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Seismic soil behaviour under an earth dyke

Comportement sismique du sol sous une digue de terre

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SYNOPSIS In situ and laboratory tests on a Champlain Sea clay, both under and outside an earth dyke, showed that the weight of the dyke has changed the sensitive clay into a normally consolidated clay, with an increase in static undrained strength and a decrease of sensitivity.

Dynamic soil behaviour was studied using the seismic crosshole technique, the resonant column device and the cyclic triaxial apparatus. The seismic crosshole technique and the resonant column device yielded the same value of initial modulus. Cyclic triaxial tests showed that unsymmetrical cyclic shear led to a compressive failure; the strength was 90% higher than that of a symmetrical cyclic shear that led to an extension failure. A deformational model was proposed to characterize the observed soil behaviour.

INTRODUCTION

Numerous earth-structures such as dams, dykes and highway embankments have been built in parts of Eastern Canada on soft sensitive marine clay foundations. Some of these structures are located in seismically active areas, such as the Ottawa and St. Lawrence valleys. The safety and serviceability of these structures during and after earthquakes are of concern to the public as well as to the geotechnical profession.

The seismic analysis of an existing earth-structure requires a knowledge of the behaviour of the subsoil, which may have undergone consolidation under a loading condition unique to the structure and its age. To obtain such information, it is desirable to conduct in situ tests or laboratory tests on high quality samples taken from under the structure. At present, however, no such test programs have been reported in the literature.

The research program recorded in this paper includes in situ and laboratory experiments to evaluate the dynamic behaviour of the clay under an existing earth dyke. Both strength and deformation characteristics have been studied. The results are useful to the assessment of the safety and deformability of the dyke during earthquake conditions.

SITE DESCRIPTION

The Champlain Sea clay tested in this study was formed by fluvial deposition in the salt water intrusion of the Ottawa and St. Lawrence river valleys, during glacial times. An earth dyke approximately 10 m high and 200 m wide, including berms, covers the marine clay at this site. The 13 m thick marine clay deposit has

been confined between the dyke fill and underlying bedrock for at least two decades. Eight to nine metres of water are retained behind the dyke. The surrounding area is relatively flat and low lying. Seismic events in this region in recent times have been estimated at between five and six on the Richter scale.

TEST PROGRAM AND PROCEDURES

Field work

The field work consisted of sampling and vane shear tests in the subsoil both under and outside the dyke. Seismic crosshole tests were carried out under the dyke.

Undisturbed soil samples were taken using a 127 mm Osterberg piston sampler (Osterberg, 1952) and the Norwegian Geotechnical Institute's (NGI) vane apparatus was used in obtaining the in situ shear strength.

The seismic crosshole survey technique was used to measure the dynamic properties of the marine clay below the dyke. Three boreholes were arranged in a linear array, equally spaced about 2.9 m apart. Two holes acted as listening holes, housing 14 Hz vertical geophones. The third, end hole was used to generate seismic body waves. The Osterberg sampler embedded approximately 0.6 m into undisturbed marine clay acted as the source of body waves, produced by hitting a sledgehammer at the top end of the rod attached to the sampler. This generated a signal rich in shear waves. The listening holes were lined with plastic 70 mm outside diameter inclinometer casing grouted in place with a cement and bentonite mixture.

Verticality surveys were performed on the holes to determine the horizontal interhole distance at depth. Body wave arrivals were recorded using a signal enhancement seismograph. Shear and compression wave velocities were determined from seismograph records of arrival times and interhole distances.

Laboratory study

The laboratory study was conducted on the Osterberg soil samples taken from the site. The testing consisted of routine tests, static and cyclic triaxial tests, and resonant column tests. The results are used for determining the soil profile and for studying the strength and deformation characteristics.

Standard methods were used to measure the Atterberg limits, density, natural moisture content and consolidation behaviour. The consolidation tests were carried out on samples trimmed to an area of 20 cm² and 2 cm thick. The load-increment ratio was 0.5.

