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EXPERIMENTAL STUDY ON SOIL-PILE-STRUCTURE INTERACTION IN LIQUEFIABLE SAND SUBJECTED TO BLAST-INDUCED GROUND MOTION

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SUMMARY

Vibration tests of a pile-supported structure in a liquefiable sand deposit were performed at Black Thunder Mine in Wyoming USA. Ground motions from large-scale mining blasts were used as input motions for the test structure. The test structure was constructed in an excavated 3m-deep pit and the pit was backfilled with water-saturated sand. Accelerations of the test site, the test structure and one of the piles were measured. Pore water pressures in the pit and strains of the pile were also measured. Vibration tests were performed six times with different levels of input motions. The maximum horizontal acceleration recorded at the adjacent ground surface increased from 20 Gals to 1,352Gals as the blast area approached the test site. The excess pore water pressures also increased with the levels of input motions and sand boiling phenomena were observed in the test pit. It was thus clarified that this vibration test method was very effective in verifying the soil nonlinearity including the liquefaction and the dynamic behavior of the pile-supported structure.

INTRODUCTION

It is important in seismic design of pile-supported structures to appropriately evaluate soil nonlinearity including liquefaction during a large earthquake and their effects on the dynamic behavior of pile foundations. Vibration tests using ground motions induced by large-scale mining blasts were performed in order to understand non-linear dynamic responses of pile-structure systems in liquefied sand deposits. Significant aspects of this test method are that vibration tests of large-scale structures can be performed considering soil-structure interaction, and that vibration tests can be performed several times with different levels of input motions because the blast areas move closer to the test structure, and that three-dimensional responses during the actual earthquake can be considered. This paper outlines the vibration tests and presents test results for the soil and super-structure responses and pile stresses.

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VIBRATION TEST METHOD USING GROUND MOTIONS INDUCED BY MINING BLASTS

The vibration test method using ground motions induced by mining blasts is shown schematically in Figure 1. Vibration tests on a pile-supported structure in a liquefiable sand deposit were conducted at Black Thunder Mine of Arch Coal, Inc. [1], [2]. Black Thunder Mine is one of the largest coal mines in North America and is located in northeast Wyoming, USA. At the mine, there is an overburden (mudstone layers) over the coal layers. The overburden is dislodged by large blasts called "Cast Blasts" and the rubble is removed by huge earthmoving equipment. After the coal surface is exposed, smaller blasts called "Coal Shots" are applied to loosen coal layers. The coal is then mined out by a truck and shovel operation. The ground motions induced by Cast Blasts were used for vibration tests conducted in this research. The smaller Cast Blasts or Coal Shots were used for checking and calibrating instrumentation.

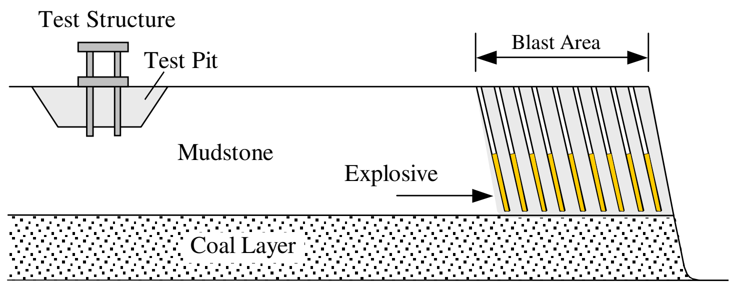

Figure 1 Vibration Test Method at Mining Site

OUTLINES OF VIBRATION TESTS

A sectional view and a top view of the test pit and the pile-supported structure are shown in Figure 2 and Figure 3, respectively. A 12x12-meter-square test pit was excavated 3 meters deep with a 45-degree slope, as shown in Figure 2. A waterproofing layer was made of high-density plastic sheets and was installed in the test pit in order to maintain 100% water-saturated sand.

Outlines of the pile-supported structure are shown in Figure 4. Four piles were made of steel tube. Pile tips were closed by welding. Piles were embedded 70cm into the mudstone layer. The top slab and the

