Shaking table test and numerical simulation on seismic performance of a bridge column integrated by multiple steel pipes with directly-connected piles

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ABSTRACT

A bridge column integrated by multiple steel pipes and connected directly to piles without a footing has been proposed to design a rational foundation of the column. Based on the past achievements, the proposed substructures possess an excellent advantage of reduction of strain at the column through strain decentralization at footing point. In addition, reduction in footing weight contributes to decrease pile strain. On the other hand, the proposed substructure has some disadvantages i.e. increase in strain and displacement of piles. But it has been revealed that the strain generated at piles can be minimised by using a beam in the ground. In this paper, the seismic performance of the bridge column structure grounded in liquefiable sand is evaluated based on the large-scale shaking table tests using a bridge column model with the scale of 1/20. Subsequently, a soil-water coupled FE analysis is conducted to reproduce the experimental results and confirm the seismic performance and the detail of the mechanism.

Keywords: damage-controlled structure, pile foundation, footing-less, liquefaction, shaking table test, FE analysis

1 INTRODUCTION

A bridge column integrated by multiple steel pipes and multiple shear panels interconnecting the pipes (Fig. 1) has been proposed and put into practical use (Shinohara et al., 2012). The bridge column is designed based on damage-control concept, in which the vertical load such as dead load and traffic load is supported by multiple steel pipes, and lateral load such as seismic load is adjunctively supported by shear panels made of low yield steel. Seismic damage will be aggregated on only the shear panels so that it enables early recovery by replacing only the shear panels after an earthquake.

Subsequently, a bridge column integrated by multiple steel pipes and connected directly to piles without a footing has been proposed to design a more rational foundation (Shinohara et al., 2013). Based on the past achievements, the proposed substructures have advantage of strain reduction at the column by strain decentralization at footing point. In addition, reduction in footing weight contributes to decrease pile strain. On the other hand, the proposed substructure has displacement of piles. But it has been revealed that strain of piles could be decreased by using a beam in the ground.

In this paper, the seismic performance of the structure in liquefiable sand is evaluated based on the

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large-scale shaking table tests using a bridge column model with the scale of 1/20, comparing to a conventional pile foundation with a footing. Subsequently, a soil-water coupled FE analysis is conducted to reproduce the experimental results and confirm the seismic performance and the detail of the mechanism.

2 OUTLINE OF SHAKING TABLE TEST

Fig. 2 shows the bridge column and foundation model with the scale of 1/20 used in the shaking table tests. Two types of models are used; one has a group of pile foundation (8 piles) with a footing and the other has directly connected piles (4 piles) without a footing. In this paper, each model is called F-type and S-type, respectively. The integrated bridge column consists of 4 steel pipes (STK 400) and three-layered shear links (LY225) to interconnect the pipes. In S-type, the underground beam is used to interconnect the piles around the pile head for the purpose of reducing displacement of the pile head. The pile and pipe spacing is set to be $2.5D_p$ ($D_p$: pile diameter).

Fig. 3 shows the schematic view of the shaking table tests. The soil chamber, which measured 4.0 m in length, 1.0 m in width and 2.0 m in depth, has cushioning material made of foam rubber on the sidewall surface in order to reduce the influence of the rigid sidewall during shaking. The liquefiable ground is modeled by pluviation in water using Tohoku silica sand #6 with a $D_r$ of 40 % using tamping method. Ground water level in all test cases is set up to G.L. -0.2 m. The soil physical properties and liquefaction strength curve for $D_r$ 40% of Tohoku silica sand #6 are shown in Table 1 and Figure 4, respectively.

The layout of measuring instruments such as displacement gauges, accelerometers and pore water pressure gauges are also shown in Figure 3. The strain gauges are attached on the column, piles and shear panels. As input, 20 cycles of a 2 Hz sinusoidal tapered wave is used. The target acceleration is 2.0 m/s$^2$.

Fig. 4. Liquefaction strength curve for $D_r$ 40% of Tohoku silica sand #6.

Table 1. Soil physical property of Tohoku silica sand #6.

| Property                     | Value |
|------------------------------|-------|
| Maximum dry density $\rho_{max}$ [g/cm$^3$] | 1.695 |
| Minimum dry density $\rho_{min}$ [g/cm$^3$] | 1.401 |
| Soil particle density $\rho_s$ [g/cm$^3$] | 2.630 |
| Maximum void ratio $e_{max}$ | 0.878 |
| Minimum void ratio $e_{min}$ | 0.551 |
| Uniformity coefficient $U_c$ | 1.52  |

Fig. 2. Schematic view of the model used in the tests.
3 TEST RESULTS AND DISCUSSION

Fig. 5 shows the time history of excess pore water pressure ratio of the ground in the G.L. -0.2 m and G.L. -1.0 m. It is obvious that the excess pore water pressure ratio of the ground in the G.L. -0.2 m sharply increases after the shaking and reaches 1 at around 5 seconds. This indicates that the ground is fully liquefied. On the other hand, although the excess pore water pressure slightly increases in the layer with a $D_r$ of 80 % assuming a non-liquefaction layer (GL-1.0 m), the layers are not completely liquefied. Thus, it proves that ground preparation and ground motion was performed as expected.

