CANLEX full-scale experiment and modelling


Abstract: A major aim of the Canadian Liquefaction Experiment (CANLEX) was to verify analysis procedures for predicting liquefaction phenomena. Towards this purpose, two loading events were carried out: a field event comprising a clay embankment built over a loose sand foundation layer, and a centrifuge test performed on a model of a sand embankment structure. Both the field event and the centrifuge model were planned so as to induce a static liquefaction failure and were instrumented to observe their response in terms of displacement and pore pressure. The fundamental mechanical characteristics of the foundation layer were determined from laboratory element tests (triaxial and simple shear). These tests formed the basis for the stress–strain modelling used in the numerical analyses. Two fundamentally different modelling techniques were used. One involved a fully coupled plasticity model, and the other involved a model based on a collapse-surface approach. The model and prototype structures were then analyzed and the predicted results in terms of displacements and pore pressures were compared with the measured values. The results from both approaches were found to be in reasonable agreement with the measurements, provided allowance was made for direction of loading and drainage effects were accounted for.

Key words: liquefaction, field experiment, embankment, centrifuge model, elastic–plastic model.

Introduction

The Canadian Liquefaction Experiment (CANLEX) included three major aspects: characterization of sand to evaluate its liquefaction characteristics; development and calibration of numerical models to predict liquefaction response; and a full-scale liquefaction event at the Syncrude site near Fort McMurray, Alberta, to validate predictive ability.

Liquefaction characterization was carried out at a number of sites. This involved in situ testing as described by Wride et al. (2000) and laboratory testing on both undisturbed and reconstituted samples as described by Vaid et al. (1995a, 1995b).

Liquefaction of sand is associated with the tendency for the sand skeleton to contract under shearing. When such contraction is prevented or curtailed by the presence of a low-compressibility fluid such as water in the pores which cannot escape, a large reduction in effective stress due to a rise in pore pressure occurs, leading to a significant loss in strength and stiffness. This behaviour can occur under static or cyclic loading conditions and is referred to as liquefaction. Liquefaction induced by static or monotonic loading can lead to flow slides in loose saturated sand slopes. These can be triggered by small additional static loading or increases in groundwater levels. Many failures of slopes, particularly those involved with mine waste disposal, have occurred in this manner. Cyclic liquefaction is commonly triggered by earthquake loading and can lead either to a flow slide if the soil is loose, or to lateral spreading in denser
sands. Much of the damage in the San Francisco area that occurred during the Loma Prieta earthquake of 1989 was due to cyclic liquefaction, as was the great damage to the port facilities at Kobe during the 1995 earthquake.

Flow slides and large movements are controlled by the stress–strain relation and strength of the soil under undrained loading. A major part of the CANLEX project involved the determination of these properties from in situ and laboratory testing. These properties could then be used in analyses to predict stability and deformations of soil structures, allowing safe design of new structures and retrofit of existing structures.

The field event provided a test for numerical modellers to predict liquefaction movements under ideal conditions. To help with the design of the field event and to help calibrate the numerical analyses, a number of centrifuge model tests on sand embankment structures including one similar to the field structure were carried out at the Centre for Cold Ocean Resources and Engineering (C-CORE) as described by Philipps and Byrne (1998). The challenge then was as follows: given the soil properties, could numerical modellers predict the response of the centrifuge test and, more importantly, could they predict the response of the field test embankment?

Field event

The intent of the full-scale event was to statically trigger a liquefaction flow slide by rapidly loading a loose saturated sand deposit. This involved the construction of a test embankment over a loose layer. The event is described in detail by Byrne et al. (1995a) and Robertson et al. (1996). A brief description will be given here and is illustrated in Fig. 1a.

An abandoned borrow pit at the Syncrude Canada Ltd. site (J-pit) was used to carry out the field event. The foundation sand was placed hydraulically into standing water up to elevation 318 m and was referred to as beach below water sand (BBW sand). A level platform was then formed at elevation 321 m by placing tailings sand above the water and was referred to as beach above water sand (BAW sand). A clay dyke 8 m high with side slopes of 2:5:1 (horizontal to vertical) was constructed slowly over the tailings so as to allow drainage of the sand to occur. A 10 m high compacted sand cell containment structure was then constructed to form an enclosure. Rapid loading was brought about by pumping tailings (contained sand) behind the clay dyke.

The site was characterized as described by Hofmann et al. (1996a). This involved in situ testing (seismic cone penetration testing, standard penetration testing with energy measurements, self-boring pressuremeter testing, and geophysical logging), undisturbed sampling using in situ ground freezing, and laboratory testing of the samples. The target sand layer was found to be very loose with an average $(N_i)_\text{lo} = 3.5$. Laboratory testing of undisturbed samples showed this very loose material to be generally strain softening in both simple shear and extension loading, but generally strain hardening in compression loading.

Five lines of instrumentation (Fig. 1b) were installed under the test embankment extending to about 30 m beyond the toe of the clay dyke, as described by Hofmann et al. (1996b). Each instrumentation line contained piezometers, tilt meters, piezo-settlement points, and surface-settlement points. In addition, a remote optical survey system was used to monitor surface movements.

The field event began on 18 September 1995. Water and tailings were poured into the cell behind the clay embankment. The water level was raised to a height of 7.5 m and the sand was placed to a height of 7 m. Loading took about 36 h.

As a result of rapid loading, the clay dyke experienced a maximum displacement of 0.054 m at the toe, with an average movement of 0.020 m (Natarajan et al. 1996). Hence, displacements due to loading were small. In terms of pore pressures, the highest piezometric head (6.7 m) was measured under the upstream slope of the clay embankment. Beneath the crest of the dam the average piezometric head was 3.5 m (Hofmann et al. 1996a). It is evident that these excess pore pressures were not sufficient to trigger liquefaction in the foundation layer underneath the dyke and beyond its toe.

