Journal of Coastal Research	SI	5	127-137	Fort Lauderdale, Florida	Summer 1989

Some Experiments on Wave-induced Liquefaction of Soft Cohesive Soils

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ABSTRACT

LINDENBERG, J.; van RIJN, L.C., WINTERWERP, J.C., 1989. Some Experiments on wave-induced liquefaction of soft cohesive soils. *Journal of Coastal Research*, Special Issue No. 5. Fort Lauderdale Florida). ISSN 0749-0208.

Experiments in a wave flume with a soft consolidated bed of kaolinite (China clay) showed that under specific conditions liquefaction of the bed can occur, resulting in a high-density fluid mud layer. From a soil mechanical analysis it can be concluded that this liquefaction process is similar to that observed in sand beds. Hence, the wave-induced excess pressures in a cohesive bed can be computed with methods well known from classical soil mechanics. The material properties of soft cohesive beds, however, cannot be determined with cyclical triaxial tests. Therefore an alternate procedure was developed, through which the required stresses are obtained by shaking a bed sample. In this way it was possible to obtain a reasonable description of the behavior observed in the wave flume. Subsequent tests on samples of a natural soil suggest that gas content and/or organic material affect the sensitivity to liquefaction significantly.

ADDITIONAL INDEX WORDS: *Cohesive sediments, liquefaction, wave action.*

INTRODUCTION

The shallow lakes, estuaries and coastal zones in The Netherlands contain large amounts of cohesive sediments. These sediments originate from the many rivers that flow through the country, or from the sea. In stagnant zones sedimentation occurs and the resulting shoaling may hamper navigation. Furthermore, some of these sediments are heavily polluted causing a severe problem for the quality of the waters in The Netherlands. The transport of these sediments is strongly affected by waves, as waves can cause rapid erosion of the deposits. Therefore Rijkswaterstaat (Ministry of Transport and Public Works, The Netherlands) initiated an extensive program to study the behavior of cohesive sediments in general, and the role of waves in particular.

From a literature survey it was concluded that waves are capable of generating a high-density mud layer close to the bed. Such a layer will behave as a density current and can be transported by, for instance, the tide or by wave-induced drift velocities. However, qualitative information on these processes is extremely scarce. Therefore, experiments were performed to study the behavior of a bed of kaolinite as a function of wave characteristics in a wave flume. A detailed description of this research is given by VAN RIJN (1985) and by VAN RIJN and LOUISSE (1987).

It is shown that under certain conditions liquefaction of the bed can occur, resulting in a high-density fluid mud layer. In order to extrapolate the results to more natural conditions (for natural sediments) this liquefaction process and the influence of the sediment properties should be understood better.

A first step is to look for analogies with the behavior of non-cohesive sediments, *e.g.* sand. The liquefaction of sand beds is well understood, and

can be modeled reasonably well. In this paper attention is focused on the onset of the liquefaction process, which is characterized by an increase of water pressure in the pores and a decrease of effective stresses in the soil. A procedure is developed to determine the response of pore pressure and effective stresses within the cohesive soil, and the results suggest that the processes involved in the liquefaction of soft cohesive sediments are similar to those of sand.

GENERATION OF FLUID MUD DUE TO WAVE ACTION

Properties of Kaolinite and Sand

Commercially available kaolinite (China clay) and fine sand were used as bed material in the wave tests. The chemical composition of the kaolinite, as given by the supplier was: SiO_2 49.5%, Al_2O_3 35.4%, H_2O 11.4%, K_2O 1.6%, Na_2O 1.1%, Fe_2O_3 1.0%. The mineralogical composition was: Kaolinite 80%, Illite 15%, Quartz 4%, and Smectite 1%.

Particle size analysis was carried out with a sedimentation method and a laser diffraction method. The sedimentation method is based on the sedimentation rate of particles in an initially uniform suspension in distilled water at 20°C. The conversion to particle size is done using Stokes' law. This method yielded a particle diameter of D_{50} = 4 μm, and D_{90} = 15 μm. The laser method uses the principle of Fraunhofer diffraction. The incident laser light is diffracted by the illuminated particles in suspension to give a stationary diffraction pattern. Using the measured diffraction pattern a computer analysis is performed to find the size distribution that gives the closest fitting diffraction pattern. This method yielded D_{50} = 6 μm, and D_{90} = 20 μm. Particles larger than 50 μm were not detected with either method.

