Review of Molikpaq Geotechnical Material

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1 INTRODUCTION

The Molikpaq is a large steel caisson to provide a year round drilling platform for exploration in Beaufort Sea. It is designed for use in water depth of 15m to 50m and to resist ice forces during the majority of the year. The core of the caisson was filled with sand to provide the major component for resisting the ice loading. Core filling was carried out hydraulically through a pipeline discharging the fill at about the sea level. There was no densification of the core sand after deposition.

About 7-1/2 months after the completion of core filling, an ice event occurred on April 12, 1986. Some notable settlements occurred over about 1/3 of the core area with a maximum value of 1.5m at the fill surface. Since then a lot of studies have been conducted to examine the performance of the caisson during that ice event and the behaviour of the core sand.

At the request of Dr. R. Frederking, Canadian Hydraulics Centre, National Research Council of Canada, Dr. Law of Carleton University to conduct a review of the Molikpaq geotechnical material as described in the 2008 report by Kevin Hewitt "Estimates of Ice Loads on the Molikpaq Based on Geotechnical Analysis", along with other related documents.

2. SUMMARY OF THE 2008 HEWITT REPORT: "ESTIMATES OF ICE LOADS ON THE MOLIKPAQ BASED ON GEOTECHNICAL ANALYSIS"

The report first examines the evidence related to the performance of the core sand. The evidence included the placement method for the core sand, cone penetrometer profiles, pressuremeter test results and observed settlements during the ice event. Conclusions are drawn that the core sand was at a very loose to loose state and was potentially liquefiable.

The second part of the report deals with analytical models for analyzing the caisson performance during the ice event. The author noted that two out of the three models assume a dense core sand and would have over predicted the ice load magnitude. The third assumes a loose core sand and analyses two conditions, one under drained state and the other liquefied state. Based on the author's estimate, the maximum ice load during the ice event was less than 200MN, much lower than the estimated value based on load measuring devices.

3. CORE SAND BEHAVIOUR

The in situ behaviour of the core sand is an important issue in the design of the Molikpaq. There is an interdependence between the performance of the core sand and the magnitude of the ice load that can develop. The stability of the core sand is controlled jointly by its in situ density and the ice load, and the magnitude of the ice load is affected by the stability of the core sand.

As early as 1935, Casagrande (Cassagrande 1976) already recognized that when loose sand is sheared, it decreases in volume (or contracts) to reach a steady state volume (or density). On the other hand, the same sand sheared in a dense state will increase in volume (or dilate) to reach the same steady state condition as that of the loose sand. He called this steady state the critical state which is dependent on the effective confining stress. The steady state can be represented by a critical state line as shown in Figure 1 in which the critical void ratio is plotted against the mean effective confining stress, p'.

Further work shows that soils represented by void ratios (e) located below or to the left of the critical state line on a e-p' plot will dilate upon shear and are safe against liquefaction. Conversely, soils represented by void ratios located above or to the right of the critical state line will contract upon shear and may lead to unstable condition or liquefaction. How safe or unstable the soil is depends on the distance of the void ratio and effective stress from the critical state line. Been and Jefferies (1985) aptly define the term 'state parameter', ψ, to describe this distance. As shown in Figure 1, ψ is defined as the difference between the void ratio of the soil as it exists and the void ratio at the critical state at the same mean effective stress. If ψ is negative, the soil is dilative and if ψ is positive, the soil is contractive.

While there are standard laboratory procedures to determine the critical state line and void ratio, there are uncertainties to accurately determine the in situ void ratio of granular soils. Hence application of the above principles is faced with some difficulties. Indirect methods of determining the in situ void ratio have to be made, commonly involving in situ testing such as the standard penetration test (SPT), the cone penetration test (CPT) and the pressuremeter test. One of the most common methods now is to measure the in situ void ratio by correlation with the CPT tip resistance, q_c, and the mean effective confining stress, p'. This is in general what has been done by Hewitt (2008) and Jefferies et al. (1985) in assessing the core sand behaviour of the Molikpaq. However, the two investigators arrive at opposite conclusions on the nature of the core sand. In order to understand why the discrepancy exists, a brief discussion of their methods is given in the following.

There are ample evidences in the literature that for a given sand there is a reasonably unique relationship between void ratio, e, mean effective stress, p', and CPT tip resistance, q_c (Schmertmann 1977, Villet and Mitchell 1981, Baldi et al. 1986, among others). The void ratio, e, can be expressed in different forms. Hewitt (2008) expresss it in terms of relative density, D_r, following the work of Baldi et al. (1986) as shown in Figure 2. This figure shows the relationship between p′, D_r, and q_c as determined from pressure chamber tests in the laboratory. Based on this relationship, and from the cone penetration resistance, q_c, measured in the field, Sladen and Hewitt (1989) are able to determine the D_r (or void ratio) of 4 hydraulic fills in the Canadian Beaufort Sea. As postulated in this approach that D_r and q_c are uniquely related for a given sand at a given p', the measured q_c profile measured in the field can be replaced by D_r as shown in Figure 3. Based on observations of cases of similar soils that have failed in liquefaction and cases that have not, Sladen and Hewitt (1989) draw a line of demarcation separating dilative (potentially non liquefiable) from contractive (potentially liquefiable) soils

(Figure 4). This line of demarcation, however, has not been used in Hewitt's report (2008). His conclusion of a loose (contractive) core sand in this case, however, is based on the following:

" • The method of placement with no attempt to densify,
- CPT profiles,
- self-bored pressuremeter tests,
- the amount of settlement and the drastic increase in density achieved during subsequent blasting at Amauligak F-24 in 1987 (but still a lesser density than was achieved by bottom dumping!)."

