SEISMIC STABILITY OF A TAILINGS IMPOUNDMENT ON SOFT CLAYEY SILT DEPOSITS

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SUMMARY

The feasibility of building an 80ft high tailings impoundment dam directly upon soft sensitive clayey silt foundation soils in a highly seismic region is examined herein. The response of such material to earthquake loading is an area of some controversy and limited field experience. Laboratory tests indicate that this material will suffer a loss in both its stiffness and strength when subjected to cyclic loading. These losses are incorporated in analyses to show that the resulting earthquake induced displacements of the dam will be tolerable.

INTRODUCTION

It is proposed to construct an 80 ft. high impoundment dam across the valley at Greens Creek on Admiralty Island, Alaska. The foundation soils comprise of a surface fibrous peat underlain by up to 80 ft. of soft to firm clayey soils. The clayey soils form basically three layers: a firm clayey silt and gravel layer underlain by; a soft clay-silt layer underlain by; a clayey-silty gravel layer. An idealized soil section in the down valley direction is shown in Figure 1.

The static stability of an embankment dam on such a foundation will require significant drainage of the underlying soils. However, because the embankment will be constructed over a long period of time, such drainage may occur naturally or can be assisted to occur by a drainage system.

The proposed impoundment lies in a highly active seismic zone. An earthquake having a Richter magnitude of 7 and a maximum acceleration of 0.3g on rock was considered appropriate for design purposes.

It is proposed to remove the surficial peat and to construct the embankment dam directly upon the firm clayey-silt and gravel. The dam will comprise of compacted rockfill with a sloping upstream core of compacted glacial till as shown in Figure 2. The water table will be at the base of the dam. The purpose herein is to investigare the stability and deformation of the proposed impoundment under the design earthquake.

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THE EFFECT OF EARTHQUAKES ON EARTH STRUCTURES

An earthquake has basically two effects on an earth structure, 1) the shaking causes additional inertia forces on the structure, and 2) the shaking may cause the soil to lose a significant portion of its strength and stiffness. Most of the severe earthquake damage to earth structures has been caused by strength and stiffness loss rather than by the additional inertia forces. Saturated sands and non-plastic silts are most prone to such losses and their behaviour has been examined in detail by many researchers and is well summarized by Seed (1979). The behaviour of certain plastic silts and clayey soils under earthquake loading has recently caused concern, (Seed, 1982), and this will be addressed herein.

Slopes comprised of plastic silts and clayey soils have generally performed well during earthquakes. In particular, embankments comprised of compacted clayey soils have suffered virtually no damage during very severe earthquake shaking (Seed et al. 1978; Seed, 1979). The earthquake performance of soil structures or slopes founded upon soft sensitive clayey silts such as are present at Greens Creek is not as well established. Samples of naturally occurring clayey soils have been subjected to simulated earthquake loading in the laboratory by subjecting them to cyclic loading. The results indicate that such soils only suffer a significant strength loss when the cyclic strains induced are large and the soil is very sensitive and of low plastic limit, (Thiers and Seed, 1968, 1969; Castro and Christian, 1976; Koutsoftas, 1978; Anderson et al., 1980; Singh et al., 1981; Ishihara, 1981).

However, significant strains can develop and can be viewed as a reduction in shear modulus. The amount of modulus reduction depends on the level of the cyclic strain and could result in damaging post-earthquake movements of the structure. For highly sensitive or "quick" clays it is possible that earthquake loading could cause a complete loss in strength leading to a flow slide, and Massarsch (1980), gives examples of flow slides in such material which were initiated by shaking due to blast loading.

Based upon Chinese data, Seed and Idriss (1982) have suggested that clayey soils having the following characteristics may be vulnerable to significant losses in strength:

Percent finer than 0.005 mm < 15%;
Liquid limit < 35%;
Water content > 0.9 times the liquid limit.

They suggest that the best way to determine the dynamic properties of such materials should they plot above the A line, is by testing. Since the Greens Creek material does have these characteristics, it was tested.

LABORATORY TESTING PROGRAM AND TEST RESULTS

A testing program was undertaken to determine the behaviour of the clayey silt under simulated earthquake conditions. The material tested had a liquid limit of 28, a plastic limit of 17 and a natural water content in the tange 24-34%. The program comprised of 11 consolidated undrained triaxial tests. Five of these tests were conducted on isotropically consolidated samples and 6 on anisotropically consolidated ones. The purpose of anisotropic consolidation was not to simulate in-situ anisotropic consolidation, but rather to simulate the static driving force or bias created by construction of the dam and the stored tailings.