Static triaxial tests were carried out by means of the NGI cell. A rotating bushing was used to reduce friction on the piston for applying load on the soil sample. Each sample was 80 mm high and about 36 mm in diameter. This size permitted four adjacent samples to be trimmed from a single section of the Osterberg sample, thus minimizing variability in the test results. Undrained strengths were obtained with an isotropic consolidation pressure of 150 kPa, which was the mean of the effective vertical and effective horizontal in situ pressures. A coefficient of earth pressure at rest (K_0) of 0.5 was used in computing the horizontal stress. Some limited K_0 - anisotropically consolidated undrained tests were also carried out. Tests at other consolidation pressures were also performed to determine the strength envelope.

The cyclic triaxial test was performed using an apparatus built in the Division of Building Research, National Research Council of Canada. The main components are a cyclic triaxial cell, an electric-to-pneumatic (E/P) transducer and a function generator, shown in Figure 1.

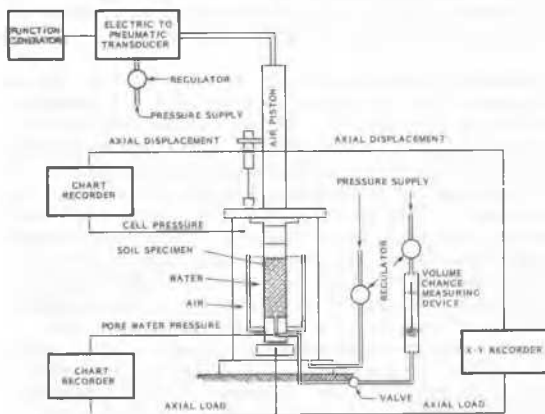


Figure 1. Cyclic triaxial test apparatus

The cyclic triaxial cell uses air as the cell fluid for applying confining pressure to eliminate the inertia effect that is imposed on the piston for cyclic loading if water is used. The soil sample, however, is surrounded by water to avoid air diffusion through the rubber membrane. This is achieved by attaching a hollow plexiglass cylinder to the base pedestal, which sits on a load cell inside the triaxial cell. A pressure transducer is used to measure the pore pressure at the bottom of the sample.

The cyclic stresses, both compression and extension, are applied axially by means of an air piston, with the air pressure supplied from the E/P transducer. This device accepts an electrical signal from the function generator and puts out a pressure in proportion to the electrical signal. The function generator can develop different shapes, frequency and amplitude. For the tests reported herein, a sine function at 1 Hz has been used.

Cyclic tests were carried out only on soil specimens from under the dyke. The specimens were of the same size and consolidated isotropically and anisotropically to the same in situ effective stresses as the static undrained triaxial specimens. Different amplitudes of cyclic shear were applied to investigate the strength and deformation characteristics of the soil. In the isotropically consolidated test, the cyclic shear was symmetrical about zero reference stress, while in the anisotropically consolidated test, the cyclic shear was unsymmetrical.

A Hardin resonant column (Hardin and Music, 1965), modified to immerse the sample in water and to measure volume change during consolidation, was used in these tests. Samples were consolidated in the device for a period of approximately 1200 minutes at the mean in situ pressure of 150 kPa prior to testing. Torsional forces of increasing strength were applied incrementally to each sample to determine shear modulus and damping at average peak shear strains ranging from about 10⁻⁴% to 10⁻¹%. Damping was determined by the log decrement method.

TEST RESULTS AND DISCUSSION

Routine tests

The results of routine tests are summarized in Figures 2 and 3. The soil profile outside the dyke (Figure 2) is typical of a soft Champlain Sea clay with average sensitivity of about 30 and liquidity index exceeding 1. The soil is lightly overconsolidated, with the average ratio of field vane strength (S_u) to the preconsolidation pressure (p_c) equal to 0.34. The soil profile under the dyke (Figure 3) is slightly different. The soil is more plastic and has a slightly higher natural moisture content in spite of consolidation under the weight of the dyke. The original moisture content before construction must have been higher because the consolidation effects

