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Evaluation of Geotechnical Performance

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SUMMARY Since the 1920's, geotechnical engineering has developed from an art-form relying largely upon experience, to a recognised science utilizing sound theory and sophisticated analysis techniques.

A substantial part of this development has relied upon observation of the behaviour of geotechnical structures, and on objective evaluation of these observations. The paper discusses the requirements and benefits of such objective evaluation, summarizes observational methods and presents examples of how evaluation of past performance has contributed to the current state of knowledge.

1. INTRODUCTION

Geotechnical Engineering is predominantly concerned with the prediction of the engineering behaviour of soil and rock masses in response to applied loads. These loads may be imposed by man made structures (eg buildings), man induced interference with the original landscape (eg excavation) or by their own geological environment (eg landslides). A natural extension to this is the use of soil and rock as an engineering material to create engineering structures such as dams, embankments and pavements.

Intuitive geotechnical engineering has been practised for centuries, and a number of developments of this practice towards a science took place up to early this century (Skempton, 1979). However, it was not until the pioneering work of Terzaghi from about 1925 onwards that the science of soil mechanics and geotechnical engineering began to develop and be recognised as an engineering discipline. Since that time, our understanding of the fundamentals of geotechnical engineering has increased many fold, together with our analytical and numerical abilities through the everyday availability of computing facilities. In parallel with these developments have been improvements in exploration methods, insitu testing, sampling techniques and laboratory test procedures. However, in all but text book examples, when trying to understand the implications of all these developments we are faced with a real world where anisotropy and inhomogeneity are the rule and not the exception. Furthermore, we are usually restricted to investigating only a very small proportion of this real world when trying to evaluate or predict the engineering performance of the soil and rock contained within an area of interest. For example, the volume of soil represented by the boreholes typically drilled for a building site represents only about 0.002 per cent of that present.

Consequently, geotechnical engineers are forced to apply experience and judgement in their evaluation in an attempt to recognise and take account of any anomalies which may exist within the soil mass, but have not been observed during an investigation. Fortunately, whilst truly homogeneous isotropic conditions do not exist, general "idealization" of the subsurface conditions at most sites can be realistically carried out. However, our ability to carry out such "idealization", and for it to be acceptable, effective, and for any anomalies to be recognised before they become a problem, has required the assimilation and dissemination of observations and records from many sites and experiences throughout the world. It is this evaluation of geotechnical performance, and its impact on today's geotechnical engineering that will be discussed in this paper. However, the breadth of geotechnical engineering is so great that it is not practical to cover all aspects. Therefore, apart from some general comments only specific areas of personal interest will be discussed in any detail.

2. EVALUATION - PREDICTION OR POST MORTEM

Reference to a dictionary will suggest that evaluation may be either the finding of a numerical expression to describe an event; or an appraisal or assessment of an event. The first represents a design or prediction situation, whereas the latter indicates consideration of an existing performance. Whilst the differences may be subtle, in this paper it is intended to address the evaluation of existing or past performance, rather than future performance which may be more logically categorized as prediction, or design, within the theme of this conference.

It is recognised that through post mortem type evaluation, design methods are able to be refined. The subsequent use of these refined methods then amounts to prediction, with such predictions then being subject to further scrutiny after construction. Hence through this iterative evaluation process, the art of geotechnical engineering is advanced.

3. OBSERVATION OF GEOTECHNICAL PERFORMANCE

3.1 The Need for Observation

The evaluation of deliberate observation may range from recording of visual observations to close monitoring of a variety of sophisticated instrumentation. Whatever the level of observation it is essential it be performed by skilled personnel aware of the reasons for and the implications of the observation, so that appropriate data is obtained and any anomalies are checked and reported. As rather forcefully put by Penman and Kennard (1981), it is equally important that having decided to consciously observe certain aspects of a project, these observations should be properly evaluated in relation to the initial reason for their being made. Such evaluations should not be permitted to be put aside until some spare time is available.

The principle reasons why observations are made of "geotechnical performance" are considered to be:-

* Geotechnical engineering is a dynamic science requiring continual feed back to check the validity of, or basis for, assumptions made in design.

* Apart from potential variability within a site as compared to that indicated by investigation, laboratory testing procedures typically do not accurately reflect the field behaviour, and testing has to be carried out on samples which have suffered varying degrees of disturbance.

* Many design procedures are empirically based and if applying such procedures with a different set of conditions to those for which they typically apply some confirmation may be required.

* To enable design assumptions to be checked and monitored, eg construction control; and to permit modifications to be made if necessary. eg water testing of oil tanks.

* To permit the results of full scale observations to be utilized in design, or to establish design and/or performance criteria for subsequent construction.

* To monitor or confirm the performance of a structure or construction technique to check whether design requirements have been satisfied.

* To provide ongoing performance monitoring, eg dams, landslides, slope protection.

* To allow back analysis to permit review of design methods or to determine what went wrong and why.

* For research purposes.

* Regrettably with increasing frequency, to permit data to be obtained to assist in arbitration or litigation.

In any attempt to evaluate the geotechnical performance of a structure or process, it is essential the benefits of careful visual observation are not forgotten. It is inappropriate to adopt a view that it will be sufficient to have installed sophisticated instruments or carried out extensive insitu testing. Instrumentation or insitu testing should be complementary to routine visual observation. For example, if using static cone test data to evaluate the effectiveness of densification of sands by vibroflotation, it would be usual to carry out only spot checks on the density achieved by the process. It follows that although seemingly trivial, good supporting construction records are essential to the usefulness of the static cone data. Such records need to identify that the entire site has been vibroflotted, include the volume of imported filling required to replace densified material and a note of the achieved amperage, or "density", at each vibroflot site. This then allows realistic comparisons between test data, as well as identification of any apparent anomalies.

Similarly, in the construction of embankments it is not sufficient to simply monitor settlement and pore pressures periodically if there is not good knowledge of the construction methods used. In particular any departures from the normal or anticipated should be recorded and may assist in evaluating the subsequent relevance of measured settlement and pore pressures. For example, it would be normal to carry out extensive in place density testing to evaluate the achieved dry density ratio and placement moisture contents. However, the volume of soil represented by such testing is a minute fraction of the total placed in the dam. Consequently, the value of such testing is limited unless it is supported by visual observation during construction, allowing a qualitative appreciation of how representative the testing has been. Again, these visual observations require good documentation, to allow their evaluation in relation to the overall performance of the structure, possibly months or years after the observations were made. It is little comfort to know that a large number of geotechnical instruments has been installed in a dam during its construction, if subsequent measurements suggest there may be a flaw in the initial design, and construction records are insufficient to check whether a rational explanation may exist for these observations.

3.2 Observational Awareness

The deliberate decision to observe and then evaluate geotechnical performance may be made for many reasons. However, in addition to the planned observation associated with some projects, most engineering structures have an unconscious or unplanned level of observation. This may range from the obvious - "Did it fall down?" or "Why has this building cracked?" to the more subtle day to day experiences and observations. This latter situation requires maintaining an engineering awareness of our (geotechnical) surroundings.

For example, such awareness may include noting that slumping is occurring in a given road cutting; that the approaches to a bridge have settled; or that driven cast-in-situ piles are being installed for a given project. By noticing such things and spending a few moments reflecting upon the observation, the geotechnical engineer is obtaining information on geotechnical performance and hopefully storing it for subsequent application in a design situation. The personal assimilation of such information hopefully will then be reflected in better engineering in the future. Whilst peripheral to the general theme of this paper, this is considered to be of great importance to the general improvement of geotechnical knowledge as it is applied to routine engineering design and construction.

