Evaluation of Seismic Stability of Foundation Soils Under Revelstoke Earthfill Dam

K. S. Khilnani
B. C. Hydro and Power Authority, Canada

P. M. Byrne
University of British Columbia, Canada

Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

Evaluation of Seismic Stability of Foundation Soils Under Revelstoke Earthfill Dam

K. S. Khilnani, Soils Engineer

B.C. Hydro and Power Authority, Revelstoke, B.C., Canada

P. M. Byrne, Associate Professor

Civil Engineering Department, University of British Columbia, Vancouver, B.C., Canada

SYNOPSIS The stability of the fine sands underlying the shells of the Revelstoke earthfill dam under earthquake shaking is discussed. The evaluation of liquefaction potential of the deposit was made from the field standard penetration resistance of the material. The dynamic analysis, made by a one-dimensional equivalent visco-elastic method, indicated that the sand under both the upstream and downstream shells had adequate liquefaction resistance and could be left in place. Adequate drainage of downstream fine sands was provided to further reduce its liquefaction potential.

INTRODUCTION

The Revelstoke Earthfill Dam, presently under construction on the alluvial terrace on the right bank of the Columbia River, will be a zoned earthfill dam with a glacial till impervious core founded on bedrock and sand and gravel shells founded on overburden. A plan of the earthfill dam is shown on Fig. 1 and a typical cross-section is shown on Fig. 2.

With the exception of the two deep buried channels which cross the dam, the overburden material under much of the length of the dam consists of dense, well graded sand and gravel. One of the two buried channels is filled with a silt and clay deposit, up to 200 feet thick and containing pockets and layers of loose fine sand. A fine sand deposit, up to 150 feet in thickness, was encountered in the second buried channel. Both these deposits could be susceptible to liquefaction if subjected to a strong earthquake shaking. Detailed dynamic stability analyses were therefore carried out to determine if it was safe to leave these materials under the dam shells.

The dynamic stability of the silt and clay deposit was reviewed in detail in another paper by Khilnani et al (1979). In general, all the silt and clay from under the upstream shell was removed. On the downstream side, the silt and clay from under the inner part of the dam shell was removed to ensure that, in the unlikely event that this material did liquefy during a severe earthquake, a failure of the dam would not occur.

The dynamic stability of the fine sands is discussed in this paper.