Jefferies and his group of researchers (Jefferies et al. 1985, Been et al. 1986 and Been et al. 1987) use a different approach in determining the in situ void ratio based on CPT test and mean effective confining pressure. Instead of determining the in situ void ratio, they go one step further to determine the state parameter, ψ, as defined above. They propose that for a given sand, the normalized tip resistance is uniquely related to the state parameter. The normalized tip resistance is defined as $(q_c\text{-}p)/p'$ where p is the total mean confining stress. Hence, they suggest:

$$(q_c - p)/p' = k \exp(-m\psi)$$

where k and m are constants to be evaluated from pressure chamber tests.

As this approach determines the value of ψ, the liquefaction potential of the soil being investigated can be directly assessed.

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According to this approach, the Molikpaq core sand is dilative, which is in contrast to the conclusion of Hewitt (2008).

3.1 Reasons for the discrepancy

There are two sets of reasons contributing to the divergent opinions between the two groups of investigators. The first set of reasons are common to both but exerting different influence on the final results, while the second set are unique to each approach.

3.1.1 *Reasons common to both approaches*

This set of reasons is inherent in using the cone penetrometer conducted in a pressure chamber for assessing the in situ void ratio (relative density or state parameter). As both approaches are based on this indirect method to determine the in situ void ratio, the weakness of this method will be reflected in both approaches as follows:

(1) For most applications, it is not practical to conduct pressure chamber tests on the sand specific to the site. Hence, some existing calibration curves for other sands have to be used to infer the void ratio in the field. As pointed out by Leonard et al. (1991) that the q_c-p'-e relationship is influenced by the effects of fines content, grain shape,

aging, and grain size distribution, different sands will have different relationships and will yield different void ratios for the same q_c and confining pressure. Hewitt's approach is based on the pressure chamber test results by Baldi et al. (1986) on Ticino sand which is different from the Molikpaq core sand. It is not clear what sand was tested by Jefferies et al. (1985) in the pressure chamber.

(2) According to Sladen and Hewitt (1989), there is a lack of direct evidence to verify what is determined in the pressure chamber test in the laboratory will truly reflect behaviour in the field even if the sand from the field is used for the laboratory test.

At present, it is not certain to what degree these factors will influence the final results in each approach.

3.2 Reasons unique to each approach

3.2.1 *Hewitt's approach*

In Hewitt's approach, three quantities are required to determine if the nature of the core sand is dilative or contractive: CPT tip resistance, q_c, mean confining stress, p', and the location of the critical state line on a void ratio-confining pressure plot. In assessing the core sand state using the CPT tip resistance, Hewitt (2008) has only two quantities clearly defined, i.e., q_c and p'. Therefore he can only infer the relative density (void ratio) based on the pressure chamber test results on Ticino sand. With the inferred D_r, Hewitt estimates that the core sand is loose with reference to textbook definition based on the compactness of granular soils expressed in terms of relative density. However, he cannot definitively say that the core sand is contractive or potentially liquefiable. As discussed earlier, relative density and confining pressure together are insufficient to distinguish if a sand is contractive or not. At the same relative density, the same sand under shear may contract or dilate depending on the mean effective pressure confining it and the location of the critical state line.

3.2.2 *Jefferies approach*

This approach assumes a unique relationship between the normalized CPT tip resistance and the state parameter, ψ. However, Sladen (1989) has demonstrated that such a relationship does not exist for all stresses. Instead, the relationship varies with stress level. Ignoring this stress level dependency may lead to an error in assessing the in situ void ratio of 0.2 or greater, i.e., more than 50% in terms of relative density.

Because of the uncertainties involved in each of these approaches, it is not surprising that they yield different conclusions on the state of the core sand.

3.3 Other observations for assessing the nature of core sand

3.3.1 *Mode of placement*

The core fill was placed hydraulically using a pipeline with discharge just below the sea surface. As the fill falls to the bottom, there is a lot of opportunity for it to entrain water between the grains. Consequently, the fill is characterized by low relative density and weak strength. Based on the Beaufort Sea experience on building artificial islands, Sladen and Hewitt (1989) point out that the bottom dumping method, the other common method for forming hydraulic fills, produces a fill about 20-30 percentage points in relative density higher than the pipeline placed material.

It is possible, however, that the pipeline placed fill was in a loose state by textbook definition but it might not necessarily be contractive or liquefiable. Other factors have to be considered before the liquefaction potential of the fill can be assessed.

3.3.2 *Variability of the core sand*

Based on the CPT results given by Sladen and Hewitt (1989), the pipeline placed core sand at Amauligak I-65 is quite variable. At a given depth, the maximum tip resistance is two to five times the minimum value (Figure 5). This is not surprising for a hydraulic fill placed with pipeline discharge under water. With such a large variability, it is likely that the core sand as placed would have strong and weak zones. Even under uniform load intensity, these strong and weak zones will respond differently. During the 1986 April 12 event, the loading intensity varies significantly with location. The east caisson face experienced large accelerations up to 8%g while the other faces experienced only 1-2%g. Soil variability and varying load intensity caused some parts of the core sand to liquefy and other parts to remain intact. This is consistent with the observed variation of surface settlement after the event (Figure 6). The settlement plates at about 0.5m below the sand surface showed a maximum settlement of 1.31m on one side (east side) of the fill but hardly any settlement at the centre, the west and the north sides.