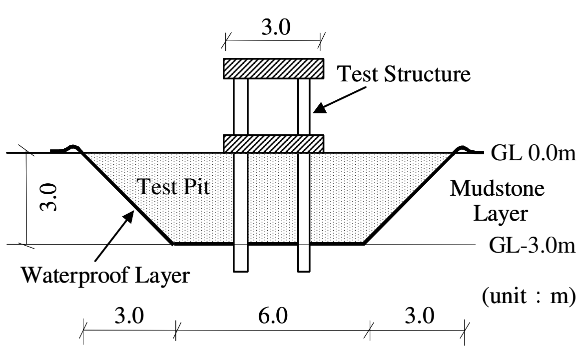

Figure 2 Sectional View of Test Pit

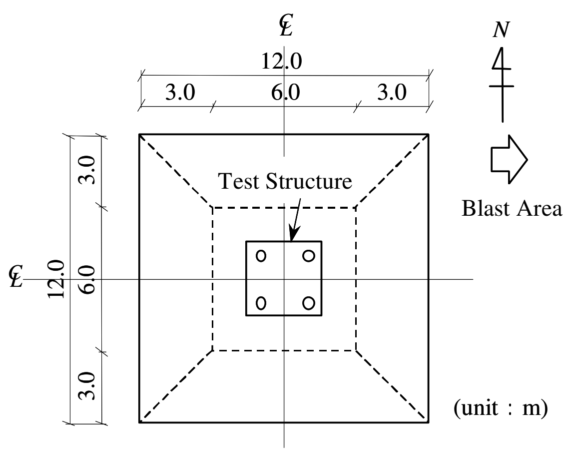

Figure 3 Top View of Test Pit

base mat were made of reinforced concrete and were connected by H-shaped steel columns. The structure was designed to remain elastic under the conceivable maximum input motions, and the main direction for the structure is set in the EW direction. The construction schedule was determined so that the structure under construction received the least influence from mining blasts.

Instrumentation is shown in Figure 5. Accelerations were measured of the structure and one of the four piles. Accelerations in the sand deposit and free field adjacent to the pit were also measured in array configurations. Axial strains of the pile were measured to evaluate bending moments. Excess pore water pressures were measured at four levels in the test pit to investigate liquefaction phenomena. Sensors embedded in the test pit were sealed and protected against water-sand mixture. Great care was taken in preparing pore pressure transducers. Each transducer was installed in a plastic casing and the casing was wrapped in silica sand and glass fiber sheet for protection. The air in front of the transducer diaphragms was removed to ensure accurate measurements. This treatment was done by heating and vacuuming in a water-filled glass container [3].

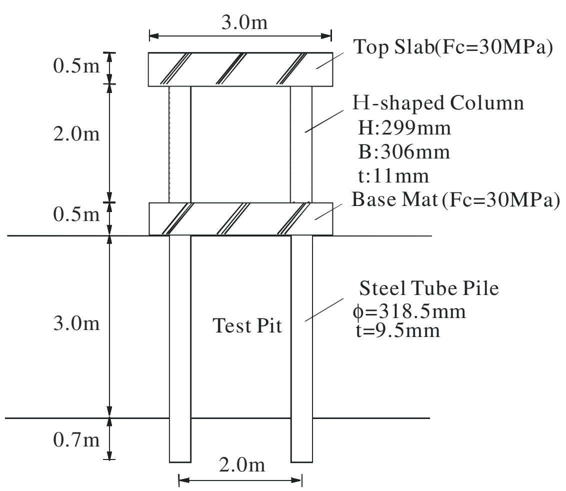

Figure 4 Pile-Supported Structure

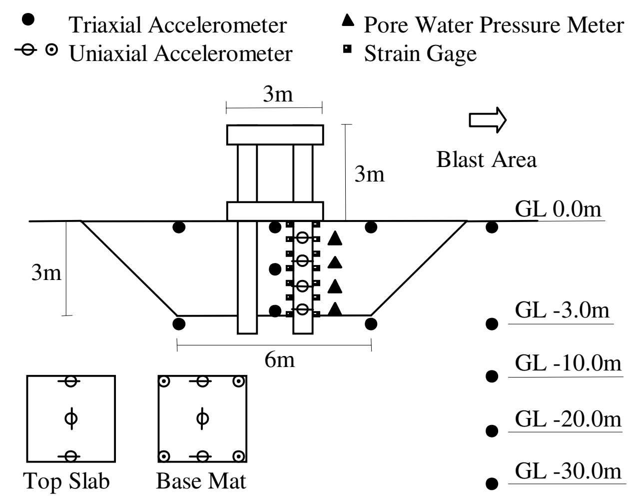

Figure 5 Instrumentation

PS measurements were conducted at the test site to investigate the physical properties of the soil layers. PS measurement results are shown in Figure 6. The overburden consisted of several layers of siltstone or mudstone. The shear wave velocity at the test pit bottom was about 200 m/s and this increased to 500 to 700 m/s with increasing depth. Core soil samples were collected for laboratory tests.

