International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1991 - Second International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

14 Mar 1991, 2:00 pm - 3:30 pm

Assessment of Seismic Stability of Dolphin Pool Slope of John Hart Dam

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Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 7.11

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SYNOPSIS: The John Hart Dam is in a high seismic risk area. To ensure dam safety, the Dolphin Pool Slope portion of the dam had to be rebuilt. Extensive field investigations and laboratory testing were conducted to obtain representative soil strength parameters for seismic stability analyses and rehabilitation design. Back analysis of the performance of the Dolphin Pool Slope under a 1946 earthquake loading confirmed the strength parameters determined from the field investigation and laboratory testing. Seismic analyses indicated that large zones of the dam could be expected to liquefy under the Maximum Credible Earthquake and, because of uncertainties regarding the residual strengths of sand layers, remedial measures were required which involved the removal and rebuilding of the Dolphin Pool Slope with compacted granular fill.

INTRODUCTION

B.C. Hydro's dam safety program continually reevaluates the safety of its dams based on the current state of practice. Attention was first focused on John Hart Dam when the first Comprehensive Inspection and Review (CIR) of the project was conducted in 1984-85 and indicated that serious deficiencies existed in the seismic stability of the earthfill dams and concrete structures. This paper is concerned with the investigations, analyses and remedial measures carried out to rectify the deficiencies of the Dolphin Pool Slope portion of the project.

BACKGROUND

John Hart Dam is located on the Campbell River in the central part of Vancouver Island about 5 km west of the town of Campbell River, B.C., Canada. Figure 1 shows the location of the dam.

The dam, completed in 1947, consists of a main concrete dam, a power intake dam and three

