

EXCAVATION AND RESPONSE ANALYSIS
OF A DAMAGED RC PILES BY LIQUEFACTION

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SUMMARY

Authors recently had the chance of excavating the damaged piles by liquefaction in 1964 Niigata earthquake in case of reconstruction of the building. Along the pile severe damages with crushing of concrete were found at the middle and the tip. Ring cracks were observed along the whole length. Lumped mass model was adopted for the building-pile-soil interaction response analysis. The liquefaction of soil layers was considered and modeled by nonlinear shear stress vs. shear strain relationships. The value and its distribution of the pile moment corresponded quite well with the observed damage.

INTRODUCTION

In 1978 Miyagiken-oki earthquake precast concrete piles were damaged in several buildings. Stimulated by the damages, the necessity to refine the earthquake resistant design of pile foundations were insisted. Usually pile foundations are adopted for the buildings on soft ground, and the soil-structure interaction has an important effects on the dynamic behavior of those structures. In the case of very strong earthquake motions the nonlinearity of subsoil is remarkable and should be taken into account. The liquefaction of sand layers is the most catastrophic case of it. Authors recently had the chance of excavating the piles damaged by liquefaction in 1964 Niigata earthquake. The piles were inspected of their damages and the simulative analyses were done using the building-pile-soil interaction model.

THE BUILDING, PILE & GEOLOGY

The Building & its Foundation

The building was a 3 storied (partly 2 storied) reinforced concrete office building, whose plan was 24.5m x 18.5m. The structure consists of open frames of 5 spans in the longitudinal direction and of frame-walls

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of 2 spans in the transverse direction as shown in Fig.1. Total 104 pieces of precast reinforced concrete piles (30cm in diameter and 10m long) were driven under the individual footings.

In 1964 Niigata earthquake the building was settled and inclined as shown in Fig.2 due to the damages of the piles caused by the liquefaction of subsoil. The maximum settlement was 1.3m. Underground pipings and the like were also damaged, though the upper structure itself was slightly damaged. After the earthquake the building was jacked up and supported by the H-shaped steel piles newly driven. It was used till recently.

Geological Condition

In the Niigata earthquake liquefaction occurred and many buildings were settled and tilted around the site. The geological formation of this site is shown in Fig.3. It mainly consists of sand with partial silty layer near the ground surface. The N values of standard penetration tests are 2~7 down to GL-10.5m, and more than 20 below it. The water table lies at GL-0.3m and almost all layers up to GL are saturated. The pile tips were at GL-12m and penetrated the fine sand layer of N value more than 20 by 1.5m.

EXCAVATION OF DAMAGED PILES

Method of Excavation

Excavation of piles was done after demolishing and removal of the upper structure and footings. Steel casing with water jet at its head was pushed down to the pile tip enclosing the piles. After the circumferential friction was cut the piles were pulled off by wire, taking care that the piles were not damaged by the work. Photo 1 shows the pile just pulled off. Total 4 piles were excavated whose locations are shown in Fig.1.

Damage on the Piles

The damage on the piles is shown in Fig.4 and Photo 2. Most severely damaged parts are around 2.2m from the tip and around 3.1~3.5m from the bottom of the footing. On both parts concrete was crushed and spalled along the length of 10~20cm. Axial reinforcing bars and spiral shear-reinforcing bars were exposed and a little rusted. Ring cracks were found along the whole length, whose widths were 0.7~1.2mm and the pitch was 10~20cm. As the subsoil 2m thick just under the footing was replaced by rubble concrete, damage of the piles around there could not be inspected.

It is noteworthy that the severe damage of concrete crush buckling of bars was found not only around the maximum moment point in the case of hinged pile head but also near the pile tip where the stiffness of the soil layers suddenly increased. It might be possible that the large bending moment was generated also at the boundary of soil layers as the displacement of the pile was restrained in the stiff soil. It is difficult to judge whether the ring cracks were due to the earthquake or construction works.

RESPONSE ANALYSIS

Building-Pile-Soil Interaction Model

The building-pile-soil interaction model used in the analysis is shown in Fig.5. It is composed of roughly two parts, the structure part and the free soil part. The structure part is composed of building, footing, piles and additive masses which are tightly connected to piles. The freedom of the model is 12 in total, 7 of structure part and 5 of free soil part. Both parts are related through the lateral springs and dashpots which transmit the response displacement and velocity of the free soil to the structure as external forces. Then the structure part accepts both the base acceleration and lateral external forces simultaneously as multiple inputs.

