INSTRUMENTATION SYSTEMS FOR AND FAILURE MECHANISMS OF AN INDUCED SLOPE FAILURE PROJECT

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Document presented for PhD in the Department
of Civil Engineering, University of Southampton.
February 1996.

VOLUME ONE

Dedicated to the
loving memory of Gillian Grant,
11 November 1938 to 24 February 1989

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D I GRANT PhD 1996

ABSTRACT

The Selborne Slope Study brought to failure, by the process of pore pressure recharge, a nine metre high slope cut in overconsolidated Gault Clay at Selborne, Hampshire. The slope was subject to a high degree of instrumentation. Following slope collapse two trenches perpendicular to the failure were excavated to locate and identify the slip surface. The slip surface was traced and surveyed in each trench. Large diameter (300mm) and block samples were taken through the slip surface for laboratory analysis.

The author, as Research Assistant for the project, was responsible for overseeing slope formation and instrumentation installation; instrumentation monitoring and management; slope monitoring; establishing initial conditions; pore pressure recharge system construction and control; slope monitoring through failure; post-collapse investigations; site decommissioning and instrument data processing.

This Thesis presents a detailed case history tracing the geometry, nature and development of the failure. Information detailing the background to the project, site geology, pore pressure recharge system and instrumentation systems is presented. The performance of the instrumentation and its ability to describe the failure is discussed. The processes and mechanisms considered to have been operating as the slope progressed from a stable to an unstable state and eventually to collapse are described and discussed.

Notable findings of the study are the demonstration of dilation occurring at the developing slip surface during failure and the identification of a progressive failure mechanism with a *bottom-top-middle* order of slope movement. Processed instrumentation results and survey data are presented.

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GENERAL INTRODUCTION

This document represents the full case history of the Selborne Slope Study project. The document is in two parts. **Part I**, Chapters 1-4, represents a general review of slope failure mechanisms, previous studies in overconsolidated clays and geotechnical instrumentation systems currently in use. **Part II**, Chapters 5-17, discusses in detail the background, set up and results of the Selborne Slope Study.

The two sections come together as Chapter 17 where the findings of the Selborne Slope Study are discussed in relation to current thinking on slope stability and geotechnical instrumentation issues. Parts I and II with associated Plates and Figures form Volume One of the Thesis.

Detailed sets of instrumentation and survey data are presented as Appendices, which form Volume Two of the Thesis. Further information, if required, is available from the Author (see Acknowledgements).

Through much of the Thesis, dates are referred to by a day number, Day zero represents the 1st January 1989. Slope formation took place between days -500 and -460 , pore pressure recharge began on day 02 and slope collapse took place on day 196. Appendix A provides a chart correlating day number to date.

PART 1 - GENERAL ASPECTS OF SLOPE FAILURE MECHANISMS AND SOIL INSTRUMENTATION

1. INTRODUCTION

All types of slopes, natural or manmade, may be subject to instability. The processes causing these instabilities and the mechanisms involved are not fully explained by current knowledge. At present the understanding of slope failure mechanisms is based almost exclusively on post-mortem analysis of failed slopes. Since all slope failures involve some unique initial combination of triggering elements, the actual conditions within the slope at failure can never be fully or accurately established in a post-failure investigation, which, takes place after the initiating effect has disappeared or been removed. A study involving the measurement of all conditions in a slope through the transition from a stable to an unstable state would therefore make a valuable contribution to the understanding of slope stability. Such an experiment would not only provide a comprehensive case history against which the various models of slope stability could be tested but could also reveal new information on the development of early creep strains within a slope and their relationship to the eventual mechanism of failure.

Clearly any such experiment, if it is to be conducted in a worthwhile manner, will involve a large amount of instrumentation, and consequently a high degree of cost. However if new and experimental instrumentation techniques and systems are used in the project, then the instrumentation performance can become an integral part of the experiment and increase the economic value of the project.

A slope study can only be as good as the field information obtained, especially with respect to the pore water pressures acting at the time of failure. Measurement of pore water pressures and measurement of displacements can only be as good as the instrumentation systems being used and the reading procedures and strategies operating throughout the study. It can hence be seen that there is a need for a greater understanding of;

a. fundamental instrument behaviour characteristics,

b. soil to instrument behaviour characteristics,

and c. the way in which the soil to instrument behaviour characteristics translate into useful data for slope monitoring purposes and how they then fit into the broader context of an instrumentation measuring system.

An instrumentation system will comprise a number of different instrumentation types, often of differing sensitivity, attempting to represent the whole slope status.

The Selborne Slope Study largely aimed to address c. of the above. The ability of an instrumentation system to "describe" a slope and the interrelationships between the varying instrument types and designs within the system was of interest. In practice despite the use of much proprietary instrumentation it was found that questions a. and b. also had to be addressed, providing some very interesting findings which are presented and discussed in Part II of this Thesis.

2. MECHANISMS AND APPROACHES TOWARDS ANALYSIS

Instability in a slope can be defined most simply as the movement of material downhill due to gravity. Nearly all forms of landsliding are observed, initially at least, to move along shear surfaces within the rock or soil. With the exception of slides such as mudflows, debris slides and flows, and avalanches (all relatively fast, unpredictable movements), during the initial stages of failure the material above a failure surface is observed to remain more or less intact (Bromhead 1992). Hence a simple diagram of the forces acting on the slope can be drawn (Figure 2-1).

In the simplest approach the weight, due to the mass of soil or rock above the slip surface, which is trying to move (the disturbing force) is resisted by a friction force along the slip surface (the available shear strength or available restoring force). For stability, the force available to resist motion must be greater than or equal to the disturbing force. At equilibrium the available restoring force is equivalent to the disturbing force. This provides the basis of the *limit equilibrium* methods of analysis, the most commonly used approach for analysis of slope stability problems. The method is based on balancing the disturbing force along a potential slip surface to the shear strength required for equilibrium. The ratio of available shear strength to that required for equilibrium being termed the Factor of Safety (Bromhead 1992).

$$\text{F.o.S} = \frac{\text{available shear strength}}{\text{mobilised shear strength}}$$

where the mobilised shear strength is that part of the available shear strength that is required to prevent sliding. Hence the mobilised shear strength is equal to the disturbing forces within the slope. From the above argument it can be seen that the slope will become unstable if the factor of safety falls below unity. At a factor of safety of unity a slope will be just static under the prevailing stress conditions (Sarma & Bhave 1974).

The friction force resisting movement (equivalent to the available shear strength) derives from the cohesion of the soil, c, and the internal friction of the soil, expressed by the angle of shearing resistance, ϕ. In reality most unstable slopes are subject to forces resulting from the presence of pore-water fluid and hence the relevant parameters for the cohesion and the angle of shearing resistance are the *effective stress* terms c' and ϕ'.

A problem arises in determining values for these parameters which characterize the condition of the slope when failure is about to take place. The shear strength of a soil is not a constant but can be dependent upon the amount of strain that has taken place and the rate at which strain is occurring. Other factors such as the stress history and the normal stress acting will also affect the shear strength of the soil. Figure 2-2 represents a typical stress-strain relationship for a brittle clay. With increasing strain the shear strength passes a peak and moves towards a residual value. The question therefore arises should the soil parameters at failure represent peak strength, residual strength, large strain strength or some other value. With the back analysis techniques usually applied, some parameter set that happens to fit the failure model to give a factor of safety of unity is often assumed to have been acting at failure, possibly with little consideration for what might actually have been happening in the soil at failure, and whether this parameter set is valid.

Tavenas and Leroueil (1981) point out that the stresses within a slope may change with time. The effective stresses within a cut slope will change (usually decrease) with time as the pore-pressures equilibrate after the initial stress relief which occurs during slope formation. In a natural slope the processes forming the slope, for example river erosion at the toe, are usually slow enough to allow complete recovery of depressed pore-pressures within the slope. At the same time, time dependent creep deformations within the soil will result in a progressive decrease in shear strength represented by the limit state curves in Figure 2-3. The geometry of a cutting and to a lesser extent the prevailing pore-pressure regime will determine the likely time effects of creep deformations. Figure 2-3 shows how in a steep slope or high cutting a point, O, will follow the stress path O to U1 to D1 to failure. It can be seen that only a small amount of pore pressure recovery is required for the point to reach the limit state line and hence failure. The effects of creep deformations in this case could rapidly reduce the length of U1 D1 so that a relatively small increase in pore pressure, say due to unusually high rainfall, could cause failure.

For a shallow slope the stress path to failure for a point O is O to U2 to D2 in Figure 2-3. Hence it can be seen that even allowing for a large amount of creep deformation the pore pressure change required to cause failure remains quite large.

Tavenas and Leroueil suggest that the effective stress parameters acting at failure can be represented by particular values of ϕ' and c'. ϕ' they claim can best be determined from CIU (consolidated undrained isotropic) tests carried out at in-situ stresses, and c' is best determined from the form of the limit state line surface of the intact clay, its value depending on the age of the cut and the intensity of the strain softening processes (this is represented by the critical state line in Figure 2-3). Tavenas and Leroueil also discuss the effects of fissuring in clay and conclude that it destroys all cohesion within the fissured region, but does not affect the value of ϕ' since no reorientation of particles takes place. Previous studies by Chandler and Skempton (1974) found that in brown London Clay and Lias Clay some cohesion is mobilized but that its value is very low.

Obviously not all points within a potential failure zone will reach the limit state curve at the same time. Once one particular point reaches this state, local failure takes place leading to strain softening. This point can then no longer support the applied shear stress and hence loads neighbouring points by shear stress transfer. If these neighbouring points are near to a limit state themselves then the transfer of stress could well force them to fail too. In this way a chain reaction can be set off leading to a *progressive failure*. So although not all of the points along the failure surface were simultaneously in a limiting state at any one time, slope failure has taken place, hence the appropriate strength parameters to be used in analysis must be somewhere between the limit state curve and the critical state line.

The most commonly used (limit equilibrium) analyses due to Bishop (1955) and Janbu (1973) make no attempt to reflect the stress path taken by a point to failure (Tavenas and Leroueil 1980) and, due to mathematical constraints, neither can they resolve correctly all the forces acting within a slope at failure. They do however still give as good an estimate of Factor of Safety as the more sophisticated methods of analysis such as those due to Morgenstern and Price (1967), Fredlund and Krahn (1977), and Spencer (1977). All these methods are limited by their basic assumptions as to the behaviour of the soil at failure.

Although these limit equilibrium methods are applicable to design, assuming care is taken in the selection of soil parameters, a better system for slope stability prediction is still needed. Methods due to Janbu, Bishop (1955) and others can be used effectively to design a "safe" slope but do not guarantee an accurate result in back analysing a slope that has already failed, since incorrect parameter assessment may introduce errors. They should therefore only be used with extreme care to analyse a failed slope or to predict future behaviour of a slope that is already failing.

A model of slope stability which examines the stresses at all points on a potential failure surface, then recalculates the effects of stress transfer from one point to the next if local failure has occurred probably holds the key to accurate and realistic slope stability analysis.

Some attempts have been made to incorporate this approach into Limit Equilibrium based analysis (eg Lo 1972, Law and Lumb 1977), but with the recent development of relatively cheap computing power the finite element method becomes a realistic option for everyday slope stability analyses incorporating considerations of progressive failure. Before new finite element techniques can be developed there is a requirement for more basic data as to the relationships between pore pressures and creep deformations early on in the development of a slip surface.

3. FIELD STUDIES OF SLOPES IN OVERCONSOLIDATED CLAYS

In practice, especially within the U.K., a large proportion of slopes cut in overconsolidated clays are for highway access. The explosion in road building within south east England in the last 30 years has forced many roads to be built on the Gault Clay, which had hitherto been avoided due to its known unsuitability as a road founding material; being subject to large seasonal swelling and shrinkage properties and corresponding changes in strength.

Studies of cuttings in Gault Clay and of embankment side slopes, which can be assumed to act in a similar way to cuttings in this particular material (Greenwood, Holt, Herrick 1985), have shown that the typical failure pattern is a shallow failure (usually less than 2.5m to slip surface) due to localised softening, occurring within 5-10 years from the end of construction. The mechanisms by which this happens are well understood and documented (Greenwood, Holt, Herrick 1985; Garret and Wale 1985; Bromhead 1986). Overconsolidated clay in the cut slope or embankment material from a borrow pit will have undergone stress relief during excavation leading to the creation of pore pressure suctions. These suctions increase with the depth of cutting or depth of excavation from the borrow pit. Surface runoff, due to rainfall or road drainage for example, will then be "sucked" into the upper layers of the slope thereby reducing the negative pore water pressures. A further addition of water due to rainfall or runoff will lead to increased positive pressures causing localised softening and eventually failure. This failure will almost always be shallow as the permeability of the clay decreases rapidly below about one metre (Anderson and Kneale 1980) and therefore the suctions below this level are maintained. The remedial measures that need to be taken usually involve the dig out of all slipped and softened material followed by replacement with a free draining granular fill. Slopes failing by this mechanism in Gault Clay are typically less than 10-15m in height and usually (where failure occurs) on a 1:2 (vertical: horizontal) or 1:3 side slope. These data largely reflect the fact that most (reported) failures in Gault Clay are concerned with highway cuttings and that economies in cutting design usually restrict cutting to less than ten metres depth and until very recently recommended designs were a side slope of 1:2 or 1:3 vertical: horizontal (TRRL RR 199).