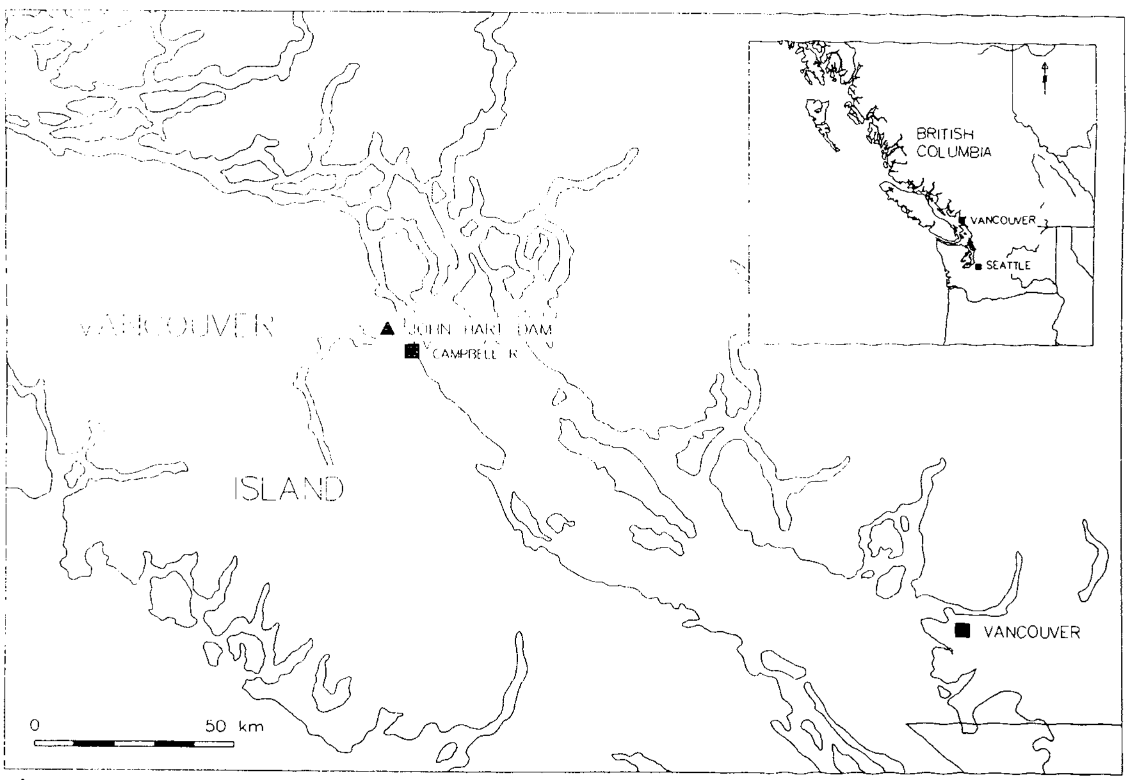


Fig. 1 Location Plan

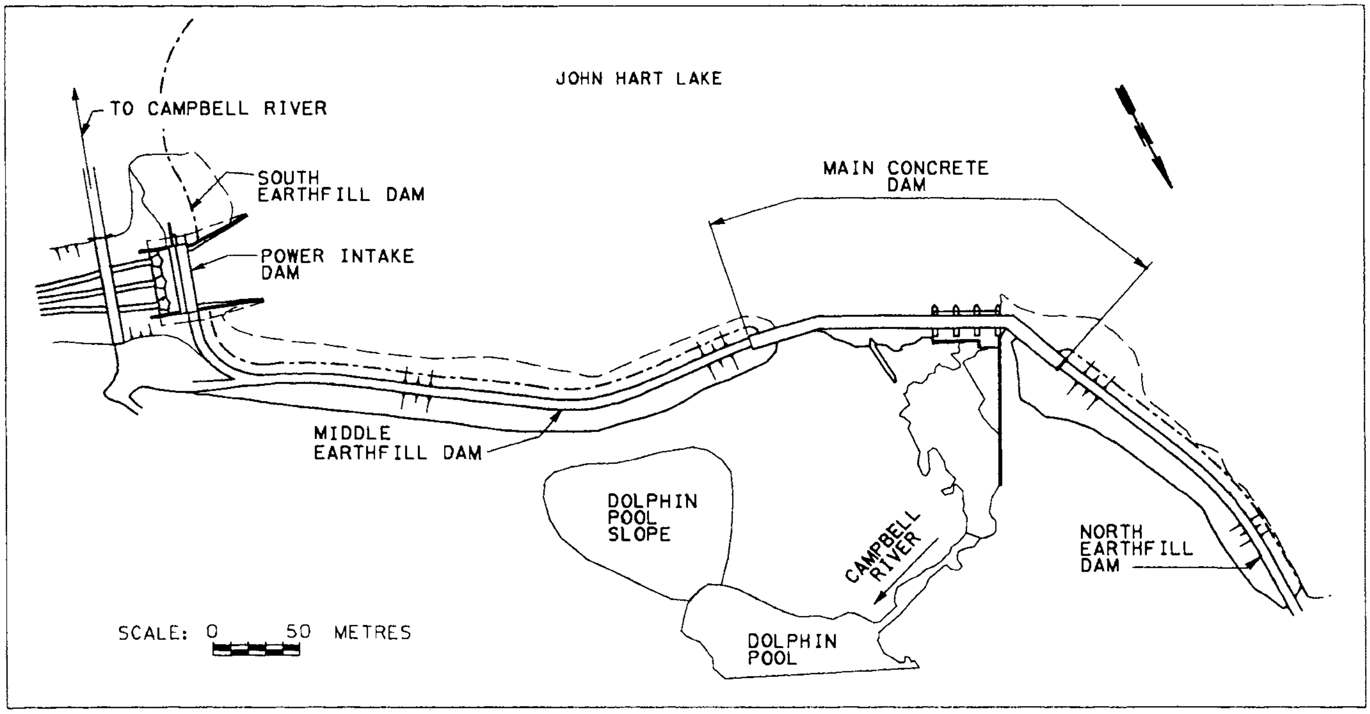


Fig. 2 John Hart Dam General Arrangement

earthfill sections (Figure 2). The north earthfill dam is north of the main concrete dam, the middle earthfill dam connects the main concrete dam to the power intake dam and the south earthfill dam is south of the power intake dam. The earthfill sections are up to 20 m high and have a total length of 600 m. Most of the earthfill sections are founded on loose, saturated sands and silts which are potentially susceptible to liquefaction under severe seismic loadings. A review of the original design documents indicated that there was no consideration given to earthquake forces in the design of the earth structures.

The earthfill embankments were largely comprised of fine to medium sands surrounding a steel sheetpile cutoff. The sheetpiles were generally driven into a low permeability silt layer to reduce the seepage. During construction the sandy dam fill was sluiced toward the sheetpile and roller-compacted.

The middle earthfill dam section consisted of a relatively small (3 m high) dike founded on a 40 m high natural interlayered sand and silt ridge which separates the reservoir basin from the river channel. The downstream slope was named the Dolphin Pool Slope. Figure 3 is an idealized cross section of this slope developed during the investigation and used for analysis purposes.

Since reservoir filling in 1947, a number of occurrences of seepage, boils, subsidences and slides have taken place at the Dolphin Pool Slope. Extensive investigations and remedial work were conducted from 1947 to 1973. These remedial measures were judged at that time to have corrected all seepage and stability related problems including the risk of failure during an earthquake generating a peak horizontal acceleration of 0.25 g.

GEOLOGICAL AND SEISMOLOGICAL SETTING

Geological mapping of the surficial deposits on which the dam is founded has shown them to be deltaic and near shore sediments. The geologic assessment indicates that at the end of glaciation, 10,000 years ago, a rejuvenated Campbell River deposited much of its bedload where the river met the sea. This is now the dam site, some 7 km upstream of the present mouth. As shown in Figure 3, the foundation soil consists of sand and fine gravel at the surface grading down to interlayered sand and silt, followed by silt, till and bedrock.

