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Technical Paper

Seismic performance of bridge pier integrated by multiple steel pipes with directly-connected piles using soil-water coupled three-dimensional elasto-plastic finite element analysis

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Abstract

A bridge pier integrated by multiple steel pipes with directly-connected piles was proposed as a way to design a rational foundation for bridge pier structures, and its feasibility was examined by comparing its seismic behaviour with that of a bridge pier with a conventional footing. Through large-scale shaking table tests, employing bridge pier and foundation models at a scale of 1:20, it was verified that the seismic performance of such a bridge pier is comparable to, if not better than, that of a bridge pier with a conventional footing. However, as the shaking table tests utilized only a 2-Hz sine wave as the input, the impact of the relationship between the structure's natural frequency and the input frequency on the seismic performance, and the response of the structure to irregular ground motion, need further investigation. Therefore, the aim of the present paper is to conduct a reproduction analysis of the results of the above experiment using the soil–water coupled three-dimensional elasto-plastic dynamic finite element analysis method. The analysis will employ a different seismic motion input from that used in the above tests. Through this analytical approach, the paper will elucidate the characteristics related to employing a bridge pier comprised of multiple steel pipes with directly-connected piles.

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Keywords: Integrated bridge pier; Footingless; Pile foundation; FEM; Liquefaction; Hysteretic shear damper

1. Introduction

With the pier being a key member that must safely transfer superstructure loads to the foundation, minimizing damage to the pier, especially during seismic events, is of concern to researchers and engineers. The present study introduces a ductile, lightweight, and cost-effective pier composed of multiple steel pipes integrated by shear links.

Specifically, the pier consists of four prefabricated steel pipes, combined with shear links at three different levels along the pier height, as shown in Fig. 1. The steel pipes (columns) are the main members of the load-resistant assembly; they are primarily employed to resist the vertical loads, namely, the dead and live loads. The shear links (panels), on the other hand, are made of low-yield-strength steel and have hysteretic energy dissipation properties; they are employed to resist the lateral loads arising from earthquake motion. During an earthquake, damage is intentionally directed so as to concentrate at the shear

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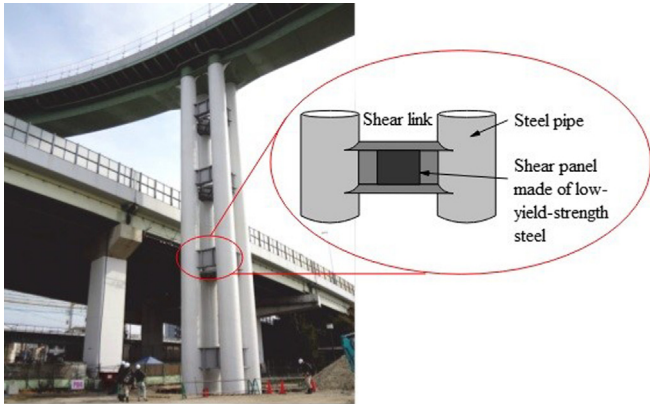


Fig. 1. Bridge column integrated by multiple steel pipes and multiple shear panels interconnecting the pipes for practical use.

links, allowing the steel pipes to maintain their strength in accordance with the damage control design concept for bridge piers (Morishita et al., 2000; Hino et al., 2001; Marwan et al., 2002). The inherent seismic resistance of the shear panels limits the damage to the bridge during seismic events, enabling its continued use by emergency and passenger vehicles during crises.

A comparative study of this pier and the conventional steel pier, using numerical analyses by Kanaji et al. (2004), confirms the damage control design response of the pier structure as the shear panels absorb energy and mitigate any possible damage to the steel pipes. Additionally, it demonstrates the reduced response displacement at the pier top in comparison to a conventional pier. Shinohara et al. (2012a) performed a series of numerical studies, consisting of pushover and dynamic analyses, to clarify the cause–effect relationship between the characteristic period of this pier and the response of the spectrum property of the seismic wave. Drawing from the findings of these studies, design standards for bridge piers integrated with multiple steel pipes were established for the first time (Hanshin Expressway Co. Ltd, 2020). Given the validation of the damage control design and its superiority over conventional steel piers, the steel pipe combined pier has already been implemented in practical applications (Shinohara et al., 2012b).

Our research group proposes a directly-connected pile foundation with steel pipes to rationalize the foundation structure for this type of pier footing. Its structural feasibility is examined by comparing its behaviour with that of a conventional pier footing under earthquake loading (Isobe et al., 2010; Shinohara et al., 2013; Isobe et al., 2015; Isobe et al., 2016; Isobe et al., 2017). Based on the seismic energy absorption of the steel pipe pier columns, the direct connection of the pier pipes to the piles is seen to redistribute the loads of the superstructure, thereby reducing the concentration of stress at the joints and allowing for the direct transmission of loads to the piles. This design will lead to a decrease in the time and cost of the construction, and will enable the use of pile foundations

in confined spaces. Shaking table tests, with pier and foundation models at a scale of 1:20, were conducted in prior studies (Isobe et al., 2015; Isobe et al., 2016; Isobe et al., 2017) to verify the advantages of the design. The structure based on the design demonstrated an outstanding seismic performance against excitations with a frequency of 2 Hz. However, assessing a structure's performance solely under a single excitation frequency is insufficient for extrapolating its response to real earthquake events.

In the present study, therefore, a three-dimensional elastoplastic dynamic finite element analysis is conducted to develop a numerical simulation method for the experimental results and to elucidate the effect of these excitation conditions. The validity of the reproduction analysis method is confirmed, and the dynamic response characteristics of the proposed structure are clarified through various analysis models, namely, static, cyclic, and seismic. Both footing and footingless types of structures are subjected to Level 2 seismic motion (in accordance with the guidelines of Hanshin Expressway) in dry sand to evaluate their responses to a real earthquake event. Furthermore, the influence of the input motion frequency on both types of structures is investigated through a parametric study by varying the input excitation frequency from 0.1 to 10 Hz. Moreover, the dynamic soil-structure interaction of the proposed structure, which considers both inertial and kinematic interactions, is briefly examined based on the results of the numerical simulation.

2. Outline of shaking table model experiment

2.1. Experiment models

The schematics of the specimens used in the shaking table test experiment are shown in Fig. 2. The specimens were reduced to models of 1:20 in size. Assuming a ramp bridge on an urban highway, and referring to Shinohara et al. (2013), single-column piers were created considering the similarity rule (Iai, 1988). Each pier was composed of four steel pipes. The horizontal connecting members (called shear links) between the steel pipes were set at the three levels shown in Fig. 2. A shear panel, having hysteretic damping characteristics, was set at the central part of each shear link, as shown in Fig. 1. Based on the findings of the previous experimental studies by Kanaji et al. (2004), Shinohara et al. (2012a), and Shinohara (2014), a vertical stiffener was installed on each shear link having an aspect ratio (the ratio of the height to length at the central part of the shear link web) equivalent to one, to reinforce the two ends of each shear link. This was done to ensure that the shear deformation would progress steadily and that the shear links would effectively perform as a hysteretic damper. The ratio of the plate width to the thickness against governing plate buckling criteria was 0.48, like that in the cyclic loading tests on the columns (Kanaji et al., 2004; Shinohara et al., 2012a).

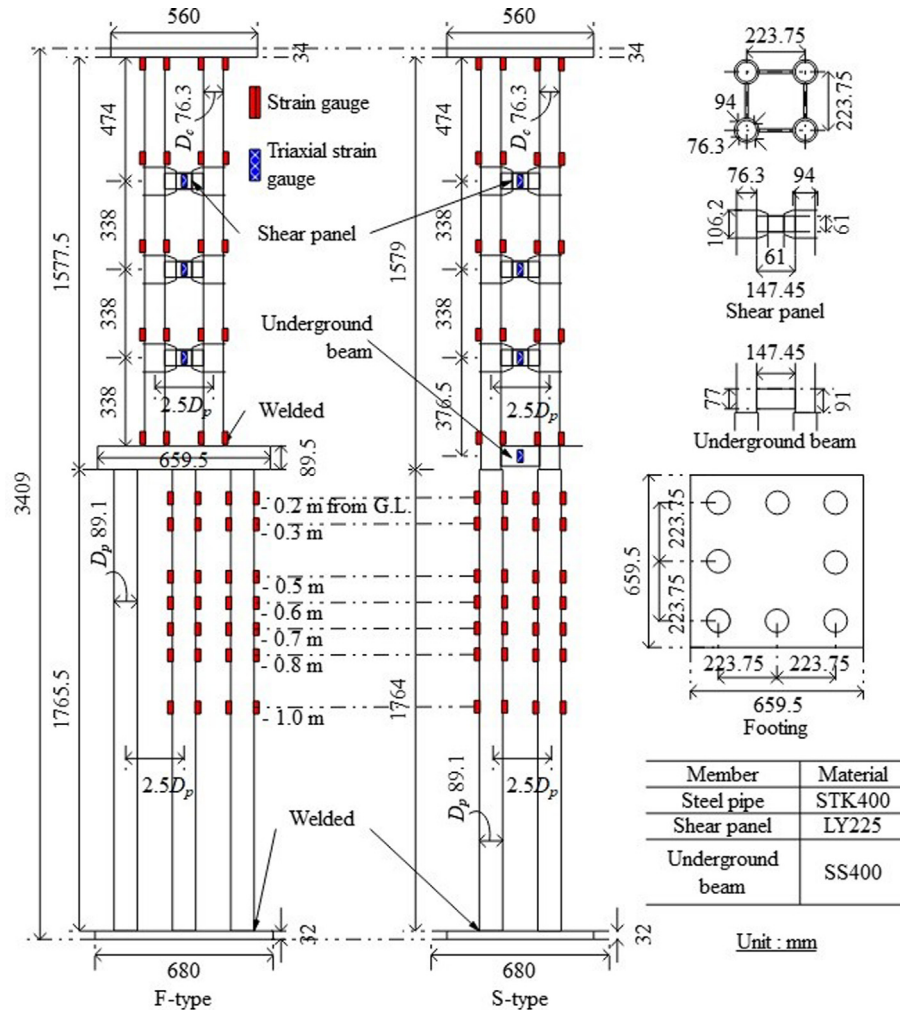


Fig. 2. Specimens and the locations of the strain gauges in the experiment (After Isobe et al., 2016).

In this series of model tests, the footing type of structure (abbreviated as F) had eight steel piles to support the pier via the footing, while the footingless type of structure (abbreviated as S) had four directly-connected piles to support the pier, which is the same number as that of the pier pipes. The piles were installed with a centre-to-centre spacing of 2.5 times the pile diameter. For the S-type, an I-shaped underground beam was welded to the base of the pier pipes (the pile heads), instead of a footing, in a similar way to the case with shear panels, to avoid excessive displacement of the piles, especially around the pile heads. To connect the pier pipes to the foundation piles, the pier pipes were inserted 20 mm into the foundation piles. Any gaps between the members were filled with the same material as that used for the underground beam, and then they were welded together to ensure the alignment of the steel pipe centres and to maintain a welding thickness at least equal to that of the steel plate. As a safety precaution in the experiment, the lower end of each pile was welded to the base steel plate, which was then bolted to the bottom of the soil tank. Therefore, the tip of each pile was completely fixed.

To emulate the superstructure, a weight of 52.6 kN, having the dimensions of 1586 mm x 1586 mm x 265 mm, was bolted to the upper part of each model (see Fig. 3). The F-type model had a steel block footing with a weight of 2.2 kN and dimensions of 659.5 mm x 659.5 mm x 89.5 mm. It was welded to the pier columns and foundation piles. The properties of the structural materials used for the specimens are shown in Table 1. It should be noted that the weight of the superstructure and footing does not conform to the similarity rule due to limitations in the shaking table's vibration capacity and the requirement to maintain a semi-infinite pile length. For this reason, its susceptibility to resonance has been thoroughly assessed in Section 4.2.