3.3.3 *Measured pore pressure*

The generation of excess pore water pressure is a complex process resulting from the long cyclic ice loading, permeability of the core sand and its state during loading, and the dewatering system installed in the core. According to Rogers et al. (1991), pore pressure transducer E1, located at around the mid height of the core sand near the east wall, shows a significant pore water pressure rise during the April 12, 1986 ice event to the point that the fill there liquefied. 9.5 m below E1 is another pore pressure transducer near the base of the Molikpag. This transducer shows a steady accumulation of excess pore pressure but not to the point to cause soil liquefaction.

4 ICE LOADS

The magnitude of the ice loads on the Molikpaq is another controversial issue in this project. This stems partly from the special design and performance of the load measuring devices and partly due the complexity of the ice loading interacting with the caisson and the core sand that leads to room for interpretation.

Two primary methods were used to measure the ice loads: 1) Medof panels mounted on the exposed surface of the structure and 2) strain gauges mounted on the steel bulkheads supporting the ice resistant face of the structure. The strain gauges were calibrated during other ice events based on the readings of Medof panels. The description and problems of Medof panels are given by Rogers et al. (1986). Briefly the readings of the panels are time dependent and the polyurethane buttons built into the panels creep upon loading, leading to errors in the results. Furthermore, both systems failed during critical moments in the major ice event on April 12, 1986.

The ice loads are the results of ice/structure interaction caused by multi-year ice. The load has both a normal and a shear component and the load transducers only measured the normal component. In spite of all the difficulties, Rogers et al. (1986), manage to consider many factors to conclude that the peak global load for the April 12, 1986 event is estimated at between 500 and 700. They also argue that this number is consistent with the crushing strength of the ice. This number, however, is strongly questioned by Hewitt (2008).

Hewitt's argument is based largely on the core sand being loose (or low in stiffness) and liquefied during critical moments in the ice event. He points out that the high ice load given by Rogers et al. (1986) cannot co-exist with the loose core sand by citing the model study that includes physical modelling (centrifuge tests), and numerical modelling by EBA and by GCRI. The results are shown in Figure 7, where the ice loads on the Molikpaq versus the induced horizontal displacements are plotted for the various models. Figure 8 further shows the result for the two EBA models, one on the core sand being drained and the other being liquefied during the ice event. Based on these models, one could use the measured horizontal movement to predict the ice load on the Molikpaq.

According to Rogers et al. (1986), the Molikpaq moved horizontally by only about 20 mm during the ice event. Even though Hewitt (2008) argues that the inclinometer casing buckled, rendering the readings for horizontal movement unreliable, he still uses the measured movements for supporting his notion that the ice load was much lower than the value given by Rogers et al. (1986).

There is another modelling based on the Sandwell model that Hewitt uses in his argument. It is not discussed here because even the author of the Sandwell model admits that the model produces questionable results in assessing the observed performance of the Molikpaq.

Hewitt (2008) further assumes that during the critical moments in the ice event, the core or a small annular zone adjacent to the caisson wall had liquefied. He then considers the core had near zero strength at the berm-core interface and behind the caisson wall. This implies that the liquefied annular zone extended from the top to the bottom of the core. Based on this, Hewitt (2008) concludes that that largest load experienced by the Molikpaq during the April 12, 1986 ice event is likely less than 200MN.

The assumption of a “liquefied core” or an annular zone extending from the top to the bottom of the core, however, is not consistent with the observed settlement distribution, measured excess pore pressure at different locations during the event, the top of the core being above water and the high variability of the core sand. If one takes these factors into account, one will likely obtain a higher estimated ice load.

5 CONCLUSIONS

(a) There are uncertainties in assessing the liquefaction potential of the pipeline hydraulically placed fill based on existing information.

(b) The core sand, placed through a pipeline with discharge just below the sea surface, is loose by text book definition, but whether or not it is contractive requires more consideration.

(c) The core sand is highly variable in nature.

(d) During the major ice event on April 12, 1986, high soil variability and different induced loading intensities lead to some parts of the core sand being liquefied while other parts remaining intact.

(e) It is difficult to accurately evaluate the ice load by means of modelling if the possibility of local liquefaction is not recognized.

