



DAMAGE TO PILES DUE TO OSCILLATION OF LIQUEFYING GROUND

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SUMMARY

During the Hokkaido Nansei-Oki earthquake with a magnitude of 7.8, in Japan in 1993, extensive liquefaction took place on a reclaimed land in Hakodate Port. A 26-m high, 2500 ton-capacity cement silo supported on a pile foundation with 64 pre-stressed concrete piles at the liquefied site suffered a 1/20 tilt and 90 cm differential settlement due to the damage to all the piles, so it was immediately condemned and demolished. The purpose of this paper is to clarify the mechanism of the damage by numerical analysis and to show the effect of oscillation of liquefying ground on the pile response. First, the distinctive damage to the pile foundation is revealed by a variety of detailed surveys. Subsequently, previously proposed method of two-step dynamic effective stress analysis for soil-pile-structure system using beam-mass-spring model is mentioned with parameter determination for an actual model. The total stress analyses are compared with the effective stress ones. The contribution of inertial and kinematic interactions is clarified. Finally, eyewitness observation clarifies the actual development of the entire damage at the site, and also verifies the effectiveness of the proposed method with good agreement with the numerical results. The significant importance of inertial and kinematic interactions in the oscillation of liquefying soil for pile response is emphasized.

INTRODUCTION

The major seismic actions on the structures supported by pile foundation are undoubtedly inertial force from the structures and seismic or residual displacement of their surrounding ground. There has been a significant amount of research on soil-pile-structure interaction problem. The structural engineers have often understood the ground as a system resisting the displacement of piles and/or as a field for the dissipation of energy due to vibration of the structure, however, they has not been often considered it as a cause of seismic action on pile foundation. The ground flow induced by liquefaction is currently believed to be a significant seismic action on pile foundation, while the ground oscillation amplified by liquefaction is not so widely noticed. The authors have already developed and proposed an analytical method for the effective stress analysis of soil-pile structure in order to evaluate the effect of the ground oscillation on the structure supported by a pile foundation.

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During the Hokkaido Nansei-Oki earthquake with a magnitude of 7.8, in Japan in 1993, extensive liquefaction took place on a reclaimed land in Hakodate Port. A cement silo supported on a pile foundation at a severely liquefied site was subjected to a tilt of 1/20 and settlements of 30 to 90 cm due to severe damage to the pile foundation [1, 2]. This damage is considered to be an excellent and typical example of the damage to pile foundation in a soft and liquefiable soil as a result of a variety of surveys, seismic ground motion record in the vicinity of the site and eyewitness observations.

The objectives of this paper are to present an outline of the damage to the silo and its foundation revealed by a variety of surveys, to illustrate the mechanism of the damage by a series of nonlinear earthquake response analyses, and finally to show the essential effect of oscillation of liquefying ground on pile response. Moreover, the paper aims at calibrating and verifying the proposed method of analysis.

DAMAGE TO THE CEMENT SILO AND ITS PILE FOUNDATION

Damage to the cement silo

The damaged silo is one of the two cement silos located at Kita Wharf in Hakodate Port. Photo 1 shows a view from the west of the silos and the surrounding ground. As seen in this photo, the silo on the right is the damaged one with a tilt forward the right and a pile of ejected gray silt can be seen around the silos. The damaged silo was designed according to the 1967 Design Code and was constructed in 1969, a year after the 1968 Tokachi-Oki earthquake causing liquefaction at this site. The 1967 code has no provisions for the seismic action on pile foundation and pile head connection. This silo had a capacity of 2500 ton, with a height of 26 m, and was supported on a foundation consisting of 64 PC (pre-stressed concrete) hollow piles measuring 22 m long and 40 and 25 cm in external and internal diameters, respectively. The other silo, which was designed according to the 1974 Code and was constructed in 1977, on the other hand had apparently no damage. This one has a capacity of 5000 ton, with a height of 26 m, and is supported on a foundation consisting of 122 PHC (pre-stressed high-strength concrete) hollow piles of 22 m length with 35 and 20 cm external and internal diameters respectively.



Photo 1 View of the two silos from the west and the surrounding ground

Fig. 1 shows the plan view of the damaged site and the location and movement trace of four eyewitnesses who were working at the site during the earthquake that went at 22:17 of July 12, 1993. The figure also shows the outline of the damage to the silo featured by a tilt of 1/20 with no cracks, a maximum settlement of 90 cm and a minimum of 30 cm, and lateral movement of the base by 47 cm toward southwest. The tilting and the lateral movement resulted in the breakage of steel pipe and pathways bridging the two silos with a gap of 1.3 m. Additionally, extensive liquefaction was observed in the vicinity with ejected silt on a massive scale. Water eruption through the silt volcanoes continued for 3 to 5 days. Considering the potential risk, the damaged silo was immediately condemned and demolished after preliminary investigation.