The grain size distribution of the backfill sand is shown in Figure 7. The sand was found near Black Thunder Mine. Great care was taken in backfilling the test pit with the sand, because the sand needed to be 100% water-saturated and air had to be removed in order to ensure a liquefiable sand deposit.

Figure 8 shows the installation of the waterproof layer. Figure 9 shows the completed pile-supported structure and the test pit. The water level was kept at 10 cm above the sand surface throughout seismic tests to prevent dry out of the sand deposit.

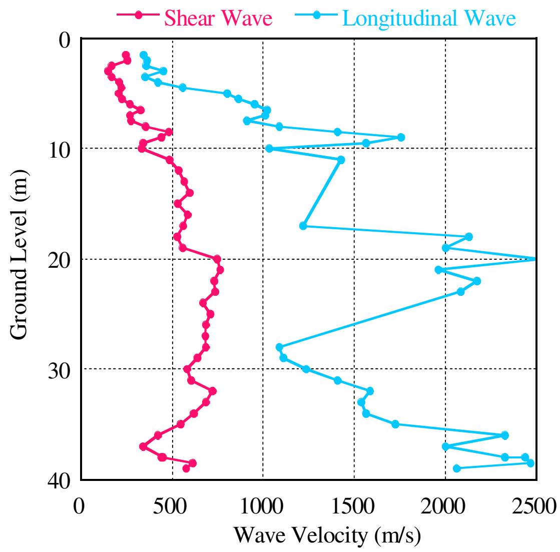

Figure 6 Results of PS measurement

Figure 8 Installation of Waterproof Layer

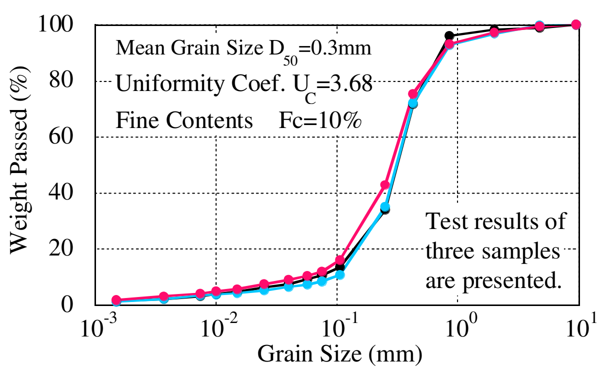

Figure 7 Grain Size Distribution of Sand

Figure 9 Completion Test Structure and Test Pit

VIBRATION TEST RESULTS

Vibration tests were conducted six times. The locations of the blast areas for each test are shown in Figure 10. The blast areas were about 60m wide and 500m long. However, their lengths depended on the mining schedule.

The results of the vibration tests are summarized in Table 1. The maximum horizontal acceleration recorded on the adjacent ground surface varied from 20 Gals to 1,352 Gals depending on the distance from the blast area to the test site. The closest blast was only 90m from the test site. These differences in maximum acceleration yielded responses at different levels and liquefaction of different degrees. Sand boiling phenomena were observed in the test pit with larger input motions. This is one of the most advantageous features of the test method employed in this project, although the input motions were not controlled mechanically or electrically.

In this paper, three tests indicated by yellow highlights in Table 1 were chosen for detailed investigations, because those tests provided three different phenomena in terms of liquefaction of the sand deposit as well as in terms of dynamic responses of the structure. Horizontal accelerations in the EW direction are discussed hereafter.

Dynamic Responses in Liquefied Sand Deposits

The maximum accelerations recorded in the adjacent free field in vertical array configurations are compared for three tests in Figure 11. The amplification tendencies from GL-32m to the surface were similar in the mudstone layers for three tests. The maximum accelerations recorded through the mudstone layers to the sand deposit are compared for these three tests in Figure 12. There was a clear difference

Table 1 Summary of Vibration Tests

Level of Input Motions	Test #	Distance (m) *	Max. Acceleration **		
			EW	NS	UD
Small	Test-1	3000	20	28	29
	Test-2	1000	32	84	48
Medium	Test-5	500	142	245	304
Large	Test-3	140	579	568	1013
	Test-4	180	564	593	332
Very Large	Test-6	90	1217	1352	3475

*: distance from blast area to test site
**: at the ground level of adjacent free field (Gals)

