Liquefaction study for the Little Jackfish Dyke

Etude de liquéfaction concernant la digue Little Jack Fish

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SYNOPSIS: The liquefaction potential of the granular deposits at the Little Jackfish dyke site was studied using the standard penetrometer, the piezocone penetrometer, the cyclic simple shear device and the triaxial cell. The study shows that the top loose fine sand layer may liquefy for two locations while soils at other locations are not liquefiable. The approach based on the steady state concept indicates that a flow slide failure for the proposed dyke is unlikely.

INTRODUCTION

The Little Jackfish River is located north of Lake Nipigon, Ontario, Canada. The river is approximately 48 km long and drops about 67 m in elevation along its length. A study is currently being undertaken by Ontario Hydro to develop about 85% of the available hydraulic head in a single stage. The current proposal calls for a new hydroelectric generating station with an installed capacity of 132 MW. The design plant discharge will be 283 m^3/sec with a gross head of 57.4 m. In conjunction with this development a main dam, two block dams, 8 dykes and two canals will be constructed.

Following a preliminary investigation, it was recognized that potentially troublesome foundation conditions existed at the site for dyke 1. A detailed geotechnical study was subsequently carried out. An important aspect of the study, which is reported here, is an assessment of the liquefaction potential of the site.

SITE CONDITIONS

Dyke 1 will be constructed across a broad valley. For about 435 m along the proposed dyke alignment the terrain of the valley slopes gently and is partly swampy. The simplified subsoil conditions are shown in Figure 1. Briefly, the subsoil is composed of four main compressible layers of varying thickness. The top layer is muskeg, followed in sequence by silty fine sand, silt and varved clay. The dyke will be about 500 m in length and will have a height of up to about 20 m. It will consist of rockfill with an impervious core and zones of filter materials. In the lower parts of the valley the surface organic soil will be replaced with sand and gravel. Seepage through the silt and fine sand deposits will be controlled by a slurry trench cutoff constructed below the impervious core and extending to the underlying varved clay or bedrock.

There is little recorded information on significant historical earthquake events in northwest Ontario where the present project is located. There is, however, some detected activity within 750 km of the site. Such a large distance implies only minor site effects based on the attenuation equations of Hasegawa et al. (1981). The maximum calculated peak horizontal ground acceleration (a_{max}) is 0.019g which is related to an earthquake of 6.3 magnitude (M) which occurred on Keweenaw Peninsula in Michigan in 1906, about 350 km away from the present site. Based on the 1985 National Building Code of Canada, a_{max} at the site is 0.04g for a probability of exceedance of 10% in 50 years (return period ≈ 475 yrs.). This acceleration is associated with a frequency of 5 Hz. For design purposes, therefore, it is safe to assume M = 6.5 and a_{max} = 0.04g for this site.

DEFINITION OF LIQUEFACTION

There are two main concepts of liquefaction. The concept of Seed and Lee (1966) is based on cyclic load tests. They defined initial liquefaction as the state when the pore water pressure is equal to the applied confining stress. Complete liquefaction is defined as the complete loss of resistance over a wide strain amplitude. For loose sand, the development of initial liquefaction and complete liquefaction happen almost simultaneously. For dense sand, complete liquefaction may not take place following initial liquefaction. A strain criterion is then used by defining liquefaction as the state when the cyclic load produces a certain double strain amplitude, typically 5% to 20%. This condition is termed partial liquefaction by Seed and Lee (1966).

The other concept of liquefaction comes from the Harvard school of thought (Casagrande, 1976). Liquefaction is defined as the response of loose saturated sand when subject to monotonic, cyclic or dynamic loading that results in substantial loss of strength and, in extreme cases, leads to flow slides. This school of thought suggests that during liquefaction the mass of soil tends towards the steady state where the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress and

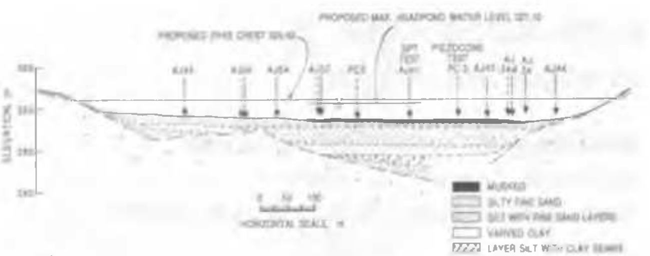


FIGURE 1

SIMPLIFIED CROSS SECTION OF SITE AND FIELD TEST LOCATION

constant rate of shear strain. Poulos et al. (1985) extended this concept and introduced the undrained steady state strength for assessing the likelihood of a flow slide.