Experimental Set-up and Procedure

The experiments were performed in a wave flume at Delft Hydraulics with a length of 17 m, a width of 0.3 m, and a depth of 0.5 m. Figure 1 shows the set-up. Regular waves were generated by a simple wave paddle. At the end of the flume a wave absorbing structure was installed to reduce wave reflections. Near the wave generator and the wave absorber a false floor (height = 0.05 m, length = 3.5 m) with a smooth wooden surface was mounted. The remaining deeper section of the flume (length = 10 m) was used as the test section where the bed was deposited from an initially uniform suspension. Before filling the test section with a suspension of kaolinite, partition walls separating the test section from the rest of the flume were installed. The kaolinite bed was allowed to consolidate for about three days, after which a bed layer of about 0.05 m thickness was obtained. Then the partition walls were removed and the flume was filled slowly with saline water (salinity = 31 ppt, temperature = 10°C). In all experiments the water depth was about 0.25 m. Wave heights were measured with a wave height gage which consisted of a small conductivity probe that could follow the oscillating water surface.

The instantaneous horizontal velocities were measured during preliminary tests with clear water with a Laser-Doppler Anemometer. Time-averaged values and root-mean-square values were determined using a time-domain analyzer. Based on these measurements, a relationship between the wave height and the rms-velocity near the bed was derived (VAN RIJN, 1985), as shown in Figure 2. The effect of the suspended sediment on this relationship (based on measurements in clear water) could not be determined.

Three methods were used to determine the sediment concentrations in the test section of the flume: a siphon method, an optical method, and a pipette method. The optical method consisted of the use of a standard, commercially available turbidity meter based on the principle of light extinction. The samples were first diluted to fall in the calibration range (0 to 3000 mgl^{-1}) of the turbidity meter.

Each test consisted of two parts. Phase 1 was initiated by starting the wave paddle to erode the bed. The wave height was kept constant for a period of about 3 hours and then raised to a higher value. Next, the wave paddle was stopped and the sediment was allowed to consolidate again for a period of about 20 to 40 hours. Then Phase 2 of the experiments was started by switching on the wave paddle again at the same conditions.

Various types of sediment mixtures were tested. In four experiments the bed consisted of pure (100%) kaolinite at two different densities. In two other experiments the bed consisted of a mixture of fine sand (D_{50} = 100 μm) and kaolinite. In the last experiment the bed consisted of pure sand

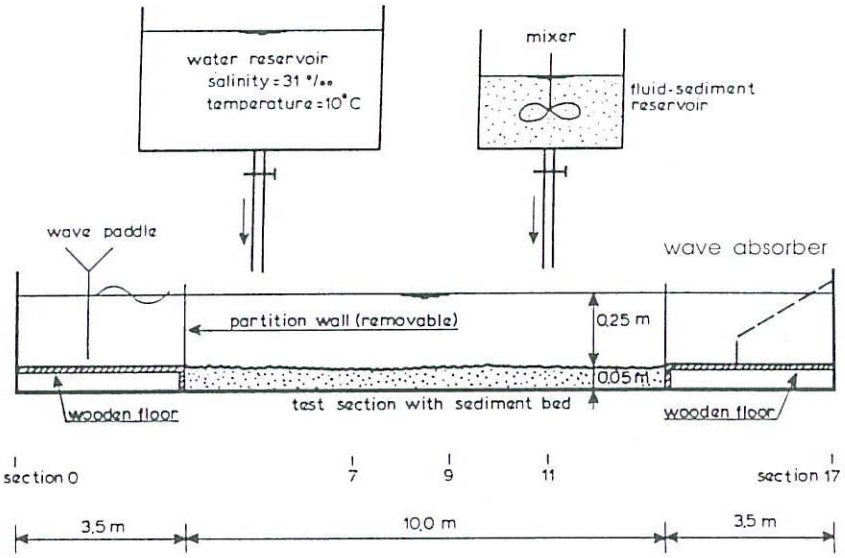

Figure 1: Wave flume and experimental set-up.

(D_{50} = 100 μm). The experimental conditions are given in Table 1.

General Observations

The experiments provided qualitative and quantitative results on the interaction between the hydraulic parameters and sediment parameters (mud layer thickness and density). Basically, the near-bed processes can be summarized as follows:

(1) Initiation of motion beyond a critical orbital velocity.

(2) Generation of a low-density suspension layer at somewhat higher velocities.

(3) Onset of liquefaction of the top layer for increasing velocities.

(4) Generation of a high-density fluid mud layer close to the bed at large velocities.