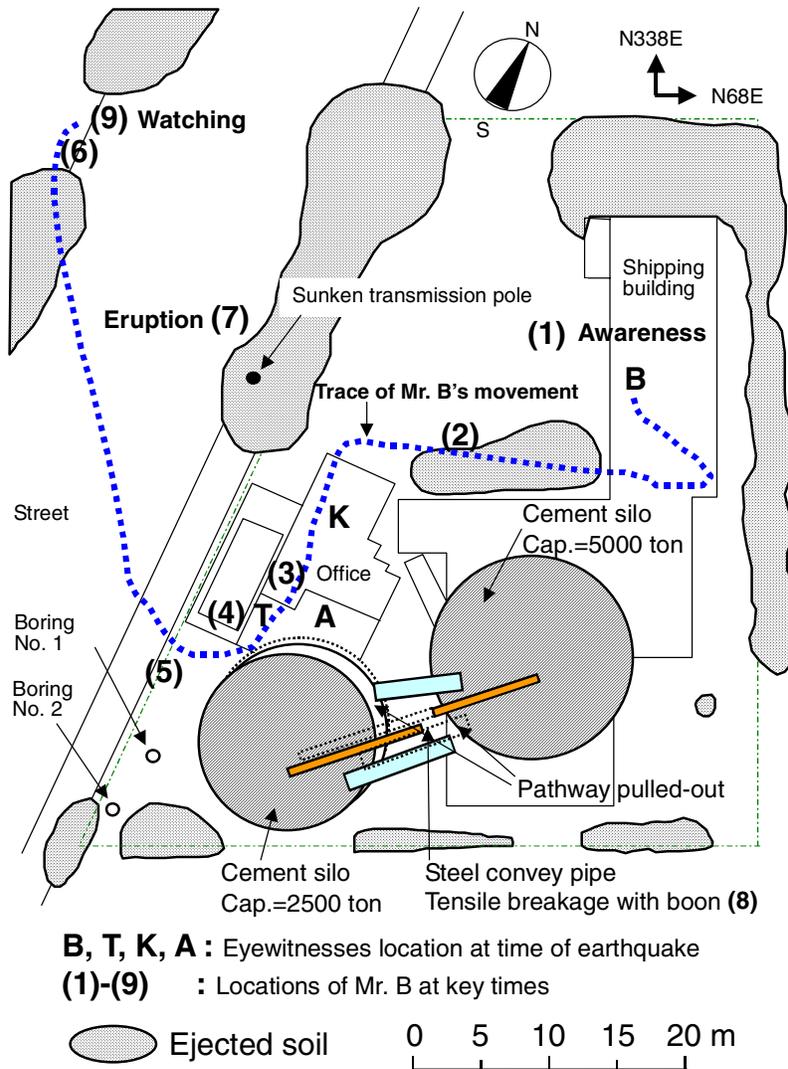


Fig. 1 Plan view of damaged site and location and movement trace of four eyewitnesses

Development of the damage according to eyewitness observation

One of the authors conducted a series of interviews with the workers at the cement company in order to clarify the development of the entire damage at the site (Mori [3]). One of the four workers (Messrs A, B, K and T), Mr. B had a very good memory of his and his colleagues' behavior during the earthquake. His behavior can be summarized in the following nine steps: (1) he was aware of the earthquake shaking on the second floor of the shipping building, (2) he judged the quake to be big and ran out to the office building to stop the machinery about 10-15 seconds after the awareness, (3) he stopped the machinery in the operation room for about 3 to 5 seconds, (4) he urged Mr. T to go immediately out of the resting room (5 seconds) and ran out of there, (5) Mr. T kept sitting on his chair and watched how things went, and fell from the chair due to floor subsidence and resulted difference in level about five seconds after Mr. B ran out, in further 5 to 10 seconds, Mr. T went out, (6) Mr. B tightly held on a transmission pole and watched how things went under strong excitation that continued even after he ran out, went across a street and reached the opposite side of the street. At that time, most of the lights had gone out due to power break, (7) he was aware of mud-water eruption up to his waist level at other transmission pole under the moonlight, (8) he heard a big boom presumably of the breakage of the steel conveyer pipe linking the two

silos at the top together with Mr. T who followed him to come across the street 10-15 seconds after the awareness of mud-water eruption, and (9) he continued to watch until the situation became calm. The time taken in each step was measured based on the repeated revival of his movement during the quake. This revived movement can be effectively used to verify numerical results in time histories of some responses of soil-pile-structure system. The time history of Mr. B's movement will be discussed later.

Damage to the pile foundation

The damaged silo was immediately condemned and demolished to prevent its overturning. After demolishing the superstructure of the silo, an excavation survey was conducted to check the damage to the pile caps and pile heads. All the pile heads were exposed and observed up to 1 to 3 meters below the pile tops. The investigation revealed that all the piles were heavily damaged near the pile heads, and their failure could be found to have two distinctive patterns as typically illustrated in Fig. 2 [1]. One of the failure patterns is shear failure or bending-shear failure near pile head with shortening of the pile length due to crushing, which is to be called as Type-A. This type was recognized in 35 piles. The other failure pattern is bending failure at a depth of 1 to 3 meters below a sound pile top with large tilting, which is to be called Type-B. This type was recognized in 29 piles. It was supposed that the Type-A is due to the bending moment predominant at the pile head in the piles connected rigidly with the pile cap, and that the Type-B is due to the bending moment at a few meters below the pile head in the piles pin-connected with the pile cap.

The variation of the two failure patterns of the pile heads is shown in Fig. 3, where the numerals indicate the serial numbers of the piles from 1 to 64. The variation is apparently seems to be irregular. Mori [1] clarified that the two failure patterns depend on the ratio of the length of penetration to the pile cap, P to the pile diameter, D based on analysis of photographs taken just after piling under construction. The criterion for classifying the failure pattern is $P/D=0.2$. The Type-A piles behaved as rigid pile-cap connection due to sufficient penetration, while the Type-B piles behaved as pinned connection due to insufficient penetration.

Damages to some deeper portion of piles were observed in all piles with the help of the internal inspection of PC piles by a CCD camera. The