Deeper failures in the Gault and failures in other overconsolidated clays tend to be more influenced by the geological properties of the material, particularly the weathering profile. Weathering affects, which diminish with depth, typically lead to a softening of the clay material and reduced discontinuity spacing. Many failure surfaces coincide with the boundary of weathered to unweathered material (Laguros et al 1982) with either a slip zone or slip surface just within the weaker weathered material. This is not surprising since the change in stiffness at the weathered to unweathered boundary will cause a concentration of stress and hence strain leading to strain softening and the eventual formation of failure planes. In the Gault the transition from weathered to unweathered material is typically at about 9.0m depth (Garret and Wale 1985), but is often overlooked during the design of cuttings. With this change in material and associated change in shear strength a standard circular slip surface analysis is no longer appropriate. A slip surface would not develop with a simple circular cross section but being influenced by the boundary of weaker to stronger material will form a slide with a translational component along the boundary of the stiffer material and rotational movements in the weaker material (Figure 3-1), a *compound* slide using the Skempton and Hutchinson (1969) classification. Barton (1984) suggests that a compound failure ought to be assumed in all slope stability analyses for overconsolidated clays until available data proves the slip to be of some other form.

The shear strength along a discontinuity (fissure or joint) is always far less than that of the intact material (Ward et al 1959, Skempton and Petley 1967, Williams and Jennings 1977 and works reviewed by Morgenstern 1977), hence a shear surface will follow a discontinuity where possible. It has also been shown that the shear strength along a bedding plane is lower than that of the intact soil (Freeman and Sutherland 1974, Loh and Holt 1974, Sugden et al 1977, Morgenstern 1977 and Lilly 1982). If shearing has previously taken place along this bedding plane then its shear strength will be even lower, possibly at a residual value.

Barton (1984) notes that the rearward surface of a compound slide will often conform to the fissure and joint pattern, whilst the lower (translational) part of the slip will have its attitude controlled by the dip of the bedding. One particular bedding plane will exhibit a lower mean shearing resistance than the surrounding bedding planes. This surface will control the elevation of the shear surface in deference to the stress distribution acting, this surface being termed the *preferred bedding plane*.

Why one bedding plane (or discontinuity) should be weaker than the surrounding ones is not clear. Many factors are involved and these have been reviewed by Morgenstern (1977) and Barton (1984). Basically the factors can be divided into two groups. The first group involves reduced shear strengths due to previous shearing along the bedding plane and the second group involves what can be termed material factors. Previous shearing can have been caused by landsliding, tectonic folding, valley rebound, glacial and periglacial activity and possibly even by non-uniform swelling. In all these cases the available shear strength along the bedding plane will be approaching a residual value.

The second group involving material factors includes processes such as seepage along bedding planes which may lead to differential softening affecting the material strength. This water may also cause chemical changes within the soil matrix close to the bedding plane. The stress/strain relationship for a bedding plane is also noted to differ from that of the intact clay (Ward et al 1959, Skempton and Petley 1967, Williams and Jennings 1977 and works reviewed by Morgenstern 1977). All of these factors will act to some degree, the complex stress history of many overconsolidated clays making it difficult to predict or assess which factor will dominate in determining the behaviour of any individual bedding plane.

4. SLOPES AND INSTRUMENTATION

To make studies of slopes quantitatively it is necessary to obtain data. These data can only be obtained by the use of soil instrumentation. Two basic measures are required, those of

- stress

and

- strain.

Stresses can be measured directly by the use of pressure cells but these are prone to errors and data interpretation is often difficult. Pressure cells are rarely used in slope stability work, their use being more concerned with embankments and structure to soil interface problems. Estimations of stresses in slopes are usually made by use of calculations (Terzaghi 1948) based on laboratory soil test data, slope geometry and field measures of pore water pressures. Often it is the pore water pressure acting at failure that is the unknown factor, and hence the state of stress at failure that is also unknown for backanalyses. Since it is often a rise in pore water pressure that triggers a slope movement it can be seen that procedures and strategies for monitoring pore water pressures are very important for the understanding and determination of slope behaviour. Measures of pore water pressure are made by use of piezometers which are described below and discussed in more detail in Chapter 14.2.

Strain or displacement measurements are important for defining the stress/strain characteristics of the slope. In engineering practice failure criteria may be specified as a fixed displacement or rate of displacement (eg Barton and McInnes 1988, Angelli et al 1988) and hence accurate measurement becomes important for defining serviceability. Absolute strains from a fixed datum or relative movement between two points, either in time or space can be measured. Displacement measuring devices include inclinometers, extensometers, tiltmeters and survey monuments.

The reader is referred to Dunnicliffe (1994) for a detailed description of the workings of individual instrument types. The following sections discuss various types of instrumentation in relation to slopes.

4.1 Inclinometers

Inclinometers give a horizontal displacement profile over a specified depth range for a single location in plan (Figure 4-1). Measurements are carried out by the use of an inclinometer torpedo run down either an aluminium or PVC access tube. The access tube is manufactured with an orthogonal set of grooves for locating the torpedo wheels. Measures are usually taken normal and parallel to the slope face.

Inclinometer torpedoes can be either uniaxial or biaxial. Uniaxial torpedoes measure displacements in one direction only, measurements being made by a servo-accelerometer. Biaxial torpedoes contain two sets of servo-accelerometers set orthogonal to one another, hence two faces can be read at once.

Inclinometer reading requires the torpedo to be passed down the access tube at least twice, the torpedo facing in the opposite direction during the second pass. The results of the two passes are then averaged to give a mean deviation, in a similar fashion to changing face when surveying using a theodolite. It can therefore be seen that to find the displacement profile in both the A-B and the C-D directions (Figure 4-2) four passes will be required when using an uniaxial torpedo, and two passes when using a biaxial torpedo.

4.1.1 Effect of inclinometer access tube stability/accuracy

Inclinometer casing is usually manufactured from either plastic or aluminium, although glass fibre is occasionally used. Aluminium is considered best in terms of deformation properties and lack of spiralling of the keyways, but has serious alkaline corrosion problems in some soils and cement/bentonite grouts and is therefore not recommended for boreholes likely to be in use for more than two years. In such cases a plastic (PVC) tubing should be used. The plastic tubing has a tendency to spiral during manufacture, the spiralling often being referred to as "groove roll". Green (1974) notes that in extreme cases spiralling can be up to 18 degrees in 24 metres of casing. Using the equations stated by Mikkelsen and Wilson (1983), derived from the mathematical formulae for rotation of an orthogonal coordinate system, the error due to this rotation can be evaluated as follows :

$$\sin A = \sin A' \cos \Delta - \sin B' \sin \Delta \quad .....(1)$$

and

$$\sin B = \sin A' \sin \Delta + \sin B' \cos \Delta \quad .....(2)$$

where sin A and sin B are the originally measured values of inclination using a biaxial torpedo. Δ is the shift in rotation due to groove roll and sin A' and sin B' the new readings (see Figure 4-3).

Using the extreme case noted by Green (1974) and assuming B equals zero, ie all true movement is in the downhill direction;

from (2)

$$\sin B' = - \sin A' \tan \Delta$$

substituting in (1) gives

$$\sin A' = \sin A \cos \Delta$$

assuming a severe case where A = 45°

$$\sin A - \sin A' = 0.035 \text{ or } 0.831\text{m in } 24\text{m}$$

and

$$\sin B - \sin B' = 0.219 \text{ or } 5m \text{ in } 24m \text{ !}$$

ISRM (1977) in their suggestions for installation of inclinometers and tiltmeters suggest that the groove roll be contained to within 1° per 3m of access tube and preferably not exceed 5° in the complete assembled length. Using these values (Δ = 5°) in the above formulae, again assuming B equals zero and taking a more realistic value of A = 10°

then ;

$$\sin A - \sin A' = 0.0007 \text{ equivalent to } 20mm \text{ in } 30m$$

and,

$$\sin B - \sin B' = 0.0302 \text{ or } 900mm \text{ in } 30m$$

which represents a considerable error especially on the B face (C-D face in U.K. notation). Since the C-D face is usually taken as the face parallel to the slope and hence actual displacements are usually far less than for the A-B face, these errors may represent a significant percentage of the observed reading on the C-D face. More importantly the error may not be readily noticed since the C-D reading will appear to increase in a believable manner as the A-B reading increases even if there is no true movement in the C-D plane.

The above calculations have taken a very much worst case scenario assuming the error to be at depth. Figure 4-4 represents a more realistic situation for a 30 metre deep access tube subject to a true cumulative deflection of 150mm downslope. With a groove roll of 0.75 degrees per metre the profiles shown in Figure 4-4 will be produced, it has been assumed that the inclinometer tube was aligned such that at its surface the A-B face was facing directly downslope. Figure 4-4 again shows how acute the errors on the C-D face can be, a true value of zero being distorted to almost 20mm.

If another scenario of movements is considered as shown by Figure 4-5, then the error on the A-B face is seen to be greater, i.e. the greater the depth of initial movement the greater the errors subtended at the surface.

If an inclinometer access tube is aligned in a true direction at its surface then groove roll will cause the apparent reading to be increasingly less than the true reading with increasing depth. The situation can be envisioned whereby a significant true reading will appear as an insignificant apparent reading and therefore be ignored as either instrument "fluctuation" or reading error. In trying to establish early signs of impending movements this is clearly a dangerous situation. For example a true reading of 5.0 mm at 22 metres depth for the access tube illustrated in Figure 4-4 will give an apparent deflection of only 3.8mm

These types of errors clearly only become significant in deeper inclinometer installations. Figure 4-6 shows the ratio of apparent to true reading with variation in angle of groove roll, Delta (Δ). For values of cumulative groove roll up to about 20 degrees the apparent reading is within reasonably good agreement with the true reading, beyond this the accuracy of the apparent reading falls rapidly. Assuming a groove roll of 0.75° per metre run of inclinometer tubing this represents a depth of only 26 metres before errors due to groove roll become significant. Even for an inclinometer within the 1° per 3m run ISRM guideline an installation of over 60 metres may be subject to significant errors. Inclinometer installations of this length are usually more applicable to mining and dam works than general civil engineering projects but the affects or measurements of groove roll in deep installations is rarely discussed within the literature.

4.1.2 Sensor azimuth shift, random and other inclinometric reading errors

If, along with the above errors, consideration is given to random errors, which are found to be higher in the measures taken on the servo-accelerometer orthogonal to the guide wheels (Devin et al 1988) and likely errors due to small drifts in the sensor azimuth (Mikkelsen and Wilson 1983) which are again more acute on the axis orthogonal to the guide wheels, then the whole practice of using a biaxial torpedo rather than an uniaxial torpedo comes into question. At current costs a biaxial torpedo is about £800 to £1000 more expensive than a uniaxial torpedo. The major time/cost consideration in the monitoring of an inclinometer is usually the time spent getting to and from site and getting the readout equipment and torpedo to the actual access tube. Once this has been done the time required to dip two more faces is relatively minimal (say an extra 30 minutes for a 25m borehole), which for the improved accuracy on the C-D face must be worthwhile. Also a uniaxial torpedo is more simply built and therefore less likely to go wrong and cheaper to recalibrate or repair. Table 4-1 gives a summary of likely inclinometer reading errors.

Error Source	Likely Value A/B	Likely Value C/D	Error over 30m A/B	Error over 30m C/D
Temp.	0.035%		±11mm	
Groove Roll (5°)	0.0027	0.0616	80mm	1.85m (metres)
Sensor ** Alignment	0.5°		23mm	185mm
Random **			8mm	12mm
Systematic**			6mm	131mm
TOTAL (excluding temp)			**117mm**	**328mm** excl groove roll

** Data from Mikkelson and Wilson, 1983

Table 4-1 Inclinometer Tube Errors

At present the subject of errors in inclinometer readings seems to have received little research attention, especially with reference to actual values of groove roll in inclinometers installed in the field by standard methods such as those detailed by ISRM (1977). There appears to be a need for a formal study to measure the amount of spiral on as many different inclinometers as possible, in as many different soil types as possible, installed by as many different contractors as possible. Say 20 samples should be enough to give a qualitative statement, more readings may be required to make a quantitative / statistical analysis.