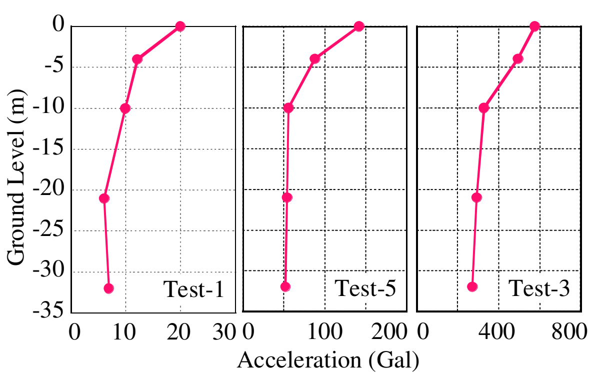

Figure 11 Max. Acceleration at Free Field

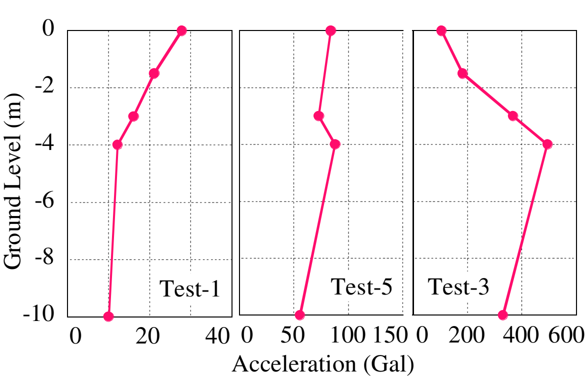

Figure 12 Max. Acceleration of Test Pit

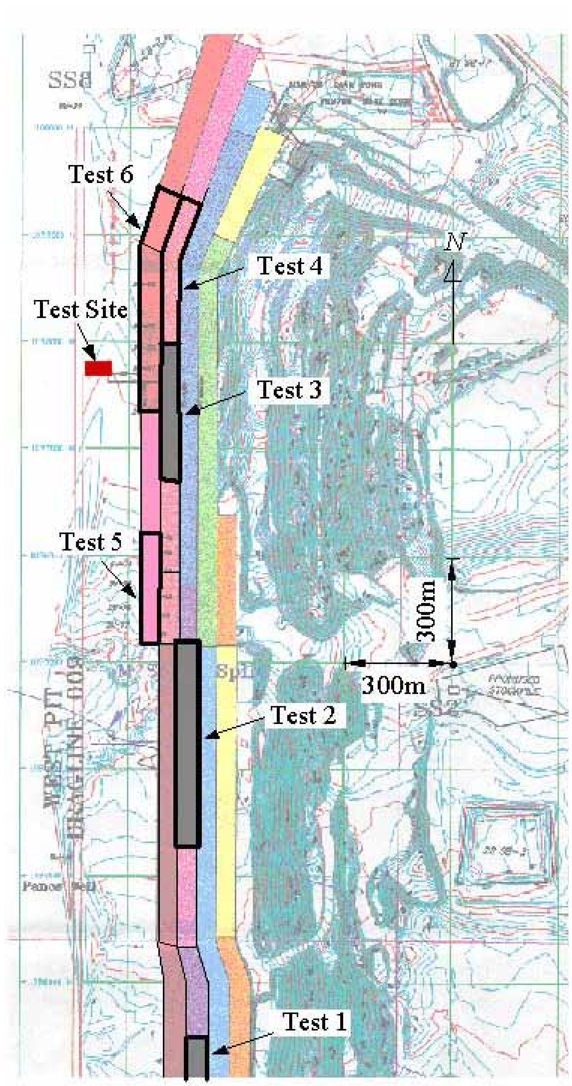

Figure 10 Locations of Blasts in Vibration Tests

among the amplification trends in the test pit. Test-1 showed a similar amplification trend to that of the mudstone layers as shown in Figure 11. Test-5 showed less amplification in the sand deposit. Test-3 showed a large decrease in acceleration in the test pit because of severe liquefaction of the sand deposit.

Acceleration time histories at the sand surface, the free field surface and GL-32m are compared for Test-1 (Small Input Level) in Figure 13. The response spectra from these records are also shown in the figure. The same set of acceleration time histories and these response spectra are shown in Figure 14 for Test-5 (Medium Input Level) and in Figure 15 for Test-3 (Large Input Level).

As can be seen from Figure 13 for Test-1, over all the frequency regions, the responses at the sand surface were greater than those at the free field surface, and the responses at the free field surface were greater than those at GL-32m. From Figure 14 for Test-5, the responses at the sand surface and the free field surface were greater than those at GL-32m over all frequency regions. The responses at the sand surface became smaller than those at the free field surface for periods of less than 0.4 seconds due to in a certain degree of liquefaction of the sand. From Figure 15 for Test-3, the responses at the sand surface became much smaller than those at the free field surface and even smaller than those at GL-32m. These response reductions in the test pit were caused by extensive liquefaction over the test pit, because shear waves could not travel in the liquefied sand.

