Modeling the Performance of the Molikpaq

with Discussion and Reply

Ameir Altaee and Bengt H. Fellenius

Department of Civil Engineering, University of Ottawa,
Ottawa, Ontario, Canada, K1N 6N5

ALTAEE, A. and FELLENIUS, B.H., 1994. Modeling the performance of the Molikpaq. Canadian Geotechnical Journal, Vol. 31, No. 5, pp. 649 - 660

Modeling the performance of the Molikpaq

AMEIR ALTAEE AND BENGT H. FELLENIUS

Anna Geodynamics Inc., 5350 Canotek Road, Unit 22, Ottawa, ON K1J 9E2, Canada
and
Department of Civil Engineering, University of Ottawa, Ottawa, ON K1N 6N5, Canada

Received July 30, 1993

Accepted May 17, 1994

A nonlinear, two-dimensional (plane-strain), monotonic and cyclic undrained finite element analysis is carried out to investigate the stability of the Molikpaq offshore structure at the Amauligak I-65 site in the Canadian Beaufort Sea. The stress–strain–strength response of the hydraulically placed sand is modeled using bounding-surface elastoplastic constitutive relations. The behavior of the sand in the field as well as in the laboratory is simulated by the constitutive relations using a single set of nine parameters. The Molikpaq structure was analyzed for up to 100 cycles of lateral cyclic ice loading in the interval 3–5 MN/m. The pore-water pressures computed at three different locations agree well with piezometer measurements made in the actual structure. The results of the study demonstrate the importance of analyzing the stability of the structure for cyclic loading rather than relying on conclusions derived from static analysis.

Key words: analysis, cyclic loading, Molikpaq, sand fill retention, offshore structure.

Une analyse en éléments finis non linéaire, bidimensionnelle (déformation plane), monotonique et cyclique non drainée, a été réalisée pour étudier la stabilité de la structure offshore de Molikpaq sur le site de Amauligak I-65 dans la mer canadienne de Beaufort. La réaction contrainte–déformation–résistance du sable déposé par méthode hydraulique est modélisée en utilisant des relations de comportement élasto-plastiques aux surfaces frontières. Le comportement du sable tant sur le terrain qu'en laboratoire est simulé par des relations de comportement utilisant un seul ensemble de neuf paramètres. La structure de Molikpaq a été analysée sous chargement cyclique latéral de glace atteignant 100 cycles dans l'intervalle de 3–5 MN/m. La pression interstitielle calculée sur trois emplacements concorde bien avec les mesures piézométriques faites directement dans la structure. Les résultats de l'étude démontrent l'importance d'analyser la stabilité de la structure sous charge cyclique plutôt que de se fier aux conclusions obtenues par analyse statique.

Mots clés : analyse, chargement cyclique, Molikpaq, rétention du remblai de sable

[Traduit par la rédaction]

Can. Geotech. J. **31**, 649–660 (1994)

Introduction

Since the early 1970s, more than 20 artificial islands have been built in the Canadian Beaufort Sea. About five of these have been constructed applying caisson (cofferdam) technology to reduce the volume of sand required in the deep-water environment (Jefferies and Wright 1988). The primary design concern of such earth-fill retention structures lies with ensuring the stability of the caisson to ice loading.

The sand islands are used as working platforms for oil exploration drilling. A typical earth-fill retention island consists of an underwater sand foundation, which is a sand-fill berm of varying thickness, on which a caisson is placed and subsequently filled with sand, becoming the sand core.

One of these caissons is the Gulf Canada Resources Mobile Arctic Caisson (Molikpaq) that was moved to the Amauligak I-65 site in the Canadian Beaufort Sea in 1985. At the Amauligak I-65 site, the structure is subjected to cyclic ice loading. Once, the cyclic ice loading was so severe that the platform was close to its limit of stability (Jefferies and Wright 1988).

This paper presents analyses of the stability of the Molikpaq structure at the Amauligak I-65 site for both monotonic and cyclic lateral ice loading. The layout of the structure, the results of laboratory tests carried out on the hydraulically placed sand, and the magnitude and character of the ice loading considered in the analysis are taken from available literature (Jefferies and Wright 1988; Jefferies et al. 1985; Hicks and Smith 1988; and Sanderson 1988).

The stress–strain–strength response of the sand is modeled using a modified bounding-surface plasticity model. The structure is approximated as a plane-strain strip. A previous study (Hick and Smith 1988) concerning the static response of the same structure has shown that plane-strain approximation is representative for the behavior of the structure. The model parameters are determined from the results of the experimental programme conducted by Been and Jefferies (1985). It is assumed that the loads were applied rapidly, as described by Sanderson (1988) and Hicks and Smith (1988), and that there was not enough time for excess pore pressure to dissipate. The latter condition is modeled by performing ideal undrained analyses in all cases considered.

In contrast to the analysis presented by the authors, previous analysis of the Molikpaq structure at the Amauligak I-65 site has been limited to monotonic ice loading (Hicks and Smith 1988).

Offshore structure and location

The Molikpaq structure at the Amauligak I-65 site is described by Jefferies and Wright (1988), Hicks and Smith (1988), and Sanderson (1988). Figure 1 shows a cross-sectional elevation of the structure and its artificial-island foundation. A surficial layer of soft sediments overlying competent soil at the Amauligak I-65 site was unsuitable for supporting the berm and the Molikpaq structure. Therefore, it was excavated and replaced with hydraulically placed sand (Jefferies and Wright 1988). The mean sea level

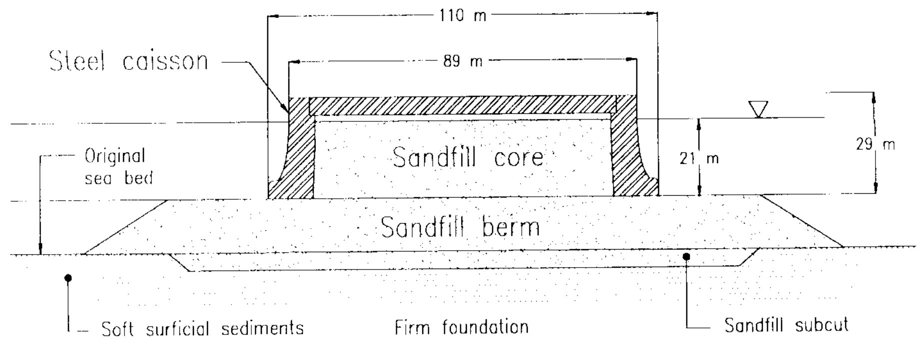


FIG. 1. Layout of the Molikpaq structure (after Hicks and Smith 1988). ∇, mean sea level.

TABLE 1. Modeling parameters for the bounding-surface plasticity model

Parameter	Symbol
Void ratio at 100 kPa mean effective stress $[p = (\sigma_1 + \sigma_2 + \sigma_3)/3]$ along the steady-state line	Γ
Slope of steady-state – critical-state line in e–ln(p) plane	λ
Unloading–reloading modulus in e–ln(p) plane	κ
Ultimate friction angle in compression	ϕ_c
Ultimate friction angle in extension	ϕ_e
Peak friction angle at largest ν value	ϕ_c
Poisson's ratio	ν
Bounding-surface aspect ratio	ρ
Hardening parameter	h_o

TABLE 2. Soil parameters specified for the fill sand at the Amauligak I-65 site (data from Hicks and Smith 1988)

	Core (less dilating)	Berm (more dilating)
ϕ_p (°)	33.5	35.5
ϕ_c (°)	30.5	30.5
Dilation $d\epsilon_{vol}/d\epsilon_{axial}$	0.1	0.2
Upsilon value	−0.025	−0.075

is assumed to be level with the upper surface of sand in the core. Other dimensions are as given by Jefferies and Wright (1988).

The Molikpaq caisson was built in Japan and towed to the Canadian Beaufort Sea in July 1984. The structure was first deployed in October 1984 at Tarsiut P-45. The design strength of the completed caisson for resisting ice loading is 8 MN/m. At the Tarsiut P-45 site, during the winter of 1984–1985, the ice loads were quite modest. In September 1985, the Molikpaq was moved to the Amauligak I-65 site and, at this location, during the winter of 1985–1986, the structure experienced very large ice loading events. In one case involving multiyear ice, the ice loads reached the magnitude of 5 MN/m (Jefferies and Wright 1988). This load corresponds to a factor of safety of 1.6 on the design strength.

Constitutive model and finite element program

In the authors' study, the stress–strain response of the sand is modeled by means of a bounding-surface plasticity model. The model was developed by Bardet (1986) and modified by Altaee (1991) and Altaee et al. (1992*b*) to overcome numerical problems associated with extreme softening of the soil in cases of low mean stress. It was subsequently implemented by Altaee (1991) into the finite element program (Advanced Geotechnical Analysis Code, AGAC) that was modified from the program SAC developed by Herrmann et al. (1986). The finite element program was used successfully in the analysis of several boundary-value problems including full-scale piles subjected to repeated axial loading tests (Altaee et al. 1992*a*, 1992*b*, 1993).

The soil constitutive model and the modified program have the capability of describing important features of sand behavior, such as stress hardening, strain softening, and accumulation of irrecoverable strains during cyclic loading.

As listed in Table 1, the model requires nine parameters for modeling soil behavior in generalized three-dimensional conditions. Details of the model formulation, the procedures for determining the parameters, and the finite element program are given by Altaee (1991). The parameters for a specific soil are established (calibrated) from results of consolidation tests and drained triaxial compression and extension tests. The parameters are independent of the initial state of the sand. The model parameters having the greatest influence on the overall response of a soil are Γ, λ, and ϕ, as defined in Table 1. Excess pore-water pressure is computed by considering a very small compressibility for the pore fluid and soil particles, as suggested by Sangrey et al. (1969), Zienkiewicz (1977), and Herrmann et al. (1986). For details of this approach as used with the constitutive model, see Altaee (1991).