Since attention is focused on the liquefaction process, the experimental results related to the thickness of the fluid mud layer are considered only. The results of tests T3-4 (100% kaolinite) and T5-7 (75% kaolinite, 25% sand) are shown in the Figures 3 and 4. The thickness of the fluid mud layer could be established by eye, as this layer could be well distinguished from the rigid bed beneath and the dilute suspension above it. The measurements are estimated to be accurate to approximately 0.005 m. Table 1 summarizes the most important characteristics of test series T3, T4 and T5. The onset of liquefaction was not observed in tests T3-3, T4-4 and T5-4.

GENERATION OF FLUID MUD BY LIQUEFACTION

Liquefaction of Cohesive Sediment Beds

After completion of the wave experiments, Delft Geotechnics was asked to investigate whether the internal mechanism leading to the onset and increase of the high concentration suspension layer could be related to the process of liquefaction of sand due to cyclic loading. An important feature of liquefaction of sand is the preceding generation of excess pore water pressure as a result of the tendency of the grain skeleton to decrease in volume under the influence of shear stresses. However, the occurrence and depth variation of liquefaction could not be proven directly during the wave experiments, because the pore pressure distribution in the kaolinite bed was not measured. Therefore it was decided to carry out an additional study aimed mainly at the following items:

(1) To investigate whether the generation of excess pore pressure and the initiation of liquefaction in a kaolinite bed is possible.

(2) To determine the relevant parameters *e.g.*

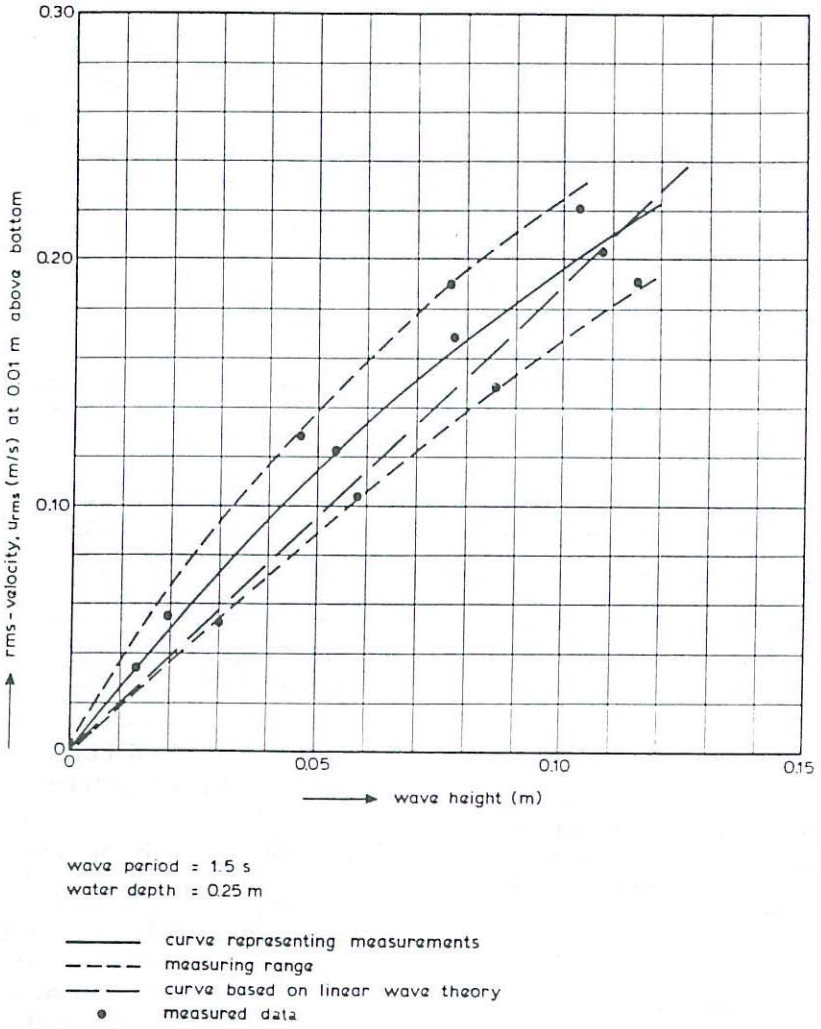

Figure 2: RMS-value of orbital velocity as a function of wave height.

mud characteristics, wave conditions, *etc.*

(3) To develop a quantitative procedure with which the measured and observed phenomena in the wave flume tests (Table 1) can be explained.

The procedure under 3 had two objectives. The first was to give a provisional methodology for more practical purposes. The second objective was to define processes and dominant quantities in the mud which have to be incorporated in a subsequent experimental program.