The west coast area of Oregon, Washington and British Columbia is believed to be a subduction zone capable of generating very large earthquakes. In fact, a large (M 7.3) earthquake occurred in 1946, about 30 km from the John Hart dam site. There were no noticeable local ground or slope movements reported although numerous slope failures and liquefaction events occurred elsewhere in the region.

A seismic review of the site undertaken in 1985 postulated a Maximum Credible Earthquake (MCE) event having a peak horizontal acceleration of 0.6 g. Since the site is considered to have a high hazard rating, the MCE was adopted as the Maximum Design Earthquake (MDE). The MCE is considerably in excess of the 0.25 g previously considered to be appropriate for the site. This demanded extensive field investigations and analyses to assess the seismic stability of the dam and slope.

A number of earthquake records were used in a sensitivity analysis, and the results indicate that the Cal Tech E-W record from the 1971 San

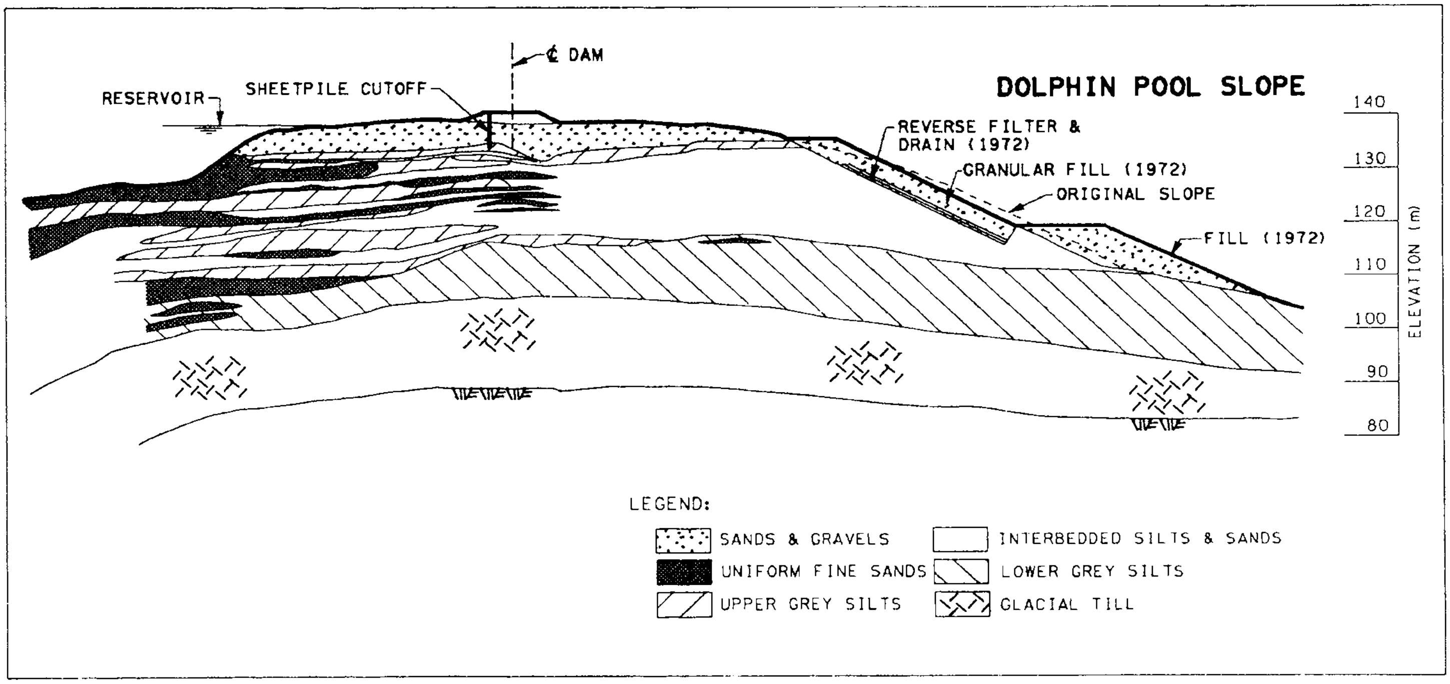


Fig. 3 Section through Dolphin Pool Slope

Fernando Earthquake scaled to a peak acceleration of 0.6 g produced the highest cyclic stress ratios in the region of concern. Therefore, the scaled Cal Tech record was selected for use in all stability analyses.

SCOPE

The scope of this paper is limited to the Dolphin Pool Slope area. This is one of the critical areas and consists of a steep 40 m high natural slope supporting the middle earthfill dam section. This slope is also interesting because it experienced the 1946 earthquake with no significant deformations reported. This information was used to back-calculate soil strength parameters for comparison with laboratory and field determined values. The paper describes the analysis and assessment of the existing slope as well as the remedial measures required.

