SEISMIC STABILITY OF THE FORT CAMP SEA CLIFFS

by

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SYNOPSIS

The seismicity of the Fort Camp Cliffs is estimated from information on large earthquakes and the geology of the site. Two important facets of seismic stability are examined. The possibility of seismically induced liquefaction failure in water bearing zones is evaluated by a combination of finite element analysis and dynamic triaxial and simple shear tests. The possibility of large block failures due to the additional disturbing forces generated by the earthquake is also investigated. It is concluded that a liquefaction failure is not likely but that block failures may be expected during the design earthquake for the present cliff configuration.

INTRODUCTION

The sea cliffs at Fort Camp on the northern edge of the University of British Columbia campus are receding at an annual rate of 1 to 2 feet. Since the land in this area is extremely valuable for both recreational and university use, many proposals for the stabilization of the cliffs have been suggested. While the recession of the cliffs is primarily caused by wave erosion and water seepage, any program of stabilization should consider the possible effects of a major earthquake on the cliffs. Vancouver lies close enough to the circum-Pacific earthquake belt to occasionally experience earthquakes of magnitude up to 7. The inertia forces generated by such earthquakes may have serious effects on the stability of slopes with already small reserves of resistance against failure. For this reason, a study of the seismic stability of the Fort Camp sea cliffs was undertaken.

The seismic stability of a slope depends primarily on the following factors:

A. the physical and hydraulic condition of the slope,

B. the magnitude and duration of the anticipated earthquake,

C. the dynamic properties of the slope material,

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and D. the static and seismic displacements and stresses in the slope.

Each of these factors is investigated herein, and conclusions are drawn about the present seismic stability of the cliffs.

A. THE PHYSICAL AND HYDRAULIC CONDITION OF THE SLOPE

Geology of the Site

The Fort Camp sea cliffs at Point Grey rise some 200 feet above sea level and have average slopes of more than 45 degrees in many places. A typical stratigraphic section of the cliffs is shown in Figure 1 and an oblique air-photograph (January 1967) of the cliffs is shown in Figure 2. Mean sea level is at Elevation 91, the base of the cliffs is at approximately Elevation 95, the crest of the cliffs is at approximately Elevation 310, and high and low tide levels are Elevation 98 and Elevation 82 respectively (Vancouver City Datum).

The cliffs are comprised of glacial and interglacial deposits. During each major glaciation the land was depressed relative to the sea by 1000 feet or more (1)[I]. The cliffs are capped by 15 to 20 feet of till-like material, of which the top 5 feet is considered to be of glacio-marine origin. This loose and relatively pervious top 5 feet is termed Newton stony clay and is thought to have been transported by floating ice and deposited as the ice melted. Below the Newton stony clay there is 10 to 20 feet of dense material termed Surrey till that is thought to have been deposited below the ice sheet by glacial action.

The Quadra formation, which is comprised of inter-glacial horizontally bedded sands and silts, lies below the Surrey till. From approximately Elevation 290 (base of the till) to Elevation 155 this formation is mainly dense, fine, silty sand with some thin silt layers and some thin sandy gravel layers. The demarcation between this silty sand and the interbedded layers of very dense silts and sands below Elevation 155 can be plainly seen in the air-photograph of Figure 2.

The Quadra formation is underlain by Tertiary rock. This bedrock outcrops at sea-level between Kitsilano Point and Jericho Beach some three miles to the east of the Fort Camp cliffs. Sandstone bedrock was encountered at Elevation -190 in a hole drilled for the Fisheries Research Board of Canada at a site approximately 1000 feet south of the Fort Camp Cliffs. It is considered that the bedrock is approximately horizontal and therefore some 280 feet below the base of the cliffs.

Ground Water Conditions

Rain water penetrates the relatively permeable Newton stony clay. However, the Surrey till acts as an impermeable boundary causing the surface and near surface ground water to flow parallel to the ground surface. In some places the Surrey till has been removed, either by natural erosion or man-made excavations, and in these locations the water seeps vertically down through the pervious Quadra sand until the silt layer at Elevation 155 is reached. The

[I] See the list of references at the end of the paper.

silt layer acts as a semi-impermeable membrane causing the bulk of the ground water to flow horizontally until it emerges at the cliff face as shown in Figure 1. Some water does penetrate the silt layers and emerges from the inter-bedded layers of sand. However, most of the water emerges above the top layer of silt.

Factors Leading to the Instability of the Cliffs

The sea cliffs in the Fort Camp area are receding at an average annual rate of 1 to 2 feet per year. Undercutting of the cliffs by wave erosion and the subsequent transportation of the eroded material into Burrard Inlet by longshore drift is one of the major causes of the cliff recession. The piping effect of seeping ground water at, and below, Elevation 155 is the other major factor contributing to the cliff recession (2).