Devin et al (1988) tested three field inclinometers for groove roll, one placed in shales, one placed in rock, and the third placed in non-homogeneous debris. Only the tube placed in rock came within the 1° per 3m groove roll tolerance suggested by ISRM (1977) and even this did not conform to the less than 5° total groove roll specification. Other studies of general inclinometer errors by Líbal et al (1987) suggest total errors (systematic plus random) of ±2mm within a 30m deep vertical borehole drilled in stable rock. No mention is made of tube spiralling.

The best way to combat or to allow for tube spiralling may be in the manufacture of the (plastic) inclinometer access tube. If manufactures were to be required to give the value of the groove roll for each batch of tubes produced an estimation of the likely errors in any inclinometer installation could be made. If it were found that these errors were significant within the context with which the installation were to be utilized then a full spiral survey of the completed inclinometer installation could be carried out.

The danger of reading only one face of an inclinometer can also be appreciated. If corrections for groove roll are to be made at some point in the future then the data for both axis will be required.

Tube/soil interaction is another important factor to be considered when evaluating the data from an inclinometer. During installation care should be taken to ensure that the grout (when set) has characteristics as close to those of the surrounding soil/rock as is possible (ISRM 1977). In practice it is difficult to control the strength of the grout unless the work is closely supervised. Care should be taken to ensure the grout is consistent in mix from one borehole to the next and between successive batches in a single borehole. Grout mix is also relevant to piezometers since either a poorly mixed grout or the junction between the grout and the borehole wall can provide a zone of increased permeability and hence locally altered pore pressure values.

The inclinometer access tube itself must be strong enough to withstand buckling and distortion so that the torpedo can still travel along its length, but not so strong as to substantially affect the magnitude of the reading of ground movements i.e. soil will be shearing around the tube rather than the tube moving within the soil mass. In practice the tube will be standard propriety aluminium or PVC. All the errors in reading an inclinometer are to some extent standard and will repeat in a similar manner each time the tube is read. Hence if change in deviation from a base reading is considered rather than actual deviation some information about what is actually happening in the ground can be deduced.

For example, again using the profile illustrated in Figure 4-4, errors in deviation over 30m are likely to be as follows :

	Face A-B	Face C-D
Temperature	Zero or very small if care is taken in operation.	
Groove Roll	3mm	19mm
Random Errors *	8mm	12m
TOTAL	**11mm**	**31mm**
% of true reading	7.3%	infinite **

* Data from Mikkelson and Wilson, 1983.

** Percentage error infinite as true C-D reading equal to zero.

Table 4-2 Errors in Inclinometer Tube Deviation.
(Table refers to 30m deep inclinometer illustrated by Figure 4-4)

Although the actual reading of the magnitude of displacement is unlikely to be accurate, the depth at which this movement is noticed is likely to be correct to within ± 5mm. Also it is possible to tell whether the movement is continuing at a constant rate, accelerating or even slowing down, and to make some comments about the rate of movement compared to other inclinometers installed in a similar manner and location.

Inclinometers provide a good system for locating depths of movement in unstable areas and assessing whether this movement is constant, accelerating or diminishing. If piezometers are also present and the inclinometers are read sufficiently regularly then some correlation can be made between the ground water regime and ground movements. Perhaps the greatest danger in interpreting and using inclinometers lies in the false impression given by the precision of the readout unit. Readings are usually given to 0.0001 metres and hence there is a tendency to believe that readings are accurate to within about 1mm, which is not necessarily the case in all soil types. Stress relief caused by the drilling process, especially in overconsolidated clays, will act on the access tube with crushing causing strains. These strains, which are usually small, will diminish with time as the tube in effect becomes part of the soil structure. The strains are likely to be greater in aluminium access tubing than stiffer plastic access tubes. Larger more prominent errors can be caused by wash out of material surrounding the access tube. This is especially likely in sands and gravels where ground water and vibration during drilling can cause voids. These voids are not always filled completely by grout placed around the access tube. The grout possibly being too viscous to fill the voids or even possibly diluted or washed away by groundwater. Voids leave the access tube unsupported and therefore susceptible to errors, these errors can actually be caused by the movement of the inclinometer torpedo itself in the unrestrained tube or there may be an underestimation of movements due to stresses not being transferred from the soil to the access tube.

4.2 Tiltmeters and Extensometers

Tiltmeters, which can be an inclinometer torpedo sensor acting at a fixed, specific depth rather than being mobile within the access tube, are prone to errors similar to those of inclinometers. In boreholes they are usually fixed in location by the use of spring loaded legs which are released once the instrument has been lowered to the required depth. They may also be fixed to say a rock surface or other accessible position by simple bolts, dowls or cement. The main error comes from not knowing the exact direction the tiltmeter is facing if it is installed in an inclinometer access tube subject to, say, groove-roll. Tiltmeters fixed to rock tend to be very accurate, the main errors being due to zero drift of the sensor (usually less than 6 seconds of arc).

Extensometers are another method of measuring horizontal displacements. They are usually used to measure movement at or near to the ground surface. Extensometers can be divided into two main types, rod and wire. Rod type extensometers are used over short distances to measure both lateral and vertical strains and may be either on the surface or placed in a borehole (vertical measurements only). Wire extensometers tend to be used over longer distances and are essentially surface instruments. As such they are prone to damage, but because they are on the surface this damage can be easily detected and repaired. Traditionally wire extensometers have used invar wire due to its low temperature strain coefficient, but recent studies by Bonnard and Steinman (1989) have used a twisted thread produced from kevlar. They found Kevlar to be far easier to install, much cheaper to produce and only slightly higher in temperature strain coefficient (2.0×10^{-6} compared to 1.2×10^{-6} per °c for invar). Unless the extensometer is being used to measure the relative displacement of two points, at least one point must be fixed in stable ground and the stability of this point checked regularly by surveying. Even if the extensometer is being used to measure relative displacements within a slope occasional surveying is still recommended.

4.3 Displacement Survey Monitoring

Survey monitoring gives a measure of both vertical and horizontal displacements. Great care must be taken in the set up of any survey monitoring system. The survey monuments must be in sufficient number and located such that they provide a true representation of slope movement i.e monuments should be placed at pre-planned locations, which may mean that they are not necessarily at points of easy access. Redundancy should also be built into any system to allow for possible loss of data points as a result of slope movements. Survey techniques can only give information as to the displacement of the slope surface. This information, however, provides a good method of establishing likely sub-surface movement regimes prior to the planning and installation of (expensive) sub-surface instrumentation. The information can also be used to predict the actual location of the slip surface, Carter and Bentley (1985) present a geographical method which uses surface movements to predict the depth to a slip surface. They show the method to be able to predict confidently the depth to a slip surface to within 2% of the distance between ground stations.

Since survey monuments are basically surface monitoring points they may be subject to swelling (in cohesive soils). Swelling due to stress relief during slope formation or due to seasonal moisture content changes in the top metre or so of the soil may mask any actual movements, possibly invalidating the elevation survey data. Reference monuments should of course be founded in such a way that they are unaffected by swelling type movements.

(Long term) Survey monitoring should not be thought of as a cheap option. Surveying is expensive in terms of manpower, not only for the initial monument installation but for each monitoring survey carried out. Data must then be downloaded, processed and checked for errors. Also all reference points (temporary bench marks) must be checked occasionally to prove that they have not moved.

Another point to remember with survey systems is that often it is early strains that are of interest to the Engineer. These early movements will be of a similar magnitude to the likely survey errors, i.e a few millimetres. For this reason every effort must be made to reduce survey errors. It is recommended that survey work be carried out with the measurement of angles only wherever possible. Good basic precision surveying techniques should be used at all times with the instrument being read on both faces. Precise levelling should be used for establishing elevations. Total station EDM's, although being extremely convenient, are not recommended for monitoring of sites where movements of less than 20mm are to be detected. EDM tacheometry is of course still recommended for basic topographical surveying to establish the extent and profile of a site. Further discussion regarding the magnitude of survey errors and early strains is given in Chapter 13.

Survey monitoring lends itself best to extensive sites where large movements are expected. It provides good data for planning future sub-surface works and also for interpolating mechanisms of movement between sub-surface monitoring points where mechanisms can be more precisely identified.

4.4 Piezometers

Piezometers are usually of a simpler design than inclinometers and hence less prone to errors. Pore-water pressures, which are typically expressed as a head of water (mH_2O), are usually read to an accuracy of 0.1m and hence any error will have to be quite large to significantly affect the readings. The major types of piezometers in use today in order of decreasing simplicity and increasing cost are;

- open standpipe
- pneumatic
- twin tube hydraulic
- vibrating wire

Standpipes and standpipe-piezometers (Figure 4-7) are cheap, easy to read, reliable and need no calibration. Wherever possible, therefore, standpipe piezometers are the preferred method of reading pore-water pressures but their use is hindered by a number of disadvantages. Their main disadvantage is their long hydrostatic time lag (response time) in low permeability soils. This slow response time is due to the relatively large water volume change required to produce a change in reading. Standpipes also interfere with construction activities since the top always has to be accessible. Readings are carried out by using either an electrical dip meter which signals when it comes into contact with water or by means of a bubbler system. Other problems associated with standpipes and standpipe-piezometers are filter clogging at the tip and freezing of water within the standpipe and also that they are unsuitable for automated logging.

Twin tube hydraulic piezometers (Figure 4-8) rely on a closed hydraulic system responding to pressure changes in the piezometer tip caused by changes in the pore-water pressure surrounding the tip. These pressure changes are then measured on a Bourdon pressure gauge. In order to ensure a rapid response time and accurate results the system must be free of air, and hence regular de-airing of the system is required. Twin tube hydraulic systems have a number of disadvantages ;

- the Bourdon gauges have to be as near to the piezometer as possible (Penman (1961) suggests less than about 300m to give a reasonable response time) and ideally below the level of the piezometer tip,

- allowance has to be made for the difference in elevation of the tip and the gauge,

- regular de-airing of the system is required, requiring on site personnel,

- the system is not easily automated for remote logging.

The system does have the advantage, when used properly, of being very accurate and able to read reliably sub-atmospheric ("negative") pore-water pressures. Also filter tips that have become only partially saturated after installation can be easily resaturated, though of course this will temporarily affect the pore-water pressures locally around the piezometer tip.

Pneumatic piezometers and vibrating wire piezometers use a similar tip design. Inside the tip there is a small diaphragm which moves back as the pore-water pressure increases and vice-versa (Figure 4-9).

Pneumatic systems use a flow of gas, either nitrogen or dried air, to equalise the line pressure to that acting on the back of the diaphragm within the piezometer tip. Once the pressure has been equalised a valve opens and air escapes along a return line (see Figure 4-10). The flow of air to the diaphragm is then halted and the air pressure is measured as the return valve once again closes at which point the air pressure is equal to the pore pressure in the tip. Care should be taken not to pressurise the system too quickly by using a high rate of air flow, since this will cause a high volume change in the piezometer tip leading to the expulsion of water from the tip and consequently an underestimation of the pore-water pressure. When using pneumatic piezometers no allowance is necessary for the difference in elevation of the piezometer tip and the readout unit, and the pneumatic piezometer lines can be up to about 1000 metres in length without any major loss of accuracy.

Vibrating wire piezometers also use a diaphragm type mechanism which deforms under the applied load (pore-water pressure). A fine wire is attached to the back of the diaphragm, the tension in the wire altering as the diaphragm moves. When the instrument is read an electrical magnet "plucks" the wire and the frequency of the resulting vibration is measured by a transducer. The frequency of the vibration will depend on the tension in the wire which in turn is dependent on the pressure acting on the diaphragm, that is the pore-water pressure. Frequency of vibration can easily be converted into engineering units, but each instrument requires an individual calibration. The cables from vibrating wire piezometers can quite easily be installed over a couple of kilometres and the electrical output lends itself to automated logging procedures.

All diaphragm piezometers suffer from the problem that the porous filter cannot be resaturated once the piezometer has been installed. This means that at pore-water pressures close to atmospheric pressure, air may enter the filter either causing a significant delay in the response time or affecting the reliability of the measurement altogether. The problem may be exacerbated if high air-entry porous filters are being used.

High air-entry filters were originally designed for use with twin tube hydraulic piezometers. They are used to exclude air and vapour from the piezometer so that the correct hydrostatic pressure is measured, the high air entry meaning high air pressures are required to allow air to pass into the filter (and consequently high pressures are needed inside the filter for the air to pass out again!). Any air that does enter a hydraulic system is removed during the standard de-airing procedures. In diaphragm type piezometers it is not possible to expel this air except prior to installation, so unless the piezometer is being used in soil that is constantly saturated, some air will inevitably pass into the filter (once partly saturated the air-entry value is reduced). Use of an high-air entry filter will then mean that this air is not easily displaced from the filter as the soil becomes resaturated. Of course if the filter is originally of sufficiently high air-entry value and care is taken during installation to keep the tip saturated then the piezometer should give accurate readings, but low air-entry filters may still be the most appropriate filters for diaphragm type piezometers especially if high excess pore pressures are the main design criterion as for example in embankment dams.

