UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
DAIRYLAND POWER COOPERATIVE	)	Docket No. 50-409-SC
(La Crosse Boiling Water Reactor)	)	(Order to Show Cause)

AFFIDAVIT OF JOHN T. GREEVES
IN SUPPORT OF MOTION FOR SUMMARY DISPOSITION

I, John T. Greeves, do hereby depose and state:

I am a Geotechnical Engineer employed by the Nuclear Regulatory Commission in the Hydrologic and Geotechnical Engineering Branch of the Division of Engineering in the Office of Nuclear Reactor Regulation. I have been employed by the NRC since 1974. A statement of my professional qualifications are attached to this affidavit. This affidavit is submitted in support of the August 1980 safety evaluation issued by the Office of Nuclear Reactor Regulation concerning liquefaction potential at the site of the La Crosse Boiling Water Reactor (LACBWR). This affidavit responds to various matters raised by the consolidated parties in response to the NRC Staff's interrogatories.

As part of my duties, I observed on site most of the test boring program conducted at the LACBWR site in July 1980. I have reviewed information submitted by Dairyland Power Cooperative and its consultant Dames and Moore and I have reviewed information provided to the NRC by its consultant, the U.S. Army Engineer Waterways Experiment Station. I assisted in the preparation of the Office of Nuclear Reactor Regulation's safety evaluation of August 1980 concerning liquefaction potential at the LACBWR site.

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Based on my professional experience, my observations, and my review of the test borir program conducted at the LACBWR site in July 1980, it is my opinion that the borings and Standard Penetration Tests were conducted in accordance with accepted engineering practices. It is my further opinion that the results of the boring program show soil density has improved as a result of pile-driving under pile supported structures. It is my opinion that the soils under the turbine building and the reactor containment building are safe against liquefaction in the event of an earthquake up to magnitude 5.5 with a peak ground acceleration of 0.12g. I believe that the Staff's safety evaluation of August 1980 accurately represents the results of the boring program and I agree with the analysis and conclusions stated therein.

I have also reviewed the responses of the consolidated parties to the NRC Staff's interrogatories. I have prepared the following responses to the matters raised by the consolidated parties.

The consolidated parties appear to assert that there is some significance regarding liquefaction potential for a case where piles are not supported by bedrock. The Staff is aware that the piles under the LACBWR structures do not touch bedrock. The 1978 report of the Staff's consultant, the Waterways Experiment Station, clearly indicated that the piles under LACBWR structures are not founded on bedrock. The Staff's safety evaluation considered liquefaction potential for the site soils in the free field and under pile supported structures. The fact that the piles do not touch bedrock does not affect the liquefaction potential of site soils. The Staff's liquefaction safety evaluation does not rely on the structural capacity of the piles themselves to preclude soil liquefaction. Rather, the increased soil density caused by the driving of piles, as demonstrated by the borings, is a major basis for concluding that the soils under the pile-supported structures are safe against liquefaction.

The consolidated parties disagree with the Staff's position that the borings performed under the turbine building and stack foundation are representative of adjacent structures that are pile-supported. They do not present a basis for their disagreement nor do they specify what their position is on this matter.

The Staff's safety evaluation stated that the borings under the turbine and stack foundations are considered representative of other adjacent pile-supported structures. See Safety Evaluation at 5. In my view, these borings are in fact a conservative representation of the range in soil conditions below pile-supported structures. The borings made through the turbine building foundation slab are considered representative of the poorest foundation soil under the pile-supported structures. The boring locations in the turbine building were carefully selected to represent the lower end of the range of soil density for foundation support for pile-supported structures. The turbine building piles are spaced relatively far apart with respect to other structures, such as the stack foundation and the reactor building. See Figure 2d-2-1 in the Dames & Moore report, Response to NRC Review Questions (July 11, 1980) which indicates pile-spacing under the LACBWR site structures. Because of the wider spacing in the turbine building, minimum improvement in soil density due to densification by pile-driving has occurred at the location of the turbine borings. In addition, soil density generally increases with the distance from the river bank. As indicated in Figure 1 in the Staff's safety evaluation, the borings under the turbine building foundation were taken in the northwest corner of the turbine building, a location closer to the Mississippi River than most other locations in the turbine building and locations in the stack foundation and reactor building. The boring locations in the turbine building therefore are representative of low initial soil density and wide pile-spacing (minimum improvement) under pile-supported structures. The borings are, therefore, a conservative representation of soil conditions under other pile-supported foundations on the LACBWR site.

The borings under the stack foundation are a conservative representation of the conditions under the reactor. As indicated in Figure 2d-2-1 of the July 11, 1980, Dames & Moore report, there is a higher density of pile spacing under the stack foundation and reactor building than under the turbine building. Spacing under the stack foundation and the reactor building is similar. The reactor building is also founded at a lower elevation than the stack foundation and the turbine building. The reactor foundation is below the hydraulic fill soils. Figure 2 in the Staff's safety evaluation of August 1980 represents a typical soil profile for the LACBWR site. As indicated in Figure 2 in the Staff's safety evaluation and in Figure 2d-2-1 in the July 11th Dames & Moore report, the plant grade is at an elevation of 639 feet. The bottom of the reactor containment is at an elevation of 610 feet. See Figure 2d-2-1 to the July 11, 1980 Dames & Moore report. As stated in the Order to Show Cause, 45 Fed. Reg. at 13,850, col. 3, and 13,851, col. 1, the Staff was originally concerned that liquefaction might occur in soils below the water table down to a depth of 40 feet. The foundation of the reactor building, which is almost 30 feet below grade, is therefore founded below most soils in which the Staff has been concerned liquefaction might occur. Tests on borings through the stack foundation, which is located at a higher elevation than the reactor containment, showed high soil density at all levels of the boring. See Figure 8 in the Staff's safety evaluation. In view of the similarity in pile-spacing between the two structures and the lower elevation of the reactor building foundation, the borings under the stack foundation are a conservative representation of conditions under the reactor building.

The consolidated parties raise a concern about the possibility of voids under the reactor building. Voids were encountered under the turbine building which is founded in the hydraulic fill. No voids were encountered under the stack foundation at a similar elevation. See Log of Borings, Plates A-3 through A-6 in Dames & Moore's report Final Assessment of Liquefaction Potential at LACBWR Site (July 25, 1980). The results of the borings under the stack indicate that no voids are under structures supported by densely spaced piles. The voids under the turbine building may be attribued to the wider spacing of piles and to the nature of the soil directly under the voids to a depth of about 25 ft. The reactor building is founded on more densely spaced piles in better soil conditions.

The consolidated parties also raise increased pore pressure as an issue. Driving of piles can increase pore pressure as well as soil density. However, any increased pore pressure dissipates shortly after the pile is driven. Increase in pore pressure has no significance after dissipation. Increased soil density remains and significantly improves the liquefaction resistance of the soils, as is discussed in Staff's safety evaluation.

In my view, dense soils can remain stable even if adjacent soils undergo liquefaction. This has been demonstrated in response to actual events. For example, one study reports that at the time of the Miyagiken-Oki earthquake of June 12, 1978, signs of extensive liquefaction were observed in the area of Ishinomaki, Japan. However, oil tanks which had been constructed on sand stabilized by the compaction pile technique did not incur any damage in spite of liquefaction that developed in the surrounding area. See K. Ishihara, Y. Kawase & M. Nakajima, Liquefaction Characteristics of Sand Deposits at an Oil Tank Site During the 1978 Miyagiken-Oki Earthquake, 20 Soils and Foundations 97 (June 1980) (Japanese Society of Soil Mechanics and Foundation Engineering). A copy of the study is attached.

In conclusion, it is my opinion that the soils under the pile-supported structures at the LACBWR site are safe against liquefaction in the event of an earthquake up to magnitude 5.5 with a peak ground acceleration of 0.12g or less. The consolidated parties have not raised any matters which would lead me to alter my opinion.

I hereby certify that the preceding information is true and correct to the best of my knowledge and belief.

John T. Greeves

John T. Greeves

Subscribed and sworn before me
this 14th day of November, 1980.