Time histories of excess pore water pressure ratios are shown in Figure 16. The excess pore water pressure ratio is the ratio of excess pore water pressure to initial effective stress. In Test-1, the maximum

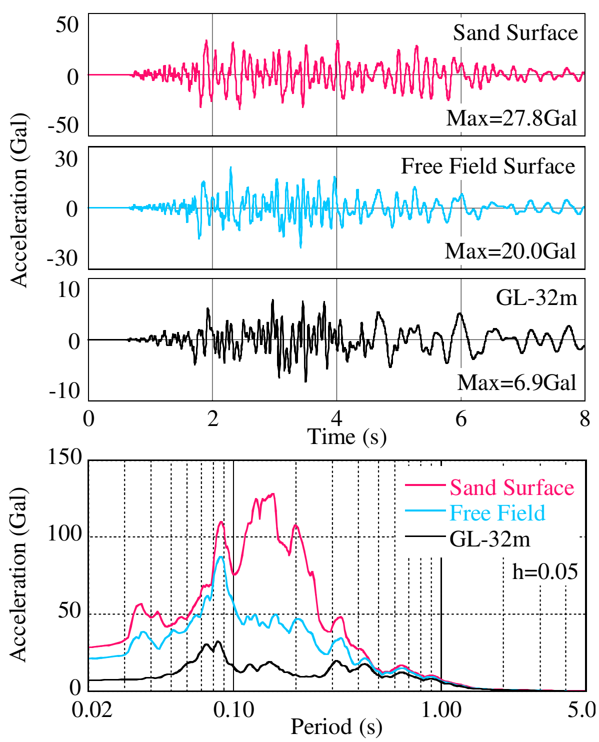

Figure 13 Acceleration Records of Test-1
(Small Input Level)

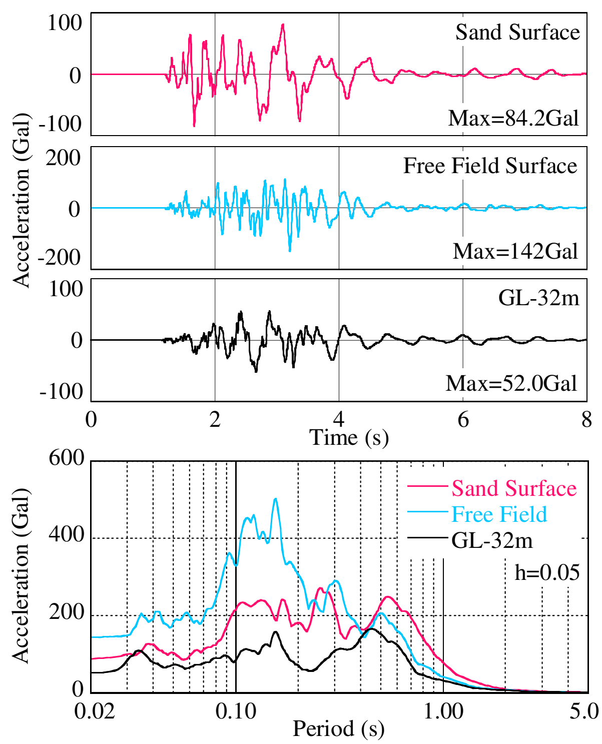

Figure 14 Acceleration Records of Test-5
(Medium Input Level)

ratio stayed around zero, which means that no liquefaction took place. In Test-5, the ratios rose rapidly,

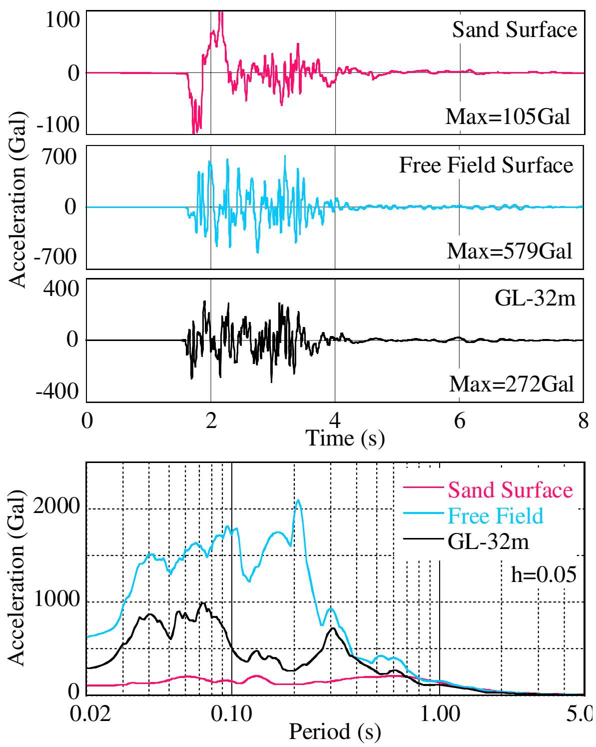

Figure 15 Acceleration Records of Test-3 (Large Input Level)