The cliffs from their base at Elevation 95 to Elevation 155 are in places as steep as 70 degrees. Above Elevation 155 and below the till the cliffs have slopes that are generally less than 45 degrees but in places are as steep as 70 degrees. Dense sands can be expected to have a friction angle of the order of 46 degrees or less (9). Hence the silty sand could be expected to have slopes of 46 degrees or less. The steeper slopes may be caused by the presence of cohesion due to cementing or apparent cohesion due to partial saturation.

It is felt that these steep slopes in the silty sand are temporary since the silt will soften with time. Evidence of slumping is present and where slumping has occurred the cone of slumped material has a slope of approximately 35 degrees. Recession of the cliffs is thought to occur as a rapid process over a short period of time rather than as a gradual continuous process. When a major storm occurs simultaneously with an unusually high tide, rapid erosion at the base of the cliffs by wave action can be expected. Undercutting at the base of the cliffs may cause immediate failure, or failure may take place some time later caused by seepage forces developed during periods of high infiltration due to snow melt or heavy precipitation.

It is clear that the stability of the face of the cliff is transient and that failure may result when the slope is subjected to additional stresses during an earthquake. The extent of the possible failure will depend on the magnitude and duration of the earthquake.

B. MAGNITUDE AND DURATION OF EARTHQUAKE

Of the large number of earthquakes that have occurred off the West Coast of North America there have been three known earthquakes of magnitude 7 or greater within 100 miles of Vancouver (10). The magnitude of an earthquake gives a measure of the energy released. However, the actual damage caused by an earthquake in a particular area depends on many factors such as magnitude, distance from the epicentre and the foundation conditions. The term intensity is used to grade the damage caused by an earthquake. The intensity scale most widely used is the Modified Mercalli Scale which is defined by details of perceptibility and damage on an increasing scale from 1 to 12. Damage to ordinary structures corresponds to an intensity of about 7 to 8.

The Campbell River earthquake of June 1946 was recorded as having an intensity of 6 in Vancouver and, as such, is the highest on record. The Seattle earthquake of 1949 had an intensity in Vancouver of 4.5. There is no record of the intensity observed in Vancouver for the 1918 earthquake ($M = 7$) off the West Coast of Vancouver Island.

The Campbell River earthquake had a magnitude of 7.3 and was centred under the Strait of Georgia. Little damage occurred in the Vancouver area, and there is no record of damage to the Fort Camp Cliffs during this earthquake. However, damage was severe in some sections of Victoria which are underlain by poor foundation materials. For this particular earthquake, Victoria is further from the epicentre than Vancouver and this would suggest that the foundation conditions for most building sites (at least up to 1946) in Vancouver were more favourable from earthquake considerations.

In order to carry out a dynamic analysis on any structure, it is necessary to have some measure of anticipated earthquake accelerations. Richter (7) has suggested, on the basis of observations in California, that a measure of the maximum surface acceleration for average ground conditions is given by:

$$\log_{10} a = I/3 - 0.5 \qquad (1)$$

where a = the maximum ground surface acceleration in cm/sec^2

and I = the Intensity.

For an earthquake of intensity 7 to 8, Equation 1 yields a maximum surface acceleration of about 0.1 g. This indicates agreement with the intensity scale definitions since an acceleration of 0.1 g is usually considered as that which can damage ordinary structures. Equation 1 indicates that the Campbell River earthquake had a maximum surface acceleration of 0.032 g in Vancouver.

From the Seismic Zoning Map for Canada (1970), Vancouver lies within Zone 3 which represents areas that may be subject to an earthquake of at least 0.06 g for a return period of 100 years (10). This corresponds to an intensity of just below 7 (6.85). An upward limit on acceleration level is not indicated for Zone 3.

The highest surface accelerations recorded during an earthquake were, until recently, associated with the El Centro, California, earthquake of 1940. This earthquake had a magnitude of 7 and an intensity of 8 in the epicentral region. The maximum recorded surface acceleration for the El Centro earthquake was 0.33 g. Using the Richter relationship (Equation 1) the maximum predicted ground acceleration would be 0.16 g or only about one half that actually recorded.

Data presented by Wiggins (11) indicates that for a number of earthquakes recorded on the West Coast of North America an earthquake of magnitude 7 could be expected to have an intensity of 8 in the epicentral area and a maximum ground acceleration of from 0.16 to 0.33 g. Wiggins concluded that there is no satisfactory relationship between ground acceleration and intensity and his data suggests that the maximum ground accelerations may range up to twice those predicted from Equation 1. Thus, it would appear that the values of 0.032 g and 0.06 g from the Campbell River earthquake and Seismic Zoning Map of Canada respectively, may not be high enough and a design figure of 0.1 g might be quite reasonable.