TABLE 3. Laboratory tests on the sand fill (data from Been and Jeffreries 1985; Hicks and Smith 1988)

Test type	Test No.	p_{initial} (kPa)	Initial void ratio	Upsilon value
Isotropic loading–unloading	CI 1			<0.00
Drained triaxial compression	CID 1	77.5	0.63	−0.090
	CID 2	151.9	0.60	−0.100
	CID 3	349.4	0.62	−0.063
	CID 4	703.4	0.64	−0.020
Undrained triaxial compression	CIU 1	300.2	0.65	−0.033
	CIU 2	504.2	0.65	−0.015
	CIU 3	508.2	0.62	−0.042
K_0 triaxial consolidation	K1			>0.00
	K2			<0.00

TABLE 4. Numerical values of the model parameters

Parameter	Symbol	Value
Void ratio at 100 kPa mean effective stress	Γ	0.713
Slope of steady-state line in e–ln(p) plane	λ	0.029
Unloading–reloading modulus in e–ln(p) plane	κ	0.006
Angle of friction (°)		
ultimate in triaxial compression	ϕ_c	30.5
ultimate in triaxial extension	ϕ_e	30.5
peak in triaxial compression	ϕ_p	35.5
Poisson's ratio	ν	0.100
Bounding-surface aspect ratio	ρ	2.100
Hardening parameter	h_0	1.000

Soil data at the Amauligak I-65 site

Table 2 presents the data specified by Gulf Canada Resources for the sand fill material at the Amauligak I-65 site. The specified initial void ratio differences to the steady-state line for the sand fill are −0.075 and −0.025 in the berm and core, respectively.

The initial void ratio difference to the steady-state line, called the upsilon value, is an important parameter. The upsilon value was called "e-prime" by Roscoe and Poorooshasb (1963), "state parameter" by Been and Jefferies (1985), and "characteristic density" by Jefferies et al. (1985). Because the upsilon value in the berm is larger than that in the core sand, the tendency to dilate during drained shearing for the sand in the berm is stronger than for the sand in the core. Consequently, they have dissimilar peak effective angles of friction (ϕ_p).

Been and Jefferies (1985) and Hicks and Smith (1988) also reported the results of a series of laboratory tests on the sand fill involving consolidation tests, and drained and undrained triaxial compression tests. The test types and the main test results are listed in Table 3. These test results enable the authors to obtain the required numerical values for the model parameters as listed in Table 4. Of these, the model parameters Γ, λ, ϕ_c, and ϕ_p are taken directly from the results presented by Been and Jefferies (1985). The remaining parameters are determined from the test data following the standard procedures described by Bardet (1986) and Altaee (1991), with the exception of the effective ultimate angle of friction in triaxial extension (ϕ_e), which value is assumed equal to that in triaxial compression (ϕ_c).

To verify that the constitutive model could adequately describe the behavior of the sand, the finite-element program was used to numerically simulate the sand response during laboratory tests as to the mean stress, deviator stress, and pore pressure versus axial and volumetric strain. Figure 2 illustrates the finite-element representation for the laboratory tests as a single axisymmetric finite element with the boundary conditions shown in the figure. The results of the numerical response are shown in Figs. 3–8 as "computed" results compared with "experimental" results, as addressed in the following.

Figure 3 shows the comparison for the isotropic loading and unloading testing, case CI 1. The solid lines represent the experimental behavior as taken from the results published by Hicks and Smith (1988). The broken lines show the behavior computed by the authors.

Figure 4 compares experimental and computed behavior of the four drained triaxial compression tests, CID 1–CID 4. The agreement is good for the deviator stress versus axial strain and not as good for the volumetric strain versus axial strain. For the latter, however, the quality of agreement lies in getting the right sequence of the tests, which is achieved in the numerical simulation. The overall agreement between the experimental and computed drained behavior shown in Figs. 4 and 5 demonstrates that the model parameters are properly determined from the drained tests.

To demonstrate the validity of the model to simulate undrained response using the parameters determined from drained testing, the behavior during the undrained triaxial compression tests, CIU 1–CIU 3, is simulated by the finite-

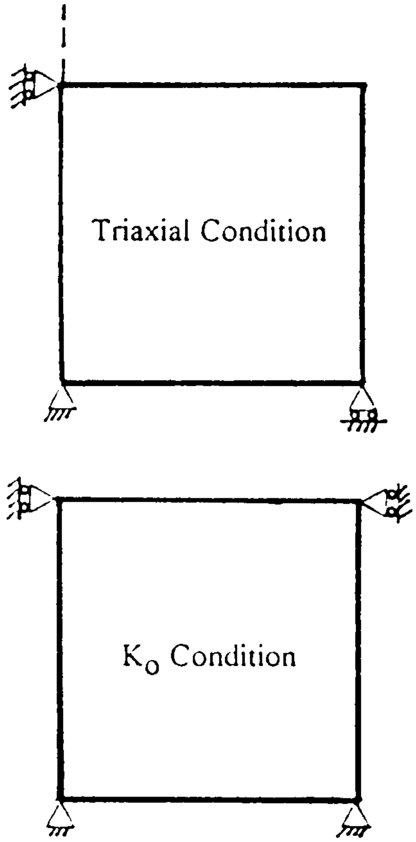


FIG. 2. Finite element idealization of laboratory test samples.

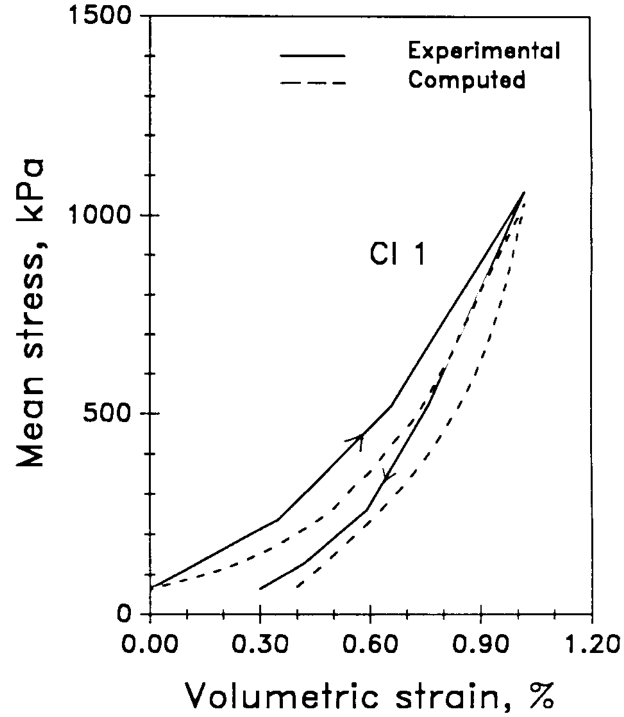


FIG. 3. Isotropic loading and unloading (CI 1).

element program without changing any of the nine parameters. Figures 5–7 show the results in terms of deviator stress versus axial strain and excess pore-water pressure versus axial strain as comparisons between experimental and computed undrained behavior. In all three undrained tests, the computed behavior agrees well with the measured behavior.

Finally, Fig. 8 presents a comparison between the experimental and computed behavior during the K_0 triaxial consolidation tests, K1 and K2, which shows that also the K_0 triaxial consolidation response is correctly modeled.

The results of the K_0 triaxial consolidation tests as well as those of the three undrained triaxial compression tests were not used in determining any of the model parameters. Therefore, the agreement between experimental and computed behavior is an indication of model generality for drained and undrained situations using one set of model parameters.

The results of the comparisons verify that the bounding-surface plasticity model using the *single* set of nine model parameters as established from the results of drained tests is capable of correctly modeling the behavior of the sand during drained and undrained tests.

Analysis of Molikpaq structure

Layout and initial states

The layout of the idealized caisson retained island (the finite element mesh) is shown in Fig. 9. The dimensions chosen in the analysis are those shown earlier in Fig. 1. The applied horizontal ice loading or the imposed horizontal displacements are applied at the mean sea level at the location represented by Q.

The nodes along the lower boundary of the mesh (representing the boundary between the berm and the sea-bed soil) were fixed. Vertical displacement was allowed for nodes located along both sides of the mesh, but horizontal displacement was prevented. The excess pore-water pressure was assumed to be zero at the nodes located at the surface of the core as well as at the surface of the berm located on both sides of the caisson.

Initial conditions similar to those applied to the monotonic analysis by Hicks and Smith (1988) were used in the present analysis: the initial effective vertical stresses in the soil mass were calculated using a total unit weight of 20 kN/m^3 for the soil; the horizontal stresses were taken as 0.7 times the vertical stresses; a vertical stress of 70 kPa was applied at the caisson "foot" to account for the caisson self-weight as well as weight of items placed on the platform; and a net surcharge of 10 kPa was assumed to act on the exposed surface of the berm sand.

Table 5 lists the cases considered in the authors' analysis. All cases assume undrained behavior. The first five cases of analysis, S1–S5, represent monotonic (static) loading conditions with the sand in the core and in the berm assumed to have equal upsilon value. These illustrate the effect of varying the degree of placement density and compaction of the sand on the overall response of the structure. Case S1 represents a very loose sand, as indicated by its upsilon value of +0.025 (contractive sand, a state above the steady-state line). Case S5 represents a dense sand; the upsilon value is −0.075 (dilating sand, a state below the steady-state line).

Cases S6 and S7 illustrate the relative importance of having different degrees of compaction of the sand in the berm and core. Case S6 represents a case having a dense sand (value of −0.050) in the core and a somewhat looser sand in the berm (value of 0.000). Case S7 is the reverse of case 6: looser sand in the core and denser sand in the berm.