The additive masses tightly connected to piles are not virtual but actual and soil column of some area is taken for it. To define the area of additive soil column static solution by Mindlin of the displacement of the elastic half space with a point load applied is used. The kinetic energy of the additive mass is equalized with that of the soil volumetrically integrated. As the additive masses of this model represent the soil column, shear springs of that area are attached between the masses. Our model is different from the model by Penzien et al (1964) on this point.¹⁾ In their model the additive masses were virtual to express the dynamic effects of spring and no shear springs existed.

The spring constants of the building were calculated from the stiffness of the frames and walls. The stiffness matrix of the piles was calculated mainly considering the bending deformation with both ends hinged. Judging from the damage of excavated piles the restraint at the pile tip might have been quite large. However considering that the most severely damaged was the upper part, hinged condition was selected for this case of analysis. Lateral spring constants between pile and free soil were calculated by Mindlin's solution. Those for group piles were obtained by multiplying the spring constants of single pile by the total number of piles and group effect factor. The group effect factor was assumed as $1/\sqrt{N}$ (N :total number of piles).

Liquefaction Model

Nonlinear response analysis was done considering the liquefaction of soil as well as the linear one. The liquefaction model was composed based on the results of dynamic torsion tests as shown in Fig.6. The model exaggerated the result and is shown in Fig.7. The stiffness is reduced to 15% of the first value at the shear strain $\gamma=0.02\%$, and the strength suddenly falls at $\gamma=0.3\%$. Also in this model the cyclic degradation of stiffness is taken into account in the following form.

$$F(n) = 0.65 - 0.35(n-1)^{0.563} \quad \text{----- (1)}$$

where $F(n)$:stiffness reduction ratio, n :repeated cycles($n \geq 1$)

Natural Periods and Modes

The natural periods and participation functions of the model are shown

in Fig.8. First and second mode is stimulated to large amount. In the first mode only the structure moves, when the free soil stands still. In the second mode the free soil moves in the shape of its first mode and the structure is in the opposite phase.

Response

EL CENTRO 1940 NS and TAFT 1952 EW were used as the popular input motions as there was no observed underground motion of the earthquake. Maximum acceleration was set as 100 Gal for the linear analysis and 200 Gal for the nonlinear one, both at the pile bottom. Damping factors were assumed element by element as shown in Table 1.

Linear and nonlinear maximum response accelerations are shown in Fig. 9. In the nonlinear response the amplification ratio is less than that of the linear response, and the mode is also varied from the linear one. The hystereses of the shear spring of the free soil at the third layer from the bottom are shown in Fig.10, where cyclic degradation of stiffness and strength is clearly noticed. The bending moment of a pile is shown in Fig. 11. The value for TAFT exceeded the crack moment to a large extent, which corresponds to the actual damage.

CONCLUSIONS

Following conclusions can be introduced from the studies previously mentioned

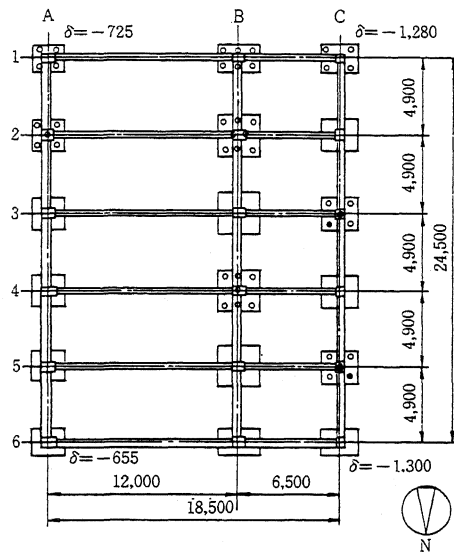
- 1) Through the excavation of piles damaged by liquefaction heavy damage as concrete crush and buckling of bars was revealed. The damaged part was not only at the maximum moment point when the pile head was hinged but also near the pile bottom which penetrated into the stiff layer.
- 2) The response of the building-pile-soil interaction model generally corresponded to the actual damage.
- 3) To rationalize the earthquake resistant design of pile foundations it is necessary to take the building-pile-soil interaction and the nonlinearity or liquefaction of subsoil into account.