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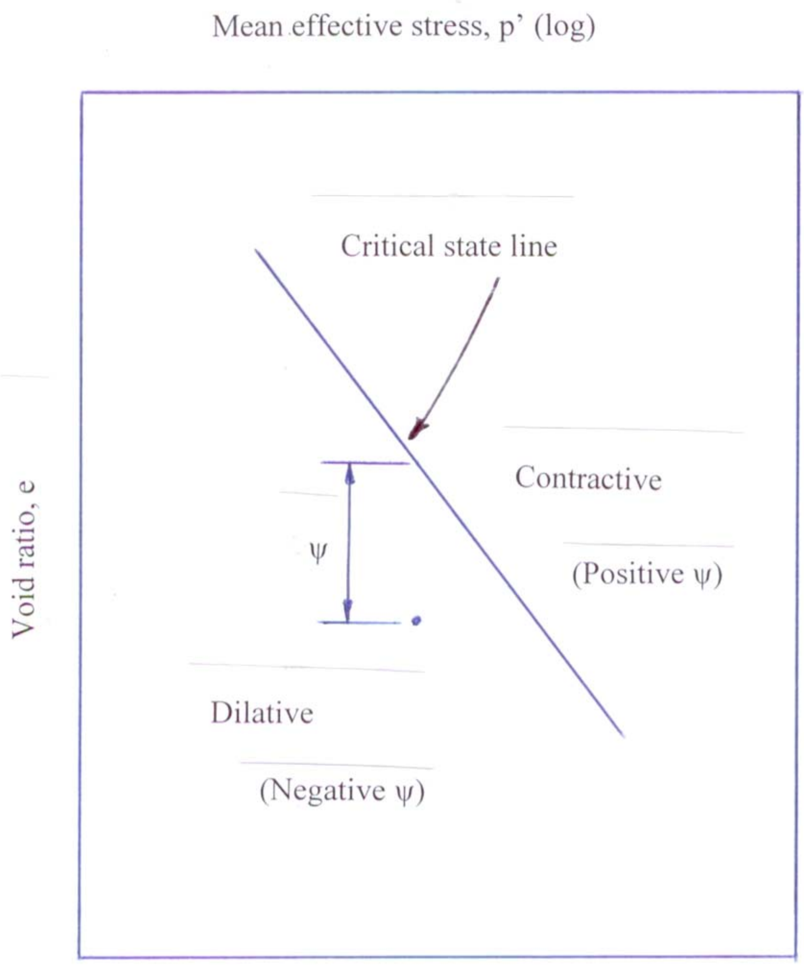

Figure 1 Definitions of critical state line and stare parameter

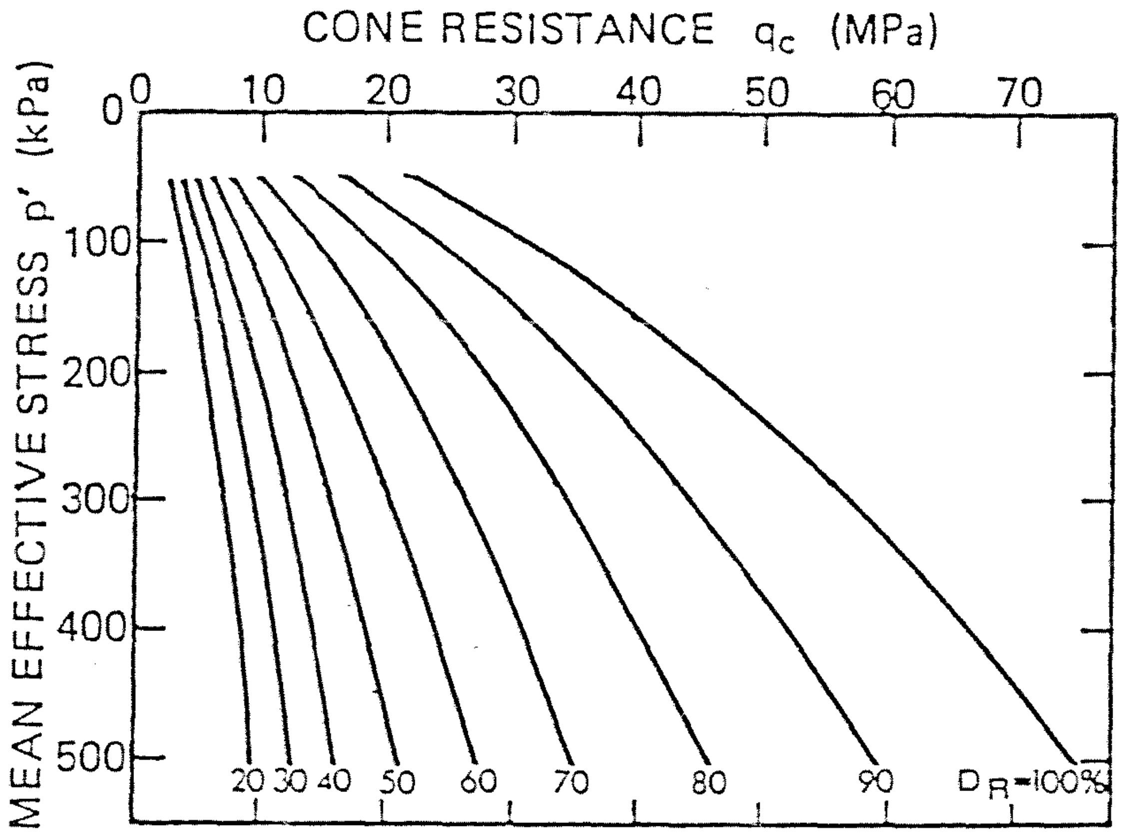

Figure 2: The relationship Between Cone Resistance and Relative Density (From Baldi et al. 1986)