Cases C1–C5 represent the analysis under cyclic loading condition. The first three cases have sand of equal upsilon

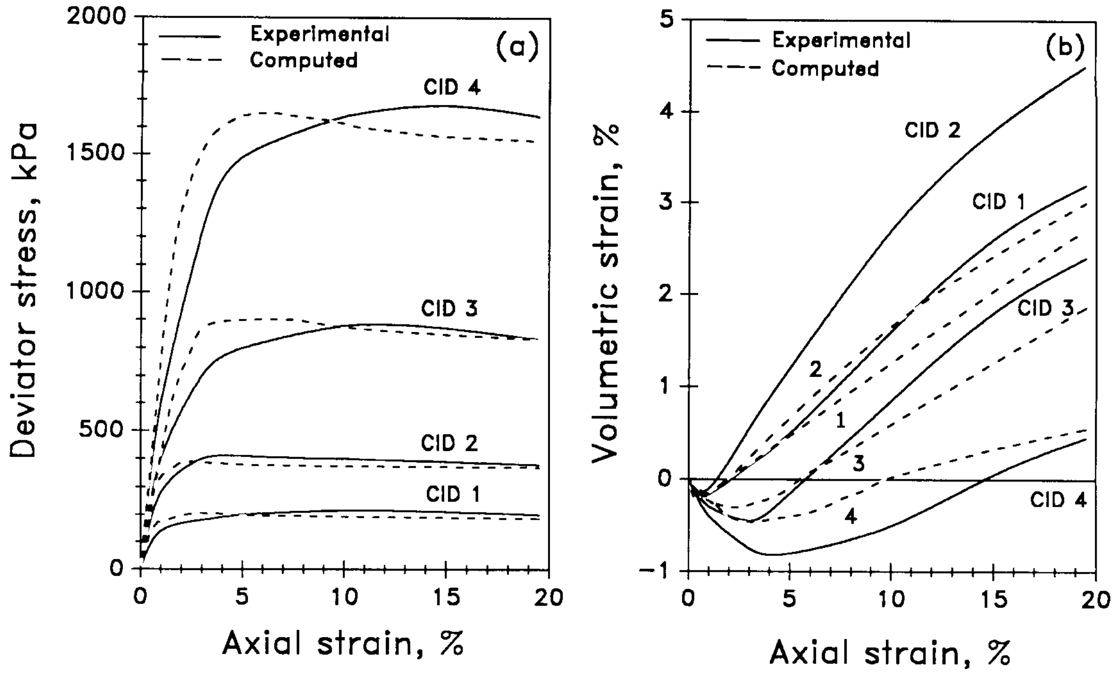


FIG. 4. Drained triaxial compression (CID 1 – CID 4). (*a*) Deviator stress vs. axial strain. (*b*) Volumetric strain vs. axial strain.

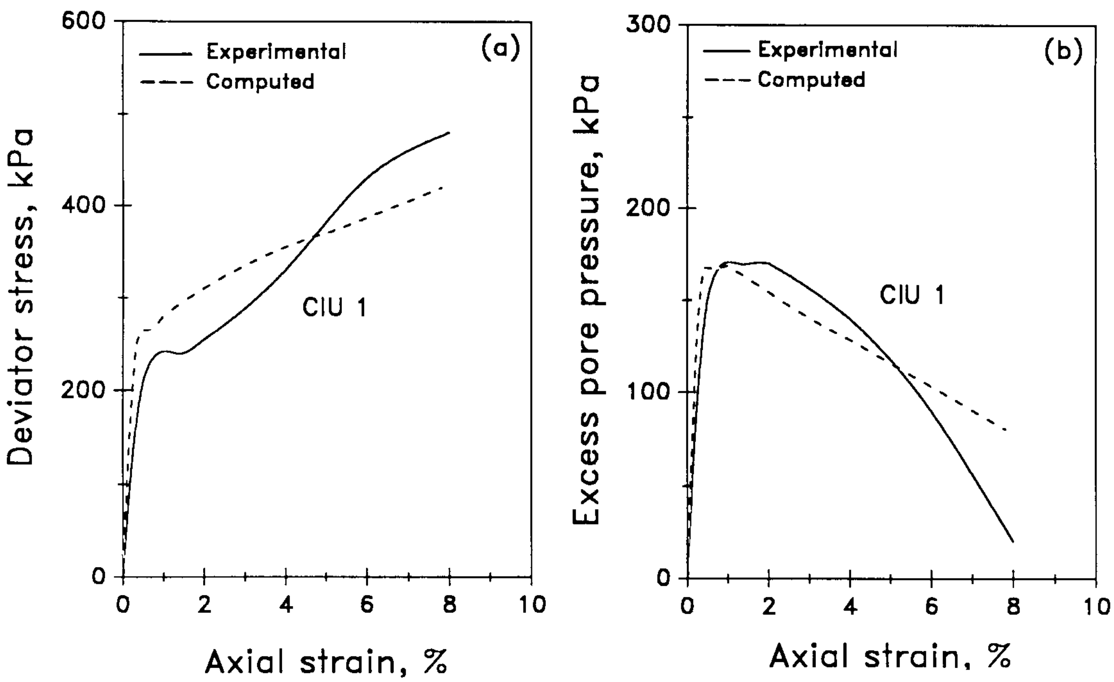


FIG. 5. Undrained triaxial compression (CIU 1). (*a*) Deviator stress vs. axial strain. (*b*) Excess pore-water pressure vs. axial strain.

value in the core and berm, ranging from 0.000 through −0.075. Cases 4 and 5 have dissimilar upsilon values in the berm and the core. Case 4 represents a case of a dense sand (upsilon value of −0.075) in the core and a looser sand in the berm (upsilon value of −0.025). Case C5 is the reverse of case C4: looser sand in the core and denser in the berm. Case C5 represents the actual condition of the structure at the Amauligak I-65 site.

Analysis results: monotonic loading

The monotonic cases of analysis, S1–S5, are carried out by imposing increments of horizontal displacement at point Q. The analysis was designed to stop when the caisson displacement reached 1 m, that is about 1% of the caisson diameter. The computed response is shown in Fig. 10 in terms of horizontal force versus horizontal displacement.

Figure 10 shows that the horizontal response depends on the value of the upsilon parameter of the soil and there is a benefit in compacting the sand: the larger the void-ratio difference below the steady-state line, the larger the resistance. Furthermore, under the observed maximum horizontal ice load of 5 MN/m, the computed horizontal displacements are relatively small, 50–100 mm, for all the five cases of analysis. This corresponds well with the measured horizontal displacement of 80 mm as reported by Hicks and Smith (1988). Jefferies et al. (1985) presented results of

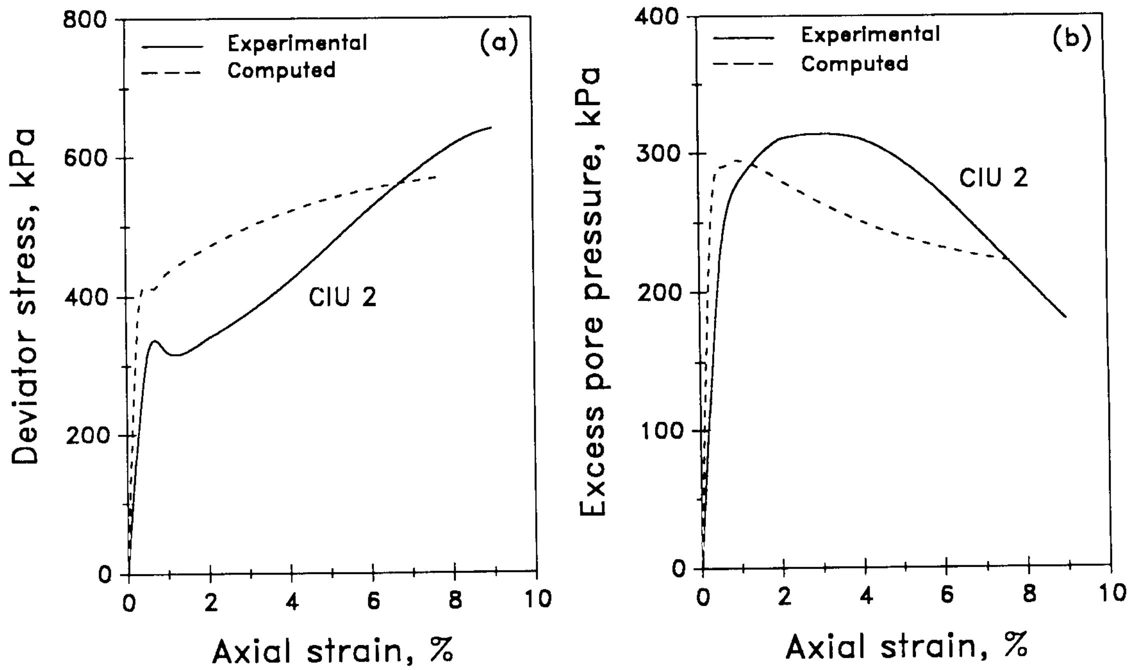


FIG. 6. Undrained triaxial compression (CIU 2). (*a*) Deviator stress vs. axial strain. (*b*) Excess pore-water pressure vs. axial strain.

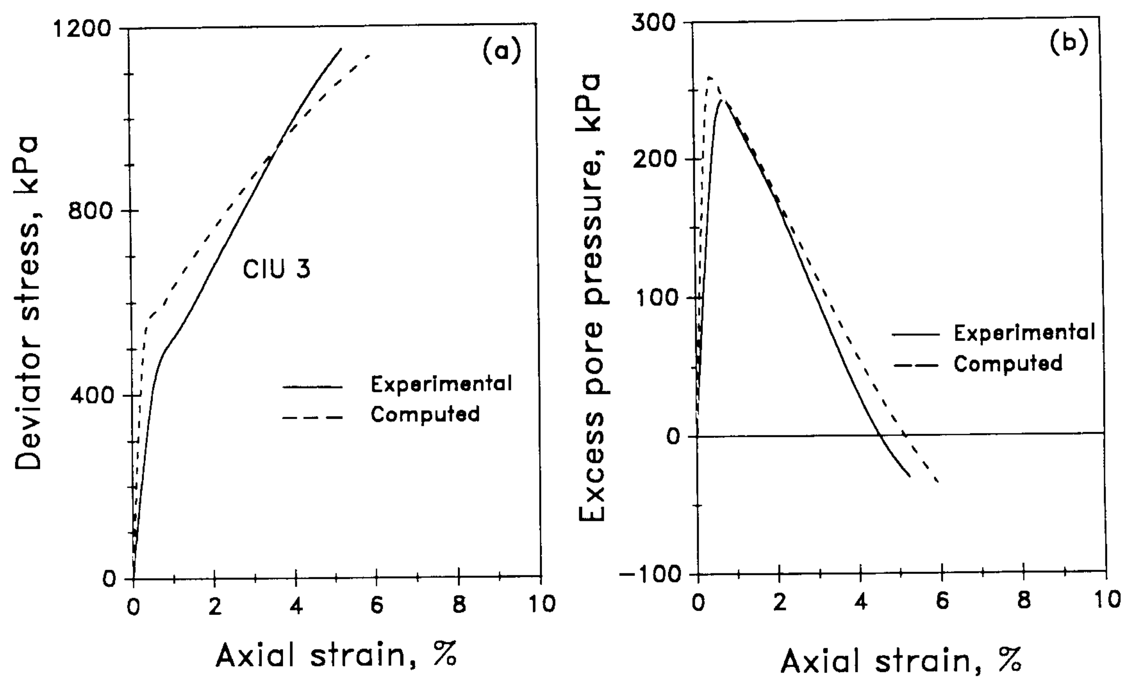


FIG. 7. Undrained triaxial compression (CIU 3). (*a*) Deviator stress vs. axial strain. (*b*) Excess pore-water pressure vs. axial strain.

numerical analysis and centrifuge testing of the Molikpaq at Tarsiut P-45 site. For this site, the computed numerical horizontal displacement (at top of caisson) under the design ice load was about 150 mm. Two centrifuge tests, using undensified Erksak sand, indicated horizontal displacement of about 250 mm results from applying the design ice loading.

