Earth Structures, Dams, Soil Improvements and Geofabrics – General Report

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1. INTRODUCTION

A total of twelve papers were submitted for this session, and the brief from the Organising Committee was to "summarise the submissions and present a review paper".

Granted the diversity of the papers presented, and the wide range of topics covered within the title of this session, it was clearly impracticable to present a review paper. I have opted, therefore, to summarise briefly the content of the papers and to comment where this appeared warranted. In several papers, additional information would have made the papers more useful to readers, and I have encouraged the authors to present this in the second volume of the conference proceedings.

In some cases, there are issues which warrant questioning or critical comment, and others where I feel that readers would do well to read alternative or additional information.

If there is a common theme to the review comments, it is that authors fail to include sufficient detailed information in the papers, to allow readers to properly relate the experiences described in the papers to their own projects. This is not always due to the lack of space required by the six page limit for the conference.

2. EMBANKMENTS AND EXCAVATIONS IN SOFT CLAY AND SILT

PALMER, S.J. — Seaview Marina, Geotechnical Design of Breakwaters.

This paper is an interesting case history which describes the investigation and design of a rubble mound breakwater which is up to 8.5m high. The breakwater is founded on 14m of "very soft to soft normally consolidated silt with some clay". The paper describes the soil properties, results of stability analysis, and pore pressures and settlements measured after construction.

There are some unusual features of the soils at the site:

- the effective friction angle of the silt is $\phi'=37°$ in triaxial compression. This is explained by the author as being due to the presence of 20% sand in the sample tested
- the undrained shear strengths determined by vane shear are very high. The lower bound envelope of values is equivalent to $S_u/\sigma'_{vo} = 1.1$.
- there is an artesian aquifer at the base of the silt, with a pressure 20kPa higher than mean sea level.

The monitoring after construction shows settlements in line with predictions, being about 0.7m one year after construction. Pore pressures monitored at mid depth of the silt layer show no dissipation over that period.

COMMENTS:

(a) The presence of 20% sand is unlikely to be the only reason for the high effective friction angle. By analogy with plots of residual strengths versus clay fraction percentage produced by Mesri and Cepeda-Diaz (1986), it would require around 40% to 50% sand before it would dominate properties. One would suspect that the silt must be very angular to give $\phi'=37°$.

(b) The undrained shear strengths obtained by the vane shear appear to be too high for a normally consolidated soil:
- Jamiolkowski et al (1983) give $S_u/\sigma'_{vo} = 0.23 \pm 0.04$.
- using the relationship between S_u and ϕ' in Wroth and Houlsby (1985) $S_u/\sigma'_{vo} = 0.43$, and using Davis in Thorne (1984), $S_u/\sigma'_{vo} = 0.23$ to 0.36 depending on Skempton's A_f.

There are several explanations:
- the soil is overconsolidated. This seems unlikely given the recent accumulation of silt at the site
- the vane is not testing in an undrained manner, ie. the soils are relatively high permeability
- the silt is cemented
- shells are affecting the vane testing.

The author's comment on this would be useful.

(c) The lack of dissipation of pore pressure shown in Figure 5 of the paper is inconsistent with the conclusion that "The accurate prediction of the rate of consolidation settlement (and hence pore pressure dissipation) has provided confidence as to the stability of the embankment".

The author is not alone in having contradictory data on pore pressure dissipation and settlement in silty soils, eg. McDonald (1988) describes similar phenomena.

It raises the question as to whether the strength gain required to raise the embankment has occurred. It would also seem possible that either the artesian pressures are having an influence on the pore pressures, and/or that the settlement is due to secondary consolidation. It would be useful to know what the pore pressures were prior to construction (including the effect of the artesian aquifer). The fluctuations in monitored pore pressure is possibly due to response to total stress loading changes under tidal water level changes. Such effects were noted in the paper by Thorne in this conference, and the general reporter knows of other instances where this has occurred.

It would also be useful to know what the imposed embankment load was for Figure 5, so that the pore pressure response due to the embankment could be assessed.