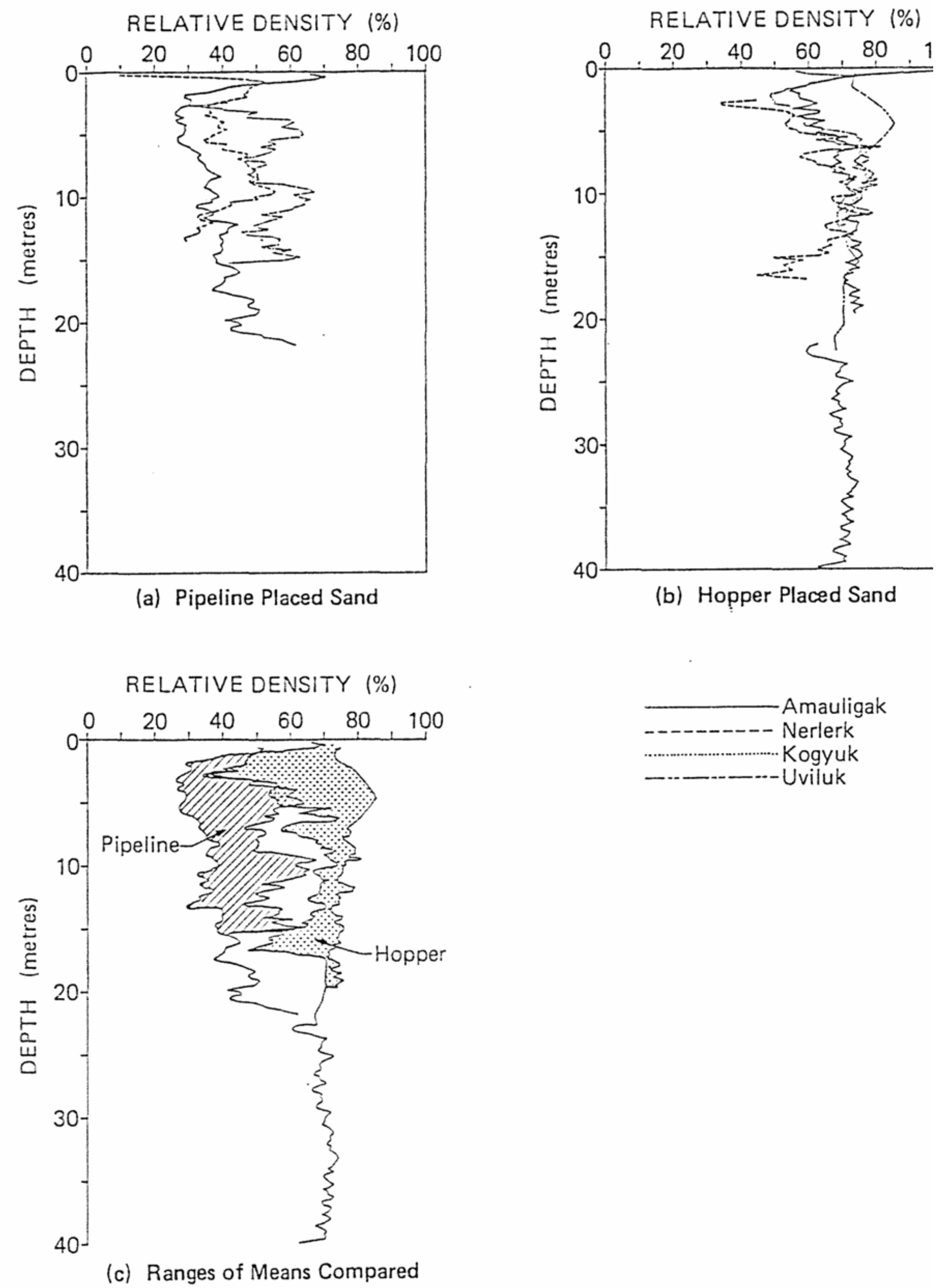

Figure 3: Comparative Profiles of Mean Relative Density for Hopper and Pipeline Placed Sands (After Sladen and Hewitt, 1989)