3.3 Observation Methods

The means of observing geotechnical performance to allow subsequent engineering evaluation fall into three categories. However, in any major structure a combination of all three is probable, and in many situations will be desireable. These categories are:-

i) Detailed visual observation, perhaps combined with simple engineering survey or measurement.

Examples of this include measurement of settlement of oil tanks both during water test and subsequently under product load; installation and visual observation of a group of line-of-sight poles to provide a check on whether any lateral movement has been initiated during an excavation process or of a natural slope; mapping and observation of cracking in masonry walls, possibly in conjunction with taking level measurements on a supporting floor slab; performance of "standard" pile load tests; and observation and possibly measurement of the downstream seepage through water retaining structures.

ii) Insitu testing.

Insitu testing is generally regarded as a site investigation tool. However, it may also be used routinely to check on geotechnical performance. For example, in the case of ground improvement by mechanical densification, before and after testing may be employed to assist with construction control and contract administration. Similarly, where staged construction of embankments on soft ground is proposed, the decision as to when the next stage may be added would usually rely on insitu strength testing (in conjunction with settlement and possibly pore pressure measurement) to support laboratory based predictions.

In the case of construction control type testing, simple, relatively inexpensive and easy to interpret test procedures are usually employed. Static cone testing or Standard Penetration Tests would be common, at least as a first order of tests. Depending upon the reason for the ground improvement, other equipment such as a self boring pressuremeter, dilatometer or screw plate may subsequently be used to better confirm that the design assumptions have been fulfilled. For embankments on soft ground the static cone or vane shear test would probably be used as the primary indicator of performance. Subject to the design assumptions and nature of the project further verification of parameters by careful insitu sampling and laboratory testing may be considered.

iii) Instrumentation

By careful selection of instruments from the wide variety currently available, following appropriate installation techniques and ensuring relevant data collection procedures are adopted, it is possible to closely monitor most aspects of geotechnical performance. Judicious application of such geotechnical instrumentation allows increased understanding of and confidence in design assumptions and of the application of new technology (eg Reinforced Earth). However, it should be recognised that instrumentation has become a specialist area of geotechnical engineering, in its own right. Furthermore, due to the relatively high cost and complexity of much of the available geotechnical instrumentation, its selection, installation and observation should be treated with respect.

Peck (1969) in his Rankine Lecture discusses the "Advantages and Limitations of the Observational Method in Applied Soil Mechanics". This so called Observational Method is principally aimed at permitting interactive design procedures, or adoption of higher risk assumptions in initial design on the basis of being able to incorporate change as appropriate during construction. In its successful application lie principles fundamental to the wider satisfactory evaluation of geotechnical performance. These include satisfactory recognition of the fundamental factors influencing a given project; an understanding of the geology of a site; being conscious of what observations or measurements might be made and what action, if any, is needed when they are observed; and the importance of reliable and relevant observations and therefore of the need for careful planning and selection of instruments. Peck also recognises that there is no substitute for careful visual observation and for simple or "quick and dirty" tests to ensure that the project is proceeding as planned. Reporting must be prompt and contain all observations, and the data then evaluated by relevant personnel at the time, not when it becomes convenient and possibly too late.

Whilst visual assessment and insitu testing make a vital contribution to our performance observations, in the context of this paper it is proposed to only consider instrumentation in any detail. This in part arises from the common sense nature of most visual observations and the general familiarity of most geotechnical engineers with insitu testing techniques, but also from the high costs typically involved in geotechnical instrumentation. Because of these high costs, it is important that the decisions made in choosing and installing instruments are based on good and up to date information, pertinent to the particular design problem being considered.

4. GEOTECHNICAL INSTRUMENTATION

4.1 General

It is neither practical nor appropriate within a paper such as this to attempt to describe fully the range of geotechnical instrumentation which is available, nor to consider in detail the relative merits of and installation methods appropriate to various instruments. The Australian Geomechanics Society (Victoria Group) recognised the increasing importance of geotechnical instrumentation and the general lack of understanding of the applications and relevance of such instrumentation, and in 1987 ran a fourteen hour extension course on Geotechnical Field Instrumentation. In the foreword to the volume containing the papers presented (Australian Geomechanics Society, 1987), it was noted that:-

"The application of field instrumentation to geotechnical engineering has become a multi-million dollar industry involving a wide range of sophisticated equipment and procedures. While the general principle behind the use of these systems concerns the collection of relevant data to assist in the evaluation and understanding of engineering performance, the details of how this can be best achieved for any specific situation can be extremely complex, time consuming and costly. There can be many problems and pitfalls for engineers contemplating instrumentation systems and unfortunately these are rarely referred to in the technical literature...."

The course was aimed at resolving some of these issues Hanna (1985), in his very comprehensive text on Field Instrumentation in Geotechnical Engineering, addresses in detail the wide range of instruments which are available and includes a large section on field applications and performance. As such, his book draws on wide ranging field experience and when used in conjunction with the extensive references included, is a most valuable contribution. This should be consulted by anyone embarking upon an instrumentation programme. Dunnicliff (1981) also provides useful guidelines for selection and use of instrumentation. Although specifically referring to long term monitoring of water retaining embankments, he offers sound experience based advice relevant to the wider applications of instrumentation in geomechanics. In particular Dunnicliff stresses the need for reliability and durability and suggests that this is more likely to come through simplicity. Although recognised as difficult to necessarily achieve, reliability is described as the characteristic of an instrument whereby a reading, if obtainable, will be the correct one. In other words, no reading may be less dangerous than an incorrect reading.

4.2 Measurement of Deformation

Deformation measurements which may be of interest to geotechnical engineers include settlement (total and differential), lateral displacement (total and relative) and heave.

For such measurements to have value, it is critical that a suitable stable datum or bench mark be established which will be free from any influence of the "structure" being measured or any other regional influence. Whilst this may appear to be stating the obvious, the influence of regional dewatering or reactive soils on surface bench marks have been overlooked at times in the past.

For example, Raisbeck and Pedler (1985) describe the monitoring of regional settlement in Victoria's Latrobe Valley, where open cut brown coal mining is carried out for electricity production. For many years the primary datum for such monitoring had been thought to be on bedrock. However, it subsequently transpired that this was actually on sediments which were settling under the influence of dewatering associated with open cut operations (Raisbeck, 1980).

This demonstrates the need to retain an open mind at all times when trying to evaluate what might otherwise appear anomalous results. A further salutary lesson from the observations made in the Latrobe Valley is the very wide area that can be influenced by the dewatering of sub-artesian aquifers. For example, Raisbeck and Pedler (1985) indicate that dewatering at the Morwell open cut has resulted in almost two metres of subsidence adjacent to this open cut, and up to 350mm of subsidence some 16km away. Settlements of this order clearly could have significant influence on existing as well as proposed structures, and need to be recognised well in advance of their occurrence or of any planned new development. Similarly in any areas where dewatering is proposed, particularly of confined aquifers, the regional influence of such activities should be carefully considered, even if such dewatering is to be of limited duration, such as during construction. In such circumstances particular attention to detail is needed when trying to establish a suitable datum for evaluation of settlement and for control purposes (Raisbeck 1980, 1986, Chapman, 1987).

Having established a datum, the choice of measurement system needs careful consideration. Dunnicliff (1981) is clear that surface monuments are preferable, at least for long term monitoring. Consistent with the theme that simplicity, reliability and maximum use of visual observation techniques should be aimed for, it is considered a fundamental form of displacement measurement incorporating simple engineering survey techniques should be mandatory to any scheme where deformation measurements are required.

