INVESTIGATION OF CYCLIC STRESS-STRAIN CHARACTERISTICS OF GRAVEL MATERIAL

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SUMMARY

Cyclic triaxial tests are reported for two gravel materials with sample diameter 150 mm permitting 30 mm maximum grain size. The usefulness of stress path diagrams to describe undrained strength behaviour is illustrated for a saturated crushed limestone (rockfill). A tentative method of normalizing the cyclic deviator stress at failure is presented, which allows a simplified evaluation of test data and a reduction of the testing program. Also presented are the stiffness and damping characteristics of an alluvial gravel from the site of a nuclear power plant. Since the samples were not saturated the cell pressure was cycled so as to maintain a constant mean principal stress.

INTRODUCTION

Coarse granular materials, e.g. natural gravels or crushed rock, play an important role in the field of geotechnical engineering. They may be used to provide a stabilizing shell for dams or they may act as a foundation for important structures such as nuclear power plants. In comparison to sands, for which a wealth of experimental data has been accumulated over the past two decades, little data is available regarding the cyclic shear resistance and cyclic deformational properties of gravelly soils. There is a need to obtain such data since those cases in which a dynamic loading analysis is required often involve this type of material. Furthermore, in the case of a dam the shells constitute a major portion of the volume of the structure.

The major difficulty associated with the testing of gravel is to be able to test a sample large enough to be representative of the in-situ material, since restrictions of the testing equipment often limit the maximum grain size in the sample. Consequently, the grain size distribution curve of the sample differs from that of the material in-situ. This will inevitably lead to some differences in the strength and deformation properties. Grain crushing also depends, among other factors, on grain size and the laboratory samples will underestimate this effect (1), which will also influence strength and deformation properties.

The present contribution is concerned with cyclic triaxial tests on two gravel materials, the first for a rockfill dam and the second for the site of a nuclear power plant. For the dam material the tests were carried out under undrained conditions. This gives conservative results. To judge if this assumption is too conservative a separate analysis of pore pressure dissipation has to be carried out (2).

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DESCRIPTION OF TEST EQUIPMENT

A general view of the test equipment is shown in Fig. 1. The equipment consists essentially of three parts, i.e. a triaxial cell, a load generating device with an electronic regulation and recording equipment. The axial cyclic loading is generated by a hydraulic system whereby the theoretical pulse shape, frequency and amplitude can be regulated. A facility of cycling the cell pressure to maintain a constant mean principal stress has been

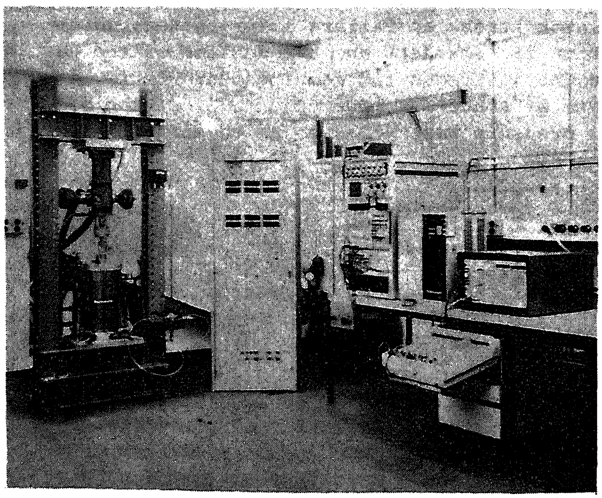

Fig. 1

General view of the cyclic triaxial test equipment.

added to the system. The change in cell pressure is of opposite sign to the axial deviator stress σ_{dc}, its amplitude equalling $\sigma_{dc}/3$. The rate of loading was fixed to 1 and 0.2 Hertz respectively in the two test series. Frequency effects did not form part of the investigation. Preliminary tests showed for this class of material a negligible influence of frequency. The lower frequency enabled more accurate regulation of cell pressure.

DESCRIPTION OF MATERIALS AND SAMPLE PREPARATION

The cyclic strength tests were carried out on a partly weathered crushed limestone, used as a shell material for a dam in Central America. The other material tested was an alluvial gravel deposit at the site of a nuclear power plant, for which site amplification and soil-structure interaction studies were carried out.

The materials were separated into several fractions and prepared to fulfil the given gradation and density requirements. Compaction was carried out in such a way as to prevent further break-down and crushing of the material. During testing some crushing is unavoidable. The grain size distribution curves and other relevant material data are shown in Fig. 2.

