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Evaluation of Pile Response Due to Liquefaction-Induced Lateral Spreading of The Ground

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ABSTRACT

A cooperative project is reported between teams at Rensselaer Polytechnic Institute (RPI) and Cornell University to evaluate the effect of lateral spreading on pile foundations. Centrifuge tests of lateral spreading and the corresponding permanent bending moments measured in instrumented model piles conducted at RPI, are used to study the problem and verify and calibrate Cornell University's computer program B-STRUCT. Individual piles and pile groups are investigated in a variety of pile and soil configurations. The paper reports in detail the results of Model 1, corresponding to the simulation of reinforced concrete piles that failed and developed two plastic hinges under the NFCH building in Niigata during the 1964 earthquake.

INTRODUCTION

Lateral spreading of liquefied soil in sloping ground or near a waterfront is a cause of significant damage to pile foundations of buildings, port facilities and bridges. Cases of distress of deep foundations due to this reason have been reported in a number of seismic events in Japan, the U.S. and other countries, including: Niigata 1964, Alaska 1964, Loma Prieta 1989, and Hyogoken-Nanbu 1995 earthquakes (Hamada et al., 1986; Ross et al., 1969; Benuzka, 1990; Tokimatsu et al., 1996). Figure 1 sketches damage to steel pipe piles under the Showa Bridge over the Shinano River in Niigata during the 1964 earthquake (Hamada et al., 1986).

In the last 5-10 years, the study of this problem, including development of evaluation methods, has focused on well-documented case histories of reinforced concrete piles damaged in the 1964 Niigata earthquake (Hamada et al., 1986; Miura and O'Rourke, 1991; Dobry, 1994). Program B-STRUCT was developed for this purpose at Cornell University. This is a beam-on-elastic foundation code including nonlinear soil springs and a nonlinear moment-curvature relation for the pile, where the lateral spreading effect is modeled by displacing laterally the soil spring supports by an amount equal to the free field permanent deformation (Meyersohn, 1994). Simultaneously, a technique to realistically model liquefaction and lateral spreading of a sloping saturated sand deposit, using an inclined laminar box excited

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by base shaking was developed at Rensselaer Polytechnic Institute, RPI (Taboada, 1995; Dobry et al., 1995). In another centrifuge project, the effect of liquefaction on the nonlinear p-y curves of saturated sand was studied at RPI by in-flight loading tests of centrifuge model piles instrumented with strain gages (Liu and Dobry, 1995; Dobry et al, 1995). In the cooperative RPI-Cornell project reported in this paper, these three efforts are further developed and applied to the study of pile response in the presence of lateral spreading (Abdoun et al., 1996). Specifically, the development of pile displacements and bending moments due to lateral spreading of layered deposits in the free field is modeled in the centrifuge, and the results are used to verify and calibrate program B-STRUCT for both individual piles and pile groups.

Pile 2 under the Niigata Family Court House (NFCH) and its response in 1964 (Figure 2) was selected for initial study and is reported in this paper as centrifuge Model 1. The 1964 NFCH building is a well documented case history for which information regarding pile properties, subsurface conditions, and extent of ground displacement are available (Hamada, et al., 1986; Kawashima, et al., 1988). The NFCH building was founded on 35 cm diameter concrete piles (Figure 2). The measured offset between the two ends of the pile after the earthquake was approximately 70 cm for Pile 2, which penetrated about 1 m into the lower nonliquefied soil layer. Figure 3 presents the pile bending moments for several values of lateral spreading, predicted with program B-STRUCT prior to the centrifuge testing (Meyersohn, 1994).

LAMINAR BOX AND MODEL PILE

A laminar box is used for the centrifuge modeling of the 1964 Niigata earthquake NFCH building case history. The container inside dimensions are 45.72 cm (length) by 25.40 (width) by 26.39 cm (height), see Figure 4. The box consists of a stack of up to 39 rectangular rings separated by linear roller bearings, arranged to permit relative movement between rings in the long direction with minimal friction (Taboada, 1995; Dobry et al., 1995).

