STRESS AND DEFORMATIONS IN EARTH MASSES

Reporter
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SUMMARY Six papers are included in the session, these cover: temperature effects on the consolidation of clay soils, the stress dilatant behaviour of sand, the vibration of an earth dam, the lateral load behaviour of a model pile embedded in sand, the modelling of the behaviour of a jointed rock mass, and the tensile cracking of compacted clay. The contribution of these papers to the subject of stress and deformation in earth masses is reviewed. The report places emphasis on the real stress-strain behaviour of earth masses rather than the common linear elastic idealisation. In particular it is pointed out that the role of prefailure dilatancy plays in determining the stresses and deformations needs further research.

1 INTRODUCTION

The subject of this session lies right at the heart of the discipline of geomechanics which is concerned with the mechanical behaviour of soil and rock masses, and which in turn is based mainly on the subdisciplines of soil mechanics, rock mechanics and engineering geology. The prime purpose of geomechanics is to provide the geotechnical engineer with the insight required to make judgements about the stress changes and deformations that will occur in an earth mass when some engineering project, such as the digging of an excavation, the cutting of a slope, the placing of fill, or the driving of a pile is executed. All the investigation work, both field and laboratory, the modelling, both numerical and physical, reported in other sessions of the conference has the one objective of contributing some insight to this judgement process.

The interest in stress changes and deformations points to the quantitative aspects of the discipline. On the other hand since soils and rocks are naturally occurring materials, subject to all the vagaries of nature, there is much uncertainty about the detailed properties and behaviour of a given earth mass. This dissension between the desire for the quantitative approach, with its inevitable simplifications, and the need to recognise the complexity of natural, rather than manmade, materials is expressed in the following two quotations. The first from the writings of Lord Kelvin:

"I often say that when you can measure what you are speaking about, and express it in numbers, you know something about it; but when you cannot express it in numbers, your knowledge is of a meagre and unsatisfactory kind".

and the second from Aristotle:

"It is a mark of an educated man and a proof of his culture that in every subject he looks for only so much precision as its nature permits".

Geomechanics based solely on the quotation from Lord Kelvin would be subject to all, and more, of the false optimism of the nineteenth century view of physical science. Recalling the celebrated controversy in the second half of the nineteenth century between Kelvin and those proposing exceedingly long periods of time during which biological and geological evolution had occurred, leads to another important point. Based on a simple mathematical analysis of the time required for the earth to cool to its present state, Kelvin arrived at the apparently irrefutable conclusion that a few tens of millions of years at most were available. As we now know, the argument was settled in favour of the geologists and evolutionists by the discovery of radioactivity, which provided a mechanism for the spontaneous generation of heat within the earth. The lesson for geomechanics in this interesting episode from the history of science is very clear: the numerical approach, particularly by way of mathematical analysis, is of no use to anyone if some essential feature of the geomechanical process is omitted. Nevertheless Kelvin's words do serve to illustrate the very useful insight into physical processes that can be gained by the measurement and modelling techniques provided by the disciplines of soil and rock mechanics.

The quotation from Aristotle serves to temper any overly enthusiastic reliance on theoretical analysis to the detriment of a thorough appreciation of the depositional details and structural features of the soil or rock mass. It thus emphasises the way in which engineering geology complements the understanding provided by soil mechanics and rock mechanics. (It is a little unfortunate that although the name geomechanics covers well the synthesis of soil mechanics and rock mechanics, it does not convey the vital contribution of engineering geology.)

The title of the session avoids the arbitrary division between soil mechanics and rock mechanics embodied in the existence of separate international societies. The materials considered in geomechanics form a continuous spectrum from hard, unweathered, and sparsely jointed rock at one end, to soft, normally consolidated deposits at the other. Since there is no clear dividing line somewhere near the middle of this spectrum, and as many of the features of the mechanical behaviour of soils and rocks are similar, it is not unreasonable to consider the materials under the one subject heading: earth masses.

The range of topics for the session is broad.
There are six papers included, these cover: temperature effects on the consolidation of clay soils, the stress dilatant behaviour of sands, the vibrational response of earth dams, the lateral load behaviour of a model pile embedded in sand, the modelling of the behaviour of a joined rock mass, and the tensile cracking of compacted clay.

The conference organising committee commissioned the reporters to cover in general the respective field of their sessions while incorporating the contribution of the conference papers. It seems appropriate to state the particular point of view of the reporter for this session. Stress and deformation in earth masses suggests that the real stress-strain behaviour of earth materials must be considered if any useful insight is to be obtained. There are two features that must be considered: the non-linear and irreversible stress-strain behaviour, and the phenomenon of dilatancy.