2.2. Model grounds

Each model ground used in the experiment was prepared in a rigid soil tank that was 4.0 m in length, 1.0 m in width, and 2.0 m in depth. Two types of soil grounds were prepared: dry sand (abbreviated as D) and liquefiable sand (abbreviated as L). Fig. 3 shows the different ground

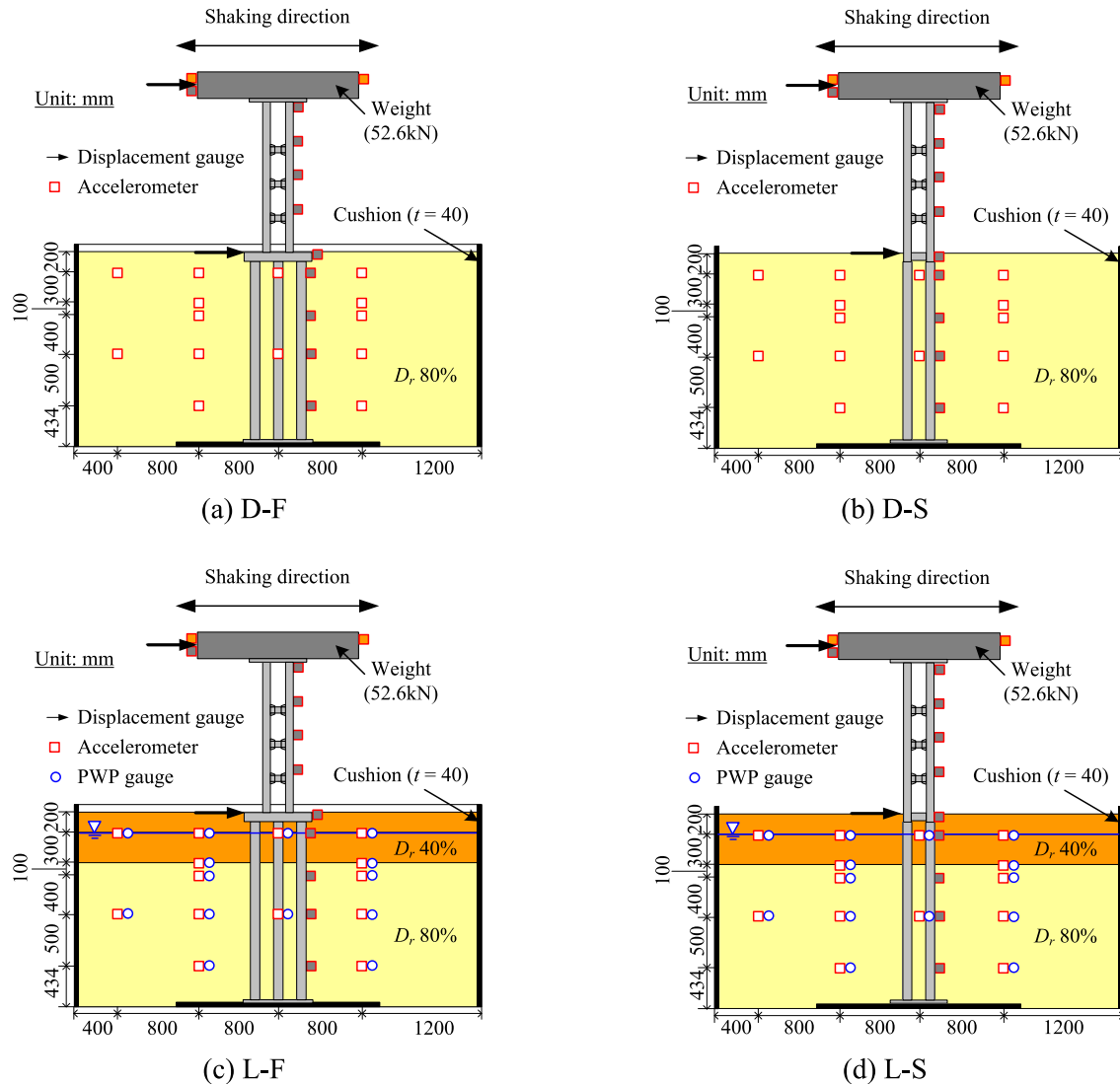


Fig. 3. Model grounds along with measurement devices and their locations (After Isobe et al., 2016).

Table 1
Properties of the structural materials.

Part	Standard	Size [mm]	Thickness [mm]	Yield stress [N/mm ²]
Pier (Column)	STK400	$\phi 76.3$	2.8	406
Pile	STK400	$\phi 89.1$	2.8	404
Shear panel	LY225	61 × 61	1.1	236
Underground beam	SS400	77 × 147	1.6	345

conditions for the F- and S-type models embedded in the D- and L-grounds set in separate soil tanks.

The L-ground was prepared using air and water pluviation methods. Dry sand, with a relative density (D_r) of 80 %, was poured into the tank to a height of 1434 mm, followed by the injection of water to raise the level to a height of 1734 mm and to create a liquefiable soil layer with a relative density of 40 % using water pluviation. In addition, a

200-mm-thick layer of dry topsoil was created by air pluviation to prevent spillage of the liquefiable soil. The D-ground, with a relative density of 80 %, was prepared similarly as one layer using air pluviation. The ground surface was set at a height of 1934 mm from the bottom plate for both the L- and D-grounds, aligning it with the upper surface of the footing in the F-type model and the underground beam in the S-type model. The sand was Tohoku silica sand #6 (soil grain density G_s : 2.63 Mg/m³, maximum void ratio e_{max} : 0.878, minimum void ratio e_{min} : 0.551, and average grain diameter D_{50} : 0.34 mm).

Fig. 4 shows the results of the static and cyclic triaxial compression tests performed on specimens prepared with the same relative density as that for the model grounds. From the figure, it is seen that the shear resistance angles are 36.5° (D_r 40 %) and 41.0° (D_r 80 %). The liquefaction strength curve values are 0.15 (D_r 40 %) and 0.32 (D_r 80 %) following the JGS standards which define the liquefaction

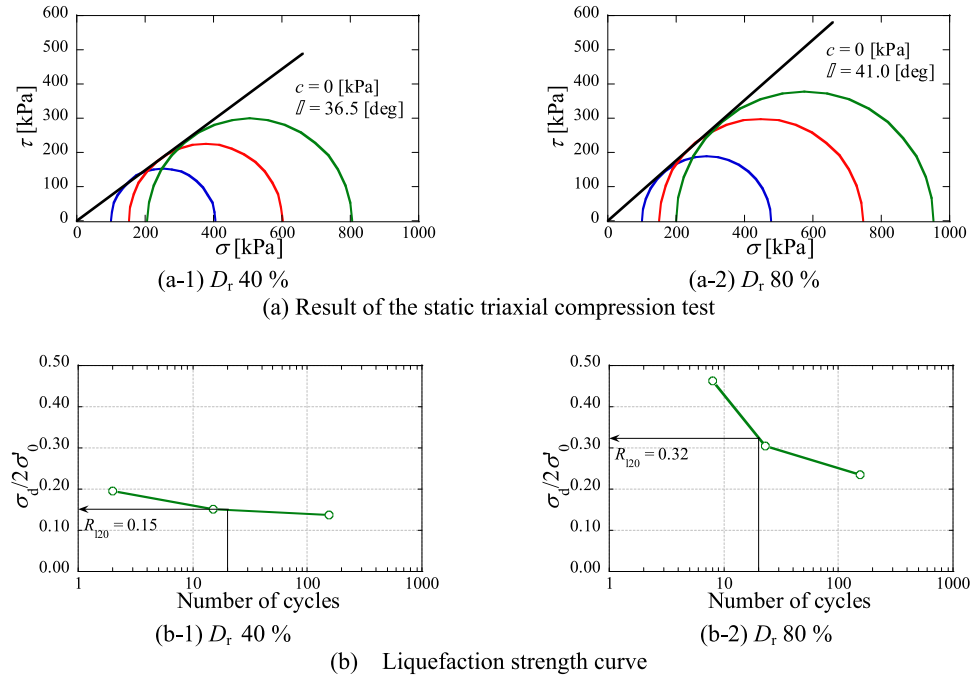


Fig. 4. Results of the static and cyclic triaxial compression tests performed on the model ground specimens (After Isobe et al., 2016).

strength ratio as the stress ratio for 20 cycles of loading based on Ishihara and Yoshimine (1992) and Matsuo et al. (2000). A 40-mm-thick rubber sheet (Young's modulus E : 0.05 GPa, Poisson's ratio ν : 0.49, and density ρ : 0.07 Mg/cm³) was installed on the surfaces of the soil tank to mitigate the influence of the rigid soil tank (Baziar et al., 2018; Lombardi et al., 2015).

2.3. Excitation conditions

Table 2 provides details of the experimental cases, each conducted in a separate rigid soil tank. Two tanks were simultaneously vibrated longitudinally, one with the S-type model and the other with the F-type model embedded in the D-ground. Later, both the F-type and S-type models were simultaneously excited in the L-ground using the Public Works Research Institute's three-dimensional large shaking table.

The input wave was a tapered sine wave applied for 10 s with a cycle of 0.5 s (frequency: 2 Hz). The input acceleration was increased in increments of 0.5 m/s² from 0.5 to 5 m/s² for the D-ground models. For the L-ground models, acceleration was applied once at 2.0 m/s². The input steps of the excitation procedure are detailed in Table 3.

Table 2
Experimental cases.

	Footing type (F-type)	Footingless type (S-type)
Dry sand (D-ground)	D-F	D-S
Liquefiable sand (L-ground)	L-F	L-S

As shown in Figs. 2 and 3, the items measured were the strain, acceleration, and displacement of the steel structures, and the pore water pressure of the ground.

3. Outline of the numerical analysis method

In this study, we performed numerical analyses based on the infinitesimal deformation theory using the soil–water coupled three-dimensional elastoplastic dynamic finite element analysis (FEM) program named DBLEAVES (Ye et al., 2007) for the above-mentioned shaking table test experiment.

3.1. Modelling the structures and grounds

In the analysis, the model elements and structures were created based on the results of the previous research by Isobe et al. (2010). The steel pipe columns and underground beam were modelled as a bilinear beam element. The shear panels were modelled as bilinear beam elements with three bilinear spring elements attached at the centre of the beam elements in the x, y, and z directions. Simple diagrams of the modelling are provided in Fig. 5(a). The hybrid element proposed by Kimura and Zhang (2000), Zhang et al. (2000) and Zhang and Kimura (2002) was employed to model each steel pipe pile, as shown in Fig. 5(b). It compensates for the weakness of the beam elements, having no volume, in simulating the interaction between the ground and the piles. When using hybrid elements, it is important to determine the shares of Young's modulus for the beam and column elements. The share of flexural rigidity (EI) is determined as 90 % for the beam ele-

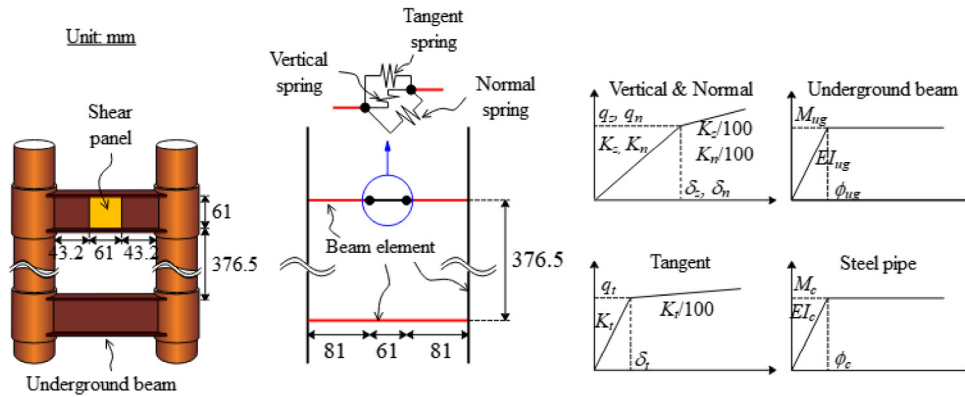
ments and 10 % for the column elements, based on Zhang et al. (2000), after considering the deformation characteristics of the piles and the relative rigidity between the column elements of the piles and the ground. Additionally, considering the volume of the column elements, the densities of the beam and column elements were adjusted so as to be

Table 3
Excitation procedure.

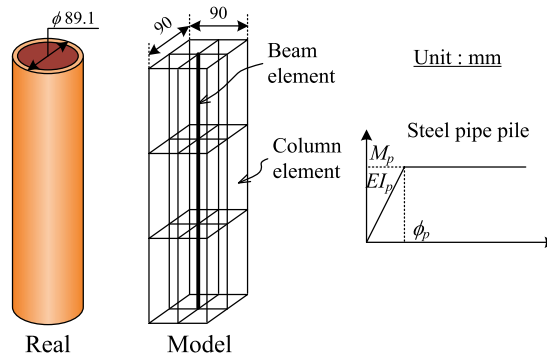
Type of ground	Excitation step	Maximum input acceleration [m/s^2]	
		Target value	Measured value
Dry sand	1st	0.5	0.62
	2nd	1.0	0.94
	3rd	1.5	1.45
	4th	2.0	2.01
	5th	2.5	2.47
	6th	3.0	3.07
	7th	3.5	3.63
	8th	5.0	5.25
Liquefiable sand	1st	2.0	1.92

equivalent to the mass of the piles. The parameters of the bridge pier specimens, comprising multiple steel pipes and steel pipe piles, are shown in Table 4.

The Cyclic Mobility model developed by Zhang et al. (2007), which incorporates the concepts of subloading and superloading as described by Hashiguchi and Ueno (1977) and Asaoka et al. (2000), was used as the constitutive model. Notable features of this model are: a yield surface with a fixed C.S.L in the yield space (Eq. (1)), incorporation of stress-induced anisotropy and its evolution rule (Eq. (2)), an evolution rule for overconsolidation (Eq. (3)) influenced by the stress-induced anisotropy, and the evolution rule for the structure (Eq. (4)). Moreover, it employs the associated flow rule for simplicity (Eq. (8)). Fig. 6 gives a conceptual diagram of the Cyclic Mobility Model, while Table 5 details the soil parameters used in the analysis. In determining the ground parameters, the above-mentioned static and cyclic triaxial test results, namely, the stress ratio in the critical state, R_f , compression index λ , swelling index κ , and void ratio N at the reference



(a) Modeling of the bridge pier consisting of multiple steel pipes



(b) Modeling of the steel pipe pile (Hybrid element)

Fig. 5. Modelling of the bridge pier consisting of multiple steel pipes and steel pipe piles.

Table 4

Parameters of the bridge pier specimens.