The model pile used in the laminar box model setup was made of 0.95 cm in diameter polyetherimide rod (ULTEM 1000). At 50g centrifugal acceleration, this model simulates a prototype pile of diameter d=47.5 cm and bending stiffness, EI = 8,000 $kN - m^2$. This value of EI is within the range of effective stiffnesses of the NFCH reinforced concrete piles in 1964, which ranged from an initial EI= 18,000 $kN - m^2$ to an effective secant EI= 4,500 $kN - m^2$ after cracking of the concrete pile took place. The model pile is quite strong and remained elastic during the tests; as no yielding took place, the same model pile could be used in several experiments.

Six pairs of full-bridge circuited strain gages were installed along the surface of the model pile to monitor bending moments during lateral spreading. Two pairs of strain gages were placed near each interface (Figure 4). As large deformations were expected in the test,

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microcrystalline wax and a soft plastic shrink tube were used to waterproof the strain gages. Sand grains were glued to the shrink tube surface to develop an adequate pile-soil roughness.

CENTRIFUGE MODELING OF PILE 2 IN NFCH BUILDING

Model 1, simulating approximately Pile 2 of the NFCH building, involved a single pile (EI=8000 $kN - m^2$) embedded in a three-layer system (Figure 4). This graph shows the soil profile in model units as well as the instrumentation used. The test was done at 50g centrifugal acceleration. The total height of the profile is 20 cm in model units, that is 10 m in prototype units. In prototype units, the top layer is a 2 m cemented sand with a cohesion of 6.5 kg/cm^2, which models the nonliquefiable sand in the field, followed by a 6 m layer of liquefiable uniform Nevada sand placed at a relative density of about 40%, followed by a 2 m layer of the same cemented sand. The soil profile is fully saturated with water and the model is inclined 2° to the horizontal. Both top and bottom cemented sand layers were perforated to make them pervious and avoid accumulation of water at the bottom of the top layer. In what follows, prototype units are consistently used.

Input acceleration-time history applied at the base of the soil profile is shown in Figure 5. Accelerations and excess pore pressure ratios recorded in the soil during the test are shown in Figures 5 and 6, respectively. The recorded accelerations and excess pore pressure ratios indicate that the 6 m, 40 % relative density Nevada sand layer did liquefy, while as expected the cemented sand remained solid. The profiles of soil lateral displacement measured by the LVDT's mounted on the laminar box rings at different times during shaking, and at the end of shaking are shown in Figure 7. The soil lateral deformation at the top after shaking is about 80 cm. All soil accelerations, pore pressures and lateral displacements measured in this test with a model pile were very similar to those measured in a preliminary experiment without a pile, revealing that the presence of the pile did not affect the free field response and lateral spreading of the soil. Figure 8 shows a comparison between the pile displacement measured at the top of the soil profile at the end of shaking and the pile displacement calculated using program B-STRUCT. It is interesting that the measured tip pile displacement is larger than the free field soil displacement; the reason for this is discussed below.

Figure 9 shows the bending moment time histories recorded at different depths along the pile during shaking. The moments recorded within the top 4 m kept increasing with time until local soil failure occurred around the pile in the top cemented sand layer. This soil failure was verified by direct observation during excavation of the soil around the pile after the test. Due to this soil failure, which was also responsible for the larger pile displacement relative to the soil in Figure 8, the pressure of the cemented soil to the pile decreased, causing the recorded decrease in bending moments. Figure 10 presents the profiles of recorded pile bending moments at different times during shaking. Figure 11 shows that the bending moments recorded in the centrifuge model pile compare well with the Cornell University

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analytical results for Pile 2 at the NFCH building during the 1964 Niigata earthquake, for the same soil surface lateral displacements in the free field, D_H, up to $D_H = 24$ cm.