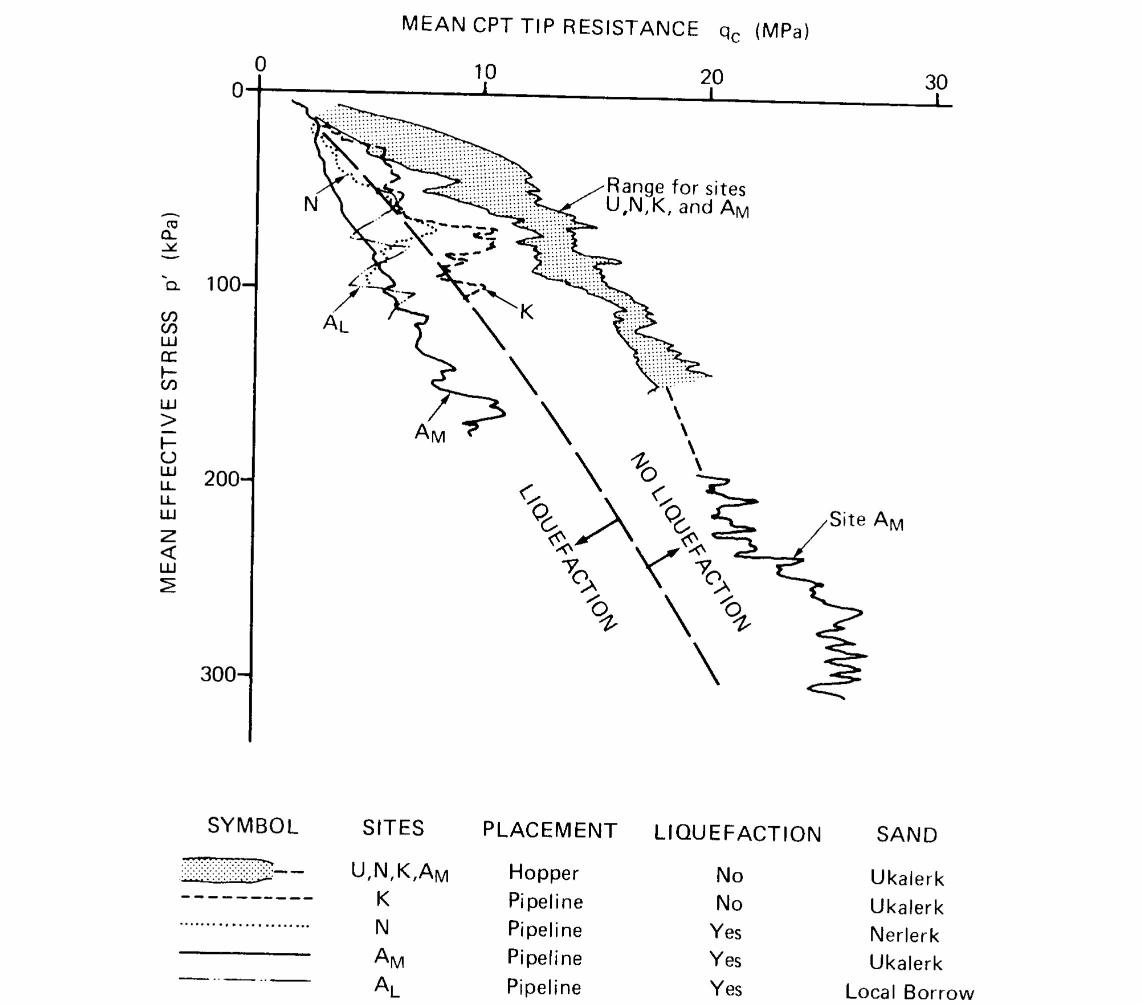

Figure 4 Mean tip resistance versus mean effective stress for the cases studied. A line separating sands that have exhibited liquefaction from those that have not is indicated. Note that although liquefaction has been reported for these sites, loading conditions were different. Key to site names: U, Uvluk P-66; Nerlerk B-67; K, Kogyuk N-67; A_M, Amuligak I65; A_L, Alerk. (After Sladen and Hewitt 1989)