Elizabeth Anne Lipton

Notary Public
My Commission expires: July 1, 1982

Attachments:
1. Professional Qualifications
2. Article from Soils and Foundations

John T. Greeves
Professional Qualifications

My name is John T. Greeves. I am responsible for geotechnical engineering evaluations for nuclear facilities. This includes evaluation of the soil mechanics, rock mechanics, earthquake engineering, and foundation engineering aspects to assure that adequate siting and foundation design measures have been taken to prevent adverse operational and safety problems. This includes development of criteria and standards for evaluating the above geotechnical engineering matters as they affect the safety of nuclear power plants, fuel reprocessing plants, fuel storage, waste disposal, and other nuclear facilities.

I am responsible for analyzing, interpreting and evaluating the soil mechanics, rock mechanics, and foundation engineering information submitted to the NRC in support of applications for the construction and operation of nuclear facilities. These evaluation duties are accomplished through the application of standard and state-of-the-art procedures to assess the safety of nuclear facilities.

I am responsible for conducting reviews of the site geotechnical conditions, and the adequacy of foundation construction procedures, including instrumentation proposed for collection of data; discussing the adequacy of these programs with technical officials in applicant's organization and their consultants and providing liaison with the U.S. Army Corps of Engineers offices in matters of soil and rock mechanics, dam design, foundation engineering, and earthquake engineering.

This includes appearances before the Advisory Committee on Reactor Safeguards, the Atomic Safety and Licensing Board Panel, and at public hearings, as required, to present and justify technical analyses and evaluations in geotechnical engineering matters.

I received a Bachelor of Science Degree in Civil Engineering from the University of Maryland in 1968. In addition, I have completed nine credit hours of graduate level studies in Civil Engineering at the University of Maryland. I have also completed numerous short courses in soil mechanics, earthquake engineering and nuclear power plant design.

I have about 12 years of professional experience working in areas related to foundation design, construction and analysis required for both nuclear and conventional electric generating plants. These include nearly seven years with Bechtel Inc., in Gaithersburg, Maryland and over six years with the U.S. Nuclear Regulatory Commission in Bethesda, Maryland.

I am a member of the American Society of Civil Engineers, International Society of Soil Mechanics and Foundation Engineers, the Earthquake Engineering Research Institute, and the Interagency Committee on Seismic Safety in Construction. In addition, I am a registered professional engineer in Virginia and Mississippi.

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Japanese Society of Soil Mechanics and Foundation Engineering

LIQUEFACTION CHARACTERISTICS OF SAND DEPOSITS AT AN OIL TANK SITE DURING THE 1978 MIYAGIKEN-OKI EARTHQUAKE

Kenji Ishihara*, Yasuhiro Kawase**, and Miharu Nakajima***

ABSTRACT

Following the Miyagiken-oki earthquake of June 12, 1978, signs of considerable liquefaction were observed over the reclaimed sand deposit in the area of Ishinomaki fishery port. Three oil storage tanks constructed in thin area but on the deposit compacted survived without any damage notwith-standing the considerable liquefaction that had developed in the surrounding area. After the earthquake, undisturbed sand samples were taken by means of Osterberg piston sampler both from the compacted deposit and from the uncompacted deposit. Laboratory cyclic triaxial shear tests were performed on these specimens to determine the in-situ cyclic strengths of the sands. The cyclic strengths thus determined were incorporated into a simple analysis to determine the potential for liquefaction in these two deposits. The results of the liquefaction analysis were discussed in the light of the construction record and the observed performances of the tanks and deposits during the 1978 earthquake.

Key words: earthquake, liquefaction, sand compaction pile, tank
IGC: C 9/D 7/H 1

INTRODUCTION

Settlements and tilts of oil storage tanks resulting from the liquefaction of sand deposit during earthquakes are of major concern for those who are working in the seismically active region of the world. Destruction of a number of oil tanks caused by the extensive liquefation at the time of the Niigata earthquake of 1964 was probably the first of this kind known to the geotechnical profession. According to a report by Watanabe (1966), nine tanks for strage of oil products 10.7 m in height and 25.2 m in diameter with a capacity of 5 000 kl sustained considerable damage involving a maximum settlement of 50 cm. These oil tanks were constructed on loose sand deposits having blow count values of 5 to 10 in the standard penetration test down to a depth of approximately 8 m. In contrast to this, two tanks having a capacity of 20 000 kl with a dimension of 13.76 m in height and 44.58 m in diameter suffered little damage. The sand deposit underlying these tanks had been compacted by means of vibroflotation technique to a density with a blow count value of 15 or more down to the depth of 7 m except for the surface layer immediately below the bottom of the tank. These experiences

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showed that artificial compaction of loose sand deposit is an effective means of reducing the potential of liquefaction and consequent damage to overlying storage tanks.

Although the effectiveness of stabilizing loose sand deposit by in-situ compaction technique has been proved through experience, little study has been undertaken thus far to clarify the behavior of sand deposit supporting oil tanks in a quantitative manner on the basis of cyclic test data performed on undisturbed samples of sand.

At the time of the Miyagiken-oki earthquake of June 12, 1978, signs of extensive liquefaction were observed in the sand deposit in the area of the Ishinomaki fishery port. However, three oil storage tanks constructed on the dense sand deposit stabilized by the compaction pile technique did not incur any damage in spite of the liquefaction that has developed in the wider area surrounding the oil tank yard. Since construction records of the oil tanks were preserved and readily available for study, detailed investigation of this site appeared to offer a unique opportunity for making a complete case history study on the performance of oil tanks and the underlying soil deposit during earthquakes. Undisturbed sand sampling from this site and testing in the laboratory were thus planned. In the following pages, the description of these investigation and the significance of the results will be presented in relation to the observed behavior of the tanks and the soil deposit during the 1978 earthquake.

MIYAGIKEN-OKI EARTHQUAKE

An earthquake with a magnitude of 7.4 occurred approximately 150 km east of Sendai city off the coast of Miyagi prefecture at 5 : 14 p.m. on June 12, 1978. The focal depth was estimated to have been 30 km. The earthquake shook widespread areas in the northern half of the country as indicated on the map in Fig. 1 in terms of the

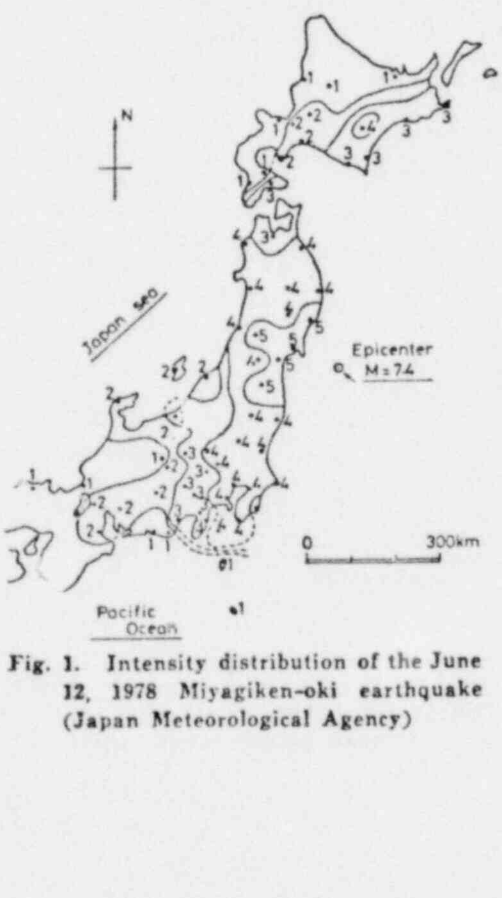


Fig. 1. Intensity distribution of the June 12, 1978 Miyagiken-oki earthquake (Japan Meteorological Agency)

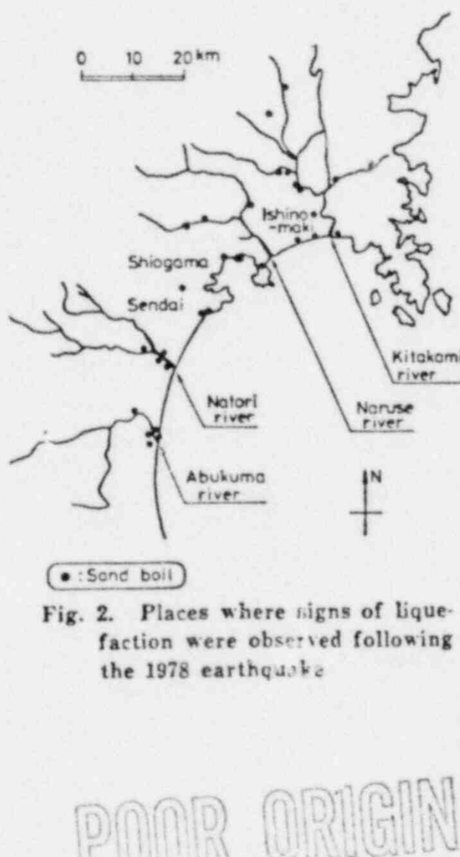


Fig. 2. Places where signs of liquefaction were observed following the 1978 earthquake

Japan Meteorological Agency's intensity scale. Considerable destruction was incurred to buildings, houses and civil engineering facilities in the area shaken with the intensity V. Several landslides were also triggered by the earthquake over the man-made residential sections in the city of Sendai, recently formed by filling valleys.