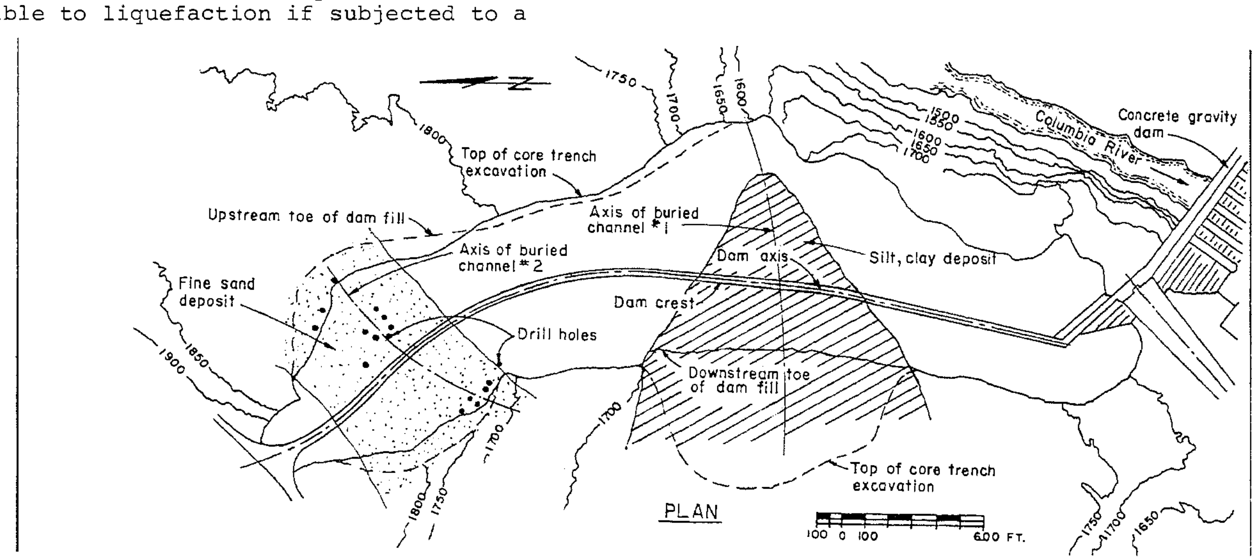

Fig. 1 - Plan of Earthfill Dam Showing Exploratory Drill Holes

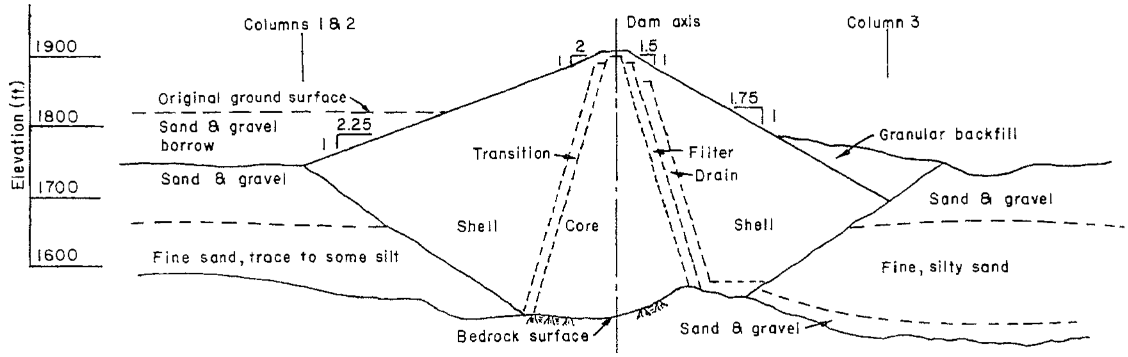

Fig. 2 Typical Cross-Section

FIELD AND LABORATORY TEST PROGRAM

The investigation program basically consisted of determination of relative density of fine sands by the standard penetration tests as well as by direct in-situ density tests. The latter tests were carried out on the core trench slopes during excavation. A total of 15 holes were drilled for obtaining the penetration resistance profile of fine sand. The locations of drill holes are shown on Fig. 1. In most of the drill holes continuous penetration tests were carried out through the fine sand deposit. In some of the drill holes, alternate shelby tube samples and split spoon samples (standard penetration tests) were obtained through the fine sand. The in-situ density tests were carried out at approximately 50-ft. spacing along the length of the core trench and at 10-ft. vertical intervals. Also a total of 6, 10-inch size undisturbed block samples, 3 from the upstream slope and 3 from the downstream slope, were obtained for determination of the laboratory liquefaction resistance.

The Laboratory maximum density for determination of relative density was obtained by both vibrating table method (ASTM D2049), and modified AASHO method (ASTM D1557), since it was considered that some of the sand samples with a higher silt content would probably compact better with the latter method. The lower of the relative density values obtained by the two methods has been used for discussions herein.

The liquefaction resistance of sand was determined by the laboratory cyclic triaxial tests. All samples tested were 3.4 inches in diameter. A total of 30 triaxial tests were performed; 14 were extruded full size from shelby tube samples, 5 were reconstituted and compacted in shelby tubes in 10 layers with 25 tamps per layer of a 5-lb. hammer and extruded, and 11 were trimmed from the 10-inch size block samples.

DISCUSSION OF TEST RESULTS

The typical penetration resistance profiles for both the upstream and downstream drill holes are plotted on Fig. 3. The values plotted are the normalized blow counts after applying the overburden pressure correction suggested by Seed, (1979). Also plotted on this figure are the relative density results obtained along the upstream and downstream slopes of the entire length of the core trench.

The results indicate that the relative density of the sand is generally in the range of 60 to 80 percent. At higher elevation, some lower relative density values of 40 to 50 percent were recorded but these represent localized loose pockets surrounded by generally dense material rather than any loose continuous layers.

