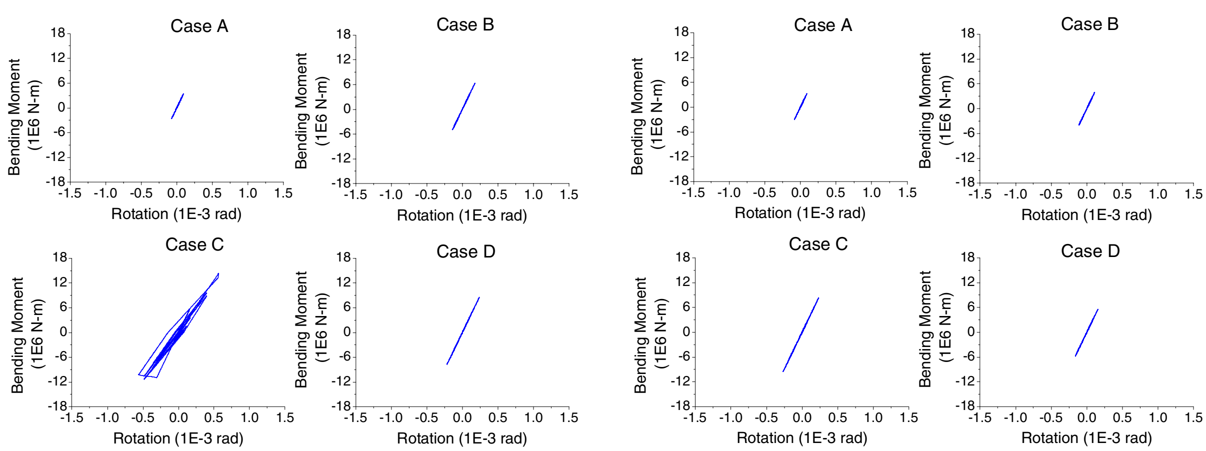
Figure 9 and Figure 10 illustrate the time history relationship between bending moment and rotation angle of pile 1. Case A and Case B lead to significant nonlinear behavior of the piles with the rotation increases at pile head, and Case C results in unfavorable response at the boundary of WIB (G.L. 11 m). Case D reduces the internal forces at the pile head, simultaneously avoiding significant increases at crucial boundaries. Since the S1-G1 loading is not so destructive as the JMA-NS earthquake loading, the piles behave almost linearly with the WIB, but we can still find the advantages of modified WIB of Case D by the response ranges attained.