In practice most problems with the use of piezometers do not arise from the piezometer itself but from damage to cables and tubing due to construction activities or vandalism. In embankment dams, for example, settlement between the core and the shells can be quite large, up to 500mm for a 150m high dam (Mikkelsen and Wilson 1983). Obviously if piezometer cables cross this zone they are likely to be sheared. There are also problems due to plant damaging equipment or unmarked cables being excavated by mistake on a congested site. A determined vandal will damage equipment regardless of how well it is protected. The best solution to this problem is camouflage, but this conflicts with the solution to construction related damage, which is to clearly mark the position of instruments and their cables. Davies et al (1989) describe how damage due to construction activities was reduced to almost zero on the temporary works for the A55 Conwy crossing, by writing into the contract specification that the contractor would be responsible for the replacement costs of any instrumentation damaged by on-site activities.

4.4.1 Piezometer accuracy and reading Errors

Before commenting on this section in relation to piezometers it is necessary to draw a distinction between what is termed an error and what is termed instrument accuracy.

Accuracy in this context is to be taken as the errors in reading caused by mechanical and material constants within the instrument. These inevitably lead to the measured reading value being different from that operating in the soil. Tunbridge and Oien (1987) in their discussion of vibrating wire instruments in geomechanics put accuracy down to three main factors as follows ;

1. Repeatability of reading,
2. Conformity - the fit of calibration formulae to true data,

and 3. Hysteresis - due to loss of energy in mechanical components

For a vibrating wire instrument Tunbridge and Oien estimate accuracy as about 8Hz or 0.5% span (usual measuring range for a vibrating wire piezometer being 800Hz to 2500Hz). For a true measure of 20 mH_2O this represents a measuring error of only 0.01 mH_2O, inconsequential when reading to an accuracy of 0.1 mH_2O.

Of more significance to the measured value are instrument and reading errors. The term error being taken to include all factors leading to a mis-reading of the instrument that are not purely a function of its manufacture, errors occur *after* the instrument leaves the factory. Where as to a certain degree accuracy is fixed, errors are a variable and as such can be minimised by good reading practice and correct instrument installation techniques.

4.5 Effect of Instrument Installation

Soil instrumentation and its relationship to what is actually happening within the soil is a valuable area of research in its own right and a large part of the Selborne Slope Study experiment was concerned with examining the performance of the instrumentation systems used.

Purely by virtue of having been installed an instrument is going to have altered, perhaps fundamentally, the characteristics and behaviour of the soil it is intended to monitor. The effects of this disturbance will diminish with time as the instrument becomes, in effect, part of the soil mass. This property is well demonstrated with the installation of piezometers, where a time period, dependent upon the permeability of the soil, is required before readings stabilise to the true value. The time for equalisation to the true reading depends on the properties of the soil and on the properties of the instrument itself (see Chapter 14.3).

There is also always a question of what is actually being monitored: is it the changes within the soil which the instrument is intended to monitor or is it just the stress/strain/temperature characteristics of the instrument itself which are being recorded? Often, it is a combination of both. Bonnard and Steinmann (1989) illustrate this problem with their extensometer's inductive displacement transducer which was found to give false displacement readings with changes in temperature. The transducer was subsequently changed for a potentiometer equipped transducer which showed no temperature effects. In practice it is often the case that the instrumentation indicates that something is happening, but it is left to the engineer's own judgement and experience to decide exactly what.

4.6 Effect of Reading Strategies

Moller et al (1989) in their development of a computerised monitoring system for landslides based upon instrument readings state that "the limitation of the system today is the lack of knowledge of what actually happens in the soil when a slide is just about to start".

At the onset of sliding, when strains will be very small, it is often difficult to distinguish between real movement indicated by an instrument and apparent movement due to natural fluctuations of the instrument signal. By plotting the results continuously against time some of the natural fluctuations can be eliminated and the trend of the results becomes more noticeable.

The siting of an instrument becomes important when small strains are to be considered. Extensometers and inclinometers are often situated towards the crest of a slope suspected of being unstable. Skempton and La Rochelle (1965), De Beer (1969) and Garret and Wale (1989) all point out that large deformations and consequently the smaller, earlier, creep deformations will first occur near the toe of a landslide i.e. in the zone of high stress concentration. Hence if the aim is to obtain early warning of incipient failure, measurable displacements will first be noticed near the toe and hence instruments for monitoring slope stability should be placed in this zone.

The reading sequence followed and the time lapse between readings may also create apparent changes in instrument readings. The obvious case is the reading of piezometers placed within a tidal zone. Since it is impossible for all instruments to be read at one time, and hence the same state of tide, or at the same state of tide each day, some allowance must be made for tidal affects. This allowance must be as accurate as the piezometer itself so as not to mask any actual change in the ground water regime. Davies et al (1989) discuss these problems with reference to the temporary works for the A55 Conwy crossing in North Wales. In this instance an in-house developed computer programme was used to predict tidal affects on each individual instrument.

Biannual reading of inclinometers can lead to false information on ground movements. For example if readings were taken in September and the following April these could well suggest that movements had taken place throughout the winter when, in fact, movement is actually concentrated within a very short length of time in the spring. This is especially true in mountainous regions where topographical surveys or inclinometer readings are rarely taken in winter due to snowfall. In these areas snow melt is often the primary cause of landsliding. Bonnard and Steinmann (1989) illustrate this with their instrument for the continuous measurement of unstable slopes.

They show that actual movements in the spring can be correlated to temperature and hence snowmelt. This information, if accurately assessed, could be used to programme construction activities to avoid, say, a six week period in the spring when landsliding is most likely to occur.

Inclinometers have occasionally been observed to "move uphill" in a manner suggesting dubious readings but, these movements are usually small and it is suspected that they are due to the swelling characteristics of the clay (Krauter 1988). These movements usually only occur in the top few meters of the inclinometer tube. Other initially inexplicable readings, especially if they are over the full length of the access tube, can be due to the inclinometer not having had its base fixed in a static stratum. ISRM guidelines for using inclinometers (ISRM 1977) suggest that if any movement is noted in the lower 3.0m of an inclinometer tube, its base should not be taken as fixed and hence the top of the inclinometer tube should be fixed by surveying (to an accuracy of 1mm) each time the inclinometer is read. It is important to carry out occasional surveying of inclinometer tops as a matter of routine to prove that the base of the access tube is fixed (see Figure 4-11).

Extensometers and tiltmeters both lend themselves to automated logging procedures, and hence to the many automated warning systems of incipient slope failure currently being developed (Barton and McInnes 1989; Bonnard and Steinman, 1989; Grabowski, 1988; Möller et al, 1989), especially where these systems are being used to monitor possible rockslides. In these type of systems care must be taken to ensure that the monitoring points are representative or, if it is a warning device, represent the worst case of the slope as a whole.

The question of whether the instrumentation is representative of the whole problem is an important one. In embankment dams, for example, instrumentation is usually installed in sections. All the cables from a given section are usually placed in a single trench and the trench backfilled by hand. The backfill in the trench may settle in a different manner to the surrounding material in the embankment compacted by machinery. Hence settlement gauges etc may monitor the performance of the trench backfill rather than the performance of the actual embankment.

Instrumentation systems must be designed such that they provide data that are representative of the problem as a whole. They must also provide enough data to be able to model the problem accurately for analysis. All of this, of course, has to be done within an often strict budget.

PART 2 - THE SELBORNE SLOPE STUDY.

GENERAL INTRODUCTION TO PART 2

The Selborne Slope Study was a government sponsored research project carried out by Southampton University in conjunction with Kingston and Warwick Universities to bring to failure, by the process of pore pressure recharge, a fully instrumented nine metre high slope cut in overconsolidated Gault Clay. The research site formed part of an operational clay pit near to the village of Selborne, Hampshire (Figure 5-1). Clay from the pit was used for the production of handmade bricks and tiles at the adjacent brickworks. The clay pit and the brickworks were both owned by the Selborne Brick and Tile Company, who generously allocated a portion of the pit for use by the research team for two years.

The slope was subject to a high degree of instrumentation. This instrumentation was monitored for 16 months prior to starting the process of pore pressure recharge. Pore pressure recharge was carried out by feeding water to 20 recharge wells which had sealed response zones at typically 6 to 12 metres depth. Following slope collapse two trenches were excavated perpendicular to the failure to locate and identify the slip surface. The slip surface was traced and surveyed in each trench. Large diameter (300mm) and block samples were taken through the slip surface for laboratory analysis at Warwick University.

The author was Research Assistant for the project and as such was present on site from slope formation and instrument installation through to slope collapse and the post failure investigations. Following site decommissioning a further year was spent at Southampton University carrying out the initial processing of the site data. The following Chapters present a detailed case history of the project tracing the geometry, nature and development of the failure. Information detailing the background to the project, site geology, pore pressure recharge system and instrumentation systems is presented. The performance of the instrumentation and its ability to describe the failure are discussed. The processes and mechanisms considered to have been operating as the slope progressed from a stable to an unstable state and eventually to collapse are described and discussed.

5. INTRODUCTION TO STUDY

The need for more experimental evidence obtained from a controlled slope failure, to further existing knowledge of the processes operating in a slope at failure, has been well documented (Tavenas and Leroueil 1981, Bonnard and Steinman 1989). With this aim in mind attempts were made to locate a suitable site. Such a site was located within the Selborne Brick and Tile clay pit at Selborne, Hampshire (Figure 5-1). Funds provided by the Science and Engineering Research Council (S.E.R.C.) allowed a feasibility study of a "nib" on the west face of the clay pit to be carried out. This work was carried out under the supervision of M.R. Cooper in 1984/5 (S.E.R.C. Research Contract GR/D02799). The feasibility study showed that the site had good potential and therefore an application was made to S.E.R.C. for funds to construct and bring to failure a fully instrumented 9.0m high clay slope (Plate 5-1).

The money was awarded in June 1987 in the form of three grants :-

Grant 1: Southampton University - Overall coordination of project, monitoring and collection of instrumentation data, publication and dissemination of results. This work was under the supervision of M.R. Cooper.

Grant 2: Kingston Polytechnic - Drilling for the installation of instrumentation and two continuously bored rotary cored samples. This work was under the supervision of Dr. E.N. Bromhead.

Grant 3: Warwick University - Laboratory testing of rotary cores and sampling and testing of slip surface samples. This work was under the supervision of Dr. D.J. Petley.

Southampton University's grant was by far the biggest since it included the initial purchase of the instrumentation and the funding for one Research Assistant and one Site Technician. The Research Assistant was to be employed for the duration of the project and the Technician for the anticipated duration of site activity. Although the three grants were to be administered separately by each grant holder, the S.E.R.C. was keen to see the three collaborators working closely together and consulting each other at regular points throughout the project. To this end and to help S.E.R.C. monitor the relatively new concept of collaborative research, a Steering Committee was proposed in the grant bid and expanded by S.E.R.C. to include the three grant holders, two S.E.R.C. coordinators, an S.E.R.C. appointed chairman and the site staff. The S.E.R.C. coordinators then reported back to the S.E.R.C. Geotechnics Committee. This Steering Committee aimed to meet either every three months or at significant points during the site works.

After some delay due to the later than planned for receipt of moneys from S.E.R.C.

site work began in mid August 1987. Slope forming and instrument installation were substantially complete by the end of September 1987, just before the Great Storm of 16th October after which time site access by vehicle was virtually impossible until the following April. In this first phase of site work 12 inclinometer access tubes, 2 string inclinometer access tubes, 23 pneumatic and 28 vibrating wire piezometers, and 20 recharge well / slip indicators (which also acted as standpipe piezometers) were installed. Instrumentation and instrumentation installation details are given in Chapters 8 and 9. A concrete hardstanding for the gauge house was also laid at this time and a small caravan brought from Kingston Polytechnic to act as the gauge house. Work notably not carried out at this time included the cutting of the side shear breaks, left uncut to aid stability through the first winter, and installation of surface wire extensometers since some construction details of their design had not yet been finalised by the manufacturers.

In a second phase of site work in the summer of 1988 seven more pneumatic and two more vibrating wire piezometers were installed. These were installed at the recommendation of the steering committee, who were worried about the lack of instrumentation cover at depth within the proposed recharge zone. Also during the summer of 1988 the wire extensometer anchor frames and anchors were installed, the benches cosmetically smartened and levelled off with sand (see Plate 5-1) and finally in late August the side shear breaks cut. The whole slope face was then covered with a protective layer of plastic sheeting as it had been the previous winter.