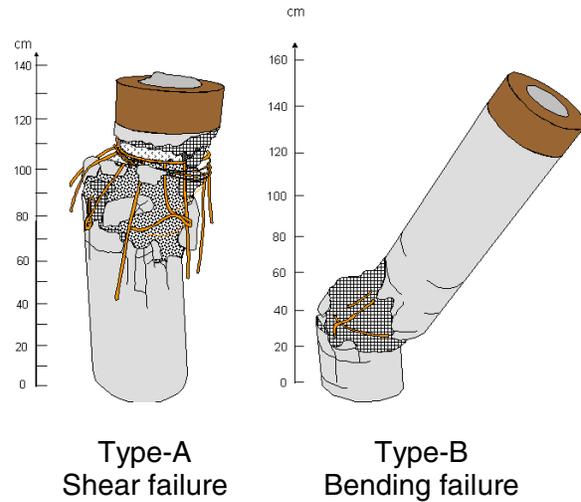


Fig. 2 Two failure patterns of the pile heads

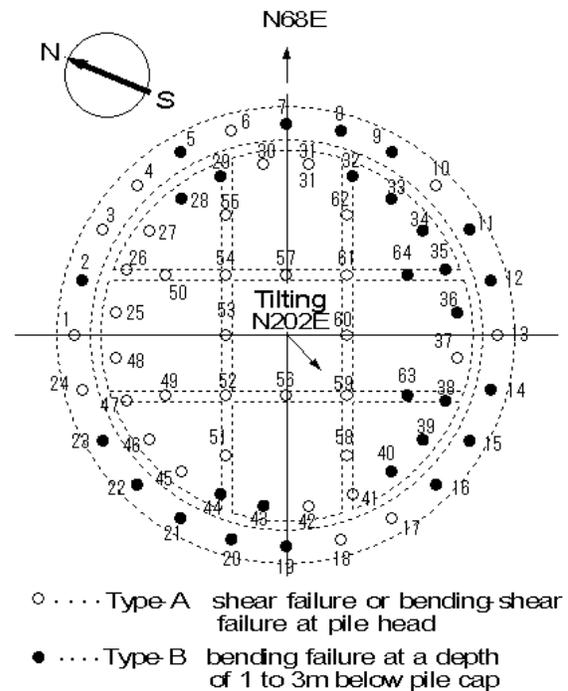


Fig. 3 Variation of the two failure patterns of pile head

results of inspection on seven piles were almost the same indicating that over one-meter portions of the piles near a depth of 6.5 m were failed completely with large shear deformations and longitudinally predominant cracks on the piles, and a lateral shift with an amount of half the pile radius was observed at the breakage depth due to the shear deformation. The depths having the shear failure of piles, i.e., about 6.5 m correspond to the bottom of reclaimed layer or the top of the old seabed (Mori [2]).

NUMERICAL ANALYSES

Fig. 4 shows the schematic models and procedure of analysis proposed by Mori [2, 4]. The proposed model, whose basic concept is somewhat similar to the model of Penzien [5], consists of three systems namely pile-structure, adjacent soil, and free field soil systems. In the model, pile-structure and adjacent soil systems are connected rigidly, and adjacent and free field soil systems are connected with springs for interaction [4]. Effective stress approach for this coupled system is carried out by two-step analysis. The first step is effective stress analysis (ESA) for free field soil system with an input motion on the bedrock. The response of excess pore water pressure (EPWP) calculated in the first step is taken in as input into the second step of ESA for the coupled system. Shear springs in the adjacent soil system and the free field soil system, and springs for interaction are nonlinear models depending on strains and confining pressures in terms of stiffness and strength. The confining pressure is transiently controlled by the time histories of the response of EPWP. If the response of EPWP were not introduced into the second step, total stress analysis (TSA) for the coupled system would be automatically performed.

Such springs are modeled based on hyperbolic model whose parameters are initial stiffness and strength, and proportional to the square roots of the confining pressures. Shear springs in the two soil systems are determined as soil columns. Their stiffnesses are calculated with the shear wave velocities and soil densities of corresponding soil layers, and their strength are simply calculated by Coulomb's formula with initial effective overburden pressures and internal friction angles of soils. As for springs for soil-pile interaction, the stiffness is determined by using Mindlin's second solution based on the idea of Penzien [5], and the strength is determined as three times of Rankine's passive pressure. These two parameters are multiplied by a factor of group pile effect, $N^{-1/2}$, where N is number of piles. ESA for free field adopts Ishihara and Towhata [6] method, which is simple one dimensional shear spring-lumped mass model with their unique seismic pore water pressure generation model requiring three parameters, B_p , B_u and κ . If parameters B_p and B_u were zero, analysis would be identical to TSA. The procedure of determination of all input parameters is identical with the authors' preceding paper [2]