REFERENCES

- 1) J.Penzien et al "Seismic Analysis of Bridges on Long Piles"
ASCE, EM3, 1964
- 2) M.Majima & J.Yamashita "Dynamic Characteristics of Saturated Sand under Reversal Loads" Reports of the Technical Reserach Institute, Taisei Corporation, No.14, 1981

Table 1 Assumed Damping Factors

Building	Frequency Proportional $h_1=3\%$ for the base-fixed 1st frequency
Pile	Frequency Proportional $h_1=1\%$ for the 1st frequency with both ends hinged
Rocking	$h=15\%$
Soil Shear Spring	Modal $h_1=20.5\%, h_2=8.1\%, h_3=3.3\%, h_4=1.7\%, h_5=2.4\%$
Soil Lateral Spring	$h=7\%$



● Excavated Piles δ : Settlement (mm)

Fig.1 Plan of the Building

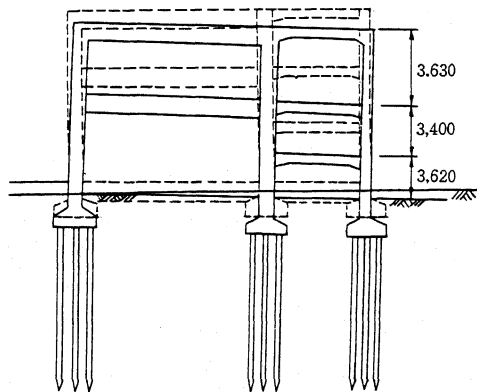


Fig.2 Section of the Building

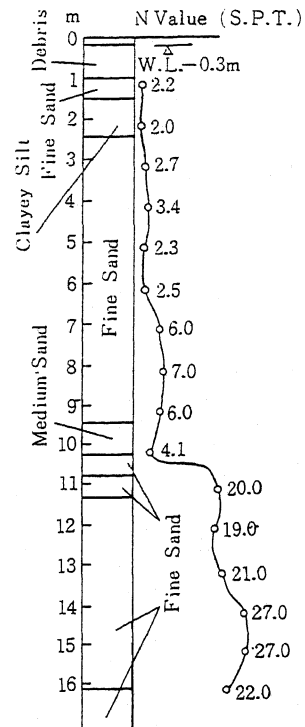
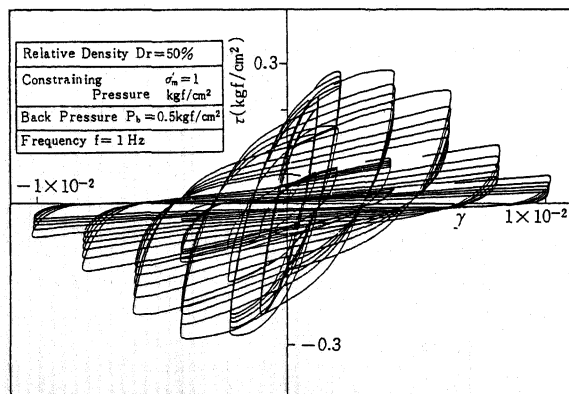
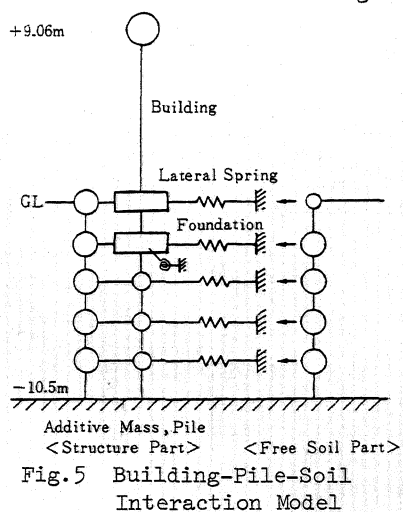
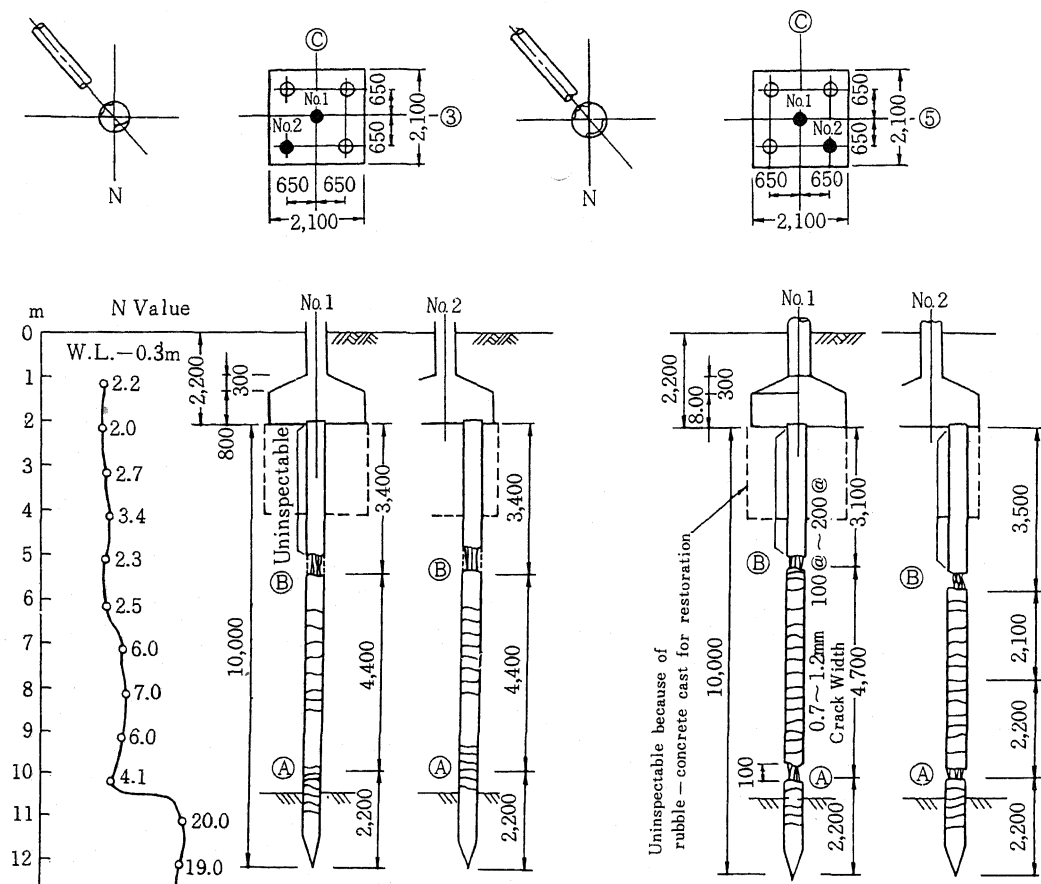