CONCLUSIONS

The previous results for centrifuge Model 1 indicate that:
1) The presence of the pile did not affect the free field response of the soil, as revealed by a comparison between these results and those of the free field test with no pile. Especially important is the consistency between lateral surface soil displacement in the tests with and without pile: about 80 cm in both cases (Figure 8). This displacement is also comparable to that measured after shaking at the NFCH building site in the 1964 Niigata earthquake.

2) In Model 1, the cemented soil in the top layer failed around the pile when the bending moments reached about 150 to 175 $kN-m$, with corresponding decrease in bending moments afterwards (Figures 9 and 10), as well as pile penetration into the soil (Figure 8). This soil failure and pile penetration were well predicted by Cornell University program B-STRUCT, indicating that the p-y curves measured at RPI for the different layers and the Cornell University analytical technique correctly account for this aspect of the response.

3) Centrifuge Model 1 results are very consistent with the analysis of the response of the NFCH concrete pile in Niigata in 1964 using program B-STRUCT, up to a surface ground displacement of about 24 cm (Figure 11). This is the range during which both soil and pile exhibited an approximately linear behavior, both in the field and in the centrifuge. Subsequently, the soil failed in the top layer around the elastic pile model in the centrifuge test when the bending moment reached about 150 to 175 $kN-m$, while the concrete pile under the NFCH building exhibited strong nonlinear moment-rotation response due to concrete cracking when the bending moments reached about 90 $kN-m$. Both effects in the centrifuge and in the field tended to reduce the pile moments in the top layer, but for different reasons. If the concrete pile in Niigata had remained linear at these high moments, the soil near the pile would have failed as it happened in the centrifuge model test.

ACKNOWLEDGMENT

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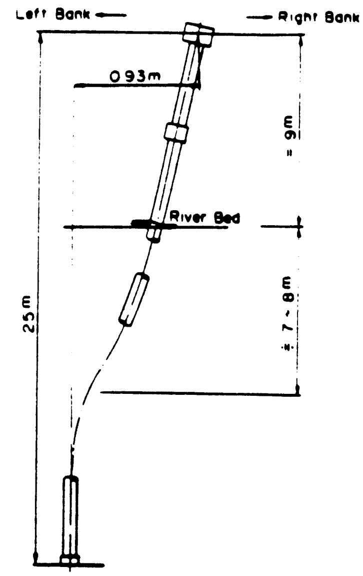

Figure 1: Damage to steel pipe piles of pier P4 of Showa Bridge during the 1964 Niigata earthquake (Hamada et al. 1986)