This design figure is further supported by Russian data relating earthquake intensity to ground acceleration. The Russian scale of earthquake intensity GOST 6249-52 is similar to the Modified Mercalli scale. According to GOST, for a magnitude 7 earthquake, the maximum ground acceleration should lie between 0.05 and 0.1 g. However, in June 1966, during an earthquake of intensity 8 at Parkfield, California, a maximum acceleration of 0.5 g was recorded. There is no reason to suppose that such a radical change in local earthquake history will not occur here. Therefore, since earthquakes of magnitude 7 have occurred close to Vancouver and since the El Centro, California, earthquake was also of magnitude 7, it is suggested that the El Centro earthquake should be considered as the extreme design earthquake for the seismic analysis of the Fort Camp cliffs.

The duration of shaking of an earthquake is also of great importance in determining the behaviour of slopes during earthquakes. Indeed, it is believed that the long duration of the 1964 Alaska earthquake was primarily responsible for the slides that occurred there (8). In the present study, the earthquakes were assumed to last between 30 seconds and 2 minutes. The effect of duration of seismic loading is discussed later when simulated earthquake tests are described.

C. DYNAMIC PROPERTIES OF SOILS

The strength of a soil and its resistance to deformation under static and dynamic conditions may be quite different (4, 5). Cyclic loading triaxial and simple shear tests on saturated sand samples indicate that the dynamic strength of a sand may be considerably less than the static strength. A tendency for volume decrease during cyclic loading causes increased pore pressures to occur. The pore water pressure may rise and become equal to the normal pressure, in which case the effective stress, and consequently the shearing resistance, becomes equal to zero. This phenomenon is termed liquefaction. Many of the slides in the Alaskan earthquake of 1964 were considered to have occurred due to strength loss by liquefaction under sustained shaking (8).

Since the sand above the silt at approximately Elevation 155 in the Fort Camp cliffs is thought to be saturated, there is a possibility of liquefaction occurring in this material. The pore water pressures may dissipate rather quickly in this zone due to vertical drainage since the zone of saturation is thought to be only from 1 to 10 feet above the silt. However, in the sand layers below the silt, vertical drainage would be restricted and it is probable that liquefaction is more likely in these zones. Dynamic triaxial and simple shear tests were therefore run to determine if liquefaction would be a possibility in these zones. The results of these studies are presented later.

D. STATIC AND DYNAMIC STRESSES IN THE CLIFFS

Since the Fort Camp cliffs are receding at an average rate of approximately 1 to 2 feet per year, the factor of safety under static conditions must continually approach unity over certain sections of the slope. In other words, the slope is in a 'meta-stable' state. Thus, it may be expected that, when the inertia forces of any significant earthquake act on the slope in its present physical condition, a failure of some kind will probably occur. The important question to be answered is the extent and nature of the potential failure.

The effect of any seismic forces will depend, among other things, on the static stress condition in the slope prior to the earthquake. The finite element method of analysis (3) was used to determine both the static and dynamic stresses in the slope. A uniform slope of 50° was selected for the analysis since such a slope would be representative of the average present conditions and might well represent a tentative design slope. The slope is idealised as an assemblage of discrete elements connected at the nodes as shown in Figure 3, a fine mesh being used in the region of anticipated high stress gradients. The region selected for analysis is large enough for the reliable determination of the static stresses in the slope. For the static analysis it was assumed that all displacements along BC were zero and that horizontal displacements were prevented on the boundary CD.

The material properties used in the finite element static and dynamic stress analysis of the 50° slope were: unit weight γ = 130 lbs/ft^3, and a Poisson's ratio of 0.30. Static stresses are independent of Young's modulus for the assumption of homogeneous isotropic material behaviour that was used. The static stress output enables the designer to assess the stability of the slope, and to determine the consolidation stresses to be used in any simulated earthquake testing.

It was determined by standard laboratory testing that the material in the slope can support a maximum principal effective stress ratio of approximately 5.5. The finite element static stress analysis showed that this ratio was exceeded only at points near the face where the material is in a 'meta-stable' state. However, the overall body of the slope is stable.

The dynamic or seismic stresses that may occur in the slope when an earthquake similar to El Centro acts on the base were also evaluated with the computer program. Since the dynamic program is extremely time-consuming to use because of the many calculations that must be performed at each time step, output was restricted to nodal points that were considered significant in assessing the seismic stability of the slope. The dynamic stresses do depend on the value of Young's modulus and a value of 100,000 psi was used for the slope.