SOIL PROPERTIES

A comprehensive field investigation, conducted from 1985 to 1987, included 83 Standard Penetration Test (SPT) holes, 34 Electric Cone Penetrometer Test (CPT) holes, 5 downhole acoustic wave propagation analysis profiles, and a hand excavated inspection shaft. Additionally, high quality undisturbed samples were obtained to evaluate soil parameters for seismic stability assessment of the earthfill dam and its foundation. The results of this field investigation were used to develop idealized profiles and cross sections for analysis purposes.

The liquefaction resistance of the sands was based on the SPT values corrected for energy and depth ($(N_1)_{60}$) and the liquefaction resistance chart (LRC) developed by Seed et al (1984) as discussed later. The liquefaction resistance of the silts was determined by laboratory cyclic shear tests.

The strength for various soil materials were obtained as follows:

1. Residual strengths of liquefiable sand layers were inferred from correlations (Seed, 1986) using carefully calibrated SPT data. These correlations are based on field experience during past earthquakes and represented the state of the art in liquefaction assessment of sandy material in 1986.

2. Residual strengths of the silts in the foundation above El. 110 were determined from CPT, field electric vane tests and back analysis based on the 1946 earthquake. Since little change in void ratio would occur in silts during the period of shaking, the residual strength was assumed to be essentially identical to the undrained strength. Undrained strengths were determined from CPT data using two methods (Robertson and Campanella, 1984) as follows:

   (a) End Bearing Method:

$$S_u = \frac{Q_c - \sigma_{ov}}{N_k} \qquad (1)$$

   where:

   Q_c = cone tip resistance
   σ_{ov} = in situ overburden stress
   N_k = 15 (cone correlation factor)

(b) Pore Pressure Method:

$$S_u = \frac{\Delta u}{7.25} \qquad (2)$$

where:

Δu = differential pore pressure

The shear strength values shown for CPT (end bearing method) are low compared to values determined by other methods. This could be due to an unusual correlation factor for the silts at John Hart. In order for these strengths to be comparable to strengths determined by electric vane tests, the correlation factor would have to be approximately 9 which is well below traditional values for insensitive, low plasticity silts.

3. Residual strength of the grey silts below El. 110 was back-calculated based on the 1946 earthquake. This is described later.

A great portion of the investigation concentrated on assessing the nature and strength of the grey silts below El. 115. This information was necessary for seismic stability assessment and final design of the Dolphin Pool Slope rehabilitation. Attempts were made to obtain undisturbed block samples by trenching near the toe of Dolphin Pool slope. This failed due to large seepage volumes entering the trench. It was therefore decided to obtain undisturbed Shelby tube samples from drill holes. These were successfully obtained and tested in the laboratory for both monotonic and post-cyclic strength. In addition, in situ vane shear testing was performed adjacent to one CPT hole and one undisturbed sample hole. This allowed a comparison of the shear strengths determined from laboratory testing of undisturbed samples with the in situ shear strengths. As well, the vane tests were used to develop a direct correlation of undrained strength with the CPT data. The in situ vane tests were conducted at a rate of rotation of 0.1°/sec (ASTM, 1984).

A total of 8 consolidated-undrained (CU) triaxial tests and 15 consolidated-undrained cyclic triaxial tests were conducted to determine the undrained shear strength of the lower grey silt deposit. Samples were consolidated under various principal stress ratios. Four cyclic triaxial tests were performed to assess the strength of the interlayered sand and silt above the silt

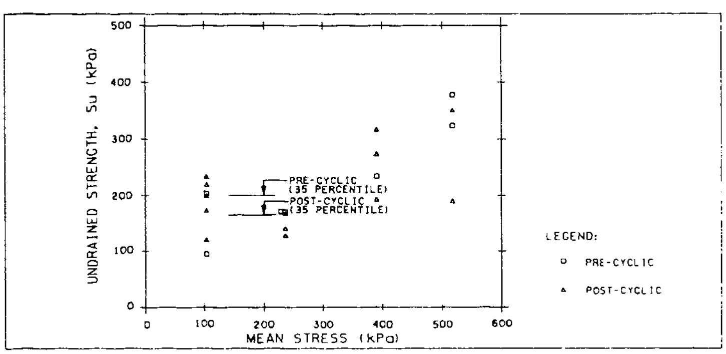


Fig. 4 Pre- and Post-Cyclic Undrained Strengths for Lower Grey Silts

deposit. The silt shear strength results fro monotonic and cyclic tests are summarized i Figure 4. A comparison of the pre- and post cyclic strengths from the laboratory tests ar shown in Figure 5 and indicated that, on average there was little difference between thes strengths. For analysis and design purposes, th post-cyclic strength was assumed to be 0.82 time the pre-cyclic strength, corresponding to degradation factor of 0.82.