Fig. 6 shows the time history of the response acceleration at the column top. Fig. 7 shows the time history of the lateral displacement at the column top and the pile top. Fig. 8 shows the time history of the relative displacement of the column top to the pile head. In the immediate aftermath of the ground motion L-S is delayed in response than the L-F, the response displacement of the column top and the pile head of L-S is greater than that of L-F. However, the magnitude relationship is reversed as the ground motion progresses. In particular, the tendency is pronounced in the results of the column top. Meanwhile, the same inversion phenomenon is confirmed even in the pile head, almost the same amount of displacement is finally obtained. This is worthy of special mention because L-F has twice the pile number of L-S and a footing with a large resistance area and mass. In response to those results, similar reversals in the time history of the relative displacement for L-F at the final stage is approximately 2 times of L-S.

Fig. 9 shows the relationship between the response acceleration and lateral displacement at the column crest. Although the initial stiffness of the L-F is greater than that of L-S, non-linear behavior is observed along with the progress of displacement and the history loop is expanded. On the contrary, the expansion of the history loop is not observed and the linear elastic behavior is shown as a structural whole system in L-S. The cause of the difference between these two vibration characteristics is discussed below from the aspect of the behavior of the shear panels.

Fig. 10 shows the relationship between the relative displacement of the column top to the pile head and the shear strain on the shear panels and the underground beam, indicating the yield strain of the panels and the underground beam (1770 $\mu$ and 2590 $\mu$, respectively). All of the shear panels in Case of L-F yields during the third wave, and the shear strain rapidly increases as shown in Fig. 11, whereas all of the panels in Case of L-S yields during the fourth wave, but the shear strain of the panels does not increase. The shear strain of the underground beam does not reach the yield strain. And the relative displacement when the shear strain of the panel reach the yield strain for L-F and for L-S is 7 mm and 13 mm, respectively. This is because L-S is much more flexible around the pile head and the column base than L-F in which a column is rigidly connected on the footing by welding. It indicates that the deformation modes are different between these structures.
Fig. 9. Relationship between response acceleration and lateral displacement at the column top.

Fig. 10. Relationship between the relative displacement of the column top to the pile head and the shear strain on the shear panels and the underground beam.

Fig. 11. Shear panels before and after shaking.

Fig. 12. Strain on the column and piles while generating the maximum lateral displacement (Left: L-F, Right: L-S).

Based on the fact that the main members such as the columns and piles yield after the shear panels (secondary member) yield, the proposed structure has a damage control performance by energy absorption due to plastic deformation of the shear panels. In particular, S-type has high seismic performance because the main member (columns and piles) holds a large residual strength even after yielding of the shear panels.

4 OUTLINE OF NUMERICAL SIMULATION

The DBLEAVES soil-water coupling FE analysis code (Ye et al., 2007) was used in the study’s simulation. 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. (2002), was used as the constitutive model. The soil parameters are determined based on the results of isotropic consolidation test, CU triaxial tests and cyclic triaxial tests to the specimens with $D_r$ of 40% and 80%. For parameters for which detailed information is unavailable, the properties of Toyoara sand has been used which has a similar grain size distribution as Tohoku sand. The soil parameters are shown in Table 2. The cushion attached on the side wall is also modeled by elastic solid elements ($E = 0.05$ GPa, $v = 0.49$, $\rho = 0.07$ g/cm$^3$).

In the analysis the integrated column, the underground beam and the shear panels are simply modeled by the bi-linear type of elasto-plastic beam and spring elements, respectively. The steel pile in the ground is modeled by the hybrid element which can consider the soil-pile interaction adequately (Zhang et al., 2000). The parameters are shown in Tables 3 and 4. The weight (52.6 kN) fixed on the structure is modeled by a single mass.

By considering symmetry of geometrical and loading conditions, only half of the domain is used in the analysis. Fig. 13 shows the finite element mesh used in the simulation. The boundary conditions are as...
follows: (a) the bottom of the ground is fixed, (b) the vertical boundaries parallel to the XOZ plane are fixed in the y direction and free in the x and z directions, (c) the vertical boundaries parallel to the YOZ plane are fixed in the x direction and free in the y and z directions and (d) the ground surface above the water table is set with a drainage condition, while the other surfaces are impermeable.