(d) The vane shear profile used for design is stepped, with relatively high values used at 3m. There does not appear to have been any reduction factor applied as recommended by Bjerrum (1973). Both would seem to be unconservative assumptions.

Overall this is an interesting case history, and it is hoped that some discussion will arise from these comments.

THORNE, C.P. — Trial Loading of Failed Section of River Bank, Fishermans Island.

This paper gives an unfortunately too brief description of a most interesting trial loading of a failed section of river bank. Details have been omitted which would have made the paper more valuable. Even so, the paper does provide some interesting data:

- The response of piezometers to tidal change. The piezometers showed no time lag, and responses varying from 0.7 to 0.3 of the total stress change due to the tidal fluctuation in water level
- pore pressure responses on loading which could be related to changes in principal stresses calculated using finite element analysis, yielding Skempton's pore pressure parameters within the range B=1, A=0.3 and B=1, A=1.0 (in some cases A > 1.0 was required)
- details of how allowance was made to correct for the relatively short length of the trial fill section, allowing adoption of higher strength values for design than those calculated by backanalysing of the trial fill.

COMMENTS:

(a) The range of Skempton's A values calculated, including values up to 1, is described as "representing an extreme response". Given that, these appear to have occurred in the piezometers near the fill/foundation interface and, hence, in material possibly disturbed by sliding, and near normally consolidation conditions, the high values would seem reasonable and consistent with those given by Skempton (1954) for normally consolidated soft clays. The lower values are consistent with slightly overconsolidated behaviour.

(b) The case history is an excellent example of where spending the necessary money on detailed geotechnical investigation and testing, and trial loading has resulted in considerable savings in remedial works. It would have been a brave engineer to ignore the low strengths obtained in residual shear testing without the proof of the trial loading.

(c) As indicated above, the paper would have been more valuable if more details were provided. Possibly the following information could be provided in the second volume of the conference proceedings:
  - the circumstances and backanalysis of the original failure, and details of the remedial works
  - comparison of the Skempton A values obtained in the paper, with those in the triaxial tests
  - discussion of the triaxial extension test(s), and if there was any reduction in undrained shear strength in extension, as compared to triaxial compression tests, as would be expected based on information published by Jamiolkowski et al (1985) and others
  - the time between the failure and the trial
  - a larger version of Figure 2, and Figure 3, with piezometer numbers.

BRABHAHARAN, P. — Ground Improvement to Reduce Geotechnical Risk: Waingate Final Water Treatment Plant.

This paper gives a general description of the use of stone columns to reduce settlements, and to increase bearing capacity under retaining walls and buildings founded on 5m to 12m of soft to firm colluvial clayey silt. The columns were constructed by compacting granular fill through 910mm dia steel casing, withdrawing the casing progressively. The columns were successful in reducing settlements.

The water treatment plant site was identified as being subject to previous overall instability, and subhorizontal drains were installed. Water was observed to flow from most of the drains, with significant flows from some.

COMMENTS:

The paper suffers from lack of detail, particularly in respect to:

- the location and depth of the subhorizontal drains
- piezometric response to the subhorizontal drains (did they reduce the pore pressures, and by how much?)
- effective stress properties and the analysis of overall stability
- the magnitude of settlements including "creep effects" which are likely to occur with the stone columns.

3. DESIGN OF EMBANKMENTS FOR SEISMIC LOADING

ELIAS, D.C., NOVELLO, E.A. and GLENISTER, D. — Design Procedure for the Seismic Analysis of Earth Structures.

This paper presents a simplified design procedure for seismic analysis of earth structures such as dams, tailings dams and embankments. It requires as input the probability, magnitude and maximum ground acceleration of the design earthquake; either in-situ SPT or CPT data, or laboratory cyclic triaxial or simple shear tests, from which liquefaction potential can be assessed; and the geometry of the embankment.