Centrifuge tests

A number of centrifuge tests were carried out to investigate the static flow liquefaction potential of embankments supported on very loose saturated sand layers. The purpose of these tests was (i) to serve as models to help design the field event, and (ii) to provide a data base from which to calibrate the numerical models. These tests are described in detail by Phillips and Byrne (1993, 1994). Test 1 (Phillips and Byrne 1994) was chosen for detailed description and analysis because it resembled the proposed field event most closely.

A profile of the model test embankment is shown in Fig. 2. During the initial loading stage, this structure was subjected to an acceleration field of 50g. The acceleration field was brought about in five increments of 10g each, allowing for pore-pressure dissipation between increments. Canola oil was chosen as the pore fluid to delay the rate of pore-pressure dissipation. Delaying the rate of dissipation was necessary to promote a near undrained type of response during the subsequent loading stage. Under an acceleration field of 50g, the model structure corresponds to a 10 m depth of foundation layer supporting a 5 m high embankment. After self-weight compression, the sand had a relative density of 29% (Phillips and Byrne 1994).

While in flight and under this acceleration field, a pressure load of 60 kPa, corresponding to about 3.5 m of soil, was applied on the crest of the slope, followed 2.6 s later by another load of 60 kPa. Thus the loading was equivalent to the rapid application of 7 m of soil over a 10 m target layer. The model was instrumented, with five pore-pressure transducers, PPT1–PPT5, and two displacement transducers, LDT18 and LDT19, as shown in Fig. 2.

Figure 3 shows the response of the centrifuge model to loading. Pore-pressure transducers PPT2 and PPT3, which were located directly beneath the load, registered sharp increases in pore pressures followed by rapid drops under each of the loading increments (Fig. 3a). However, a very different pore-pressure response can be noted below the toe of the slope, as registered by PPT4. Here, the pore pressure rose rapidly but remained almost unchanged with application of the second load increment.

© 2000 NRC Canada
Fig. 1. (a) Plan and cross-sectional views of liquefaction event site. BAW, beach above water; BBW, beach below water. (b) Plan view of instrumentation lines through embankment.
Fig. 2. Initial centrifuge model and instrumentation configuration. All dimensions and vertical scale in mm. PPT, pore-pressure transducer; LDT, longitudinal displacement transducer.

Fig. 3. Centrifuge model response to loading: (a) pore pressure during loading, (b) surface settlement during loading, (c) deformation field due to first surcharge of 60 kPa.
The vertical movements recorded by LDT18 (on the crest) and LDT19 (in the toe area) are shown in Fig. 3b. LDT18 indicated that the crest settled 4 mm under the first load and continued to deform upon application of the second load, up to 6.7 mm. Scaled to 50g, these settlements correspond to 0.2 and 0.33 m, respectively. In contrast, LDT19 shows upward movement of about 3 mm, or 0.15 m scaled to 50g.

The pattern of displacements after the first load was applied but prior to application of the second load is shown in Fig. 3c. The pattern is deep seated and consistent with a liquefaction failure.

### Liquefaction response of Syncrude sand

Tests carried out by Vaid et al. (1995a) showed that the undrained response of loose Syncrude sand to first-time loading is strongly influenced by the direction of loading relative to the direction of deposition (vertical), i.e., the angle of the major principal stress, $\sigma_1$, as shown in Fig. 4. Triaxial compression ($\alpha_o = 0^\circ$) as compared with triaxial extension ($\alpha_o = 90^\circ$) is shown in Fig. 4a, which shows that resistance in compression is strain hardening, whereas in extension it is strongly strain softening. Simple shear loading, in which $\alpha_o$ varies from 0 to about 45° during the test, is neither hardening nor softening, and its ratio of undrained shear strength to vertical consolidation stress ($s_u/\sigma_{vc}$) is approximately 0.2. Hollow cylinder tests (Fig. 4c) show the response for $\alpha_o$ values of 0, 30, 45, and 90° and illustrate the enormous effect of the direction of principal stress on undrained shear response. For $\alpha_o = 0^\circ$, the undrained shear strength ratio ($s_u/p' v$ in which $p' v$ is the effective mean normal stress) is approximately equal to 0.3, whereas for $\alpha_o = 90^\circ$, $s_u/p' v = 0.06$, i.e., a factor of five difference in strength. Since the direction of major principal stress will range from extension to simple shear to compression in both the model and field events, it is very important that this be considered in an analysis.

### Numerical modelling

In stress deformation analyses of soil structures, equilibrium and compatibility of every element in the domain should be satisfied as well as the boundary conditions, and these constraints must be fulfilled for the appropriate element stress–strain relations. Equilibrium, compatibility, and the boundary conditions can be reasonably satisfied using finite element or finite difference formulations. Many computer codes exist for carrying out such calculations, e.g., PISA (Chan 1997) and FLAC (Cundall 1995). The difficulty arises in modelling the complex stress–strain relations of soil, particularly under undrained conditions when liquefaction can occur, as was shown in Fig. 4. The commonly used elastic–plastic models in which the soil is assumed to be isotropic and elastic below the failure envelope and plastic at failure will not simulate the strain softening observed in monotonic liquefaction. Two basic approaches were taken to model the fundamental element liquefaction response.