Connection of the instrument cables onto terminal boards inside the gauge house and then testing of the instruments took place throughout the autumn and winter of 1987/88. The weather conditions plus the presence of two inexperienced site staff and problems with condensation in the pneumatic piezometer air lines initially made progress slow, but the first full set of "confident" piezometer readings was taken on the 13th January 1988. Base readings on the 12 inclinometer access tubes were taken as the average of three successive rounds of readings in November 1987. The two String Inclinometers were installed on the 11th January 1988 and connected up to reading equipment inside the gauge house a few days later. Details of pre and post installation testing and calibration procedures are given in Chapters 8 and 14.

An automated logging system for the vibrating wire piezometers, string inclinometers and wire extensometers was installed in June and July 1988. This system was initially capable of reading and storing data taken at one minute intervals for each instrument for in excess of 24 hours. The system was also capable of running on a threshold basis whereby instrument readings were only stored if they had changed by a pre-determined value from the last set of stored readings, consequently saving a vast amount of logger memory. Later on an alarm system was added that could be triggered at a pre-determined threshold. Once triggered the alarm system could set off a number of external devices and also reset the logger to a new pre-set logging strategy. The external devices used at Selborne were an on-site audible alarm and flood lighting system plus a telephone autodial system that contacted chosen telephone numbers with a message that a certain alarm level had been reached. Despite numerous teething problems with this new and, at the time, still largely experimental system everything was up and running by Christmas 1988. At this point all site activity began to focus on the construction and monitoring of the Recharge System and the eventual build up to failure.

The recharge system was initially constructed from 1.5" and 1.25" plastic waste pipe with water fed from small central heating tank reservoirs, one tank for each row of recharge wells. These small tanks were in turn fed from a 40 gallon tank with a permanent mains connection placed on top of the gauge house (Plate 5-2). The 40 gallon tank was intended to act as a large reservoir should the mains connection ever fail, for example due to freezing (which happened only twice due to the exceptionally mild winter of 1988/89) or due to other site activities. The small tanks acted not only as secondary reservoirs for each row of recharge but also as a convenient method of controlling the head acting on each row of wells since the head could be raised simply by raising the level of the tank. This also had the advantage of allowing the head acting on each row of recharge wells to be raised independently. The sequence of recharge is fully explained later (see Section 11) but basically involved introducing recharge at some distance behind the crest of the slope and slowly bringing it forward by introducing extra lines of recharge until the whole of the area immediately behind the crest and the top 40% of the slope were being recharged. Once this point had been reached the head acting on the whole system was simply increased until the pore pressures in the slope were raised sufficiently to induce failure. Failure actually occurred in the week preceding the 16th July 1989 with spectacular movements taking place on the 16th. At each stage of the head being raised during recharge the system was left until a steady state had been noticed in the piezometer readings. A crisis point was reached with the system at about 5m head when the joints on the plastic waste pipe began to leak under the applied pressure. This meant that the pressure was not being maintained in the recharge wells and hence it became necessary to rebuild the system using 0.75" reinforced garden hose, which suffered from the problem of airlocks. The airlock problem was solved by setting up a daily schedule of bleeding each recharge well firstly to clear any air at the top of the well and secondly, to check that the well was still running.

Although failure was beginning to occur at about 5m head, due to time constraints, the site owner wanted repossession of the land by the end of August 1989, the recharge was increased to 7m head on the 13th July 1989. By this time it was impossible to dip completely any of the inclinometer access tubes on the slope face and hence it was decided to induce failure and collapse as soon as possible.

In the two weeks following failure two inspection trenches were cut perpendicular to the slope face (Plate 5-3). The slip surface was identified and surveyed in each trench (Plates 5-4 and 5-5) and large diameter (150mm and 300mm) samples taken through the slip surface for laboratory testing at Warwick University. Also a series of 150mm diameter samples of undisturbed material were taken from the outside of the side shear break for comparison with the rotary cored samples of undisturbed material.

A danger throughout the project, from the first winter, had been that local softening could cause a shallow slump failure of the top half metre or so of the marginally stable 1:2 slope face. For this reason following a small slump of the top bench the slope was covered with agricultural polythene sheeting from October 1987 onwards. The polythene was removed for a short time during the fieldwork season of summer 1988 to allow for final bank trimming and digging of the side shear breaks. The concern with respect to shallow slumping was proved valid towards the end of the project when a small scale slump took place along the top two benches (see Section 16).

The pre-existing topography of the site prior to slope formation is described in the feasibility study report (Cooper 1996), and expressed diagrammatically as Figure 5-2.

6. GEOLOGY OF THE GAULT

Selborne lies within the Wealden District (Figure 6-1). The Wealden District comprises a large anticlinorium composed of numerous smaller asymmetrical east-west folds of which the steeper limb dips to the north. The district covers the South East of England from the Thames Estuary to the South Downs. The Selborne research site is situated on an outcrop of the Gault Clay approximately 4km east of Selborne (National Grid Reference SU 768342).

Most factors affecting the engineering properties of the Gault clay are due to its geological history. Relevant strata in the Selborne area date from Cretaceous times. Generally in the Weald, Chalk overlies Upper Greensand which in turn overlies the Gault with the Folkestone Beds of the Lower Greensand below this (Figure 6-2). The basal bed of the Chalk is often a chalk marl of almost clayey consistency and where the Upper Greensand is not present gives rise to a gentle transition of ground slope between the Chalk escarpment and the Gault outcrop. In the western part of the Weald, however, Upper Greensand is present above the Gault resulting in slopes along the outcrop which are fairly steep. These slopes are especially noticeable around the Selborne area where the wooded ones are referred to locally as *The Hangers*. Landslides have been recorded along the Hangers since the 18th century (G. White 1883).

The Gault was deposited during the Middle and Upper Albian period. It increases in thickness westwards across the Weald from about 50 metres at Folkestone, Kent to over 100 metres towards the south and west Sussex areas. During it's period of deposition conditions were not uniform, leading to the formation of several lithologically different beds. The Gault and Upper Greensand are, in fact, just lithological variants of the same sequence, Upper Greensand having been deposited in shallow, current-swept conditions close to the shoreline whilst the Gault was deposited, again in relatively shallow water, in quieter conditions farther from the source of the sediments.

Palaeontologically the Gault can be divided into a number of different zones. Up to thirteen beds have been logged at Folkestone but these have not generally been recognised elsewhere in the region, although however, the Gault can be split into at least the Upper Gault and Lower Gault throughout the Weald.

Lower Gault is usually dark grey in colour and increases in thickness from about 10m at Folkestone, westwards to 47m at Upper Beeding, Sussex. The interface with the Upper Gault which has a paler grey colour is marked by a band of phosphatic nodules.

Upper Gault contains several layers of phosphatic nodules and in some areas a band of glauconitic clay occurs at the top of the sequence. In the western Weald during the period of deposition of the Upper Gault (the Upper Albian Stage) the material deposited was of a silt size. This material forms the Upper Greensand which represents the Upper Gault in this area. The distribution of Upper Gault clay and Upper Greensand is illustrated diagrammatically by Figure 6-3.

Gault clay can be also be classified by its degree of weathering or decomposition, and can be presented in the following forms :-

- Solifluction deposits
- Cryoturbated (frost-shattered) Gault
- Weathered Gault

And
- Unweathered Gault

Unweathered Gault is characterized by fissure bounded, intact lump sizes which are rarely less than 100mm sides. The mean size of intact lumps tends to decrease towards the ground surface, suggesting stress relief as a mechanism of fissure formation. Fookes and Denness (1969) have established that bedding is the dominant factor influencing the fissure pattern in the Gault. Lithology, stress relief and tectonism were found to have only minor affects on the actual fissure pattern.

Weathered Gault is usually found within the top nine metres or so of the formation. It is rarely found at all where the Gault underlies the Chalk. Weathered Gault is usually less stiff than the underlying unweathered material and can be differentiated structurally by the scale of fissuring, which is likely to be variable; average fissure lump sizes tending to be between about 50mm and 100mm sides. The alteration in structure of weathered Gault is mainly due to groundwater percolation, the lower horizon marking the extent of chemical weathering within the formation. Weathered Gault changes from a grey to a brown colour as the degree of weathering increases.

Frost shattered or Cryoturbated Gault occurs in a zone which has been affected by severe mechanical weathering and also possibly some degree of chemical weathering under periglacial conditions leading to advanced disintegration. Fissuring is usually at less than 25mm spacing. This type of material does not occur everywhere, it is thickest on hilltops and absent in valleys and rarely extends below six metres depth.

Solifluction deposits are found across almost all the Gault outcrop. They consist of material derived from Chalk and Upper Greensand as well as Gault Clay. The latter has suffered almost complete disintegration, having been moved by solifluction processes (solifluction after Anderson 1900). This zone is usually less than 1.5 metres thick and may be absent in valleys.

In the Selborne area the Upper Gault is not present, Upper Greensand rests directly on the Lower Gault. The research site itself was located approximately 4km east of

Selborne at National Grid Reference SU768342 on an outcrop of Lower Gault. The Lower Gault is confirmed by Owen (1963) and also by the fossil record . A number of Montoniceras, a Cretaceous ammonite, of up to 100mm diameter were found during the site works and identified by a palaeontologist present on site during the rotary drilling works.

6.1 Geotechnical Properties of the Gault Clay

At the Selborne research site the Gault clay was only 17.8 metres deep. With the Chalk estimated to have exceeded 500 metres at its peak this represents a considerable pre-consolidation stress, about 1.5 to 3 times that of the London Clay (Bishop et al 1969, Skempton and Hutchinson 1969)

During the site works samples from three rotary cored drillholes were taken to Warwick University for detailed laboratory analysis. Examination of the cores lead to the establishment of four zones :-

a) a soliflucted layer approximately 3 metres thick,

b) an upper weathered zone of Gault Clay approximately 2-3 metres thick,

c) a lower weathered zone of Gault Clay about 6 metres thick,

and

d) a zone of unweathered Gault Clay.

This information together with results obtained from the feasibility study and the post-failure investigations lead to the establishment of the general centreline soil profile shown by Figure 6-4.

The water content, liquid limit and plastic limit profiles as determined from tests on selected samples are presented as Figure 6-5.

ZONE	PEAK ϕ_p	c_p	RESIDUAL ϕ_r	c_r
a) Solifluct.	21°	5 kN/m^2	13°	0
b) Upper Weath.	22°-24°	10kN/m^2	13°	0
c) Lower Weath.	22°-25°	15kN/m^2	14°	0
d) Un weath.	23°-26°	20-25kN/m^2	15°	0

Table 6-1. Laboratory Shear Strength Parameters.

Much of the laboratory testing programme concentrated on the determination of peak and residual shear strength parameters. Both 75mm triaxial and 60mm square shear box tests were used. The results of these tests are given in Table 6-1, Appendix M gives further details of the testing. Ring shear tests were also performed on selected samples and the residual shear strength values found to be in close agreement with the values given in Table 6-1. Tests to investigate the effect of shearing rate found that a change in shearing rate from 0.0001 mm/min to 0.01 mm/min resulted in an increase of ϕ_r of about 2%.

7. EXPERIMENTAL DESIGN

The Selborne Slope Study had to be designed on a "one chance only basis". In this one chance as much data as possible had to be collected about not only the intended failure event, but also about pre-failure strains and ground water conditions. The One Chance Only philosophy ran throughout the design process, hence the logging systems were split into two independent systems with both hard (paper output) and soft (floppy disk) copies of data. Another aim of the project had to be to collect general data about the soil properties of the site, hence Warwick University's grant under Dr. Petley (S.E.R.C. Final Report No.GR/E/34094). In order to achieve these aims the third grant was awarded to Kingston Polytechnic under the supervision of Dr. Bromhead to install the instrumentation and take samples from three rotary cored drill holes for delivery to Warwick University (S.E.R.C. Final Report No.GR/E/33981).

7.1 Instrumentation Layout Design

The instrumentation system was designed to three broad constraints ;

1. COST - It was known that the bid for funds was unlikely to succeed if the total cost of the project exceeded £200,000.

2. COVERAGE of failure event

and 3. MONITORING of progress towards failure event i.e control of the recharge system.

7.1.1 Piezometers

The layout philosophy for piezometers was as follows. Some placed deep near the back of the slope to monitor the recharge, some moderately deep and some shallow to give an idea of the general ground water regime, with a concentration around the expected slip zone. The more expensive, quicker reacting, logged instruments (vibrating wire piezometers) were to be placed nearer to the expected slip zone and the slower reacting manually read instruments (pneumatic piezometers) in the less sensitive areas, that is, at the base of the slope and behind the slope (Figure 7-1).