The dynamic response of a slope is markedly affected by the boundary conditions assumed for the slope. To cover a wide range of boundary conditions the following three dynamic analyses were run on the slope in Figure 3 under the action of the El Centro 1940 earthquake: (1) the boundary CD was assumed completely free, (2) the boundary CD was restrained horizontally, and (3) the boundary CD was moved several hundred feet to the right to minimize the effect of horizontal restraint. Examples of results obtained from the dynamic analysis are shown in Figures 4a, 5a and 6a where the horizontal dynamic stresses, σ_x, are shown for particular times during the earthquake. The static stresses are also shown. Since it is the combined static and dynamic stresses to which the soil responds, these combined stresses are shown in Figures 4b, 5b and 6b.

SEISMIC STABILITY OF THE CLIFFS

Under the action of a given design earthquake the slope may fail in one of three modes:

1. chunks may break off the cliff face whenever sufficient tensions are developed as a result of the superposition of the alternating seismic stress system on the initial static stresses,

2. slippage of a block of soil at the cliff face may occur along some slip plane,

or 3. the material in the cliff below the water table may liquefy under the continued excitation of the earthquake and the entire slope may move seaward. This motion would result in the formation of trenches (grabens) behind the face of the slope and the loss of material from the slope face (8).

Failure Modes 1 and 2 will be discussed in this section; Mode 3 is examined in the next section in conjunction with a discussion of the laboratory testing.

Mode 1

If material losses from the cliff face, particularly near the crest, are to be significant then tensions must develop near the face to some depth. It is considered here that tensions developing to a depth of 50 feet could cause significant chunks of material to break off the cliff face.

From the dynamic computer output for nodes at a depth of 50 feet, cases of extreme tension were abstracted and shown in Figures 4, 5 and 6. In Figure 4b it is seen that the combined horizontal static and dynamic stresses are in tension to a distance of 60 ft from the cliff face. It could be concluded that a maximum of about 60 ft of the cliff crest may break off under the action of an earthquake such as El Centro.

Mode 2

Newmark's (6) method was used to investigate the possibility of significant movement of a block of the cliff face along some slip plane. This method assumes that displacements will occur due to the sliding motion of a block such as shown in Figure 7a. The block is assumed to behave as a rigid plastic single degree of freedom system. Using a computer program, Newmark determined a relationship between displacement and the ratio N/A where N is the maximum resistance coefficient, and A is the maximum earthquake acceleration in gravity units. This relationship is shown in Figure 7b for four earthquakes which have been normalized to a maximum acceleration of 0.5 g and a maximum ground velocity of 30 in./sec.

For the wedge shown in Figure 7a, the maximum resistance coefficient, N, is given by:

$$N = \cos\alpha \tan\phi - \sin\alpha \qquad (2)$$

Results from triaxial tests indicated that a friction angle ϕ of 44° may be operative in the silty sand, hence:

$$N = 0.966 \cos\alpha - \sin\alpha \qquad (3)$$

Assuming α to range between 30 and 44 degrees, values of N can be calculated using Equation 3. For the normalized earthquakes used by Newmark, the maximum acceleration A was 0.5 g. Hence the ratio N/A can be calculated and, using Figure 7b, the standardized displacement for the normalized earthquake can be determined.

Displacements so obtained will be for a normalized earthquake. For other earthquakes an estimate of the displacements can be obtained if it is assumed that, for the same ratio of N/A, the displacements are proportional to the square of the maximum ground velocity. Hence for the El Centro earthquake, which had a maximum ground velocity of 16.3 in./sec, the standardized displacements should be reduced by the factor $(30/16.3)^2$, i.e. 3.4. The necessary steps to obtain displacements to the El Centro earthquake are shown in Table 1. The relationship between the angle of the potential sliding block, α, and the displacements is shown in Figure 7c.

It may be seen that for α less than about 40° the displacements are small. For α greater than 40° the displacements commence to increase rapidly. Therefore, it would appear that, if a flattening of the slopes is being considered, a slope angle of less than 40° should be adopted. If construction close to the face of the present cliffs is being considered, then any structure should be located back of the intersection of the 40° line with the ground surface. These conclusions are based on the assumption that erosion at the face of the cliffs is arrested.

If large displacements were to occur along a slip plane leading in effect to a slumping failure, then the failure would extend back from the crest for about 100 ft. It was previously noted that for the same 50° slope a tension break-off could be expected to progress back as far as 60 ft from the crest. Thus it seems that the crest of the slope for a distance of at least 100 ft from the edge may be affected by an earthquake of the magnitude of El Centro.