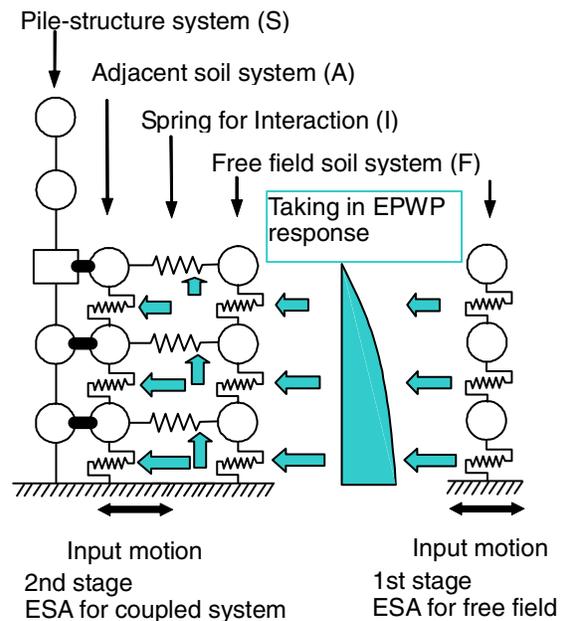
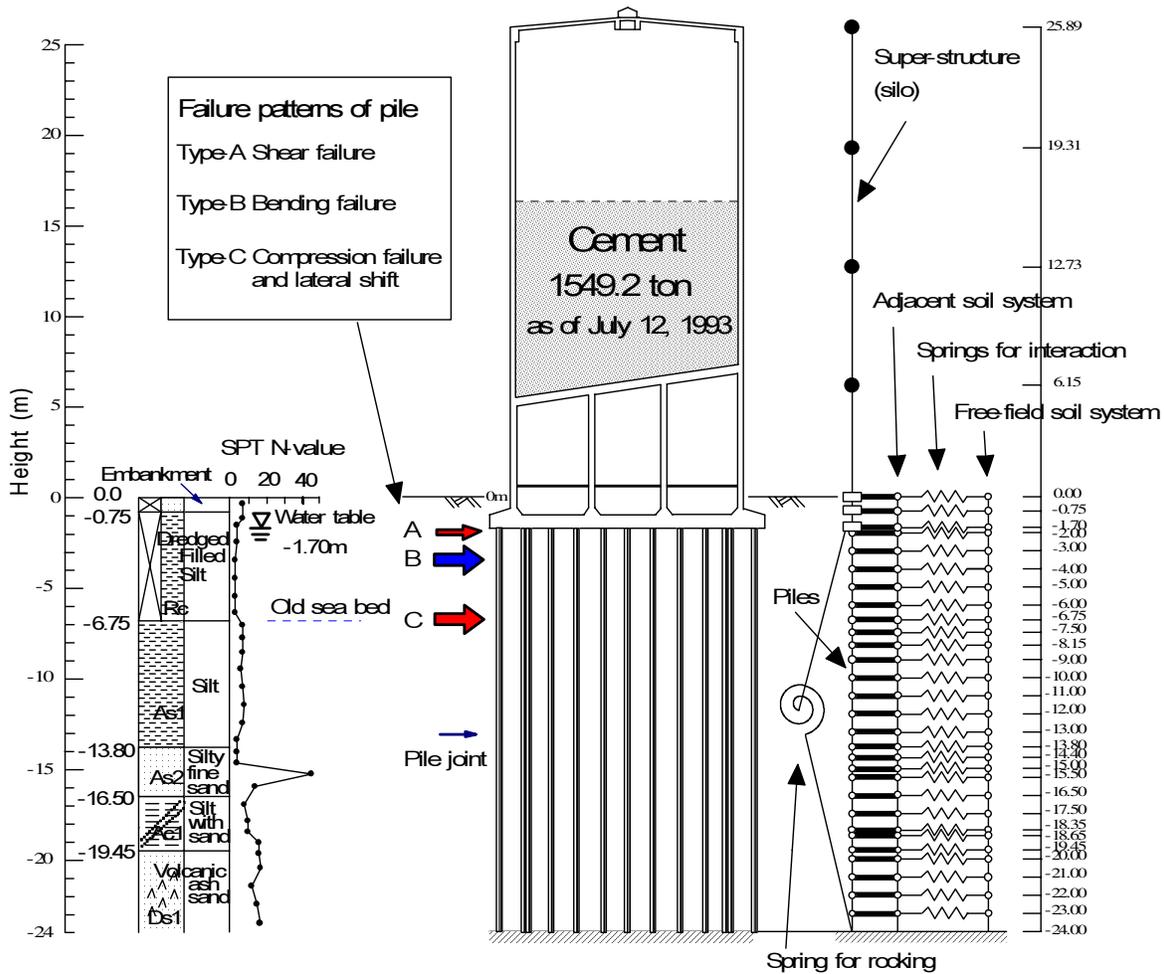


Fig. 4 Schematic models and procedure of analysis proposed by Mori [2, 3]

Fig. 5 shows the specific model of analysis in association with ground profile of the site, the three depths of damage to piles and the entire structure of silo and piles. The lower boundaries of the coupled systems and free field soil systems are located at a depth of 24 m, which is identical to the bottom of the piles. The lower boundaries are modeled as fixed ones. Two soil and pile systems are sufficiently discretized for distinguishing the three depths of damaged portion. The mass of cement contained as of the time of the earthquake is incorporated with masses of superstructure in the model. Two cases of pile cap connection were considered, which include a case of rigid connection in all piles and other of pinned connection in all piles. In this paper, however only the case of rigid connection will be discussed to focus on to the triggering mechanism of the damage.

A boring and SPT test was conducted 12 days after the earthquake (Boring No.1 in Fig.1). The test showed the N values to be zero and one respectively at depths of 7.5 and 6.5 m, whereas it was found three above a depth of 5.5 m. This indicates that the bottom two meters of the filled silt layer was severely liquefied. Therefore, another boring and SPT test was performed 120 days after the quake (Boring No.2 in Fig.1), which showed that the recovery of the N values 7.5 and 6.5 m depths was up to 1.5 and 2.5,



(a) Soil profile (b) Pile damage (c) Structure and foundation (d) Analysis model

Fig. 5 Specific model of analysis in association with ground profile of the site, the three depths of damage to piles and the entire structure of silo and piles

respectively. So, the above-mentioned speculation appears to be true. Moreover, a series of physical property tests were carried out for the soil samples from every one meter up to a depth of 24 m. As a result, the properties of ejected soils were found to coincide with those of the filled silt layer varying from a depth of 0.75 to 7.75 m. Accordingly, the entire filled silt layer was estimated to have liquefied and the bottom two meters of the layer was estimated to have severely liquefied, based on the above investigation.

For determination of parameters required for dynamic effective stress analysis of the coupled system, various kinds of surveys were conducted, which include PS logging and thin-wall sampling of undisturbed soil. Furthermore, triaxial tests were carried out for rigidity and damping ratio dependent on cyclic strain, for cyclic liquefaction resistance, and for static strength.

The input motion is determined as a total motion response at the lower boundary of the free field soil system of the site obtained by SHAKE analysis, in which the input incident motion for the model is calculated from deconvolution analysis by another SHAKE analysis with a strong motion recorded during the quake on a site where the strong motion record was acquired (Hakodate Office of Hokkaido Development Bureau). The time history of the incident motion will be shown, together with some responses of the free field in a figure to be shown later, as an outcrop motion that is double amplitude of the incident one. The input motion is NS component, which is predominant in amplitude. The north direction is positive in this study.