However, the loss of accuracy associated with using distant bench marks may be sufficient that the expense of establishing insitu measurement is also necessary. Raisbeck (1986) illustrates how the use of engineering survey from a bench mark some 500 metres from a structure resulted in errors in setting out the steelwork of up to 10mm, as shown on Figure 1. At this site the repeatability of settlement measured using an array of magnetic settlement gauges (see below) was within 2mm. This data could have been more reliably used for set-out purposes than the traditional survey data.

Notwithstanding the above, even if reliable and repeatable surface measurement of vertical displacement can be achieved, such measurements are of little value if the variation in compressibility with depth, or in different layers, is required. Where some form of stiffness categorization over the depth of influence is necessary,direct measurement of settlement/strains at various levels within the depth of influence of the applied loading should be considered. Having recognised that subsurface measurement of displacement is required, the selection of instrument type will be dependent upon need, and needs careful consideration. Hanna (1985), Dunnicliff (1981) and Chapman (1987) describe in some detail various techniques which are available and discuss their relative merits for given applications.

Measurement of settlement due to a structure, dewatering, etc., at various depths can be relatively inexpensively obtained using multi point vertical settlement gauges installed in boreholes. Such devices typically consist of ring magnets located on telescopic casing, but attached to the sides of the borehole at desired locations. By means of a steel tape attached to a reed switch, it is then possible to determine the relative locations of each magnet to within about ± 2mm (as shown on

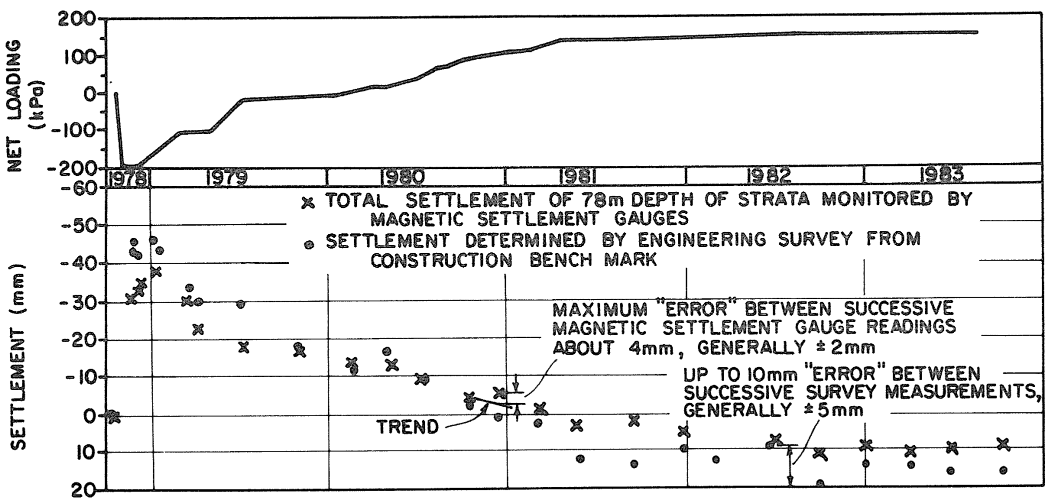

Figure 1 - Comparison of settlement by magnetic settlement gauges and engineering survey at No. 1 chimney foundation, Loy Yang Power Station (after Raisbeck 1986).

Figure 1). Better accuracy can be achieved with more sophisticated reading systems (Chapman, 1987).

If settlement measurements are required during the course of construction, say of an earth dam, the above simple settlement gauges may not be appropriate due to potential interference from construction equipment, or burial of the reference points. In such circumstances overflow weir devices or remote reading hydrostatic cells may be employed. Such devices rely on the manometer or U tube principle, with either remote direct reading of one leg of the manometer, or electronic/pneumatic measurement of the applied pressure. For direct reading it is necessary for the readout station to be at the same elevation as the settlement cell, which can cause problems in large structures. Hanna (1985) and Chapman (1987) discuss these matters in more detail.

For lateral displacements, below surface instrumentation may range from a simple shear strip or strips installed in a grouted borehole and which break if relative displacement occurs, to the Trivec borehole probe (Koppel et al, 1983), which permits vertical strain as well as lateral rotation between discrete points to be determined. Whilst shear strips only provide a yes/no answer, they are relatively inexpensive and can be automatically interrogated, thus acting as a remote early warning system for discrete slope movement. On the other hand, the Trivec probe, or sliding micrometer, is costly to install and requires time consuming semi-manual data collection, but with the advantage that a full three dimensional "picture" of movements and strain can be determined. The more widely known sliding inclinometer allows good information on lateral movement, but does not permit measurement of vertical strains. The introduction of semi-automatic logging of inclinometer output and on-site microprocessing of this data has made the use of such instruments easier and therefore increased their application. Data collection is nevertheless time consuming, particularly in remote locations, and therefore may also be expensive.

4.3 Measurement of Pore Water Pressure

The behaviour of saturated soils under applied or self imposed loads is controlled by the principle of effective stress, $\sigma' = \sigma - u$, as described by Terzaghi. As such the pore water pressure, u, has a major influence on geotechnical performance. In many unsaturated soils, and in jointed rock masses the pore water pressure (or suction) is also of extreme importance in determining engineering performance. Of particular interest is the influence of pore water pressure on shear strength and slope stability, and its dissipation with respect to consolidation. However, prediction of pore water pressures in practical field situations is inherently difficult and often inaccurate due to the scale effects associated with laboratory testing and the problems associated with predicting groundwater flow under changing stress conditions in a complex soil or rock mass.

For these reasons, the insitu measurement of pore water pressure by piezometer has generally assumed greatest importance in the development and utilization of geotechnical instrumentation devices. Fell (1987) provides a valuable summary of typical applications for pore pressure measurement. When proposing the use of piezometers, and subsequently evaluating the results obtained from them, Fell emphasises the need to understand the geotechnical engineering principles relevant to the particular problem being considered, and to recognise the influence of geology on this problem.

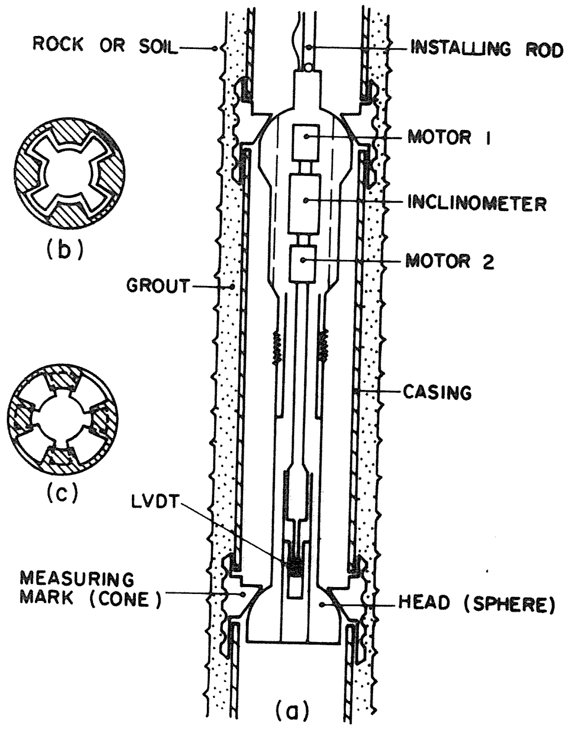

Figure 2: TRIVEC Sliding Micrometer (After Koppel, et al, 1983)
a) schematic view of the instrument
b) sliding position
c) measuring position.