The explanation of liquefaction in a seabed of sand due to wave action is supported strongly by the results of laboratory tests (*e.g.* triaxial tests). In these tests periodic stresses are imposed in a controlled way, resulting in a gradual increase of internal pore pressure and a corresponding decrease in effective stresses. When the imposed stresses are sufficiently high, the effective stresses may vanish entirely, and the sample will liquefy.

Because of the very low consistency of a kaolinite bed, the use of triaxial tests is impossible. Therefore, a special type of shaking test was developed

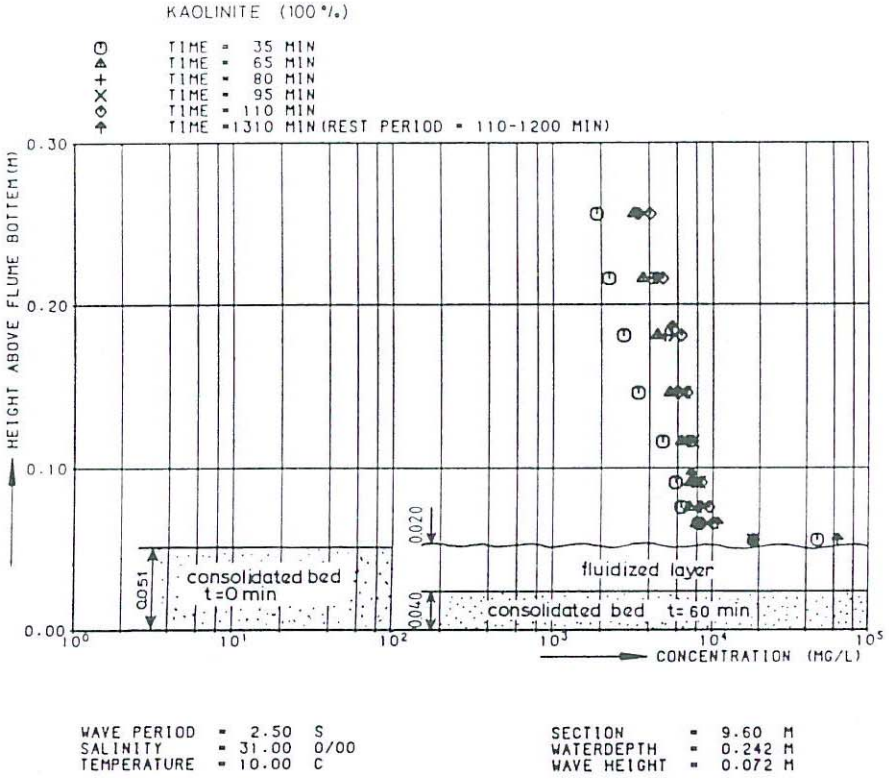

Figure 3: Measured concentration profiles in test T3-4.

Table 1. Main characteristics and results of test series T3, T4, and T5.

Test	Bed Material	Mean Concentration Before	After Consolidation	Layer Thickness After Consolidation	Water Depth	Waves Height	Period	Test Duration
		[kgm^{-3}]		[m]	[m]	[m]	[s]	[min]
T3-3	Kaolinite	500	640	0.049	0.25	0.048	2.5	180
T3-4	Kaolinite	500	640	0.049	0.25	0.072	2.5	60
T4-4	Kaolinite	500	640	0.048	0.25	0.047	1.5	240
T4-7	Kaolinite	500	640	0.048	0.25	0.070	1.5	150
T5-4	75% Kaol. 25% Sand	600	790	0.048	0.25	0.048	1.5	210
T5-7	75% Kaol. 25% Sand	600	790	0.048	0.25	0.070	1.5	180