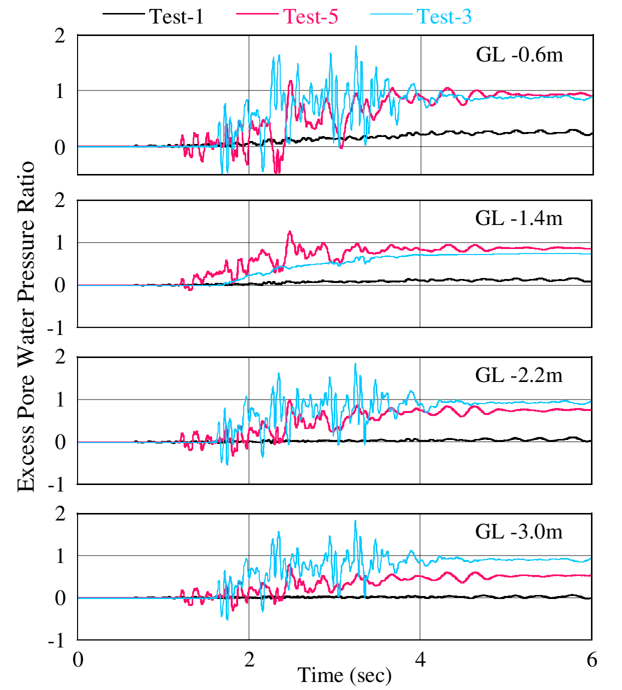

Figure 16 Measured Time Histories of Excess Pore Water Pressure Ratio

reaching around one at GL-0.6m and GL-1.4m after the main vibration was finished. Ratios at GL-2.2m and GL-3.0m were about 0.7 and 0.5. The measurement showed that the liquefaction region was in the upper half of the test pit. In Test-3, ratios at all levels rose rapidly, reaching around one, which indicates extensive liquefaction over the entire region. The large fluctuations in pressure records during main ground motions were caused by longitudinal waves.

Structure Responses Subjected to Blasts-Induced Ground Motion

Figure 17 compares the maximum recorded accelerations of the pile-supported structure and in the test pit for three tests. As can be seen for Test-3, there were differences between the maximum responses of the pile-supported structure and the sand deposit, which means that the pile and surrounding sand did not behave in the same manner. In Test-3, unlike in the other cases, the maximum acceleration decreased as motions traveled upward. Figure 18 compares the acceleration time histories at the top slab, the base mat and GL-3m of the pile for Test-1 (Small Input Level). The response spectra from these records are also shown. The same set of acceleration time histories and their response spectra are shown in Figure 19 for Test-3 (Large Input Level).

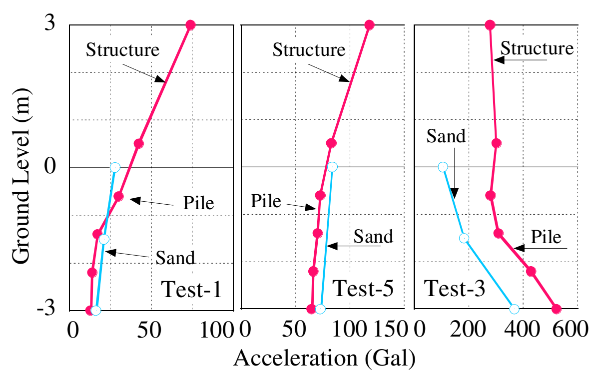

Figure 17 Max. Acc. of Test Structure

As can be seen from Figure 18 for Test-1, the maximum accelerations increased as motions went upward. For all frequency regions, the responses at the top slab were greater than those at the base mat, and the responses at the base mat were greater than those at GL-3m of the pile. The first natural period of the soil-pile-structure system was about 0.2 seconds under the input motion level of Test-1. For Test-3, the maximum accelerations decreased as motions went upward, which were different from those of Test-1. The responses at the top slab and the base mat became smaller than or similar to the responses at GL-3m of the pile. Compared with Test-1 results, it became difficult to identify peaks corresponding to natural periods of the soil-pile-structure system from response spectra diagrams. These results show that soil nonlinearity and liquefaction greatly influence the dynamic properties of pile-supported structures.