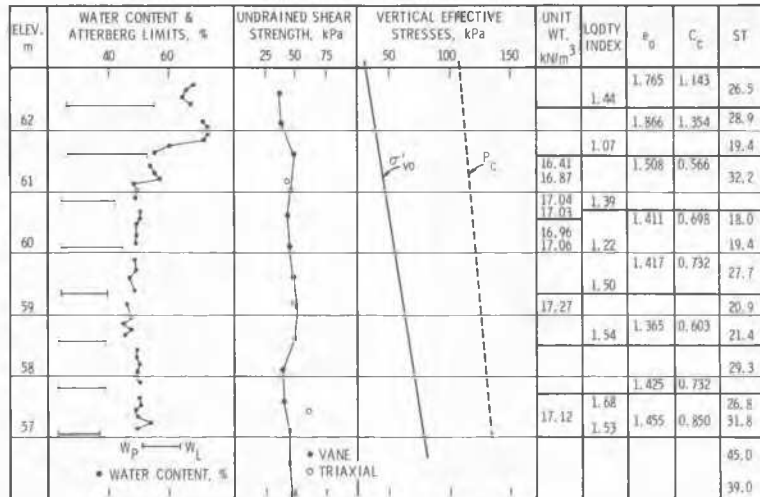


Figure 2. Soil profile outside dyke

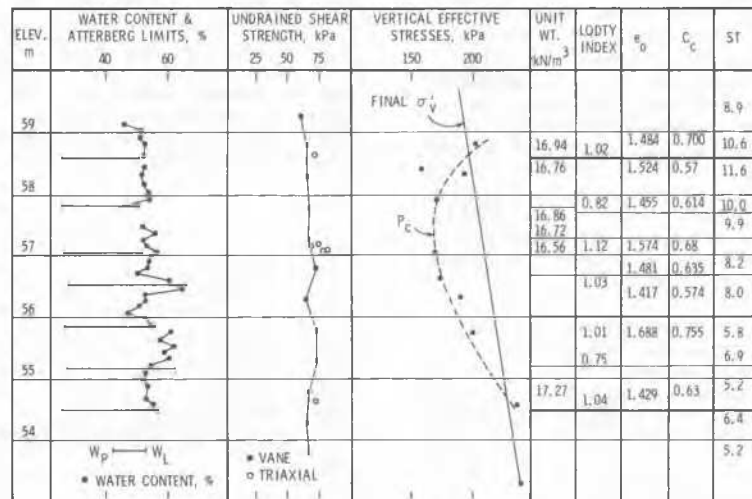


Figure 3. Soil profile under dyke

are clearly evident from other test results such as the significant decrease of liquidity index and sensitivity, and the substantial increase in shear strength and preconsolidation pressure. The values of p_c follow a parabolic distribution under the dyke and are, in general, lower than the computed final effective vertical stress. Some excess pore water pressure, therefore, may still exist and further consolidation settlement and strength gain may be anticipated.

With the change of soil behaviour under the dyke, these tests conducted in situ at the dyke or on samples taken from under it, are important in order to study its performance during earthquake conditions.

Seismic crosshole tests

Shear wave velocities (V_s) were extremely consistent in the marine clay under the earth

dyke. In eight tests, performed at depths ranging from 16.1 to 21.2 m, V_s values (shown in Table I) ranged from 148 to 162 m/s and averaged 153 m/s. An average shear modulus (G_0) of 40.5 MPa was obtained by substituting the average bulk density (γ) value of the marine clay (1747 kg/m³) and the average shear wave velocity into the equation:

$$G_0 = \frac{\gamma}{g} (V_s)^2 \tag{1}$$

where g = acceleration due to gravity. For this clay deposit, the ratio of shear modulus (G_0) to undrained shear strength (S_u) is about 600.

Compression wave velocities (V_p), also shown in Table I, average 1492 m/s. These values indicate that the clay is nearly 100%

TABLE I. Shear Wave and Compression Wave Velocities in Marine Clay Under the Earth Dyke

Test depth (m)	Shear wave velocity V_s (m/s)	Compression wave velocity V_p (m/s)
16.1	153	1694
17.1	151	1400
17.9	151	1339
18.6	149	1400
19.1	148	1620
19.9	153	1620
20.5	155	1400
21.1	162	1466

saturated, since compression wave velocity in water is 1524 m/s.