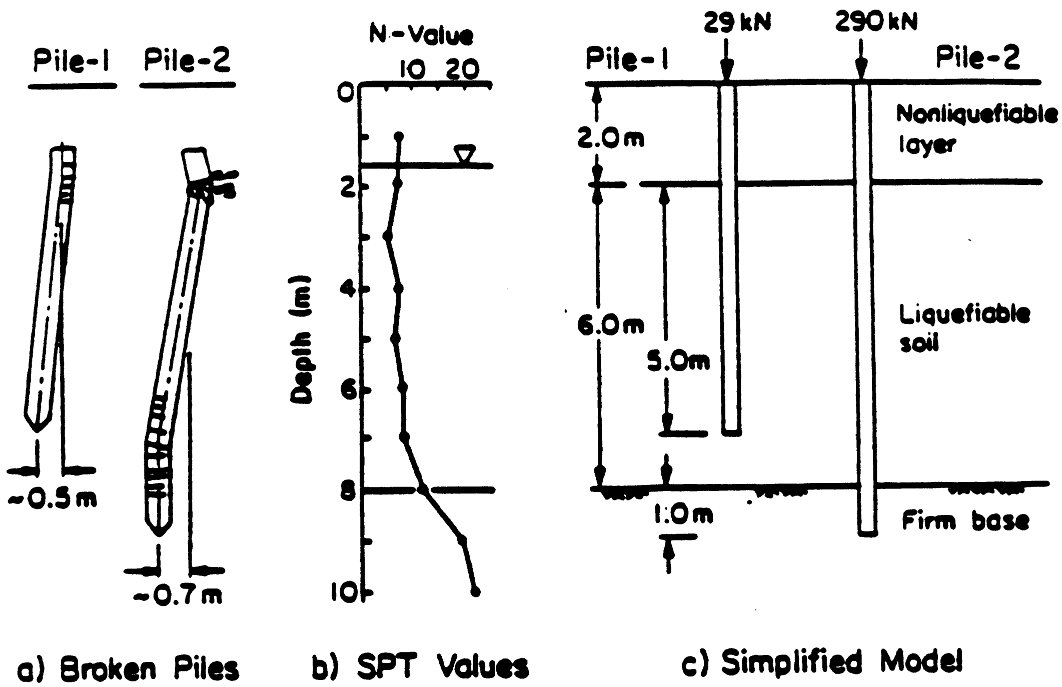

Figure 2: Observed pile deformation at NFCH building, Niigata earthquake (Yoshida and Hamada, 1991; Meyersohn, 1994)

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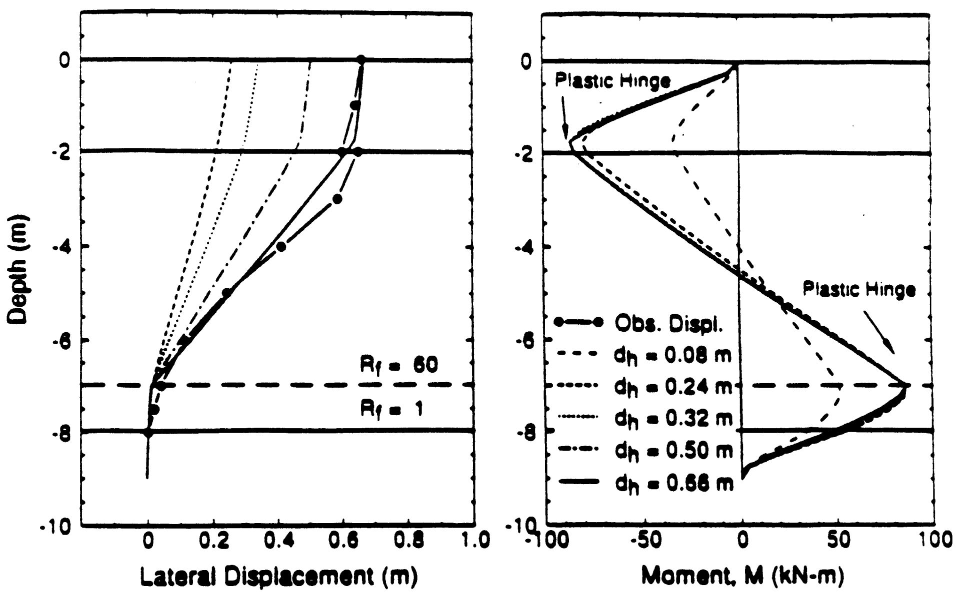

Figure 3: Analytical results using program B-STRUCT for Pile-2 at NFCH building (Meyersohn, 1994)