(a) Bridge pier consisting of multiple steel pipes (non-linear beam element and non-linear spring element)		
Model	Item	Properties
Bilinear beam element	Cross-sectional rigidity [kN]	1.29×10^5
	Flexural rigidity [kN*m ²]	8.74×10^1
	Fully plastic moment [kN*m]	6.13
	Damping constant	0.01
	Density [Mg/m ³]	7.87
Bilinear springelement	Vertical/Normal direction spring constant [kN/m]	7.22×10^4
	Vertical/Normal direction yield displacement [m]	1.16×10^{-4}
	Tangent direction spring constant [kN/m]	3.02×10^5
	Tangent direction yield displacement [m]	4.27×10^{-5}
(b) Piles (hybrid elements)		
Model	Item	Properties
Bilinear beam element	Cross-sectional rigidity [kN]	1.52×10^5
	Flexural rigidity [kN*m ²]	1.27×10^2
	Fully plastic moment [kN*m]	8.41
	Damping constant	0.01
	Density [Mg/m ³]	0.69
Column element	Cross-sectional rigidity [kN]	2.18×10^{-2}
	Flexural rigidity [kN*m ²]	1.41×10^1
	Poisson's ratio	0.30
	Damping constant	0.01
	Density [Mg/m ³]	0.69

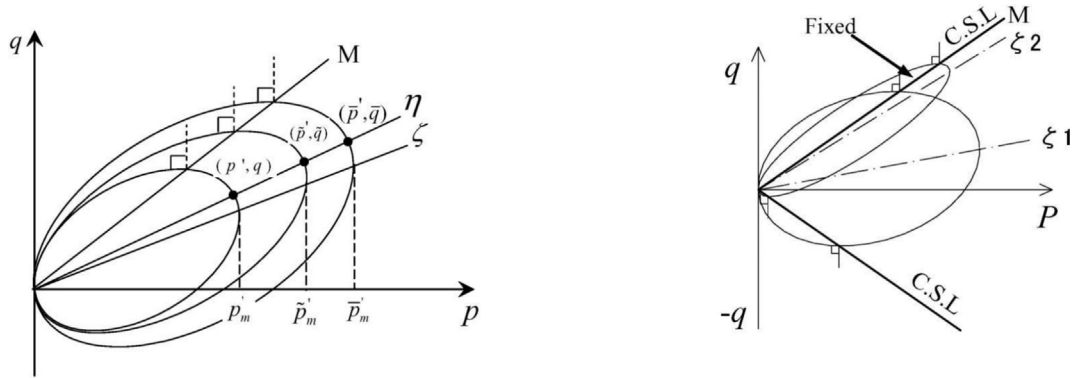


Fig. 6. Conceptual diagram of the Cyclic Mobility Model (After Zhang et al., 2007).

pressure ($p = 98$ kPa), are derived from the isotropic consolidation test results. Additional parameters were determined through a simulation of the triaxial consolidation tests, as shown in Fig. 7. In the shaking table tests, the influence of the smaller confining pressure than that in the triaxial tests was adjusted by changing the initial over-consolidation ratio $1/R_0$.

$$f = MD \ln \frac{p'}{p'_0} + MD \ln \frac{M^2 - \zeta^2 + \eta^{*2}}{M^2 - \zeta^2} + MD \ln R^*$$

$$- MD \ln R - \int_0^t J \text{tr} \mathbf{D}^p d\tau = 0$$

$$\dot{\beta} = \lambda \frac{JM \sqrt{6} b_r (b_l M - \zeta) \hat{\eta}}{(M^2 - \zeta^2 + \eta^{*2}) p'}$$

$$\dot{R}^* = J \frac{a}{D} R^* (1 - R^*) \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\| = \lambda \frac{2JM a R^* (1 - R^*) \eta^*}{(M^2 - \zeta^2 + \eta^{*2}) p'} \quad (4)$$

$$\begin{aligned} \dot{R} &= -J \frac{m}{D} \left\{ \frac{(p'/p'_0)^2}{(p'/p'_0)^2 + 1} \right\} \ln R \|\mathbf{D}^p\| + \frac{R}{MD} \frac{\eta}{M} \frac{\partial f}{\partial \beta} \cdot \dot{\beta} \\ &= d\lambda \frac{-mJM \ln R \sqrt{6\eta^{*2} + \frac{1}{3}(M^2 - \zeta^2)^2}}{(M^2 - \zeta^2 + \eta^{*2}) p'} \left\{ \frac{(p'/p'_0)^2}{(p'/p'_0)^2 + 1} \right\} \\ &\quad + \frac{R}{MD} \frac{\eta}{M} \frac{\partial f}{\partial \beta} \cdot \dot{\beta} \end{aligned} \quad (3)$$

$$\dot{R}^* = J \frac{a}{D} R^* (1 - R^*) \sqrt{\frac{2}{3}} \|\mathbf{D}_s^p\| = \lambda \frac{2JM a R^* (1 - R^*) \eta^*}{(M^2 - \zeta^2 + \eta^{*2}) p'} \quad (4)$$

where a is the degradation parameter of the structure, which controls the collapse rate of the structure, m is the degradation parameter of the over-consolidation state con-

Table 5
Soil parameters (Cyclic mobility model).

Item		D_r 80 %	D_r 40 %
Compression index	A	0.028	
Swelling index	K	0.005	
Stress ratio in the critical state	M	3.92	
Poisson's ratio	ν	0.30	
Void ratio at 98 kPa	N	0.66	
Overconsolidation parameter	m	0.31	
Soil structural parameter	a or m^*	0.65	
Anisotropic parameter	b_r	0.10	
Initial void ratio	e_0	0.63	0.76
Initial soil structural ratio	R^*_0	0.80	
Initial overconsolidation ratio at 9.8 kPa	$1/R_0$	73.4	1.00
Inherent anisotropy	ζ_0	0.30	
Damping constant	h	0.04	
Dry (in water) weight per unit volume [kN/m ³]	γ^d (γ')	15.8 (10.2)	— (9.50)

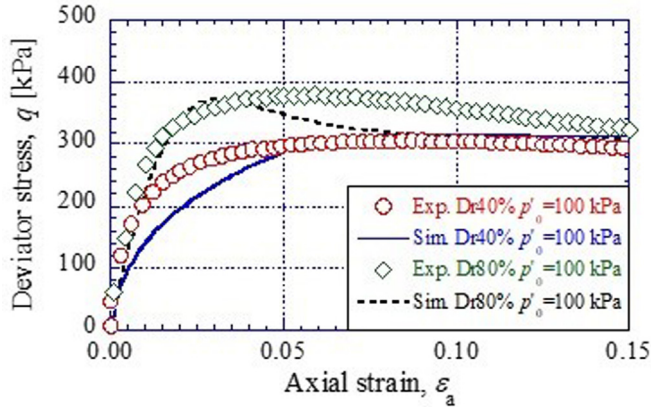


Fig. 7. Results of the drained static triaxial tests and the simulation.

trolling the rate of the loss of over-consolidation, b_r is an evolution parameter of anisotropy used to control the developing rate of stress-induced anisotropy when the soil is subjected to shearing or compression, and b_l is a limitation parameter used to avoid the singularity in Eq. (1) at the point $\zeta = M$; 0.95 is used for the value of b_l throughout this paper.

The other variables included in Eqs. (1)–(4) are defined as follows:

$$\begin{aligned} \eta^* &= \sqrt{\frac{3}{2}} \hat{\eta} \cdot \hat{\eta}, \hat{\eta} = \eta - \beta, \eta = \frac{\mathbf{S}}{p'}, \mathbf{S} = \mathbf{T}' + p' \mathbf{I}, p' \\ &= -\frac{1}{3} \text{tr} \mathbf{T}', \zeta = \sqrt{\frac{3}{2}} \beta \cdot \beta, \eta = \sqrt{\frac{3}{2}} \eta \cdot \eta \end{aligned} \quad (5)$$

where $\mathbf{S}$ is the deviatoric stress tensor, β is the anisotropic stress tensor, and $\mathbf{T}'$ is the Cauchy effective stress tensor. J is the Jacobian determination of deformation gradient tensor $\mathbf{F}$, and D is the dilatancy parameter which can be expressed by λ and κ , as follows:

$$D = \frac{\lambda - \kappa}{M(1 + e_0)} \quad (6)$$

D^p denotes the plastic component of stretching D which is assumed to be positive in tension; it is related to the plastic volumetric strain rate.

$$\varepsilon_v^p = - \int_0^t J \text{tr} \mathbf{D}^p d\tau \quad (7)$$

The following associated flow rule is employed in the model:

$$\mathbf{D}^p = d\lambda \frac{\partial f}{\partial \mathbf{T}'} \quad (8)$$

The consistency equation for the subloading yield surface can then be given as

$$\frac{\partial f}{\partial \mathbf{T}'} \cdot \dot{\mathbf{T}}' + \frac{\partial f}{\partial \beta} \cdot \dot{\beta} + MD \frac{\dot{R}^*}{R^*} - MD \frac{\dot{R}}{R} - J \text{tr} \mathbf{D}^p = 0 \quad (9)$$

3.2. Analysis conditions

The analysis domain was set as the half-sectional domain shown in Fig. 8 in consideration of the symmetry of the structures and the loads used in the tests. The number of nodes was 15423, and the number of elements was 13464. For the boundary conditions, the nodes on the bottom surface were fixed in all directions, while the side surfaces were roller supported in the vertical direction. The cushioning material, installed to diminish the influence of the boundaries, was modelled as a solid element. The influence of the cushioning material will be discussed in the Appendix.

The analysis utilized an initial stiffness proportional type of viscous damping model. Natural frequencies for the D-F and D-S types were calculated using the waveforms at the pier top during the free vibration time using the Ricker wavelet (Ricker, 1945), which has a minute acceleration amplitude. The damping constants of the ground and the structure were set to 4 % and 1 %, respectively. Under these analysis conditions, the natural frequencies were calculated as 2.78 Hz for D-F and 1.86 Hz for D-S, which roughly agree with the experimental values of 2.70 Hz for D-F and 1.84 Hz for D-S. Similarly, a Ricker wavelet analysis was performed using dry soil only in the soil box. The soil surface response measured at the centre of the soil box indicated a natural frequency of 8 Hz for the soil. The time increment in the analysis was 0.005 s, and the Newmark- β method was used for the time integration.

The dead load of the superstructure was simply modelled as a mass point of 5.36×10^3 kg (to be exact, it was 2.68×10^3 kg because of the half-sectional domain). The differences in the superstructure inertial force and pile bending moment between a point mass modelling and a solid mass modelling are almost negligible, as will be shown in the Appendix for both the F- and S-type structures.

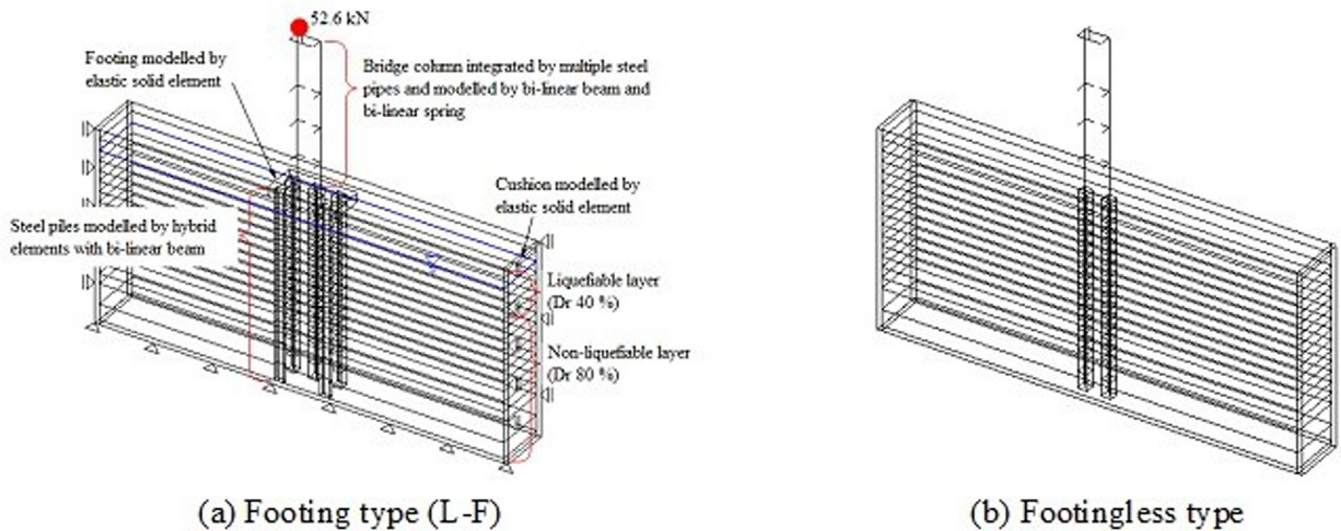


Fig. 8. Analysis mesh used in the reproduction analysis.

3.3. Analysis cases

3.3.1. Cyclic loading

(a) Reproduction analysis

We conducted a reproduction analysis through a numerical simulation of the shaking table tests to assess the seismic performance of a bridge pier constructed using F- and S-type foundations. The selected cases for the numerical simulation included the first to fourth and eighth input excitations on a dry sand ground, as well as the first input excitation on a liquefiable ground. Throughout the simulation, we varied only the amplitude of the input cyclic wave for numerical stability, and did not consider the influence of the previous motions on the stress and strain histories. However, the findings presented in Section 4.1 confirm that the influence of the previous motions on the stress and strain histories was minimal compared to the impact of the variations in amplitude.

(b) Parametric study

In the parametric study, we adjusted the input frequency from 0.1 Hz to 10 Hz, and the acceleration amplitude from 0.5 m/s² to 5.0 m/s². These adjustments were made to ensure that the shear panels experienced plastic deformation during the ground motion.