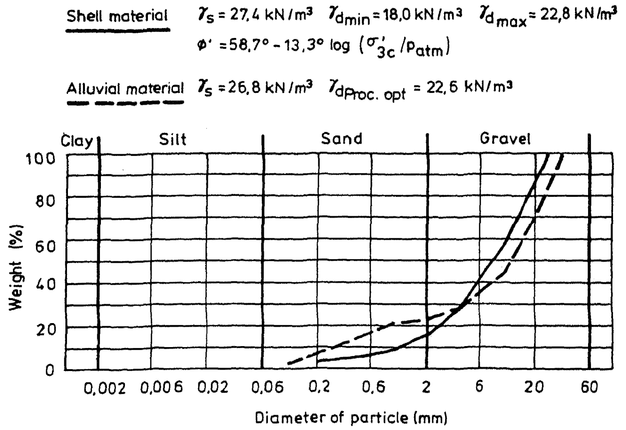

Fig. 2

Grain size distribution curves for shell and alluvial material for cyclic laboratory testing program.

For the strength tests care was taken to ensure that the samples were completely saturated by means of sample preparation under water and where necessary using back pressure until the B-value equalled at least 0.97. The sample size was 300 mm in height and 150 mm in diameter, allowing a maximum grain size of about 30 mm.

CYCLIC SHEAR RESISTANCE

In general, the state of static stress in a rockfill dam is anisotropic. Thus the cyclic triaxial test program has to be carried out for various K_c-values, where K_c is the ratio of static principal stresses $\sigma'_{1c}/\sigma'_{3c}$. Typical test results are shown in Fig. 3.

A static finite element analysis had shown that locally higher values of K_c may exist. The limiting value is given by the static strength of the material, which for $\Phi' \simeq 50°$ gives a value of $K_c \simeq 7$.

The results of the cyclic loading tests are best illustrated by means of stress-path p'-q diagrams, where $p' = 1/3(\sigma'_1 + \sigma'_2 + \sigma'_3)$ and $q = \sigma_1 - \sigma_3$. Anisotropic consolidation conditions correspond to the initial stress coordinates p'_o, q_o. If at the end of cyclic loading the stress path lies on the "Coulomb strength envelope" it may readily be shown (3) that the residual pore pressure is given by

$$\Delta u_r = \left\{\frac{K_c + 2}{3} - \frac{K_c - 1}{M}\right\} \sigma'_{3c} \quad \text{in which} \quad M = 6\,\sin\Phi'/(3 - \sin\Phi')$$

For the given material at low mean effective stresses $M \simeq 2.1$, so that $\Delta u_r/\sigma'_{3c}$ results in the values 1.0, 0.86 and 0.71 for the selected K_c-values, i.e. $K_c = 1.0, 2.0, 3.0$. The experimentally determined values of Δu_r were very close to these values indicating that during cyclic loading the failure envelope had been reached.

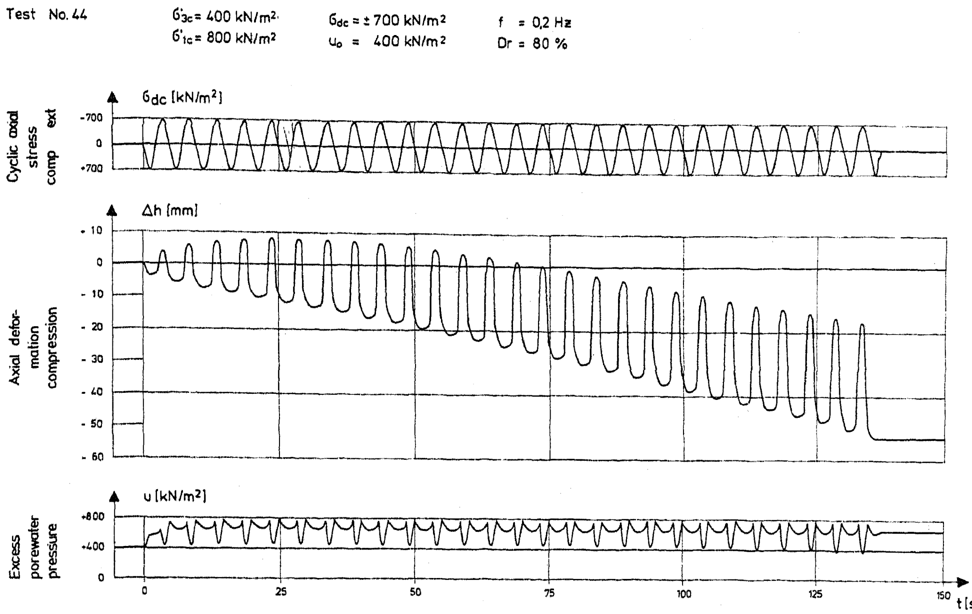

Fig. 3 Typical cyclic triaxial test data on shell material.