The computed excess pore-water pressure generated by the loading is shown as determined for two locations, one in the core and one in the berm. The locations coincide with the actual locations of three piezometers installed in the core (P1 and P2) and berm (P3), as reported by Hicks and Smith (1988).

The excess pore-water pressures computed for the five cases are shown in Fig. 11*a* for the berm location (P3) and in Fig. 11*b* for the core location (P2). In the berm, the pore-pressure development is as one would expect: positive excess pore-water pressure develops initially. Then, for the cases having looser sand, the excess pore-water pressure continues to increase, whereas for the cases having denser sand, the excess pore pressures reduce, even becoming negative at large movement.

For the core location, however, the pore-pressure development is very different: independently of the initial sand state, excess pore-water pressure is generated that continues to increase with increasing horizontal displacement of the structure. The larger the upsilon value (negative values), the higher the computed excess pore-water pressure for the same horizontal movement.

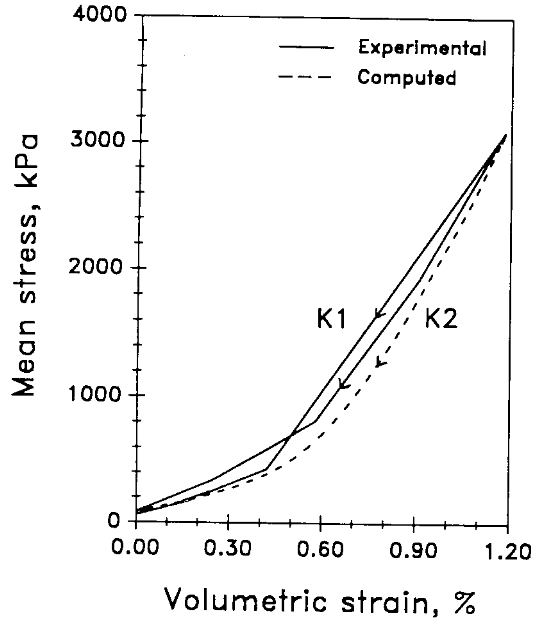


FIG. 8. K_0 unloading (K1 and K2).

TABLE 5. Cases analyzed

Case	Type of analysis	Upsilon value	
		Core	Berm
S1	Static	0.025	0.025
S2	Static	0.000	0.000
S3	Static	−0.025	−0.025
S4	Static	−0.050	−0.050
S5	Static	−0.075	−0.075
S6	Static	−0.050	0.000
S7	Static	0.000	−0.050
C1	Cyclic	0.000	0.000
C2	Cyclic	−0.025	−0.025
C3	Cyclic	−0.075	−0.075
C4	Cyclic	−0.075	−0.025
C5*	Cyclic	−0.025	−0.075

*Actual condition.

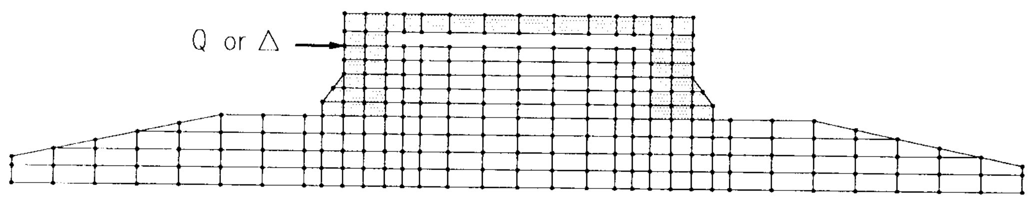


FIG. 9. Finite element mesh used in the analysis.

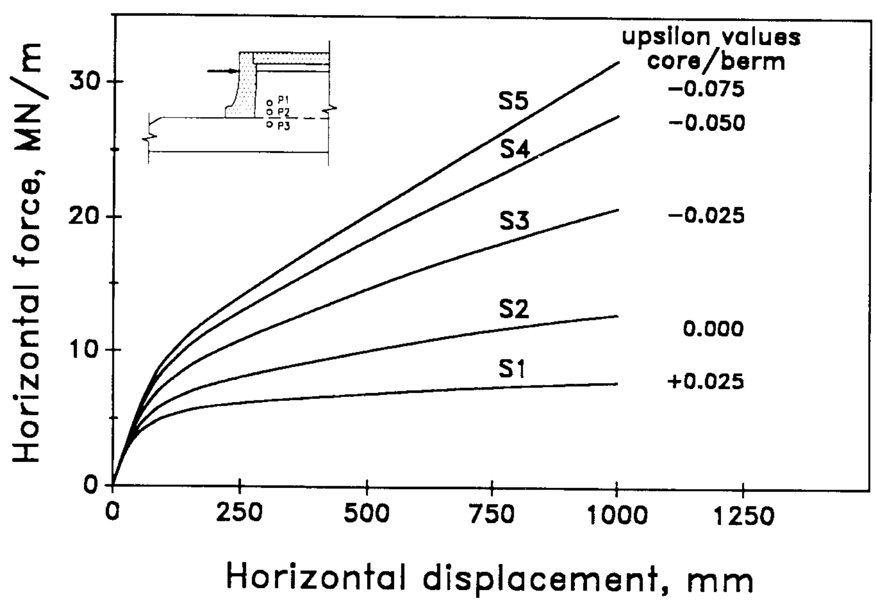


FIG. 10. Horizontal force vs. horizontal displacement for cases S1–S5.

That the berm develops negative excess pore pressure, while the core does not, explains the higher resistance of the structure built with denser berm sand. The resistance increase correlates to the higher negative excess pore-water pressure generated in the berm.

To further demonstrate the influence of the different initial density as represented by the different values of the upsilon parameter, the analysis results of cases S6 and S7, which have dissimilar upsilon values for berm and core sand, are compared and referenced to the results of cases S2 and S4 where the values are equal in both berm and core. Figure 12 shows the computed horizontal force versus displacement, indicating that a dense berm combined with a loose core results in a structure of higher strength than a

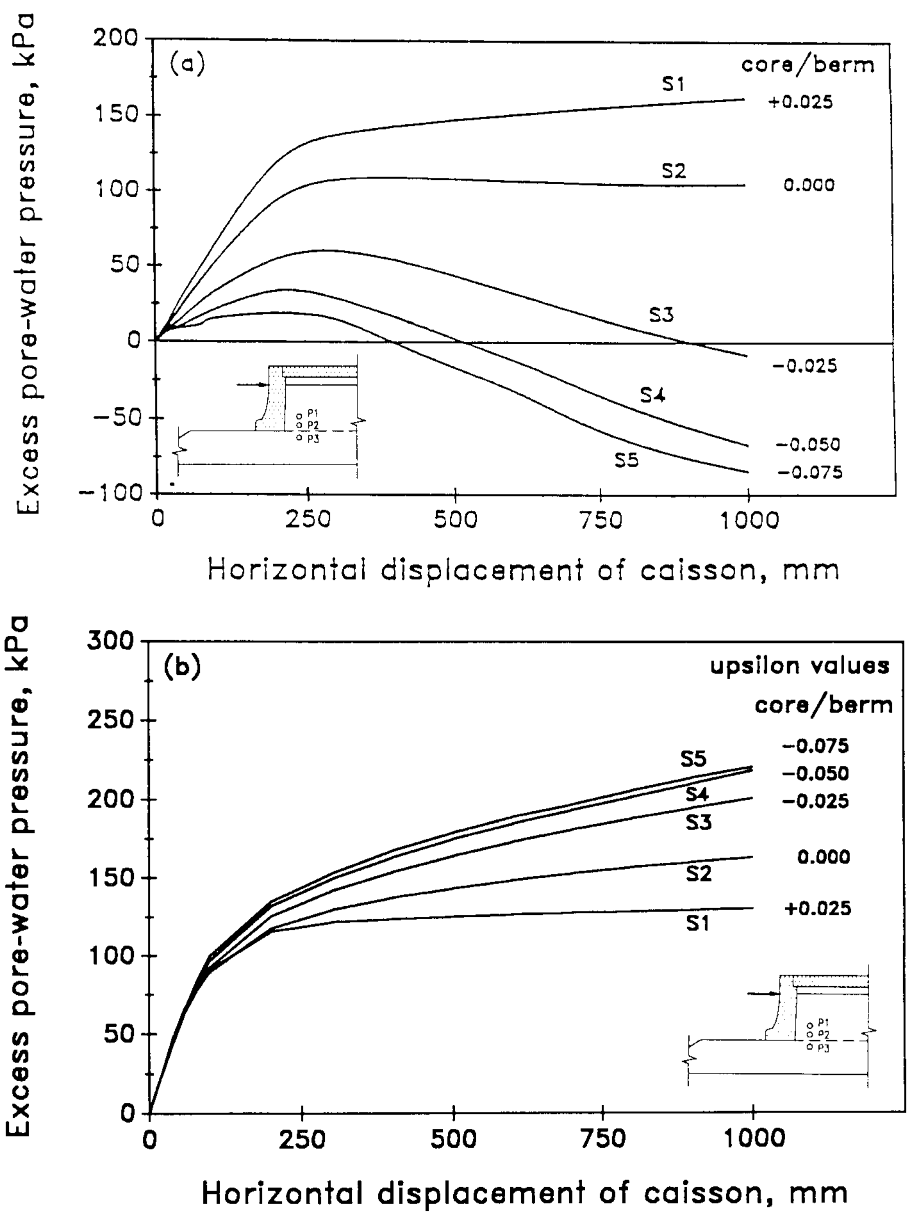


FIG. 11. Pore-water pressure at the leading wall side for cases S1–S5. (*a*) In the berm at location P3. (*b*) In the core at location P2.