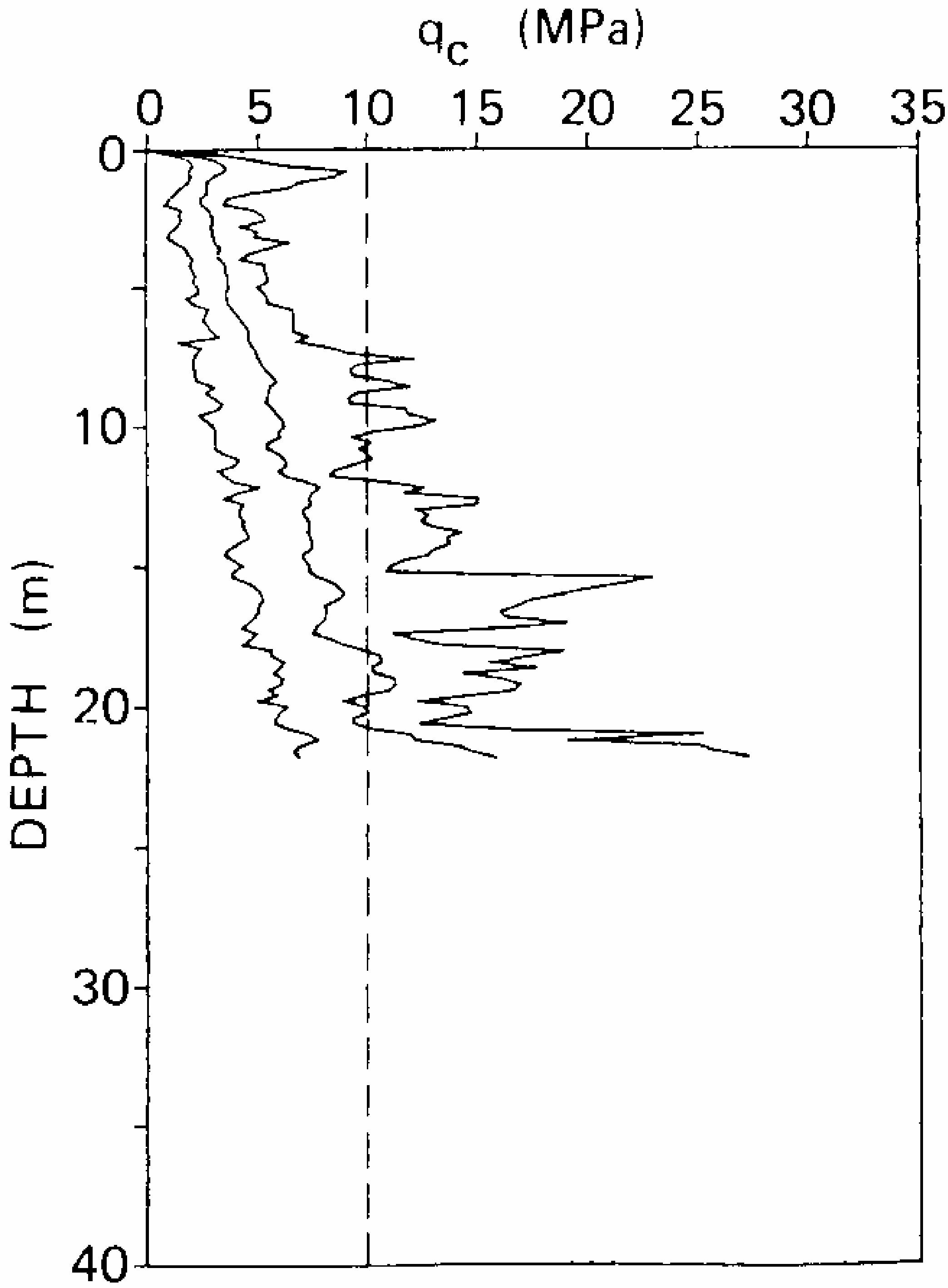

Figure 5 CPT Tip Resistance Profile for the Amuligak I-65 Core Sand

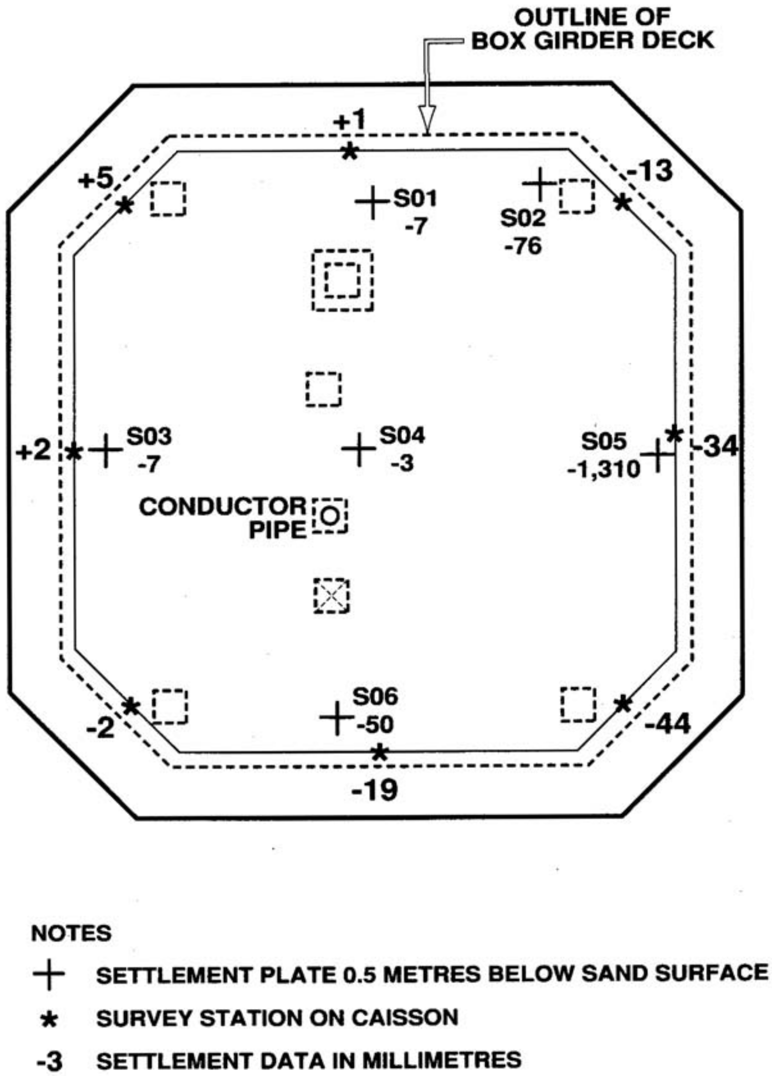

Figure 6 Settlement recorded from April 12, 1986 ice event (Rogers et al. 1986)