TEST PROGRAM

Standard penetration tests (SPT) and piezocone penetration tests were conducted in the field at locations as shown in Figure 1. Sampling by means of a 76.2 mm diameter Osterberg sampler was carried out at AJ37. The SPT was carried out in accordance with common North American practice. The piezocone penetrometer with a 60°, 10 cm^2 area cone, measures tip resistance, sleeve friction and pore pressure. The pore pressure was measured through a 4 mm thick cylindrical porous stone located immediately above the cone.

In the laboratory, three types of tests were conducted on the silt samples taken from the site: 1) routine tests to determine the basic soil properties, 2) cyclic simple shear tests to study the liquefaction resistance and 3) load and strain rate controlled triaxial tests to study the steady state characteristics of the soil.

A Seiken cyclic simple shear apparatus was used to perform the consolidated, undrained cyclic simple shear tests on undisturbed and reconstituted samples. This apparatus has provisions for drainage control, application of a cell (horizontal) pressure and shearing under a back pressure. While shearing, the soil sample was constrained to a constant area by a stack of teflon-coated metal slip rings which have an inside diameter equal to the diameter of the rubber membrane and the sample. Consolidation for this test series was anisotropic with the horizontal consolidation stress equal to 0.45 of the vertical consolidation stress (σ'_{vo}). This ratio is approximately equal to the coefficient of earth pressure at rest (K_o) for the deposit.

Load controlled and strain rate controlled triaxial tests were also conducted on undisturbed and reconstituted samples. A special triaxial cell was built for the load controlled tests with the vertical load being applied by means of an air-piston. For the strain rate controlled tests, an ordinary triaxial cell was used.

Reconstituted samples were prepared by compacting moist soil particles (4% moisture content) in a mould mounted on the base of the simple shear or triaxial cell. The inside of the mould was lined with a rubber membrane for enclosing the sample throughout the test. After the sample was prepared, the mould was removed and the test cell was assembled and filled with enough water to cover the top cap on the sample. A thin layer of oil was added to prevent air from entering into the soil sample by permeating through the water and the membrane. The sample was saturated by passing carbon dioxide through it under a small cell pressure to displace the air in the void space and then, by passing distilled water to replace the carbon dioxide. Any trace of carbon dioxide that remained in the sample would dissolve in the pore water when the back pressure of at least 300 kPa was applied.

TEST RESULTS

The penetration resistance (N) in terms of number of blows per foot from the SPT and the tip resistance (q_c) in Kg/cm^2 from the piezocone tests may be compared as a ratio. For the silt deposit, the average ratio of q_c/N for each location is shown in Table I.

Table I. q_c/N ratio for silt deposit

Location	AJ34A	PC2	PC3	AJ37	AJ50
q_c/N	5.97	4.25	4.38	4.12	7.78

At locations AJ34A and AJ50, an artesian pressure was observed. This condition created an unstable bottom of the borehole for the SPT. Hence, the measured penetration resistance erred on the low side. The piezocone test results were not affected by the artesian pressure. Therefore, the resulting q_c/N ratio for these two holes is too high and is discarded for estimating the mean q_c/N value of the silt deposit for the whole site. This value is equal to 4.25.

Similar comparison on the fine sand layer shows that the mean value of q_c/N is about 5.

The response of a typical cyclic simple shear test on an undisturbed silt sample (Fig. 2) resembles that of a dense sand. In the initial stage of shearing, the pore pressure (u) steadily increased, the total vertical pressure for maintaining constant sample thickness