The landslide problem illustrated in Figure 3 illustrates some of these points. Each of the piezometers shown is recording steady state conditions. However, the claystone band acts as an aquiclude, and being overlain by the coal seam aquifer behind the slide, permits high pore pressures to be generated along the claystone even though the watertable in the overlying conglomerate may indicate lower static water pressure. Hence, if an insufficient number of piezometers were installed, or they were incorrectly positioned, use of data from these piezometers to evaluate this landslide problem could easily be misleading. Similarly, a good understanding of the geology of this site would be essential to a satisfactory evaluation of the problem. With too few exploratory boreholes or trial excavations a misleading interpretation of the mechanism might occur. Consequently, any "evaluation of the geotechnical performance" of this problem could be quite erroneous.

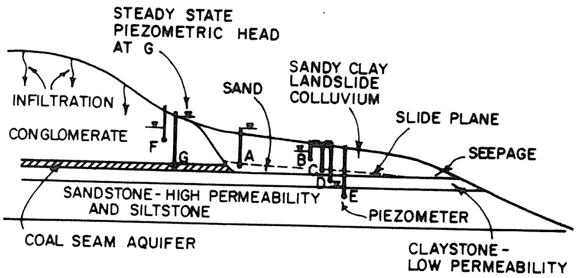

Figure 3 - Landslide piezometric conditions (after Fell, 1987)

The basic principle behind measurement of pore water pressure by piezometer is the introduction of a porous element into the soil/rock mass which will preclude soil from entering the measuring device but allow flow of water, and in some circumstances air, through to the measuring element. The measuring element may range from simply an open standpipe (Casagrande piezometer) to an electronic pressure transducer.

Open standpipe piezometers provide reliable long term measurement of static pore water pressure, but due to the requirement that there be an inflow (or outflow) of water to register a change in water pressure they are inappropriate to register transient changes in pore pressure unless they are installed in very permeable soil with a high recharge capacity. Where a quick response time or remote recording of pore pressures is required, it is necessary to use one of the hydraulic, pneumatic or electrical piezometer types. The various types of piezometer and their advantages and disadvantages are described by Dunnicliff (1981), Sherard (1981), Hanna (1985) and Fell (1987) and these references should be considered when contemplating piezometer installation.

Sherard (1981) concludes that double tube hydraulic piezometers require more skill and care in their maintenance than the diaphragm type pneumatic and vibrating wire piezometers, and suggests there is not comparative evidence to support the proposition that hydraulic piezometers have a longer life. In considering the relative merits of pneumatic and vibrating wire piezometers, Sherard also concludes that both vibrating wire and the better types of pneumatic piezometer are satisfactory instruments, but believes the vibrating wire instrument has substantial advantages over the pneumatic type. Sherard suggests that these vibrating wire instruments will be more widely used in the future. However, Fell (1987) points out that the pneumatic instruments are the preferred instrument for dams in the USA. The choice between the two therefore will probably involve personal preference, cost constraints and availability. Due to their relative simplicity, low cost and ease of use, pneumatic piezometers are believed to be in more common use in Australia for "everyday" monitoring of pore pressures.

4.4 Measurement of Stresses

In this paper it has been necessary to limit the scope of topics to be covered. As such no attention has been given to rock mechanics and consequently measurement of insitu or applied stresses in rock has not been specifically addressed. This topic alone is worthy of a comprehensive paper.

The stresses imposed on the ground by most engineering structures can be reasonably well predicted, and the use of instrumentation to establish or confirm such predictions is not usual, except for research purposes. However, for retaining structures the prediction of earth pressures is still often based on empirical design methods developed over the past few decades. Even then, there remains a divergence of opinion as to how stresses will be distributed (e.g. Golder et al, 1970). Notwithstanding the presence of such empiricism, except when using the "Observational Method" to provide potential savings in tie back or strutted support, or increased confidence in the design (e.g. Peck, 1969, Gartung, 1986), it is not common to routinely install stress or load measuring devices for retaining structures. However, it is common that earth pressure cells form part of the instrumentation package installed during construction of large water retaining structures (Penman and Kennard, 1981; Murley, 1987), so that when considered in conjunction with pore pressure measurements, a better estimate of effective stress can be obtained.

The measurement of stress in a soil or rock mass typically requires the installation of a diaphragm or pressure transmitting element. Because earth pressure is likely to be strongly directional, it is necessary in advance of installation to have an understanding of the probable principle stress directions and to orient the pressure calls accordingly. If not able to be satisfactorily predicted, rosettes of pressure cells may be required.

The two types of pressure cells in common use are the diaphragm cell and the hydraulic pad. The diaphragm cell type rely upon sensing the deflection of a diaphragm using, for example, bonded electrical resistance strain gauges or a vibrating wire transducer. Due to difficulties introduced with the resistance of long lead wires to strain gauges, vibrating wire sensors are more widely accepted (Cummins, 1987). Hydraulic cells are typically oil filled "flat jacks", with the applied external pressure transmitted to the oil and then measured remotely via a connecting tube and pressure gauge/transducer. Alternatively the pressure can be measured at the cell by means of an electrical or pneumatic pressure transducer connected to the pressure cell, with appropriate lead wires taken to a remote reading station.

The mere presence of a pressure cell within an earth mass may be sufficient to alter the applied stresses in the vicinity of the instrument, due to the probable difference in stiffness between the soil mass and the instrument. To minimize this effect a pressure cell should match as well as possible the anticipated modulus of the soil mass and have a high diameter to thickness ratio.

4.5 Application of Instrumentation

The above only skims the surface of available geotechnical instrumentation. However, it should be appreciated that it is only through careful investigation and observation of our past successes and failures that geotechnical engineering has reached the level of sophistication it can currently claim. Such observation has frequently required development and installation of new instruments and thorough analysis of the data obtained. Through such evaluation, revision of design parameters and analysis techniques has been possible. Instrumentation has been a very useful servant in this respect.

For the future we should guard against complacency with respect to ongoing use of instrumentation in relevant projects. Whilst instrumentation should continue to be our servant, it is important to recognise the need for careful consideration of instrument type, and number, and not to allow the appropriate engineering decision to be overridden by cost constraints or lack of client appreciation. Having made the decision that instrumentation is required, and obtained approval for its use, it then remains to ensure that appropriately skilled personnel are used to plan, select, install and monitor the instruments and ensure prompt and appropriate evaluation of the data obtained is then carried out.

5 INTERPRETATION OF PERFORMANCE OBSERVATIONS

5.1 Selection and Location of Instruments

For the evaluation of performance to be carried out with a maximum of objectivity, it is essential that observations made are reliable, and complete. Similarly, where instrumentation is installed, its selection must be carefully considered to ensure the maximum level of reliability is achieved. Having chosen what instrumentation is required, it is then important that the location of instruments be well considered and installation is carried out carefully by suitably experienced people. By applying considerable care to each of these steps the chances of subsequently obtaining data which can be relied upon with confidence are considerably enhanced. Redundant or back-up instrumentation will often be necessary, due to the almost inevitable loss or malfunction of some instruments after the rigours of construction. However, by careful consideration of the location of instruments, or measuring points, rationalization of the number of back-up instruments and measurements will be permitted.

With respect to deciding upon the location of instruments, and ensuring reliable and relevant information is obtained, a sound understanding of the probable behaviour of the proposed structure is required. For example, areas of potentially high stress or deformation gradient should be avoided. In earth and rockfill dams such zones may occur within filter layers, due to the probable differential stiffness between the earth core and adjacent rockfill. Trying to relate the results of instruments in such zones could be misleading because, for example, the stress conditions relating to a total pressure cell could be quite different to those at the location of a nearby piezometer.
It is common to monitor the settlement of large oil tanks under both water test, and product load. It is usually very difficult to monitor internal settlements by direct measurement, and remote measurement by settlement tubes or gauges is rarely specified. Consequently, monitoring tends to be limited to edge settlement at a few discrete perimeter points. In the absence of internal measurement, understanding the settlement behaviour and the contribution of various subsurface layers therefore can be difficult. For this reason alone, it becomes important to understand the probable nature of the edge settlements and to obtain an adequate number of relevant measurements in the zone immediately adjacent to the tank. Similarly, it is important to ensure settlement readings are carefully correlated against the applied load and the time of its application. By only measuring at a number of discrete and conveniently chosen locations, qualitative data may be all that is achieved. Whilst this may be satisfactory for routine water testing of the majority of tanks, it may result in failing to recognize a potential problem if it exists, or arises.