The penetration resistance of the sands on the upstream side is quite high with N_1 of about 30 to 50. The downstream fine sands have somewhat lower penetration resistance with N_1 in the general range of 20 to 40. A zone of low N_1 values, between approximate elevations 1615 and 1580, was encountered on the downstream slope in some areas but this again was believed to represent an isolated loose pocket rather than

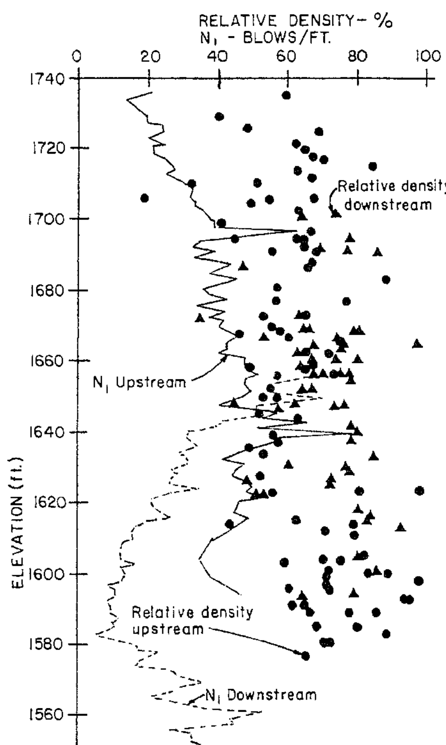

Fig. 3 Penetration Resistance and Relative Density Profile

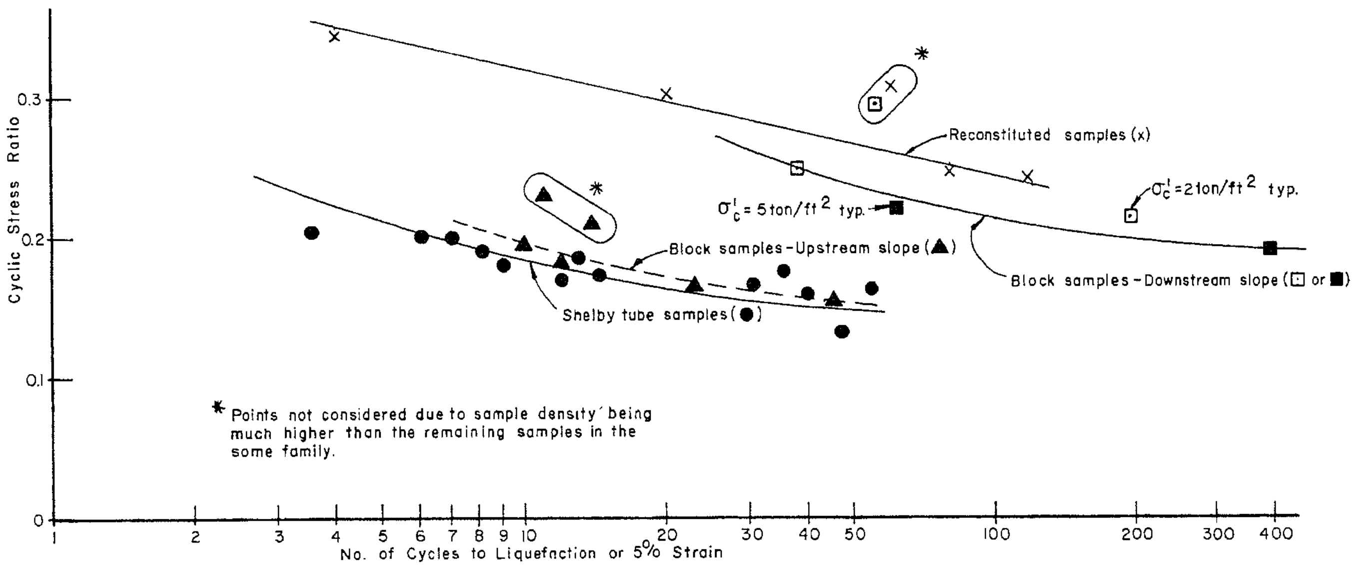

Fig. 4 Laboratory Cyclic Triaxial Test Results

a continuous layer and was not detected by the direct in-situ density tests. Also, it is considered that the standard penetration resistance of the downstream fine sand which has a high silt content (ranging between 10 and 60 percent), may not be a good indicator of its relative density. In general, no definite correlation between the N_1 values and the relative density can be established from the available test data.