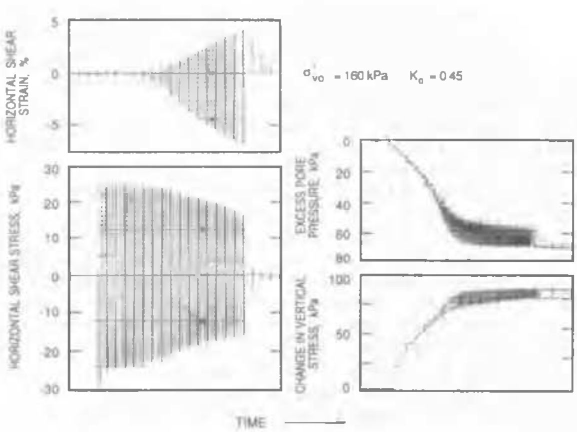
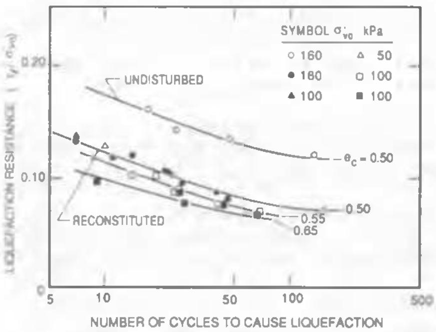


FIGURE 2

RESPONSE OF CYCLIC SIMPLE SHEAR ON AN UNDISTURBED SAMPLE decreased and the horizontal strain showed very small change. At u equal to about 70% of the horizontal consolidation pressure, the sample started to deform appreciably. The sample, however, did not collapse even at many cycles after 100% pore water pressure was developed.

The initial shearing response of a reconstituted sample was similar to that of the undisturbed sample. At 100% pore water pressure, however, the sample collapsed quickly.



e_c = Void ratio after consolidation

FIGURE 3

LIQUEFACTION RESISTANCE vs NUMBER OF CYCLES TO CAUSE LIQUEFACTION IN CYCLIC SIMPLE SHEAR TESTS.

The liquefaction resistance normalized by σ'_{vo} at the beginning of 100% pore pressure is summarized in Figure 3. This resistance is independent of σ'_{vo} within the studied pressure range. It decreases with increase in void ratio or in the number of cycles to cause liquefaction. The resistance of the undisturbed samples is significantly higher than that of the reconstituted ones.

The results of the triaxial tests for studying the steady state characteristics on reconstituted samples are shown in Figure 4. It appears that both the load controlled and the strain rate controlled tests yield the same steady state line and the same steady state strength line. Tests on undisturbed samples within the capability of the test cells (maximum confining pressure = 1 MPa) showed that the soil is dilative and the results are, therefore, not shown here.

LIQUEFACTION POTENTIAL

The liquefaction potential at the site can be assessed by comparing the normalized induced seismic stress (τ_h/σ'_{vo}) with the normalized liquefaction resistance (τ_ℓ/σ'_{vo}) of the deposit. The seismic stress can be estimated by the method of Seed et al. (1971):

$$\tau_h/\sigma'_{vo} = 0.65\,\frac{a_{max}}{g}\,\frac{\sigma_v}{\sigma'_{vo}}\,r_d \tag{1}$$

where a_{max} = maximum ground surface acceleration
g = gravitational acceleration
σ_v = total overburden pressure
r_d = a stress reduction factor=1-0.015 z
z = depth in m.

As noted earlier, the maximum bedrock acceleration at the site is equal to 0.04g for

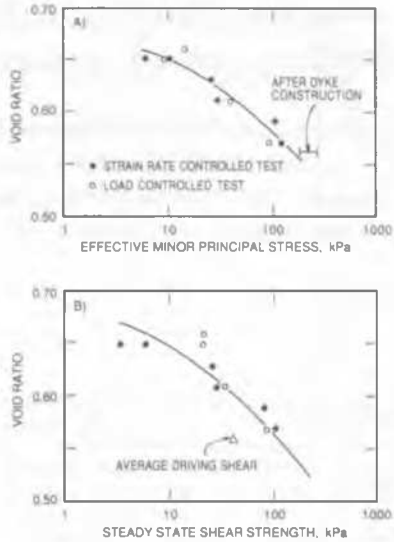


FIGURE 4

STEADY STATE CHARACTERISTICS

a return period of 475 yrs. and at a frequency of 5 Hz. It is unlikely that significant acceleration amplification will take place at this frequency during the wave transmission to the surface. Therefore, a_{max}/g is taken equal to .04.

The liquefaction resistance (τ_ℓ/σ'_{vo}) can be estimated from the piezocone data or the cyclic simple shear tests. For the piezocone data, q is first converted to N_1 using:

$$N_1 = C_N q_c/K \tag{2}$$

where C_N is a coefficient to account for the effective overburden pressure (Seed et al. 1983); N_1 is the modified penetration resistance and K = q_c/N. K was determined earlier as equal to 4.25 and 5 for the silt and the fine sand of this site, respectively.