It was also desirable to have some overlap of the two piezometer types so that they could be cross calibrated and their performances compared. The final piezometer layout design is shown by Figure 7-2.

The only significant failure in the planning of the pore pressure monitoring scheme was in the zone at the base of the slope, see Figure 7-1, where pneumatic piezometers were installed. Stress relief during slope formation caused the pore pressures in this area to become negative (negative in this case meaning below normal atmospheric pressure), a condition under which pneumatic piezometers usually become inoperative. Hindsight would have suggested installation of some vibrating wire piezometers or other system capable of operating at below atmospheric pressure in this zone. Due to cost and inaccessibility for machinery at the bottom of the slope (which throughout the winter months was under 0.5 metre of standing water), it was impossible to install any further instrumentation once it had been realised that pore pressures were falling to below atmospheric pressure. A system was therefore developed to read "negative" pneumatics, the full operating details of this system are described in Section 14.5.

Data collection and analysis during the first winter also identified a small "gap" in the piezometer coverage at depth around the base of the recharge wells - a zone that was obviously going to become important for early recharge monitoring. To rectify this nine more piezometers were installed during a second season of fieldwork in the summer of 1988.

7.1.2 Inclinometer access tubes

The coverage of displacement type instruments was controlled almost entirely by cost constraints. Inclinometer access tubes are relatively cheap and were therefore installed at 3m centres in section spread out in plan to cover approximately three lines, as shown by Figure 7-3. This gave a total of twelve access tubes. Each access tube was "keyed" into a depth of about 3m below the expected failure zone. An expected failure zone had been predicted as part of the feasibility study by Cooper (1986), using a Bishop analysis for a range of predicted parameters. The total length of inclinometer tube was 117.5m. Although more inclinometers could have been installed for a relatively small increase in cost, the time consumed in reading the instruments would have become excessive.

7.1.3 In-Place Inclinometers/Wire Extensometers

Inclinometer access tubes are clearly useful for monitoring pre-failure movements, but could not be automatically logged nor, for obvious safety reasons, could they be monitored through failure. Hence with the aid of Geotechnical Instruments Ltd. the In-Place Inclinometer and 5 string Wire-Extensometer cradles were developed. Three Wire-Extensometer cradles and three In-Place Inclinometer strings would have been ideal, covering approximately the same three sections as the standard inclinometer access tubes, but unfortunately only two of each type could be afforded. These were positioned as shown in Figure 7-4. Loss of information associated with the cost cutting is clearly demonstrated by Plate 7-1 which shows the slip surface emerging in front of String Inclinometer 01.

Other instrumentation systems such as time-lapse photography and video filming were considered but rejected on either cost grounds, or as being too time consuming since manpower was another tightly budgeted commodity on this project. Detailed descriptions and workings of the individual instruments are set out in Section 9.

7.2 Programme

The experimental design at Selborne not only involved the physical design and planning of the monitoring equipment but also the programming of the work to fit into a relatively tight schedule. It was known from the beginning of the project that the site owner might wish to re-occupy the site for the clay-winning season of 1989 starting in April or May, though an opportunity for renegotiation had been promised. Enough time had to be left to monitor insitu conditions before bringing the slope to failure. Prior to this there had to be enough time to install and check the instrumentation and allow it to settle. Also once the slope had failed, time had to be allowed to dig out, locate and sample the slip surface. Hence a programme, Figure 7-5, was drawn up. The programme consisted of two summer seasons of fieldwork with one winter of monitoring insitu conditions followed by a second winter of recharging the slope with the help of natural seasonal recharge with the aim of failing the slope the following spring.

The project slipped significantly from the scheduled programme due to two main reasons. Late allocation of moneys by S.E.R.C delayed the first season of fieldwork until late summer 1987. The first winter was then extremely wet making site conditions very poor, a problem exacerbated by the Great Storm of October 16th. This first very wet winter was then followed by one of the driest summers and winters on record, which slowed the natural recharge element of the failure mechanism. The failure eventually occurred in July 1989 and due to pressure from the landowner the dig-out and slip surface sampling was carried out in only two weeks and the site decommissioned shortly thereafter.

8. INSTRUMENTATION DETAILS

The total cost of the instrumentation at Selborne came to in excess of £69,400 at 1987 prices in a project with a total budget of just under £200,000. The instrumentation consisted of 60 piezometers, 12 inclinometer access tubes, 2 in-place inclinometer strings (a total of 19 monitoring elements), 10 wire extensometers (5 each to two recording cradles), 20 recharge well/slip indicators and an automated logging system. All these instruments were installed to monitor an area covering 24m by 42m giving a density of one instrument per 9.7 square metres. This represents an extremely high intensity of instrumentation, the intensity of instrumentation as compared to previous slope failure studies is discussed in more detail in Chapter 17.1 and by Cooper and Grant (1989). The instrumentation had three major functions to perform;

1. To provide an extremely accurate and intensive bank of data for a detailed case record of a cutting up to, through and if possible beyond failure.

2. To give useful information on the pre-failure strains and displacements associated with first time slides in stiff, brittle clays for use in research areas such as how progressive failure mechanisms develop.

3. To provide a means of assessing the condition of the slope and hence the

effect of the recharge system at any particular time, so that some level of control could be exercised over the intended mechanism of failure. This involved monitoring factors such as; the variation of r_u within the slope and with time, accelerations within the rates of displacement being measured and any unexpected variation in measurements considered as representing 'background' or steady state conditions.

Since the aim of the project was to fail the slope, a large amount of duplication and redundancy had to be built into the system to ensure adequate coverage of the "final event", knowing that a large proportion of the instrumentation was likely to be lost. To help avoid instrument loss, especially in the event of a slow and prolonged failure, each instrument connection tube or cable had a 4m slip loop loosely coiled under the gauge house. Positioning of the gauge house also posed some problems since it obviously had to be at the top of the slope for safety reasons, but every metre it was positioned back from the crest of the slope, cost approximately another £250 in cabling, on a project that was already very tightly budgeted. Eventually a position with the front of the gauge house about 6m back from the crest was chosen which, in the end, proved to be just about right, leaving enough room in front of the gauge house, after failure, for safe working and allowing the remaining instruments to be read until the gauge house had to be moved off site.

9. INSTRUMENT INSTALLATION PROCEDURES

All instruments were installed in 150mm nominal diameter boreholes drilled using a continuous flight auger technique. Borehole depths were measured down the auger string. Each instrument cable was colour coded with waterproof sticky tape at 1m intervals immediately after installation.

Figure 9-1 and Plates 9-6 and 9-7 illustrate the procedure for instrument installation on the slope face. Instruments were installed on benches nominally 3.0 metres longitudinally and 1.5 metres vertically apart. At each stage a slightly steeper than 1:2 (vertical to horizontal) slope was formed. Boreholes were then drilled and instruments installed as close to the newly cut face as possible. When all work was complete the next bench was formed starting 0.5 metres from the toe of the previous cut face. In this way an overall 1:2 slope was formed with five narrow benches left for access to the instruments and recharge wells. Plates 9-1 to 9-4 show the procedures adopted for the installation of a typical piezometer and Plate 9-5 the final slope profile after the first season of field work.

9.1 Piezometers

The piezometers were embedded in a 250mm deep filter of saturated sand. A plastic waste bin of the correct volume was used to measure the volume of sand for each piezometer hole to ensure uniformity. The holes were then sealed with a bentonite pellet plug before being backfilled with a liquid cement/bentonite grout. The short filter length ensured a precise location for the measured pore pressure. All the pneumatic piezometers were pressure tested to 15m head of water immediately prior to, and immediately after, installation. The pressure test consisted of raising the applied head to 15m H_2O, and maintaining this head for 10 minutes without any loss of pressure.

Due to a fault on the vibrating wire piezometer readout unit the vibrating wire piezometers could not be tested immediately after installation, but all of these piezometers had had their signal cables fitted, cut to the correct length and tested by the manufacturer prior to delivery and hence it was considered safe to install without any testing. They were then tested within a few days of having been installed.

9.2 Recharge Wells/Slip Indicators

The recharge wells were constructed from 19mm PVC standpipe tube with sets of orthogonally alternating 8mm holes drilled at approximately 100mm intervals along the proposed response length (see Figure 9-2). The boreholes were backfilled with pea-gravel over the response length and then sealed with a bentonite pellet plug before being backfilled to the surface with a mixture of cement/bentonite grout and fine arisings. The slip indicators were simply 0.5m lengths of 6mm reinforcement bar connected to 18m lengths of non-stretchable fishing twine lowered to the bottom of the wells in the Autumn of 1988, before recharge started. Another 0.5m length of bar was kept to lower down from the top of the well after failure so that the position of the slip surface could be interpolated.

9.3 Inclinometer Access Tubes (including In-Place Inclinometer guide tubes)

Holes for the inclinometer access tubes were over drilled by about 100-200mm to guarantee the bottom readings should silt gather at the bottom of the access tubes. The access tubes were installed and the holes backfilled with a liquid cement/bentonite grout. Some of the access tubes were filled with water to prevent them becoming buoyant in the liquid grout whilst it set. All the joints in the access tubes, and the bottom end caps of the tubes were pop-rivetted in place to provide strength during installation. The joints and end caps were then sealed with mastic and wrapped in "Denso" tape. This prevented both ingress of groundwater and silt into the access tubes and also prevented water, added to the tubes to prevent buoyancy, leaking out and thereby diluting the liquid grout.

The inclinometer tubes were aligned during installation such that the locating grooves headed orthogonally downslope (A-B) and along the slope (C-D). This process was carried out "by eye" aligning grooves with survey pegs in place along the slope crest. Groove roll within the access tubes was not investigated for any of the inclinometers. This may have been an oversight but with access tube lengths ranging from 6 metres to only 15.5 metres, with the majority of tubes less than 9 meters in length, it is considered that any error due to groove roll will be minimal. Initial misalignment of the tubes however, is more likely to have lead to errors and I believe it was a mistake not to have carried out a full survey of inclinometer alignments post installation.

9.4 Wire Extensometers

The wire extensometers were installed in the summer of 1988. The extensometers were grouped into two sets of five, one set along grid line 3/4 and the other along grid line 4/5. Each set of five wire extensometers shared a common measuring frame positioned at the slope crest. An invar wire from the individual anchor to the frame was attached to a length of cycle chain which passed over a toothed wheel to a tensioning weight which was free to move in its own borehole. The frames, which were constructed from galvanised steel, were bolted at each corner to a 2m long reinforced concrete pile as illustrated in Figure 9-3. Plate 9-8 shows the wire extensometer frame at grid line 3/4.

Movement of the frame or anchor was recorded by a circular displacement transducer connected to the toothed wheel. Each anchor consisted of a 12mm galvanised reinforcement bar concreted into hand augured piles, the invar wire was passed through a hole towards the top of the anchor and held in place by means of two small grub screws. Due to the hardness of the clay, especially towards the bottom of the slope, some of the anchor piles were under 1m in depth (see Table 9-1). The holes for the tensioning weights were simply 5" augured holes (the largest available auger for the Minute Man mini drill rig being used) widened by hand auguring to 6" and lined with 6" PVC soil pipe. The holes varied in depth depending on the range of the particular extensometer.

Wire Extensometer Anchor Number	Grid Position	Anchor Pile Depth m
051	K10/11	0.83
052	R10/11	1.13
053	N10/11	0.78
054	D10/11	1.20
055	B10/11	2.20
056	L4/5	0.81
057	S4/5	1.15
058	P4/5	0.73
059	E4/5	1.20
060	B4/5	2.20

Table 9-1 Wire extensometer anchor pile depths.

10. AUTOMATED LOGGING SYSTEM

The "one-off" nature of the project and the remoteness of the site indicated a need for some kind of automatic recording system. The system used was one being developed by Geotechnical Instruments Ltd. At the time the system was still very much under pre-production development by the manufacturers and as such found to be quite complex and often laborious to programme and download, but gave an almost infallible data collection and storage facility. A system such as this could, obviously, only be connected to the "electronic" instruments, that was, the vibrating wire piezometers, in-place inclinometer strings and the wire extensometers. Figure 10-1 gives a general outline of the logging procedure. The facility for manual intervention at nearly any point in the system is an important feature of the system. This facility was incorporated for two main reasons, one; to guard against any component failure and, two; to check that the information being saved to disk was actually the same as the data arriving at the multiplexors (which, incidentally, was not the case when the system was first set up). A set of readings was taken manually every day using the DTM and compared to the logger printout to check the logging system. In the first few months after installation a number of faults were discovered in this way - mainly in the logging software but also a few hardware problems which were quickly rectified. Taking a set of readings manually every day also helped keep the site staff aware of the state of the slope and recharge system. Figure 10-2 shows the actual logging path of all the instruments connected to the system.