MODE 3 - LIQUEFACTION OF SLOPE MATERIAL

Under prolonged shaking by an earthquake, the pore water pressures in undrained saturated silts and sands may increase until the effective stresses in the deposit are reduced to very low values. At this point, the shearing resistance of the material is much reduced and the material is said to have liquefied. From a practical point of view the material may be said to have liquefied whenever, as a result of continued shaking, the shear strength has dropped below the value needed to resist any disturbing forces which are present.

Since the Fort Camp cliffs contain beds of saturated silts and sands, one of the crucial questions in assessing the stability of the cliffs is whether these deposits will liquefy or not during an earthquake. To assess the possibility of liquefaction, simulated earthquake loading tests must be run on representative samples of material from the cliff.

Initially the action of an earthquake was simulated by alternating or cyclic stress tests on triaxial specimens. However, at the time these tests were carried out, it was not possible to induce principal stress reversals in this particular machine, a phenomenon which may occur during an earthquake and which is considered particularly conducive to liquefaction. Later, the development of a dynamic simple shear apparatus was completed and complete stress reversal tests were run. These dynamic triaxial and simple shear tests require special mechanical equipment, test procedures, and monitoring and recording equipment.

Seven dynamic triaxial tests were run on a sample taken from Elevation 190 in the cliffs. The material was a dense, grey, silty fine sand and was tested in an undrained saturated state at a dry density of γ_d = 103.7 pcf. To encourage pore water pressure build-up, the sample was consolidated to rather light stresses; the vertical stress σ_1' was 1.425 Kg/cm^2, and the horizontal stress σ_3' was 0.665 Kg/cm^2, giving a static deviator stress σ_d of 0.76 Kg/cm^2.

For each test a dynamic deviator stress equal to a given percentage of the static deviator stress was applied at a rate of 2 cycles per second and the applied load, axial deformation and the increase in pore water pressure were recorded. The results are summarized in Table II. It may be noted that, although the pore water pressure did increase, liquefaction did not occur in any of the dynamic triaxial tests.

Dynamic Simple Shear Tests

Liquefaction under alternating stresses can only occur in undrained saturated sands and silts. Thus, in the Fort Camp cliffs liquefaction is possible only below the water table and Node 18 (Figure 3) was selected as a representative point at which liquefaction might occur. Since the static stress analysis indicated that the normal effective stress at Node 18 was about 7 Kg/cm^2, all of the dynamic simple shear tests were run with a normal effective stress of 7 Kg/cm^2.

Various dynamic horizontal shear forces were applied at 2 cycles per second to simulate the horizontal inertia forces or lateral forces for earthquakes of different magnitudes. The dynamic stress analyses indicated that the maximum dynamic shear stress at Node 18 was approximately 1.86 Kg/cm^2 for an earthquake similar to El Centro. Therefore, a maximum cyclic shear stress of 2 Kg/cm^2 was used. Tests were also run with dynamic shear stresses of 1.68 Kg/cm^2 and 1.0 Kg/cm^2.

A total of 10 tests were run with sample dry densities ranging from 102.5 pcf to 107.0 pcf. Pertinent data on all of the dynamic simple shear tests is summarized in Table III. The relationship between void ratio e and cycles to liquefaction is shown by the straight line on Figure 8. This line is valid for layers in which the ratio of dynamic shear stress, τ_d, to normal stress, σ, lies between 0.25 and 0.30.

From data based on drill logs for the cliffs, it would appear that a void ratio of 0.60 represents the loosest possible state for either the saturated sand or silt. It may be inferred from Figure 8 that in this state the saturated material would require approximately 150 cycles of shear stress at a level of at least 1.6 Kg/cm^2. Judging from the past earthquake history of this area, it is considered unlikely that such a large number of significant shear stress cycles of the order of 1.6 Kg/cm^2 would occur at the Fort Camp cliffs. Therefore, it is concluded that the saturated material in the slope will not liquefy.

CONCLUSIONS

The probable seismic stability of the Fort Camp cliffs under the action of an earthquake similar to that which occurred at El Centro, California, in 1940 was evaluated. This earthquake had a maximum acceleration of 0.33 g. Although earthquakes of similar magnitude to El Centro (magnitude 7) have occurred near Vancouver, such high accelerations are considered to have a low probability of occurrence here in view of the local earthquake history. Thus, El Centro is considered to represent an extreme case. There is a reasonable expectation, however, of the occurrence of an earthquake with a maximum ground acceleration of greater than 0.06 g.

Three possible modes of failure of the cliffs have been examined: (a) cracking and spalling due to excessive tensions, (b) large displacements due to progressive sliding deformations and (c) large slope movements due to the liquefaction of saturated sand and silt layers.

The following conclusions appear to follow from the results of the theoretical and experimental studies.

1. The material in the slope will not liquefy under the extreme earthquake represented in magnitude and duration by the El Centro 1940 earthquake.