The University of British Columbia (UBC) approach

A fundamental plasticity approach that captures the behaviour of the sand skeleton including its plastic contraction upon shearing was developed at The University of British Columbia. The approach is described by Byrne et al. (1995b) and Puebla et al. (1996, 1997) and will be referred to here as the UBC approach. When the volume of the sand skeleton is constrained, either by the presence of water in the pores that cannot escape or by constrained boundaries such as in the simple shear test, plastic contraction upon shearing can lead to a large reduction in effective stress and a strain softening shear stress – shear strain response. The parameters for such a model can be obtained from drained shear tests in which shear and volumetric strains are measured or from undrained tests in which pore pressures are measured in place of volumetric strains. The model can be used to solve the drained or undrained response as well as the coupled stress–flow problem. In addition, the plastic component of the model is anisotropic, so the marked difference between the undrained response observed in triaxial compression and that observed in extension can be captured. This was considered to be very important for both the centrifuge and the field event. The model parameters (Table 1) were selected to give reasonable agreement with the test data over a wide range of consolidation stresses and stress paths. The predicted response for isotropically consolidated ($p'_o = 100$ kPa) triaxial compression and extension tests and simple shear tests (vertical consolidation stress, $\sigma_{vc} = 100$ kPa) are compared with laboratory test data in Fig. 5 and show good agreement. In this approach, strain softening and an undrained collapse line are predicted for extension loading rather than specified and result from the softer

### Table 1. Model parameters used in the UBC approach to simulate undrained behaviour of loose Syncrude sand.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Undrained triaxial and simple shear tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic bulk modulus number, $k_B$</td>
<td>300</td>
</tr>
<tr>
<td>Elastic shear modulus number, $k_G$</td>
<td>300</td>
</tr>
<tr>
<td>Elastic bulk modulus exponent, $m_e$</td>
<td>0.5</td>
</tr>
<tr>
<td>Elastic shear modulus exponent, $n_e$</td>
<td>45</td>
</tr>
<tr>
<td>Plastic shear modulus number, $k_G^p$</td>
<td>310</td>
</tr>
<tr>
<td>Plastic shear modulus exponent, $n_p$</td>
<td>0.67</td>
</tr>
<tr>
<td>Peak friction angle, $\phi_f$</td>
<td>33.7°</td>
</tr>
<tr>
<td>Constant volume friction angle, $\phi_{cv}$</td>
<td>33.0°</td>
</tr>
<tr>
<td>Failure ratio, $R_f$</td>
<td>0.95</td>
</tr>
<tr>
<td>Factor of anisotropic plastic response, $F$</td>
<td>0.32</td>
</tr>
</tbody>
</table>

© 2000 NRC Canada
plastic modulus used in extension. The FLAC computer code was used with this model.

University of Alberta (U of A) collapse-surface approach

In this approach, the undrained strain softening response of loose sand depicted in Fig. 6 is captured directly in the model. The approach, referred to here as the U of A approach, was developed at the University of Alberta by W.H. Gu and D.H. Chan and is described in more detail by Chan et al. (1995) and Cathro and Gu (1995). The initial response is captured using an elastic formulation with conventional Skempton $A$ and $B$ undrained parameters to generate pore pressures. Strain softening is captured by specifying a collapse surface in terms of plastic
parameters of cohesion and friction. By assuming that the residual strength, \( s_u \), is related to the consolidation pressure, \( p_0' \), the collapse surface can be moved up or down to reflect changes in void ratio arising from drainage. In addition, both the residual strength and the slope of the collapse surface can be varied with the direction of the major principal stress to capture the response characteristics shown in Fig. 4, i.e., the collapse surface could also depend on the direction of loading as well as the void ratio or effective consolidation pressure. The strain softening is captured by specifying strength as a function of shear strain. The pore pressures \( (u) \) on the collapse surface are computed by subtracting the predicted effective stresses \( (\sigma'') \) from the total stresses \( (\sigma) \), i.e., \( u = \sigma - \sigma'' \).

**Analysis of centrifuge tests**

The University of British Columbia

The gradual buildup of stress during centrifuge “swing up” was not modelled. The effective stresses due to self weight under an acceleration field of 50g were computed by assuming a fully drained response and using the UBC stress–strain model described in detail by Byrne et al.
The densities of the sand were based on the laboratory data reported by Phillips and Byrne (1993). Two different densities of the sand were used: a submerged density of 0.96 t/m$^3$ below the oil table, and a total density of 1.87 t/m$^3$ above the oil table.

The response to rapid loading was modelled as undrained. The undrained response of the foundation sand observed in laboratory element tests over a range of stress states and paths was captured with the stress–strain model using the parameters listed in Table 1. The slope was then modelled as a collection of such elements. The boundary conditions on the model were as follows: zero horizontal displacements on the vertical boundaries, and zero vertical displacements on the bottom horizontal boundary, as shown in Fig. 7. The water table (oil table) was horizontal and coincided with the surface of the target layer.

The rapid loading was simulated by a pressure of 60 kPa, increased shortly afterwards to 120 kPa, applied at the crest of the slope under undrained conditions. A very stiff and weightless structural beam member connected the loaded nodes simulating the effect of the rigid weight. A comparison between the measured and predicted pore pressures and displacements at the monitored points in the centrifuge test is presented in Table 2, which shows reasonable agreement between measured and predicted values.

The displacement pattern upon application of the first weight is shown in Fig. 8. It compares reasonably well with the pattern observed in the actual test (Fig. 3c) from the first weight. Both patterns show that the region directly beneath the loading plate was mainly sheared in compression mode, whereas in regions below the slope and beyond the toe shearing was mainly in simple shear and extension modes. A strain-hardening type of response was predicted by the numerical model for elements making up the “compression” region, whereas a strain-softening type of response was predicted for elements in the “simple shear” and “extension” regions.