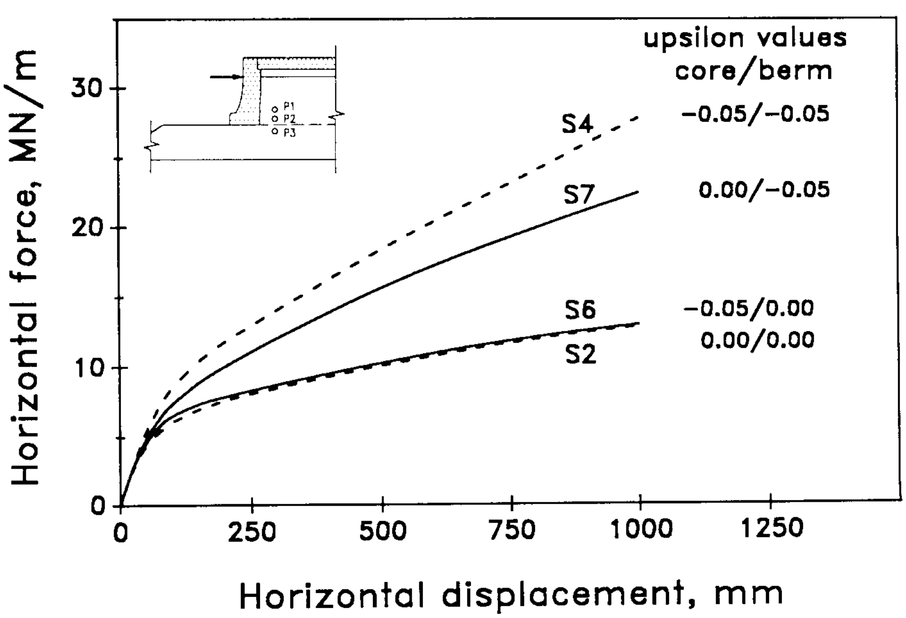
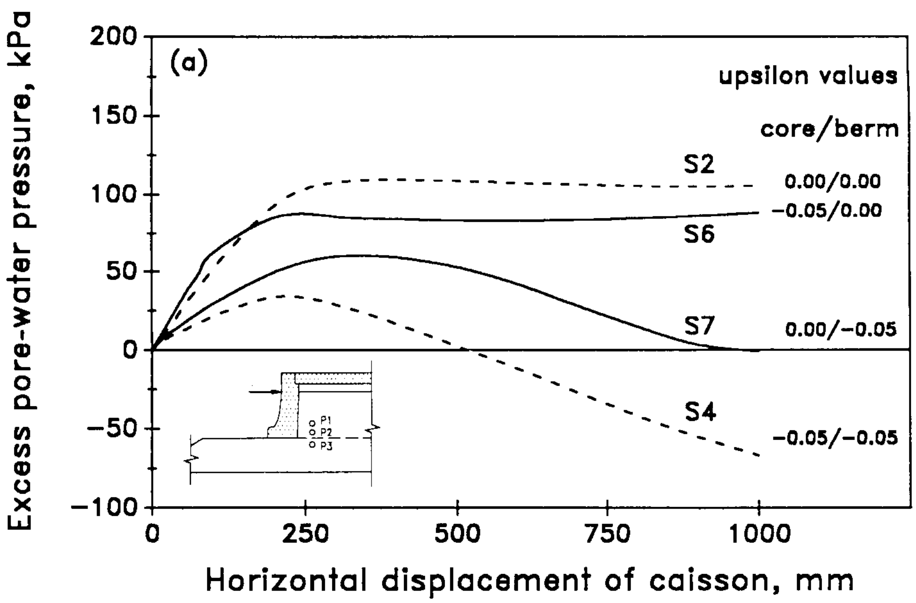


FIG. 12. Horizontal force vs. horizontal displacement for cases S2, S4, S6, and S7.



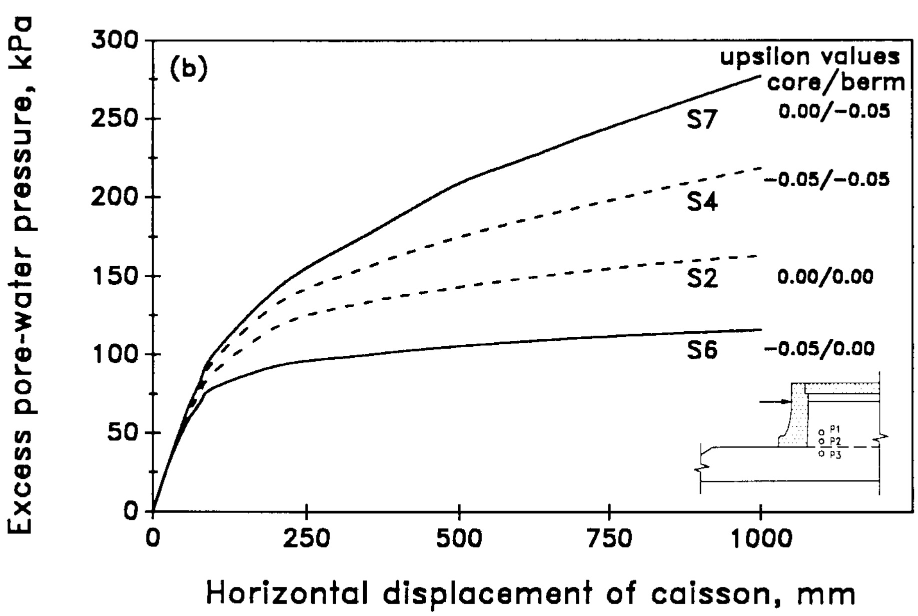


FIG. 13. Pore-water pressure at the leading wall side for cases S2, S4, S6, and S7. (*a*) In the berm at location P3. (*b*) In the core at location P2.

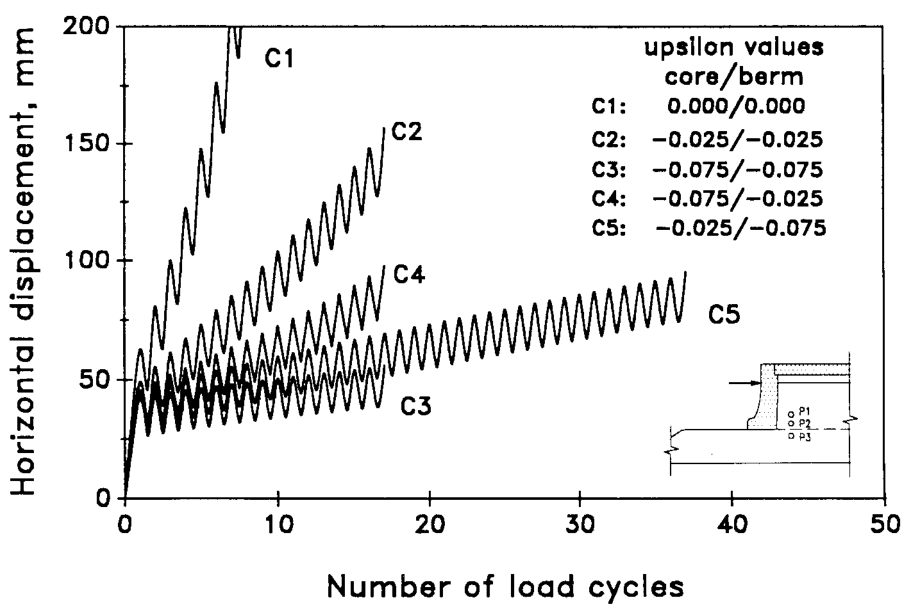


FIG. 14. Horizontal displacement vs. number of cycles for cases C1–C5.

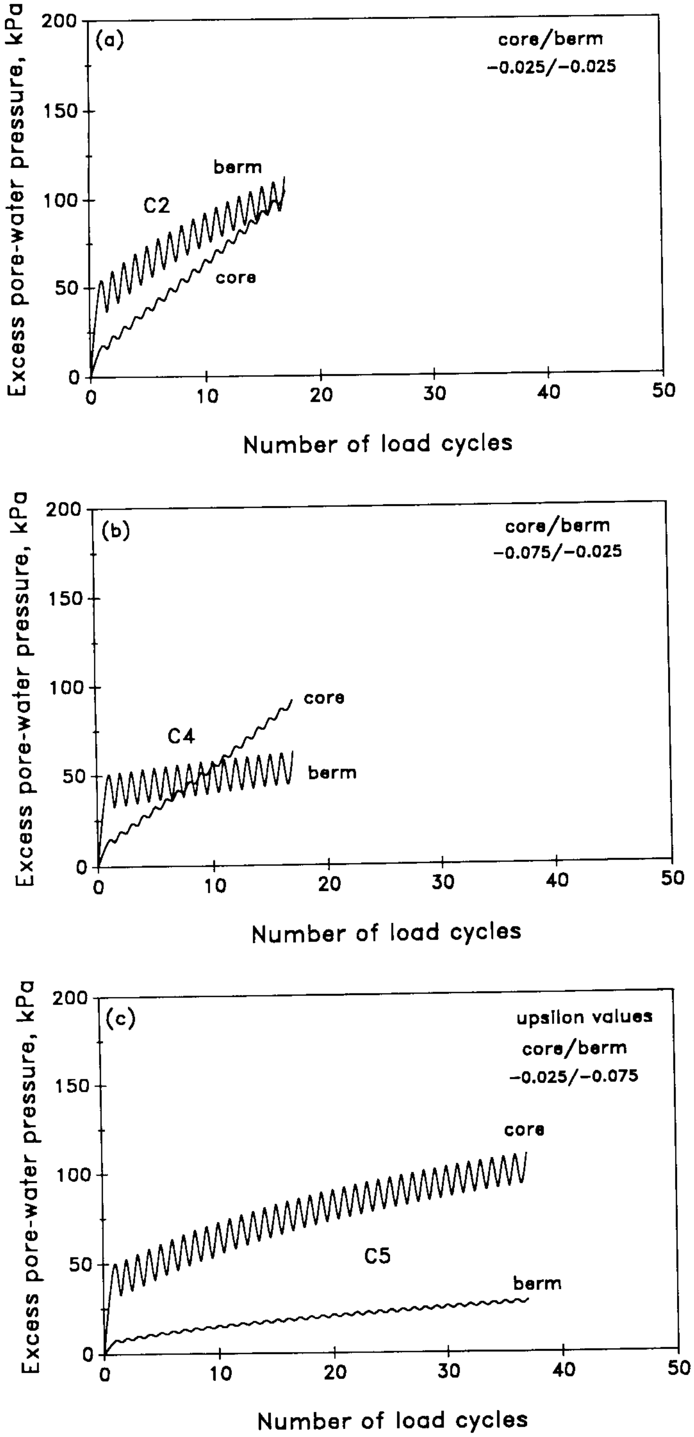


FIG. 15. Pore-water pressure vs. number of cycles for cases C2 (*a*), C4 (*b*), and C5 (*c*).

loose berm combined with a dense core. The diagram demonstrates clearly that there is but little benefit in densifying the core sand beyond the density of the berm sand.