Surface evidences of liquefaction were numerous in alluvial sand deposits along the middle and lower reaches of rivers and also in filled deposits along the sea coast. Fig. 2 shows the places where sand volcanoes, surface fissurings and other signs of liquefaction were identified as a result of field investigations carried out after the earthquake.

DAMAGE FEATURE AT THE OIL TANK SITE IN ISHINOMAKI FISHERY PORT AREA

Of a number of places where the occurrence of liqufaction was identified as shown in Fig. 2, a site near the fishery port in the city of Ishinomaki was of particular interest because three fuel storage tanks constructed there did not suffer any damage in spite of the extensive liquefaction that had occurred in the sand deposit adjacent to the tank yard. Fig. 3 shows the fishery port area where the fuel tanks are located. The individual spots where sand volcanoes and surface fissurings were observed near the tanks are shown in Fig. 4.

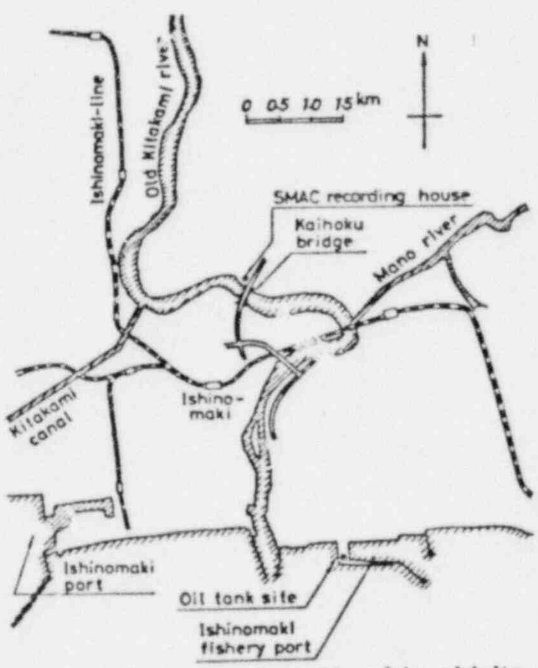


Fig. 3. City of Ishinomaki and its vicinity

An operator of the tanks who was stationed to the workroom witnessed many sand boils show up about 10 minutes after the main shock. The sand spouts reached a height of approximately 1 meter above the ground surface. He also testified that the bodys of the tanks were shaken violently, while the ladders attached on the flank of the tanks collided against the tank and chattered loudly.

Twenty days after the earthquake, accurate measurements were made of the settlements of the tanks along their peripheries as shown in Fig. 5. The result of the measurements made after the earthquake are presented in Fig. 6, together with the records of a similar survey previously performed for the tanks since they had been put into operation in 1975. It is noted in Fig. 6 that the amount of annual settlement prior to the earthquake was on the order of 10 mm, a level which did not affect the satisfactory operation of the tanks. It should also be noted that the amount of settlements for approximately one-year period including the earthquake between 1977 and 1978 was on the same order of magnitude as that recorded for the previous one-year period between 1976 and 1977. This fact would indicate that there was substantially no damage to the main body of the oil tanks due to the earthquake.

STABILIZATION OF FOUNDATION SOILS FOR OIL TANK CONSTRUCTION

The site planned for the construction of the oil tanks consisted of deposit formed by

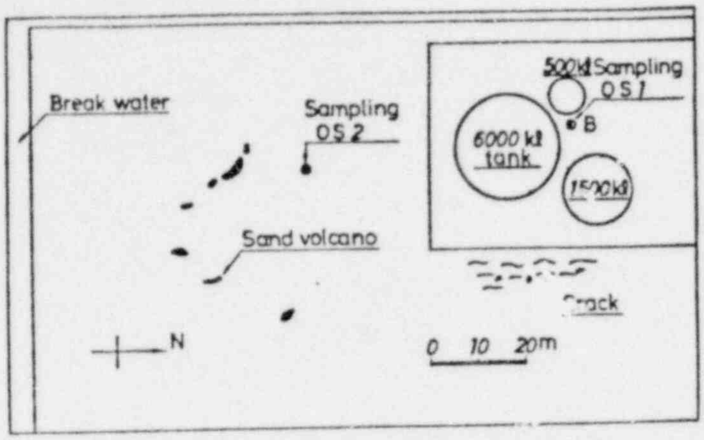


Fig. 4. Ground damage near the tank yard

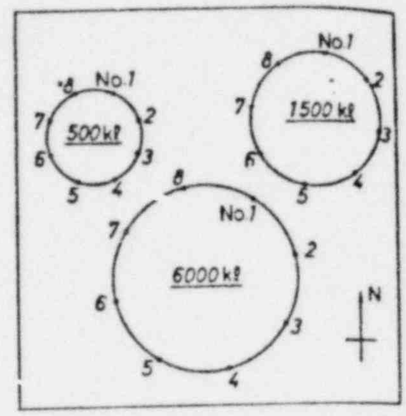


Fig. 5. Points of settlement measurement around the tanks

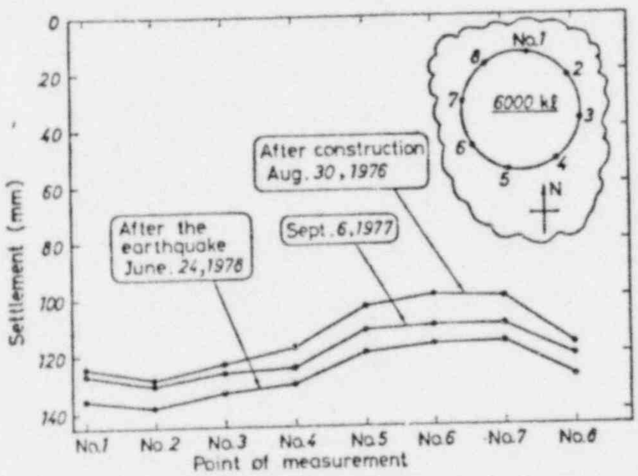


Fig. 6. Settlements of 6 000 kl tank during the period 1976–1978

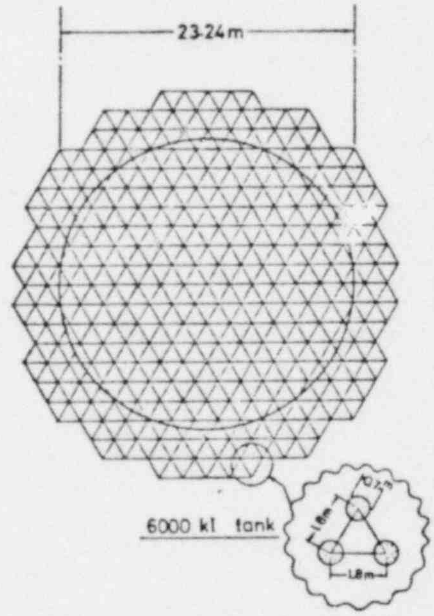


Fig. 7. Installation of compaction piles in plan

fine sand reclaimed from the nearly seabed through the hydraulic method. The sand as it had been deposited was loose, and, therefore, it was stabilized by installing compaction piles to provide a sufficient amount of bearing capacity for the foundation of the oil tanks. The compaction piles were installed at each node of a triangular mesh shown in Fig. 7 with a spacing of 1.8 m. The diameter of the piles was 0.7 m. The plan for the installation of the compaction piles covered the circular area beneath the tank plus the annular belt area having a width of 2.8 m extending out from the periphery of the tank as shown in Fig. 8. The section half enclosed by the three tanks as shown in Fig. 8 was also stabilized with the compaction piles. The compaction was carried out down to a depth of 1.55 m as illustrated in Fig. 9 where a relatively dense sand deposit lay.