### University of Alberta

Two separate analyses using the collapse-surface approach were carried out, one by Chan et al. (1995), and one by Cathro and Gu (1995). These analyses are described in more detail by the authors.
**Chan et al. (1995) analysis**

The centrifuge experiments were modelled using the finite element program PISA. The program incorporates a collapse-surface plasticity model for modelling undrained deformation of sand. A two-dimensional plane strain finite element mesh was used to model the prototype embankment. In performing the analysis, an initial stress field was incorporated using the switch-on-gravity technique. This is similar to the process of applying the centrifugal loading on the soil. The application of the loading by the drop weight was modelled using a uniform pressure boundary. The numerical model assumes that the soil deforms in an undrained manner with pore-water pressures determined from the pore-pressure parameters. Stresses in the soil are determined and then compared with the collapse surface, which is a function of the undrained shear strength of the material. If the stresses result in a collapsible state, collapse analyses will be performed and the stresses and pore pressures will be redistributed to obtain a new equilibrium state. If the zone of the collapsible material is sufficiently large, overall failure may occur.

The material parameters used in the analysis are shown in Table 3. These parameters are determined based on triaxial test results. The pore-pressure parameters were back-calculated from the equation of Henkel (1960) using observed pore-pressure response during swing up in the centrifuge experiments.

Since it is known that the undrained shear strength increases with an increase in mean effective stress, a constant $s_u/p_o'$ ratio of 0.2 was used in the analysis to incorporate the variable $s_u$. The numerical model indicated failure of the slope after the application of the first weight. The calculated distorted mesh at failure is shown in Fig. 9 and the surface profile compares reasonably well with the observed shape shown in Fig. 3c.

**Cathro and Gu (1995) analysis**

Cathro and Gu (1995) extended the collapse-surface approach for liquefaction analysis to account for volume change due to pore-pressure dissipation using the consolidation theory of Biot (1941). With this approach, drainage during swing up of the centrifuge model was simulated. In addition, the loading stage was modelled by considering undrained conditions during application of the first 60 kPa surcharge load. The pore pressures generated during this process were then allowed to dissipate until the application of the second 60 kPa surcharge load, causing an additional undrained rise in pore-water pressure. Pore pressures from both loadings were then allowed to dissipate. The predicted pore pressures at the locations of PPT2 and PPT3 agreed
Table 3. Material parameters used in the U of A finite element liquefaction analysis of the centrifuge test.

<table>
<thead>
<tr>
<th>$\gamma_s$ (kN/m$^3$)</th>
<th>$E$ (kPa)</th>
<th>$\nu$</th>
<th>$\Phi_{ss}$ ($^\circ$)</th>
<th>$\Phi_{us}$ ($^\circ$)</th>
<th>$\alpha$</th>
<th>$s_{ur}$</th>
<th>$\alpha_s$</th>
<th>$\alpha_m$</th>
<th>$\alpha$</th>
<th>$s_i$/$\rho_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.00</td>
<td>9000</td>
<td>0.3</td>
<td>33</td>
<td>24</td>
<td>0.001</td>
<td>0</td>
<td>5</td>
<td>0.3</td>
<td>0.2</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Note: $\gamma_s$, submerged unit weight in Canola oil; $E$, Young’s modulus; $\nu$, Poisson’s ratio; $\Phi_{ss}$, angle of steady state line; $\Phi_{us}$, angle of collapse surface; $\alpha$, post-peak factor in hyperbolic softening model (Chan and Morgenstern 1989); $A_s$ and $A_m$, pore-pressure parameters $A$ in the liquefaction model (Gu et al. 1993); $\alpha_s$, Henkel’s pore-pressure parameter ($B = 0.97$) was used in all analyses; $s_{ur}$, undrained shear strength at steady state; $\rho_o$, mean normal effective stress.

Table 4. Material parameters used in the UBC liquefaction analysis of the field event.

<table>
<thead>
<tr>
<th>Material</th>
<th>$B$ (kPa)</th>
<th>$G$ (kPa)</th>
<th>$c$ (kPa)</th>
<th>$\phi$ ($^\circ$)</th>
<th>$k_x$ (m/s)</th>
<th>$k_y$ (m/s)</th>
<th>$\rho$ (t/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay dyke</td>
<td>30 000</td>
<td>30 000</td>
<td>30</td>
<td>0</td>
<td>$1 \times 10^{-4}$</td>
<td>$1 \times 10^{-4}$</td>
<td>2</td>
</tr>
<tr>
<td>Sand cell</td>
<td>30 000</td>
<td>30 000</td>
<td>0</td>
<td>36</td>
<td>$8 \times 10^{-6}$</td>
<td>$5 \times 10^{-7}$</td>
<td>2.1</td>
</tr>
<tr>
<td>Slurry</td>
<td>10 000</td>
<td>100</td>
<td>2</td>
<td>0</td>
<td>$1 \times 10^{-4}$</td>
<td>$1 \times 10^{-4}$</td>
<td>2.2</td>
</tr>
<tr>
<td>Soft clay</td>
<td>10 000</td>
<td>2 000</td>
<td>10</td>
<td>0</td>
<td>$1 \times 10^{-11}$</td>
<td>$1 \times 10^{-11}$</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Note: $B$, bulk modulus; $G$, shear modulus; $c$, cohesion; $\phi$, friction angle; $k_x$, horizontal hydraulic conductivity; $k_y$, vertical hydraulic conductivity; $\rho$, density.

remarkably well with the measured values (Fig. 10c). At the location of PPT4, the predicted rate of pore-pressure dissipation was too fast compared with the measurements (Fig. 10b). In addition, a greater peak value of pore pressure under the second load increment was predicted. The vector plot of final displacements obtained in the numerical analysis is shown in Fig. 10c and shows good agreement with the test results.