Static triaxial and field vane tests

The triaxial undrained strengths are also plotted in Figures 2 and 3 along with the vane strengths. The triaxial undrained strength consolidated to the in situ pressure system is in close agreement with, or slightly higher than, the vane strength for soils both under and outside the dyke. Such an agreement is also reported for other sites before any man-made loading is applied (Bozozuk and Leonards, 1972; Bozozuk, 1977); however, agreement is found only for those structures with a wide base, as in this case (Law, 1985). For those with a narrow base, a discrepancy stemming from a lack of vane strength increase has been noted (Law et al, 1977).

The strength envelope in terms of effective stresses is shown in Figure 4. There is essentially no difference between soils from under and outside the dyke. A similar observation was found for the Gloucester clay near Ottawa, Canada (Law et al, 1977).

Cyclic triaxial tests

The cyclic triaxial strength is defined as the maximum shear (S_c) at failure and is given by:

$$S_c = \tau_s + \tau_c \quad (2)$$

$$\text{where } \tau_s = (\sigma_{vc}' - \sigma_{hc}')/2 \quad (3)$$

and σ_{vc}' , σ_{hc}' = consolidation pressures in the vertical and horizontal directions respectively;

$$\tau_c = \sigma_d/2$$

σ_d = cyclic deviatoric stress (single amplitude) in excess of static deviatoric stress.

Failure is defined as the development of failure plane or a 10% double amplitude strain, whichever comes first.

The mode of failure in an isotropically consolidated undrained and symmetrical cyclic shear (ICUSC) test is very different from that in an anisotropically consolidated undrained and unsymmetrical cyclic shear (ACUUC) test. The strain time plot for a typical ICUSC test is shown on Figure 5. In the first few cycles, the strain is symmetrical, as is the applied cyclic shear. Upon further straining, however, extension strain exceeds compression strain, indicating the presence of modulus anisotropy, with the compressional modulus stronger than the extension modulus. This leads to a failure under an extension state. The strength of an ICUSC test, therefore, corresponds to an extension strength.

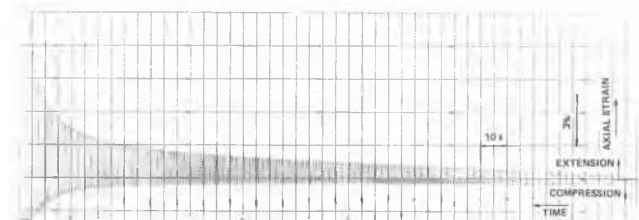


Figure 5. Strain-time plot of an ICUSC test

The strain-time plot of an ACUUC test is shown in Figure 6. The total shear stress ($\tau_c + \tau_s$) did not change in direction in this test. It remained in the state of passive shear and the resulting strain cycled exclusively in the compression domain. This leads to a compressional mode of failure, yielding a higher cyclic strength than that of the ICUSC test. The average ACUUC strength is about 90% higher than the ICUSC strength for this clay. Observations of unsymmetrical cyclic strength higher than symmetrical cyclic strength have also been reported by Seed and Lee (1969) and Lee (1979).

Figure 7 shows the decrease of ICUSC and ACUUC strengths with respect to N_f , where N_f is the number of cycles to failure. Both strengths are normalized by the static isotropically consolidated undrained triaxial compression