5.2 Objective Evaluation

Fundamental to effective or useful evaluation of any observed geotechnical performance is a thorough understanding of the geotechnical engineering principles pertinent to the particular problem being considered. Having obtained observational or instrumentation data, analysis should be carried out objectively. It will be human nature to try to fit the data to a preconceived model for behaviour. However, if a satisfactory fit is not achieved of all data points which are considered to be reliable, both the model and the relevance of the data to the particular model need careful consideration. For example, time effects, due perhaps to construction sequencing or pore pressure dissipation, may influence the measured data in relation to the model behaviour. Any attempt to massage the soil model to suit the data obtained, or to dismiss data which does not fit the initially chosen model, suggests a lack of confidence in either the data or the design model, and doesn't allow that the real behaviour may be quite different to that initially assumed. Such lack of objectiveness should be assiduously avoided.

This is discussed in more detail by Leroueil and Tavenas (1981), who suggest that back analysis of case histories has become a frequently used, if not preferred, method of improving geotechnical knowledge. They recognise this approach generally is useful and appropriate. However, they suggest that through a lack of objectiveness in selection of instrumentation, or in the fitting of observed data to a soil model not based on sound soil mechanics principles, back analysis can prove to be unreliable and in some cases misleading. For example, they suggest it is often reported that "settlements were correctly estimated but the model failed to accurately predict pore pressures and effective stress". Given that these are all closely inter-related, a more appropriate conclusion may have been "a satisfactory model to predict the data obtained could not be found".

Walker (1983) discusses how reinterpretation of settlement data from instrumented test embankments on soft clay making allowance for probable behaviour and time difference in construction could alter significantly the conclusions reached. In this example, three test embankments were constructed, one with no subsurface drainage and the other two with vertical drains, either sand or cardboard wick. The initial interpretation of the data obtained was made on the basis of comparing measured settlement against a linear time scale (Fig 4). The conclusion reached is reported to have been that accelerated settlement required the use of one of the types of vertical drain. Walker's (1983) interpretation adopted the more commonly used logarithm of time basis, and made allowance for the considerably longer construction period for the wick drained embankment (86 days) to the sand drained embankment (21 days) and the undrained embankment (15 days). Analysis of the data presented in this form (Figure 5) showed that comparable creep rates existed for each embankment. Furthermore, it was concluded that the influence of the installed drainage was to increase the probable total settlement that would occur. Hence, contrary to the inference of the initial interpretation of the data, it appeared desirable that the prototype embankments be constructed without any vertical drainage.

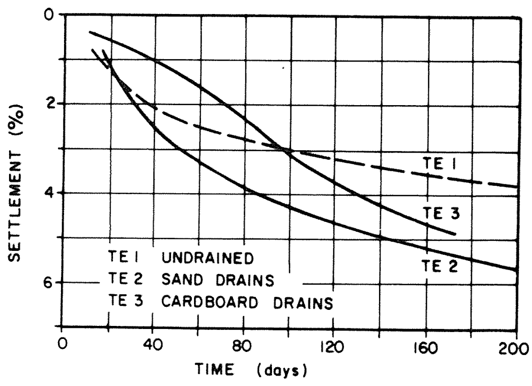

Figure 4 - Embankment Settlement - Linear Time (Walker, 1983)

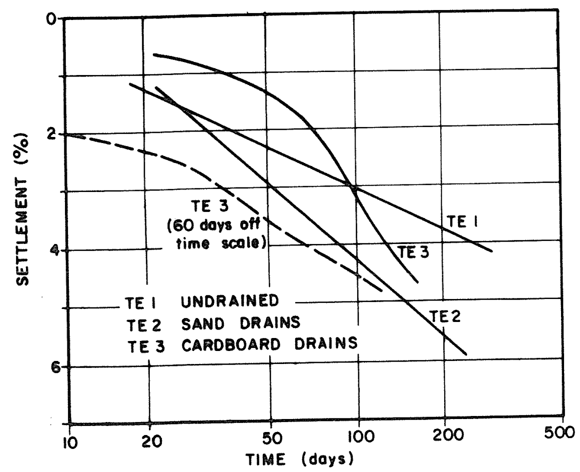

Figure 5 - Embankment Settlement - Log Time (Walker, 1983)

An example of trying to fit a model to observed behaviour is the work of Skempton in relation to the stability of cuttings in London Clay. Because of his initial assumptions made with respect to the soil model, specifically that the London Clays are sufficiently fissured to have high permeability, Skempton (1964) found it necessary to suggest that the effective stress parameters were time dependent. This concept of peak frictional values reducing to residual parameters by only the passage of time and without displacement is inconsistent with the usual concept of residual strength. This is illustrative of how fitting data to observations without fully considering the principles of soil mechanics can be misleading. As expected, Skempton (1970) realized this and reanalysed the original data. However, he again assumed rapid equilibration of pore pressures and concluded that it was only the effective cohesion which is time dependent. Again his initial assumption with respect to pore pressure was controlling his evaluation.

In 1977, Skempton again presented the results of further back analyses. He then reviewed all his initial assumptions and concluded that the principle reason for the delay of many years after construction before the failures occurred was the very slow rate of pore pressure equilibration. At this time he also recognised the importance of the insitu strength measured along joints and fissures. Hence, although initially adopting incorrect assumptions, through continued objective review of the data and of his assumptions, Skempton was able to resolve a rational explanation for the behaviour of these cuttings and for design of future cuttings.

These examples reinforce the point that not only is it important to obtain reliable, relevant and unbiased data, the interpretation of this data in relation to evaluating the performance of the structure being monitored must be carried out using sound soil mechanics principles, and maintaining a high level of objectiveness in trying to satisfy any previously adopted performance models. Calibration of a model against field data whilst neglecting this can be misleading or inappropriate, particularly if extended to another application. As put by Mitchell (1986), "we need to learn better how to expect the unexpected, especially when confronted with new problems in new environments or settings".

6 EXAMPLES OF BENEFITS ACHIEVED BY EVALUATION OF PERFORMANCE

6.1 General

Through the astute evaluation of past geotechnical performance, the art or science of geotechnical engineering has been advanced, and confidence has been provided in new technology. This has highlighted the dynamic nature of geotechnical engineering and allowed the inherent empiricism in many design procedures to be better understood. Examples of how such design procedures have developed through observation and research using field situations fill a large proportion of the technical literature and have been periodically summarized in "State of the Art" text books, such as Terzaghi and Peck (1948 and 1967) and Winterkorn and Fang (1975); in an abundance of specialist texts such as Tomlinson (1977) and Poulos and Davis (1980) on piles; and in the proceedings of specialist conferences or symposia such as published from time to time by the ASCE, ASTM, ICE and IEAust.

To attempt to illustrate the influence of other than a small fraction of such research in a paper such as this would be futile. However, by reviewing the literature produced over the past 20 years or so it becomes very apparent that our working knowledge of the fundamental principles of geomechanics as they apply to geotechnical engineering has been substantially increased. This has rarely been through a "lighting bolt" type discovery but rather through the constant questioning and review of past procedures, and attempts to better relate observed with predicted performance. This has only been possible by the accurate collection and reporting of field observations with which to make such comparisons. The availability of such data has also permitted calibration of computer based analytical techniques so that analysis of previously "impossible" interactive problems can now be carried out routinely (e.g. Fraser and Wardle, 1976, Poulos, 1979).