Fig.3 Geological Formation



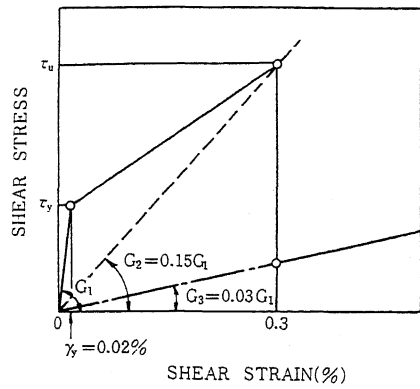


Fig. 7 Liquefaction Model

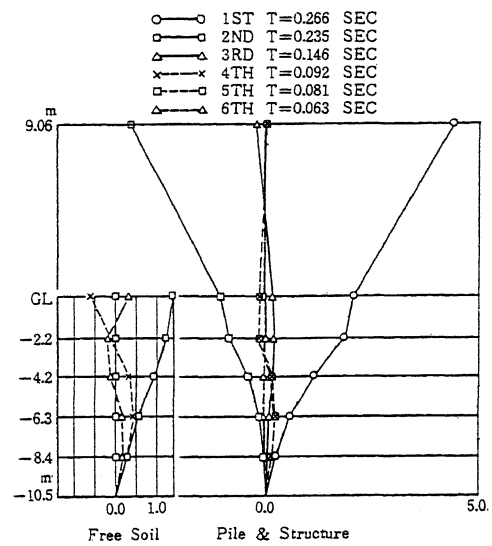


Fig. 8 Natural Period and Participation Function

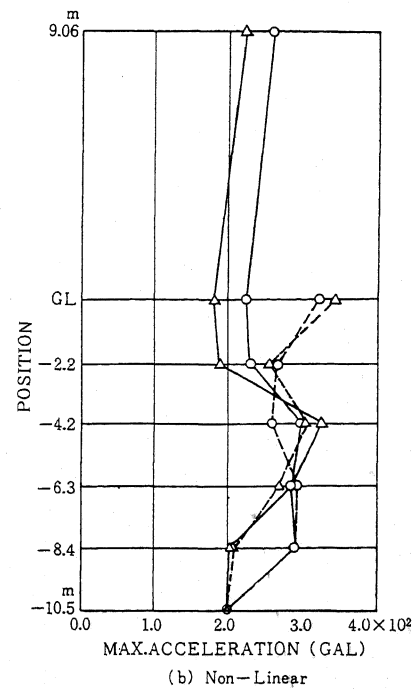
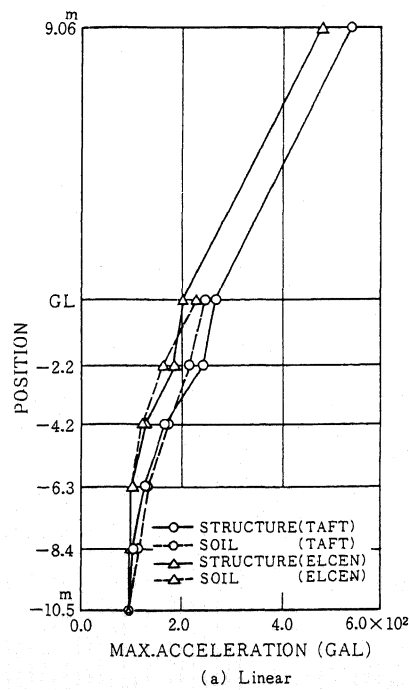


Fig. 9 Maximum Response Acceleration

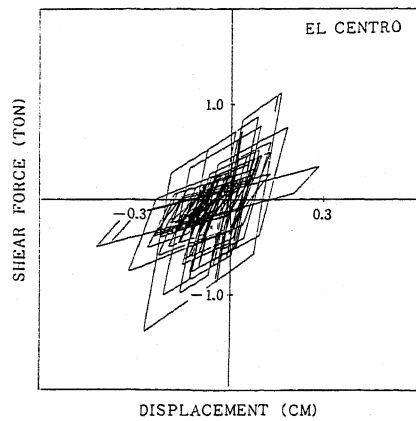
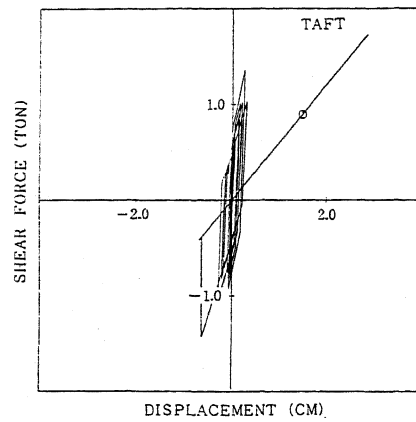


Fig.10 Hystereses of Free Soil

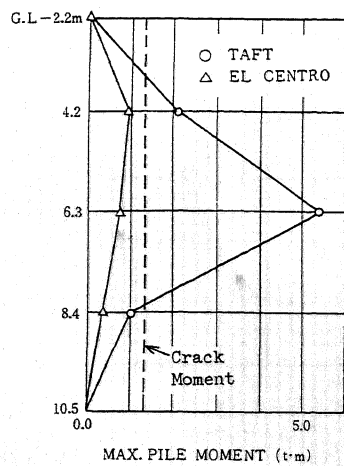


Fig.11 Bending Moment of a Pile

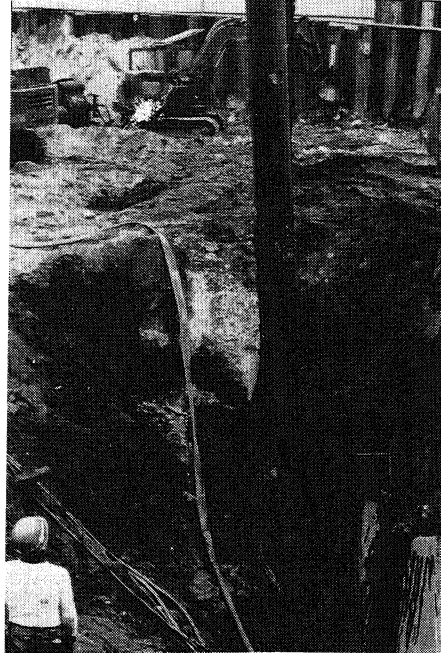


Photo 1 Pile just Pulled off



Photo 2 Damage of a Pile