2. Material may be lost from the crest of the slope to a distance of 60 feet in from the crest due to cracking and spalling caused by excessive tensions.

3. The Newmark analysis for sliding displacements under the El Centro earthquake indicates that for a 50° slope a sliding block extending about 100 feet into the slope at the crest would undergo excessively large displacements.

4. If the slope were cut back to 40°, the sliding displacements under El Centro would be small.

All conclusions regarding the seismic stability of the Fort Camp cliffs are only valid for the slope in its present unloaded state and with its present hydraulic characteristics.

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TABLE I

RELATIONSHIP BETWEEN ANGLE OF THE SLIDING BLOCK AND DYNAMIC DISPLACEMENT - EL CENTRO EARTHQUAKE

α in degrees	N	A	N/A	Standardized Displacement Inches	Displacement for El Centro Earthquake Inches
30	0.337	0.33	1.020	≈0	≈0
34	0.243	0.33	0.735	1.3	0.4
36	0.194	0.33	0.588	3.0	0.9
38	0.146	0.33	0.442	7.0	2.1
40	0.097	0.33	0.294	18.0	5.3
42	0.050	0.33	0.151	44.0	13.0
43	0.026	0.33	0.079	180.0	53.0

TABLE II

DYNAMIC TRIAXIAL TESTS - SERIES 2

Test No	Dynamic Deviator Stress as percentage of Static Deviator Stress	No. of Cycles	Change in Pore Pressure Δu Kg/cm^2	σ_1'/σ_3'	Remarks
1	± 21%	100	+ 0.04	2.47	No Liquefaction
		Drained			
2	± 50%	100	+ 0.08	2.91	No Liquefaction
		Drained			
3	± 75%	100	+ 0.14	3.51	No Liquefaction
		Drained			
4	± 100%	500	+ 0.28	4.96	No Liquefaction
		Drained			
5	+ 125% - 100%	1000	+ 0.24	5.16	No Liquefaction
6	+ 150% - 100%	200		5.32	No Liquefaction
7	+ 175% - 100%	200		5.38	No Liquefaction
8	+ 200% - 100%	200		5.55	No Liquefaction
+ indicates an increase in pore water pressure					

TABLE III

SUMMARY OF UNDRAINED DYNAMIC SIMPLE SHEAR TEST RESULTS

Material taken from cliff face above silt.

2 Bulk Density tests in field indicated γ_d = 116 pcf and 101 pcf with e = 0.456 and 0.667.

Tests performed with initial effective normal stress, σ_n = 7 Kg/cm^2.

Dynamic shear stress τ_d applied at a frequency of 2 cps.

Test No.	Void Ratio e	Dry Soil Density γ_d - pcf	Dynamic Shear Stress τ_d Kg/cm^2	τ_d/σ_n	No. of Cycles	Pore Pressure Rise Kg/cm^2	Remarks
1	0.640	102.5	1.00	0.143	40	0.80	No Liquefaction
2	0.620	103.5	1.15	0.165	240	2.50	No Liquefaction
3	0.680	101.0	1.68	0.240	12	6.00	Liquefied
4	0.630	103.0	1.68	0.240	20	6.25	Liquefied
5	0.605	105.0	1.68	0.240	144	6.10	Liquefied
6	0.580	106.5	1.68	0.240	146	2.03	No Liquefaction
7	0.636	103.0	2.00	0.286	10	6.00	Liquefied
8	0.626	103.5	2.00	0.286	23	5.75	Liquefied
9	0.570	107.0	2.00	0.286	199	1.55	No Liquefaction
10	0.600	105.0	2.00	0.286	145	6.35	Liquefied

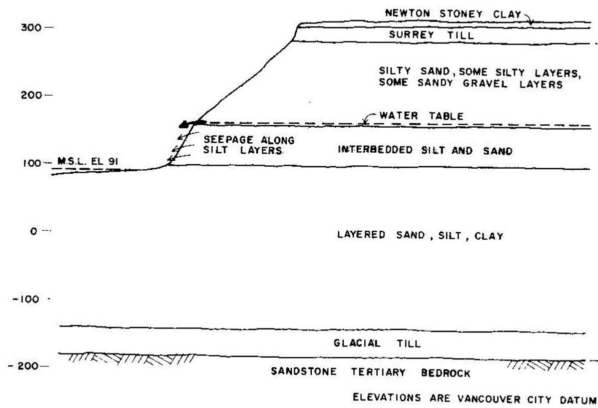

FIG. 1 TYPICAL STRATIGRAPHIC SECTION OF THE FORT CAMP SEA CLIFFS.