Positive pore pressures can be induced also in dense gravels, Figs. 4a) and b). However, there is no question of a flow of the material (i.e. liquefaction) occurring. In fact, if large compressive deviator stresses are applied subsequent to cyclic loading the material dilates and is able to sustain high shear stresses q according to the basic strength relationship $q = M \cdot p'$ until the critical state condition at large strains is reached. If one examines the stress path diagrams for typical reversing cyclic shear tests, i.e. q becomes negative (see Fig. 4a)), it is apparent that the stress path does not reach the theoretical failure envelope for negative q values, but seems to follow a failure envelope corresponding to a smaller M (or Φ') value. The sample then exhibits at failure both necking and bulging in the course of one load cycle. Necking is more pronounced the denser the sample. Once necking occurs it is meaningless to evaluate axial strains as they are concentrated in a very limited zone of the sample. This suggests that it might be preferable to test under conditions of non-reversing, and ideally with free ends to reduce to a minimum non-uniform strains due to the dead zones in contact with the rough platens. Some previous investigators have found that conditions of non-reversing deviator stress do not produce failure in the sense of $\Delta u = \sigma'_{3c}$. In the cyclic strength curves produced here, however, (see Fig. 5a)) all data points correspond to a maximum pore pressure defined such that the stress paths reach the failure envelope in the course of cyclic loading.

It may be noted in Fig. 5a) that for a particular value of K_c the strength curve for higher σ'_{3c} lies below the curve for lower σ'_{3c}. This is

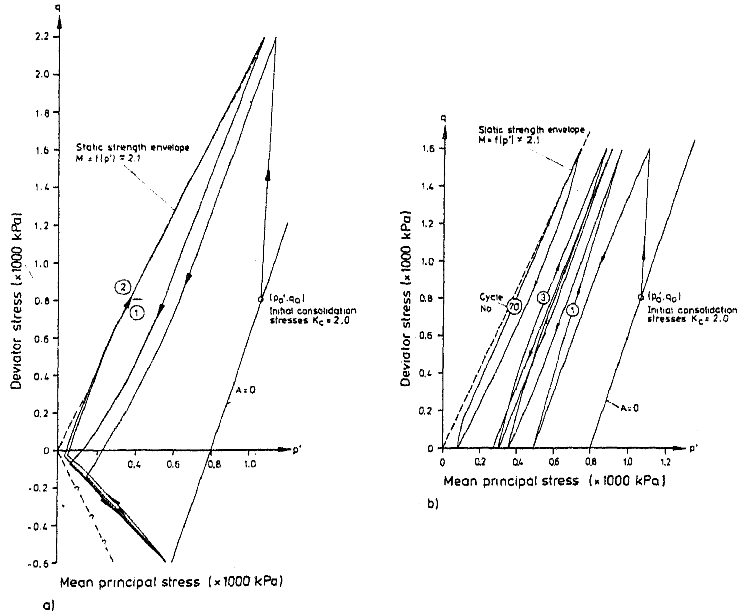

Fig. 4 Stress paths under undrained conditions in p' - q diagrams for tests with K_c = 2.0 and σ'_{3c} = 800 kPa.
a) reversed deviator stress
b) non-reversed deviator stress

reasonable because volume change characteristics (for drained tests) are influenced by σ'_{3c} and grain crushing is greater, the higher the value of σ'_{3c}. Fig. 5a) is the standard way of presenting results. An alternative way is shown in Fig. 5b), whereby the major principal stress σ'_{1c} is used to normalize the cyclic stress σ_{dc}. The tests reported here then show a relatively small scatter, indicating that for practical purposes and within a limited range of K_c values, a unique strength relation is given. There is experimental evidence to show that the void ratio under anisotropic consolidation conditions is primarily related to σ'_{1c} (4, 5). This tentative result would indicate that the existence of a static shear stress ($K_c > 1$) does not intrinsically increase the cyclic strength for a given void ratio. Consolidating triaxial samples anisotropically to get initial shear stresses simply has the effect of producing a denser sample. With regard to the a-seismic design of earth dams the behaviour of different soil elements must be compared on the basis of relative density, σ'_{1c} and as a secondary parameter σ'_{3c}. This would mean that zones with higher K_c-values, e.g. near the slopes of a dam, are not necessarily safer. For two elements in the dam at different levels subjected to the same seismic stress ratio (τ_{cyc}/σ'_{1c}), the one with the higher value of σ'_{3c} will build up pore pressures more quickly.

Curves of the type presented in Fig. 5 can also be produced for cyclic or accumulated strains.