RESULTS OF ANALYSES

Response of the free field

The distributions of maximum responses, including displacement, acceleration, shear stress, shear strain and excess pore water pressure (EPWP) ratio, along the depth in free field are shown in Fig. 6. Each graph, except for EPWP ratio, has the results of ESA and TSA. Comparison between ESA and TSA results helps us understand the effect of generated excess pore water pressure and liquefaction in the free field soil system.

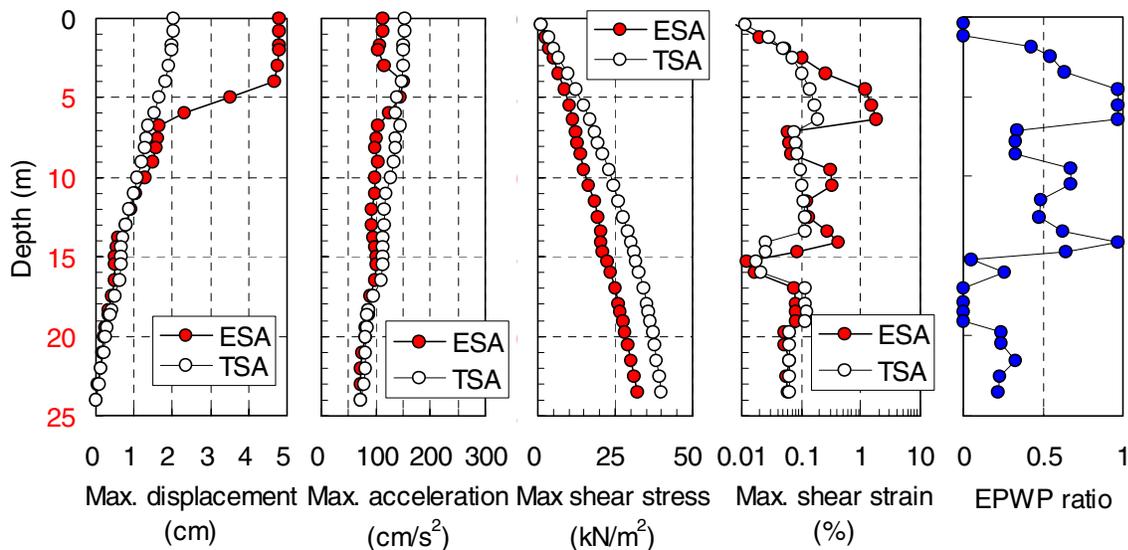


Fig. 6 Distributions of maximum responses, including displacement, acceleration, shear stress, shear strain and excess pore water pressure (EPWP) ratio in free field

The distribution of EPWP ratio indicates two ranges of depths where the ratio almost reaches one, meaning the liquefaction take place. The shallower part of the liquefied ranges corresponds to the lowest part of dredge-filled silt layer just above the old seabed. Therefore, the result of numerical analysis matches the actual situation that was observed at the site and is understood from the various kinds of surveys and tests on the soil samples obtained from the site.

At the depths with EPWP ratio greater than 0.5, the maximum shear strains by ESA are much greater than those by TSA. Such greater strains are particularly significant in liquefied depths, resulting in the increase of maximum displacement. In contrast, those can be understood to reduce the maximum acceleration and the shear stress in the soil.

The time histories of acceleration of the ground surface both by ESA and by TSA are shown together with the time histories of EPWP ratios for four liquefiable sub-layers in Fig. 7. Between the acceleration time histories of ESA and TSA, no difference before about 20 seconds and slight difference before 45 seconds can be seen. The acceleration by ESA after 45 seconds, however, is de-amplified and is poor in shorter period component. This change is due to gradual generation of EPWP and liquefaction near two depths at and after about 57 seconds. It should be noticed that the 57-second corresponds to the times when liquefaction occurs in ESA and the maximum amplitude of the acceleration appears in TSA.

Fig. 7 also shows the time histories of displacement of ground surface by ESA and TSA. A drastic change starts at about 50 seconds and obviously appears at and after 57 seconds when liquefaction occurs. In TSA, displacement reaches maximum value at the 57 seconds and attenuates beyond this time, which is similar to the acceleration time history. On the other hand, the displacement in the case of ESA is suddenly amplified and reaches its maximum value at the time, and such amplitude repeats even after 57 seconds. This amplification of displacement is undoubtedly due to liquefaction at the depths of 4 to 7 m according to the distribution shown in Fig. 6.

Responses of pile-structure in different pile cap connections

The distributions of the maximum responses including acceleration and displacement along the height of the silo-pile system are shown in Fig. 8. With regard to the acceleration response of the silo, the results of ESA are smaller than those of TSA, because of the de-amplification of the ground acceleration due to

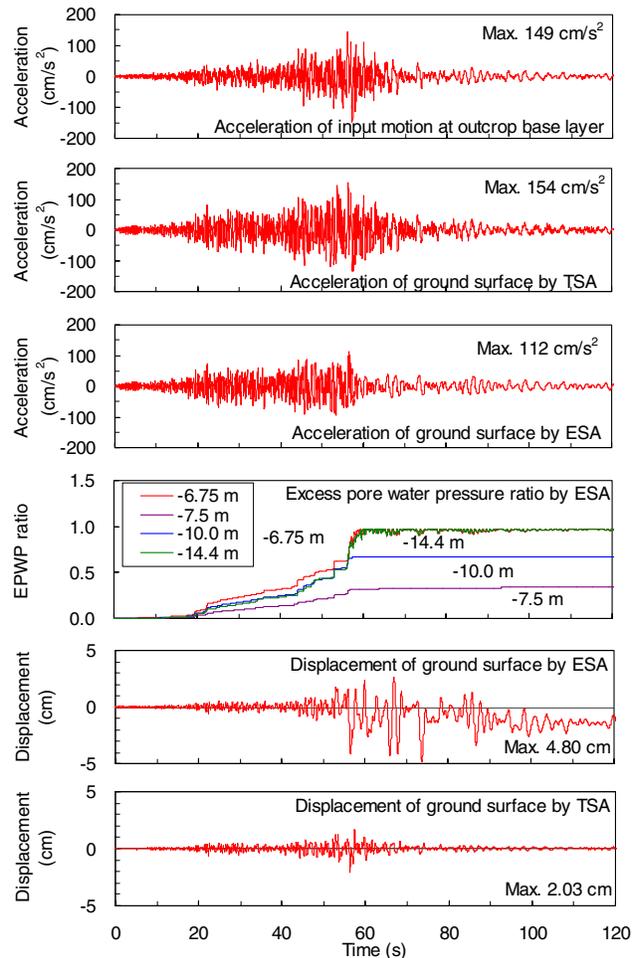


Fig. 7 Time histories of acceleration and displacement of ground surface by ESA and TSA, and excess pore water pressure ratio by ESA