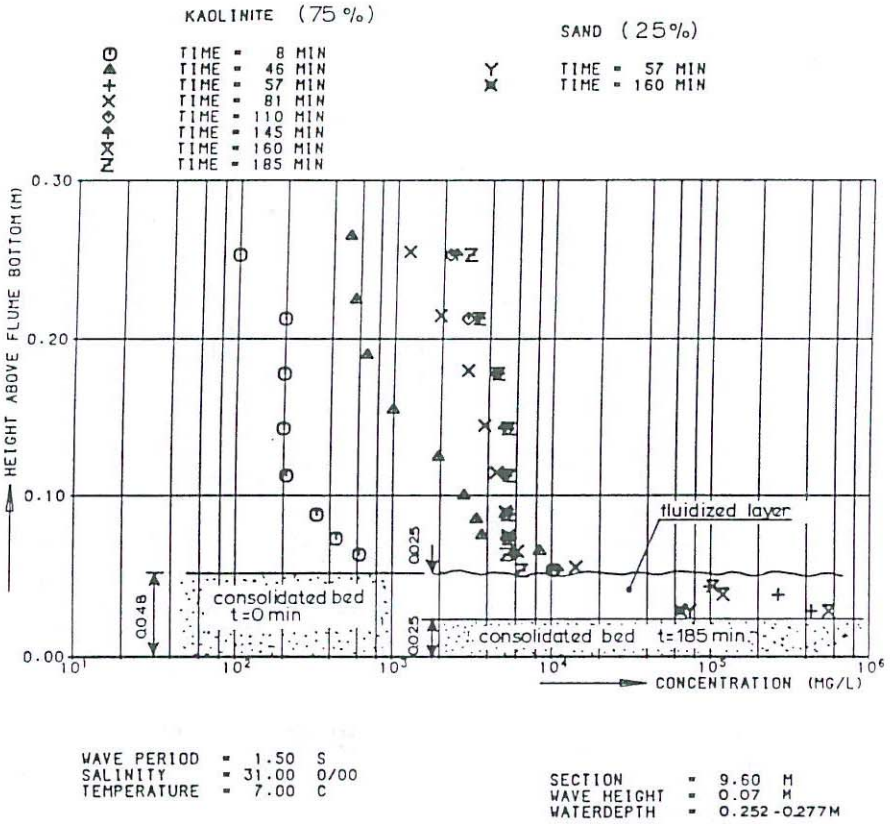

Figure 4: Measured concentration profiles in test T5-7.

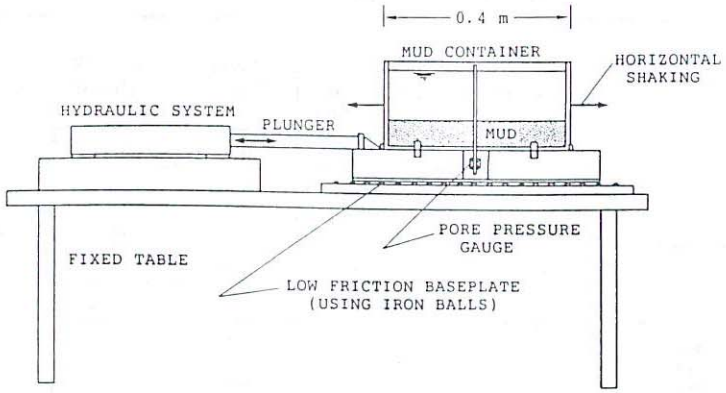

Figure 5: The shaking device.

to study the possibility of liquefaction. The test facility is shown schematically in Figure 5. Kaolinite was mixed with saline water to a required initial concentration. This mixture was poured into a transparent container to settle and consolidate. During consolidation the pore pressure in the mud layer just above the container bottom was measured continuously. The end of primary consolidation is defined by zero pore pressure, which occurred after a period of four to seven days.

After consolidation the container was placed on a low friction base plate mounted to a hydraulic system. During the actual test the mud container was shaken horizontally at a controlled combination of frequency and amplitude to simulate periodic stresses. Thus, the periodic stresses in the mud were induced by the acceleration of the container whereas in nature these stresses are mainly caused by oscillating wave-induced gravity pressures.

The dominant stress parameter characteristic of liquefaction is the ratio $\Delta\tau/\sigma_{vo}'$ of the shear stress amplitude $\Delta\tau$ and the initial effective vertical stress σ_{vo}'. The shear stress amplitude is related to the shaking intensity af^2 in which f is the frequency and a is the amplitude of the oscillations. With increasing $\Delta\tau/\sigma_{vo}'$ the excess pore pressure increases faster and liquefaction becomes more likely. Furthermore it was expected that the sensitivity of mud to liquefaction would depend strongly on the mud density. This would mean that the liquefaction sensitivity would be larger at lower densities. This is a dominant aspect in sands.

A number of tests with different mud concentrations were carried out, each at various frequency/amplitude combinations. Each test was carried out for 80 to 4000 shaking cycles. During these tests the pore pressure response was recorded continuously.

In Figure 6 the wave pressure response of the kaolinite mud layer is presented in a schematic way. The rate of pore pressure increase and the asymptotic value at the end of each test appear to be strongly dependent on the shaking characteristics. In Table 2 some of the main results are summarized. Here, the pore pressure ratio r is defined as the ratio of the actual pore pressure p measured near the bottom of the container and the initial (*i.e.* before the shaking tests) vertical grain effective stress σ_{vo}'. $r = 1$ (100%) means that the entire mud layer is liquefied. Because the density of the mud, and therefore σ_{vo} increases with depth, the sensitivity for liquefaction decreases with depth. This also implies that when only a small layer near the surface liquefies, the value of r further down in the bed will be smaller than unity, *i.e.* complete liquefaction in a thin top layer may occur for $r < 1$.