Figure 13*a* shows the computed excess pore-water pressure in the berm (location P3). As would be expected, compared to a loose state of the berm and core (case S2 with upsilon values of 0.00), densifying the berm sand reduces excess pore pressure in the berm sand (S7 versus S2), while densifying the core sand beyond that of the berm sand has only little reducing effect on the pore pressure in the berm (S6 versus S2). Furthermore, densifying the core becomes useful when the density of the core does not exceed that of the berm (S4 versus S7 as compared to S6 versus S2).

Similarly, Fig. 13*b* shows the results of excess pore-water pressure computed for the core (location P2). The diagram shows that densifying the core sand reduces the excess pore pressure in the core. However, the diagram also shows the not so trivial result that densifying the berm sand increases the excess pore pressure in the core. Notice, however, as indicated in Fig. 12, that densifying the berm still increases the overall resistance of the structure.

To study more closely the stability of the structure to monotonic loading, the excess pore pressures need to be compared in more than two locations. However, monotonic loading has little relevance to the actual loading conditions, which are cyclic. Therefore, the structure must be analyzed taking into account the actual cyclic nature of the ice loading as demonstrated in the following.

Analysis results: cyclic loading

Table 5 includes five cases of cyclic loading, cases C1–C5. The first three cases consider sand having equal upsilon values in the core and berm of 0.00, −0.025, and −0.075, respectively. Cases C4 and C5 have dissimilar upsilon values in the berm and core. Case C4 is assigned upsilon values of −0.025 and −0.075 for the sand in the berm and core, respectively, and case C5 has these values in reverse.

According to Sanderson (1988), the maximum cyclic ice loading during the winter of 1985–86 varied between a crest value of about 500 MN (5 MN/m) and a trough value of about 300 MN (3 MN/m). Details of this loading event, such as total number of cycles, average magnitude of loads, and cycle periods, are not available. For the analyses of cyclic loading as applied to cases C1–C5, the imposed load was increased from 0 to 5 MN/m then cycled between 3 and 5 MN/m boundaries for a total of about 20–40 cycles.

Figure 14 shows the horizontal displacement of the structure at point Q (location of load application) versus the number of load cycles for the five cases of cyclic loading. The computation results indicate that case C1, having the loosest sand, would not have been stable for the imposed ice loading.

Of course, increasing the density of both the berm and the core sand improves the stability. As in the monotonic loading, increasing the density of the core beyond that of the berm provides little benefit. That is, Fig. 14 suggests that the stability is governed more by the density of the berm than by the core is also true in the case of cyclic loading.

Case C1 is unstable and so are, possibly, also cases C2 and C4, as indicated by the progressive increase in accumulated horizontal displacement with increasing number of load cycles. On the other hand, cases C3 and C5 are stable as indicated by the declining rate of increase of horizontal displacement with number of load cycles.

Case C1 corresponds to case S2 and case C2 corresponds to Case S3 of the preceding monotonic analyses. The monotonic analysis of these cases (at a factor of safety of at least 1.6) does not indicate instability of the structure, however. This is alarming because it means that the static response of the structure cannot be used to design for the stability of the structure for cyclic loading.

The suspected instability of cases C2 and C4 can be verified by studying excess pore pressure versus number of cycles as illustrated in Figs. 15*a* and 15*b*. Figure 15*a* shows large excess pore pressures that are stabilizing in the core,

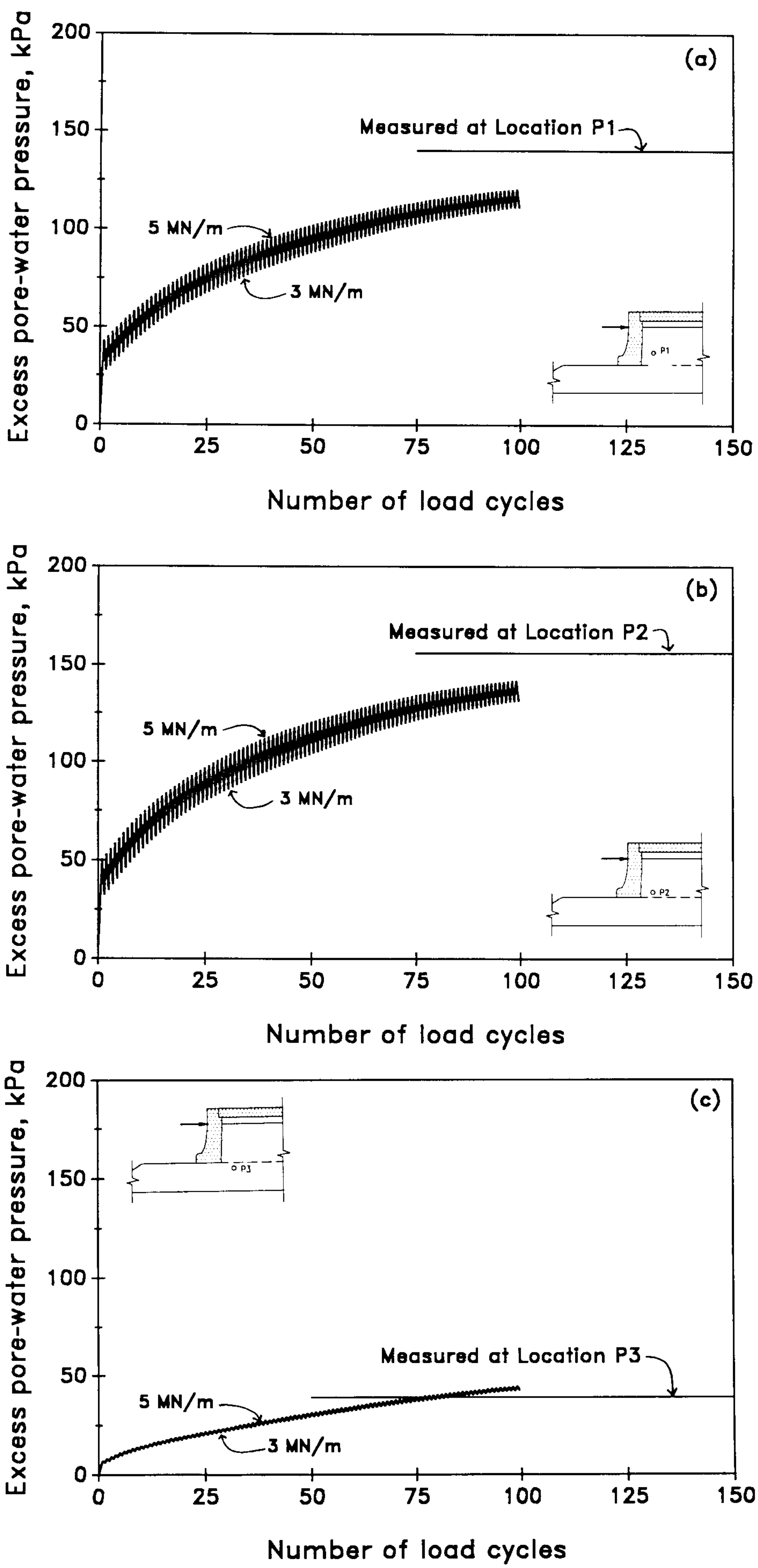


FIG. 16. Computed and measured pore-water pressure for case C5 at (*a*) location P1, (*b*) location P2, and (*c*) location P3.

but not in the berm. As shown in Fig. 15*b*, densifying the core sand has a beneficial effect on the pore pressures in the core, but little or no effect in the berm.

Figure 15*c* shows the behavior of case C5 and the effect of densifying the berm rather than the core—the reverse of case C6. A comparison with Fig. 15*a* indicates that the pore pressures in the core have reduced. The most beneficial effect, however, lies with the berm behavior, however, which clearly indicates a stabilizing trend.

Notice that no negative excess pore-water pressure is generated at the berm location similar to that computed for the static analysis (Figs. 11*a* and 13*a*). This is because the strains induced during the cyclic loading are far smaller than those required to cause dilatancy of the sand.

Pore pressures and horizontal displacement of actual case

The numerical observation obtained for case C5 (Fig. 15*c*) that the excess pore-water pressure at the core location is higher than at the berm location agrees with field observations made during the ice-loading event of the structure (Jefferies and Wright 1988; Hicks and Smith 1988). (As mentioned, case C5 represents the conditions of the actual structure.) Maximum excess pore-water pressures of about 140, 156, and 40 kPa were measured at locations P1, P2, and P3, respectively.

To further demonstrate the ability of the model and program to correctly simulate actual behavior, the analysis of case 5 is repeated letting the number of cycles between 5 and 3 MN/m continue to 100 cycles. Figures 16*a*–16*c* show the pore-water pressure generated numerically at locations P1, P2, and P3. For reference, the figures also include the mentioned measured maximum pore-water pressure at these locations. In the core, locations P1 and P2, the computed pore-water pressure approaches the measured values with increasing number of load cycles. In the berm, location P3, the computed pore-water pressure slightly exceeds the measured value. (For full agreement at location P3, the analysis would have to be repeated with a slightly denser sand in the berm.)

The computed pore-water pressure indicates that no significant additional increase in computed pore-water pressure takes place after the first 100 loading cycles. This is indicated by the stabilizing trend of the pore-water pressure with increasing number of load cycles.

The accumulated horizontal displacement due to cyclic loading is still relatively small. For example, case 5 was analyzed for 100 load cycles, and the computed horizontal displacement was about 170 mm. The numerical computation indicated that continuous accumulation of horizontal displacement with declining rate takes place with increasing number of load cycles. It is anticipated that no significant increase in horizontal displacement will take place for load cycles beyond about 200 cycles. The actual measurement results with regard to the accumulated horizontal displacement are not available to the authors for comparison.

Conclusions

The results of the simulations of the laboratory tests verify that the bounding-surface plasticity model using the *single* set of nine model parameters as established from the results of drained triaxial tests is capable of corectly modeling the behavior of the sand not only during the drained tests but also during the undrained tests.