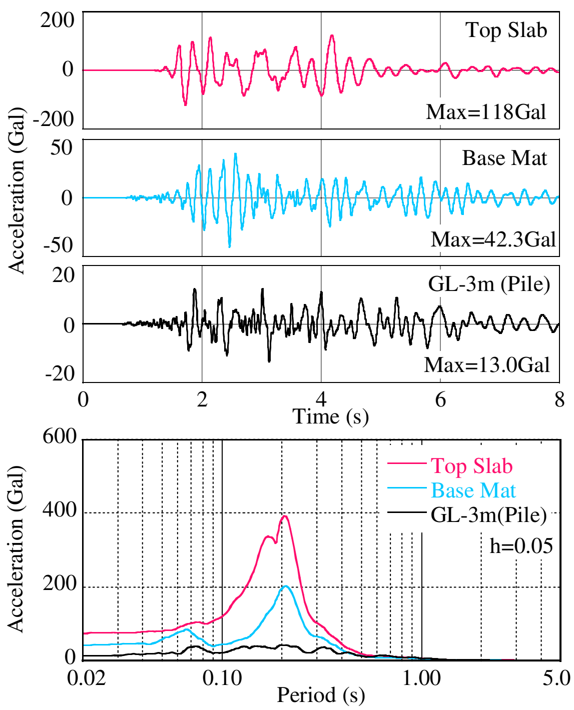

Figure 18 Acceleration Records of Test Structure
(Test-1 : Small Input Level)

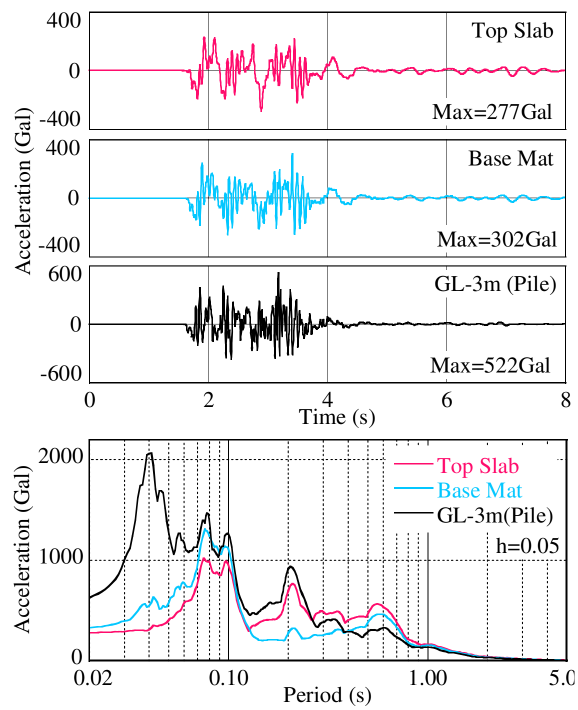

Figure 19 Acceleration Records of Test Structure
(Test-3 : Large Input Level)

Measurement Results of Pile Stresses

The distributions of maximum pile stresses, bending moments and axial forces, are shown in Figures 20 and 21. The bending moment took its maximum value at the pile head for all cases. However, the moment distribution shapes differed and the inflection points of the curves moved downward in accordance with the input motion levels, in other words, the degrees of liquefaction in the test pit. However, the axial forces are almost the same regardless of the depth and similar tendencies are shown in all the test results.

Time histories of the pile stresses are shown in Figure 22. Records of bending moments have the inverse phases at the pile head and at GL-2.9m that is the interface between the liquefied sand and the supported layer, and those occurrence times of maximum values are different due to the super-structure responses.

The axial forces occur in almost the same phase at the pile head and at GL-2.5m. These results show that pile axial forces are caused by the superstructure response and almost all stresses propagate from the pile head to the pile tip without dissipation to the soil around the pile foundation.