The onset of liquefaction near the surface of the mud layer and the gradual penetration to lower levels could also be observed visually during the tests by the increase of horizontal movement of the mud. Here the disturbing effect of the container walls became obvious because near these walls vertical movements also occurred.

At the beginning of a new frequency/amplitude combination the initial pore pressure started at the final value of the previous combination, as illustrated in Figure 6. This was done for practical reasons, as the period required for consolidation and pore pressure dissipation is fairly long. During two test series complete reconsolidation of the sample was allowed. These tests showed that the effect of an

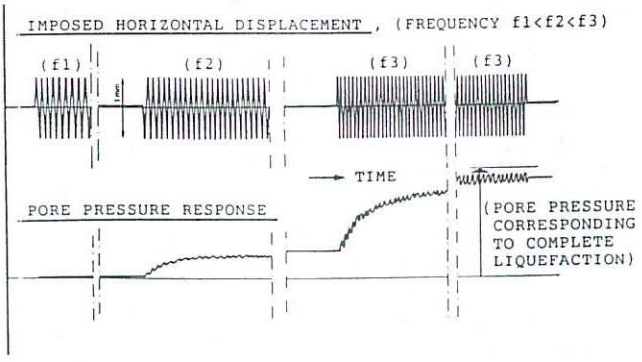

Figure 6: Pore pressure response to horizontal shaking.

Table 2. Results of shaking tests on kaolinite.

Test	Material	After Consolidation		Shaking Conditions		Maximum Pore Press Ratio r	
		Average conc.	Sample height	Frequency	Amplitude	N_1	r_{max}
		[kg m^{-3}]	[m]	[Hz]	[mm]	[cycles]	[%]
S3-3	Kaolinite	580	0.069	4	0.5	700	90
S4-3	Kaolinite	625	0.064	4	0.5	2000	55
S4-4	Kaolinite	625	0.063	5	0.5	300	90
S5-2	75% Kaol. 25% Sand	730	0.055	3	0.5	450	12
S5-3	75% Kaol. 25% Sand	730	0.055	4	0.5	80	90
S5-4	75% Kaol. 25% Sand	730	0.055	55	0.5	80	85

initial non-zero pore pressure only affected the rate of pore pressure increase (and thus N_1; see further below), and not the maximum pore pressure ratio r_{max}. From Table 2 it can be seen that, in spite of the increase of concentration or mud density, the sensitivity to liquefaction increased when 25% sand was added to the kaolinite (compare the number of cycles N_1 required for obtaining r_{max}; Tests S4-4 and S5-4).

From the results of these tests it was concluded qualitatively that liquefaction can occur in a kaolinite bed layer in a way quite similar to the liquefaction process of a sand bed.

Quantification of Liquefaction and Comparison with Flume Tests

The first step in quantifying the liquefaction process of kaolinite is the computation of the wave-induced stresses in the bed. This was done by approximation with the two-dimensional, elastic approach outlined by YAMAMOTO *et al.* (1978), giving the variations of the shear stresses with depth as a function of wave height, length, period, and water depth. Together with the (measured) density profile and σ_{vo}' as a function of depth, the cyclic stress ratio $\Delta\tau/\sigma_{vo}'$ was determined as a function of depth within the bed.

Next a quantitative procedure was developed to predict the liquefaction behavior of kaolinite as a function of the imposed stresses. For sand this can be done with triaxial tests. As noted, however, kaolinite is too soft to be tested in this way. Therefore, the stress characteristics of kaolinite were determined with the shaking device. The characteristics of this device could be determined from tests with fine sand (D_{50} = 135 μm), as the liquefaction behavior of sand is known from the cyclical triaxial tests. Thus the rate of increase of the excess pore pressure as a function of imposed stresses, and the number of cycles N_1 required to obtain liquefaction, could be derived. Because of the relatively high frequency of loading during shaking, these values of N_1 could be conceived as applicable for complete undrained conditions. In this procedure it is assumed that the pore pressure response is not a function of the frequency itself, but only of the magnitude of $\Delta\tau/\sigma_{vo}'$, which itself is related to the shaking characteristics.