The AGAC finite-element program is capable of producing simulations of the behavior of the Molikpaq structure for both monotonic and cyclic loading. The results show that the development of excess pore pressures is very important to the Molikpaq stability, as also argued by Jefferies and Wright (1988). The excess pore pressures cause a reduction of effective stress within the core and the berm that could cause the structure to collapse. The sustainable cyclic loading levels are far smaller than the sustainable monotonic load levels.

During an extreme ice loading event with several cycles of loads from 3 and 5 MN/m, excess pore-water pressures were measured in the core and the berm to values of 140 and 156 kPa at locations P1 and P2, respectively, and 40 kPa at location P3. The numerical analysis was carried to 100 cycles of loading and the computed excess pore-water pressures are in good agreement with the measured values.

The results of the study demonstrate the importance of analyzing the stability of the structure for cyclic loading rather than relying on conclusions derived from static analysis.

Altaee, A. 1991. Finite element implementation, validation, and deep foundation application of a bounding-surface plasticity model. Ph.D. thesis, Department of Civil Engineering, University of Ottawa, Ottawa.

Altaee, A., Evgin, E., and Fellenius, B.H. 1992*a*. Axial load transfer for piles in sand. II. Numerical analysis. Canadian Geotechnical Journal, **29**: 21–30.

Altaee, A., Evgin, E., and Fellenius, B.H. 1992*b*. Finite element validation of a bounding surface plasticity model. Computers and Structures, **42**: 825–832.

Altaee, A., Fellenius, B.H., and Evgin, E. 1993. Axial load transfer for piles in sand and the critical depth. Canadian Geotechnical Journal, **30**: 445–463.

Bardet, J.P. 1986. Bounding surface plasticity model for sands. ASCE Journal of the Engineering Mechanics Division, **112**(EM11): 1198–1217.

Been, K., and Jefferies, M. G. 1985. A state parameter for sands. Géotechnique, **35**: 99–112.

Herrmann, L.R., Kaliakin, A.M., Shen, C.K., Mish, K.D., and Zhu, Z.Y. 1986. Numerical implementation of plasticity model for cohesive soils. ASCE Journal of the Engineering Mechanics Division, **113**(EM4): 500–519.

Hicks, M.A., and Smith, I.M. 1988. Class A prediction of Arctic caisson performance. Géotechnique, **38**: 589–612.

Jefferies, M.G., and Wright, W.H. 1988. Dynamic response of Molikpaq to ice-structure interaction. *In* Proceedings of the 7th International Conference on Offshore Mechanics and Arctic Engineering, Vol. 6. American Society of Mechanical Engineers, New York, pp. 201–220.

Jefferies, M.G., Stewart, H.R., Thomson, R.A.A., and Rogers, B.T. 1985. Molikpaq deployment at Tarsiut P-45. *In* Proceedings of the Conference Arctic'85, Civil Engineering in the Arctic Offshore, San Francisco. *Edited by* F.L. Bennett and J.L. Machemehl. American Society of Civil Engineers, New York, pp. 1–27.

Roscoe, K.H., and Poorooshasb, H. 1963. A fundamental principle of similarity in model test for earth pressure problems. *In* Proceedings of the 2nd Asian Regional Conference on Soil Mechanics, Bangkok, Vol. 1. pp. 134–140.

Sanderson, T.J.O. 1988. Ice mechanics, risks to offshore structures. Graham & Trotman, London.

Sangrey, D.A., Henkel, D.J., and Espig, M.I. 1969. The effective stress response of a saturated clay soil to repeated loading. Canadian Geotechnical Journal, **6**: 241–252.

Zienkiewicz, O.C. 1977. The finite element method. 3rd ed. McGraw-Hill, London.

DISCUSSION

Modeling the performance of the Molikpaq: Discussion[1]

Michael Jefferies

In their paper on the Molikpaq, A. Altaee and B.H. Fellenius have missed several points about the measured data. The effect is to raise questions about some of the insight gained from their analysis. These topics will be briefly discussed leading to a suggestion that a different conclusion should be drawn from that suggested by Altaee and Fellenius.

For those readers unfamiliar with this case history, it should be noted that it is one of the few full-scale liquefaction events with detailed geotechnical data and response measurements; arguably, it is the only high stress level liquefaction event measured to date. The potential significance of the information was recognized immediately the load event occurred, and considerable research was carried out on the case history under the sponsorship of the oil industry using the vehicle of a "joint industry project" (JIP). Such projects are normally restricted by confidentiality agreements to protect the commercial interests of the funding companies, and this is the case with the Molikpaq case history. Because of the downturn in oil prices, and consequent affect on the time scale for Arctic development, the confidentiality agreement for the Molikpaq research runs through to 1997. Nevertheless, the JIP participants have been quite generous, and quite a bit of the geotechnical information has been published (it is the ice-loading studies that are the principal commercial interest). The paper has not used all the presently available public-domain information.

The principal missing information is in Jefferies et al. (1988) and Jefferies et al. (1989). This additional public-domain information reveals the following key discrepancies in the paper:

(1) loading was not undrained;
(2) all 18 fast-response piezometers were in the core;
(3) only 2 out of 18 piezometers show liquefaction; and
(4) more than 700 significant load cycles were experienced.

Some comments on these facts follow.

That the loading was partially drained is readily seen from published pore pressure records. Jefferies et al. (1988) presented the piezometric time history of the maximally responding piezometer, which was at mid height of the loaded side (P1 on the Authors' figures). This data establishes that the total duration of "significant" cyclic loading was about 12 min, that this piezometer sustained a liquefied condition for about 6 min, and that the subsequent time for 95% dissipation of this excess pressure was also about 6 min. Comparison of this dissipation time with the duration of loading shows that the assumption of undrained analysis is untenable.

The lower piezometer shown as "P3" in the paper was not in the berm. It was lower than the base of the caisson, but this was because the center of the berm was lower than the leveled perimeter used to set the structure down; contours of the constructed and leveled berm show the core region was mostly at about 20 m below mean sea level (BMSL) with about 15% of the area (actually in the northeast quadrant) at about 21 m BMSL. Although the significance of this error is much reduced because of the need to use a fully coupled solution, it appears inappropriate for Altaee and Fellenius to present this fit as obviously caused by the changed material properties of the denser berm. Yes, the berm may have sucked in water by dilation, but was there sufficient dilation given the small horizontal displacements? And, it should also be noted that the Molikpaq dewatering system was pumping during the liquefaction event and that the core dewatering intakes were towards the base of the caisson and arguably well within the influence zone for the P3.

Although attention has concentrated on the loaded side of the caisson, the whole point of cyclic analysis, such as presented by the Authors, is to understand what is going on so that confident decisions can be made. In this context, it is simply not good enough to fit a small portion of the data and then claim full insight. There may be other solutions, models, and idealizations that also fit. In terms of both intellectual curiosity and practical engineering significance the most interesting question is: Why did liquefaction not spread across the entire core? For it is only full liquefaction that could lead to catastrophic loss of the structure. There were 18 piezometers with high-speed data acquisition measuring the core response, and only 2 of these (P1 and P2 in the notation of the paper) showed liquefaction; on the opposite side, and even the center of the core, only small to negligible excess pore-water pressures were measured. We need to see solutions that predict the entire piezometric pattern and that specifically show why liquefaction was so limited, for it is only such solutions that will begin to give us the required insight. Perhaps the Authors could present their results for the opposite face of the caisson for the three piezometers located in the

Received May 5, 1995. Accepted June 20, 1995.

M. Jefferies. Golder Associates (U.K.) Ltd., Landmere Lane, Edwalton, Nottingham NG12 4DG, England.

[1] Paper by A. Altaee and B.H. Fellenius. 1994. Canadian Geotechnical Journal, **31**: 649–660.

same relative position as P1–P3? It would certainly contribute to the subject if they would comment on the extent to which their computed excess pore-water pressure varies across the core and to what extent does the measured low excess pore-water pressures on the opposite face constrain the parameter choices for the core sand properties.

Some changes in assessed properties are also required because of the duration of loading. Unlike an earthquake, during phase-locked crushing as experienced by the Molikpaq on 12 April, 1986, each ice-load cycle was similar to the next so that each cycle is significant; accordingly there should be no averaging or other factoring down of the number of cycles. The number of significant load cycles experienced, then, is about 12 min at 1 Hz, which equates to about 700. This an order of magnitude more significant load cycles than used in the Authors' analysis and requires further consideration in viewing the results of their numerics. Perhaps there is compensating errors at play here: the apparently over-conservative assumption of undrained conditions has been balanced by the use of an understated severity of cyclic loading.

So what changes do the above discrepancies have on the insight suggested by the paper?

Most of the points raised above relate to parameter selection and modeling details, although the understanding as to why liquefaction was localized may eventually prove of considerable practical significance. But, there is one issue in particular where the current information suggests a very different conclusion from that drawn by the Authors. The issue is the type of cyclic analysis.

As pointed out in the paper, both the Authors and the Discusser agree on the need for cyclic analysis (conveniently ignoring, for the moment, the inability of current ice mechanics to deliver a cyclic loading function for arbitrary ice conditions). However, it is not self-evident that an undrained analysis is appropriate or sufficient. When assessing the assumption of undrained conditions (the fundamental assumption of the Authors' analyses) the first reaction might be to see this as a conservative assumption, and it may well be conservative in terms of the rate at which pore pressure is generated by the most stressed part of the sand. But, it is by no means obviously conservative when considering the structure as a whole. The ability of a sand to withstand load under cyclic conditions depends upon dilation offsetting the cyclic effects; clearly if water migrates one can no longer rely on the shakedown-caused excess pore-water pressure being more than offset by subsequent dilation (at least in the case of a sand initially denser than critical). If anything is learnt from the Molikpaq case history, it should be that the undrained assumption (usually encountered in the context of the steady state school) is simply wrong at field scale for cyclic loading of sand.

Hopefully, fully coupled analyses will reveal just how unsafe is the undrained assumption. It is for this reason in particular that the use of coupled analyses must not be seen as a intellectual nicety adding refinement to the type of calculations presented by the Authors. Partially drained analyses (particularly those including localization) are essential if we are to have a proper insight into liquefaction for practical engineering. Thus, the Authors have contributed to the liquefaction literature, but what they present is not a sufficient framework. There is some way to go before we have the tools needed for confident practical engineering of liquefaction.