Similarly, astute interpretation of field data has resulted in the development of new design procedures, which although semi-empirical are rationally based and permit economic development where otherwise it may not have been practical. For example, the development of a design procedure for high capacity bored piles socketed into weak rock (Williams, Johnston and Donald, 1980) permitted rational design of the foundations for the elevated structures of the Westgate Freeway in Melbourne. Applications of this method in other areas where soft rock foundations exist has also been demonstrated to provide substantial savings when compared to alternative design procedures previously in use (e.g. Ervin 1983).

The prediction of settlement of foundations on sand deposits has been frequently addressed in the literature (e.g. Parry, 1971; Schmertmann, 1970; Jorden 1977; Schmertmann et al, 1978) with substantial variation in approach and subsequent predicted values. This has recently been reviewed again by Bowles (1987), who suggests a further computational procedure. He then compares the predictions so obtained with measured settlements using examples reported in the literature and achieves good agreement. With such ongoing iterative review and back analysis of measured performance, improved future predictions are probable. Unfortunately the estimates of settlement provided in routine geotechnical reports are frequently scoffed at by structural engineers who believe that a guess would be better, even though refinements in computational methods have occurred.

The author has experience of one such structural engineer who quite proudly advised that he always halves the estimates given to him. Whilst this may be his own personal evaluation of geotechnical performance, possibly based on experience, it is strongly suspected he is unable to recognise the difference between a soundly based estimate and a "guessed" estimate when provided with his geotechnical report. It is therefore appropriate when design predictions are given, that the recipient be aware of the bases of such predictions and of the continued upgrading of our predictive abilities.

To illustrate further how careful observation and evaluation of performance has resulted in positive benefits, three examples will be presented. These have been chosen to illustrate a range of geotechnical problems, from foundation design to introduction of new technology to planning and execution of remedial works.

6.2 The Axial Capacity of Piles in Clay

The evolution of design procedures for estimating the frictional resistance of piles driven or bored into cohesive soils is an excellent example of the influence careful evaluation of accumulated data from performance records has had on current procedures. Whilst the use of piles to support structures goes well back into history (Kerisel, 1985), it was not until relatively recent times that sufficient data were accumulated to permit pile design in clays from laboratory data with reasonable confidence.

In 1943, Terzaghi claimed "Our knowledge of the influence of the method of installing the piles on the skin friction and on the intensity of the shearing stresses ... is still rudimentary and the prospects for evaluating this influence by theory are very slight". Consequently, the application of pile driving formulae tended to be relied upon. However, Terzaghi (1943) noted that such dynamic pile formulae are fundamentally deficient and can be used only as an empirical yardstick, to which local experience should be added.

Moore (1949) attempted to arrive at a more rational design method for pile design, and carried out pile load tests to evaluate his design method, finding reasonable agreement. This procedure, relying upon frictional resistance rather than cohesion, does not appear to have been widely used but was probably the first attempt at rational pile design procedures. However, it is interesting to note that Moore's paper attracted substantial discussion, mostly complimentary and informative. Sowers in his discussion to the paper notes that "Probably many so-called practical engineers will scoff at the detail involved in a rational analysis of a pile foundation ... Foundation engineering will find its place with other branches of civil engineering when as much time and effort are put into it as the relative cost of the foundation warrants". Although significant advances in design methods have occurred since then, this comment probably can be equally applied today.

During the 1950's several authors presented the results of pile load tests and their interpretation of them. Tomlinson (1957) back analysed much of this data plus additional load test data and suggested that a limiting adhesion applied for piles driven into clay, with the ratio of achieved adhesion to cohesion decreasing with increasing shear strength (Figure 6). Based on this, tentative design criteria were offered, with design adhesion falling to about 40% of cohesion for stiff clays.

Skempton (1959) examined the available adhesion for bored piles in London Clay, and set the basis of design procedures for bored piles in clay still commonly used today. From his work, it was suggested the adhesion, c_a, could be related to the cohesion, c, by a factor $\alpha = 0.45$, except that a limiting value of about 100 kPa was placed on adhesion.

It appears little advance was made over the next decade, with the recommendations presented in Terzaghi and Peck (1967) being essentially a restatement of the above. Terzaghi and Peck also suggested in this text that "any attempt to establish the rules for the design of pile foundation necessarily involved radical simplifications, and the rules themselves are useful only as guides to judgement". Hence, though there had been significant contributions towards rational design of piles in clay through careful evaluation of performance data, we were still being reminded to rely upon judgement.

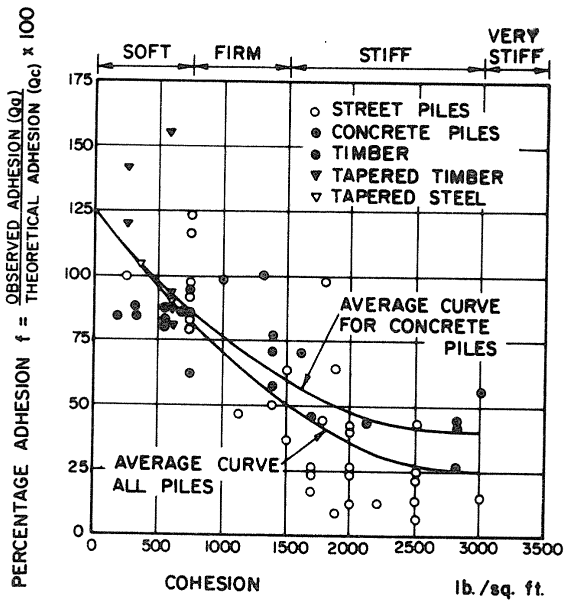

Figure 6 - Relationship of observed adhesion (expressed as a percentage of theoretical adhesion) to cohesive strength of clay (after Tomlinson, 1957)

Reliance on empiricism to modify classical soil mechanics methods in the design of piles in clay continues today, but through further evaluation of pile load tests, the guidelines have been refined and confidence levels increased. Tomlinson (1971), on the basis of his research, presented adhesion factors for displacement piles driven into stiff clay which take account of the nature of the strata penetrated by the pile. The adhesion factors given by these curves (Figure 8, Tomlinson, 1977) are higher than presented in many texts and codes, and are understood to have only limited acceptance. Tomlinson justifies his recommendations by comparing them with the results of 93 pile load tests, or again by evaluation of geotechnical performance.