This stage of analysis ended up with an expression for the number of cycles N_1 required for complete liquefaction under undrained conditions as a function of mud density and cyclic stress ratio $\Delta s/\sigma_{vo}'$ caused by wave pressure fluctuations in the mud bed. Subsequently, one-dimensional numerical soil consolidation calculations were carried out using the actual (measured) density profile, and the cyclic shear stress distribution computed with the above mentioned two-dimensional model. This numerical approach is similar to the method published by SEED and RAHMAN (1978), who included an equivalent pumping term for excess pore pressure generation.

The results of some of these calculations for Tests T3-4 and T5-7 are shown in the Figures 7 and 8. In the calculations, the 0.05 m thick kaolinite mud layer in the wave flume is simulated by 10 layers. These layers are thicker further down in the bed. The actual wave characteristics in the tests are prescribed to arrive at the number of load cycles

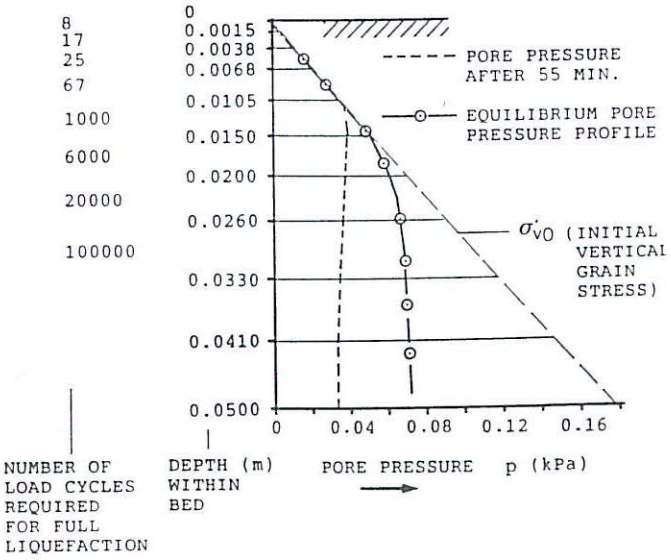

Figure 7: Calculated pore pressure as a function of depth and shaking time for test T3-4.

N_1 required for complete liquefaction under undrained conditions. Values of N_1 are given in Figures 7 and 8.

The numerical calculations yield increasing pore pressures as a function of time until an equilibrium is attained. At this stage (theoretically after infinite time) the pore pressure is balanced by an increase due to wave action and a decrease due to consolidation and drainage. Because of the low consistency and permeability of mud compared with sand, the influence of drainage is relatively small. The figures show that liquefaction starts at the surface and then penetrates to deeper layers. Because of the increasing mud density (in fact increasing value of N_1) the rate of penetration of liquefaction decreases considerably with increasing depth.

In Table 3 the measured (visually observed) and calculated liquefaction thickness for the wave tests are compared. It is seen that the computations describe the observations reasonably well. Only test T4-4 shows a considerable difference in the results.

A final remark should be made about the observed growth of the thickness of the bed in the flume after liquefaction. Due to wave-induced stresses, the grain structure in the bed collapses, generating an increase in pore pressure. Through drainage the excess pore pressure can dissipate, resulting in a decrease in bed thickness (and an increase in bed density).

However, it is hypothesized that under wave conditions the sediment will be entrained into the water column by the orbital velocities, which results in lower bed densities, and thus a larger thickness of the bed. The entrainment rate is affected favorably by the weakening of the bed due to the breakdown of the grain structure. This hypothesis is supported by the observation of small craters in the bed surface after the tests, indicating an outflow of water from the bed.

Additional Influences

After it was shown that liquefaction may occur in a low-density kaolinite bed and that the response under wave loading can be predicted, the study was extended to natural sediments. A special sample container was developed to take mud samples in natural waters. It consisted of a transparent case with a removable metal bottom and the same dimensions as the container in Figure 5. The container without bottom was pushed into the mud by a diver. Water could escape through openings in the top plate. After removal of some mud around the container, the bottom plate could be mounted by the diver. By firmly tightening bottom and top plates, the container was sealed and could be lifted out of the bed and carried above the water level.