The results of laboratory dynamic triaxial tests, plotted as the liquefaction resistance ratio, $\sigma_d/2\sigma'_{3c}$, versus the number of cycles to liquefaction or 5 percent double amplitude axial strain, are shown on Fig. 4. With the exception of 3 samples that were consolidated at effective confining pressure, σ'_{3c} of 2 Tons/Ft^2 , all the samples were consolidated to σ'_{3c} of 5 Tons/Ft^2. The liquefaction resistance ratio for the shelby tube samples and the upstream block samples is much lower than what would be expected for the material with N_1 of about 30 to 40. The average dry unit weight of the material was about 96 lb/ft^3. The reconstituted samples had a dry unit weight of about 105 lb/ft^3 and showed a considerably higher liquefaction resistance.

The above results indicate that the very high N_1 values recorded in the field do not represent a very high relative density but are caused by bonding. This was further confirmed by the results of the pocket penetrometer tests which yielded values in the range of 3 to 5 Tons/Ft^2 in the field as compared to 0.75 Tons/Ft^2 on the reconstituted samples compacted in the laboratory to a relative density of 100 percent. Chemical analysis of the sand samples was carried out to determine the possible presence of a cementing agent. Some minor amounts of calcium carbonate and iron oxide were found and these would possibly cause some cementing of sand.

The relatively low liquefaction resistance of the shelby tube samples and the upstream block samples was believed to be due to sample disturbance. Disturbance of shelby tube samples was quite evident as the cutting edge of most of the shelby tubes was badly damaged due to very high resistance to penetration. Although the block samples were obtained with great care, the upstream sands were generally clean and cohesionless and some disturbance during transportation and trimming of laboratory samples from the block could not be avoided. The liquefaction resistance of the downstream block samples is much higher due to these samples being relatively undisturbed and having a higher silt content than the upstream samples.

The liquefaction resistance of the sand at confining pressures of 2 Tons/Ft^2 and 5 Tons/Ft^2 was found to be nearly the same as seen from Fig. 4. However, the test data are insufficient to conclusively establish this and additional tests at higher confining pressure are planned to study the effect of confining stress on the liquefaction resistance of sand.

DYNAMIC STABILITY ANALYSIS

The method of analysis used herein to estimate the seismic response considers both the dynamic forces caused by the earthquake and the possibility of strength loss or liquefaction of fine sands. It is essentially that proposed by Seed and his co-workers at the University of California, Berkeley, and involves the following steps:

(1) Determine a design earthquake acceleration record.
(2) Determine the pre-earthquake static stresses in the dam and foundation.
(3) Estimate the dynamic stress-strain properties of the dam and foundation soil.
(4) Determine the time history of dynamic stresses within the dam and its foundation from an equivalent viscoelastic dynamic analysis.

(5) Determine the dynamic resistance of the dam and foundation soils under cyclic loading conditions.
(6) Evaluate the liquefaction potential by comparing the dynamic resistance with the dynamic stresses caused by the earthquake.

The section of the dam that was analyzed is shown in Fig. 2. The material of concern is the fine silty sand that underlies the dam shells.

Design Earthquake

The design earthquake was an M6 with a maximum acceleration of 0.2g on firm ground. The time history used was the N-S component of the Long Beach earthquake record of 1933 and scaled to have a maximum acceleration of 0.2g.

Static Stresses

The pre-earthquake static stress required for a dynamic analysis is the mean normal effective stress, $\sigma'm$. It was assumed herein the $\sigma'm$ was given by:

$$\sigma'm = 1/3\, \gamma\, Z\ (1+2k_o)$$

in which γ = the effective weight of the soil, total or submerged.
Z = the depth below the ground surface.
k_o = an earth pressure coefficient assumed to be 0.5

Dynamic Stress-Strain Relations

An equivalent viscoelastic analysis requires estimates of the following dynamic properties of the soils comprising the dam and its foundation:

(1) the maximum shear modulus, G_{max}, at low strain levels
(2) the reduction or decay of G_{max} with the level of shear strain
(3) the equivalent viscoelastic damping

The following empirical equation proposed by Seed and Idriss (1970), was used to determine G_{max}:

$$G_{max} = 1000\ (k_2)_{max} (\sigma'_m)^{1/2}$$

in which $(k_2)_{max}$ = a shear modulus constant and $\sigma'm$ = the mean normal effective stress. G_{max} and $\sigma'm$ are in units of lb/ft^2. $(k_2)_{max}$ depends primarily on the relative density of the soil. A $(k_2)_{max}$ value of 60 for the fine sand and 90 for the sand and gravel was used in the analysis.