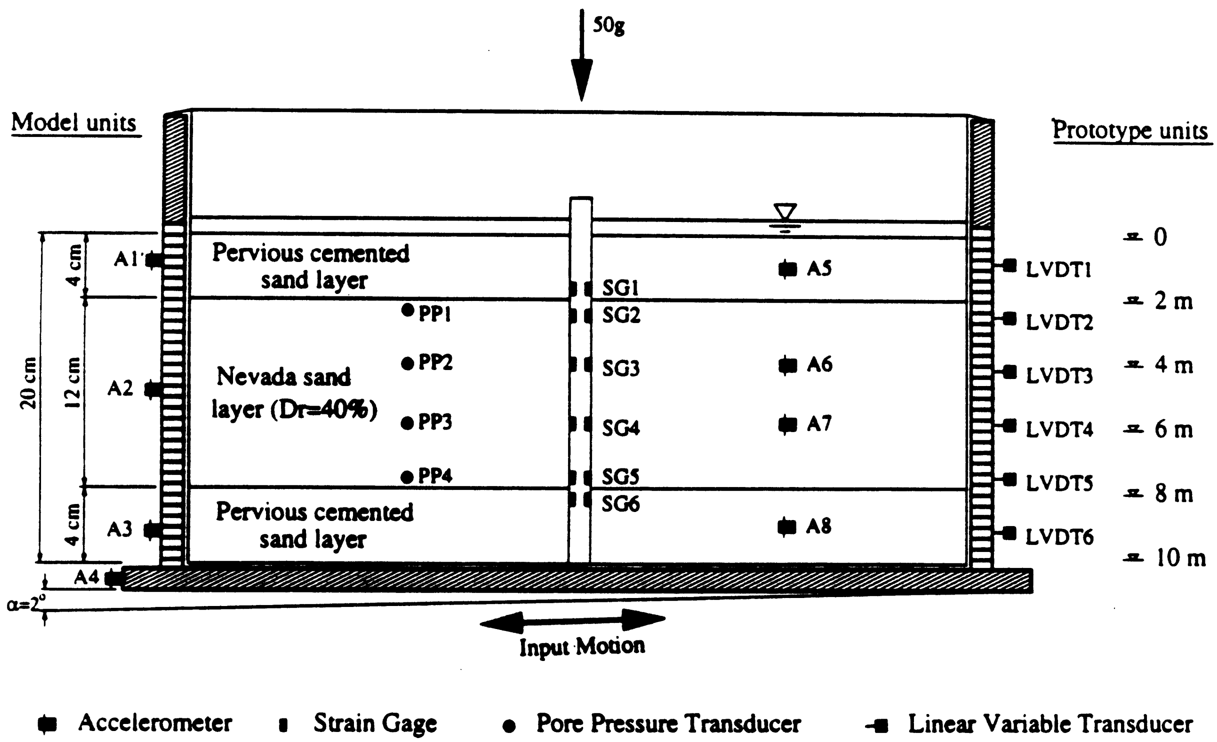

Figure 4: Centrifuge lateral spreading model setup of Pile-2 at NFCH building, using RPI laminar box

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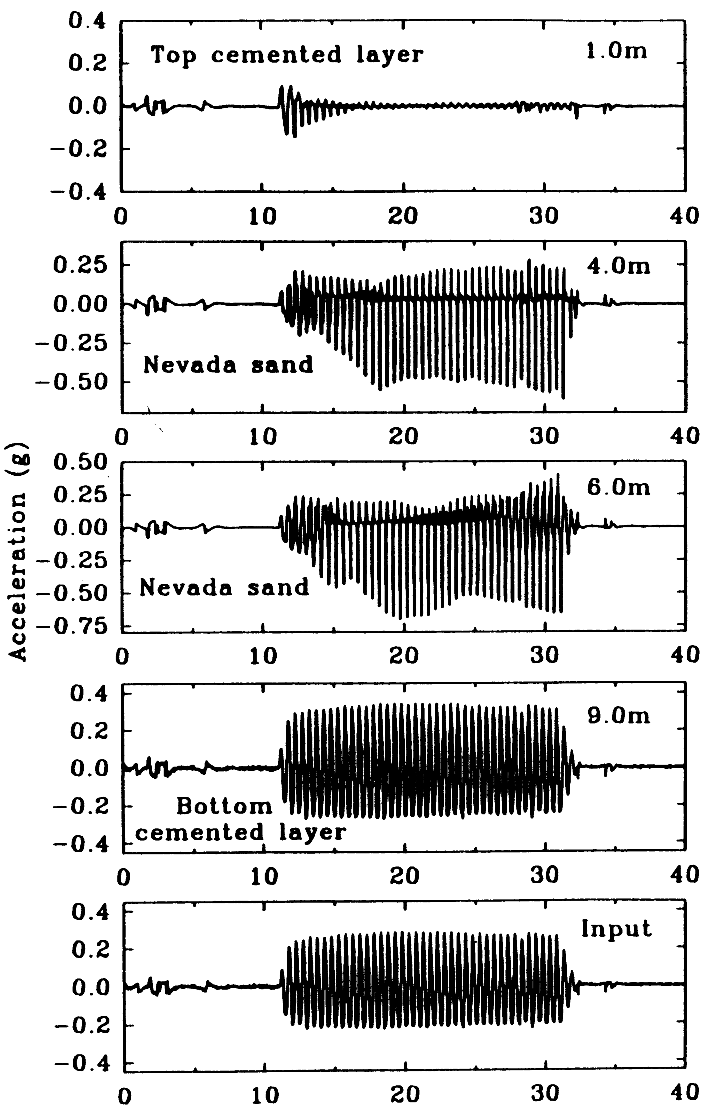