liquefaction. With regard to the displacement response of the silo, on the other hand, the results of ESA are greater than those of TSA because of the amplification of the ground displacement due to liquefaction.

Response of the piles

The distributions of maximum bending moment of the pile along the depth as obtained by ESA and TSA are shown in Fig. 9. From the figure, two distinctive features can be pointed out. The first feature is that the maximum bending moment is generated at the pile head in both the analyses and the moment in ESA is greater than that in TSA. The second feature is that the second maximum bending moment is produced at a depth of 5 m in TSA and at 7 m in ESA with greater magnitude. As the maximum bending moment in ESA as well as TSA are produced near the pile head, it may be appropriate to take a look at the pile head.

The traces of pile stress at the pile head in a plane of bending moment and axial force are shown in Fig. 10, together with ultimate and cracking surfaces. As seen in the figure, in TSA, the pile stress reaches the cracking surface but does not reach the ultimate surface, whereas, the pile stress in ESA once goes beyond the ultimate surface and repetitively reaches it thereafter.

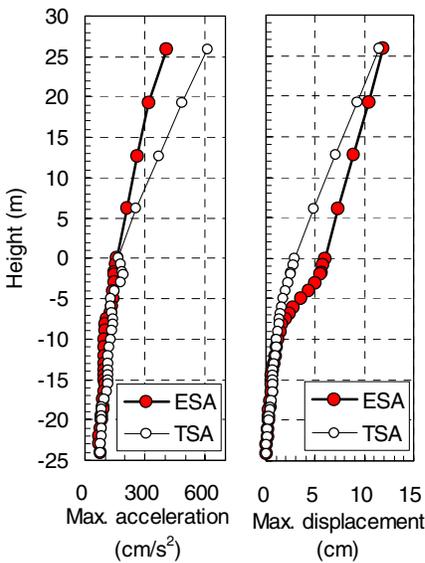


Fig. 8 Distributions of maximum acceleration and displacement along height in silo-pile system

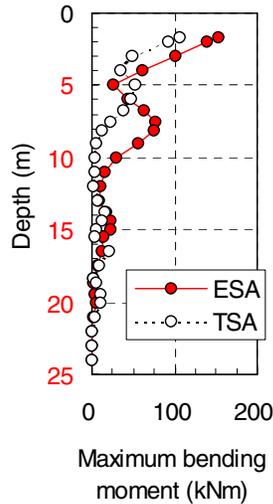


Fig. 9 Distributions of maximum bending moment of pile

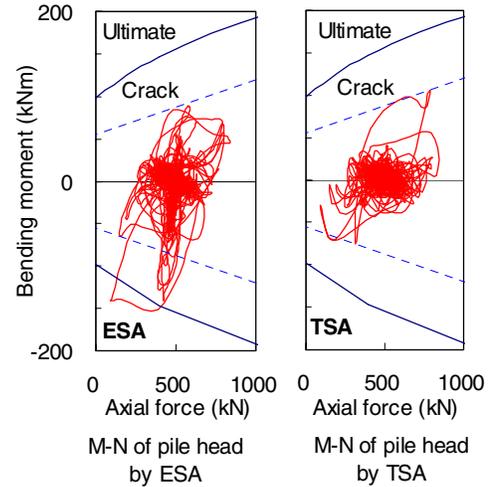


Fig. 10 Traces of pile stress at the pile head in a plane of bending moment and axial force

The time histories of bending moments of the pile head are shown in Fig. 11. The difference between the two, as seen in the figure, appears after 45 seconds, and the maximum moment is observed at about 57 seconds in both the analyses.

In TSA, the moment reaches the maximum magnitude at 57 seconds and attenuates thereafter. The waveform is roughly similar to the time histories of acceleration and displacement of the ground surface. In ESA, on the other hand, the moment is suddenly amplified and reaches the maximum value at 57 seconds with repetition of such amplitude. The waveform is very similar to that of the displacement of ground surface, especially after 57 seconds when liquefaction occurs.

If the maximum values at about 57 seconds were mainly induced by inertia force of the silo and foundation, the actual maximum value of the bending moment would be 1.3 times greater than the calculated value as mentioned earlier. Moreover, the bending moment of the pile seems to be significantly dominated by the displacement of liquefied ground. The residual bending moment of the pile and the displacement of the ground can be seen in Figs. 11 and 7, respectively. They match with the direction of residual movement of the silo base observed after the quake.