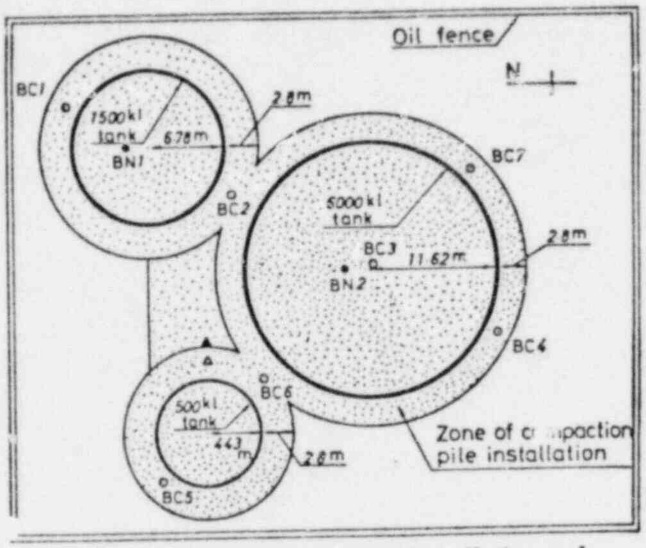


Fig. 8. Zone of compaction pile installation and locations of drilling and sampling

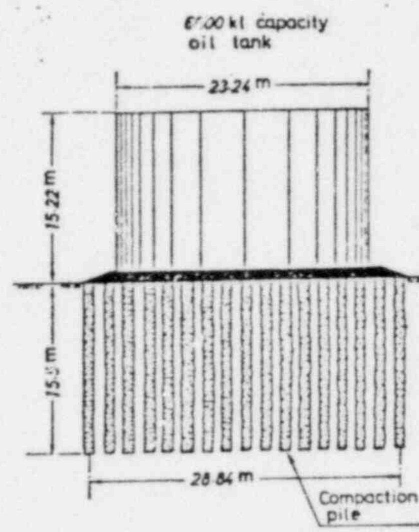


Fig. 9. Extent of compaction pile installation

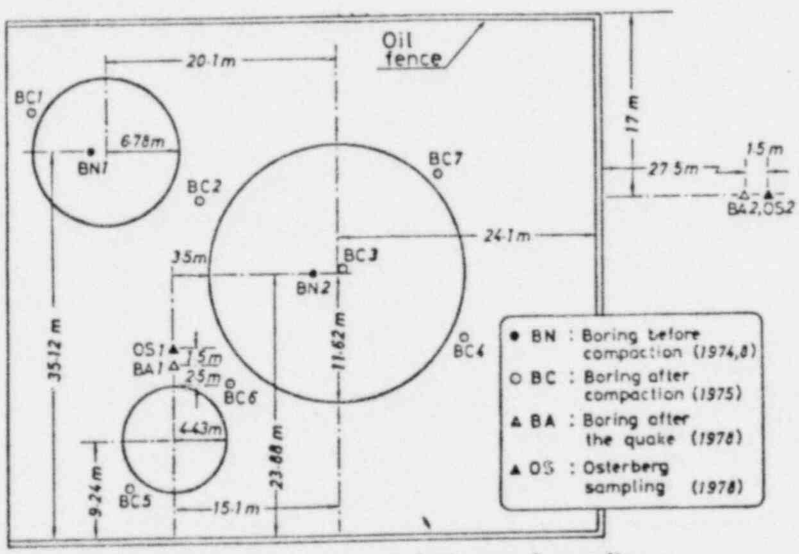


Fig. 10. Locations of drilling and sampling

SUBSURFACE SOIL CONDITIONS

Before starting the construction of the tanks, the subsurface soil profile at the proposed site had been investigated by drilling 2 bore holes. The locations for these borings are indicated by BN 1 and BN 2 in Figs. 8 and 10. The soil profile and blow count values of the standard penetration test at BN 2 are shown in Fig. 11. It may be seen that the reclaimed portion of the sand deposit had been loose to a depth of approximately 12 m having blow count values on the order of 5.

After completion of the stabilization by means of the compaction pile, the standard penetration tests were performed again at seven locations to confirm if the compaction

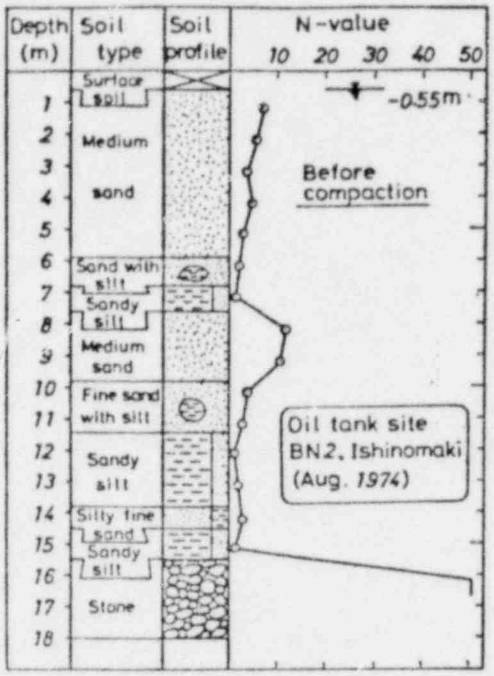


Fig. 11. A soil profile and standard penetration resistance before compaction

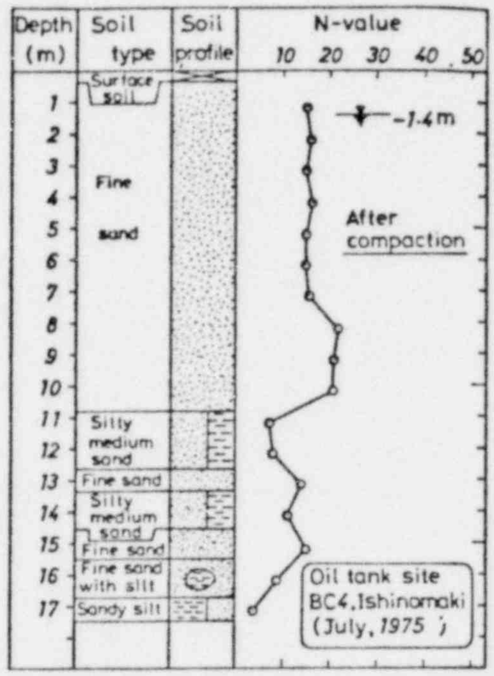


Fig. 12. Standard penetration resistance after compaction

pile installation had sufficiently compacted the sand deposit. The locations of the check-up standard penetration test are indicated by the sign BC in the plan of Fig. 10. One of the results of the tests is presented in Fig. 12. It is apparent that the reclaimed sand layer had been compacted to N-values of approximately 15 down to the depth of compaction pile installation. The sand deposit thus compacted was considered dense enough to provide sufficient resistance to liquefaction.