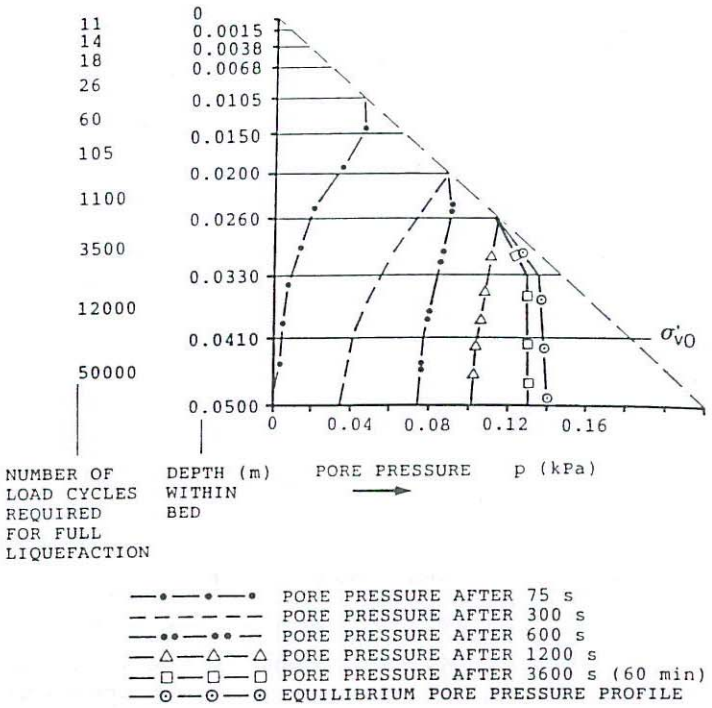

Figure 8: Calculated pore pressure as a function of depth and shaking time for test T5-7.

Table 3. Observed and calculated thickness of liquefied layer in wave flume experiments.

Wave Flume Experiment	Observed Thickness of Liquefied Layer	Calculated Liquefaction Thickness (equilibrium depth)
T3-3	No liquefaction	0.0015 m
T3-4	Between 0.01 and 0.02 m	0.015 m
T4-4	No liquefaction	0.0105 m
T4-7	Between 0.01 and 0.025 m	0.026 m
T5-4	No liquefaction	No liquefaction
T5-7	Between 0.01 m and 0.025 m	0.026 m

With this device a number of mud samples were taken from a shallow lake, the Markermeer in The Netherlands.

The grain size characteristics of this mud were: $D_{12} = 2\,\mu$m, $D_{50} = 12\,\mu$m, and $D_{90} = 30\,\mu$m. Organic material content was 5% by weight. In the laboratory the container with the sample was placed directly on the low friction base plate, connected to the hydraulic plunger and subjected to horizontal shaking. However, even at extreme conditions liquefaction did not occur. A subsequent analysis led to the following preliminary conclusions. The liquefaction resistance of the natural mud layer may have increased as a result of previous storm occurrences of limited strength. This history effect on liquefaction resistance is a well known phenomenon in the case of sand. Thus, it can be hypothesized that liquefaction is more likely to

occur in fresh mud deposits and less likely in somewhat older deposits which have withstood other, less severe storms. During such a milder storm, the pore pressure rises to a limited extent as a result of smaller wave loading. After the storm this pore pressure dissipates. The rate of pore pressure generation during a following storm will be reduced, basically because of a plastic hardening effect. Decrease of the excess pore pressure rate after previous drainage were also observed during the shaking tests.

The field samples contained a rather high amount of gas. This gas probably stems from organic processes. From theoretical geotechnics it is well known that the sensitivity to liquefaction reduces considerably when small amounts of gas are present in the pores of a soil. This hypothesis was supported by additional tests after a further treatment of the samples. First the samples were disturbed by mixing them thoroughly. During this mixing process only the large gas bubbles were removed. The aforementioned history effects, if present, probably disappeared during this mixing process. Furthermore, the organic material in the sample may have continued to generate a certain amount of gas. Again, the shaking tests did not show any liquefaction and/or increase in pore pressure.

Next, this disturbed sample was heated to remove the organic material and saturated again with de-aired water. This time the shaking tests showed a behavior very similar to the behavior observed with kaolinite including liquefaction of the top layer. One of the consequences of these findings is that data concerning gas content or organic material should be measured when the liquefaction behavior of natural muds has to be determined or explained.

CONCLUSIONS

A consolidated bed of kaolinite can be liquefied under wave conditions, resulting in a fluid mud layer. The liquefaction process seems to be similar to that of sand, and can be described by methods well known from non-cohesive soil mechanics. The liquefaction behavior of cohesive sediments seems to be affected strongly by the presence of gas and/-or organic material in the soil. The liquefaction analysis described in this paper must be conceived as a pilot study. In future attention must be paid among other things to verification of the liquefaction phenomenon under wave conditions (wave flume experiments including pore pressure measurement in the mud bed) and to a more detailed study of the stresses in mud shaking tests in relation to wave induced stresses.

ACKNOWLEDGEMENT

The authors would like to thank Rijkswaterstaat (Ministry of Transport and Public Works, The Netherlands) for funding the reported research and for their permission to publish the results.

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