Analysis of field event

The three modelling groups had simulated the field event prior to the event taking place. These results were presented at a CANLEX meeting held at Fort McMurray on 7 and 8 September 1995, 10 days before the event took place. Based on the centrifuge modelling and the site investigation carried out to that time, both U of A groups predicted that the test embankment would fail when loaded. The UBC group predicted that it would not. The main reason for the difference at that time was the incorporation of the stress-path or anisotropic effect on strength and stiffness in the UBC model, i.e., the effect shown in Fig. 4. The results presented here are based on papers dated after the event had occurred.

The University of British Columbia

The field event was modelled in the analysis by simulating the construction and loading conditions. It was assumed that all materials behaved in a fully drained manner during placement, except the contained sand (Fig. 1), which was assumed to behave as a heavy fluid. The clay dyke and compacted cell sand were modelled as elastic—perfectly plastic materials and their parameters were based on Duncan et al. (1980) and Byrne et al. (1987). These parameters are listed in Table 4. The parameters for the stress—strain model used to simulate the target foundation sand response were obtained by back analysis of laboratory test data from Phillips and Byrne (1994) and Vaid et al. (1995b) and are listed in Table 1.

The boundary conditions on the numerical model were as follows: zero horizontal displacements on the vertical boundaries at $x = 0$ and 300 m (Fig. 1), and zero horizontal and vertical displacements on the bottom boundary (natural ground surface, Fig. 1). The water table was horizontal and coincided with the surface of the target layer (BBW sand) before the load was applied. After loading, the water level was assumed to coincide with the crest of the embankment on the upstream side of the clay dyke and with the surface of the sand layer at elevation 321 m on the downstream side (Fig. 1a).

The rapid placement of the contained sand comprised the “loading.” Under this loading, two different assumptions were made regarding the drainage conditions of the target sand: (i) undrained, and (ii) coupled stress—flow. This led to two separate sets of analyses and results.

The predicted displacement pattern and contours of total pore-water pressure obtained by numerical simulation under undrained conditions are presented in Fig. 11. The maximum predicted horizontal displacement occurred in the toe area and had a magnitude of about 10 cm. The pattern of predicted displacements shown in Fig. 11a indicated that significant displacements would occur to the full depth beneath the upstream side of the dyke and would drop rapidly with depth beneath the downstream slope of the embankment. Beyond the toe of the dyke, the predicted movements were small. The contours of predicted total pore-water pressure presented in Fig. 11b show large pore pressures beneath the applied load, but beyond the downstream slope of the dyke the effects of the load on pore pressure were not significant. Lack of excess pore-water pressure beyond the toe of the dyke was the likely reason for the small displacements in this region.

The coupled stress—flow was simulated by allowing the generated pore pressures to dissipate during and after load application. Once the contained sand (heavy fluid) was placed rapidly, its surface became a drainage boundary with pore pressure equal to zero. The surface of the tailings downstream of the toe was also specified as a drainage boundary with pore pressure equal to zero. The vertical boundaries at $x = 0$ and 300 m (Fig. 1) and the bottom boundary (natural ground surface) were considered impermeable. Seepage flow into all the zones within the boundaries was considered. The hydraulic conductivity of the foundation sand was based on...
field experience at the Syncrude site which indicated that the horizontal hydraulic conductivity \( (k_x) \) was 15 times greater than the vertical hydraulic conductivity \( (k_y) \). The values used for the analysis were \( k_x = 7.5 \times 10^{-6} \) m/s and \( k_y = 5.0 \times 10^{-7} \) m/s. The hydraulic conductivity of the clay dyke was assumed to be \( k = 1.0 \times 10^{-8} \) m/s (Cedergren 1989), and the contained sand (heavy fluid) was considered to have a hydraulic conductivity \( k = 1.0 \times 10^{-4} \) m/s (Cedergren 1989).

Results of the coupled stress–flow analysis are shown in Fig. 12. The maximum predicted horizontal displacement occurred in the toe area and had a magnitude of about 5 cm. The pattern of displacements is presented in Fig. 12a and the contours of predicted total pore-water pressure after the steady state seepage condition was reached are shown in Fig. 12b. The patterns of displacements and pore pressure are similar to those obtained for the undrained case; however, the values are smaller. Allowing for dissipation of the excess pore pressures leads to predicted displacements of about half the values predicted for undrained conditions.

The measured and predicted values of excess pore-water pressure for both undrained and coupled stress–flow analyses are compared in Table 5. The values obtained for the undrained case are lower than the measured values.

The predicted and measured displacements in the toe area are also shown in Table 5. The prediction from the coupled stress–flow analysis agrees well with the measurements, whereas the undrained prediction was twice that of the measured value.

University of Alberta

The field event was modelled using the finite element computer program PISA and the collapse-surface approach described earlier. A certain depth of the foundation was included in the finite element mesh, and the mesh was extended laterally far enough to minimize boundary effects. Initial in situ stresses in the tailings sands were generated using the switch-on-gravity technique. The material parameters used in the analysis are shown in Table 6. These parameters were determined based on field and laboratory tests. Strength and deformation parameters for the tailings were estimated from laboratory test results and the average cone resistance obtained during site characterization. Based on the site-characterization information, the unit weight and strength parameters for the top 3 m of the tailings were assumed to be slightly higher. Pore-pressure parameters used in the undrained analysis are the same as the values used in the U of A analyses of the centrifuge tests.

**Modelling of embankment construction**

The construction of the clay embankment over the submerged tailings was simulated in three ways representing drained, undrained, and partially drained conditions. A drained analysis was performed to simulate slow construction of the embankment. This may not represent the real conditions due to the fact that a long time is required for pore pressures to completely dissipate. However, the drained condition represents one extreme case in the evaluation of the stability of the embankment. The finite element results of this analysis indicated that no significant deformations would occur during embankment construction. An undrained effective stress analysis was carried out to model the rapid construction of the embankment over the tailings with no dissipation of excess pore-water pressures in any of the construction lifts. This represents the other extreme condition that may be experienced by the tailings sands. Results of the analysis showed that failure would occur at an early stage of embankment construction. Failure was indicated by nonconvergence of the numerical solutions.