SAMPLING AFTER THE EARTHQUAKE

In order to determine the cyclic strength of soils in the sand deposit at and near the oil tank site, undisturbed sand samples were obtained from two sites for testing in the laboratory. It was considered of interest to compare the cyclic strength of undisturbed samples from one site where liquefaction is known to have occurred against the cyclic strength of samples from another site where liquefaction is known not to have occurred during the June 12, 1978 earthquake. Consequently, undisturbed sampling was performed first near the 500 kl tank in the tank premises as shown in Fig. 10. Needless to say, the sand deposit at this place had been compacted by means of compaction piles. A standard penetration test was also conducted next to the sampling hole as indicated in the plan view of Fig. 10. The result of the standard penetration test is presented in Fig. 13. The blow count values as well as the soil profile in general are nearly identical to those shown in Fig. 12 where the soil conditions investigated prior to the earthquake is presented. Another sampling site was chosen outside the tank premises where surface signs of liquefaction were observed in the form of ejection of sand and water following the 1978 earthquake. The sampling site relative to the positions of sand volcanoes is shown in Fig. 4, and the exact sampling location is indicated in Fig. 10. Result of the standard penetration test performed in the immediate vicinity of the sampling hole is

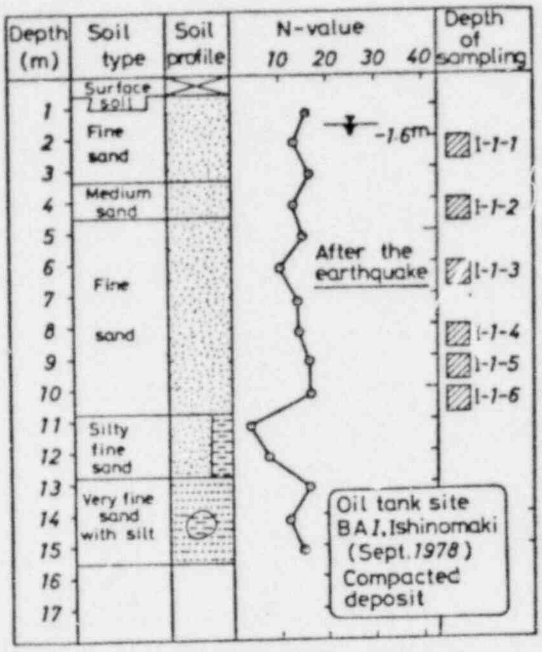


Fig. 13. Standard penetration resistance and depths of Osterberg sampling at the compacted site after the earthquake

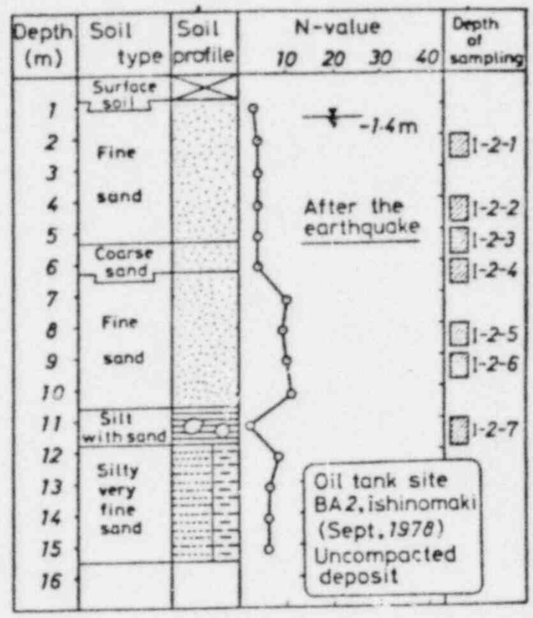


Fig. 14. Standard penetration resistance and depths of Osterberg sampling at the uncompacted site after the earthquake

shown in Fig. 14. The soil profile and the blow count values were nearly the same as those obtained in the investigation performed before the earthquake (see Fig. 11).

SAMPLING BY MEANS OF PISTON SAMPLER

Undisturbed sand samples were obtained by means of a piston sampler originally developed by Osterberg (1952). This sampler consists of a steel tube 76.3 mm in diameter and 90 cm long, driven by mud water pressure at the bottom of a drilled hole. The sample was kept in a vertical position for about 2 days to drain out excess water existing in the voids of the sand. It was frozen in the field and stored in a ice cream freezer until tested to preserve the insitu density and fabric of the sample.

Detailed procedures for sampling and handling are described in a paper by Ishihara, Silver and Kitagawa (1979).

CYCLIC TRIAXIAL TEST

The frozen sample encased in the sampling tube was severed into 15 cm long lengths using a power hacksaw. The tubes were then cut lengthwise using a band saw to split the tube for removing the intact sand specimen. The specimen was then enclosed in a rubber membrane and placed in a triaxial test cell. After finishing necessary processings such as thawing and saturation, the triaxial specimen was consolidated under an effective confining pressure of 100 kN/m² and subjected to the cyclic axial stress until the specimen deformed to a peak-to-peak axial strain of 10%. During cyclic loading the chamber pressure was kept constant and the axial load, axial deformation and change in pore water pressure were monitored and recorded with time. Detailed description of the test procedures is given in the paper by Ishihara, Silver and Kitagawa (1978).

RESULTS OF LABORATORY TESTS

Test Results for the Samples from the Compacted Deposit

The grain size distribution curves of the Osterberg samples from each depth are presented in Fig. 15. It is noteworthy that all the sands from each depth are very fine sand having a mean particle size, D_{50}, of approximately 0.15 mm. The results of cyclic triaxial tests are shown in Fig. 16. The cyclic stress ratio, $\sigma_{dl}/(2\sigma_0')$, required to induce initial liquefaction, 5% and 10% double amplitude axial strains is plotted in these figures versus the number of cycles, where σ_{dl} denotes the amplitude of cyclic axial stress and σ_0' is the effective confining stress. Each of these figures plots the cyclic strength of the Osterberg specimens from a given depth. It may be seen from these figures that the shape of the cyclic strength curve is rather flat and for 20 cycles failure occurred at a cyclic stress ratio of approximately 0.22 to 0.28.

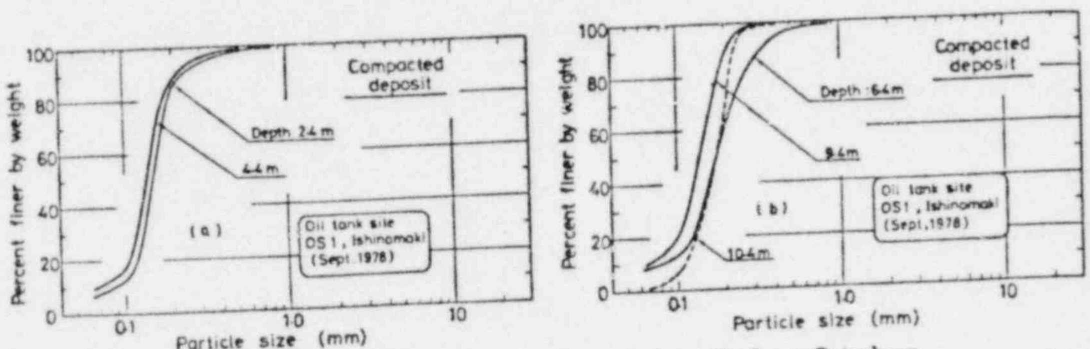
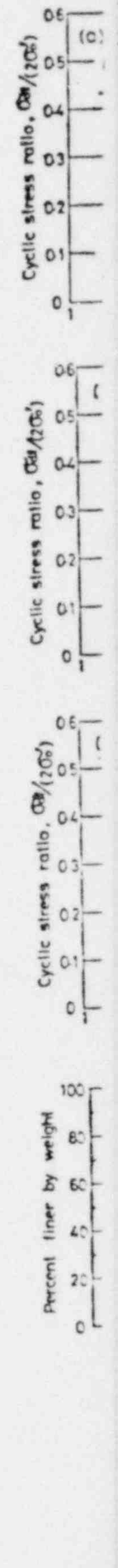


Fig. 15. Grain size distribution curve of the soils from Osterberg samples at the compacted site

As noted above, the average grain size of the sand at this site was small compared to most of the clean sands that have been tested so far for the studies of liquefaction. Because of this, the void ratio values at the consolidated state of the undisturbed test specimens were extremely large, ranging mostly from 1.0 to 1.4. The maximum void ratios were in the range of 1.3 and 1.65, corresponding to the large values of void ratio, and the minimum void ratios ranged between 0.85 and 1.0. Inasmuch as the use of relative density is open to question for such unusually high values of void ratio, relative density values are not presented in Fig. 16. Instead, average values of the void ratio itself and their ranges of variation are indicated in the figures.