2 STRESS-STRAIN BEHAVIOUR OF EARTH MATERIALS

The characteristic phenomenon which differentiates geomechanics from other branches of solid mechanics is the phenomenon of dilatancy. This is defined as a volume change which occurs in earth materials when there is a change in shear stress. If saturated material is sheared under the constraint of constant volume, dilatant behaviour is manifested in the generation of excess pore water pressures. It is this of considerable relevance to consider the effects of dilatancy on the stress and strain distribution in earth masses.

Insight into nonlinear stress-strain behaviour has been derived mostly from laboratory testing on small specimens. For reasonable changes in stress it is well known that the stress-strain curve is not linear, that deformations are not recoverable, and that stiffness depends on effective stress. Furthermore during cyclic loading energy is dissipated during each cycle, but the mechanism is hysteretic, rather than the usual viscous idealisation, because the apparent damping is found to be independent of frequency. There is only one situation where soil stress-strain behaviour approximates to elastic behaviour and that is for very small strain amplitude (strains less than 10^-5) cyclic loading, Richart (1975).

The above features are well established for soils and established to a lesser extent for jointed rock masses, and even for the so-called intact, but in fact microfractured, rock, Walsh and Brace (1973).

Much traditional stress and deformation analysis in geomechanics simply ignores the above features of soil and rock behaviour and assumes the material is linearly elastic. For other than small changes of stress this idealisation is a gross approximation. However, the attraction of linear elasticity is the availability of complete solutions to a number of boundary value problems. These solutions account not only for the stress-strain behaviour but also for equilibrium and kinematic conditions. Such solutions, which include realistic stress-strain behaviour, are generally not available for the boundary value problems of interest in geomechanics.

The advent of finite element analysis has meant that a number of interesting and useful solutions can be obtained with realistic stress-strain modelling. One interesting conclusion that has been reached from such studies relates to the distribution of vertical stress. When a uniform pressure or displacement loading is applied to a finite element representation of a half space, and a non-linear incremental analysis performed, the resulting vertical stress distribution is found to be very close to that given by the elastic solution. This result has been found by a number of separate investigators and some typical results from Graham (1982) are given in Fig. 1. On the other hand the horizontal stresses are somewhat larger than in the elastic case.

This result is the basis of the numerical approach to the stress path method given by Burland (1971), and developed further by Hira (1980). Since non-linear incremental finite element analysis is hardly a design tool, further developments of such stress path methods would seem to be a worthwhile way of incorporating modern constitutive models into design methods for estimating soil displacements.

3 SESSION PAPERS

3.1 Consolidation around a heat source - Booker and Sawgod

The authors have produced a very elegant analysis of the generation and dissipation of excess pore water pressure around a heat source buried deep in a saturated soil deposit. It is nothing short of a splendid achievement to have obtained an exact solution about a spherical heat source, and an

Fig. 1: Vertical stress distribution in a non-linear medium beneath a rigid plane strain footing. (a) beneath the centre of the footing, (b) beneath the edge of the footing, (c) a distance 2B/3 beyond the edge of the footing (B footing width). (From Graham 1982)

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approximate solution about a cylindrical source, which takes account of: elastic deformations of the soil skeleton due to changes in effective stress and temperature, flow of pore water and flow of heat from the source.

The mathematical formulation of the problem arises from a prior qualitative understanding of the physical situation and the need for more detailed results. The essential step is to idealise the problem sufficiently so that a set of equations capable of solution is obtained.

This paper provides a particularly good example of the improvement in insight gained by moving from a qualitative to a quantitative analysis. From the qualitative point of view there was reason to believe that fracturing of the soil around the heat source might be possible. The result of the analysis shows that this will not occur. There has been a deepening of understanding that Kelvin had in mind when he emphasized the need for appreciation of phenomena in numerical terms.

The real problem has to be extensively idealised before the mathematical analysis can proceed; on balance the return for the idealisation seems more than worthwhile. The deformation of the soil skeleton is limited to elastic behaviour, but this will not be a serious limitation if the soil is overconsolidated as the pore pressures dissipate. Another possibility for extension of the analysis is to consider whether some of the soil properties, particularly the coefficient of consolidation, might be temperature dependent. Such an extension would add greatly to the complexity of the analysis and might well mean that a numerical solution is needed. If this was attempted the analytical solution presented in the paper would then provide a valuable benchmark solution to which the numerical result should converge as the temperature sensitivity of the various properties is reduced to zero.