To guard against component failure the whole system was divided into two, with half of the instruments of each type going to separate loggers. Hence in the event of one logger being "lost" half the results should still be retained. In the event of the gauge house being involved in the failure it was likely that the power supply would be lost. Hence each logger ran off its own 12 volt battery which was connected to a charger permanently plugged into the mains. The batteries were of the fully sealed type so that they would continue to function even if they became inverted. Each logger also had its own internal Ni-Cad battery which was capable of keeping the logger memory functional for up to 24 hours should the 12 volt supply ever fail.

10.1 Logging Strategy

The logging system had a number of logging paths or strategies. The simplest and most straightforward strategy was the fixed interval or normal log, whereby a set of readings was taken at a fixed interval, say every 6 hours. The next level up from this was to only log selected channels at each pre-set interval. This system was obviously very wasteful in terms of memory since many sets of identical readings could be being stored, but was very good for day to day monitoring of data on a, say, 4 or 6 hour logging interval. In fact when the system was first being tested at Selborne this strategy was used, with a 6 hour logging interval, and the data compared each day to a set of readings taken manually. Also, all four sets of daily readings were compared to each other to spot "floating" readings - a good indication of loose connections in the multiplexors or more serious problems within the switching cards.

To save on memory a strategy could be set whereby a set of data was only stored if some reading, or readings, had changed by a pre-set "threshold" value from the last set of stored reading. In this way the logging interval, or scan time, could be reduced substantially without an equivalent increase in memory consumption. A problem with this system was that the threshold had to be set to a value larger than the natural "flicker" of the instrument to prevent a set of readings being taken after every scan. The thresholds could be set for all or any number of the instrument channels but once one threshold had been exceeded a full set of readings was stored to memory. A system that notified which channel number had exceeded the threshold would have been useful. At Selborne about 30 instruments were connected to each logger with usually only the displacement measuring instruments set to a threshold logging strategy. The thresholds were set quite low since comprehensive data collection was the aim. Thus the chances of one instrument exceeding its threshold value every scan time were quite high and hence the logging trips tended to be about 3 out of every 5 scan times. This still represented a 40% saving in logger memory over a normal logging strategy for the same level of data collection.

A strategy similar to the threshold logging strategy was the alarm strategy. Here certain alarm levels could be set along with a scanning interval. The logger would scan the channels without storing any data until one of the alarm levels was reached. Once this happened the logger would then switch to a new pre-set normal logging strategy. A second "lower" alarm could also be set which, when reached, would switch off the alarm strategy and return it to a passive scanning mode until another alarm level was reached. Up to four different alarm strategies could be set at any one time. To prevent accidental tripping of the alarms when, for example, an animal tripped over an extensometer wire, three successive scans had to reach or exceed the alarm level before the alarm strategy "tripped". On a one minute scan time this only represented 2 minutes before the alarms tripped. When the alarm strategy tripped, a pair of contacts within the logger closed, which could then be used to set off an external circuit. At Selborne this was used for two purposes, firstly it was used to trip the other logger so that both loggers would be acting on the same strategy, and secondly it was used to trigger on site alarms (buzzer and flood lights) and an off-site autodial system to warn staff resident in Southampton that the particular alarm level had been reached.

10.2 Logger Memory

The loggers used at Selborne had internal memory allocations of 170 kbytes, enough for about 86,000 readings , which represented about 44 hours of continuous logging at minutely intervals on Logger 01 and about 53 hours of logging at the same rate on Logger 02 (see Table 10-1). With the system set to a threshold strategy these times expanded to about 75 hours for Logger 01 and over 88 hours for Logger 02, that is plenty of time to cover from 5.00pm Friday to 10.00am Monday (65 hours), although the site was usually visited at least once each weekend from Easter 1989 until failure occurred in July. This visit was usually made due to worries about the state of the recharge system rather than the logging system. Once the logger memories were full the system automatically stopped logging and switched off. A system whereby logging continued with alternate sets of stored readings being over-written was promised but never materialised. Once full it took over an hour to download a logger to the Interrogator Unit - representing an hour of possible lost logging time. Hence a policy of keeping at least 10,000 readings space of memory free was operated. By doing this a second logging strategy could be run whilst the logger was downloading to the Interrogator Unit. Once the file had been downloaded from the Logger to the Interrogator Unit and from the Interrogator Unit to floppy disk via the micro-computer, an operation taking another 30-40 minutes, the logger memory could be cleared and a new logging strategy set for the next 24 hours. In this way the loggers were operating for literally 24 hours a day and hence the failure event was very unlikely to be "missed". It can be seen that since two loggers were being run in parallel, downloading, checking data and reprogramming the logging strategies took up a large proportion of the working day. This became even more time consuming towards the end of the contract when regular tripping of alarm levels meant constant downloading, clearing of memories (to re-set alarms) and reprogramming of strategies. Limitations within the logging system at this stage of its development associated with the re-setting of the alarms made this process more cumbersome by requiring total clearance of all memories once an alarm strategy had been tripped before the next alarm strategy could be programmed.

Logger Channels Instrument Type	Logger ABCD01 Channel Numbers	Logger ABCD02 Channel Numbers
VWP Piezometers	001-016	017-030
Wire Extensometers	050-055	056-060
String Inclos.	101-111	201-208
TOTAL No CHANNELS	32	27
NUMBER OF FULL SETS OF READINGS	86,000/32 = **2687**	86,000/27 = **3185**
Logging every 1min	44 hrs 47 min	53 hrs 5 min
Threshold Log (3 out of 5 readings say)	**74 hrs 38 min**	**88 hrs 28 min**

N.B. 5.00pm Friday to 10.00am Monday = 65 Hours

Table 10-1. Logger Memory Capacities.

11. RECHARGE SYSTEM AND PROCEDURE

The aim of the project was to use the recharge system to raise porewater pressures behind the slope, and by raising the head on successive rows of wells, slowly bring the zone of raised porewater pressures forwards towards the zone of slope instability. Hopefully, thereby, a "deep seated" failure mechanism would be initiated (see Figure 11-1).

The idea of failing a slope by the use of recharge was largely a new idea and the likely behaviour of recharge wells very much an unknown quantity. The Recharge Wells, which also acted as Slip Indicators, were constructed from 19mm PVC piezometer tube as described in Section 9.2.

11.1 Recharge Well Test Bed

A single recharge well plus two pneumatic piezometers were installed in a test bed approximately 30m away from the main slope in the summer of 1988, Figure 11-2. Very little conclusive information was gained from this trial, possibly due to the lack of time given to allow equilibrium conditions to be achieved in the ground prior to commencing recharge. The only real conclusions gained from the test were that:-

1. Large volumes of water could be involved.

and that

2. The water supply/recharge well connection detail would require careful consideration.

11.2 Recharge System Design and Construction

Flexibility within the system was a major design consideration. A system that allowed the water supply system to each row of wells to be built one row at a time was important. In this way, if required, the system design could be modified in light of experience. Also new lines of recharge wells could be brought on line with the minimum of disruption to existing systems and hence the minimum disruption to the current pore water regime acting within the slope.

The system needed to give approximately equal head to all rows, therefore the system design had to be designed on a parallel supply and not a series supply basis. Other considerations were resistance to frost and the ability to withstand temporary interruptions of water supply (due to feeding cattle, filling kettle and such like). It was decided to build the system from standard household waste-pipe, this was easily available and apart from joints relatively cheap. It could be easily and quickly constructed and was very flexible : T-bends, L-bends, end stops, and such like were all readily available. Waste-pipe also had the advantage of being available in a number of diameters. Some match, therefore, of supply flow to likely demand could be made. Standard garden hose could have supplied the required quantities of water, but it was hoped that the extra volume of the waste-pipe would help prevent freezing. The additional pipe volume would also act as an extra reservoir during interruptions to the mains supply.

Figure 11-3 shows a schematic view of the Recharge System. Water was fed from the mains supply to the large (40 gallon) central heating tank on the Gauge House roof. Water was gravity fed from here via 1½" waste-pipe to small (5 gallon) central heating tanks, one for each row of recharge wells. A ball valve on the inflow pipe controlled the water level within each small tank. Water was then allowed to flow through 1¼" waste-pipe from the small tanks to the individual recharge well. The waste-pipe to recharge well connection details are illustrated in Plate 11-1. The small central heating tanks were removed and replaced with in-line connectors when the recharge head was raised to the height of the gauge house roof. When further head increases were required a scaffold tower was built adjacent to the gauge house (see Plate 11-2), it was considered unsafe to put the large (40 gallon) tank on top of this scaffold so a 5 gallon tank with a permanent mains connection was used instead. Once the recharge head exceeded 5 metres above ground level the joints on the plastic waste pipe could no longer sustain the pressure and hence the whole system was replaced with ¾" reinforced garden hose. This lead to the system now being connected in series (see

Figure 11-4) rather than parallel and the site staff became concerned as to head losses in the system. An experiment was therefore devised whereby a length of hose was connected from the last well in the series, grid position K2, to the top of the scaffold tower. The recharge system was then set running and left for half a day. The head loss in the system was found to be less than 0.1 metres i.e. the accuracy to which the piezometers were being read.

11.3 Sequence of Recharge

A summary of the recharge events is given in Table 11-1 and expressed diagrammatically as Figure 11-5. A full diary of recharge events is attached as Appendix F.

The sequence of recharging can be summarised into nine basic steps leading to failure (Figure 11-5). The first four steps introduce recharge behind the slope gradually bringing it forward in stages to the slope crest. Step five introduces recharge to the front line of wells (grid line K) on the slope and step six raises the head to the same level as the other wells. The next three stages then gradually raise the applied head over the whole site until complete failure (collapse) is achieved. Failure was actually reached at stage eight where it became impossible to read any of the inclinometer access tubes on the slope. At this point the project was running almost three months behind schedule and there was pressure from the land owner to re-occupy the site. Since no additional manually read data could be obtained from the inclinometer access tubes it was decided to bring the slope to collapse as quickly as possible. This was achieved by raising the applied head from 5.35m to 7.00m on the 13th July. Collapse occurred three days later.

Recharge System - summary of events		
date	time	event
01/12/88	14:00	Recharge switched ON,grid line D to GROUND LEVEL
21/12/88	10:30	Recharge switched OFF - Christmas break
11/01/89	11:00	Recharge ON
31/01/89	14:00	Recharge grid line F switched ON to GROUND LEVEL
21/03/89	14:00	Recharge OFF - for connection of next line and raising of header tanks
22/03/89	12:15	Recharge switched ON, grid lines D and F to 3.0m head grid line H to GROUND LEVEL
12/04/89	13:00	Recharge OFF - preparatory work for raising recharge head on grid line H
13/04/89	15:30	Recharge switched ON, grid lines D,F and H to 3.0m head
25/04/89	10:50	Recharge OFF - preparation for connection in of grid line K
26/04/89	15:00	Recharge switched ON, grid lines D,H & F to 3.0m head, grid line K to GROUND LEVEL i.e 3.0m above bench level
12/05/89	14:00	Grid line K up to full head 3.0m above ground level
16/05/89	13:45	Recharge switched OFF - for raising of head
date	time	event

17/05/89	15:15	Recharge head raised to 4.23m a.g.l.
17/5/89 to 22/5/89		numerous small leaks therefore :-
25/05/89 and 31/05/89		all connections on grid line D fibre glassed and all connections on grid lines F,H and K fibre glassed
01/06/89		Recharge ON to 4.23m head
02/06/89	14:00	Recharge head raised to 5.35m a.g.l.
8 & 9/06/89		leaks everywhere on system, joints bursting at regular intervals therefore :-
12/06/89		Recharge switched OFF - system rebuilt using 12.5mm dia. reinforced hosepipe
15/06/89	12:00	Recharge switched ON all rows to 5.35m head
06/07/89		recharge noticed to be running "as if there were a leak in the system" - no leak found therefore a well may have split at depth. (Theory discounted following post-collapse digout.)
13/07/89	14:40	Recharge head raised to 7.00m a.g.l.
16/07/89		COLLAPSE OF SLOPE - recharge switched off at 14:30 hrs.

Ground level is taken as the height of a peg randomly installed at a point 7.5m back from the slope crest, adjacent to the gauge house.