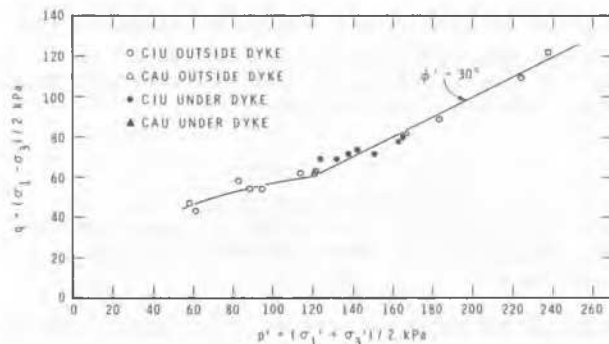


Figure 4. Strength envelope from static triaxial tests

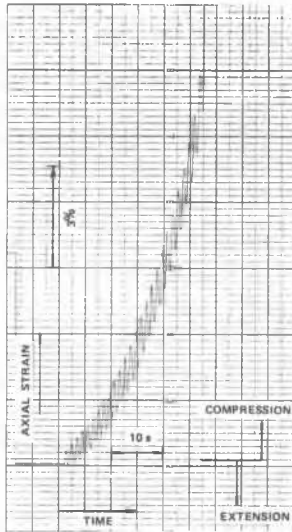


Figure 6. Strain-time plot of an ACUUC test

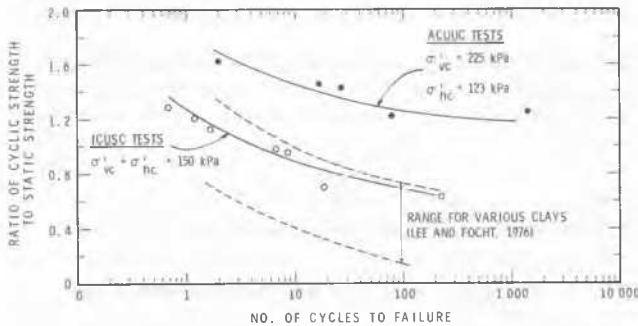


Figure 7. Triaxial cyclic strength for soil under dyke

strength. For comparison with existing information, the range of results for many soils summarized by Lee and Focht (1976) is shown on the same figure. This range is for saturated clays failing under symmetrical cyclic shear. Both strengths of the present clay seem to follow the general pattern of other clays, with the ICUUC strength lying close to the upper limit and the ACUUC above it. At $N_f > 4$, the ICUUC strength starts to drop below the static strength, while the ACUUC strength is higher than the static strength throughout the entire range of N_f values observed in the experiments.

The question now is which cyclic strength should be used for design or analysis. More common in earthquake loading is the simple shear condition, which is different from the compression or extension condition. For clays, there are very few comparisons between cyclic shear strength and cyclic triaxial compression or extension strength. The results of Thiers and Seed (1969) showed, for San Francisco Bay mud, that there is no difference between simple

shear cyclic strength and unconsolidated undrained cyclic triaxial strength of specimens trimmed at 45° from the vertical. Both strengths, however, are about 20% higher than the cyclic strength of specimens trimmed in the vertical direction, the usual direction for the majority of tests recorded in the literature. Lee's (1979) work on a Champlain Sea clay shows that the ICUUC strength is higher than the simple shear cyclic strength at $N_f < 10$ and the opposite is true at $N_f > 10$. The ACUUC strength is invariably higher than the simple shear cyclic strength. There is a need for further research along this line.

For the time being, the pseudo-static analysis coupled with Seed's (1979) criterion (Table II) may be applicable to the present case. The criteria are based on acceptance of limited deformation and a crest acceleration less than 0.75 g. The unsymmetric dynamic strength due to earthquake loading is assumed not to drop below 85% of the static undrained strength, a condition which is satisfied with the present soil.

TABLE II. Design Criteria for Pseudo-Static Analysis (after Seed, 1979)

Earthquake magnitude	Design criteria
6½	FS = 1.15 for seismic coefficient = 0.1
8½	FS = 1.15 for seismic coefficient = 0.15

Resonant column tests

Six tests were performed on material obtained from below the dyke and seven tests on material from outside the dyke. The shear modulus (G_r) and the average peak shear strain (γ_r) were deduced using the method by Drnevich et al (1978). The results for the low amplitude tests are shown in Table III; the marine clay from under the dyke had a mean initial modulus (G_{ro}) of 40.0 MPa at a mean γ_r value of 0.0066%. This compares well with the field crosshole test value of 40.5 MPa at a comparable strain level. Such a good agreement indicates the high quality of the Osterberg samples. Samples from outside the dyke, consolidated to the same 150 kPa confining pressure for the same length of time, have a G_{ro} value of 36.5 MPa at a mean γ_r value of 0.0067%. This G_{ro} value is within 10% of that for soil from under the dyke.