Test Results of the Samples from the Uncompacted Deposit

Fig. 17 shows the results of grain size analysis performed on the specimens from the uncompacted deposit adjacent to the oil tank yard. It may be seen that the soil at this site was composed of very fine sand with a mean particle size, D_{50}, of approximately 0.15 mm for all sands at all depths. At depth 11.4 m the soil was clay, having a void ratio of approximately 1.8, specific gravity of 2.709, liquid limit of 53%, and plastic limit of 39%. The grain size characteristics of the sand at this site was essentially the same as that of the sand at the compacted deposit within the premises of the tank yard except for the clay existing at depth 11.4 m. Fig. 18 shows the results of cyclic triaxial shear tests on the undisturbed Osterberg samples obtained from the uncompacted deposit. It may be seen in the figures that the cyclic stress ratio required to cause 5% double amplitude axial strain in the test specimens was approximately 0.22 except for the larger cyclic stress ratio values measured for the clay specimen from the depth of



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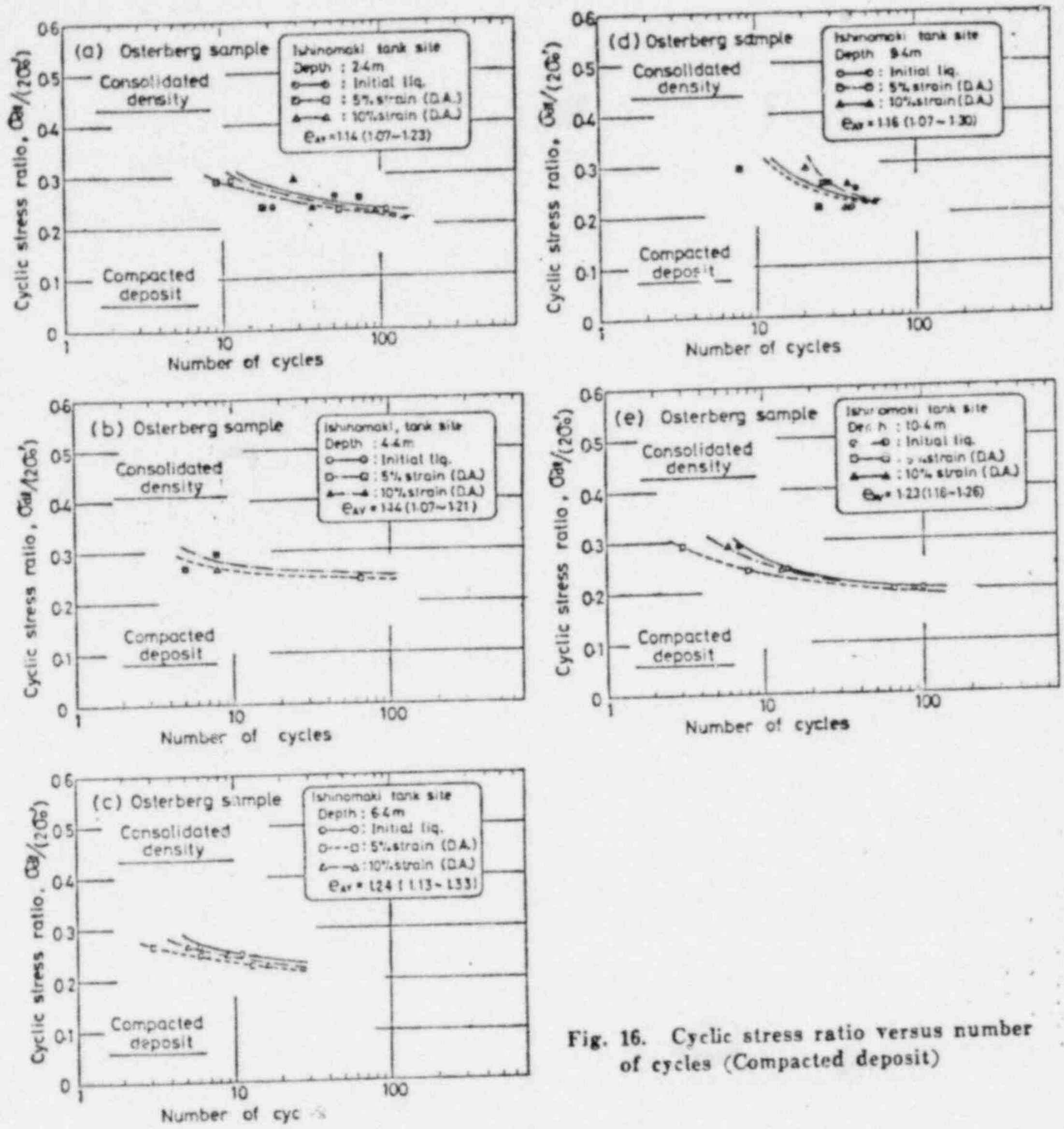

Fig. 16. Cyclic stress ratio versus number of cycles (Compacted deposit)

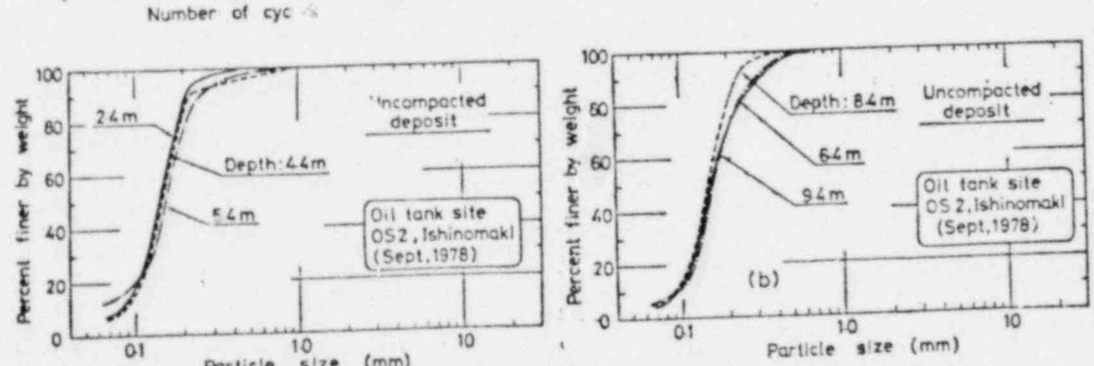

Fig. 17. Grain size distribution curves of the soils from Osterberg samples at the uncompacted site