FIG. 2 AIR-PHOTOGRAPH OF THE FORT CAMP SEA CLIFFS

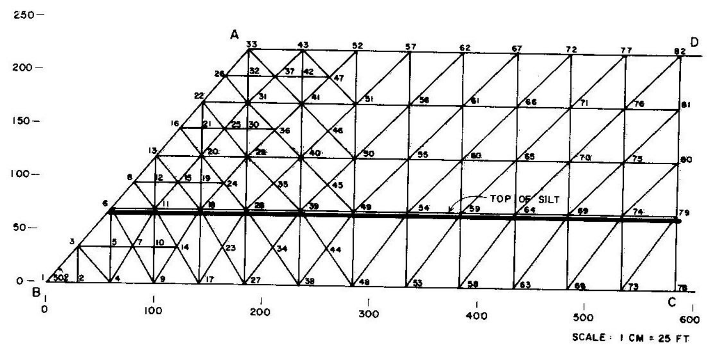

FIG. 3 FINITE ELEMENT REPRESENTATION OF EXISTING CLIFF SECTION

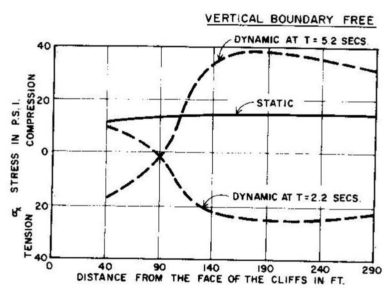

FIG. 4a STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT BELOW TOP OF CLIFFS.

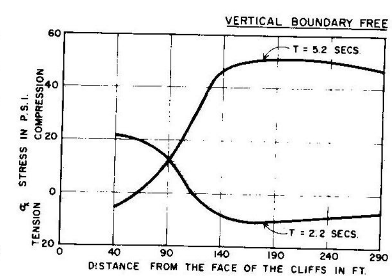

FIG. 4b COMBINED STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT BELOW TOP OF CLIFFS.

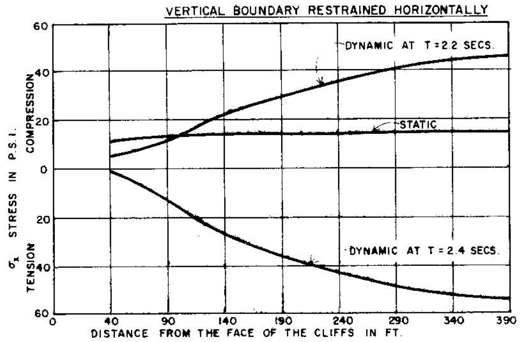

FIG. 5a STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT BELOW TOP OF CLIFFS.

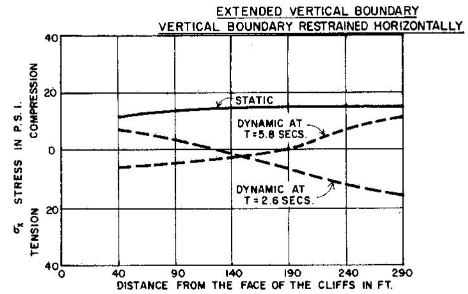

FIG. 6a STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT. BELOW TOP OF CLIFFS.

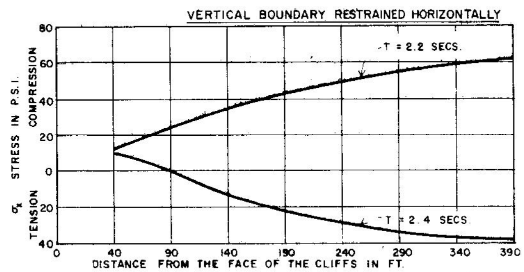

FIG. 5b COMBINED STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT. BELOW TOP OF CLIFFS.

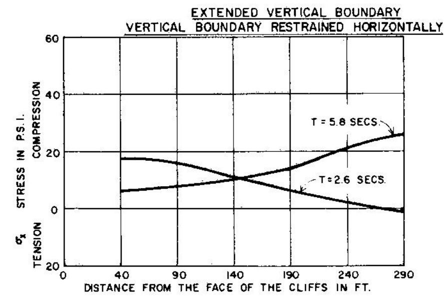

FIG. 6b COMBINED STATIC AND DYNAMIC STRESSES (σ_x) AT DEPTH 50 FT. BELOW TOP OF CLIFFS.

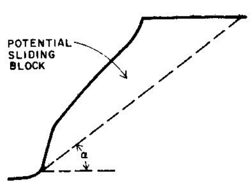

FIG. 7a POTENTIAL SLIDING WEDGE.