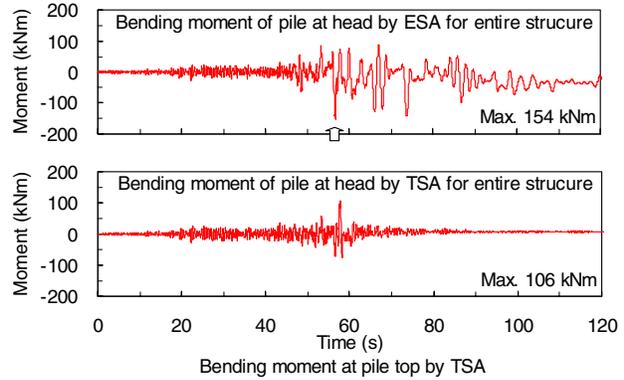


Fig. 11 Time histories of bending moments of pile at head

DISCUSSIONS ON THE MECHANISM OF DAMAGE AND THE VERIFICATION OF ANALYSES

Effects of inertial and kinematic interactions

In order to clarify the contributions of inertia force of silo (superstructure) and ground displacement, additional ESA and TSA for a coupled system without the silo were performed for comparison with the standard coupled system (system with the silo). Comparisons with regard to the distribution of bending moment of pile along the depth in ESA and TSA between the systems with and without the silo are shown in Fig. 12. In TSA, the system with the silo shows greater bending moment than the system without the silo. This means that the inertia force of the superstructure is influential to pile response in case of no liquefaction. On the other hand, little difference is found between the two systems in ESA, which means that the inertia force of the superstructure is not essential in maximum pile response in case of a liquefying ground.

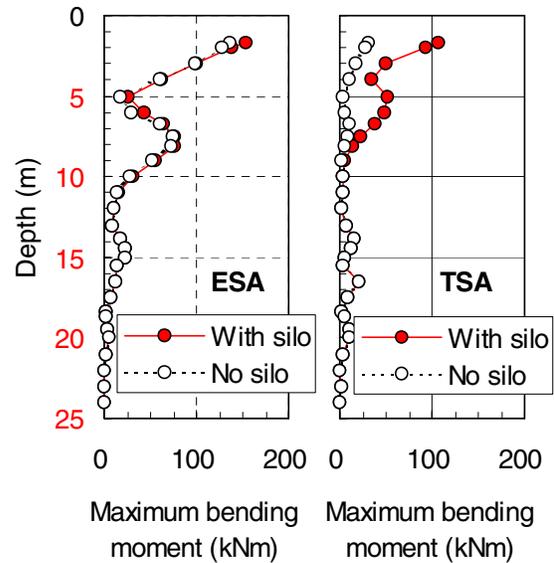


Fig. 12 Comparisons with regard to distribution of bending moment of pile along a depth in ESA and TSA between systems with silo and with no silo

The mechanism of pile stress response is discussed by comparing the time histories of the bending moment of pile near the head for the entire structure and the system with no structure as well as by comparing the ESA and TSA. In a coupled system with no superstructure, major seismic action on the piles is the displacement of the surrounding ground, while the inertia force of the superstructure acting on the pile foundation are added to the major action on the piles in a coupled system for the entire structure. The time histories of bending moment of the pile near the head by ESA and TSA for the system with no superstructure are shown in Fig. 13.

The ESA for the system with no superstructure results in a time history that is very similar to time history obtained by ESA for the system for the entire structure after 60 seconds when EPWP distribution reaches

a maximum value at every point along the depth. The maximum bending moment in this case is generated at 74 seconds and the magnitude is almost the same as the bending moment in case of the entire structure at the same timing, which is the second maximum. Accordingly, it is understood that the kinematic action due to ground displacement is highly dominant in the bending moment of piles in liquefying soils.

On the other hand, the TSA for the system with no superstructure results in a time history that is proportionally smaller than that from TSA for the system consisting entire structure except for the time range from 50 to 60 seconds where the amplitudes are differ largely. The maximum value is merely about 30 % of the value in the case of entire structure. Therefore, it is understood that the inertia force of the superstructure is more dominant for the bending moment of the pile in non-liquefiable soil than the kinematic action due to ground displacement.

This understanding will be confirmed from the difference in time histories of bending moment in case of the entire structure and case of no superstructure, as shown in Fig. 14. This figure represents the effect of the superstructure on bending moment of piles in terms of inertial action. The difference between the two cases is negligible until 47 seconds when EPWP ratio is smaller than 0.5 all along the depth. Sudden amplifications appear at around 48 and 57 seconds in ESA and 57 seconds in TSA. The amplifications in these time ranges can be understood to be induced by transient resonance of the superstructure to transient ground motion due to nonlinear behavior of soil. In a series of ESA, the bending moment at 57 seconds in the entire structure has a maximum value of 154 kNm including the contribution of 94 kNm due to kinematic action. The difference between them is almost equal to the maximum difference in the bending moment time histories. This indicates that the effects of two actions, i.e. inertia force from superstructure and kinematic action due to ground displacement, can be estimated to be 40 % and 60 % dominant respectively, and these two effects can be simply added in this case. In a series of TSA, bending moment due to inertia force is slightly greater than the total moment. Accordingly, it is understood that the kinematic action has an effect of reducing the total bending moment, while the inertia force of superstructure is highly dominant. Eventually, the inertia force from superstructure amplified due to transient resonance induced by nonlinear soil-pile structure interaction is significant in evaluating the performance of pile foundation as well as kinematic action due to ground displacement in liquefying soils.

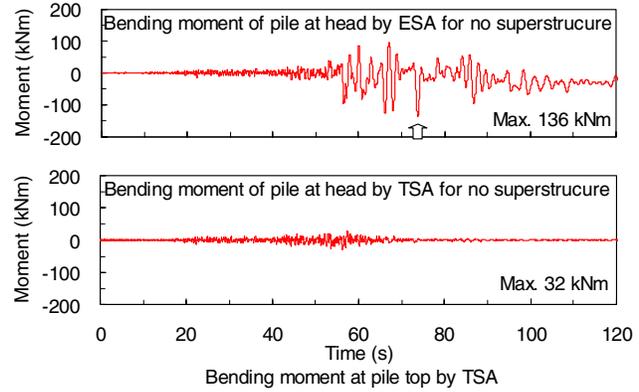


Fig. 13 Time histories of bending moments of pile at head for coupled system with no superstructure