3.3.2. Seismic loading

Additionally, we employed Level-2 input seismic motion as specified in the Guidelines for Seismic Design of Cut and Cover Tunnels published by Hanshin Expressway Co., Ltd. The amplitude and time axis of the waves recorded at a depth of 83 m beneath Port Island during the Hyogo-ken Nanbu Earthquake were adjusted to match the scale of

the model tests. This waveform encompasses a range of frequencies with predominant components close to 2 Hz and a maximum acceleration magnitude of 6.0 m/s². However, as the model does not perfectly satisfy the similarity law of weight and the associated natural frequency (Isobe et al., 2016), the time axis was further adjusted in various cases to ensure that the dominant frequency of the irregular waves would comprehensively cover the natural frequencies of both structure types. This loading simulation was carried out for both types of structures, namely, those with footings and those without footings, under dry sand conditions, using the same Finite Element Method (FEM) model as in the previously mentioned simulations.

3.3.3. Static loading

In the static loading analysis, a pushover analysis was conducted to assess the structure's response. In other words, a horizontal force equivalent to the inertial force was applied statically at the pier top for both D-F and D-S structures, employing the same FE model as that utilized in the preceding simulations.

4. Numerical analysis results and discussion

4.1. Dry sand ground cases

4.1.1. Results of reproduction analysis

Fig. 9 depicts the relationship between the inertial force and the lateral displacement at the pier top under dry sand conditions, considering various sets of input excitations ranging from the first to the fourth and the eighth input accelerations. The figure showcases both the experimental and simulated outcomes for the D-F and D-S structures. However, as a precautionary measure, the tests for D-F were halted due to excessive strain observed at the pier base

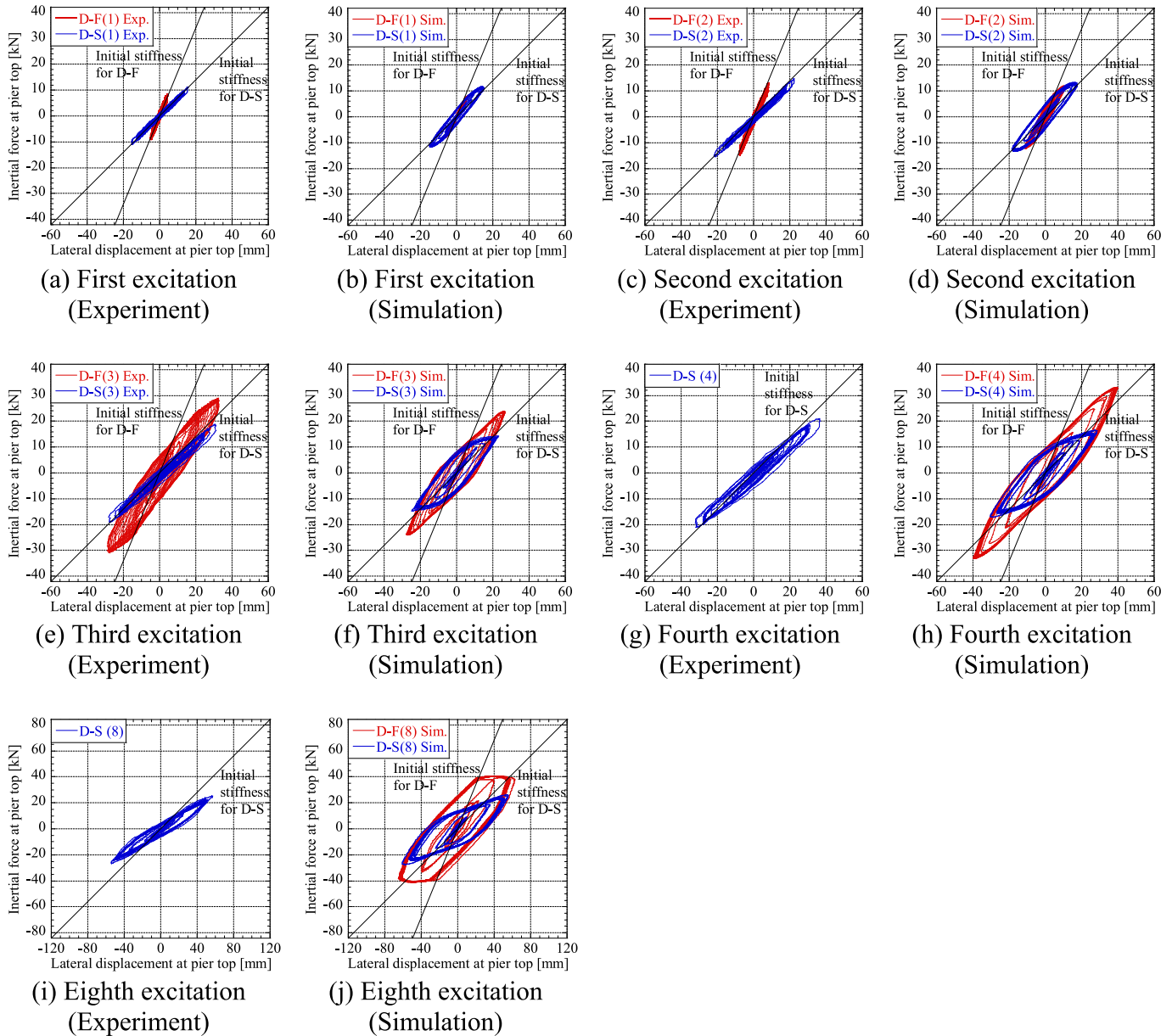


Fig. 9. Comparison of the experimental and simulation results for the relationship between the inertial force and the lateral displacement at the pier top.

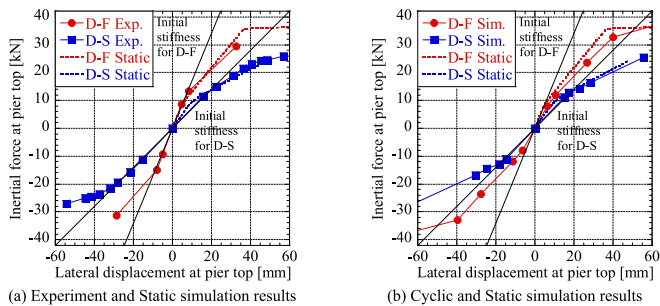


Fig. 10. Comparison of the experimental and simulation results for the relationship between the maximum inertial force and the maximum lateral displacement at the pier top.

during the third input excitation. Consequently, the response of D-F to the remaining excitations was derived through the numerical simulations only.

The inertial force was calculated as the product of the response acceleration observed at the pier top and the mass corresponding to the weight of the superstructure. The comparison in the figure between the experimental and the numerical simulation results indicates that the numerical simulations tended to underestimate both the inertial force and the lateral displacement at the pier top. Specifically, in the case of D-F, only the initial stiffness was marginally underestimated, while in the case of D-S, both the initial stiffness and the hysteresis loop were slightly overestimated. This overestimation may be attributed to simplifi-

cations made in the model, such as representing the superstructure weight as a point mass, modelling piles as hybrid members, and treating shear panels as springs. Although the differences caused by these individual simplifications are minor, their combined effect might explain the quantitative discrepancies observed in the numerical simulation results. Despite these discrepancies, the peak values of the inertial force and pier top displacement are nearly perfectly simulated from the first to the eighth steps.

Based on these findings, Fig. 10 presents skeleton curves illustrating the relationship between the maximum inertial force and the lateral displacement at the pier top. The skeleton curves obtained from the experiments have been

effectively reproduced through the reproduction simulation and static pushover analysis. The results indicate that the inertial force and pier top displacement in D-F gradually increase until the second excitation input, and then exhibit exponential growth thereafter. In the third step, these response values for D-F exceed those for D-S, and continue to do so in subsequent steps. Although no experimental results for D-F were available for comparison after the third acceleration input, the simulation results indicate that the inertial force reaches a plateau at approximately 36 kN, and that the displacement increases sharply, consistent with the pushover analysis results. In the case of D-S, the skeleton curve is almost accurately reproduced, and the

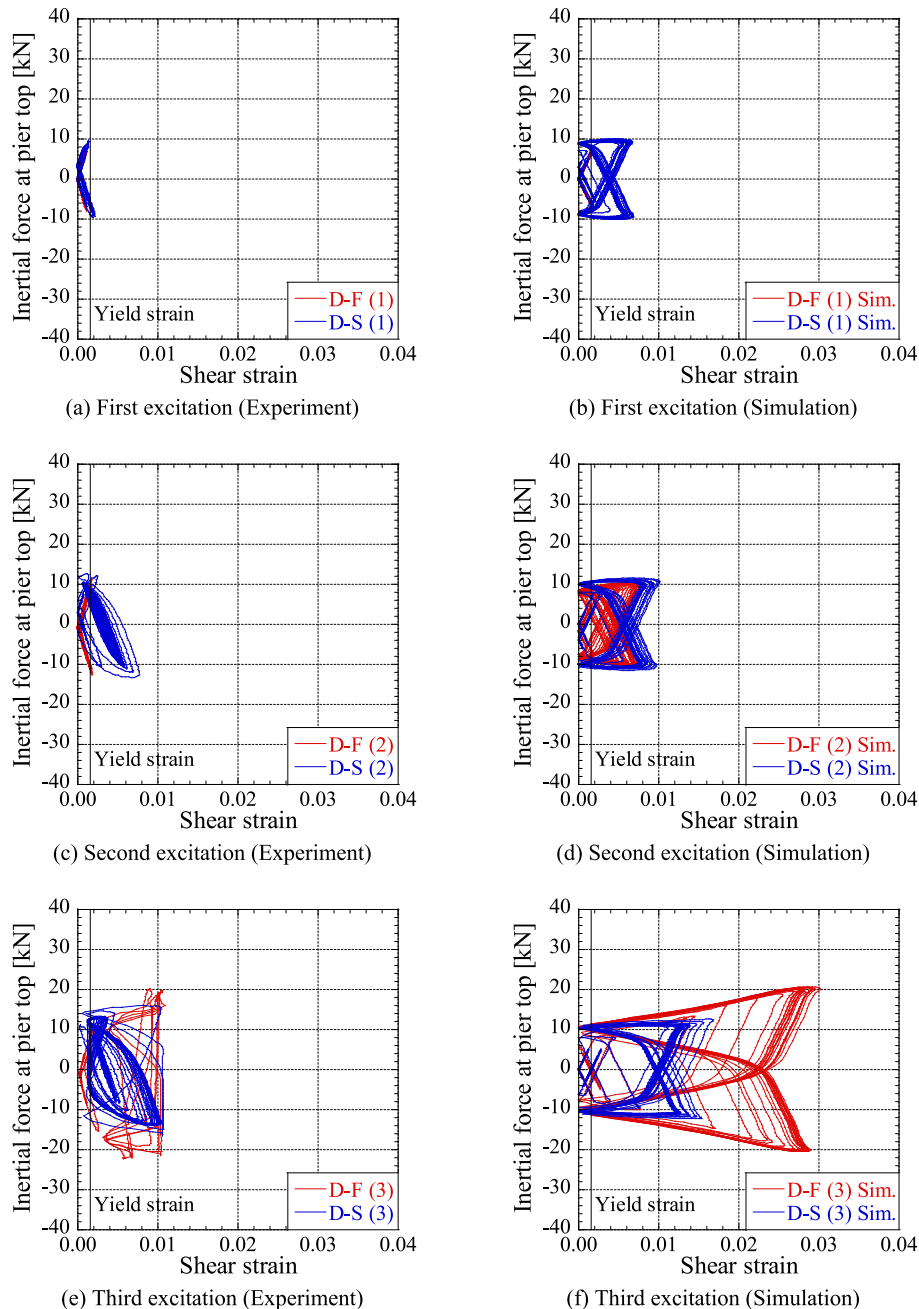


Fig. 11. Relationship between the inertial force at the pier top and the shear strain in the upper shear panel.

increments in the inertial force and pier top displacement are gradual until the eighth step. The suppression of the decrease in rigidity in D-S is also reproduced in the 4th and subsequent steps.