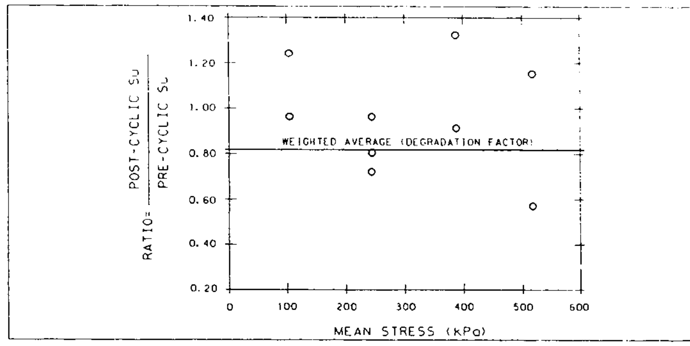


Fig. 5 Reductions of Undrained Strengths After Cyclic Loading

Table I summarizes the shear strength of the silt deposit established by the field and laboratory testing and the back analysis. Except for the back analysis method and discounting the results of the CPT end-bearing method, 65% of the undrained shear strength test results were greater than the values shown in the table. The majority of results indicate the undrained shear strength to be 170 kPa or higher. To account for the potential strength loss due to high strains that may be induced under the MCE, the degradation factor of 0.825 was applied to 170 kPa to give a lower bound value of 145 kPa. This value was used in the stability analyses for the rehabilitation design.

TABLE I. Shear Strength of Silt Deposits

Method of Test	No. of Tests	Shear Strength* (kPa)
CPT (end bearing)	60^m	100
CPT (pore pressure)	60^m	170
Field Vane	148	190
Laboratory CU Tests	8	200
Laboratory Cyclic Triaxial Tests	15	160
1946 Back Analysis	1	170
Pocket Penetrometer	11	150

Average = 170
X 0.825 = 145

* These values represent a 35 percentile value - i.e., 65% of the test results were higher than this value.

BACK ANALYSIS OF 1946 EARTHQUAKE

A large earthquake (M7.3) occurred in June 1946, approximately 30 km from John Hart Dam which was under construction. Since much of the earthfill section comprised a small dike on top of the natural Dolphin Pool Slope, the analysis of this slope for the condition that existed in 1946 allows a check on the dynamic strengths of the native soils, obtained from laboratory and field tests and used in the analysis.

In 1946 the groundwater level in the native soils was approximately El. 123, the level of a swamp just upstream of the Dolphin Pool Slope. Since no slides or significant movements were reported as a result of the earthquake, it has been assumed that movements of no more than 0.5 m occurred in the Dolphin Pool area. In this analysis sand layers above the groundwater were assumed to behave in a drained manner so that triggering of liquefaction would not occur and residual strengths equal to the drained strengths could be assumed.

Based on a peak acceleration of 0.3 g and a peak velocity of 30 cm/sec estimated to have occurred at the site during this event, a 'Newmark' analysis (Newmark, 1965) was conducted. Figure 6 shows the critical surface where maximum movements occur. By varying the strengths of the lower grey silts, it was determined that these silts should have a strength of at least 170 kPa to keep movements along the critical surface under 0.5 m.

ANALYSIS

In the downstream area, the soil's response to earthquake loading depends on the stress-strain and liquefaction characteristics of the soil. The earthquake performance of the earthfill dams and foundation were assessed in a uncoupled procedure to determine deformation rather than a factor of safety against yielding. Deformations induced by the following mechanisms were considered:

1. Stresses which exceed the yield strength of the soil resulting in permanent deformation.

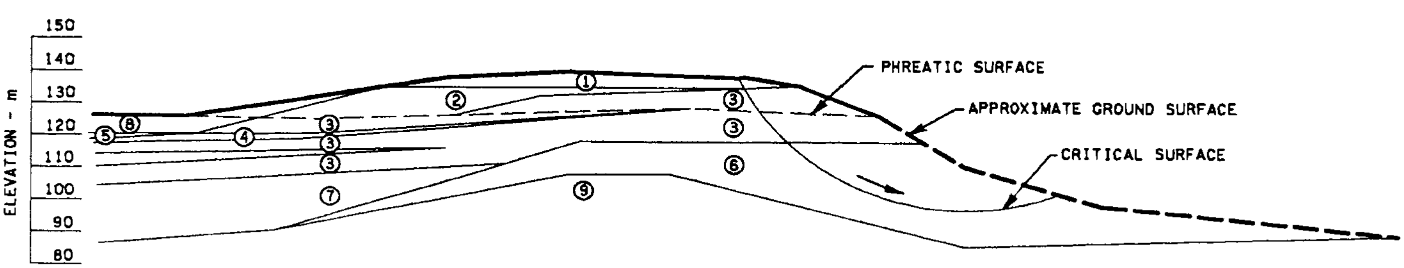
2. A softening or reduction in modulus of the soil which results in added deformation under the gravity loads. In this case, an equivalent elastic finite element analysis was conducted using "Soilstress" (Byrne and Jansen, 1981).