Figure 5: Time histories of accelerations recorded in the soil during shaking

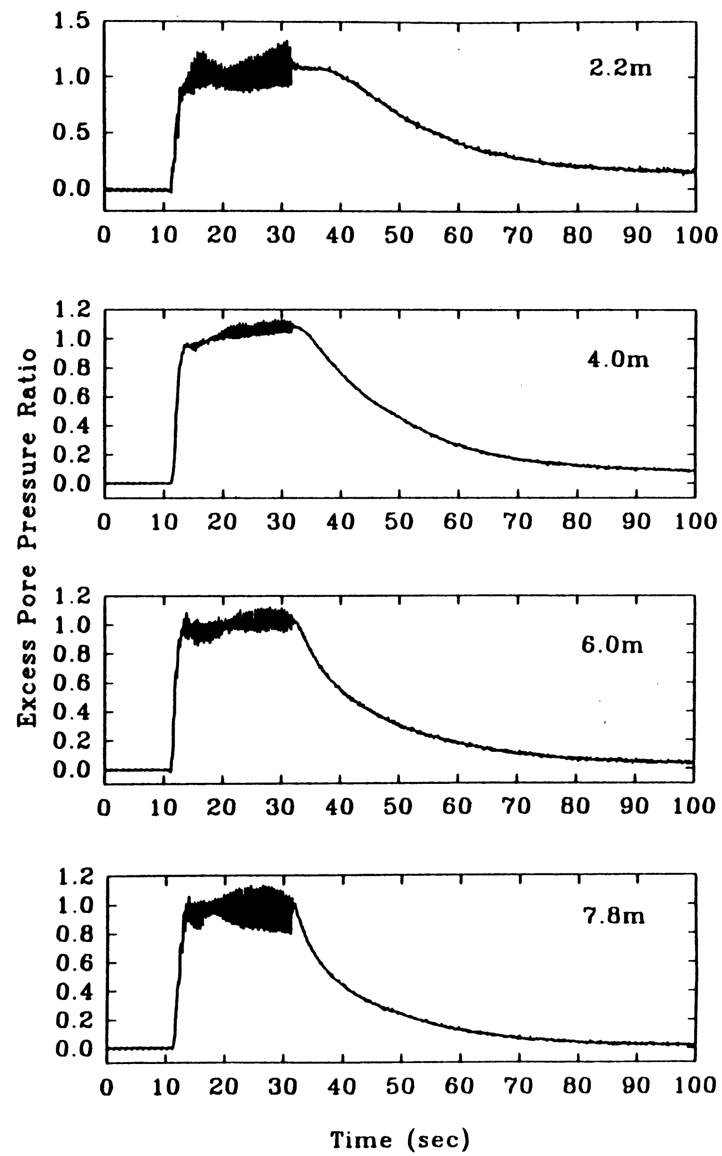

Figure 6: Time histories of excess pore pressure ratio in the Nevada sand layer (Dr=40%)