The above two cases represent extreme conditions that may be induced in the sand during embankment construction. Realistically, partial drainage (or complete drainage, depending on the permeability of the tailings sand and the rate of construction) will occur. To simulate partial dissipation of pore pressures between each lift of fill placement, an undrained analysis was carried out with excess pore pressures in the sand being reduced between lifts. The pore pressure was reduced to sufficiently low values that would allow stable construction of the embankment. Piezometric head measurements taken during embankment construction confirmed that dissipation of pore pressures in the foundation sand did occur between lifts. Results of the partially drained analysis are plotted in Fig. 14, which represents the maximum allowable excess pore-water pressures at the mid-depth of the tailings sands (under the centreline of the embankment) to avoid instability during the embankment construction. This figure was provided to Syncrude Canada Ltd. as a guide for construction control during the embankment construction. During construction, except for two piezometers in line 5, the responses of each piezometer located in the tailings sands did not exceed the recommended values given in Fig. 14. This resulted in safe construction of the clay embankment.
Modelling of loading due to contained tailings

A number of analyses were carried out to investigate the effect of contained tailings on the stability of the embankment. In these analyses, contained tailings were assumed to be placed after all excess pore-water pressures in the tailings sands due to the embankment construction had dissipated. This assumption was made based on the observed response of the piezometers, which indicated relatively fast dissipation of the excess pore water pressure in the tailings sands.

The analyses are categorized into three main cases:

Case 1: The contained tailings were assumed to be placed rapidly into the area between the embankment and the compacted sand dyke so that the tailings imposed undrained loadings on the structure. No dissipation of excess pore-water pressures was assumed during filling.

Case 2: Steady state seepage in the tailings sands was assumed during filling of the contained tailings. In this case, the tailings sands and the embankment were loaded under drained conditions.

Case 3: The contained tailings were assumed to be placed in three lifts, with reduction of excess pore pressures in the tailings sands between lifts. This analysis was performed to study the effect of the loading rate on the stability of the embankment.

In situ testing, including cone penetration tests (CPT) and standard penetration tests (SPT), before the embankment construction indicated that the tailings sands were generally loose, although highly nonhomogeneous. Based on the in situ and laboratory test results, the average undrained steady state strength ratio of the tailings sands ($s_u/p_o'$) before the embankment construction was estimated to be about 0.1. Ratios of $s_u/p_o'$ in the range of 0.1–0.2 were used in the analysis to examine embankment sensitivity to this parameter. To investigate the effect of higher strengths in the top 3 m of
the tailings sand, analyses were carried out with higher $s_u/p'_{o}$ ratios for the top 3 m of the tailings. In addition, an increase in the $s_u/p'_{o}$ ratio of the tailings sands under the clay embankment was expected due to consolidation. In this regard, the undrained strength, $s_u$, beneath the embankment would increase under constant $s_u/p'_{o}$ because $p'$ increases as the embankment is placed and dissipation occurs. It was felt that additional densification resulting in an increase in $s_u/p'_{o}$ would occur due to vibration associated with compaction of the overlying dyke. To examine this effect for the tailings under the embankment, cases 1, 2, and 3 were analyzed using different $s_u/p'_{o}$ ratios, as summarized in Table 7.

Results of analyses by the University of Alberta

Based on the information summarized in Table 7, the major findings of the liquefaction analyses were as follows:

(1) Rapid filling of the contained tailings behind the clay embankment, with no dissipation of excess pore-water pressures during filling, would trigger liquefaction flow failure in the tailings sands if the average undrained steady state strength ratio of the tailings sands did not exceed 0.1.

(2) Liquefaction flow failure (if any) would occur as a result of undrained loading before steady state seepage is established in the tailings sands.

(3) Results from the case 3 analysis indicated that if loading was not fast enough, a liquefaction failure would not occur. Rapid loading of the contained tailings is essential in order to trigger a flow failure. Therefore it was recommended that the contained tailings should be filled continuously and be completed in about 24 h to minimize dissipation of excess pore-water pressures in the tailings sands. Figure 15 shows contours of computed excess pore-water pressures at the end of loading for the case 3 analysis.

Although the loading rate was relatively fast from a feasibility point of view, it was not fast enough to induce high enough excess pore pressures in the tailings to reach a state of instability. It is important that the rate of loading be studied together with the rate of dissipation of excess pore-water pressure. Piezometer measurements during construction of the clay embankment showed that the tailings sands are highly permeable. Almost half of the induced pore-water pressures were dissipated after 12 h from the beginning of lift construction, and piezometer response during loading behind the embankment indicated that a large amount of excess pore pressure had been dissipated during loading. Results of the case 3 analysis showed that if the loading was not fast enough (i.e., most of the excess pore pressure was allowed to dissipate during loading), no liquefaction flow failure would occur. Table 8 compares the observed and calculated changes in water level for case 3 in line 1 of the instrumented section; the piezometer locations in line 1 are shown in Fig. 16. The instrumented section along line 1 showed the highest pore-pressure response during the experiment.

Figure 17 compares the measured excess pore-water pressure at piezometer P18A during loading with the calculated
excess pore-water pressure for cases 1, 2, and 3. As indicated, the computed excess pore-water pressure from case 1, which assumes fully undrained conditions, is much higher than the observed excess pore-water pressure. However, the excess pore-water pressures calculated from cases 2 and 3 were close to the measured values. These results indicate that the assumption of fully undrained loading was not realistic due to the relatively high permeability of the tailings sands.