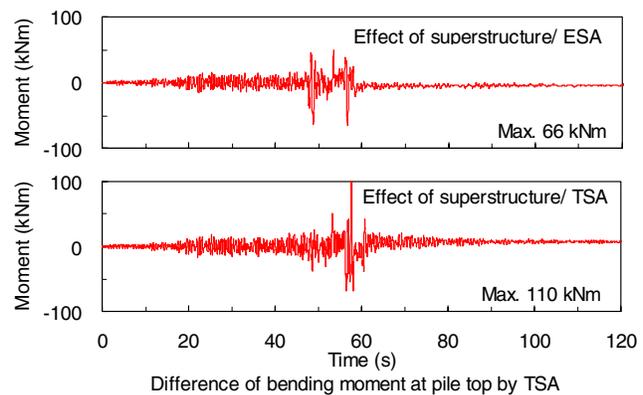


Fig. 14 Time histories of bending moment of pile at head due to inertia force of superstructure

Verification of the analytical results by eyewitness observation

As described earlier, eyewitness' observation based on the interview and revival of their movement after the quake will be used to verify the time histories of responses of ground, silo and pile foundation obtained by the dynamic analyses. Actual behaviors of the eyewitnesses during the quake will be matched with the time histories based on some assumptions. It is assumed that Mr. B's awareness of the earthquake for the first time corresponds to the surface ground motion of 30 cm/s^2 and his judgment of the quake being big corresponds to the acceleration of 50 cm/s^2 . The remaining featured times of the eyewitnesses, ground, and structural behaviors will be assumed based on the elapsed time measured in the revival of movement.

The development of the entire damage at the site estimated based on the eyewitness observation incorporated with the time histories of ground surface acceleration and bending moment of pile near the head is shown in Fig. 15. The subsidence of the floor of resting room due to separation between the room and the foundation of the damaged silo is estimated to have occurred at around the time of maximum ground motion, liquefaction occurrence and maximum bending moment of pile near the head in the ESA for the entire system. Mud-water eruption due to liquefaction is estimated to have been recognized just after the liquefaction in the ESA. The breakage of the conveyer pipe is estimated to occur during the time range of repetition of bending moment with amplitude reaching ultimate surface. The excellent match of these timings is considered to verify the effectiveness of the effective stress analysis in this study.

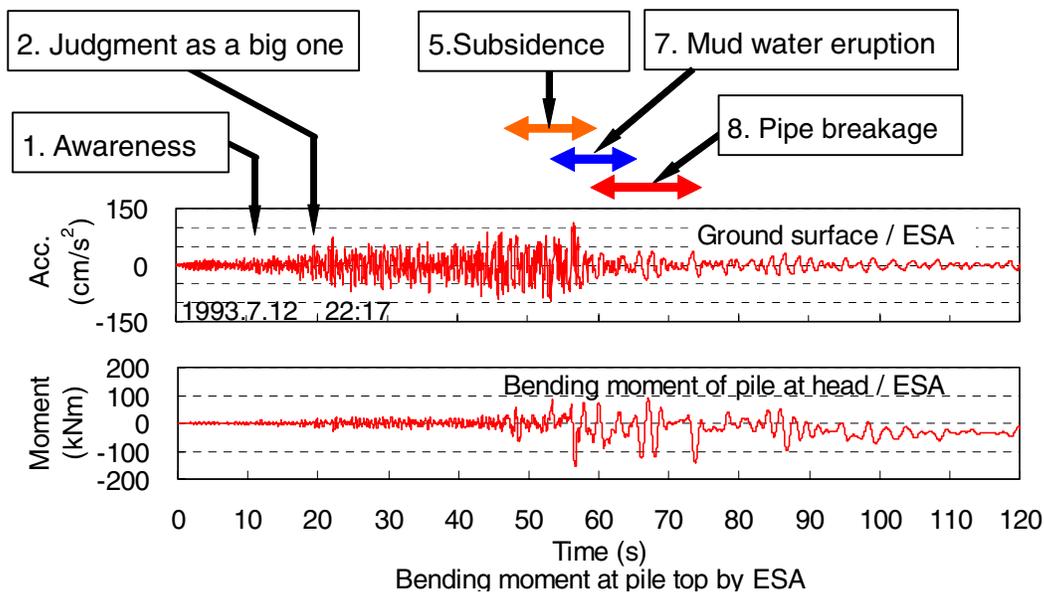


Fig. 15 Development of entire damage at site estimated based on eyewitness observation incorporated with time histories of ground surface acceleration and bending moment of pile at head

CONCLUSIONS

The severe damage to cement silo supported on a pile foundation in the liquefied ground in Hakodate during the 1993 Hokkaido Nansei-Oki earthquake was investigated by a systematic series of effective and

total stress analyses (ESA and TSA) based on the model and the procedure proposed by Mori [2, 3]. The results of this study can be concluded as follows.

- (1) According to the site observation and various kinds of investigations, 7 m thick filled silt layer above an old seabed is estimated to have entirely liquefied. The piles that were rigidly connected with the pile cap suffered complete failure near the pile head and near a depth corresponding to the bottom of the filled layer.
- (2) According to the traces of pile stress in the plane of bending moment and axial force by ESA and TSA, only the trace in ESA repetitively reaches the ultimate surface and also goes beyond it. So, the liquefaction played a significant role in the damage.
- (3) If liquefaction did not occur, the pile stress would be highly dominated by the inertia force from the superstructure, which is influenced by soil-pile structure interaction. However, the major dominant seismic action on the pile is kinematic action due to ground displacement amplified in the liquefying soil. The inertia force from the superstructure is simply added to the major seismic action. Such inertia force is amplified through the interaction by transient resonance due to nonlinear ground behavior. Therefore, oscillation of the liquefying soil is emphasized to be of significant importance in terms of inertial and kinematic interactions.
- (4) Eyewitness observation clarified the developments of the entire damage at the site, and verified the effectiveness of the effective analysis based on the proposed.

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