The variation of both modulus and damping with strain level was assumed to be in accordance with the empirical relationship proposed by Seed and Idriss (1970), for sand.

Dynamic Stresses

The maximum dynamic shear stresses were computed from a one dimensional equivalent linear viscoelastic analysis using the computer program "SHAKE". The simpler one dimensional analysis was used because Khilnani et al (1979) had shown that it gave similar results to the two dimensional analysis. This finding is also in agreement with the study by Vrymoed and Calzascia (1978).

A total of 3 columns were analyzed; 2 on the upstream side and 1 on the downstream side, as shown on Fig. 2. Columns 1 and 2 on the upstream side are at the same location. The material overlying fine sand on the upstream side is well graded sand and gravel that will be borrowed for use as fill for the dam shells. Column 1 represents the condition where backfill has been placed to El. 1800 following excavation of borrow material. Column 2 represents the condition on the downstream side. The equivalent uniform stress ratio $\tau_{eq}/\sigma'o$, was computed at the various depths within each column and is shown on Fig. 5. τ_{eq} was taken as 0.65 times the maximum computed dynamic shear stress and $\sigma'o$ is the vertical effective stress.

Liquefaction Resistance

There are two methods to determine the liquefaction resistance of soils:
a. from laboratory cyclic triaxial tests, and
b. from standard penetration resistance, using its correlation with the liquefaction resistance, derived from field observations during earthquakes.

As discussed earlier, a proper estimation of the liquefaction resistance of the sands was difficult due to the difficulty of obtaining good undisturbed samples, particularly under the upstream shells, and it was considered that the penetration resistance data would provide a more reliable evaluation of liquefaction resistance.

The relationship between the normalized standard penetration resistance, N_1, and the dynamic shear stress ratio proposed by Seed (1976), was used and is shown on Fig. 6. This resistance chart is based on field experience during earthquakes and is appropriate for level ground when the vertical effective confining stress is in the range 0-2 T/ft^2.