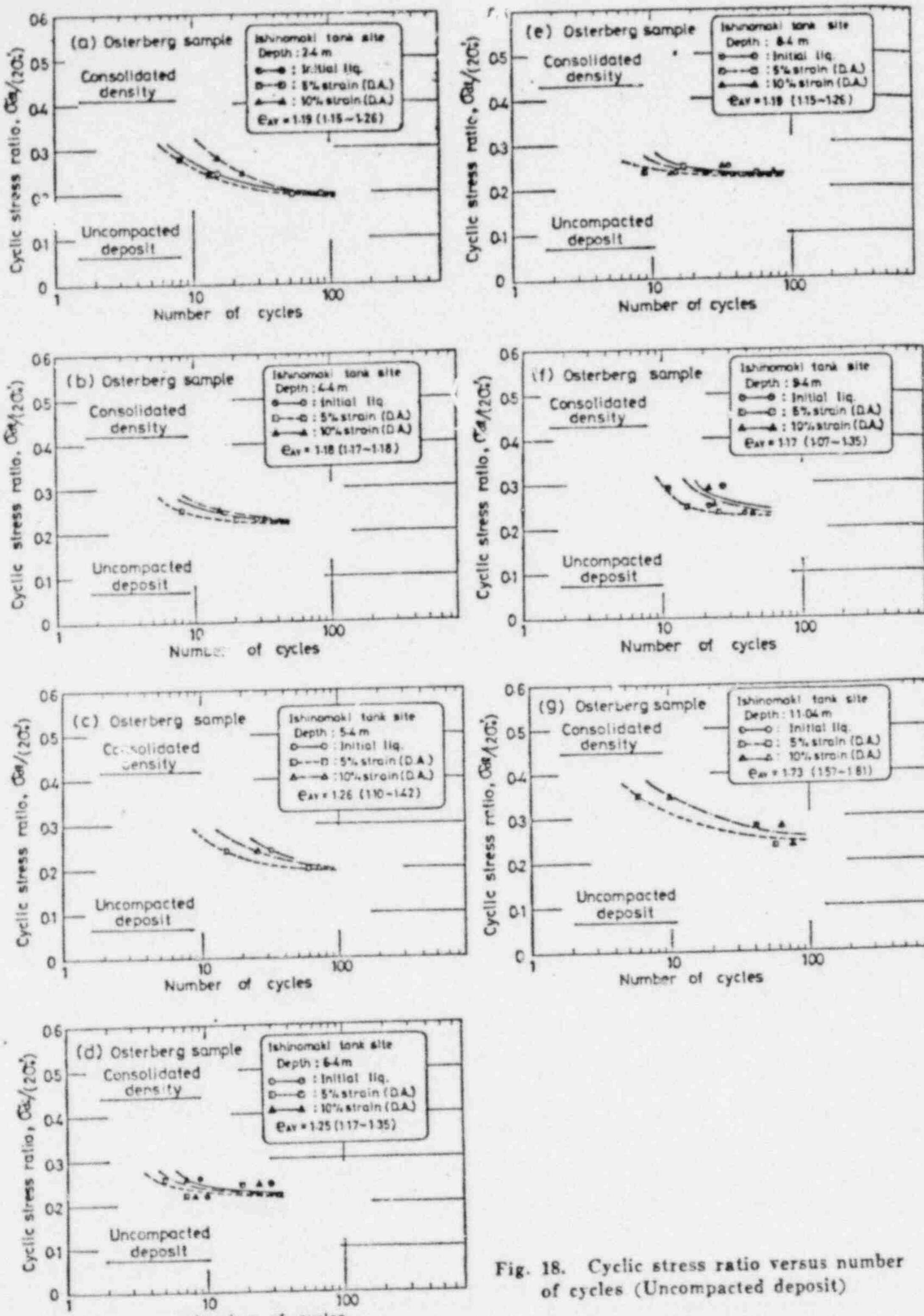

Fig. 18. Cyclic stress ratio versus number of cycles (Uncompacted deposit)

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COMPARISON OF CYCLIC STRENGTHS AT THE COMPACTED AND UNCOMPACTED DEPOSITS

In order to compare the cyclic soil strength at the compacted site and at the uncompacted site, the cyclic stress ratios causing 5% double amplitude strain in the undisturbed test specimen in the course of 20 cycles of uniform loading were read off from the test data shown in Figs. 16 and 18, and replotted in Fig. 19 versus the depth of the deposit. The figure shows the cyclic strength in terms of cyclic stress ratio as being almost constant for the uncompacted deposit down to a depth of approximately 8 m, whereupon the cyclic strength increased gradually. As for the compacted deposit, the cyclic strength was about 0.26 except at the two depths of 6.4 m and 10.4 m. The figure also shows the sand at the compacted deposit as exhibiting greater cyclic strength than the sand at the uncompacted deposit. In order to provide other aspect of comparison, average values of void ratio were computed from individual data at each depth of the two deposits. The average void ratios are plotted versus the depth in Fig. 20, where the average void ratio at each depth was shown as being generally higher for the uncompacted deposit than it was for the compacted deposit, except for the clay layer existing at the depth of 11.4 m.

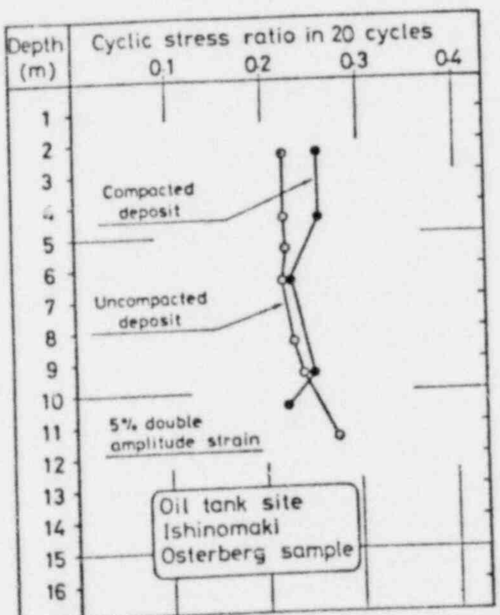

Fig. 19. Comparison of cyclic strengths between the compacted and uncompacted deposits

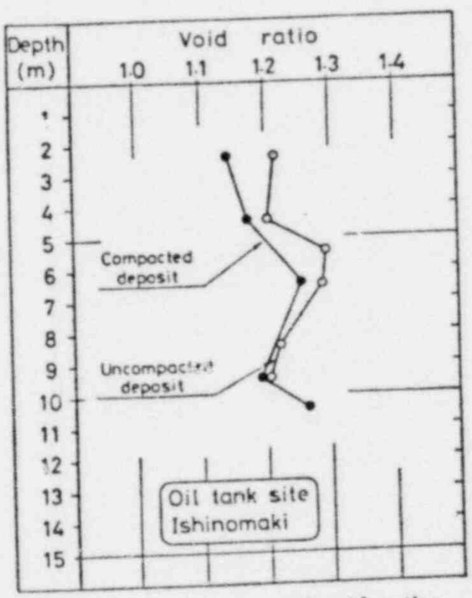

Fig. 20. Comparison of void ratios between the compacted and uncompacted deposits

Comparison of cyclic strength with reference to the N-values shown in Figs. 13 and 14 shows that, for the sands at depths 2.4 m and 4.4 m, the rate of increase in cyclic strength of the compacted deposit over that of the uncompacted deposit is nearly equal with the rate at which the blow count values increase. At the depths of 6.4 m and 9.4 m, the difference in cyclic strength between the two sites was unproportionately narrow in view of the large difference observed in the resistance of the standard penetration test.

SIMPLE ANALYSIS OF LIQUEFACTION

The analysis of liquefaction during earthquakes requires a knowledge about the intensity of shaking most preferably in terms of the time history of acceleration on the ground surface at a given site in question. The acceleration records obtained during the 1978 earthquake at a place nearest to the Ishinomaki port were those obtained on the rock outcrop at Kaihoku bridge site (Fig. 3), located approximately 5 km north of the oil tank site (Iwasaki et al., 1978). The maximum horizontal acceleration recorded was 289 gals in N-S direction and 200 gals in E-W direction. An assessment of the maximum ground acceleration on the soft soil deposit just at the oil tank site may be made based on the acceleration record obtained on the nearby rock outcrop. Pending further detailed analysis, however, it may well be assumed that the maximum ground acceleration at the oil tank site might have taken a value ranging between 0.16 g to 0.20 g. The fact that the maximum acceleration was estimated to be lower on the soil deposit than the recorded accelerations on the rock outcrop may be justified by allowing for the effects of nonlinearity and higher damping and also the effect of stiffness degradation due to pore water pressure build-up exhibited by soft soils. With these facts in mind, it will provisionally be postulated in the following simple analysis that the maximum ground acceleration at the oil tank site had been approximately 0.185 g. This value would appear reasonable as a rough estimate considering the magnitude of the maximum horizontal accelerations as listed in Table 1 that have been obtained thus far on soft soil deposits where liquefaction is known to have occurred.

Table 1. Maximum horizontal ground accelerations ever recorded where liquefaction occurred

Earthquake	Place of recording	NS-comp.	EW-comp.
Niigata (1964)	Kawagishicho, Niigata	0.162 g	0.158 g
Tokachioki (1968)	Aomori harbor	0.217 g	0.184 g

The simple analytical procedure developed by Ishihara (1977) was used to make a crude analysis. The maximum stress ratio, τ_{max}/σ_v', at each depth induced by earthquake loading was computed by the following formulas,

$$\frac{\tau_{max}}{\sigma_v'}=\frac{a_{max}}{g}\cdot r_d\cdot h\left(\frac{Z}{H}\right) \tag{1}$$

where

$$\sigma_v'=\gamma H+\gamma'(Z-H) \tag{2}$$

$$h\left(\frac{Z}{H}\right)=1+\frac{\dfrac{\gamma_w}{\gamma}\cdot\dfrac{Z-H}{H}}{1+\dfrac{\gamma'}{\gamma}\cdot\dfrac{Z-H}{H}} \tag{3}$$

$$r_d=1-0.015Z \tag{4}$$

In the above formula, a_{max} is the maximum horizontal ground acceleration and g is the gravity acceleration. γ is unit weight of soil above the ground water table, γ' submerged unit weight of soil below the ground water table and γ_w unit weight of water. The depth of the ground water table is denoted by H and Z represents the depth of deposit in question. The vertical effective stress is denoted by σ_v'.