(4) If the $s_u/p_0^c$ ratio of the tailings sands under the embankment was increased to 0.2 due to construction of the clay embankment, failure would not occur.

(5) Figure 4 showed the strong dependency of strength on stress path. Based on these results, it is useful to investigate stress paths in the tailings sands during loading. Figure 18 shows contours of the angle ($\theta_w$) between the major principal stress ($s_1$) and the depositional (vertical) direction. As indicated, most of the tailings, including the tailings under the upstream side of the embankment, were sheared predominantly in compression. However, towards the downstream side, some regions were subjected to simple shear type loading and only the surface material near the toe was subjected to the extension mode of shearing. Therefore, it is reasonable to conclude that a rigorous liquefaction analysis based on the above undrained steady state strength ratios (0.3 for compression, and 0.06 for extension) most likely will result in nonfailure of the embankment.

Discussion of field event predictions

Two very different techniques were used by the modellers:

(1) The UBC approach is a fully coupled plasticity procedure that can capture the sand skeleton behaviour over a wide range of loading paths and drainage conditions, i.e., drained, undrained, and coupled stress–flow conditions. The model parameters are selected to give the best fit to laboratory element test data.

(2) The U of A approach is a procedure in which the undrained strength and slope of the collapse line are specified, together with a strain-softening function and a stress-redistribution procedure to assure that elements do not violate the collapse line and the steady state strength. The undrained steady state strength can be increased to account for consolidation or loading path, and the slope of the collapse line can be adjusted with the direction of loading.

Despite the differences in analysis procedures, the results were found to depend more on the assumption made about the stress–strain and strength of the soil than on the procedure itself.

© 2000 NRC Canada
Both penetration testing and results of undisturbed sampling and testing were available at this site. The laboratory testing showed that direction of loading was a key factor affecting the shear resistance of the target sand. If loaded in compression (direction of deposition), the sand was predominately strain hardening and had an average residual strength ratio \( s_u/p' \) » 0.3. When loaded in extension, the sand was predominately strain softening and had an average residual strength ratio \( s_u/p' \) » 0.06. The key question is what value of \( s_u/p' \) should be used in the modelling.

The UBC approach amounted to using a variable \( s_u/p' \) based on the laboratory test data, so zones of compression loading had \( s_u/p' \) = 0.3 and zones of extension loading in the toe area had \( s_u/p' \) = 0.06. However, because the strength in the compression zone upstream of the crest of the dyke was so high, the stresses in the potential strain-softening zone in the toe region never rose to their peak value and so did not strain soften. Hence, high pore pressures in this zone which could lead to a liquefaction failure did not develop.

The U of A modellers based their \( s_u/p' \) value on penetration-resistance values and the laboratory data, and while they did use a variation in \( s_u/p' \) to reflect looser and denser zones, they did not include the effect of the direction of loading. They concluded that, had they considered this, they would have predicted that the dyke would not have failed, even if the loading had been undrained.

Although the analyses carried out were quite different in principle, the key item affecting stability was the assumption concerning the residual strength. If \( s_u/p' \) = 0.1 was selected, which, based on current practice, would not be unduly conservative for such a loose sand, an undrained failure would have been predicted by both modelling groups. The field test showed that advantage can be taken of the large variation in residual strength with direction of loading when designing against a static liquefaction failure.

© 2000 NRC Canada
Table 6. Parameters of tailings sands used in the U of A analysis of the event.

<table>
<thead>
<tr>
<th>Material</th>
<th>$\gamma'$ (kN/m$^3$)</th>
<th>$E$ (kN/m$^2$)</th>
<th>$v$</th>
<th>$\Phi_{\text{s}}$ (°)</th>
<th>$\Phi_{\text{cs}}$ (°)</th>
<th>$A_\text{u}$</th>
<th>$A_\text{m}$</th>
<th>$s_u / p'_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tailings (lower 8 m)</td>
<td>8.95</td>
<td>14 000</td>
<td>0.49</td>
<td>34</td>
<td>24</td>
<td>0</td>
<td>5</td>
<td>0.2, 0.15, 0.1</td>
</tr>
<tr>
<td>Tailings (top 3 m)</td>
<td>9.4</td>
<td>18 000</td>
<td>0.49</td>
<td>36</td>
<td>25</td>
<td>0</td>
<td>5</td>
<td>0.2, 0.15, 0.1</td>
</tr>
</tbody>
</table>

Note: $\gamma'$, effective unit weight.

Fig. 13. Piezometer locations in liquefaction event site along line 2 instrumented section.

Fig. 14. U of A approach: maximum excess pore pressure at mid-depth of tailings sands (under centreline of embankment) for safe construction of dyke.
Table 7. Summary of the analyses of loading (U of A approach) due to contained tailings.

<table>
<thead>
<tr>
<th></th>
<th>$s_u/p_0'$</th>
<th>Case 1</th>
<th>Case 2</th>
<th>Case 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Constant $s_u/p_0'$</td>
<td>0.1</td>
<td>Failure</td>
<td>No failure</td>
<td>No failure</td>
</tr>
<tr>
<td>$s_u/p_0'$ of the top 3 m of tailings is higher*</td>
<td>&gt;0.1</td>
<td>No failure</td>
<td>No failure</td>
<td>No Failure</td>
</tr>
<tr>
<td>$s_u/p_0'$ of tailings right under embankment is higher†</td>
<td>&gt;0.2</td>
<td>No failure</td>
<td>No failure</td>
<td>—</td>
</tr>
</tbody>
</table>

*A value of 0.1 is used for $s_u/p_0'$ of the lower 8 m of tailings.
†A value of 0.1 is used for $s_u/p_0'$ of the tailings which are not located under the clay embankment.