3. Dissipation of excess pore water pressure leading to consolidation and additional deformation.

A key factor involved in such analyses is whether the earthquake is likely to trigger liquefaction. If liquefaction is triggered, then stability is based on the residual strength.

Prior to 1985, it had been common practice to assume that once liquefaction was triggered, the deformations would be unacceptably large. However, recent research and field experience indicates that this is not necessarily true and hence triggering of liquefaction may be acceptable provided the resulting deformations are small and can be safely accommodated.

The resistance to liquefaction and the post-liquefaction residual strength of the soil, in principal, can be determined by testing soil samples under cyclic loading conditions in the laboratory. Undisturbed samples of the silt material were obtained and tested in the laboratory. However, at John Hart, it was very difficult to obtain high quality undisturbed samples in the saturated loose clean sands. Therefore, both the triggering resistance and



ZONE	DESCRIPTION	ASSUMED DYNAMIC STRENGTH	
		Ø	Su(kPa)
1	DENSE SAND (DRY)	38°	-
2	COMPACT SANDS (DRY)	36°	-
3	INTERLAYERED SAND &SILT	-	*
4	LOOSE SAND	-	36
5	COMPACT SAND	-	115
6	SILT	-	*
7	DENSE SAND	38°	-
8	ORGANIC SAND & SILT	-	5
9	TILL	45°	100

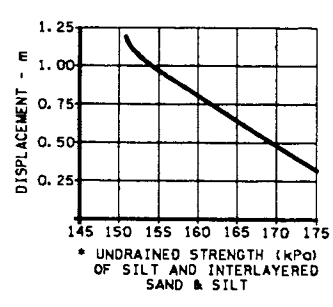


Fig. 6 Back Analysis of 1946 Earthquake

post-liquefaction residual strength of the sands were inferred from normalized SPT values and the correlations developed by Seed (Seed et al, 1984 and Seed, 1986) based on field experience during earthquakes.

The predicted zones of triggering were determined by comparing cyclic stresses caused by the design earthquake with the triggering resistance of the soil. The cyclic stresses were computed from a one-dimensional shear wave propagation analysis utilizing the computer program "SHAKE" (Schnabel et al, 1972). The equivalent cyclic stress ratio (CSR) was computed from these stresses. The cyclic resistance ratios (CRR) of the in situ soils were based upon $(N_1)_{60}$ values and the liquefaction resistance chart (Seed et al, 1984) for the sands and results of laboratory cyclic shear tests for the silts. The potential for triggering liquefaction was assessed by comparing the CRR with the equivalent CSR induced by the MCE. The ratio of CRR to CSR can be interpreted as a Factor of Safety against liquefaction. The results of these analyses indicated that significant zones of embankment and foundation soil would be triggered to liquefaction for the MCE.

Analyses were conducted to determine if the predicted zones of liquefied soil could lead to a flow slide and the extent of such a slide. In the event that a flow slide was not predicted, analyses to predict the likely deformations were conducted.

Based on the SPT and CPT data, the densities and continuities of sand layers above El 123 were a serious concern with the probability of some continuous loose sand layers being very high. Back analysis could not be applied since these layers were not saturated during the 1946 earthquake. The possibility of a flow slide along one of the sand layers during a major seismic event could not be ruled out.

Remedial measures including the installation of an improved upstream cutoff to allow the sand layers to drain was then assumed. Under thi condition, a flow slide was not predicted. Usin the strengths determined by CPT's and bac analysis, the modified Dolphin Pool Slope wa reanalysed for movements under the MCE (a_{max} 0.6 g, v_{max} = 60 cm/sec) assuming that all san layers above El. 123 to be drained. N continuous loose sand layers were identifie below El 123. Yield accelerations were determine using Sarma limit equilibrium analysis (Sarma 1973).

The deformation analysis was based on yield unde seismic loading (Newmark, 1965). Results of th analysis indicated that even with assume drainage of the sand layers, movements of up t 1.5 m could still occur within the silt layers Softening and reconsolidation would furthe increase the overall movements.

Several alternative remedial measures wer considered for the Dolphin Pool Slope. Simple drainage of the sand layers was deeme impractical and difficult to control. It was, therefore, decided that most of the slope had to be removed and backfilled with compacted sand and gravel.

The Dolphin Pool Slope was reanalysed assuming excavation of the interbedded sands and silts to El. 115 and replacement with compacted sand and gravel as shown in Figure 7. The shear strength parameters were determined to be C=0, Ø = 38° based on laboratory data. A shear strength of 145 kPa was used for the underlying silt deposits. Newmark analyses indicated downstream movements of about 1 m for the MCE.