3.2 Dilatancy of Sand under a General Stress System - Mesary

This paper considers further the stress dilatancy relation proposed by Rowe (1962). In its standard form this applies to dense sand in the conventional triaxial apparatus. It relates the principal effective stress ratio to the ratio of the volumetric to axial strain rates. It thus gives part of the stress-strain relationship for sand, but it is incomplete in that it relates to drained behaviour only. Also the relationship requires correction at small strains; it applies better to the middle and failure strain ranges. Mesary generalises the relationship to general stress systems. From a series of tests on two different sands, he found that the principal effective stress ratio \( \sigma'_1/\sigma'_3 \) was a linear function of the dilatancy rate up to a maximum value. The maximum dilatancy rate was found to be independent of the intermediate principal stress, and a relation between the maximum effective stress ratio, maximum dilatancy rate and relative porosity was proposed. The maximum dilatancy rate generally occurred at the peak stress of the sand. It is of interest to note that the shear strength behaviour of a rough rock joint face surface exhibits a similar phenomenon. The peak shear resistance is associated with the peak 'dilation' rate (the peak value of the rate of normal displacement), Barton (1976). This illustrates the comment made in the introduction that there is a continuity between the behaviour of rock masses and soil masses.

There is a pressing need in geomechanics to explore the consequences of dilatancy beyond the constitutive modelling phase. The onset of dilatancy generally occurs well before the peak strength of a material is mobilised and the question thus arises as to how this affects the behaviour of an earth mass prior to failure. As dilatancy has been recognised since the early days of soil mechanics, it is surprising that this aspect of the phenomenon has received so little attention. Borrowing again from the rock mechanics literature, a very clear statement of the effect of prefailure dilatancy was given by Goodman (1974). His demonstration of the effect of a constraint preventing the dilatant deformation during the shearing of a rough joint surface is shown in Fig. 4. The geophysical community has realised the significance of prefailure dilatancy as a premonitory phenomenon for earthquake prediction, Nur (1974). Apart from the paper by Bishop and Bjerrum (1960), considered in the next paragraph, discussions in the soil mechanics literature of the significance of prefailure dilatancy are rare, particularly the effect of dilatancy on the solutions to boundary value problems of interest. One exception is the observation of

![Fig. 2: Effect of test mode on shear deformation curves for dilatant joints. A: shear at constant normal stress, B: shear with no normal displacement (From Goodman 1974)](image)

![Fig. 3: Changes in pore pressure and safety factor during and after excavation of a cut in clay (From Bishop and Bjerrum, 1960.](image)

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Atkinson and Potts (1977) that the surface settlement profile above a tunnel driven in a dilatant medium is less noticeable than that in a non-dilatant medium. Another exception is the Cambridge group work on the passive pressure behind retaining walls in dense sand. Dilatancy was included in this work and the earlier results are summarised by Roscoe (1970). However the retaining wall response involves several phenomena and the effect of dilatancy is not easily isolated. A clearer presentation of the effect of dilatancy on the displacements of the sand behind a retaining wall rotated about its toe has been presented by Wroth (1975).

The clearest appreciation of prefailure dilatancy in the soil mechanics literature is given by Bishop and Bjerrum (1960). They illustrate the effect of various amounts of dilatancy, as expressed in the undrained pore pressure response through the pore pressure parameter A, on the time dependent pore pressure changes after undrained loading*. Their diagram is repeated in Fig. 3.

3.3 Response of a two dimensional wedge taking the effects of shear and bending moment into account - Nomaishi, Kurioiva, Matsunaka and Itahio.

The authors investigate the natural frequencies of a wedge shaped earth dam bounded by a rectangular canyon. The dam is inhomogeneous in that the shear modulus increases with depth. In addition to the well known shear beam approach the authors included bending effects in the analysis.

The authors conclude that for a wide dam bending in the vertical plane has a noticeable effect on the first and second natural node frequencies when the side slopes are steeper than about 20°. However the effect does not become really marked until the side slopes are steeper than 45°, so the shear beam theory is still acceptable for assessing the modal frequencies of most dams. As expected the side walls of a narrow canyon have a stiffening effect on the natural frequencies. Once the width of the dam is greater than five times the height the effect of the side walls on the natural frequencies disappears. But even for a dam with width twice the height the effect of the canyon walls is surprisingly small.

A recent comparison between two and three dimensional earthquake response analysis of earth dams relates to the subject of this paper. Mejia and Seed (1983) computed both the two and three dimensional earthquake response of a finite element model having the soil properties, dam profile, and canyon geometry of the Oroville Dam. This has a ratio of crest length to maximum height of 7.0. They also analysed a dam with the same maximum section profile and properties, but with a ratio of crest length to maximum height of 2.0. The earthquake record used was for an earthquake of magnitude 5.7 that had occurred near the dam. They reached similar conclusions to Nomachi et al regarding the effect of crest length on the natural period of the dam. For L/H less than 2.0 the presence of the canyon walls has a substantial effect on the natural frequency. The two dimensional analysis of the maximum section of the Oroville dam (L/H = 7) gave shear stresses within 20% of those from the three dimensional analysis. On the other hand there were considerable differences between the two sets of shear stresses when L/H = 2.