Two of the tests were conducted under static loading. The stress-strain curves for these tests are shown in Figure 3 and indicate that the material is plastic rather than brittle with no significant strength loss occurring for strains of up to 10 percent. The undrained strength ratio, $s_u/p \approx 0.33$, in which $s_u = (\sigma_1 - \sigma_3)/2$ at failure and p = the mean normal effective consolidation pressute.

The 4 cyclic tests were performed on isotropically consolidated samples at a cyclic stress ratio, $(\Delta\sigma_1)_{cy}/2p$, of 0.25. The number of cycles to cause 5% double amplitude or peak to peak axial strain was obtained for each test as shown in Figure 4. The tests indicate that 5 or 6 cycles of this stress ratio would result in 5% strain.

The post cyclic strength of each of the 4 tests was obtained and the results are also shown on Figure 4. They indicate that the post-cyclic strength ratio, $(s_u)_{pcyc}/p$ ranges from 0.15 to 0.33. The reason for the large scatter in post-cyclic strength is thought to result from the fact that the cyclic strain was not limited to 5% but that much larger cyclic strains actually occurred before the cyclic loading was stopped. In particular, test #4 which shows the lowest post-cyclic strength was subjected to a very large unknown strain and hence this test should be discounted. The lower limit of the post-cyclic strength ratio would then be 0.2. The very limited tested data indicates that for peak to peak cyclic strains in the range 5 to 10%, the post-cyclic strength ratio will likely exceed 0.2. Since the static strength ratio was 0.33 this means that the post-cyclic strength will likely exceed 60% of its static value. Such a strength reduction is in accord with data presented by Ishihara (1981) for clayey soils of low plasticity and zero static shear stress.

The reduction in post-cyclic strength as a function of single amplitude cyclic shear strain level for a number of clayey soils including the Greens Creek soil is shown in Figure 5 and is seen to be in accord with the general body of test data. It is seen that cyclic shear strains in excess of 1% are required to cause a significant reduction in srrength.

The reduction in secant shear modulus as a function of single amplitude cyclic shear strain level for these same soils is shown in Figure 6. It may be seen that a very significant drop in post-cyclic shear modulus may occur for cyclic shear strains of 1 percent.

The above data is appropriate for level ground conditons where there is no driving force or static bias. When there is a static bias present, there will be an accumulation of strain between each cycle as shown in Figure 7 where it may be seen that a strain accumulation of about 15 percent occurs after 20 cycles. The double amplitude strain, however, is still quite small being less than 2 percent.

The cumulative axial strain versus the combined static and dynamic stress ratio is shown in Figure 8 where it may be seen that no significant strain accumulation occurs until the combined stress ratio exceeds the static strength ratio 0.33. This suggests that the material is behaving mainly in an elastic-plastic manner with plastic strains occurring because the applied dynamic stress exceeds the static strength of the soil.

Because this is so, the cumulative strains will depend on the frequency of loading. It is more meaningful in this case to examine the cumulative strains in terms of cyclic <u>strain</u> rather than stress ratio as shown in Figure 9. It may be seen that for single amplitude cyclic shear strains below 0.15%, essentially zero cumulative strain occurs, while for cyclic strain above 1%, very large cumulative strains will result for the static bias in question.

PREDICTED RESPONSE OF EMBANKMENT

The response of the embankment was predicted using the following three approaches: a) The Newmark Method, b) The Dynamic Stress and Strain Path Approaches, and c) A Nonlinear Dynamic Analysis.

a) Newmark Method

Newmark (1965) presented a simple method of predicting the earthquake induced displacement of a slope based upon a single degree of freedom rigid plastic system. If it is assumed that the soil has a strength corresponding to $s_u/p = 0.33$, its static value, which seems reasonable based on Figure 8, the computed displacement is 13 in. If, on the other hand, based on the isotropic consolidation tests, the strength ratio was to drop to $s_u/p = 0.20$, then an infinite displacement is computed. It will be shown later that such a strength drop is not likely to occur.

b) The Dynamic Stress and Strain Path Approaches

The dynamic stresa path approach was devised by Seed and his coworkers over the past 20 years and is described in a number of papers including Seed & Idriss (1982). Basically, the computed dynamic stresses are applied to representative samples and the cumulative strains observed. These strains are then used to compute the earthquake induced displacements. Herein an equivalent viscoelastic dynamic analysis was used to compute the dynamic stresses. These in turn were divided by the mean normal stress, p, to form the equivalent cyclic stress ratio $\tau_{dy}/p \approx 0.3$. When this is added to the static stress ratio value of 0.2 caused by the tailings, a static plus dynamic stress ratio of 0.5 is obtained. Based on Figure 8, such a stress ratio would cause unlimited strains.