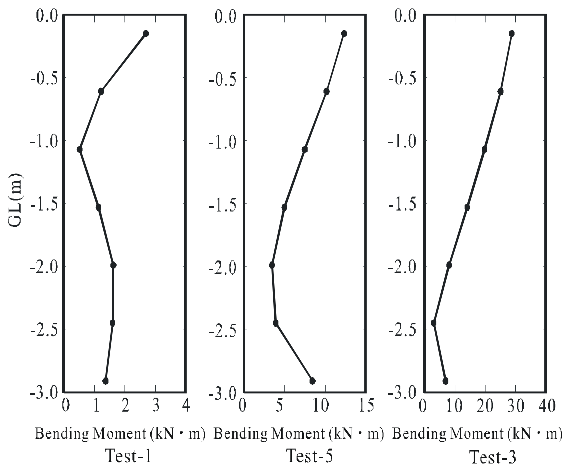

Figure 20 Maximum Bending Moments of Pile

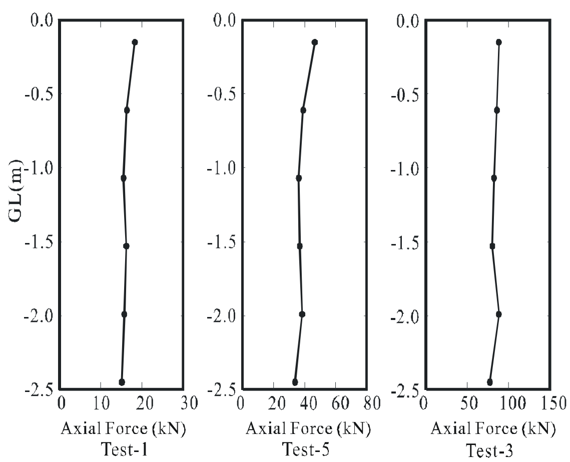

Figure 21 Maximum Axial Forces of Pile

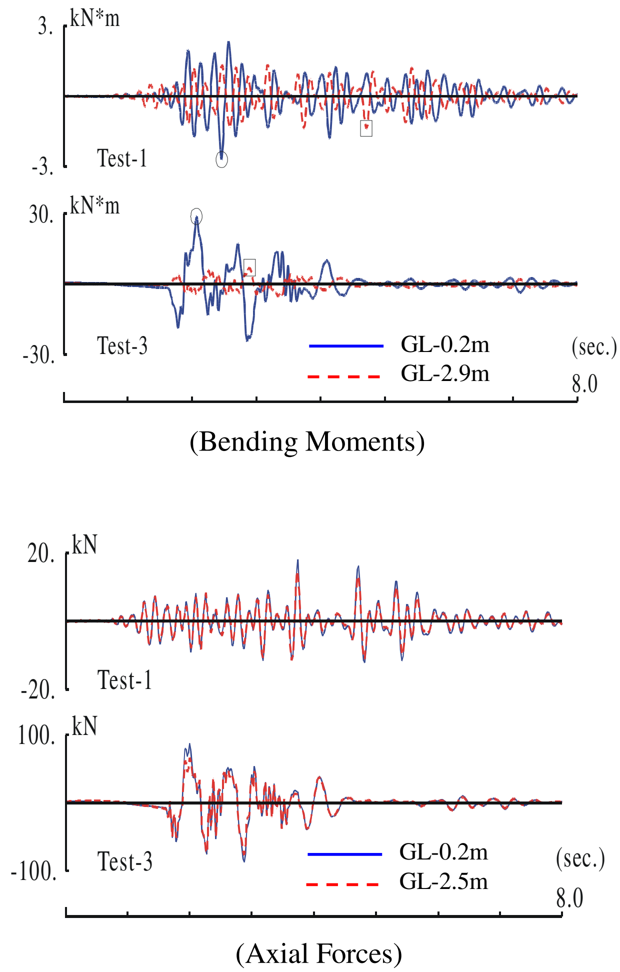

Figure 22 Time-Histories of Pile Stresses

CONCLUSIONS

(1) Vibration tests were conducted of a pile-supported structure in a liquefiable sand deposit six times using ground motions induced by large mining blasts. Nonlinear responses of the soil-pile-structure system at different levels were obtained for various levels of liquefaction in the test pit.

(2) The maximum horizontal acceleration recorded at the adjacent ground surface varied from 20 Gals to 1,352 Gals depending on the distance from the test site to the blast areas. This is one of the advantages of the vibration test method employed in this project.

(3) In the adjacent free field, motions were amplified from GL-32m to the ground surface, regardless of input motion levels. In the test pit, amplification of ground motions depended on input motion levels due to soil nonlinearity including liquefaction.

(4) Generation of pore water pressures depended upon input motion levels. Liquefaction started from the shallow part and extended to the deeper part of the test pit, and finally the test pit was completely liquefied as in Test-3 (Large Input Level).
(5) Bending moments were a maximum at the pile heads, regardless of input motion levels. However, the moment distribution shapes varied and the inflection points of the distribution curves moved downward in accordance with input motion levels, in other words, the degrees of the liquefaction in the test pit.
(6) The vibration test method employed in this experimental research was found to be very useful and effective for investigating the dynamic behavior of large model structures under severe ground motions.

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