The results in the paper apply to undamped elastic behaviour. Thus they are relevant only to very small levels of excitation. A particularly interesting series of tests and analyses of the response of a full scale earth dam has been reported by Abdel-Ghaifer and Scott (1979 a and b, 1981 a and b). In these tests the response of the Santa Felicia earth dam in California was compared to forced vibration, ambient vibration and to two earthquakes (magnitude 6.3 and 4.7). The recorded response of the dam was interpreted to give apparent shear moduli and damping for the dam material. The results are reproduced in Fig. 4 of this report. Abdel-Ghaifer and Scott emphasise that the behaviour of the dam is too complex to explain all the changes in response with increasing excitation as strain dependent soil behaviour, nevertheless the results plotted in Fig. 4 are sufficiently reminiscent of

* The demonstration of the effect of dilatancy on pore pressure response was just one facet of the Bishop and Bjerrum paper. It is notable for the very lucid statement of the concepts of the short term and the long term (or undrained and drained, or the $\Phi = 0$ and effective stress) methods of stability analysis. In recent years a spate of elementary and middle level soil mechanics texts has appeared. The majority of these texts present a deficient exposition of soil stress-strain-strength phenomena. The authors seem to be unaware of the Bishop and Bjerrum paper or the concepts of short term and long term stability analysis.

![Fig. 4: Estimated shear moduli and damping factors and the corresponding dynamic shear strains. (Evaluated from the hysteretic response to two earthquakes as well as from full scale dynamic tests). (From Abdul-Ghaifer and Scott 1981)](image-url)
the familiar variation of apparent shear modulus and equivalent viscous damping ratio with strain amplitude, that strain dependent behaviour must be an important factor. These results are a rare demonstration of the non-linear hysteretic behaviour of soil at full scale.

The natural frequencies discussed by Nomachi et al correspond to the low strain amplitude forced vibration and ambient vibration test results shown in Fig. 4 of this report.

3.4 Experimentally Determined Distribution of Stress around a Horizontally Loaded Model Pile in Dense Sand - Williams and Parry

The lateral load behaviour of a pile is one of the basic problems of soil-structure interaction. Although much attention has been devoted to the problem there still seems to be much scope for further work.

At the experimental level there are three possibilities: full scale testing, model testing in a centrifuge and model testing at 1g. The first of these is very expensive and the opportunity occurs only rarely. Also lateral loads sufficiently large to get well into the non-linear range, particularly for dynamic tests, are difficult to mobilise. Dynamic pile tests reported by Scott, Tsai, Steussy, Ting (1982) and Jennings, Edmonds and Theen (1986) indicate that the comparison between centrifuge modelling and full scale lateral load tests of a pile in sand is encouraging. However a centrifuge is a major experimental facility and requires considerable resources for its operation. It is of interest of note that the authors of the present paper have chosen to model at 1g, despite the fact that the department in which they were working has, at the time of writing, one of the largest and most sophisticated geotechnical centrifuges in the world.

The chief objection to scale model testing in the laboratory is that the stress levels in the sand are too low. At the bottom of the model pile the vertical effective stress in the sand is only about 10 kPa. It is known that at very low normal stresses the strength behaviour of dense sand is not as simple as the usual constant friction angle zero cohesion assumption, Seed and Lee (1967). In fact the failure envelope is curved in such the same way as is the failure envelope for a closely jointed rock mass. The normal stresses generated against the model pile during the lateral loading, shown in Fig. 4 of the paper, are very high in relation to the local vertical stresses in the sand. Pendall (1979) reports similar large lateral pressures generated by model piles in a bed of dense sand. In both cases the lateral pressures are much greater than the likely passive resistance available. Despite these difficulties the results presented by Williams and Parry show that there is still interesting insight to be obtained from model tests at 1g.

The very large normal stresses at the pile tip, at the completion of the driving, are also worthy of comment. The jacking of the model pile displaces the sand and also causes shearing in the sand surrounding the tip. By analogy with Fig. 2 of this report, the differential tendency of the dense sand can be suppressed if there is a large increase in the confining pressure as observed by the authors.