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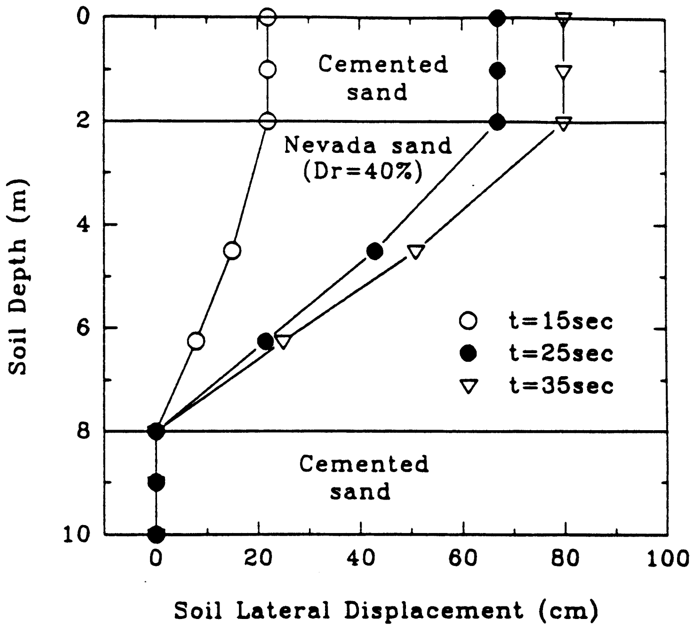