Fig. 11 illustrates the relationship between the inertial force at the pier top and the shear strain of the upper shear panel. Despite a tendency for the analysis results to overestimate the shear strain in both structures as the excitation

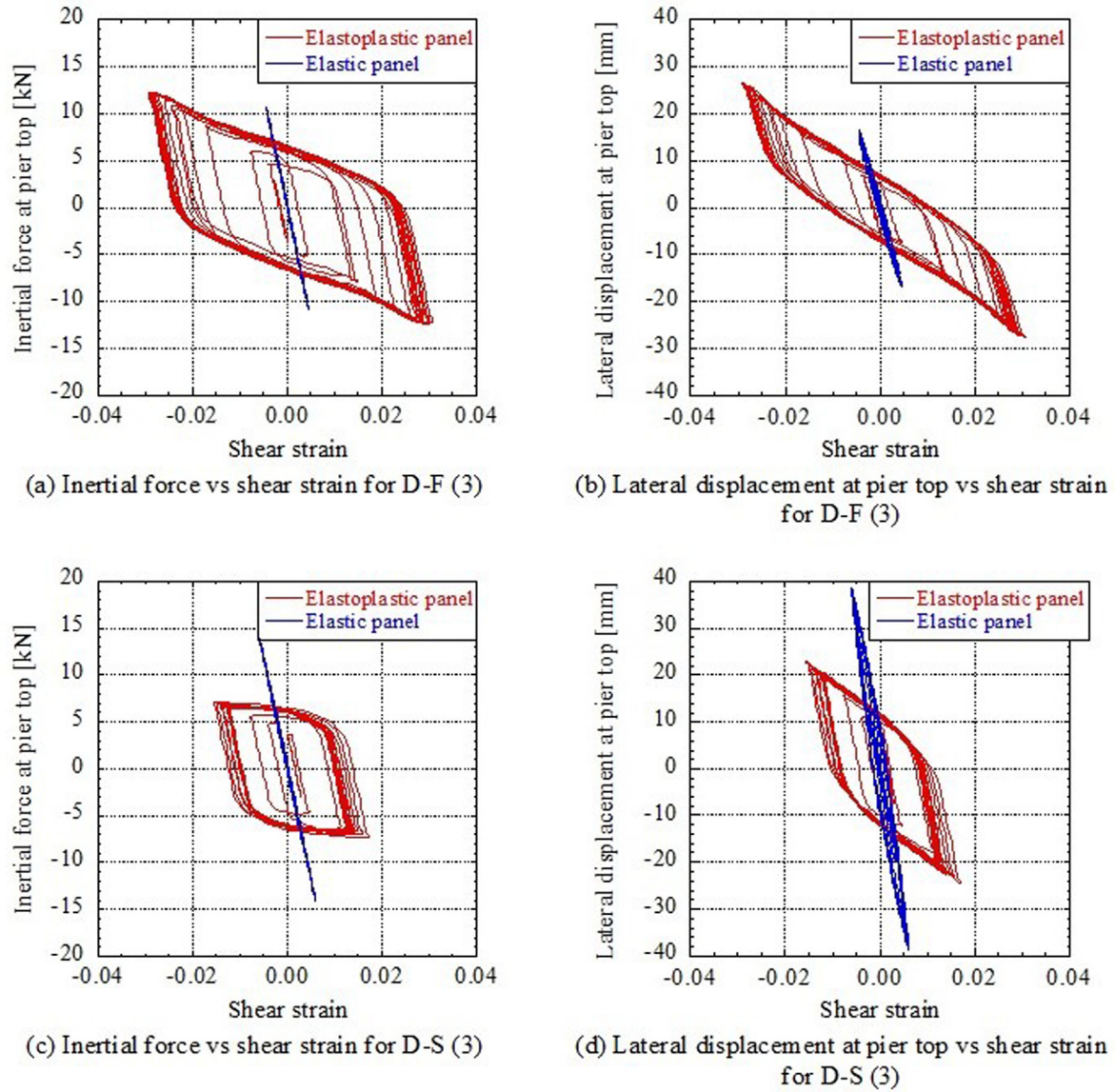


Fig. 12. Responses of the superstructure against shear strain for elastoplastic (EP) and elastic shear panels under the third excitation step.

Table 6

Comparison of the response parameters for the F- and S-type structures under elastic and elastoplastic shear panels corresponding to the third excitation step.

Structure type (Shear panel modelling)	A_{PT} [m/s ²]	M_c [kN*m]	D_{PT} [mm]	S	A_{PH} [m/s ²]	M_P [kN*m]	D_{PH} [mm]	Q [kN]
D-F (EP)	4.56	2.86	27.5	0.0306	3.44	0.90	8.75	5.27
D-F (E)	3.99	1.12	16.8	0.0045	2.72	0.86	8.32	5.12
D-S (EP)	2.79	1.02	24.5	0.0171	2.54	0.97	9.04	3.54
D-S (E)	5.38	1.58	38.7	0.0060	3.17	1.44	19.6	6.82

A_{PT} : pier top acceleration, M_c : pier base moment, D_{PT} : pier top displacement, S: shear strain of the upper panel, A_{PH} : pile head acceleration, M_P : pile head moment, D_{PH} : pile head displacement, and Q: pile head shear force.

input increases, the behaviour of each structure is qualitatively reproduced with sufficient accuracy for understanding the mechanism. In the first and second steps, D-S generates a larger shear strain than D-F, consistent with the observations in Figs. 9 and 10. Only D-S exceeds the yield strain in the first step, while D-F exceeds the yield strain after the second step. Subsequently, D-F exhibits larger shear strain than D-S in the third step. These findings highlight a strong correlation between the yielding of the shear panels and the response of the pier top. In D-F, the stiffness of the entire structure decreased significantly after the yielding of the shear panels, leading to a sharp increase in the inertial force and lateral displacement of the pier top. In contrast, in D-S, the decrease in stiffness of the entire structure, due to the yielding of the shear panels, was relatively small compared to that of D-F.

To strengthen our understanding, a numerical simulation is performed with shear panels modelled as perfectly elastic springs and compared against an elastoplastic panel case for D-F and D-S under the third excitation step. Fig. 12(a) and (b) show that, in the case of perfectly elastic springs, the lateral displacement and inertial force at the pier top are significantly decreased. This is because the shear panels just reach their yield strain and do not perform their intended role as an energy dissipator, whereas the lateral displacement at the pier top undergoes drastic amplification after the plastic deformation of the shear panels. This suggests that excessive deformation of the shear panels could modify the flexural stiffness along the pier columns, potentially lowering the structure's natural frequency to closer to 2 Hz and amplifying its response. In the case of D-S, an opposite trend is observed. By modelling the shear panels as perfectly elastic springs, the lateral displacement and inertial force at the pier top are increased significantly. A contrasting consequence is shown for the shear panels just reaching their yield strain and not performing their intended role as an energy dissipator. This leads to the finding that the plasticization of the shear panels becomes beneficial as it brings the natural frequency of the structure away from the input motion frequency in the given case. This indicates that the relationship between excitation frequency and structure frequency plays a key role in determining the response of a structure. A comparison of the rest of the response variables is provided in Table 6.

Focussing on the response characteristics and deformation mechanism of the structure and the ground, the third excitation results were selected, as the magnitude and nature of the relationship between the maximum pier top acceleration and the lateral acceleration is contrary to the first and second excitation cases.

Figs. 13 and 14 illustrate the longitudinal distributions of the bending moments and lateral displacement of the piles and the pier, respectively, at the point when the maximum lateral displacement occurs at the top of the pier. The lateral displacement of the piles and the pier is derived through a second-order integration of the values measured

using accelerometers installed on both the piles and the pier, as depicted in Fig. 3. The bending moments of the piles are computed using strain gauges installed along their lengths. In Fig. 13, the trends in the bending moments at both the column base and the piles were effectively reproduced. D-F exhibits a greater bending moment at the column base, while D-S demonstrates a greater bending moment at the piles. Moreover, the bending moments at the pile tips tend to converge. Fig. 14 demonstrates that the analysis satisfactorily captures the trends in displacement and shear deformation of both structures. The F-type exhibits a greater displacement at the pier top and tends to show increased shear deformation at the pier, whereas the S-type exhibits similar pile head displacement to the F-type, probably due to the slight overestimation of the stiffness in the simulation. The behaviour of the pier and the piles aligns with the characteristics of the shear panels discussed earlier. The reduced pile head deflection in D-F may be attributed to the variance in foundation stiffness, where a group of eight piles and pile cap possesses greater stiffness than the 2x2 piles directly connected to the columns in D-S.

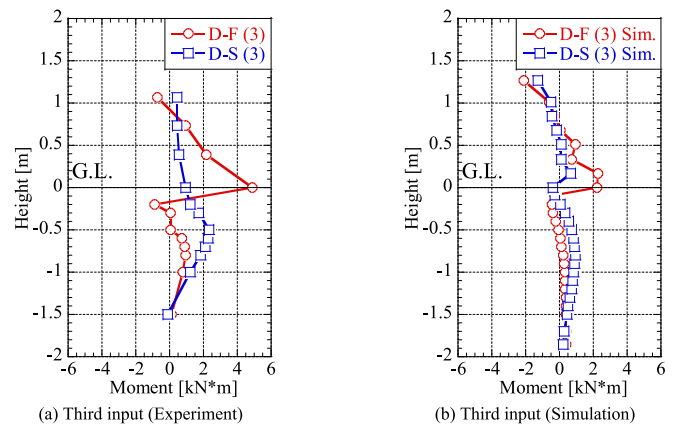


Fig. 13. Distributions of the bending moments generated in the piles and pier when the maximum lateral displacement occurs at the pier top.

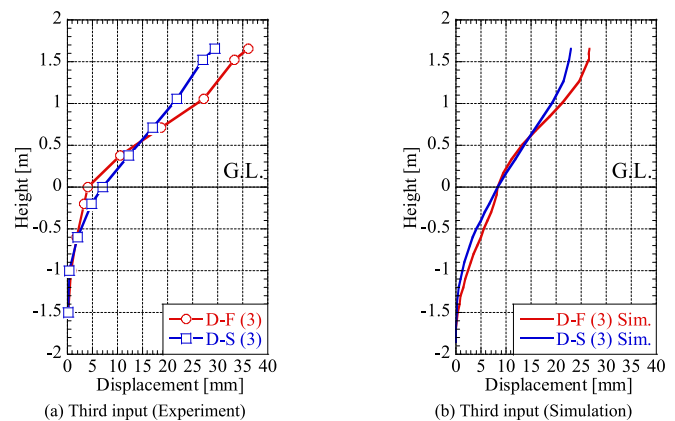


Fig. 14. Distributions of the lateral displacement of the piles and pier when the maximum lateral displacement occurs at the pier top.

The decrease in stiffness in the D-F structure, after the yielding of the shear panels, leads to significant increases in acceleration and lateral displacement at the pier top, while the D-S structure mitigates the displacement effectively due to the minor decrease in stiffness. However, it is possible that these observed effects are influenced by the unusual condition of the excitation occurring between the natural frequencies of the two structures. The experiment does not clarify the relationship among the structure's natural frequency, input wave frequency, and impact of the decrease in stiffness on the natural frequency of the structure. Thus, in the next section, the seismic performance will be evaluated against the varied frequency sine and random waves to address these gaps in knowledge.

4.1.2. Results with L2 seismic input

Fig. 15 displays the time history and the Fourier spectrum of the random input acceleration waveform for bed-rock whose predominant frequency components are 1.59 Hz and 2.07 Hz. Subsequent response analysis results for the random wave are depicted in subfigures Fig. 16 (a) to (d). Fig. 16(a) presents a comparison of the inertial force and lateral displacement at the pier top. The relationship

between the inertial force at the pier top and the shear strain in the upper shear panel is illustrated in Fig. 16(b). Fig. 16(c) shows the distribution of bending moments in the pile and the pier when the maximum lateral displacement occurs at the pier top, while Fig. 16(d) shows the distribution of lateral displacement of the pile and the pier under the same conditions.

It is noteworthy that similar trends were observed for the sine wave as described previously. The response values for each structural member are provided in Table 7. Although the maximum response acceleration (A_{PT}) and maximum lateral displacement at the pier top (D_{PT}), moment at the pier base (M_C), and maximum shear strain in the shear panels (S) are larger for D-F compared to D-S, these values did not reach those for the fully plastic moment (8.41 kN*m) of the pier columns in D-F. The amplification of the response after the yielding of the shear panels could be the reason behind the larger values observed in D-F. Similarly, while the maximum lateral displacement of the pile head (D_{PH}) and maximum bending moment of the piles (M_P) are larger in D-S than D-F, these values also did not reach the fully plastic moment. The lower stiffness of the substructure in D-S would make the

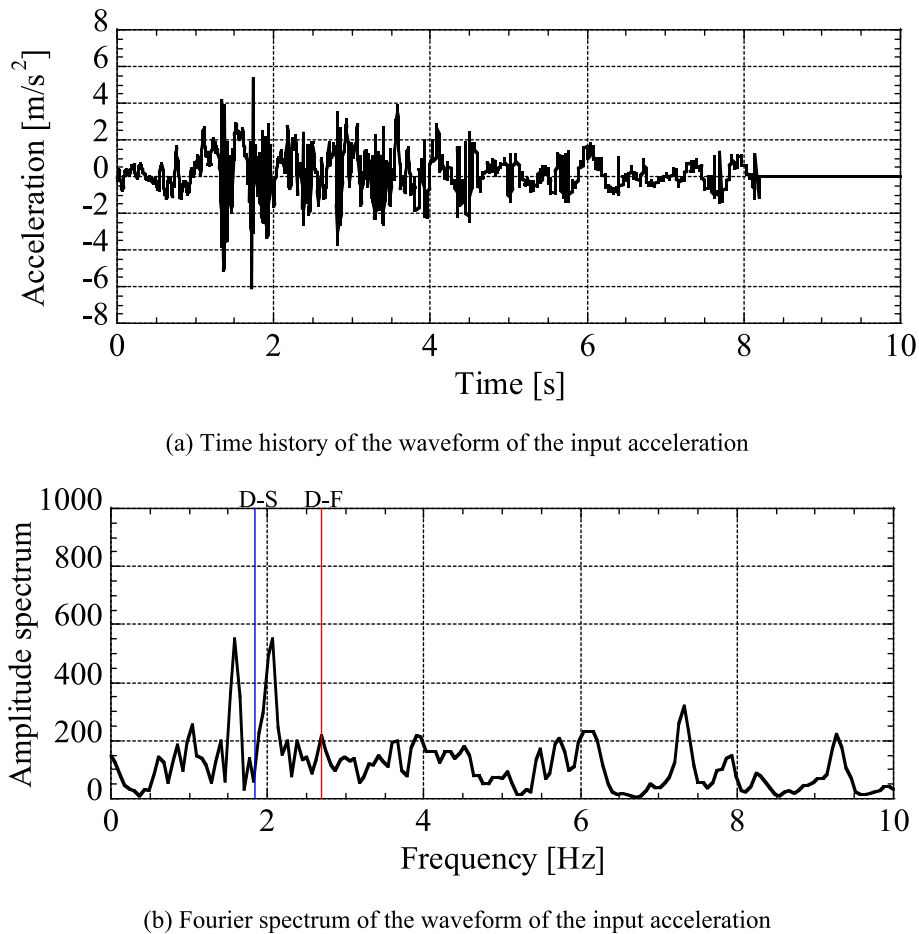


Fig. 15. Level-2 input seismic motion specified in the Guidelines for Aseismic Design for Cut and Cover Tunnels published by Hanshin Expressway Co., Ltd.