More recent refinements in design methods introduce the effect of pile length and allow for normalizing the undrained shear strength with respect to effective overburden pressure. For example, Semple and Rigdon (1986) analysed in detail the results from over 50 pile load tests from 24 clay sites, and concluded that the peak skin friction should be reduced to account for pile scale effects. Figure 8 illustrates their suggested correlation between the adhesion factor and the strength ratio c_u/σ_v', thus reflecting the degree of soil overconsolidation. The influence of pile stiffness is then introduced through a reduction factor, F, which depends upon the pile aspect ratio L/D. Hence $c_a = \alpha_p F c_u$ related predominantly to steel pipe piles in offshore applications. For conventional foundation engineering piling, application of this length reduction factor would rarely be necessary, because L/D will typically be less than 50

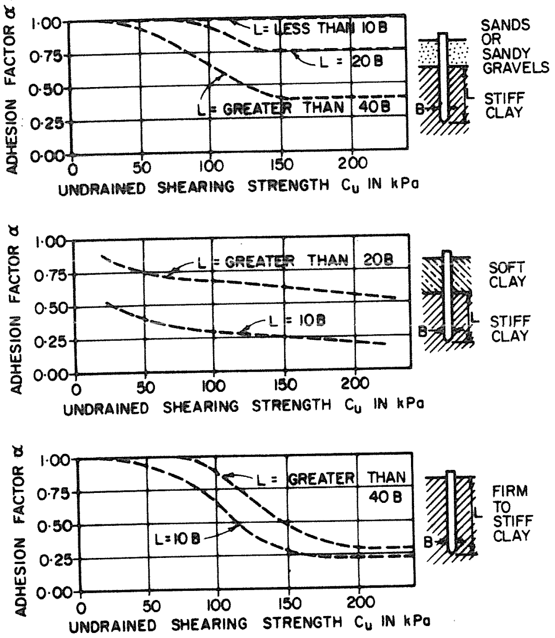

Figure 7 - Design Curves for Adhesion Factors for Piles Driven into Clay Soils (Tomlinson, 1977)

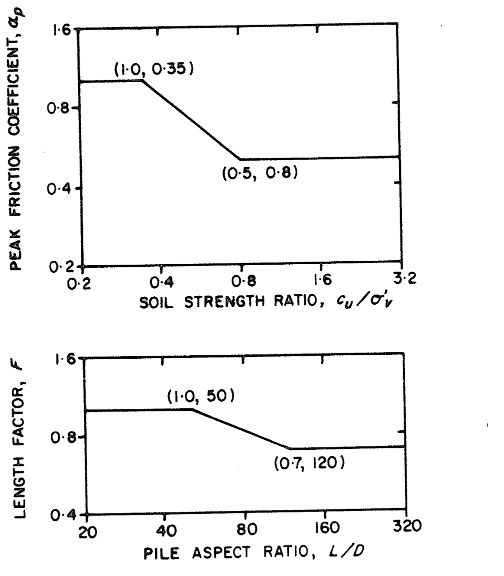

Figure 8 - Criteria for Capacity Prediction (Semple and Rigdon, 1986).

Hence, through the efforts of many researchers and the collection and analysis of data from many sites throughout the world, semi empirical design methods for determining the axial capacity of piles in clay have been developed, and now can be used with reasonable confidence. However, as reminded by Terzaghi and Peck (1967), judgement must always form part of geotechnical design because more often than not the ground conditions for design will not exactly match those from which the design basis evolved. In such situations, test piling to confirm or modify design assumptions is appropriate and will often result in significant savings (e.g. Ervin and Pells, 1985), and at least will provide confidence. Such test piling should desirably be to failure, to allow any savings to be maximised and to permit further re-evaluation of our geotechnical knowledge. Above all, it is desirable that the results of such testing, even if routine, are reported in the literature so that broad scale evaluation by others is also permitted.

6.3 Reinforced Earth Walls

Perhaps one of the best examples of how the combination of an enquiring mind, good visual observational skills and detailed evaluation of performance have resulted in a new and widely used concept, is the evolution of the Reinforced Earth wall. Vidal (1978) describes how he had been lying on the sand of a Mediterranean beach and in the course of playing with the sand, and some pine needles, conceived the idea that granular material could be reinforced by discrete flexible strips. He then set about several years of theoretical and experimental studies until he had the concept well developed.

During this period of research, Vidal claims to have theoretically examined the use of the reinforced earth concept in a number of engineering applications. However, the ideas then had to be sold to the community. It was not until 1966, three years after his first publication on the method, that he was able to put his ideas into practice, successfully. Over the next few years several Reinforced Earth structures were successfully constructed, a number of which were instrumented.

Through the evaluation of the performance of these early structures, supported by insitu measurement from this instrumentation, Vidal claims he was able to demonstrate the effectiveness of his original concept. However, with the natural conservatism and skepticism of the engineering profession, such acceptance has not always come easily, with experimental walls often being required prior to general use of the system. Consequently, research has been carried out in many countries to satisfy local authorities of the validity of the process. The ASCE Symposium on Earth Reinforcement (1978) presents the results of many such applied research endeavors.

Through this type of evaluation of performance, Reinforced Earth structures have been widely accepted and are now routinely employed throughout the world. Without thorough and well documented field evaluation of early structures, it is improbable such wide acceptance would have been achieved, in what is a relatively short time frame for engineering.

An example of the perceived need for detailed field evaluation of performance of a Reinforced Earth structure prior to its acceptance is presented by Yong (1983). This refers to the first Reinforced Earth wall built in New Zealand, for the New Zealand Ministry of Works. Through instrumentation and careful observation during construction, data were obtained which suggested the design method adopted by the Ministry of Works was conservative and therefore that the structure, in principle, would be entirely satisfactory. Consequently a second thirteen metre high and 140 metre long wall was sanctioned (Yong and McLarin, 1985) and also thoroughly instrumented during construction.

The instrumentation adopted was aimed at:-

- identifying the distribution of tie tension along selected reinforcing strips;
- measuring the deformations of the face of the wall;
- measuring the settlement of the foundation;
- determining the friction between special short strips and the soil by pullout tests;
- determining the rate of corrosion of the strips.

Consequently, a comprehensive array of instrumentation was installed, (Figure 9), at three sections along the wall, with monitoring continuing after the completion of the project. In evaluating the data collected up to the end of construction, Yong and McLarin concluded the adopted static design theory was satisfactory, and confirmed that some of the design assumptions adopted by the Ministry of Works designers were conservative (e.g., the apparent friction between the reinforcing strips and the soil).

As a result of these two projects, Reinforced Earth has now been accepted as an appropriate construction process in New Zealand, thus demonstrating the practical and economic benefits which can be achieved by careful planning and evaluation of the geotechnical performance of structures.

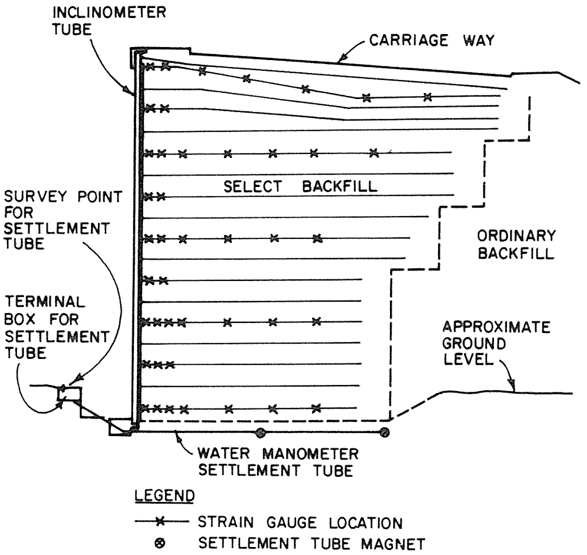

Figure 9 - Ngauranga Reinforced Earth Wall, Instrumented Section (Section through wall). (After Yong and McLarin, 1985.)

6.4 Remedial Works at Silvan Dam, Melbourne

Much of the geotechnical instrumentation in common use today has been developed to satisfy the needs of dams engineers to monitor their structures. This need has arisen historically as a consequence of failures and legislation demanding routine surveillance (e.g. Penman and Kennard, 1981), as well as from a desire to better understand the design principles applicable to such structures. It is also probable that in view of the scale of many dam projects, it has been easier to convince owners of the need for and benefits of close observation, both visual and by means of instrumentation.