The expression for the soil modulus given in equation 2 is of a form that requires the coefficient to have units which depend on the power to which the depth is raised. The expression Nomachi et al avoids this by using the ratio (z/h) to a power, in this way the coefficient has the same units regardless of the value of n. Another difference between these two papers is the range of values for n. Nomachi et al used the range 0 ≤ n ≤ 1, whereas Williams and Parry are specifying a value of 3 in their equation 2. The suggestion that the modulus increases as the depth cubed gives a very rapid build up in stiffness with depth. Perhaps this is another indication of anomalous behaviour that arises from tests at small scale and on soil contained in a tank. From the readings of normal pressure at each of the load cells and the displacements obtained with the dial gauges it should be possible to plot a pressure-displacement curve for the sand at each load cell. It would be surprising if these curves were linear, so the question arises as to the displacement range equation 2 is associated with.

3.5 Deformational mechanisms of a rock mass - Chappell

The problem presented to the geotechnical engineer by a jointed rock mass is very challenging. There are frequent discussions in the soil mechanics literature about sample disturbance, the need for in situ testing, and artefacts introduced because of features of the testing apparatus and experimental technique. However at least it is possible to recover a sample and test specimens of reasonable size. With a jointed rock mass there is no such possibility, and even in situ testing may not solve the problem because of the scale of the jointing. Assuming, as is generally the case, that the joint properties control the rock mass behaviour it is necessary to measure the properties of a volume of the rock mass sufficiently large to represent the joint system. For all but exceedingly closely jointed systems this is simply not possible. One approach is to map the joint system, assign strength and stiffness properties to the individual joints, and then to calculate the response of the rock mass. Once again this is a pragmatic approach after the event and not the quotation of the quotation from Kelvin, but it is very much under the shadow of the quotation from Aristotle. For one thing the mapping of joint systems is an uncertain business as much has to be inferred from limited data. Secondly the assignment of joint properties is very difficult. Apart from the difficulty of measurement of joint stiffness and shear strength there is the question of scale effects. Pratt et al (1975) have shown that measured joint shear strength decreases as the area sheared increases. Finally the problem of calculating the behaviour of the rock mass given the geometry of the joint system and the joint properties is not a simple matter. Many workers have used finite element analysis with special elements representing joints. An alternative procedure developed by Cundall (1977) shows considerable promise.

Another approach, which is suggested by Chappell in the paper, is to attempt to model the rock mass with an equivalent anisotropic continuum so that, for prefailure behaviour anyway, the joint system is replaced. This approach is more attractive than the method outlined above when the joints are closely spaced. In many practical situations approaches to formulating the method, Gerrard (1982), Pande and Gerrard (1983), Goodman (1980), Amadei and Goodman (1981). The approach suggested
Yet another example of the significance of prefaillure joint dilatancy, or in this case, more correctly, interface dilatancy, is given by Rowe and Pells (1980). They described a finite element analysis of the interaction between the surrounding rock mass and a pile socket with a rough, and hence dilatant, interface between the pile and the rock. It was found that the angle of dilatancy of the interface had a profound effect on the skin friction available. In their calculations, Rowe and Pells used modest values for the angle of dilatancy of the interface. Further work on dilatant interface behaviour for pile sockets is presented by Chiu and Dight (1983).

3.4 Cracking Behaviour of an Earth Dam — Moore and Hop.

Cracking in the clay cores of earth dams has been of concern for some time. Abutment irregularities provide a clear focus for such cracking. The paper reports a model study of the behaviour of clay compacted across a short abutment irregularity. The paper adds to research which has been underway at the University of Melbourne for a number of years.

The most interesting finding of the paper is that the vertical stress required to cause cracking is smallest when the compaction water content is close to optimum. Presumably as the compaction water content decreases below optimum the available tensile strength of the soil increases. As the compaction water content increases above optimum, the soil becomes increasingly able to deform plastically around the abutment irregularity. Descriptions of the crack pattern observed would be of interest.

The paper also shows, once again, how difficult many phenomena of interest to the geotechnical engineer are to quantify with a "simple" test. Bending tests on compacted beams seem, at first sight, to be a convenient means of assessing the susceptibility of a soil to cracking. However the tests reported in the paper show that this is not the case for cracking over abutment irregularities.

4 CONCLUSIONS

A major conclusion of this report, which reflects the point of view of the reporter, is that prefaillure dilatancy is a phenomenon that will have a significant effect on stress and deformation in earth masses.

The title of the occasion could easily have accommodated papers covering the results of field measurements of stress and deformations. Although these are much more expensive than laboratory or analytical work, and require great persistence to execute, they are a most important means by which our understanding of the behaviour of earth masses will be advanced. Such studies are also most effective in fostering an awareness of the need in geomechanics to use "... only so much precision as its nature permits".

5 REFERENCES