Five extensive sand beds were identified under the upstream slope beneath the reservoir. Three of these sand layers are very dense and were determined not to liquefy under the MCE. If liquefied, one loose sand layer at approximately El. 118 would develop a residual strength of 36 kPa and another layer at El. 112 would develop a residual strength of 115 kPa based on Seed's residual strength chart (Seed, 1986).

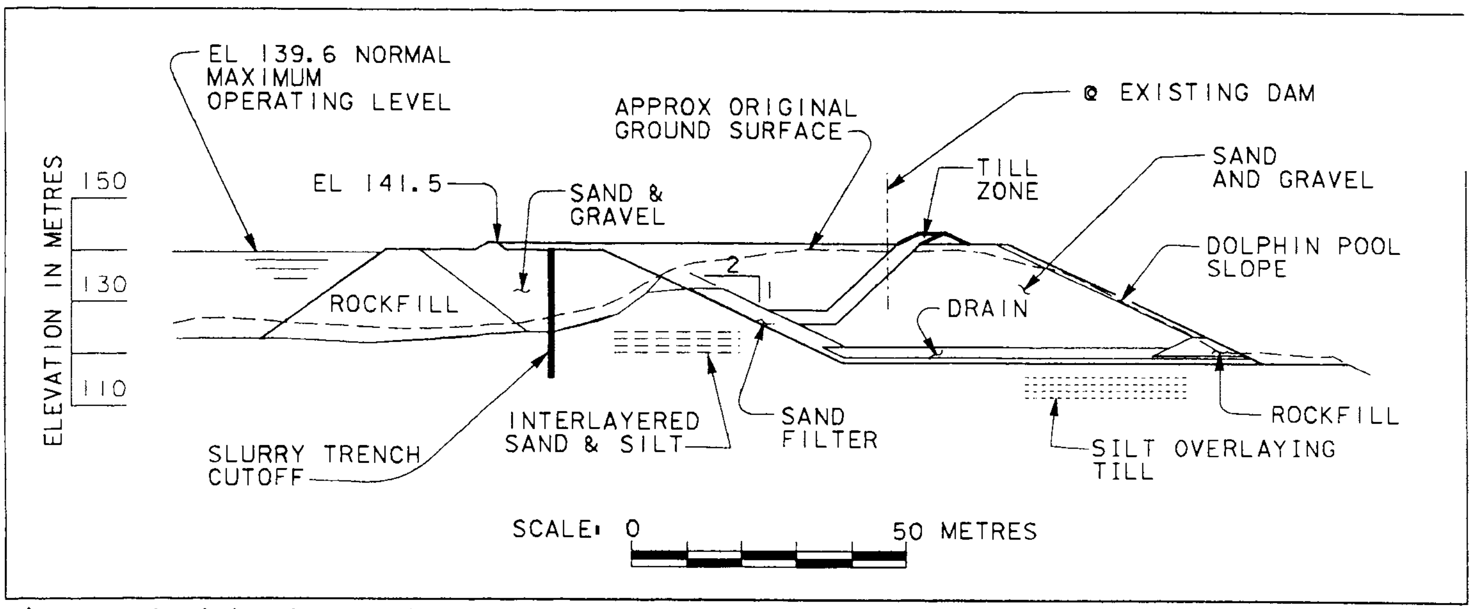


Fig. 7 Rehabilitation Section

'o assess upstream damage during the MCE, a onservative two-stage failure mechanism was ostulated, as shown in Figure 8. The first stage nalysis predicted that an upstream flow slide ould occur along the surface having the minimum ritical acceleration. This surface corresponds o the portion of the El 118 sand layer upstream f the toe of the embankment. The second stage nalysis is based on the assumption that all of he first stage slide would move significantly way from the slope and would not contribute any assive resistance at the toe. The second stage ailure surface would continue along the iquefiable portion of the loose sand layer until t reached the slurry trench. The backscarp would ross the slurry trench and extend up to the urface. Under MCE conditions Newmark analyses redict movements of approximately 1.8 m using hese assumptions.

ith the construction of compacted sand and ravel in the Dolphin Pool Slope, the critical pstream and downstream movement surfaces would ot meet ensuring that any reduction in freeboard ould be due only to modulus reduction and econsolidation of the lower silts. This was etermined to be no more than 0.3 m.