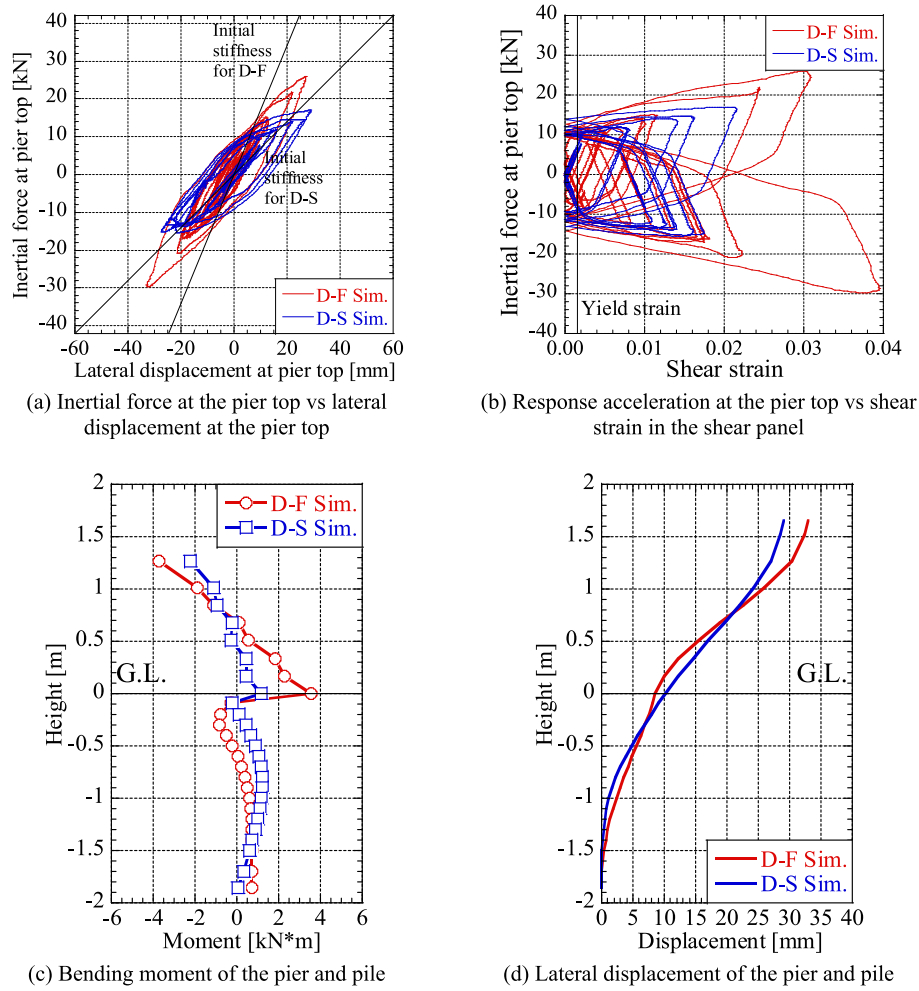


Fig. 16. Response analysis results against Level-2 seismic motion.

Table 7
Comparison of the response values for each member against Level-2 seismic motion.

Part	Item	D-F	D-S
Pier	Maximum response acceleration at the pier top, A_{PT} [m/s^2]	5.61	3.24
	Maximum horizontal deformation at the pier top, D_{PT} [mm]	32.9	29.0
	Moment at the base, M_C [$kN*m$]	3.54	1.17
	Ratio to the fully plastic moment	0.58	0.19
Panel	Maximum shear strain, S	0.039	0.022
	Ratio to the yield shear strain	21.19	12.20
Pile	Maximum horizontal deformation at the pile top, D_{PH} [mm]	8.55	11.5
	Maximum bending moment, M_P [$kN*m$]	0.94	1.23
	Ratio to the fully plastic moment	0.10	0.13

piles deform significantly, thereby resulting in larger bending moments. In summary, the superstructure response amplifications in D-S are significantly reduced compared to D-F, making it a favourable option against the L2 input motion used in this analysis despite the relatively large pile head deflection.

To address the similarity rule for susceptibility to resonance, we conducted an additional analysis on the relationship between the predominant frequency of the seismic motion and the natural frequency of the structure. By vary-

ing the L2 input motion's predominant frequency from 1.2 Hz to 4.85 Hz (adjusting the time step from 0.006 to 0.0015 s), while holding Arias Intensity (Arias, 1970) constant, it was found that the D-F and D-S response trends remained consistent with the original case (0.004 s), as shown in Fig. 17. The predominant components of those additional L2 inputs thoroughly consider the ranges in natural frequency of both D-F and D-S types.

As the predominant frequency decreased to 1.2 Hz, the D-S responses increasingly exceeded those of D-F, particu-

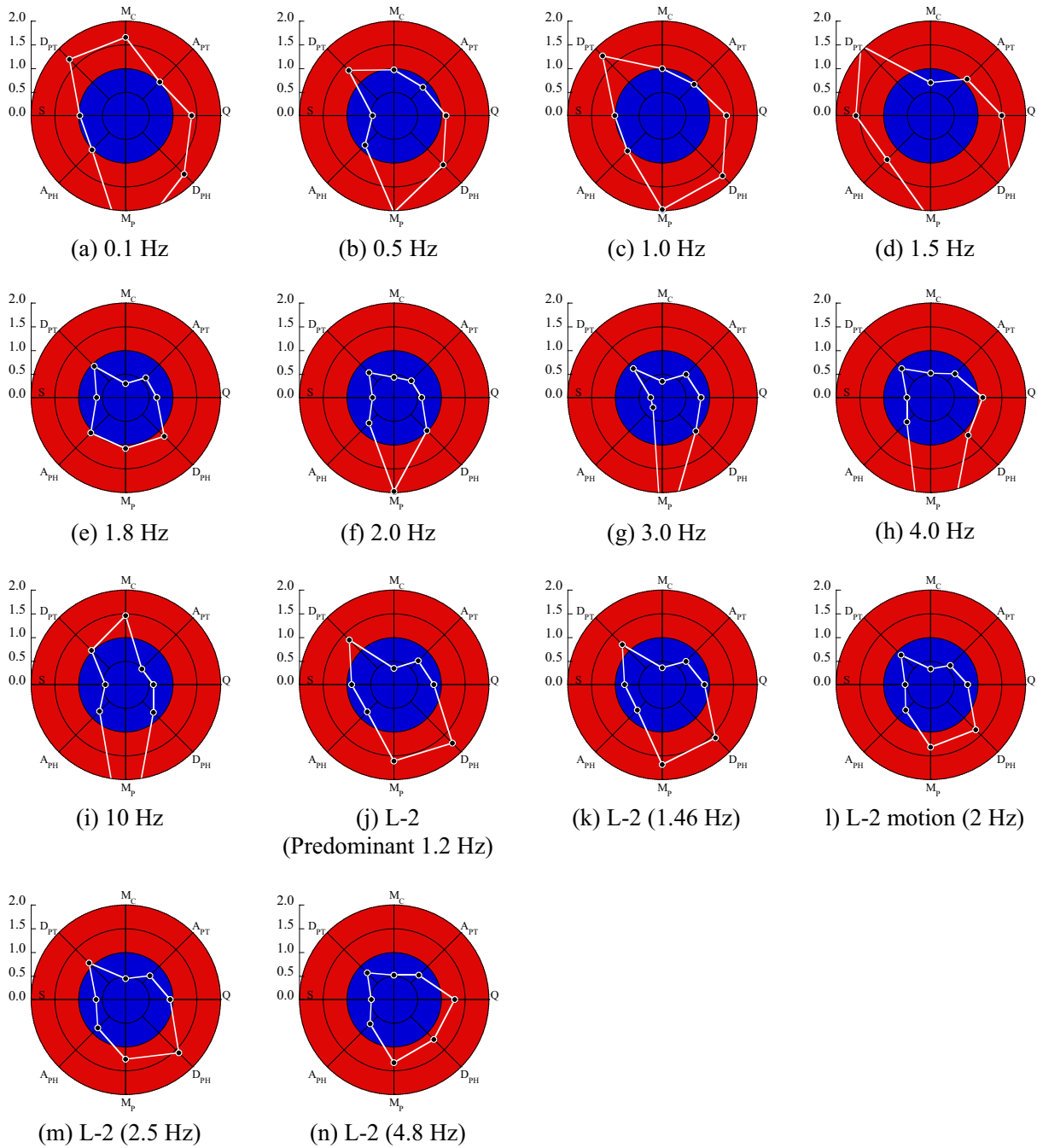


Fig. 17. Comparison of the results of the response analysis against the input waves with various frequencies (D-S/D-F).

larly in pier top displacement, pile bending moment, and pile head displacement, aligning with the reduced natural frequency of D-S after shear panel yielding. The unique behaviour at 1.85 Hz corresponds to its proximity to the reduced natural frequency of D-F. Although the weight similarity rule was not fully met, our findings indicate that this discrepancy did not significantly impact the relative structural responses.

4.1.3. Results with varying input frequencies

Fig. 17(a) to (i) present radar charts displaying the ratios of the response values for each member of the D-S type

compared to those of the D-F type across different frequencies of the input sine wave. The charts suggest that lower input wave frequencies result in smaller response values for the footing type, whereas higher input wave frequencies lead to smaller response values for the footingless type. Furthermore, in Fig. 18, it is evident that the shear strain remains minimal until 1 Hz for both D-F and D-S. However, a notable increase in shear strain occurs at 1.5 Hz for D-S and 1.8 Hz for D-F. The lower response values of the superstructure in D-F (Fig. 17) at lower frequencies can be attributed to the insufficient shear strain of the panels, which fails to alter the stiffness and amplify the struc-

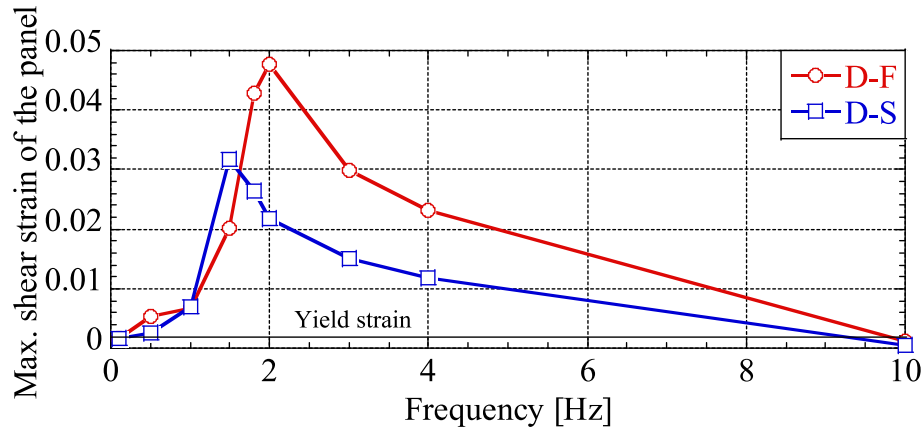


Fig. 18. Variation in the maximum shear strain of the panels against the input frequencies.

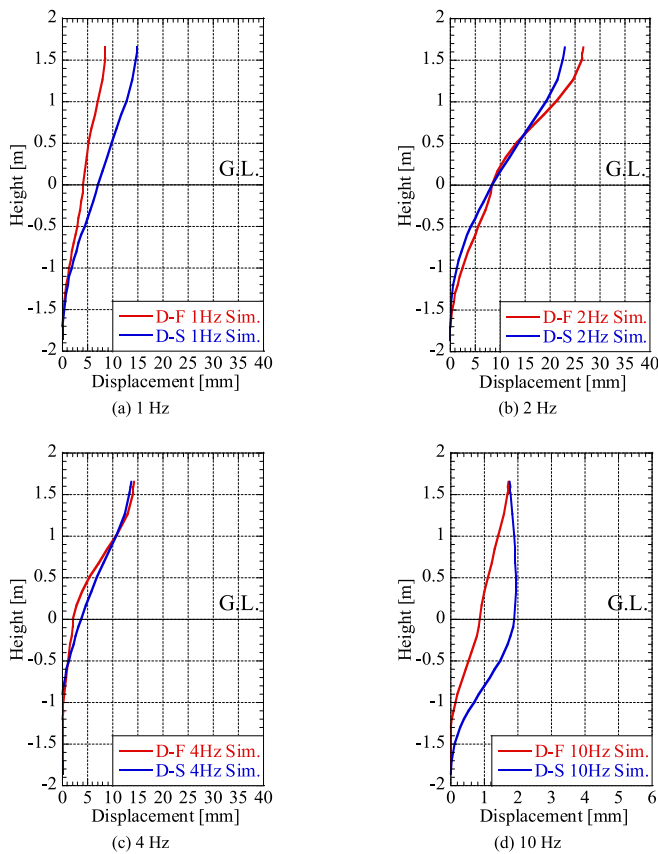


Fig. 19. Lateral displacement profiles of the pier and piles against various frequencies of input motion.

ture's response. Additionally, the proximity of the input frequency to the system frequency governs the system's response, resulting in the amplification of the inertial force (Baziar et al., 2018). It is presumed that the yielding of the panels reduces the system frequency closer to 1.5 Hz for D-S and 2 Hz for D-F, leading to resonance phenomena and significantly larger amplifications in the structure.