Table 2. Soil parameters.

| Parameter | Common | D. 40% | D. 80% |
|-----------|--------|--------|--------|
| $\lambda$ | 0.0500 | 0.9    | 0.7    |
| $\kappa$  | 0.0064 | 1/10   | 1.19   |
| $R_0$     | 3.290  | 0.00   | 0.00   |
| $N$       | 0.74   | $10^{-4}$ | $10^{-4}$ |
| $\nu$     | 0.30   | 2.01   | 1.93   |
| $m$       | 0.01   | 1.01   | 0.93   |
| $a$       | 2.20   |        |        |
| $b_r$     | 1.50   |        |        |

Table 3. Parameters of the column and the pile.

| Parameter | Column | Pile |
|-----------|--------|------|
| Material  | STK400 | STK400 |
| Model type| Beam   | Hybrid |
| Diameter (mm) | 89.1 | 76.3 |
| Thickness (mm) | 2.8  | 2.8  |
| Cross section stiffness (kN) | $1.5 \times 10^6$ | $1.3 \times 10^6$ |
| Flexural rigidity (kN-m²) | $1.4 \times 10^6$ | $8.7 \times 10^5$ |
| Yield moment (kN-m) | 3.73  | 2.69  |

Table 4. Parameters of the panel and the underground beam.

| Parameter | Panel | Underground beam |
|-----------|-------|------------------|
| Material  | LY225 | SS400            |
| Model type| Spring| Beam             |
| Size (mm) | 61.0 x 61.0 | 77.0 x 147.5 |
| Thickness (mm) | 11.1 | 11.1 |
| Shear stiffness (kN/m²) | $7.7 \times 10^6$ | $1.2 \times 10^6$ |
| Yield shear stress (kN/m²) | $3.2 \times 10^4$ | $2.3 \times 10^4$ |
| Flexural rigidity (kN-m²) | $2.3 \times 10^5$ | $1.0 \times 10^5$ |
| Yield moment (kN-m) | 0.68  | 0.68  |

The initial stress field is calculated in advance via self-weight static analysis. The value of the initial OCR is based on that of the initial stress and the consolidation yield pressure. The input wave observed on the shaking table is applied at the bottom of the analytical domain. Stiffness proportional damping with the damping constant and the proper period observed during the free vibration in the tests was applied for the damping model, and the Newmark-$\beta$ method was used for time integration. The time increment is 0.005 sec.

5 SIMULATION RESULTS AND DISCUSSION

Fig. 14 shows the time history of the response acceleration at the column top obtained from numerical simulation. As with the experimental results, in the immediate aftermath of the ground motion L-S is delayed in response than the L-F, the response displacement of the column top and the pile head of L-S is greater than that of L-F. However, the magnitude relationship is reversed as the ground motion progresses.

Fig. 15 shows the comparison of the relationship between the response acceleration and lateral displacement at the column crest. It is obvious that the numerical analysis results are in good agreement with the experimental results for both cases (L-F and L-S). It means that although the initial stiffness of the L-F is greater than that of L-S, L-F behaves non-linearly along with the progress of displacement and the history loop is expanded. On the other hand, the history loop is slightly expanded in L-S and the whole structural system behaves relatively linear.

Fig. 13. Finite element mesh used in the simulation (L-F).

Fig. 14. Time history of the response acceleration at the column top obtained from numerical simulation.

Fig. 15. Comparison of test and simulation: Relationship between response acceleration and lateral displacement at the column top.
Fig. 16 shows the comparison of the relationship between the response acceleration and the shear strain on the upper stage panel. For the both cases, the simulation results reproduce the relationship well. The shear strain for L-S is considerably suppressed compared to that for L-F.

On the other hand, the bending moment distribution on the column and the piles, at which time the maximum lateral displacement happens, is different from the test results. The bending moment around the pile tip is overestimated by the simulation. This is because the stiffness of the non-liquefiable layer with Dr of 80% is underestimated, considering the results of the time history of the excess pore water pressure ratio at G.L. -1.5 m as shown in Fig. 18. Therefore, it is essential to evaluate the rigidity of the soil material under low confining pressure and further research need to be performed in this aspect. According to the additional simulation results, the relative stiffness of the structures to the ground and the stiffness of the pile head of the proposed structure (S-type) including the underground beam, have been found to sensitively affect the analysis results. Depending on these factors, there is a possibility that the magnitude relationship of the seismic response for these structures is reversed.

Further research is being done in these aspects and the achievements of these types of proposed structures in engineering practice will be published in future.

6 CONCLUSIONS

A bridge column integrated by multiple steel pipes and connected directly to piles without a footing is proposed as the reasonable damage-controlled structure. In this paper, the seismic performance of the structure in liquefiable sand is evaluated based on large-scale shaking table tests and a soil-water coupled FE analysis. As with the past achievements, the proposed substructures have advantages of strain reduction of column by strain descentralization at footing point. It is recognized that it has high seismic performance and high toughness if the conditions are right in view of the fact that the main member (columns and piles) holds a large residual strength after yielding of the shear panels.

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