Figure 8 shows the change in shear modulus and damping with increasing shear strain. In the strain range from 0.006% to 0.2%, normalized shear modulus values decrease about 40%. Damping increases from 1.75% to about 5.25% in the same strain range. No difference in behaviour is noted between samples from under

TABLE III. Resonant column test results

Depth (m)	Shear modulus (MPa)	Average peak shear strain (γ_r) $\times 10^{-3}\%$
(A) Below dyke samples		
18.1	42.8	7.5
18.5	40.5	6.8
19.7	44.6	6.3
20.0	37.5	7.1
21.1	34.7	6.3
21.7	40.1	5.7
(B) Outside dyke samples		
4.2	32.9	5.8
4.8	37.0	6.7
5.6	36.3	6.4
6.6	37.9	6.8
7.2	36.5	7.5
8.1	38.2	6.9
8.9	36.5	6.8

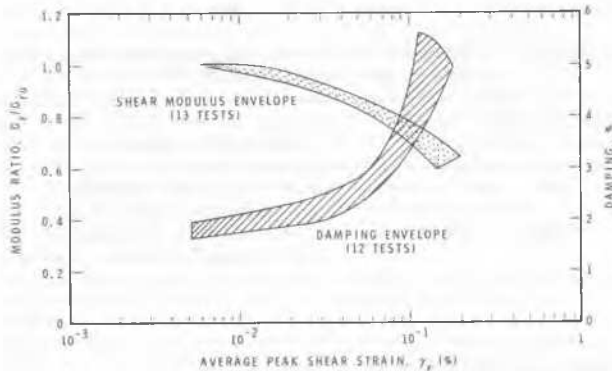


Figure 8. Resonant column tests

and outside the dyke. All results fell within the envelopes shown in Figure 8.

Deformational characteristics

A stress-strain model is required for the analysis of deformation or serviceability of the dyke during earthquake loading. A model is used here to describe the characteristics displayed in the resonant column and the symmetrical cyclic shear (ICUSC) tests. There are three aspects to this model: 1) first cycle modulus (G_1); 2) modulus degradation; 3) formation of residual strain. The definition of symbols is shown in Figure 9.

The first cycle shear modulus (G_1) can be represented by a Ramberg-Osgood model (Richart, 1975). Expressed in terms of cyclic deviatoric stress (σ_d) and static undrained strength (S_u) the model can be written as:

$$\frac{G_0 - G_1}{G_1} = \alpha \left| \frac{\sigma_d}{2S_u} \right|^{R-1} \quad (4)$$

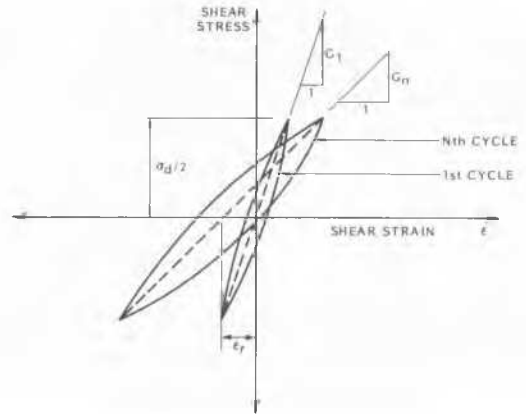


Figure 9. Symbols used in the proposed model

where R and α are parameters of the model and G_0 is the initial shear modulus, which can be obtained in this case either from the seismic crosshole or the low amplitude resonant column test, since both give the same value.

For the resonant column test, the term $\frac{\sigma_d}{2}$ is given by:

$$\frac{\sigma_d}{2} = G_r \times \gamma_r \quad (5)$$

If the model applies, a plot of $\log \frac{G_0 - G_1}{G_1}$

versus $\log \left| \frac{\sigma_d}{2S_u} \right|$ will yield a straight line

with the intercept equal to $\log \alpha$ and the slope to $R - 1$. Figure 10 shows the test results plotted in this manner and a straight line seems to provide a reasonable fit. α is 13.69 and R is 6.17.

Modulus degradation is the term describing the decrease of modulus during cyclic loading. It may be attributed to pore pressure increase with strain and the deterioration of the clay structure during cyclic straining. This degradation has been observed and studied (Aisiks and Tarshansky, 1969; Castro and Christian, 1976). For San Francisco Bay mud sheared at low cyclic strain level ($\pm 0.4\%$), Idriss et al (1979) proposed that:

$$\frac{G_n}{G_1} = N^{-t} \quad (6)$$

where G_n is the shear modulus at the N th cycle and t is the degradation parameter. While the degradation at such a low strain level has not been studied for the present soil, a different relation is proposed here for the degradation at higher cyclic triaxial strain levels ($\pm 0.1\%$ to $\pm 2\%$):