The liquefaction resistance is evaluated using functional relationships suggested by Seed et al. (1983):

$$\tau_\ell/\sigma'_{vo} = f(N_1) \tag{3}$$

Different functions apply for sand and for silt. A typical τ_ℓ/σ'_{vo} profile for an earthquake magnitude (M) of 6.5 is shown in Figure 5.

The liquefaction resistance from the cyclic simple shear tests on undisturbed samples corresponding to M = 6.5 is also plotted on Figure 5. This resistance is about half of that from the piezocone tests. The discrepancy has been attributed to mechanical disturbance in the soil samples and the nonplastic nature of the silt (Zhu and Law, 1988). In this study the design liquefaction resistance of the silt layer was obtained from the piezocone data corrected by reduction to the value from the cyclic simple shear tests. Undoubtedly, this produces a conservative estimate of τ_ℓ/σ'_{vo} as the laboratory value includes the effects of mechanical softening due to disturbance.

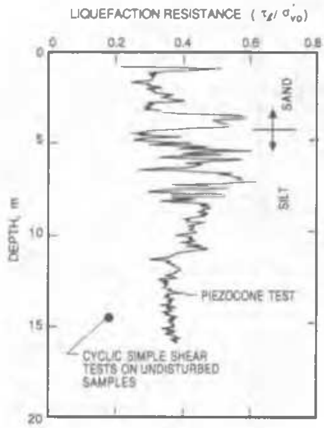


FIGURE 5

LIQUEFACTION RESISTANCE PROFILE AT AJ37 FOR EARTHQUAKE MAGNITUDE = 6.5

For the fine sand layer, τ_ℓ/σ'_{vo} from piezoncone data does not require correction because plasticity is no longer a factor.

In Figure 6 a corrected τ_ℓ/σ'_{vo} profile from piezocone data for the weakest location (AJ34A) is compared with the seismic stress for that which could be developed for an earthquake of M = 6.5. The comparison shows that the top 3 m of fine sand may liquefy under such conditions and, therefore, requires strengthening. A similar observation was also made for the top 2.5 m at PC2 while for all other locations liquefaction is unlikely.

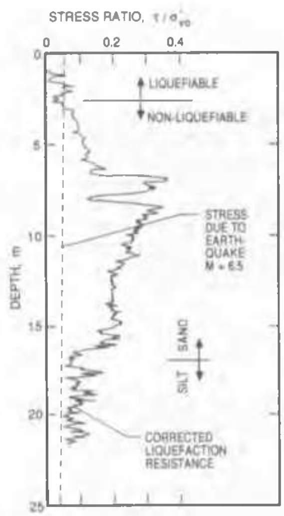


FIGURE 6

LIQUEFACTION POTENTIAL AT AJ34A FOR M = 6.5

Figure 4 shows the results of another approach for assessing the liquefaction potential based on the method of Poulos et al. (1985). The average void ratio and effective horizontal stress of the top 15 m of subsoil after construction of the dyke are represented by a range on Figure 4A. It appears that the soil layer will be contractive and the possibility of a flow slide failure needs to be examined. Hence, a stability analysis based on limit equilibrium was carried out to calculate the average driving shear stress (τ_d) along the critical slip surface. The calculated (τ_d) is plotted on Figure 4B. As this point lies below the steady state strength line, the dyke is unlikely to fail in the form of a flow slide.

SUMMARY AND CONCLUSIONS

The Little Jackfish dyke is to be constructed over a thick layer of nonplastic silt and fine sand. The liquefaction potential of these granular soils was studied using in situ and laboratory methods. The study shows the following:

1. The ratios of tip resistance in kg/cm^2 to penetration resistance in blows per foot are 4.25 and 5 for the silt and the fine sand, respectively.
2. The liquefaction resistance of the silt from piezocone tests is double that determined from cyclic simple shear tests on undisturbed samples.
3. The top loose sand layer at two tested locations was found to be liquefiable under the design earthquake while for the other studied locations, both the sand and silt layers are not likely to liquefy.
4. Based on the steady state approach, the proposed dyke is unlikely to fail in a flow slide condition.

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