If the material is indeed elastic-plastic as it appears to be, such large dynamic stresses would not in fact occur during an earthquake. Instead, the material would yield plastically when the stress reached the strength of the soil. The increment of strain for the elastic-plastic system would be about the same as for the equivalent elastic system. This assumption is commonly made in earthquake analysis of steel and concrete structures. If this is so, then the oscillating strains rather than stresses obtained from the equivalent viscoelastic analysis are appropriate. An equivalent shear strain, $\gamma = 0.3\%$ was computed. Based on Figure 9, ten cycles of such a shear strain would result in an accumulated strain of 0.8%. Based on an 80 ft. thickness of foundation soil, such a strain would lead to a downslope movement of 8 in. This approach, in which the cyclic strains rather than stresses are applied to the sample, is called the strain path approach.

The predicted low cyclic shear strain value of 0.3% will not cause the soil to lose significant strength (Fig. 5) and hence the assumption of no strength loss for the Newmark analysis is appropriate.

A better estimate of the earthquake induced displacements can be obtained from a static finite element analysis in which a reduced modulus is used to allow for the earthquake induced strains (Byrne and Janzen, 1981). The original and deformed pattern of the dam is shown in Figure 10. Displacements are seen to be in the range of 1/2 ft.

c) Nonlinear Analysis

A truly nonlinear dynamic analyais using an elastic plastic stress-strain law with a strength corresponding to $s_u/p = 0.3$ was performed. The cumulative strain in elements was found to be about twice the cylic strain, and the maximum displacement was about 6 in.

CONCLUSIONS

The earthquake response of a tailings impoundment to be founded upon a soft sensitive clayey silt is examined herein. Laboratory test results indicate that the material behaves in an elastic-plastic manner to cyclic loading in the presence of a static bias. For such a material, it is suggested that the Seed dynamic stress path approach may be inappropriate. The Newmark approach, together with the dynamic strain path approach and a truly nonlinear analysis, indicate that the earthquake induced displacements will be less than 1 ft. Such displacements are quite acceptable for a tailings impoundment.

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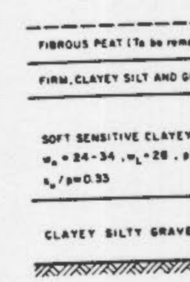
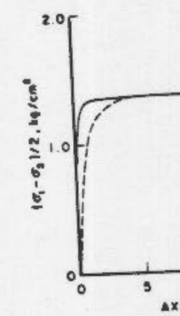


Figure 1.



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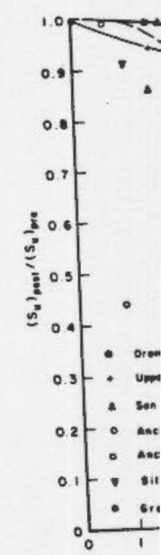

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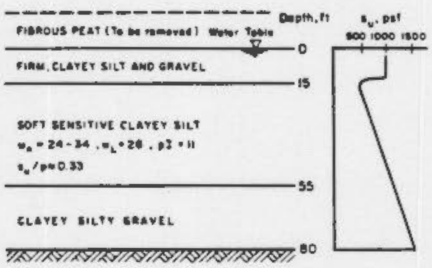

Figure 1. Soil Profile

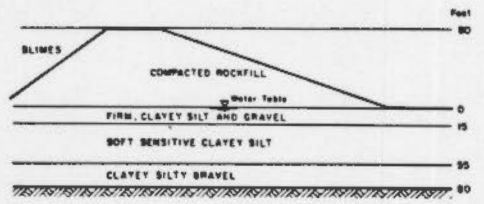

Figure 2. Profile of Tailings Impoundment.

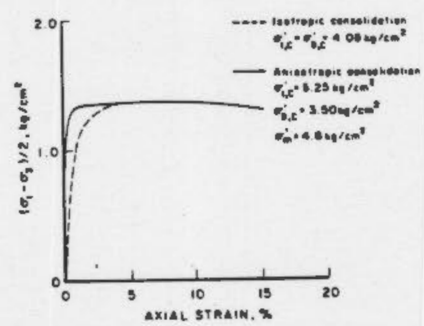

Figure 3. Stress-Strain Relations for Static Loading.