Likewise, Fig. 19 presents the lateral displacement profiles of the piles and the pier across various input motion frequencies. The profile at 1 Hz reveals a relatively substan-

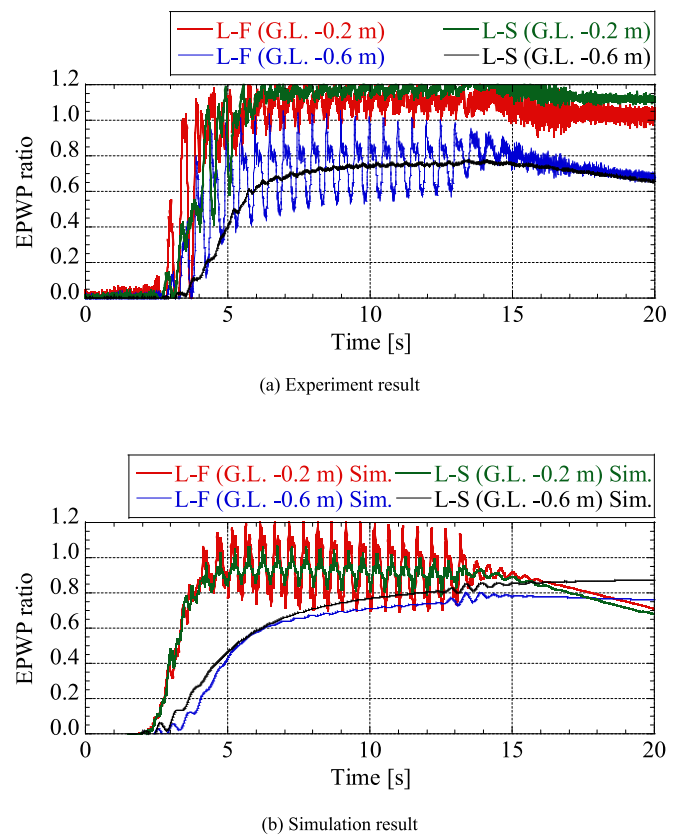


Fig. 20. Comparison of the time histories of the excess pore water pressure ratio of the ground for the experimental and simulation results.

tial lateral deformation of both the piles and the pier in D-S type compared to D-F. At 2 Hz, the lateral deformation increases in both structures, particularly at the pier top, where the displacement exceeds three times its value at 1 Hz for the D-F type. This increase is directly related to the substantial shear strain of the panels, as depicted in Fig. 18. At 4 Hz, the same tendency as 2 Hz is observed, but the amounts of displacement at the pier top and the pile head at 4 Hz are less than those at 2 Hz. At 10 Hz, the magnitude of the lateral displacements is further

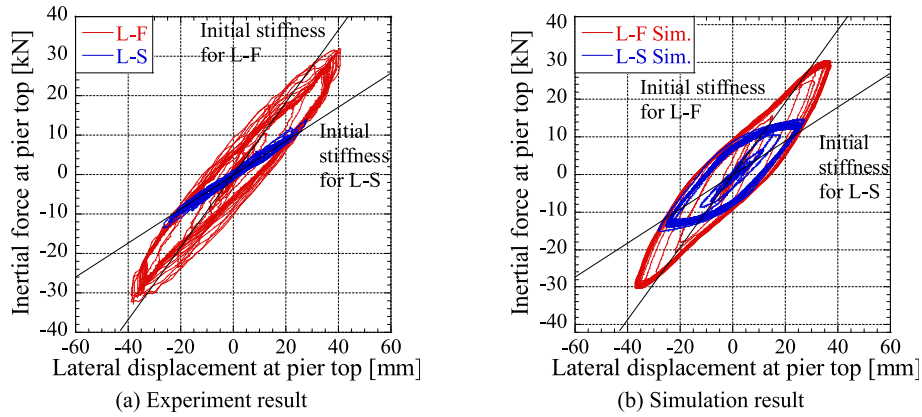


Fig. 21. Comparison of the experimental and simulation results for the relationship between the inertial force and the lateral displacement at the pier top for the case with liquefiable sand.

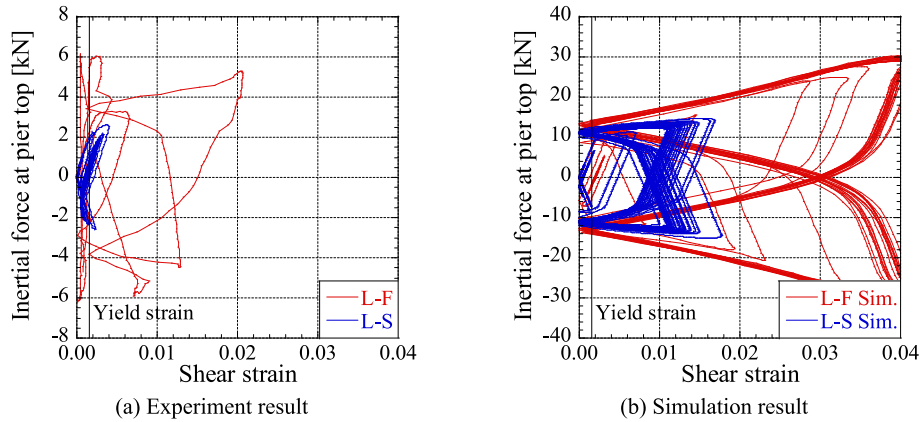


Fig. 22. Relationship between the inertial force at the pier top and the shear strain in the upper shear panel for the case with liquefiable sand.

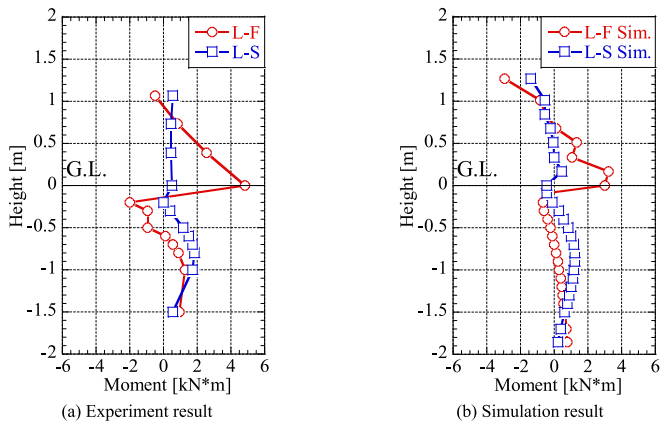


Fig. 23. Distributions of the bending moments generated in the piles and pier when the maximum lateral displacement occurs at the pier top for the case with liquefiable sand.

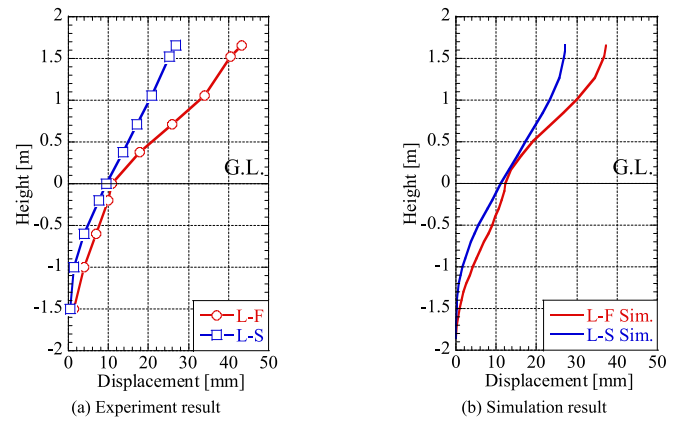


Fig. 24. Distributions of the lateral displacement in the piles and pier when the maximum lateral displacement occurs at the pier top for the case with liquefiable sand.

reduced and the deformation pattern exhibits slight changes. The shear deformation of the pier is notably minimal compared to that of the piles, indicating less influence of the inertial action. Conversely, the additional reduction in pile head deformation could be attributed to the heightened kinematic action of the soil as the ground deforma-

tion becomes excessive near its natural frequency (8 Hz). Consequently, the pier structure exhibits a secondary mode, as noted by Hussien et al. (2015). Despite the favourable soil conditions, according to the Japanese design code, both kinematic and inertial interactions appear to influence the soil response, as noted by

Toyooka et al. (2012). Furthermore, supporting evidence for the observed response is found in the findings of Murono et al. (2000), Tokimatsu et al. (2005), and subsequent studies, suggesting that, when a superstructure's frequency is lower than that of the ground, the ground displacement tends to be out of phase with the inertial force, and vice versa.

4.2. Liquefiable ground cases

Fig. 20 presents a comparison of the time histories of the excess pore water pressure ratio (EPWPR) of the ground between the experimental and the analytical results. The EPWPR at G.L. -0.2 m indicates a liquifiable layer, while that at G.L. -0.6 m indicates a non-liquifiable layer. In both L-F and L-S, the soil reaches liquefaction at approximately 5 s, which is consistent for both the experimental and simulation analyses. The natural periods of the L-F and L-S models after liquefaction were determined as 0.481 sec and 0.683 sec, respectively, based on observations of the free vibration phase immediately after the main

shaking during experiments (Isobe et al., 2016). The numerical simulation estimated these periods as 0.45 sec and 0.60 sec for L-F and L-S, respectively, which are slightly underestimated but reasonable given the simplifications and assumptions applied here.

Fig. 21 illustrates the relationship between the inertial force and the lateral displacement of the pier top and compares the simulated responses with the experimental observations. Similar to the cases with the D-ground, the analysis tends to overestimate the initial stiffness and hysteresis loop of the L-S, but accurately reproduces the hysteresis loop of the L-F. Furthermore, the tendencies for the larger maximum acceleration and maximum lateral displacement in L-F than in L-S are successfully reproduced, aligning with the experimental results.

Fig. 22 displays the relationship between the inertial force at the pier top and the shear strain in the upper shear panel. Similar to the D-ground case, the reproduction analysis slightly overestimates the shear strain in both structures, with a more pronounced tendency in L-F. Although measured strain values in the large strain region

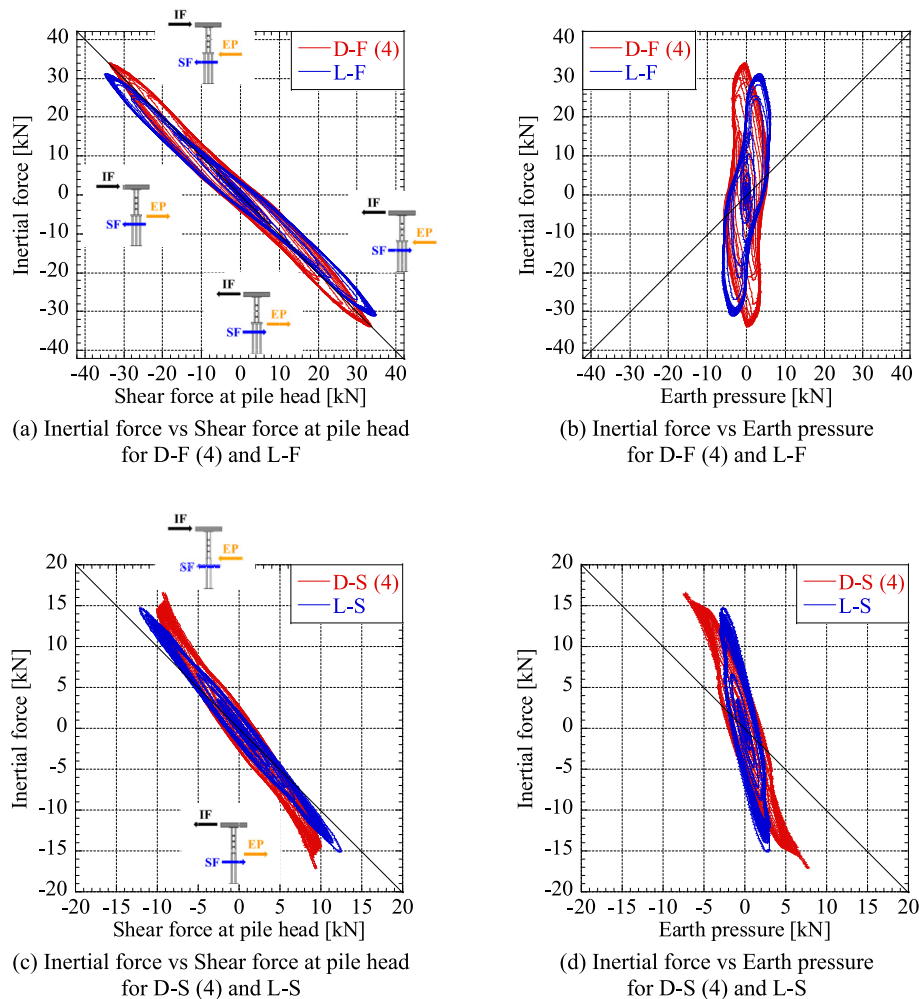


Fig. 25. Relationships between the inertial force and the shear force at the pile head and between the inertial force and the earth pressure for D-F, D-S, L-F, and L-S.

of the experiment were unavailable, due to the detachment of the strain gauges on the shear panels after out-of-plane deformation, the analysis successfully replicated the trend of the shear strain values to exceed the yield strain in both cases, with larger shear strain being observed in L-F than in L-S.

Figs. 23 and 24 depict the longitudinal distributions of the bending moments and lateral displacement, respectively, in the piles and the pier, at the occurrence of the maximum lateral displacement at the pier top. The bending moment in the pier base is larger for L-F compared to L-S, while the moment in the piles is larger for L-S than for L-F. Additionally, the bending moment tends to converge at the pile tip similar to the experimental case where the pile tips are fixed. However, there is a slight difference that is probably due to an imperfect modelling of the relative stiffness between the piles and the ground in the deeper sections of the model. The analysis successfully captures the observed tendencies in the bending moment distribution in the piles and the pier, the nearly equivalent levels of lateral displacement values at the pile heads for both L-F and L-S, a larger displacement at the pier top for L-F, and a tendency for L-F to exhibit larger shear deformation in the column than L-S.