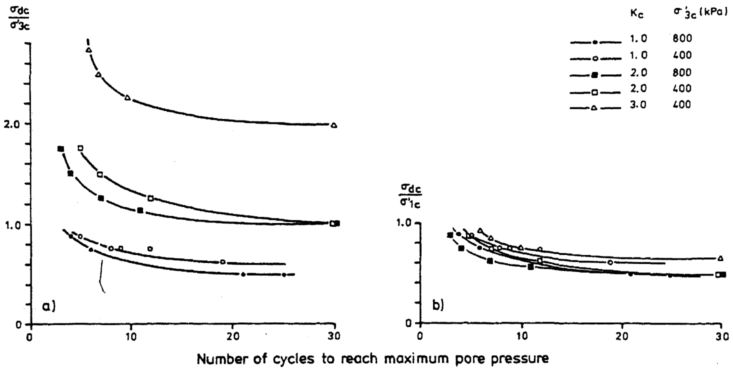

Fig. 5 Normalized cyclic stresses as a function of the number of cycles to reach maximum pore pressure.
a) stress normalization with σ'_{3c}
b) stress normalization with σ'_{1c}

STIFFNESS AND DAMPING CHARACTERISTICS

The stiffness and damping properties of soils are strongly dependent upon the shear strain, and this fact needs to be accounted for in dynamic analysis. The non-linear stress-strain behaviour is known to be of plastic hysteretic nature, but for convenience equivalent viscous damping and secant moduli are evaluated for use in standard dynamic finite element codes.

The most common test equipment used to obtain the relevant cyclic deformation properties is the triaxial apparatus. Using this device the shear modulus G and shear strain γ are usually found in terms of the Young's modulus E and axial strain ε assuming a value for Poisson's ratio ν, i.e.

$$G = E/2(1+\nu), \qquad \gamma = \varepsilon\,(1+\nu)\,.$$

Some of the typical features of the test results and difficulties encountered are described by Pyke (6). With respect to gravels the published data seems to be limited to the work of Seed and Idriss (7). Gravels exhibit more or less the same behaviour as sand, except that the stiffness values are somewhat higher (by a factor of about 1.5 to 2.5). The initial tangent stiffness is dependent amongst other factors on the mean confining pressure. In the test series conducted here under isotropic confining stress conditions the empirical square root relation that applies to sands was found to hold good (Fig. 6). However, during the cyclic triaxial test the mean principal stress varies so that for negative deviator stresses the secant modulus is less. This influences the general shape of the hysteresis curve, which somewhat resembled a banana. Cycling the cell pressure to maintain a constant mean principal stress improved the form of the hysteresis loop, but one is still left with an asymmetric form. In this case the average of the secant moduli may be taken.

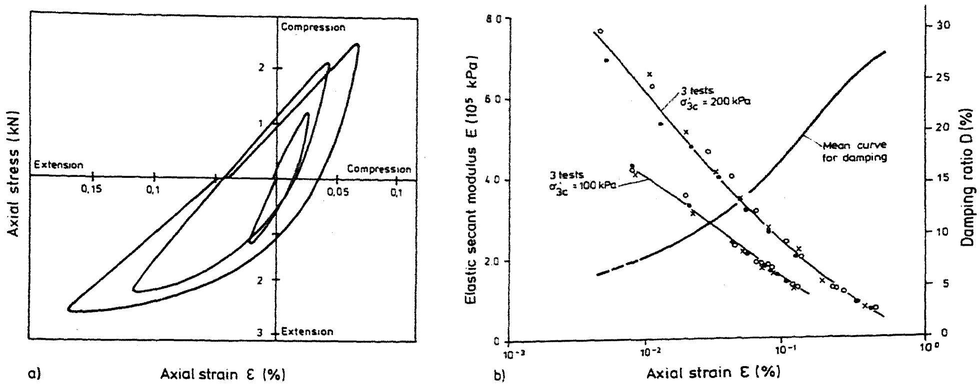

Fig. 6 Triaxial determined stiffness and damping characteristics of the alluvial gravel as a function of the axial strain.
a) typical hysteretic loops for a test with K_c = 1.0 and σ'_{3c} = 100 kPa
b) elastic secant modulus and damping ratio

A close examination of the loops indicates, however, that for large negative deviator stresses, even at low strains, either strain-softening or non-uniform strains are present.

CONCLUSIONS

1. The induced residual pore pressure is limited for anisotropically consolidated samples by the fact that the stress path is confined within the Coulomb failure envelope. Thus after removal of the cyclic deviator stress the induced pore pressure is always less than the static confining pressure.

2. If the cyclic stress is normalized with respect to the major principal stress the strength curves plot-for a practical range of in-situ confining pressures-within a fairly narrow band for different K_c-values. This provisional result simplifies evaluation procedures and allows the number of tests to be cut down considerably. This is an important point considering the laboriousness in preparing and handling large gravel samples.

3. The nature of strain development (both cyclic and accumulative) depends on the stress level and whether reversing or non-reversing deviator stresses are applied. The triaxial extension (reversing condition) of compact gravel samples generally leads to necking. This renders strain-evaluation difficult and can lead to conservative results.

4. The stiffness curves of the gravel material, i.e. secant elastic modulus as a function of axial deviator strain, are of a similar form to those for dense sand, but lie somewhat higher. They also exhibit a square root law. Damping values are approximately equal to those of sands.

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