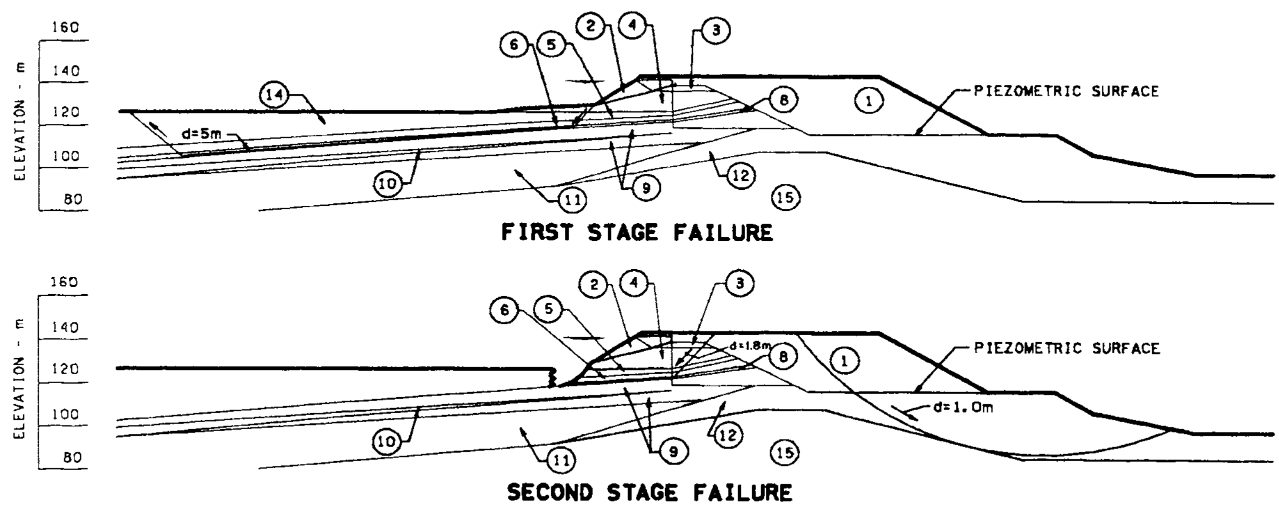
EHABILITATION

o ensure safety for the potential MCE, a large ortion of the Dolphin Pool Slope has been emoved and rebuilt as shown in Figure 7. In addition an embankment consisting of a rockfill dike, vibro-compacted sand and gravel fill, and a slurry trench cutoff were installed upstream of the slope.

Because the dam is crucial for power supply to central Vancouver Island and the reservoir supplies water to the town of Campbell River and a major pulp mill, the entire reconstruction was conducted with full pool.

The new upstream embankment is designed to have limited movement for a 0.2 g earthquake and would likely need remedial work if subjected to seismic accelerations greater than this value. The rebuilt Dolphin Pool Slope is designed to provide reservoir containment for earthquakes up to the MCE.

Dolphin Pool Slope reconstruction requires that a wedge of silt and sand be left in place downstream of the slurry trench cutoff. There was concern that the groundwater level might be higher than expected, and dewatering would be necessary to ensure that the loose sand layers were permanently drained downstream of the slurry trench. A drainage system was designed so that, if necessary, the system could be incorporated quickly during the reconstruction phase. Detailed mapping of the excavation surfaces indicated that the loose sand layers within the wedge were not continuous and were much more contorted and deformed then previously assumed. Therefore, the



LAYER	MATERIAL	Ø	Su (kPa)
1	SAND & GRAVEL FILL	38°	-
2	ROCKFILL	45°	-
3	VERY DENSE SAND	38°	-
4	DENSE SAND	36°	-
5	SILT	-	145
6	INTERLAYERED	-	145
7	LOOSE SAND (SATURATED)	-	36
8	LOOSE SAND (UNSATURATED)	35°	-
9	INTERLAYERED	-	145
10	SAND (SATURATED)		115
11	DENSE SAND	32°	-
12	SILT	-	145
13	TILL	45°	100
14	SURFACE MUD	-	5
15	DISTURBED SOIL	-	5

'ig. 8 Rehabilitation Stability Analyses drainage system was not required.

The dam crest was raised 1.8 m by a dike (till plug) to ensure adequate freeboard after MCE induced movements. To control erosion along the backscarp and to reduce seepage from the reservoir after the MCE, an impervious till zone was incorporated into the downstream fill downstream of the critical sliding surface and extended to the upstream face of the dike.

CONCLUSION

Seismic stability assessment of the Dolphin Pool Slope of John Hart Dam depends on the strength value of the foundation silt deposit. Although extensive field investigations and laboratory testing were conducted, the strength value of the silt material was still difficult to determine. Back analysis of the slope under a 1946 earthquake provides a check on the selection of a proper strength value for the stability analysis and the rehabilitation design. Since the residual strengths of liquefiable sand layers cannot be relied upon, the sand layers were removed. The design and reconstruction of the Dolphin Pool Slope have been completed with no loss of power generation or water supply. B.C. Hydro now considers John Hart Dam safe according to current standards.

ACKNOWLEDGEMENTS

The authors appreciate the permission given by B.C. Hydro to present this paper and thank B.C. Hydro engineering staff for their assistance in preparing this paper.

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