COMMENTS:

(a) While the method is "simplified" compared to full scale dynamic finite element analyses, it encourages the use of cyclic triaxial tests for the design of embankments. In most cases in Australia, this would be an unnecessarily complicated and costly approach for design of embankments, dams and tailings dams constructed of compacted earth and rockfill.

ICOLD (1986), who tend to take a fairly conservative but realistic approach, quote Seed (1979):

> "virtually any well-built dam on a firm foundation can withstand moderate earthquake shaking, say with peak accelerations of about 0.2g, with no detrimental effects"

and

> "since there is ample field evidence that well-built dams can withstand moderate shaking with peak accelerations up to at least 0.2g, with no harmful effects, we should not waste our time and money analysing this type of problem — rather we should concentrate our efforts on those dams likely to present problems either because of strong shaking involving accelerations well in excess of 0.2g or because they incorporate large bodies of cohesionless materials (usually sands) which, if saturated, may lose most of their strength during earthquake shaking and thereby lead to undesirable movements".

(b) Tailings dams constructed of sand or silty sand, using the upstream or centreline methods, need to be considered more carefully. In this case, the author's approach may be justified but readers should also refer to the other "simplified" methods suggested by Lo and Klohn (1990), McLeod, Chambers and Davies (1991), Lo, Klohn and Finn (1991). For more important structures, a method such as that outlined by Finn et al (1990) might be employed.

(c) In many cases, it will be the potential for liquefaction of saturated granular soils in the dam or embankment foundation which will be critical. An initial assessment of this can be done using the method described by Seed and de Alba (1986), with allowance for static driving shear stresses and in-situ effective stresses greater than 100kPa described by Seed and Harder (1990). The general reporter's experience is that this will rule out most sites in Australia as having a problem (but not all).

(d) The general reporter does not agree with the recommendation in section 2 of the paper that "we recommend that an annual probability of exceedance of 1:500 is used for structures with short operating or saturated lives, such as tailings dams, and an annual probability of exceedance of 1:1000 for structures with long design lives such as railway embankments and water supply dams". The design earthquake should be assessed on the basis of the hazard rating of the structures, ie. the consequences of failure of the dam (in ANCOLD terms). ANCOLD (1983). and ICOLD (1986) give guidelines.

4. SEEPAGE CONTROL FROM STORAGE DAMS

O'FLAHERTY, P.J., BARKLEY, D. and TRUSCOTT, E.G. — Seepage and Salinity Control for the Wurdee Boluc Reservoir Enlargement Project.

This paper describes the investigation and design of seepage control measures to allow control of salinity surrounding the Wurdee Boluc Reservoir, when its full supply level is raised. The reservoir is underlain by tertiary clays, silts, silty sand and sand, and tertiary basalt along one boundary of what was a natural lake. Details are provided of seepage analyses, a trial drainage trench and the design of the drainage trench and relief wells, which are needed to control piezometric pressures in saline aquifers. The paper describes how a staged "observational" approach has been adopted to reduce the costs.

COMMENTS:

(a) The permeability of the "organic clay", "surface clay", and "cover clay" are shown as in the range approx 10^{-8} m/sec to 10^{-9} m/sec. The permeability of the basalt is shown as 10^{-7} m/sec to 10^{-8} m/sec approx. These values appear very low for naturally occurring clays, which usually have fissures, relic joints, root holes and/or other lamination and deposits which result in mass permeabilities orders of magnitude higher than those quoted. The value for basalt, in particular, seems low. The permeabilities are said to be based on laboratory and field permeability tests, but no details are given. If the values are based on "pump in" type tests in boreholes, the general reporter would doubt their correctness. Again, it would be the basalt clays where subvertical defects are likely to be present where the greatest error would occur. That this layer may be more permeable is indicated by the recording of saturation and waterlogging along the edge of the basalt flows. It would be useful if the authors commented on the methods used to assess the permeabilities.

(b) The authors adopt a permeability of 10^{-6} m/sec for a slurry trench cutoff wall which was assessed as an alternative. This figure is consistent with the data published by Powell and Morgenstern (1985), and the general reporter would agree it is a realistic figure.