(a) JMA-NS

(b) S1-G1

Figure 9. Bending moment-Rotation relationship at G.L.2.5 m of pile 1

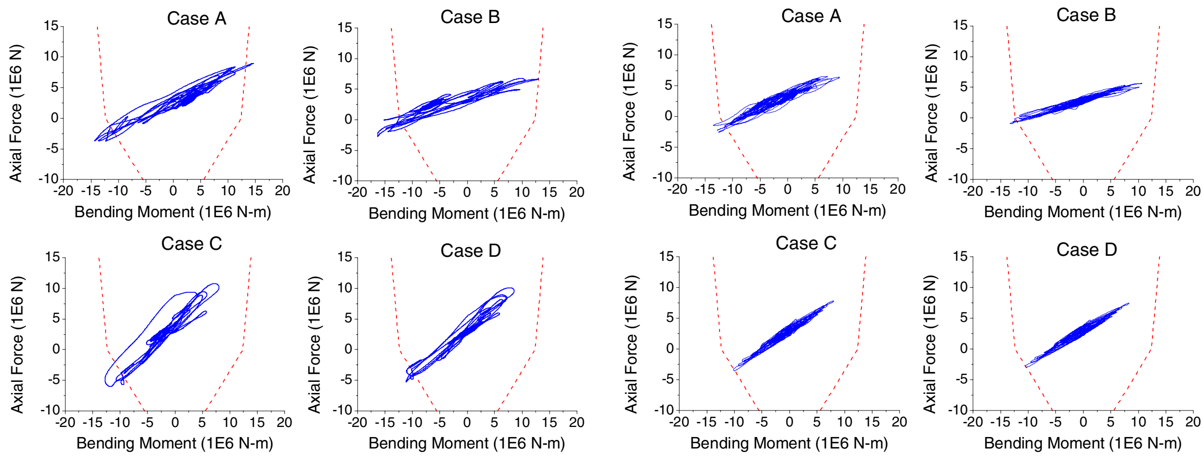


(a) JMA-NS

(b) S1-G1

Figure 10. Bending moment-Rotation relationship at G.L.11.0 m of pile 1

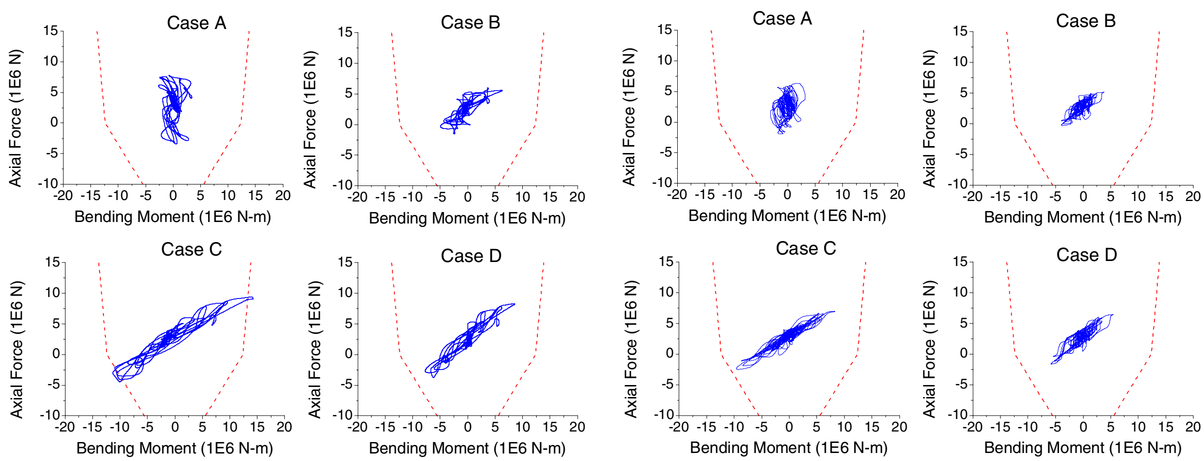
Figure 11 and Figure 12 describe the time history relationship between bending moment and axial loading of pile 1 at pile head and the boundary of WIB. The corresponding yielding diagram of bending moment and axial force is indicated for reference. Both JMA-NS and S1-G1 loading cases show evidently the advantages of the optimized WIB measure of Case D. At the pile head, the WIB prevents the piles from failure by constraining the bending moment-axial force loops in a much safer zone. At the bottom of WIB, cases without WIB are far from yielding, while Case C has failure possibility seriously. The well-designed WIB of Case D lessens such unfavorable effect to a high extent, which is acceptable in practical engineering.