Figure 7: Lateral displacement soil profiles measured during shaking

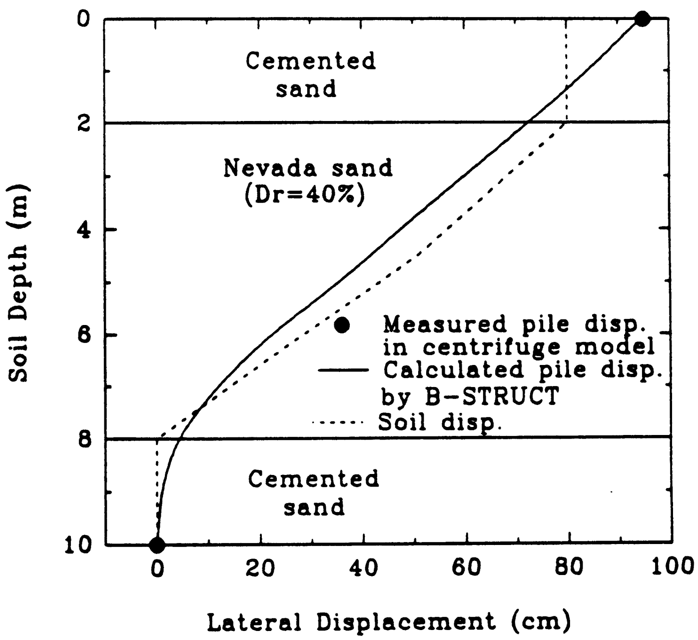

Figure 8: Soil and pile lateral displacement profiles at end of shaking

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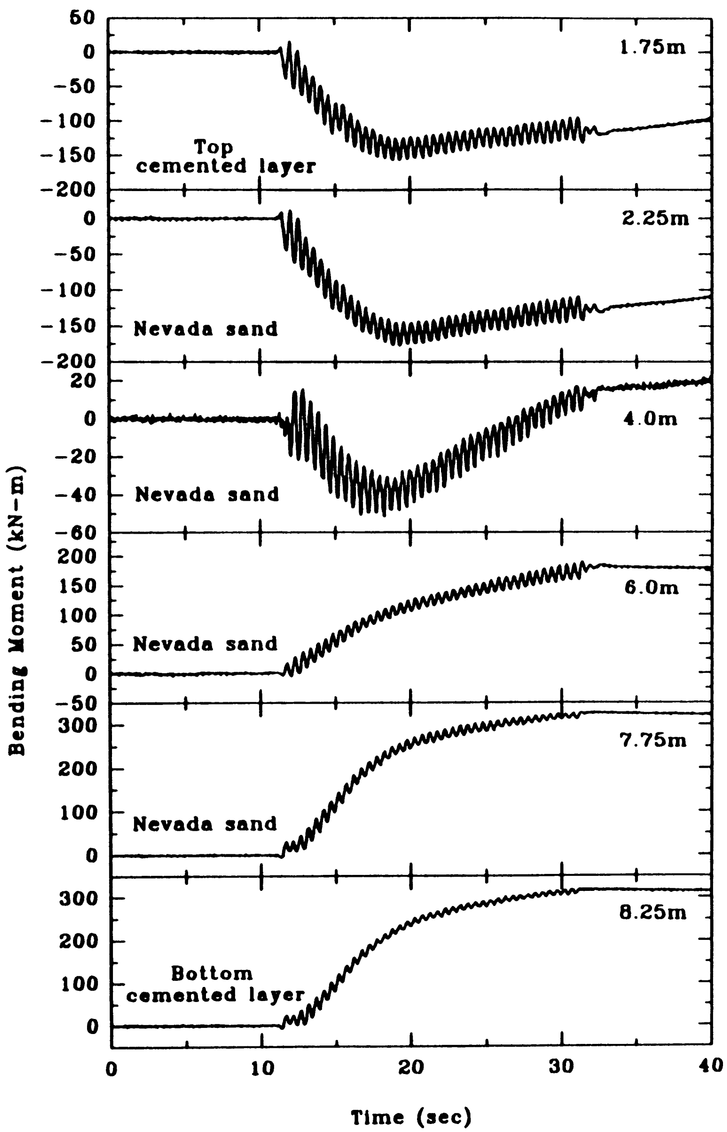

Figure 9: Time histories of measured bending moments along the pile

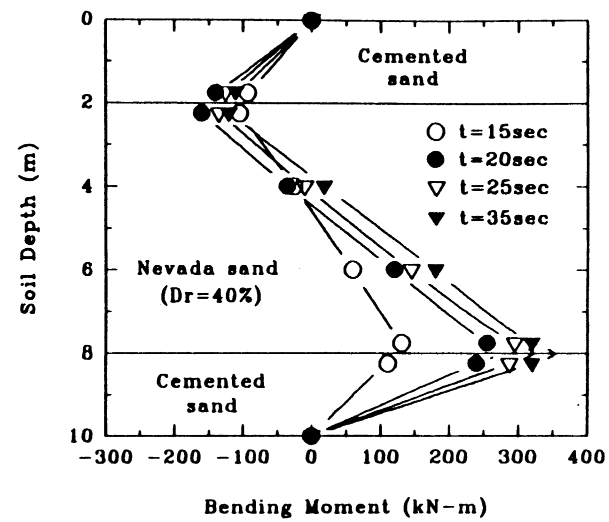

Figure 10: Measured bending moments profiles during shaking

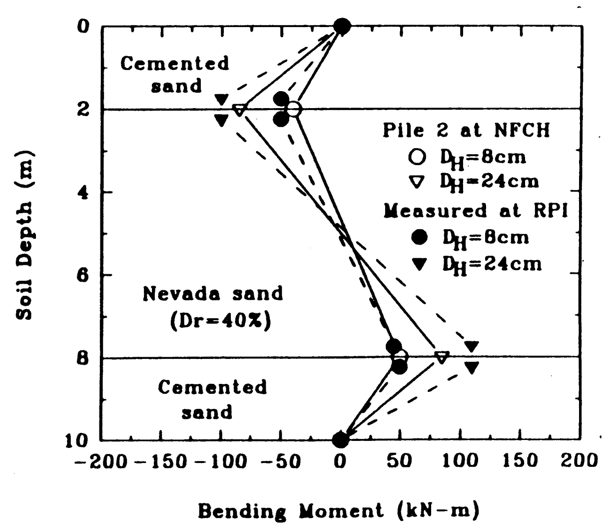

Figure 11: Comparison between bending moments measured in RPI centrifuge test and those calculated by B-STRUCT for Pile 2 at NFCH building in the 1964 Niigata earthquake

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