(c) Two and three dimensional seepage analyses were carried out, which included modelling of pressure relief wells. It would be useful to know how the pressure relief wells (which are to be spaced 20m apart) were modelled in the two dimensional model. It would also be useful to know what percentage of seepage is intercepted by the wells.

(d) It is indicated that the seepage collector trenches were sited near the property boundary, rather than at the toe of the dam. While this might be desirable from the viewpoint of intercepting seepage passing outside the property, it is not generally the best location when considering control of seepage pore pressures, under and downstream of the dam. With the presence of confirmed aquifers , such as those at this site, one would normally place the wells at the toe of the dam, or even under a berm to ensure that pore pressures are controlled against the possibility of "blow out" failure.

It is noted that there are few piezometers downstream of the collector trenches, which means that monitoring of the effectiveness of the trenches and wells will apparently be limited.

5. EARTH AND ROCK REINFORCEMENT

HAUSMANN, M.R., SADLIER, M.A.. and BECKINDALE, C. — Geomembranes, Geotextiles and Slope Stability.

This is a useful paper which describes some of the practical problems which may be encountered in designing heap leach facilities for gold or other treatment, where a geomembrane is used as a liner to the base of the heap. It is shown that supporting the geomembrane on geotextile can result in instability, because the friction angle between the two may be low. The information presented would have use in other applications, such as design of landfills.

HOWARTH, D.F. and RENWICK, M.T. — The Development, Testing and Application of a Non-Steel Tendon for Artificial Ground Support.

This paper describes the properties and some application of a highly oriented polyethylene "bar" or "rock bolt". It is claimed to have several advantages over steel and other non-steel bars in some applications, although creep and low melting point and poor bonding resistance between the bolt and grout would seem to be restricting factors in some applications.

BOYD, M.S. — The Application of Reinforcement Materials for Earth Structures — Risk and Acceptability.

This is a useful paper describing the concepts which may be applied to assist in deciding on which system of soil reinforcing might be applicable to a particular project, depending on the assessed risk. In this context, risk is defined in broad terms as a function of "hazard" (the consequences of failure of the structure), the "criticality of the reinforcement" to the structure in terms of stability and deformation, and "uncertainty", ie. the degree of confidence one has of long term behaviour in the particular project environment.

COMMENTS:

(a) The paper emphasises applications where vertical or near vertical faces are required for the structure, where the creep characteristics of polymer reinforcements is potentially more of a problem than it will be in other applications, where some minor long term deformation will not be so important.

(b) The difference between "durability experience", ie. the length of time a particular type of reinforcement has been used in practice, and durability, ie. the ability of the reinforcement to maintain its desired function in the environment of the particular project, is not made clear enough in Table 1. Strictly, it is the latter which is more important.

(c) When discussing the consequences of failure the mode of failure will be important, eg. failure of "inextensible" (the author's term) strips due to strip fracture will occur with little warning, but structures reinforced with the more "extensible" materials should exhibit large strains before collapse, allowing remedial action, and/or reduction of the hazard.

FULLER, S.R.. and WALLACE, K.B. — Strain Compatibility and Design Criteria for Reinforced Earth.

This is an interesting paper which describes the results of analysis of reinforced soil slopes using an analysis method which modifies the Bishop limit equilibrium method, to make some allowance for compatability of strain in the reinforcement and the soil. The results are compared to an elasto-plastic analysis using the finite difference program FLAC.

The "modified displacement method" (MDISMET) has been developed into a computer program by the first author, and can be used as a design aid.

COMMENTS:

(a) Unfortunately the paper does not describe how the displacements along the trial rigid body surfaces within the limit equilibrium analysis are calculated, so one cannot get any feel for the validity of the approach. The method apparently modifies the normal stresses estimated by Bishop analysis, but again details are not provided.