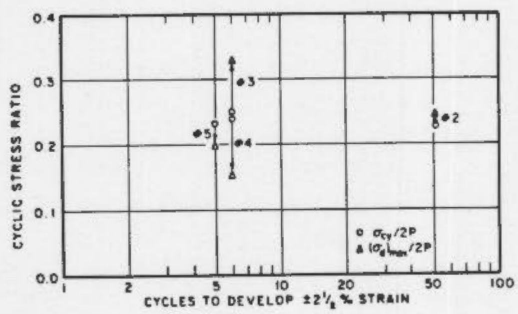

Figure 4. Dynamic and Post-Cyclic Resistance Ratios (Isotropic Consolidation)

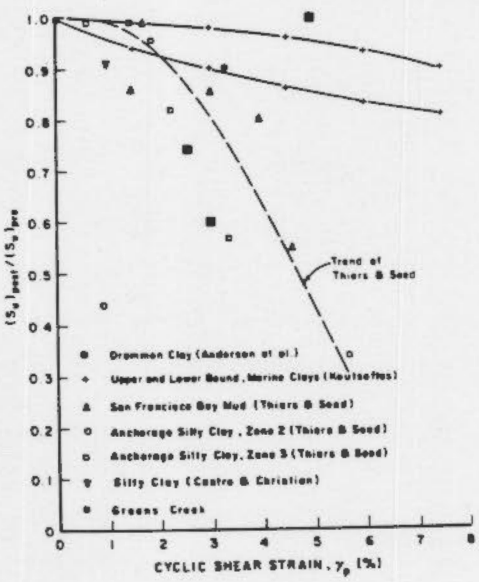

Figure 5. Strength Reduction Versus Cyclic Shear Strain (Isotropic Consolidation).

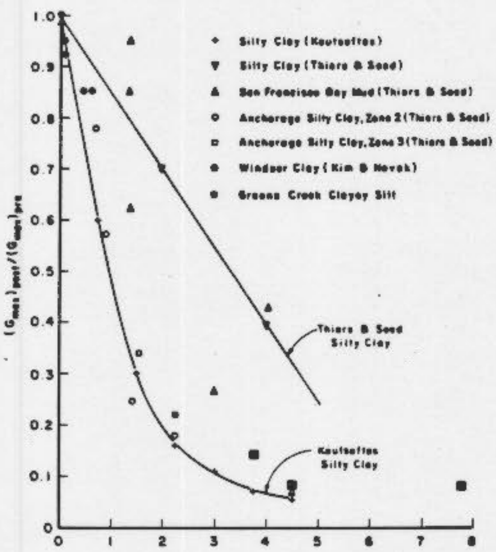

Figure 6. Modulus Reduction Versus Cyclic Shear Strain (Isotropic Consolidation).

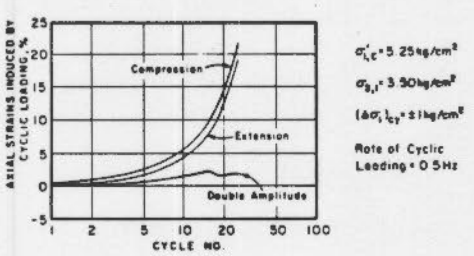

Figure 7. Cumulative Strain Versus Cycles of Stress (Anisotropic Consolidation).

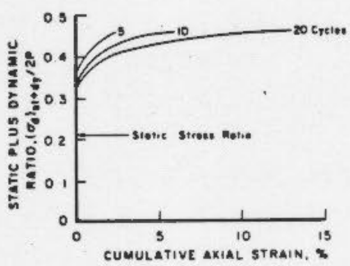

Figure 8. Cumulative Strain Versus Static Plus Dynamic Stress Ratio (Anisotropic Consolidation).

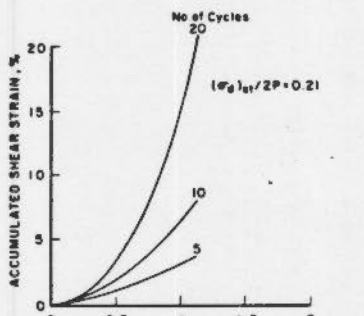

Figure 9. Cumulative Strain Versus Cyclic Strain (Anisotropic Consolidation).

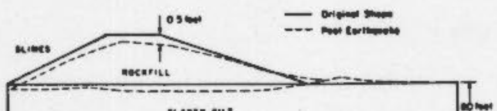

Figure 10. Earthquake Induced Displacement From Finite Element Analysis.

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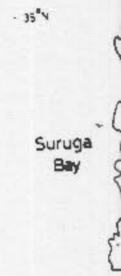

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