Fig. 15. U of A approach: contours of excess pore pressures (kPa) at the end of loading in the case 3 analysis.

![Figure 15](image1)

Table 8. Comparison between the observed and calculated (U of A) changes in water level for case 3 in line 1 of the instrumented section.

<table>
<thead>
<tr>
<th></th>
<th>P18A</th>
<th>P13A</th>
<th>P13B</th>
<th>PF19C2</th>
<th>PO9A</th>
<th>PF19T2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Observed water level (m)</td>
<td>6.8</td>
<td>3.4</td>
<td>3.8</td>
<td>1.8</td>
<td>1.6</td>
<td>0.7</td>
</tr>
<tr>
<td>Calculated water level (m)</td>
<td>7.1</td>
<td>2.9</td>
<td>3.4</td>
<td>1.2</td>
<td>1.0</td>
<td>0.9</td>
</tr>
</tbody>
</table>

*Note: See Fig. 16 for location of piezometers.*

Fig. 16. Location of piezometer tips of the instrumented section along line 1. WT, water table.
The knowledge of stratigraphy and material properties were very well known at the Syncrude test site. In particular, (i) the investigation and testing of the target sand were extensive and much greater than is generally available in practice, and direct measures of stress–strain and strength were available from testing on undisturbed samples and indirect measures in the form of penetration resistance tests; (ii) the site investigation, laboratory testing, and field inspection all indicated that the target tailings sands were generally in a very loose state prior to the field loading event; and (iii) model centrifuge data using the same Syncrude sand were available. In light of this, how well did the modellers do?

The results of the analyses showed that the key item controlling the stability of the test embankment was the residual strength and its variation with direction of loading and drainage conditions. The UBC modelling group incorporated the direction of loading directly into their analyses and predicted results both before and after the event in good accord with field observations. They concluded that the dyke would not fail under undrained conditions and drainage effects would reduce movements by about 50%.

The U of A modellers initially based their residual strength values on penetration tests and state of practice de-
sign, and concluded the test embankment would fail. Based on the direct assessment of $s_u$ from laboratory tests and taking loading path into consideration, they later concluded that the test embankment should not fail under undrained conditions. They also concluded that drainage was significant during the event and would have a stabilizing effect.

In practice, engineers are required to give assurance that a slope will not fail or deform excessively. This is generally achieved using a limit equilibrium analysis approach. Appropriate soil shear strengths, and an appropriate factor of safety. This simple approach is not really applicable to the liquefaction problem described here because the soil has both peak and residual strengths, and the level of strain required to achieve these strengths must be considered. In addition, the strength depends very markedly on the direction of loading. Consequently, a higher level of analysis, which considers strain compatibility and equilibrium, is required to consider these factors. This was the approach taken by both modelling groups. The main difference was in the steady rate of loading. Both modelling groups used a normalized undrained strength ratio ($s_u/p_o$) approach; the main difference lay in whether the effect of direction of loading on $s_u/p_o$ was considered or not. The current practice is not to consider direction of loading effects and to use a residual strength based on penetration resistance and field experience, i.e., an indirect measure of strength. The analyses and field event presented here show that this approach may be unduly conservative because (i) direction of loading is important, (ii) the undrained strength in the compression zone may be very much higher than assumed, and (iii) drainage leading to increased strength may be significant. All of these factors were present at the Syncrude test site and prevented the occurrence of a failure where the current-practice approach would have predicted a failure.

Generally, the problem is to assure, with some degree of certainty, that a failure will not occur. In doing this, low values of shear strength based on penetration testing are usually used in analyses. On occasion, a direct measure of residual strength based on recovery and testing of undisturbed samples is undertaken, e.g., Duncan Dam (Byrne et al. 1994). At the Syncrude site, the problem was to predict the likelihood that failure would occur. In such a case, a high estimate of shear strength should be used in analyses, together with a factor of safety less than unity, to assure that failure will occur. Although the analyses that took stress-path effects and drainage into consideration indicated that a flow failure would not occur, no reasonable engineer would have advised that the dyke would not fail and that safety precautions did not need to be considered during filling of the test embankment.

The results of the field event and analyses indicate that steep slopes can be built over very loose sand, provided the rate of loading is such that consolidation can occur. Once built, the resistance to shock loading is greatly influenced by the direction of loading. In areas of vertical compression loading, the undrained strength ratio may be high (i.e., $s_u/p_o > 0.3$), even for very loose sand, whereas in regions of vertical extension (toe area) the strength ratio may be very low (i.e., $s_u/p_o < 0.06$). Considerable savings and (or) safer designs are possible through consideration of these aspects in the analysis and design of such structures.

**Acknowledgements**

This research was funded by operating grants from the Natural Sciences and Engineering Research Council of Canada (NSERC) and the Canadian Liquefaction Experiment (CANLEX), which is a project funded through a Collaborative Research and Development Grant from NSERC, B.C. Hydro, Hydro Quebec, Syncrude Canada Ltd., Suncor Inc., Kennecott Corporation, and Highland Valley Copper. The collaboration includes the geotechnical consultants, AGRA Earth and Environmental Ltd., Kohn-Crippen Consultants Ltd., Golder Associates Ltd., Thurbur Engineering Ltd., and EBA Engineering Consultants Ltd., as well as faculty and students from the University of Alberta, The University of British Columbia, Université Laval, and Carleton University. The authors wish to thank Mrs. K. Lamb for her typing and arrangement of the paper.

**References**


© 2000 NRC Canada


Phillips, R., and Byrne, P.M. 1993. CANLEX, Phase I, Activity 8e-Stage 1, centrifuge testing. Centre for Cold Ocean Resources Engineering, St. John’s, C-CORE Contract Report 93-C14.