(b) The paper shows that the normal stresses calculated using Bishop, are higher than those using the Fellenius assumption when the angle of the base of the slice exceeds 40°, and uses the FLAC results to indicate the Bishop results are more reasonable. As a word of caution, my colleague Garry Mostyn has noted some inconsistencies with the Bishop method for steep slopes and failure surfaces, such as those being analysed by the authors. In particular, the factor of safety is calculated to increase, as ϕ' decreases. The reason for this is being explored.

(c) There seems to be an inconsistency between Figures 3 and 5, in that from Figure 3 one would expect the normal stresses calculated from the Fellenius method to be lower than those calculated using Bishop, not higher as shown in Figure 5. The authors might clarify this.

(d) Given that FLAC allows the estimation of stress strain behaviour of the soil and the reinforcement, rather than using some empirical approximation to estimate displacements, and the program's ready availability, there would seem some argument that it would be better to concentrate on using FLAC rather than MDISMET. It is not clear what benefits MDISMET would have over FLAC.

6. EXPANSIVE SOILS AND ROADS

HOLDEN, J C. — Reduction of Pavement Damage for Expansive Soils Using Moisture Barriers.

This paper presents the results of trials where vertical moisture barriers, consisting of polythene sheeting placed in trenches backfilled with clay, were used to control seasonal heave of the pavement. For the site of the trials, barriers up to 1.8m deep were not deep enough to prevent significant seasonal movements caused by roadside trees. A barrier 2.5m deep greatly reduced seasonal movements.

The information would be useful not only for road engineers, but would be relevant to building construction.

COMMENTS:

(a) The data would be more useful if shrink-swell test results, suction properties and water content profiles had been presented. Atterberg limits are presented, but they are not a good guide to shrink-swell behaviour

(b) It is indicated that "to substantially stop the lateral moisture migration in the subgrade, the moisture barrier must extend below the depth of any crack which would provide an easy moisture path through the relatively impermeable clay". This could be misunderstood. It is true that vertical cracks will allow easy ingress of rainwater, and hence facilitate swell; and that the depth of cracking is a guide to the depth of seasonal moisture variation (although moisture variation will often extend below the depth of cracking); but the crack will prevent moisture transfer horizontally. Apart from roots, the transfer of moisture horizontally from under the pavement is by soil suction, which will be broken by a crack.

LOOK-HONG, B., WIJEYAKULASURIYA, V.. and REEVES, I. — A Method of Risk Assessment for Roadway Embankments Utilizing Expansive Materials.

This paper sets out to evaluate the risk of distress of road pavements to shrink-swell effects in the subgrade, when roads are constructed on embankments. The paper presents a useful literature review on many of the issues which have to be considered, as well as some data from two field sites.

COMMENTS:

(a) A reader is likely to be left somewhat confused as to what is meant by "risk", because the terminology used in the paper is not consistent. At the beginning of the paper risk is defined as

$$[\text{Risk}] = [\text{Hazard}] \times [\text{Vulnerability}] \times [\text{Value}]$$

where	Hazard appears to be defined as the probability of occurrence
	Vulnerability as the degree of damage or loss of functions
and	Value as the value of the road and/or the value to users

but no clear definition is given.

In the next section "hazards" which may produce distress are detailed, including landslips, consolidation of compressible clays and volume changes in expansive clays.

This definition of "hazard" is maintained in section 3.1, but neither definition or assessment seems to be fed back into the risk equation.

Vulnerability is assessed in terms of the swell potential of a pavement under likely changes in soil moisture content, and this is related to acceptable differential movement criteria. However, this is done on acceptable limits (Table VIII) rather than in any way which could allow use of the risk equation. There is no further discussion of "value". Later in the paper (4.1) risk seems to be equated to maximum swell potential, and related to Table VIII. To further confuse the issue in 6.0, Figure 7 is described as showing the vulnerability of embankments to expansive behaviour, but is titled construction risks.