Unlike the D-ground cases, the displacement at the pile heads for L-F is larger than that for L-S. This difference can be attributed to several factors. One factor is the excessive response displacement of the unsaturated surface layer above the liquefied ground, which may lead to increased subgrade reaction forces around the pile heads. Another factor is that the liquefaction-induced reduction in the subgrade reaction forces beneath the piles could contribute to the increased pile head deformation under the influence of the inertial force of the footing. This hypothesis is supported by studies indicating that liquefaction decreases the lateral resistance provided by the surrounding soil, leading to increased pile flexibility (Wilson et al., 2000; Rollins et al., 2005). Studying the role of the subgrade reaction force as kinematic action may further corroborate these findings in the next section.

4.3. Kinematic interaction

Kinematic interaction refers to the effects of relative displacement and velocity between the soil and the structure during dynamic loading, such as earthquakes. It plays a crucial role in influencing the behaviour and response of structures, particularly piles and foundations, subjected to seismic forces. Wang et al. (2017) noted that pile deformation is influenced by both kinematic and inertial actions in liquifiable sand. Fig. 25 illustrates the relationships between the inertial force and the pile head shear, and between the inertial force and the earth pressure, during the fourth excitation. Earth pressure is calculated as the difference between the inertial force and the net pile head shear, following the approach of Tokimatsu et al. (2005). It shows that, for D-S, the inertial force exceeds the shear

force at the pile heads, suggesting that the surrounding ground provides a supporting mechanism, with the earth pressure causing a decrease in the pile bending moments and shear forces. In contrast, for D-F, the earth pressure tends to be out of phase with the inertial force, causing most of the inertial force to transfer directly to shear force. While earth pressure occasionally acts as a supporting force, its magnitude is relatively low.

In the case of the L-S type, the earth pressure counteracts the inertial force, thereby reducing the shear force transmitted at the pile heads. However, the magnitude of the earth pressure force is reduced by 62 % compared to the D-S type. On the other hand, the earth pressure is in alignment with the inertial force, thereby increasing the pile head shear force for L-F. Earth pressure and inertial force are not exactly in the same phase; however, they have a greater tendency to act in the same direction. When the earth pressure acting as kinematic force aligns with the inertial force, it acts as additional force on the structure, making it an unconventional and critical case.

Although the kinematic forces in L-F and D-F are similar in magnitude, their impact appears more pronounced in liquefiable grounds for several reasons. Firstly, the pile head deformation in L-F exceeds that in D-F. Secondly, the inertial force in L-F is slightly lower than that in D-F and, importantly, the earth pressure in L-F acts in the same direction as the inertial force. Kinematic force may not serve as a supporting force when aligned with the inertial force, depending on the relationship between the natural frequency of the structure and the input motion. Conversely, studies by Kaynia et al. (1996) and Hussien et al. (2015) suggest that when a structure's natural frequency is lower than that of the ground, the kinematic interaction can be the primary contributor to the seismic force in the piles, particularly when the excitation frequency does not closely match the natural frequency of the coupled soil-pile-structure system (fSSI).

For F-type structures, the input seismic excitation falls within a relatively narrow band of the soil-pile-structure system's natural frequency compared to S-type structures, meaning that the inertial interaction generally prevails. Based on the above discussion, it is evident that the earth pressure acting as a kinematic force on the footing block, aligned with the inertial force, amplifies the pile head deformation more in L-F than in L-S.

Overall, the S-type structures demonstrate the potential for an improved seismic performance, while leveraging the stiffness and damping of the underground beam and under the favourable influence of the kinematic action in the soil. It should be noted that this discussion provides a preliminary overview of kinematic forces in F- and S-type structures under the specified conditions, without extensively examining the relationship between the natural frequency of the structure and ground excitation. A detailed analysis of the influence of the inertial and kinematic interactions on the structural response will be addressed in a separate manuscript.

5. Conclusions

In order to comprehensively understand the seismic response and deformation mechanisms of a bridge pier with directly-connected piles, a soil–water coupled dynamic finite element analysis was conducted. The impact of the natural frequency, changes in the input wave frequency, and Level-2 seismic motion on the performance of the proposed structure was examined via a parametric study by comparing it with a conventional pile foundation. The key findings are as follows.

- (1) The numerical simulation qualitatively and quantitatively reproduced the shaking table test results with sufficient accuracy.
- (2) The simulation clarified that the preference of the S-type bridge pier over the F-type one was seen to depend on the relationship between the natural frequency of the structure and the input wave frequency. This finding therefore suggests the necessity of analysing the superstructure, substructure, foundation, and ground as one unit for dynamic analyses.
- (3) It was found that the F-type bridge pier incorporates a mechanism that effectively utilizes the hysteretic damping function of the shear panels, resulting in the formation of a plastic hinge at the base of the pier in the final state. However, in instances where significant strain leads to out-of-plane deformation or cracks in the shear panels, due to a sharp reduction in the overall structure's stiffness, there may be a sudden increase in the inertial force and lateral displacement at the pier top. Therefore, the reduction in stiffness and the variation in natural frequency of the entire structure, due to the plastic deformation of the shear panels, should also be considered during the design phase.
- (4) The S-type bridge pier maintains the efficacy of the hysteretic damping function of the shear panels for a relatively extended period of time by leveraging the strength of the underground beam, owing to the gradual reduction in the overall structure's stiffness.
- (5) The numerical simulation showed that the S-type model exhibited a better performance against deformation in soft grounds, where the excessive response displacement and liquefaction of the ground are of concern, courtesy of the favourable influence of the kinematic force and the lower resistance area. The lateral displacement of the piles in the F-type model, having a large superstructure mass-to-footing mass ratio, could be partly attributed to the heightened kinematic action of the soil at the footing block. In real structures, with superstructure mass-to-footing mass ratios close to one, the influence of the footing's inertial force and kinematic action on the lateral displacement of the piles is expected to be enhanced. These findings underscore the need to revise the

design methodology for the proposed structure, ensuring proper consideration of the effect of the inertial and kinematic interaction.

CRedit authorship contribution statement

Muhammad Mahmood Ul Hassan: Writing – review & editing, Writing – original draft, Visualization, Validation. **Koichi Isobe:** Writing – review & editing, Writing – original draft, Visualization, Validation, Supervision, Project administration, Funding acquisition, Conceptualization. **Yasumasa Soga:** Writing – review & editing, Project administration, Funding acquisition, Conceptualization. **Yasuo Sawamura:** Writing – review & editing, Visualization, Validation, Supervision, Methodology. **Hiroki Sugiyama:** Writing – review & editing, Project administration, Methodology. **Masatsugu Shinohara:** Writing – review & editing, Validation, Funding acquisition, Conceptualization.

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Appendix. To examine the influence of cushioning material on a structure, a numerical simulation without cushioning material was performed and compared against a simulation model with cushioning material for case D-F. Under seismic excitation of 2 m/s^2 , the soil element near the boundary was seen to exhibit a uniform stress–strain pattern for the model with cushioning material, mirroring the real ground conditions. However, an erratic stress–strain pattern was recorded in the cushion-less case, and significant shear stress was observed near the boundary due to the rigid confinement of the soil box (Fig. A1(a)). Consequently, the rationale for incorporating cushioning material was validated. Nevertheless, even in instances without cushioning material, neither the pile heads nor the superstructure exhibited any substantial influence in their response (Fig. A1(b)). Consequently, the contribution of a rigid soil tank to structural deformation appears to be relatively insignificant.

The wide top weight used in the experiment (Fig. 3) implies significant rotational inertial, which could affect the pier structure's response. However, a comparison of the responses with a solid element superstructure to a point mass model for cases L-F and D-S showed negligible differences (Figs. A2 and A3). Thus, a point mass approach was adopted for reproducing the proposed structure's analysis.

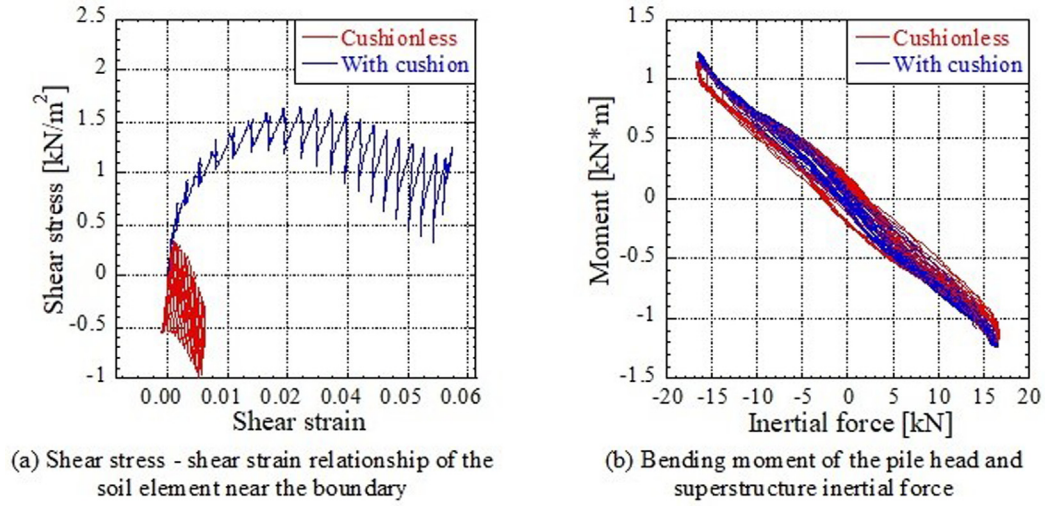


Fig. A1. Comparison of the soil and structure's responses with and without cushion.

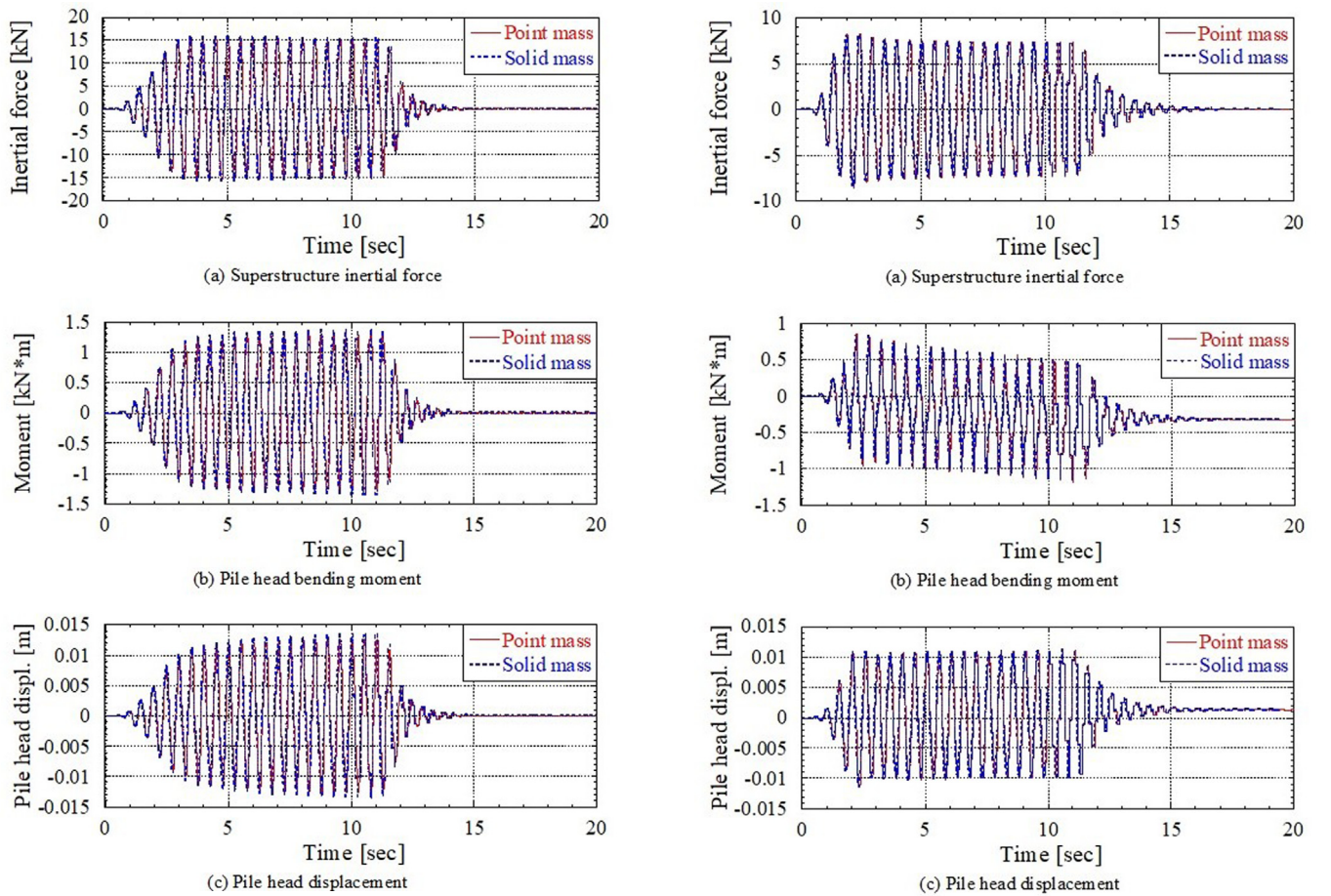


Fig. A2. Comparison of the superstructure and the pile head responses for the point mass and solid mass superstructure case (L-F).

Fig. A3. Comparison of the superstructure and the pile head responses for the point mass and solid mass superstructure case (D-S).

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