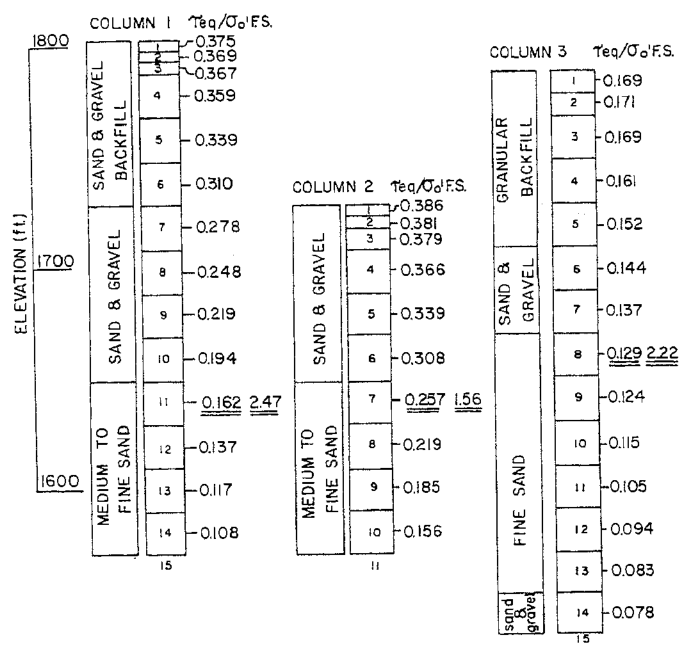

Fig. 5 Dynamic Stress Ratio From SHAKE Program

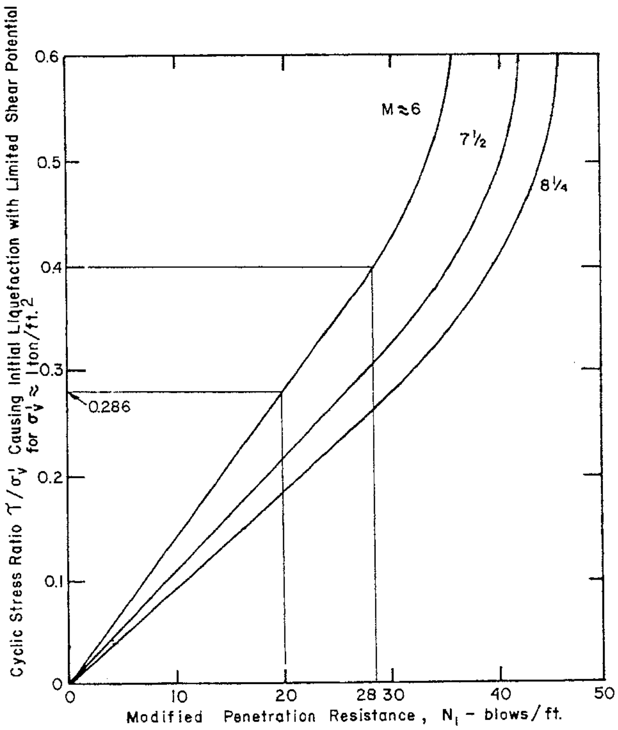

Fig. 6 Correlation Between Field Liquefaction Behaviour and Penetration Resistance for Sands

It has been a general belief that the liquefaction resistance ratio τ_{dy}/σ'_o of a normally loaded sand is independent of the value of the effective confining stress, σ'_o. However, recent data by Seed (1980), have indicated that the resistance ratio may decrease with increasing effective confining stress. As discussed earlier, the limited amount of laboratory test data did not show any appreciable change in the liquefaction resistance of sand in the confining pressure range of 2-5 Tons/ft^2. It was concluded, therefore, that the chart would be appropriate for the range of vertical stresses encountered.

The field N_1 values obtained in the sand deposit range widely. Because this range represents pockets of looser and denser material, or perhaps more or less bonded material, rather than any continuous loose layer, it is appropriate to use a weighted average N_1 rather than the lowest N_1 to represent dynamic resistance of the sand. This is so because, although the zones of lower N_1 value may have less liquefaction resistance, they will also be subjected to lower dynamic stresses because of the arching effect caused by their lower stiffness. An N_1 value which is exceeded by 75% of the field values was considered appropriate. A statistical plot of the N_1 values based on the data obtained from all the drill holes is shown on Fig. 7. From this figure, a weighted average N_1 value of 28 for the upstream sands and 20 for the downstream sands, was computed.

The liquefaction resistance ratios for N_1 values of 28 and 20 and for a number of cycles corresponding to an M6 earthquake were obtained

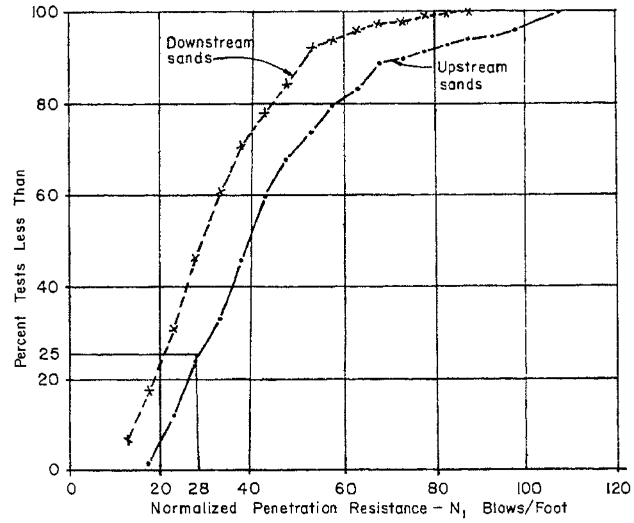

Fig. 7 Statistical Plot of Penetration Resistance Data

from Fig. 6, and are 0.40 and 0.286 respectively.

The above values are appropriate for level ground. For sloping ground conditions and for relative densities in excess of 50 percent, the liquefaction resistance will be considerably higher than for level ground as shown by Lee and Seed (1967), and Vaid and Finn (1979). Thus, the above values will be conservative for sloping ground conditions such as exist in some areas of the foundation sand.