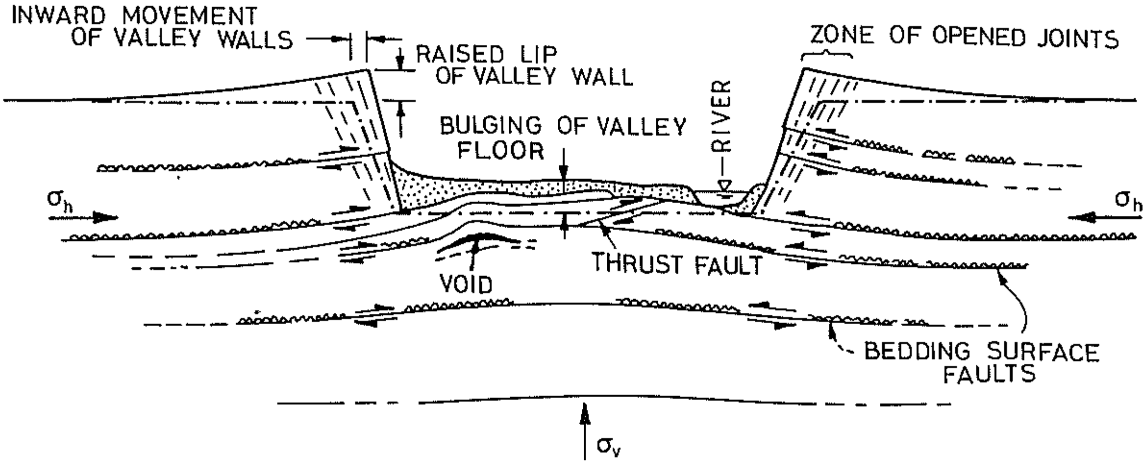

Figure 1 Complex valley structures related to stress release in weak, flat lying rocks (from Fell, MacGregor and Stapledon, 1992, based on Patton and Hendren, 1972).

(b) These problems are unfortunate, because there are some useful concepts in the paper. In reality, the paper seems to set out to develop a method for assessing the magnitude of potential swell, and comparing this to acceptable criteria, and doesn't assess "risk" as originally defined. Methods of construction which will reduce the potential swell are described, including control of construction water content to be near the in service equilibrium water content.

(c) The in service equilibrium water content was found to be near the plastic limit of the embankment soil, and for the soils at the test site, this was 1.3 times the optimum moisture content. Unfortunately, whether this is "standard" or modified optimum water content is not stated. The paper also uses the term relative dry density for what is more commonly called density ratio.

(d) Observed changes in water content at varying depths are shown in Figures 4 and 5, but it is not clear whether these are under the pavement or adjacent. The large changes recorded in Figure 5 would appear to indicate they were not beneath a pavement. It would be helpful if the authors would clarify these points of detail.

7. BEDDING SURFACE SHEARS

JAMES, P.M. — Flexual Slip, An Often Neglected Hazard.

This paper presents a case for engineers and geologists to be aware of the likelihood of the presence of flexual slip seams, particularly in interbedded argillaceous and stronger sedimentary rocks, and metamorphic rocks. Like the general reporter, the author's experience has been that these features, also known as bedding shears, bedding surface faults, or intraformational shears are often missed during site investigations, sometimes with severe consequences, as the effective shear strength on the planes is often near residual.

COMMENTS:

(a) The paper fails to refer to the paper by Hutchinson (1988), which discusses in more detail the formation of such features, and readers are encouraged to refer to that paper. The paper also fails to acknowledge the other potential mechanisms of formation. These include differential consolidation during the formation of the rock, differential slip along bedding and joints, faulting, and stress relief.

One of the most common causes of bedding surface shears is the differential movements which occur on stress relief as valleys are formed. Figure 1 shows examples of such features.

(b) The concentration of differential movement on the contact between "weak" and "strong" layers of rock, when in-situ ground stresses are relieved, has recently been observed in several excavations in the City of Sydney and also occurred in the 40m deep excavation for the Sydney-Newcastle Freeway near Wyong. Given this evidence, there would be a case for expecting the formation of such shear zones in deep excavations in interbedded sedimentary rock, eg. in coal mines, even if they are not present before excavation. This may be considered a form of progressive failure but the result is the same, ie. low strength bedding plane shears.

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