References

Jefferies, M.G., Rogers, B.T., Stewart, H.R., Shinde, S., James, D., and Williams-Fitzpatrick, S. 1988. Island construction in the Canadian Beaufort Sea. Proceedings, ASCE Speciality Conference on Hydraulic Fill Structures, Fort Collins. ASCE Geotechnical Special Publication 21.

Jefferies, M.G., Hardy, M.D., and Rogers, B.T. 1989. Instrumentation and monitoring of an offshore Arctic platform. Proceedings, Conference on Geotechnical Instrumentation in Civil Engineering Projects, Institution of Civil Engineers. Thomas Telford Ltd., London.

Modeling the performance of the Molikpaq: Reply[1]

Ameir Altaee and Bengt H. Fellenius

It is unfortunate that data from the Molikpaq have only been made available in a few papers, each offering, as it were, but "a few raisins from the cake." When the Authors wrote the paper, data were available by Jefferies et al. (1985), Hicks and Smith (1988), Jefferies and Wright (1988), and Sanderson (1988). By combining information gleaned from these papers, the Authors could compile the following information:

(1) six values of measured *maximum* excess pore pressure (from two sets of three piezometers, one at the ice impact side and one at the opposite side of the structure);
(2) the ice loading event consisted of almost 1000 cycles at about 1 Hz frequency and the *maximum* total ice load was about 500 MN;
(3) soil parameters necessary for performing the finite element analysis by means of the Advanced Geotechnical Analysis Code (AGAC) program (this involved getting soil data from more than one of the referenced papers and calibrating the input soil parameters against the diagrammatic presentations of laboratory soil test results available in one of the papers);
(4) the excess pore pressure measurements show the behavior to be essentially undrained (stated by Hicks and Smith 1988);
(5) the maximum measured movement of the structure was about 80 mm.

The Authors were at the time not aware of the two conference presentations (Jefferies et al. 1988, 1990) stated by the Discusser to be a source of "additional public-domain information." However, on now reviewing these two references, they are found to contain no new information related to the subject of the paper. Each gives the response of "a typical piezometer in the core" showing 10 cycles of ice loading and pore pressure during a 10 s duration (both force and pore pressure scales are unquantified), plus parts of the overall pore pressure response during one loading event of 20 min. The information contained in these two conference presentations is already available in the references mentioned in the paper.

The Authors notice that the Discusser now provides additional "raisins." One, the piezometers P1 and P2 located near the loaded side actually indicated that liquefaction occurred; and, two, the piezometers placed in the center of the structure registered only "negligible" pore pressure changes (about the same negligible values as the piezometers placed at the opposing side to the loading side?). Moreover, the Discusser reveals that dewatering by pumping occurred near piezometer P3, which could have affected the pore pressure measured at this location.

To respond to the Discusser, the Authors find it necessary to summarize his many statements, as listed below. The quotations marks identify verbatim quotations.

(1) The Authors have used only a small portion of the currently public-domain information, yet they "fit a small portion of the data and then claim full insight."
(2) The public-domain data show that the loading was "not undrained."
(3) All fast-response piezometers were located in the core.
(4) The "whole point of the cyclic analysis, such as presented by the Authors, is to understand what is going on so that confident decisions can be made."
(5) The Authors should have shown the effect of 700 cycles of loading, as actually experienced, rather than limit the presentation to 100 cycles, as this would have resulted in a significantly different computed pore pressure response.
(6) The Authors should present "their results for the opposite face of the caisson for the three piezometers located in the same relative position as P1–P3" (i.e., those used in the paper).
(7) The Authors should comment on the extent of the computed pore pressure variation across the core.
(8) Partially drained (coupled) cyclic analysis is essential.

Statement 1 is incorrect. The Authors used all the available information, scarce as it was and is, and nowhere in the paper do the authors claim full insight of any kind. Further, the Authors have not fitted the analyses to the data. The analytical procedures are clearly described in the paper.

Statement 2 is in conflict with the full journal paper by Hicks and Smith (1988), who also had access to measurement results. They state emphatically that the pore pressures follow an undrained behavior. The Discusser's conflicting statement that the pore pressures did not is not convincing, and the available information does not indicate that the drainage played any major role in the pore pressure development during the short-duration ice-loading event. This notwithstanding that it is logical for at least some drainage to have taken place. Thus, that the measured excess pore pressures are slightly smaller than the Authors computed values can be considered an indication of that some drainage occurred at piezometer P3. Of course, the pumping now mentioned by the Discusser can also be the cause. It must be noted, however, that the absolute

Received June 16, 1995. Accepted June 20, 1995.

A. Altaee and B.H. Fellenius. Urkkada Technology Ltd., 1010 Polytek Street, Unit 6, Ottawa, ON K1J 9H8, Canada, *and* Department of Civil Engineering, University of Ottawa, ON K1N 6N5, Canada.

[1] Discussion by M. Jefferies. This issue. Canadian Geotechnical Journal, **32**: 922–923.

Fig. 1. Computed pore-water pressure at locations P1 and P2.

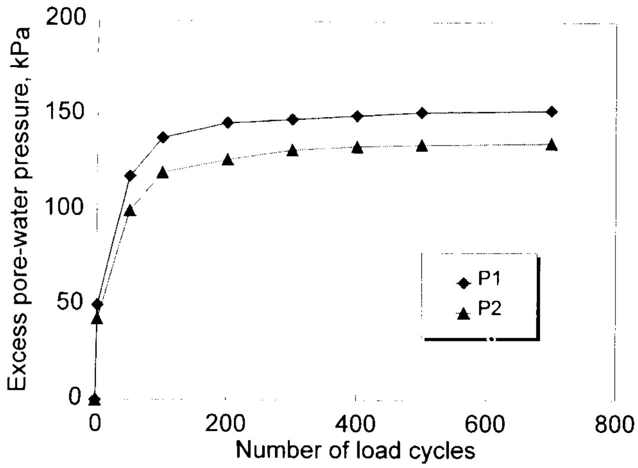
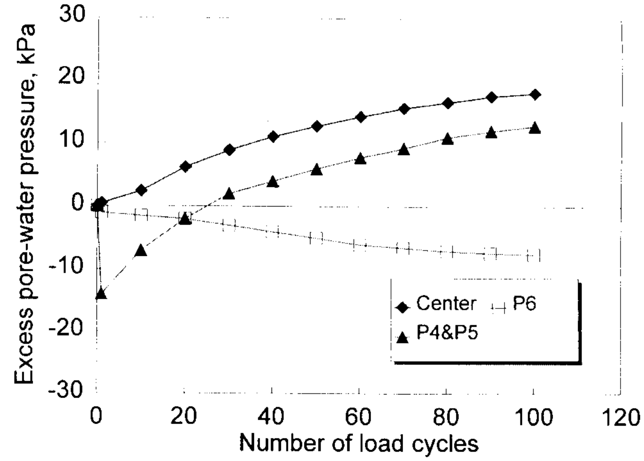


Fig. 2. Computed pore-water pressure at locations P4, P5, P6, and core center.



magnitude is of less importance than the trends resulting from the various cases of densities presented in the paper.

Statement 3 implies that the paper (Hicks and Smith 1988) that shows the location of piezometer P3 to be in the berm is in error. However, whether or not the piezometer tip is in sand similar to the berm sand or to the core sand has very little influence on the actual pore pressures as well as on the analysis results. The pore pressure at a specific piezometer location is a function of the material and behavior of the larger volume of soil surrounding the piezometer, and less of the material at that specific location.

Statement 4 is incorrect, and how the Discusser can read this out of the paper is odd. The main point, though by no means the whole point, of the analysis presented in the paper was to show the importance of considering cyclic response in analysing and designing a sand island structure and to show that the analysis method and the AGAC program used by the Authors indeed is capable of determining a realistic response, including liquefaction. Indeed, the Authors computed also a movement very close to the reported magnitude of movement. In contrast, some others, who published results of static analyses, presented calculated values of movement that were one to two orders of magnitude greater than the actual movement.

Statement 5 is addressed in the paper. The analysis shows that once a maximum pore pressure was reached, which occurred after about 100 cycles at the location of the ice impact, the effective stress approaches zero (liquefaction occurs) and no further pore pressure increase is obtained (at this location). To respond to the Discusser's suggestion, the Authors have extended the analysis to 700 cycles (all parameters are identical to those used in the analysis reported in the paper). The results are presented in Fig. 1 showing that more than 90% of the maximum pore-water pressure occurred during the first 100 cycles. Of course, the liquefaction will spread out progressively from the location of the ice-raft impact. Therefore, full liquefaction will require a larger number of cycles at locations further away from the impact location. In the extreme, the structure will fail.

Statement 6 is easy to respond to. To limit the paper, the Authors concentrated on the location of the high values of measured pore pressure. However, the same analysis from which the information in the paper was obtained provides the pore pressures at the side opposite to the impact side. The computed pore pressures at piezometers P4, P5, and P6 (same relative positions as piezometers P1, P2, and P3 at the impact side) are shown in Fig. 2. The values measured (at unknown time) for P4 and P5 are about −10 kPa, and the measured value is about −2 kPa at P6. Both measured and computed values are considered to be negligible.

Statement 7, if taken to suggest that the Authors should provide the computed pore pressure response across the entire structure, goes beyond the objectives of the paper. (Though, this could be done and the results be shown as contour lines, for example). The Authors prefer to limit their answer to showing computed values only when they can be compared with measured values. For instance, Fig. 2 includes also the pore pressure response computed for a piezometer at the centre of the structure (placed at same depth as the piezometer P1 at the impact side). Indeed, compared with the pore pressures computed and measured near the loading side, the response shows negligible increase, as stated by the Discusser to have been the actual case.

Statement 8 is of course correct per se. However, to perform a coupled analysis, time becomes a very important parameter. Data must be available showing not just a few maximum values, but covering the entire loading event. The Authors have the means to perform the coupled analysis. However, unless the complete ice-loading development is made available for input to the analysis, the results would not be relevant to the objective of comparing computed results with measured data. Of course, should field data of the actual ice-loading history become available to support a coupled analysis, the Authors will be glad to perform the analysis to compute the response for comparison with measured values.

The pore pressure development shown in the paper indicates that liquefaction would occur at the location of piezometers P1 and P2 for a structure built as the actual case. It is with no little satisfaction that the Authors now learn from the Discusser that liquefaction indeed occurred at these two piezometers locations.