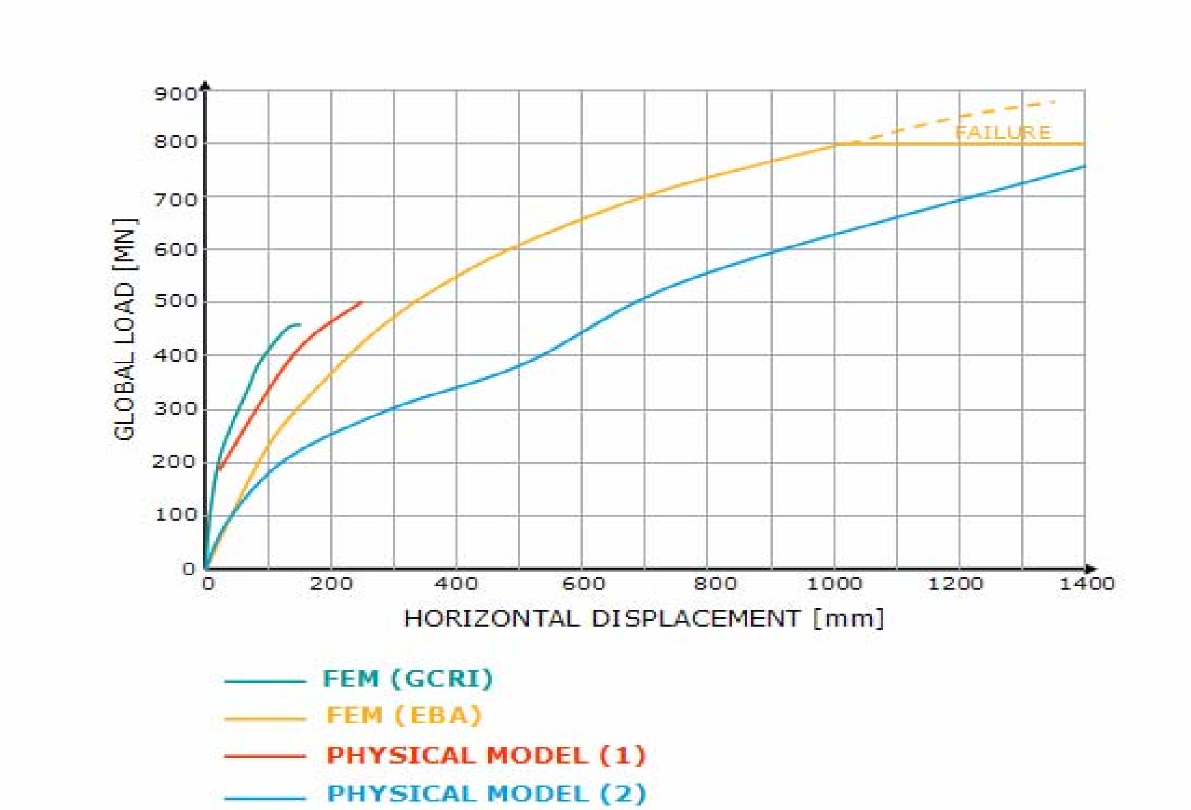

Figure 7: Predicted Horizontal Displacement at Point of Load Application (Hewitt 2008)

Note: Physical model (1) = centrifuge test on medium dense sand
Physical mode (2) = centrifuge test on loose sand
EBA analysis is based on loose sand
GCRI analysis is based on dense sand

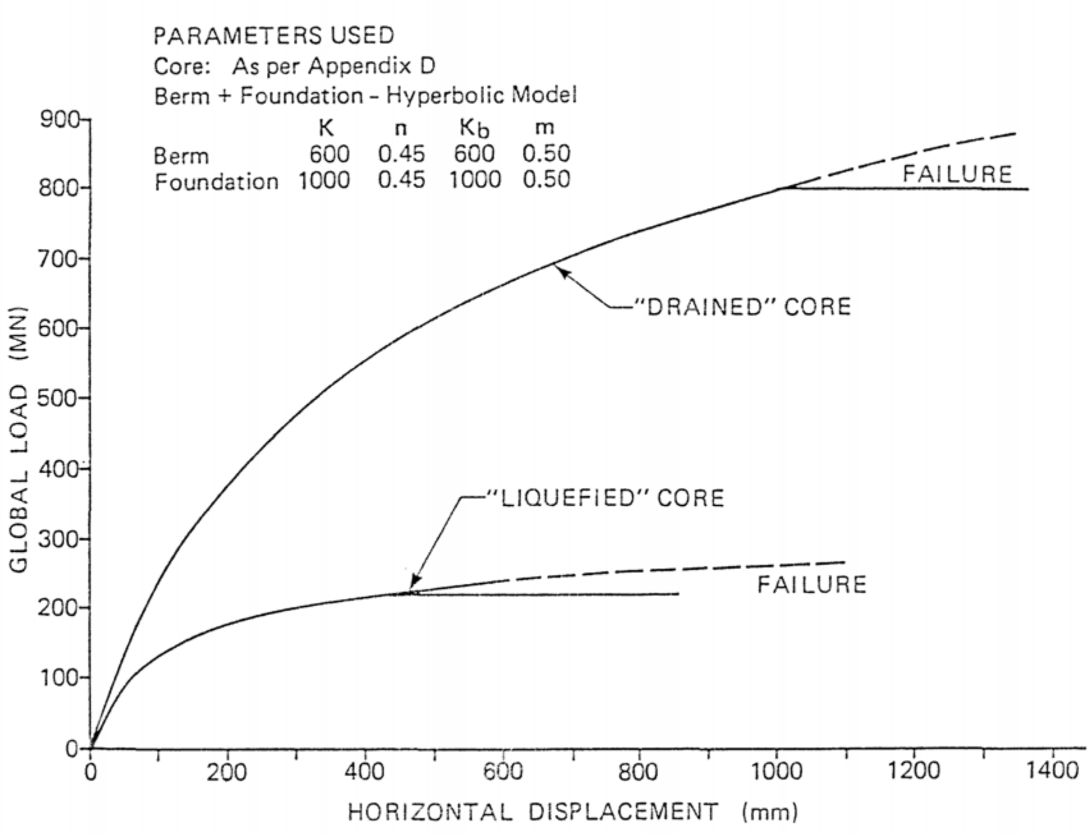

Figure 8 Load Displacement Relationships for Static Loading ("Drained Core") and for a Completely Liquefied Core by EBA analysis (Hewitt 2008)