$$\frac{G_n}{G_1} = 1 - \beta \log N \quad (7)$$

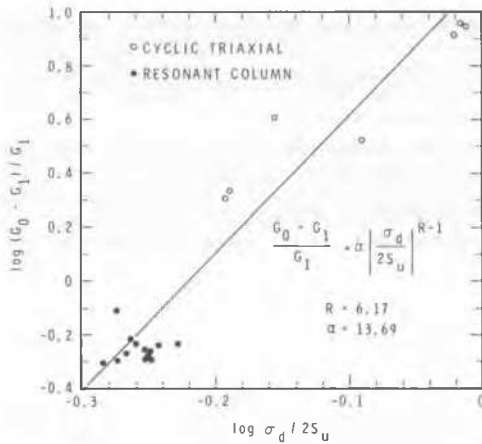


Figure 10. Determination of deformation model parameters from cyclic triaxial and resonant column tests

where β is a degradation parameter. For strain level exceeding $\pm 2\%$, failure is imminent and a stability consideration is more appropriate.

β for the given test series ($\sigma'_c = 150$ kPa) is a function of the cyclic stress level. The empirical relationship can be approximately given by:

$$\beta = 1.57 \left| \frac{\sigma_d}{2S_u} \right| - 0.63 \quad (8)$$

The residual strain (ϵ_r) is the average value between the maximum and minimum strains during a cycle of loading. It is a measure of the strain shift of the centre of hysteresis loop (Figure 9) from the origin, i.e., the permanent deformation after the cyclic shear stops. The amount of residual strain depends on soil type, stress level, number of cycles and testing method. The value of ϵ_r is theoretically zero for a cyclic simple shear applied along a plane free of initial or static shear. Some non-zero values, however, were noted by Moriwaki et al. (1982) partly because of the difficulty of maintaining perfect cyclic stress symmetry during the experiment. In the ICUSC test, higher ϵ_r will be expected because of straining in an anisotropic material, as discussed earlier. Ideally the test should be conducted with proper recognition of the direction of principal stresses in the field with respect to the direction of the seismic shear. For the triaxial cyclic test data a simple approximate relationship is:

$$\epsilon_r = AN^B \quad (9)$$

where A and B are parameters depending on soil type, stress level, and consolidation stresses. More data are required to clearly determine the values. A similar relationship was also observed by Marr and Christian (1981).

SUMMARY AND CONCLUSIONS

In situ and laboratory tests were conducted on a Champlain Sea clay under and outside an earth dyke. The in situ work consisted of seismic crosshole tests, field vane shear tests and soil sampling. The laboratory tests included determination of Atterberg limits, together with consolidation, static triaxial, cyclic triaxial and resonant column tests.

The results show that the weight of the dyke has caused a significant consolidation in the clay, changing it into a normally consolidated clay. Associated with this change are an increase in undrained shear strength and a reduction of sensitivity. The static strength parameters, in terms of effective stress, however, remain unchanged under the dyke.

The majority of the dynamic tests were conducted on clay from under the dyke; the results can be summarized as follows:

- (1) The laboratory resonant column tests yielded an initial shear modulus of 40.0 kPa, which is the same as the value obtained in the field using the seismic crosshole technique. The ratio of initial shear modulus to undrained shear strength is 600. When consolidated under the same pressure, the clay from outside the dyke yields similar results in shear modulus and damping ratio.
- (2) The 127 mm diameter Osterberg sample produces excellent quality samples for study of dynamic soil behaviour.
- (3) An extension failure mode prevails in the isotropically consolidated, symmetrical cyclic triaxial (ICUSC) tests, while compression mode of failure predominates in the anisotropically consolidated, unsymmetrical cyclic (ACUUC) tests. As in the static case, in which the compression strength generally exceeds the extension strength, the ACUUC strength is 90% higher than the ICUSC strength.
- (4) A model is proposed to describe the deformational characteristics of the soil. The model has three components to deal with 1) the variation of first cycle modulus with stress or strain level, 2) modulus degradation with number of stress cycles, and 3) residual strain or permanent strain with stress cycles.

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