Silvan Dam, located about 40 km east of Melbourne, Australia, was commissioned in 1932. As such the design of this dam predated soil mechanics design principles as they developed after the work of Terzaghi. The dam has a maximum crest height of about 45 metres, and a crest length of about 600m. It consists of a central cellular concrete core supported by earth fill shoulders with 3 to 1 upstream and 2 to 1 downstream batter slopes. The construction of this dam is described by Kelso (1934) in a fascinating paper containing excellent records of detailed visual observation. As an aside, Kelso also reports on compaction control testing procedures used at the site which were similar in principle to those in common use today. This is the first known use of this type of control testing procedure and predates the work of Proctor.

As described by Barnes et al (1984), monitoring of Silvan Dam was required by the designers, and provision for measurement of deflections was incorporated in the core wall. Deflection surveys have been carried out since the time of initial filling and have provided a continuous record of crest movement. In addition, seepage through the core wall internal drainage system and foundation drain has been monitored throughout the life of the dam.

By 1941, less than 10 years after commissioning, the crest had moved downstream by some 330mm at the centre of the dam and slip scarps had developed on the upstream slope. At the time, these slips were not considered to be serious, as the rate of deflection appeared to be reducing. In consequence, other than continuing to monitor the deflection and seepage, no action was initiated. However, following the formation by the Melbourne and Metropolitan Board of Works of a Dams Surveillance Unit in 1979, the behaviour and condition of Silvan Dam was thoroughly investigated. By this stage, crest deflections of up to 800mm had been measured, and the vertical displacement at the slip scarps had increased to some 430mm (Barnes et al, 1984). This investigation concluded the condition of the dam was inferior to that required by current standards. Of interest is that the investigation relied heavily upon Kelso's paper for understanding of construction methods and materials, confirming the need for maintaining careful and detailed records of construction. Kelso's observations and attention to detail in recording them, particularly considering the state of geotechnical knowledge at the time, were invaluable and contributed substantially to the information collected as part of the detailed investigation carried out some fifty years later.

In view of the findings of this investigation, it was decided to raise the stability of the downstream flank of the dam by adding rockfill (Figure 10). Prior to carrying out this work and to assist in confirming the findings of the investigation, a comprehensive instrumentation system was installed to assess the short term effects of construction and to permit enhanced long term monitoring. The instrumentation included (Coffey and Partners, 1984):-

- 23 Geosystems pneumatic piezometers installed in boreholes advanced using cable tool drilling techniques, thus avoiding the introduction of drilling fluid to the earth fill where potential opening of existing fissures by "hydraulic fracturing" was of concern.

- conversion of existing Casagrande standpipe piezometers for remote reading, using Petur mini pneumatic piezometers capable of installation within 25mm diameter tubing;

- installation of Sinco aluminium inclinometer casing in two boreholes.

In addition, it is understood additional surface movement pillars and internal settlement instruments were installed, together with an improved seepage monitoring system (Barnes et al, 1984).

Although the effect of adding the downstream rockfill toe has not yet been published, it is understood that the addition of this rockfill resulted in further downstream movement of the core wall by some 60 mm, and continued activity of the slip scarps on the upstream batter.

It is also understood that the finite element techniques used to model the original post construction behaviour were able to predict that downstream movement would occur, albeit more slowly than was observed but of greater magnitude. Because the initially observed rate of movement was substantially greater than anticipated, it is quite probable considerable concern may have arisen during construction, had the comprehensive instrumentation system not been installed and regularly monitored and interpreted in respect to the observed behaviour. As it transpired, it was possible to assess the behaviour, compare it to predictions and allow construction to continue uninterrupted and the reservoir to remain in service.

This combination of carefully made and recorded observations during construction, continued monitoring of the behaviour of the dam during service and the subsequent detailed investigation, design and instrumentation associated with remedial works, illustrates the benefits of careful and systematic evaluation of geotechnical performance. In the absence of any of the above factors, it is conceivable sufficient concern could have arisen from the presence of the slip scarps (usually taken as a sign of failure or impending failure) to have withdrawn the reservoir from service, or to have hastened unnecessarily into remedial works. Given that the scarps were in the upstream face it may never have been realized that these were due to downstream movement and not an upstream slope failure. In consequence any hastily planned and executed remedial works may have had disastrous consequences or at best resulted in heightened concern.

7 CONCLUSION

Geotechnical engineering is a dynamic art which has continually benefited from on-going re-evaluation of past successes and failures. As a result, our general understanding of geotechnical processes has been enhanced and refinements in design methods have been progressively introduced. The parallel advances in numerical capabilities and in field and laboratory investigation methods have contributed to this evolutionary process.

Through the dissemination and rationalization of the resulting geotechnical engineering principles, it is now practical for geotechnical investigations for a wide range of projects to be routinely carried out. Notwithstanding this, it is important that complacency be avoided in respect to the need for, and the requirements of, geotechnical investigations. All too often cursory or simplistic investigations are carried out, consistent with owners' perceived needs and financial restraints. The data so collected will be sparse and possibly inappropriate, and reliable design utilizing the benefits derived from evaluation of past performance is unlikely to be possible. In a large percentage of cases, the resulting inbuilt conservatism avoids problems arising. However, it is believed there is an increasing tendency to extend this philosophy to more important structures and to less amenable foundation conditions. The end result sooner or later must be a failure.

Such a failure has the potential to benefit the profession through the inevitable scrutiny and back analysis that will follow. However, it is probable the evaluation of the geotechnical performance carried out by the legal profession will be the one most remembered, particularly by the designer who believed he could get away with minimal and low cost investigation. It is feared that this legal evaluation of our performance will dominate the geotechnical profession unless the potential implications of inadequate and ill-conceived investigations are widely recognised.

This recognition process should be mindful of the substantial benefits which have arisen from careful observation of past performances, and from the subsequent objective evaluation of these observations. A number of examples of such evaluations have been presented. Any scan of the geotechnical literature will reveal that these examples represent only a small percentage of those which could be cited as having contributed to our current state of knowledge. The geotechnical engineering profession must continue to question

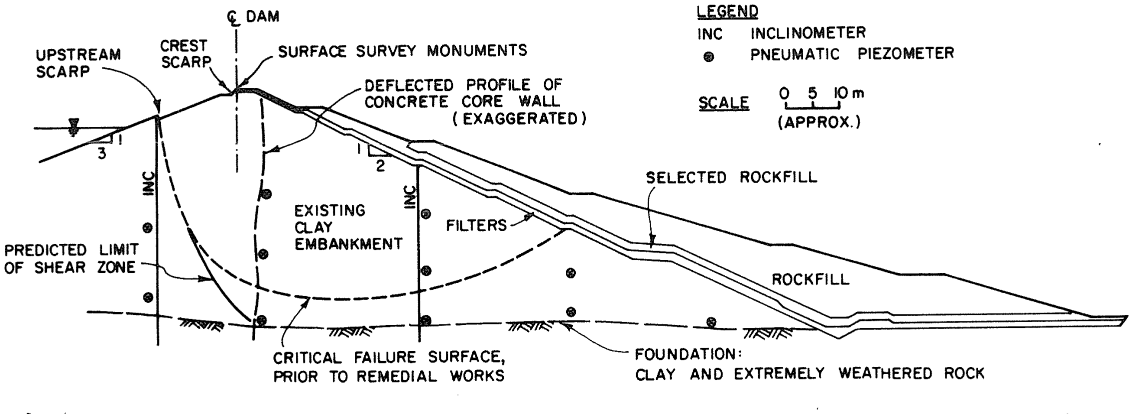

Figure 10 - Typical Section with Stabilizing Rockfill (simplified), (After Barnes et al, 1984).

objectively the performance of its structures, and to ensure that a sensible balance of theory, analysis and experience based judgement is maintained in addressing future problems.

8 ACKNOWLEDGEMENT

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