Evaluation of Liquefaction Potential

The factor of safety against the occurrence of liquefaction is the ratio of the liquefaction resistance of the soil to the dynamic stresses caused by the design earthquake. The maximum dynamic stress ratio in fine sand in each of the 3 columns analyzed is underlined in Fig. 5. A sample computation of factor of safety for Upstream Column 1 is shown below:

Maximum dynamic stress ratio in fine sand=0.162

Liquefaction resistance or dynamic stress ratio corresponding to weighted N_1 value of 28=0.40 (from Fig. 6)

Factor of Safety = 0.40/0.162 = 2.47

The factors of safety for the various conditions are as follows:

Upstream Slope - Without backfill	1.6
Upstream Slope - With backfill	2.5
Downstream Slope - With backfill	2.2

All of the above values are higher than the minimum desirable value of 1.4.

The factors of safety are based on the assumption that the liquefaction resistance of sand is independent of the confining stress. Should the laboratory tests currently planned indicate a reduction in the liquefaction resistance with the increase in confining stress, the factors of safety of both upstream and downstream slopes, with backfill, would be lower than 2.5 and 2.2 indicated above.

The factor of safety of the upstream slope without backfill is adequate. The factor of safety for the downstream slope is based on the conservative assumption of groundwater level at El. 1650, which is the highest groundwater level expected in the area. A deep drainage system will be installed at the downstream slope and this system will have the capability to lower the groundwater level to the bottom of the fine sand deposit. If the factor of safety was found to be lower than the acceptable value, groundwater level will be lowered to the bottom of fine sands to minimize their liquefaction potential.

CONCLUSIONS

Based on the results of the analyses discussed above, it was concluded that the fine sands under both the upstream and downstream shells of the earthfill dam had adequate resistance against possible liquefaction under a 0.2g earthquake and could therefore be left in place. Also the factor of safety of the upstream slope following removal of the upper sand and gravel borrow material was adequate and that it would not be necessary to place any additional backfill. The downstream fine sands will be drained, if necessary, to minimize their liquefaction potential.

ACKNOWLEDGEMENTS

The authors wish to thank B.C. Hydro and Power Authority for permission to publish this paper.

Dr. H. Bolton Seed, Consultant to the Comptroller of Water Rights, Province of B.C., reviewed the dynamic stability of the foundation soils for the earthfill dam. His valuable guidance and comments are gratefully acknowledged.

REFERENCES

Khilnani, K.S., P.M. Byrne and K.K. Yeung (1979), "Stability of Earthfill Dam on Soft Silt Under Seismic Loading" Proc. 32nd Canadian Geotechnical Conference, September, pp. 3.148 to 3.168.

Lee, K.L. and H.B. Seed (1967), "Dynamic Strength of Anistropically Consolidated Sand" Journal of Soil Mech. and Foundation Div, ASCE, Vol. 93, No. SM5, September, pp. 169-190.

Seed, H.B. and I.M. Idriss (1970), "Soil Moduli and Damping Factors for Dynamic Response Analysis" Report No. EERC 70-10, University of California, Berkeley, December.

Seed, H.B. (1976), "Evaluation of Soil Liquefaction Effects on Level Ground During Earthquakes" State-of-the-Art, ASCE National Convention, Philadelphia, October.

Seed, H.B., I. Erango, C.K. Chan, A. Gomez-Masso and R. Grant Ascoli (1979), "Earthquake Induced Liquefaction Near Lake Amatitlan, Guatamala" Report No. EERC 79-27, University of California, Berkeley.

Seed,H.B. (1980), Private communication with P.M. Byrne

Vaid, Y.P. and W.D.L. Finn (1979), "Static Shear Stress and Liquefaction Potential" Journal of Geotechnical Division, ASCE, Vol. 105, GT10, October pp. 1233-1246.

Vrymoed, J.L. and E.R. Calzascia (1978), "Simplified Determination of Dynamic Stresses in Earth Dams", Proc. ASCE Geotechnical Div. Speciality Conf. on Earthquake Engineering and Soil Dynamics, Pasadena, California, pp. 991-1006.