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The cyclic strength to be compared against the induced maximum stress ratio, τ_{max}/σ_v', was determined by the following formula, based on the laboratory test data which were expressed in terms of the cyclic stress ratio, $\sigma_{dl}/(2\sigma_0')$, required to cause 5% double amplitude strain in 20 cycles,

$$\frac{\tau_{max.l}}{\sigma_v'} = \frac{0.9}{0.55} \cdot \frac{1+2K_0}{3}\left(\frac{\sigma_{dl}}{2\sigma_0'}\right)_{N=20} \qquad (5)$$

where $\tau_{max.l}/\sigma_v'$ indicates the maximum stress ratio required to produce failure in the soil as represented by the development of 5% double amplitude strain under uniform loading condition. K_0 is the coefficient of earth pressure at rest. The value, 0.55, on the right side of Eq. (5) represents a factor to take into account the effect of irregular nature of time history changes of shear stress during an earthquake. A factor, 0.9, was incorporated on the right side of Eq. (5) to allow for the effect of changes in direction of shear stress application in the horizontal plane during an earthquake.

The maximum stress ratio given by Eq. (1) was compared against the corresponding maximum stress ratio required to cause failure in soils as given by Eq. (5). The result of comparison is expressed in terms of factor of safety against failure, F_l, which is defined as

$$F_l = \frac{\tau_{max.l}/\sigma_v'}{\tau_{max}/\sigma_v'} \qquad (6)$$

The simple analysis as above was applied for both the uncompacted and compacted deposits. For the uncompacted deposit the assumption that no surcharge load was present over the ground surface was made, and also, that the horizontal surface extended indefinitely. The analysis for the compacted deposit was carried out for the soil layers just beneath the center of the tank by assuming that the surcharge due to the weight of oil existed all over the ground surface so that one-dimensional stress conditions

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2, CD w/ q w/at
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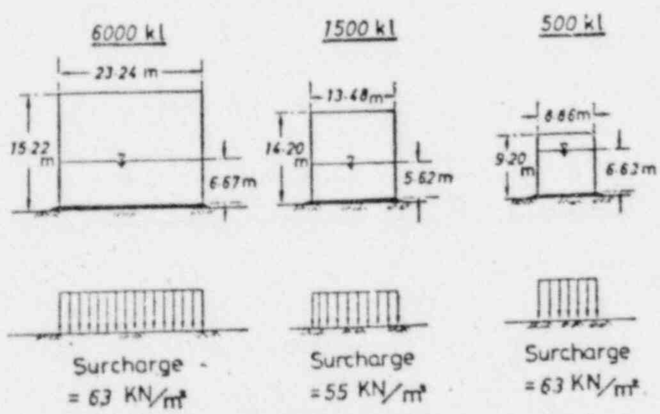

Fig. 21. Oil storage at the time of the 1978 earthquake

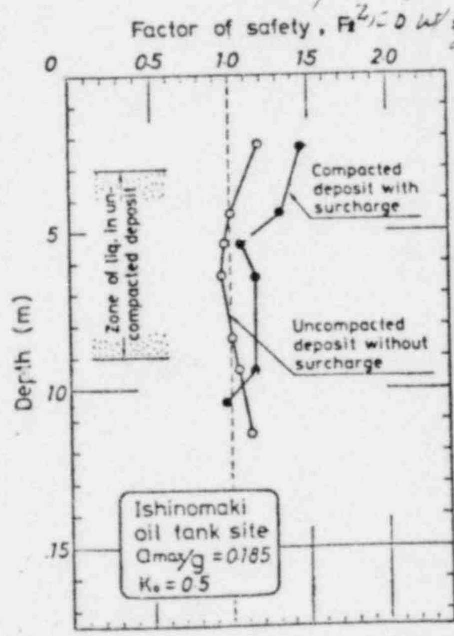

Fig. 22. Factor of safety against failure in the compacted and uncompacted deposits

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prevailed. This assumption neglects possible effects of static shear stresses that existed near the edge of the tank. However, as a first approximation, the above assumption was considered acceptable.

According to the operational records each tank contained oil to a height of 5.6 m to 6.7 m at the time of the 1978 earthquake as shown in Fig. 21. Assuming that the unit weight of oil is 8 kN/m^3, the overburden pressure is estimated to have been approximately 60 kN/m^2 as illustrated in Fig. 21. The value of effective overburden pressure, σ_v', through the depth beneath the center of the tank was calculated by adding the surcharge pressure of 60 kN/m^2 to the effective stress due to the weight of solis from the ground surface down to the depth in question.

It was further assumed that the presence of oil did not affect the shaking intensity on the ground surface during the earthquake. The ground surface at the bottom of the tank was, therefore, assumed to have moved with the same maximum acceleration as that at the nearby free surface of the uncompacted deposit. This assumption neglected possible effects of dynamic interaction that must have taken place between the tank and the ground during the earthquake. The above assumption also neglected the effect of the compaction pile installation which certainly increased the stiffness of the soils and accordingly the maximum acceleration on the ground surface. In spite of these important effects being disregarded, the approximate analysis based on the assumption as above was considered to yield some meaningful comparison in the behavior during the earthquake between the uncompacted and compacted deposits.

The factor of safety as defined by Eq. (6) was calculated for both the compacted deposit with surcharge and for the uncompacted deposit without surcharge on the assumption that the ground surface had been shaken equally with the maximum acceleration of 0.185 g. The result of the analysis is shown in Fig. 22. The factor of safety against failure was seen as being below or close to unity for the uncompacted deposit through the depth of approximately 3 m to 9 m, indicating that the liquefaction type failure must have occurred within this depth during the 1978 earthquake. On the other hand, the factor of safety for the compacted deposit is above unity throughout the depth except at the depth of 10.4 m. The distribution of the factor of safety as above may indicate that the liquefaction type failure might have taken place in a thin layer around the depth of 10 m. However, it would appear that the occurrence of the liquefaction, if any, must have extended only slightly, and was never extensive enough to exert any harmful influence on the behavior of the oil tanks resting on this compacted deposit.

CONCLUSIONS

In order to clarify the non-occurrence of liquefaction in a compacted sand deposit at an oil tank yard and the occurrence of liquefaction in an uncompacted sand deposit adjacent to the yard at the time of the 1978 Miyagiken-oki earthquake, undisturbed sand sampling was carried out after the earthquake by means of an Osterberg piston sampler. The undisturbed samples were tested in the laboratory using a cyclic triaxial test apparatus to determine the cyclic strength of the sands in the compacted deposit and also in the uncompacted deposit. The test results showed that the cyclic stress ratio causing 5% double amplitude axial strain was 2% to 18% greater for the specimens from the compacted deposit than for the specimens from the uncompacted deposit.

On the basis of the acceleration records obtained on the rock outcrop approximately 5 [illegible] from the oil tank site, the maximum horizontal acceleration on the ground [illegible]face was estimated roughly to have been on the order of 0.185 g at the site of the oil tank. At the time of the earthquake the oil tank contained some oil which exerted a surcharge pressure of about 60 kN/m² on the compacted deposit beneath the tank.

Taking into account the effect of the surcharge and assuming the maximum ground surface acceleration of 0.185 g, a simple analysis of liquefaction was made for the sand in the compacted deposit based on the cyclic strength obtained for the undisturbed specimens. The result of the analysis showed that the liquefaction must have been only minor around the depth of 6 m, but that the soil at a shallower depth did not liquefy, producing no harmful influence on the oil tank.

A similar liquefaction analysis was also made for the uncompacted sand deposit without surcharge near the tank yard by assuming the maximum acceleration of 0.185 g, on the basis of the cyclic strength obtained for the undisturbed specimen. The result of the analysis indicated that the observed surface signs of liquefaction during the earthquake were in coincidence with the low compacted factors of safety against failure which were below or close to unity through the depth between 3 m and 9 m.

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(Received July 9, 1979)

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY & LICENSING BOARD

In the Matter of	)	
	)	
DAIRYLAND POWER COOPERATIVE	)	Docket 50-409-SC
(La Crosse Boiling Water Reactor)	)	(Order to Show Cause)

CERTIFICATE OF SERVICE

I hereby certify that copies of the NRC STAFF'S MOTION FOR SUMMARY DISPOSITION in the above-captioned proceeding have been served on the following by deposit in the United States mail, first class, or as indicated by an asterisk, through deposit in the Nuclear Regulatory Commission's internal mail system, this 14th day of November, 1980.

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