(a) JMA-NS

(b) S1-G1

Figure 11. Bending moment-Axial force relationship at G.L.2.5 m of pile 1



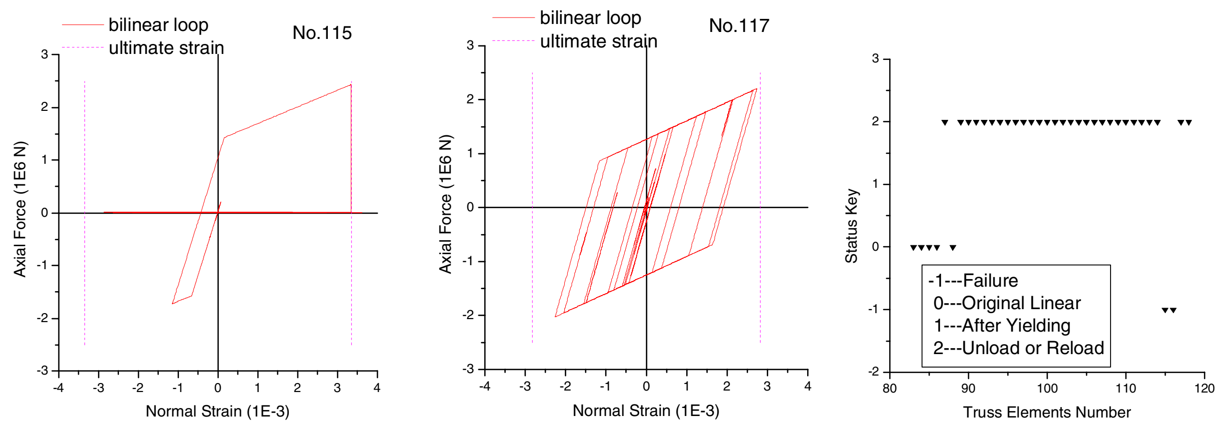
(a) JMA-NS

(b) S1-G1

Figure 12. Bending moment-Axial force relationship at G.L.11.0 m of pile 1

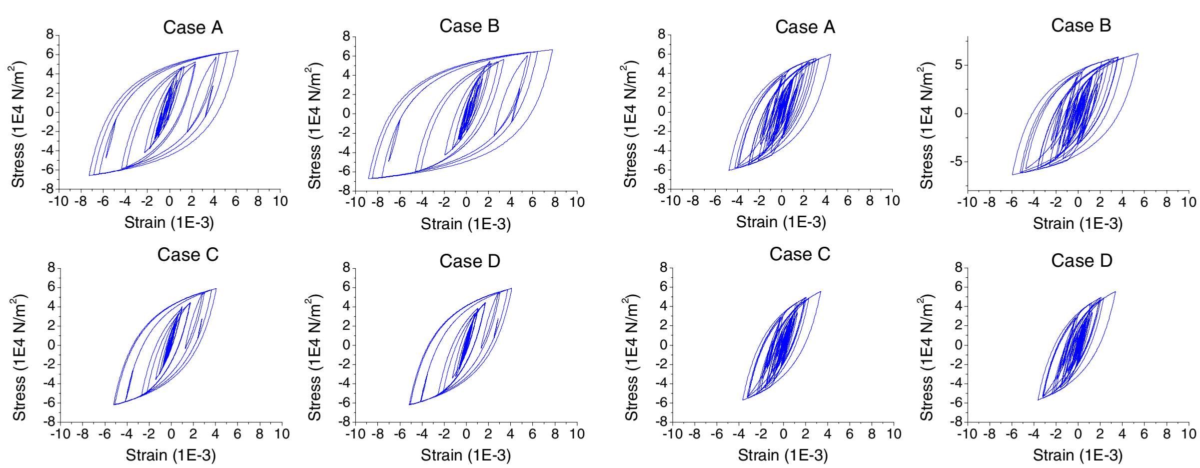
The truss elements are introduced here to simulate the horizontal shear forces of WIB columns. Those elements are assumed to behave nonlinearly to meet the large deformation condition. The bilinear behavior of the truss elements of WIB of Case D subject to JMA-NS loading are shown in Figure 13. No.115 and No.117 are two truss elements at the bottom of WIB. The vertical dashed lines are the ultimate normal strains corresponding to the ultimate shear strains obtained from experiments. The ultimate strains are different depending on the various geometric conditions of truss elements. Figure 13 (b) illustrates the final status of all the truss elements of Case D. The elements No.115 and No.116 get damaged as shown in this figure. According to this result, the WIB can be expected to get preceding partial damages while keeping the inside piles undamaged.

The responses of piles are induced not only by the inertial interaction with the superstructure but also due to the kinematic interaction directly from surrounding soil deformation. Figure 14 shows the stress-strain loops of a soil element inside the honeycomb WIB cells. From the comparison among the investigated cases, we can see the stress histories are similar; however, the strain range is reduced substantially by the WIB installation, especially for the JMA-NS input. It indicates that the WIB measure restrains the deformation of the inside soil of WIB to a significant extent to lead the reduction of pile response. In view of these restoring force characteristic in piles and WIB columns, optimizing the indirect reinforcement by the honeycomb WIB benefits the performance based rational design of the total system.



(a) Bilinear loops (b) Final strain status

Figure 13. Bilinear behavior of truss elements of WIB



(a) JMA-NS (b) S1-G1

Figure 14. Shear stress-strain loops of a soil element inside WIB cells

CONCLUSIONS

An innovative countermeasure called honeycomb shaped WIB has been applied to a viaduct pile foundation to reduce the seismic response. The honeycomb WIB is designed with the knowledge of wave field with respect to the target frequency to control for the plan view and the active length of a pile deformation in soil for depth. It is made of soil-cement columns by the soil improvement technique.

Herein, a 2-dimensional time domain FEM-BEM analysis was conducted to simulate the behavior of the WIB-enhanced pile foundation for the Level 2 earthquake inputs. The parametric study was performed in order to gain the optimum size for the WIB in which the smooth variation of the bending moment and shear force profiles could be attained along pile. Such a well-designed WIB-enhanced foundation keeps the internal forces of piles below the critical values at important sections. Further, in the case of server earthquakes the preceding damage at the WIB that results in absorbing the seismic input energy can save the pile foundation without damage. High damping materials may of practical use for the in-fill in the honeycomb WIB to surround the piles for benefiting the more response reduction for seismic input.

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