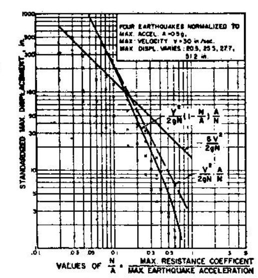

FIG. 7b STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (UNSYMMETRICAL RESISTANCE), AFTER NEWMARK (6)

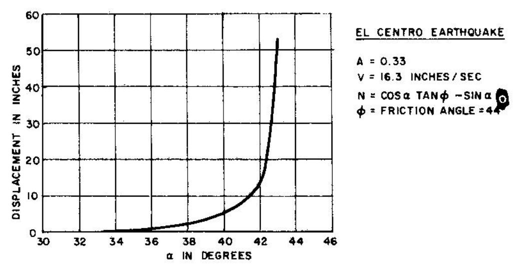

FIG. 7c RELATIONSHIP BETWEEN WEDGE ANGLE α AND DISPLACEMENT FOR EL CENTRO EARTHQUAKE.

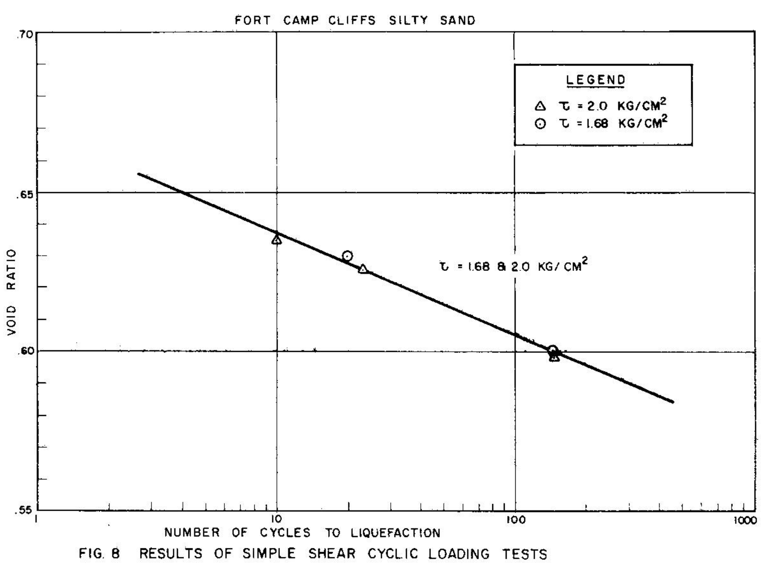

FIG. 8 RESULTS OF SIMPLE SHEAR CYCLIC LOADING TESTS

DISCUSSION OF PAPER NO. 9

SEISMIC STABILITY OF THE FORT CAMP SEA CLIFFS

by

W.D.L. Finn, P.M. Byrne and R.A. Spence

Question by: D.A. Sangrey

In your wedge analysis of the sea cliffs you have used a soil strength parameter of $\phi' = 44°$ and implicitly a drained condition.

Would it not be more appropriate to expect the soil to respond in an undrained mode under the short term earthquake loading and, if negative pore pressures result, to have a less critical situation than you describe?

Reply by: P.M. Byrne

The silty sand material (Fig. 1) which comprises most of the slope is above the water table and is not saturated. Hence, pore pressures developed during undrained shear should be insignificant. Therefore, the effective soil strength parameter was used.

The bottom 70 ft. of soil which is comprised of saturated interbedded silt and sand will undergo pore pressure changes during undrained shear. Since this material is very dense, it is likely that these pore pressure changes would be negative. It was considered prudent not to rely on negative pore pressure changes reducing the actual pore pressure below zero. Hence, if the pore pressure is assumed to be zero, the effective strength parameter would be appropriate.

Question by: J.H. Rainer

It would appear that particularly for the assumed sliding wedge mode of failure, vertical earthquake motions might become important. Has any consideration been given to the inclusion of these vertical motions?

Reply by: P.M. Byrne

For the sliding wedge mode of failure, the earthquake excitation was assumed to occur at an angle α to the horizontal (Fig. 7a). This was the assumption made by Newmark (1965). Therefore, both horizontal and vertical accelerations have been included in this analysis.

The assumption implies that the horizontal and vertical components of the earthquake are linearly related. This is not generally the case. However, the sliding wedge method of analysis is a very simplified approach to the problem and consequently a more sophisticated method of determining the earthquake excitation is hardly warranted.

Question by: S.R. Swanson

What will be done about the cliffs?

Reply by: P.M. Byrne

Many proposals have been put forward for stabilizing the sea cliffs. The current proposal is for a protective beach at the base of the cliffs to prevent wave erosion. In this proposal, it is anticipated that with the toe protected, the cliffs would eventually stabilize to an average angle of about 38°. At this angle the sliding block mode of failure would indicate only a small movement under an earthquake such as El Centro.