Table 11-1. Summary of Recharge Events

11.4 Monitoring Response to Recharge

The response of the slope to recharge events was monitored by the use of the piezometers. The response of the piezometers can basically be divided into three types as shown in Figure 11-6

Type A - no response or very limited response

Type B - damped response

Type C - direct response

Type A instruments were totally unaffected by individual recharge events and either totally unaffected or only very slightly affected by the general increase in pore water pressure. They mark the boundaries of the zone of influence of the recharge system. Type B instruments showed some response to individual recharge events but were more influenced by the general increase in the soil pore water pressure. They showed no response to short interruptions in the recharge flow. Type C instruments showed a direct response to recharge events, responding within hours to rises or falls in recharge head. During recharge these instruments were often used as indicators of either leaks or airlocks within the recharge system. The way in which Type C instruments were used to monitor the status of the recharge system (as opposed to monitoring the response of the slope to the recharge) is illustrated below. Figure 11-7a shows the readings for pneumatic piezometer PP24, a Type C response piezometer, for the whole of the recharge period and Figure 11-7b shows the readings for PP24 through February and March 1989. The appropriate entries from the site diaries (Appendix F) for Points A to G on Figure 11-7b are listed below :

- **A** Recharge switched OFF for connection of wells on line H and raising head on lines F and D to 3m a.g.l.
- **B** Recharge switched ON.
- **C** Leak in supply line.
- **D** Leak mended and recharge switched on again.
- **E** Leak in supply line.
- **F** Repaired.
- **G** Recharge switched OFF for connection of wells on grid line K.

Figures 11-8 and 11-9 show the distribution of B and C type response piezometers within the slope. The actual area of affect due to recharging is well contained within the intended recharge zone although it possibly did not extend quite as deep or as far forward as originally envisaged. Early on in the recharge process this was considered to be possibly due to smearing of the recharge wells during installation, the smearing preventing the full transmission of hydrostatic head from the wells to the ground. A small quantity of gypsum was therefore added to the recharge header tank to try and restructure the clay within the smeared zone. No effects were noted following addition of the gypsum and its use was discontinued before any permanent chemical changes were made to the clay within the slope. Maximum pressures created in the recharge zone equate to a maximum r_u of 0.43. This value is thought unlikely to have caused any hydraulic fracture within the fissure system (Bjerrum et al 1972). If hydraulic fracturing had occurred the area affected by the recharge would have extended much further forward.

Figure 11-9 also clearly illustrates the zone of negative piezometer readings where the pore water pressures dropped to below atmospheric pressure, and were observed to remain so for the duration of the project. This phenomenon is discussed in more detail in Section 14.5.

11.5 Assessment of Recharge System Performance

Despite an admittedly ad-hoc development of the water supply system to the wells, the recharge system performed extremely well and gave an excellent method of controlling the pore water pressure regime within the slope. Post failure examinations of recharge wells during the dig out following collapse showed that none of the wells investigated had ruptured despite considerable deformation as illustrated by Plate 11-3. The system, as finally composed, can be highly recommended as a relatively cheap, flexible and accurately manageable method of controlling pore water regimes or boundary conditions in any future experiment of this type.

12. TWO DIMENSIONAL ISOLATION

In order to try and obtain a plane strain type failure mode, and hopefully to alleviate as far as possible the influence of end effects, side shear breaks were formed at the north and south extremities of the slope. These side shear breaks or isolating trenches were constructed during the second summer of site works when the marginally stable slope had successfully survived intact over the first winter. The side shear breaks were formed by placing a double layer of agricultural grade PVC sheeting into a trench dug using a Poclain VC15 long reach excavator. Each trench was dug in two sections, one from the base and one from the top of the slope. In each case the PVC sheeting was carefully placed into the trench and the trench completely backfilled before the second half of the trench was dug. In this way each section of trench was left open for only a matter of hours. Lateral face readings were taken on the inclinometers adjacent to the isolating trenches immediately prior to and soon after trench construction. No discernable movements were noted in any of the inclinometers.

Figure 12-1 shows the profiles of the isolating trenches, the shape basically being determined by the reach and digging power of the Poclain VC15 excavator. Below about 10 metres depth (from top of slope) the digging was found to be extremely difficult due to the hardness of the clay, the lower section of the side shear breaks represent the maximum depth that could be achieved with the power of the VC15 excavator. Plate 12-1 shows the southern side shear break exposed during the post-failure dig out. The double layer of PVC sheeting can clearly be seen. Note that the movement appears to have involved the inner layer of soil moving against the plastic rather than a plastic to plastic movement as had originally been expected.

The influence of the side shear breaks on the eventual failure pattern is discussed in Section 16.5.

13. SURVEY MONITORING

13.1 General

A survey programme was initiated firstly to establish the finished slope profile and secondly to try and monitor any perceivable ground strains within the slope during the duration of the project. Survey work was also carried out post-failure to establish the exact slip surface position and final slip-mass profile.

As part of a final year undergraduate project (Dover 1988) three survey reference pillars were built at the locations shown on Figure 13-1. Three pillars were used since this represents the minimum number required to establish whether or not any of the reference objects were moving, and if so to quantify the direction and magnitude of any such movements.

For monitoring purposes the slope had to be fully visible from at least two of the reference positions, a task made difficult by :

a) the presence of a corrugated iron clay drying shed located towards the centre of the pit,

and,

b) continued earth moving activities within the pit, which often meant earth mounds obstructing the view to the slope would come and go daily.

It was essential that the reference pillars remained stable for the duration of the project. To try and ensure this each pillar was founded on three 2.75m deep reinforced concrete piles drilled using Southampton University's own "Minute Man" rotary drilling rig. The construction details of the survey pillars are shown on the insert to Figure 13-1. Each reference pillar had an Ordnance Survey brass triangulation spider set and accurately levelled into its top. The spider was to allow accurate placing and re-placing of the survey instruments during monitoring.

No free-standing targets as described in M Dover's report were actually installed nor were any inclinometer targets produced by Geotechnical Instruments Limited, but reflecting inclinometer top targets were produced by technicians within the civil engineering department at Southampton University, to a specification drawn up by the author (see Figure 13-2).

The reflective targets were considered necessary since it was intended to bring the slope to a state of collapse, at which point it would be desirable, if possible, to obtain some real-time survey data of the movements. Obviously with the slope in such an unstable condition it would be dangerous for personnel to go onto the slope, therefore some form of passive EDM reflecting system was required. Targets coated with

Scotchlite survey reflective material were produced to fit into the top of the twelve inclinometer access tubes. Details of the targets are shown on Figure 13-2.

13.2 Equipment and Personnel

The equipment used for all the survey work was a Wild T1600 theodolite connected to a Wild DIOR 3002 EDM distance measuring device (edm) siting onto either a tri-brach mounted Wild Triple Prism, a pogo-stick mounted Wild triple-prism or the reflective inclinometer targets. The equipment was held, and consequently the survey work largely carried out by, the Coastal Research Department at Southampton University with site staff (usually the author) acting as chainman to ensure accurate positioning of the pogo-stick at the monitoring positions. Using a pogo-mounted prism inevitably lead to some errors due to non verticality of the pogo-stick, but this was minimised by using a centring level bubble on the pogo-stick.

The black plastic sheeting covering the slope throughout the time that surveys were being carried out and the corrugated iron clay drying shed in the centre of the pit both gave rise to strong heat-hazes being formed on sunny days, making siting onto many of the lower slope survey positions virtually impossible after about 10am during June and July. All the surveys during this time were therefore carried out early in the morning, but inevitably some errors must have occurred, especially in the EDM readings to the inclinometer targets which were only just above the black plastic sheeting.

13.3 Survey Procedure

The surveys can be classified into three main types as follows:

a) Topographical Survey,

b) Survey Monitoring of 22 "Key Points",

c) Survey Monitoring of Reflective Inclinometer Targets.

a) and c) were carried out with the theodolite at one reference pillar only whereas surveys of type b) were carried out from two reference pillars ('A' and 'B').

13.3.1 Topographical Surveys

Topographical surveys were taken with the theodolite set up at reference pillar 'B'. This station gave the best view of the slope and had the advantage of being accessible by Land Rover at all times (access by vehicle to stations 'A' and 'C' was impossible from October to May). The surveys were carried out by siting onto a hand held-pogo mounted triple-prism and booking horizontal angle, vertical angle and slope distance on one face of the instrument only.

The first full survey of the finished slope following final bank trimming, installation of side shear brakes and installation of wire extensometer anchors and frames, took place on 27th September 1988. Further topographical surveys were carried out on the 17th, 20th and 24th July 1989 and 4th August 1989 following slope collapse on the 16th July to establish the post-failure slip-mass profile and the position of the slip surface located within the two inspection trenches.

13.3.2 Monitoring of 22 Key Points

The 22 "Key Point" survey monitoring was the most comprehensive survey monitoring carried out at Selborne. The 22 points represent the top of the twelve inclinometer access tubes, the two string inclinometer positions and the wire extensometer anchor frames and on slope anchor positions. All these points were permanent, easily relocatable positions where continuous displacement data would also be available from the instrumentation logging system. Cross calibration of data would therefore be possible and some estimate of a three dimensional fix for the one and two dimensional instrumentation output could be made. An initial survey of the 22 positions was carried out on the 6th March 1989 from reference pillars 'A' and 'B', readings being taken on both faces of the instrument. Further surveys were carried out from these two pillars with readings being taken on one face only on the 5th, 11th and 13th July 1989 prior to final slope collapse on the 16th July, and twice following collapse on the 17th and 20th of July 1989.

13.3.3 Monitoring of Reflective Inclinometer Targets

Surveys of the reflective inclinometer top targets were carried out from pillar 'B'. Readings of horizontal angle, vertical angle and slope distance were taken on one face of the instrument only. Due to the length of the targets, and to check the EDM was reflecting off the target, readings were taken with the instrument siting to the top and to the base of the target.

13.4 Survey Error Analysis

The major source of survey error at Selborne was expected to be a systematic error due to the reference pillar 'B', which was founded on made ground, moving due to consolidation or slipping of the mound it was on. The systematic error could easily be located and relatively easily be allowed for in the data analysis. In practice the largest source of errors appears to have been random errors from the EDM.

Reading discrepancies due to seasonal swelling and contraction of the clay give the appearance of data errors. These movements although possibly masking the displacement data being sought from the survey cannot truly be described as errors since they represent the real behaviour of the slope. The reference pillars A, B, and

C were installed such that they were founded below a depth likely to be subject to seasonal moisture changes (Figure 13-1). Monitoring points on the slope face were founded below an impermeable plastic sheet which reduced seasonal moisture content changes. Material on the slope face, especially towards the base of the slope, was also undergoing swelling due to unloading affects following slope formation. This swelling will have been of a greater magnitude than seasonal swelling and also have extended to a greater depth from the slope face.

13.4.1 Movement of Reference Object

Random survey errors have been analyzed in accordance with the methods stated in Bannister and Raymond (1985). Using these methods any result differing by greater than three standard deviations (3σ) from the mean has a less than 1:400 probability of being correct. The 3σ value has been used as the acceptable error criteria at Selborne.

Analysis of data to the reference pillars for all the surveys gives the following results:

Included Angle @ Reference Pillar	Size of Field (Number of Readings)	3σ (equivalent mm over 200m)
A	16	17
B	34	15
C	–	-

Table 13-1 Survey errors -Included angles

Edm measurement along line :-	Size of Field (Number of Readings)	3σ (mm)
AB	25	14
AC	7	132
BC	24	82

Table 13-2 Survey errors - EDM

From this it can be seen that the general level of survey accuracy achieved at Selborne was not great. Closer inspection of the data, however, indicates that the spread of data associated with the measurement of angles is far less than that associated with the measurement of distances.

For example the surveys of 27th September 1988 and 6th March 1989 both indicate errors of 24mm in the EDM data measurements to the reference pillars but errors of only 0.1 and 0.36 seconds of arc in the horizontal angles. These angles are equivalent to errors of about 1mm and 3.6mm over a distance of 200m. Consequently the series of "22 point" surveys represents the only survey data where the magnitude of expected early strains is greater than that of the random survey errors.

Also note that the line AB, the base line for the "22 point" surveys, has a random error of only 14mm. It is therefore believed that the data for the "22 point" surveys provides an important and reliable case history for the surface strains leading up to failure at Selborne. However any data used for analysis using EDM measurements must be treated with caution unless verified by other data.

All the survey data with error analyses is presented as Appendix G.

14. INSTRUMENTATION PERFORMANCE

14.0.1 General

The instrumentation at Selborne was fulfilling two functions, firstly it was being used to monitor the conditions within the slope as it progressed from a stable to an unstable condition and, secondly the instrumentation itself was acting as the research subject. This included not only the newly developed forms of string or "in-place" inclinometers and wire extensometers but also the performance of the proprietary off-the-shelf inclinometers and piezometers. A major aim of the project was to try and establish the exact relationships between instrumentation performance and ground conditions. It was important to discover to what extent ground-instrumentation is just an indicator that "something" is happening and to what extent it can be used as an accurate gauge to say how much of that something has happened. In this aim the Selborne experiment has been a great success providing an extensive and valuable case load of field data to cross examine and prove the accuracy of the instrumentation.

The degree of instrumentation was extremely high. Excluding the recharge wells which also acted as standpipe piezometers prior to being brought "on-line", there were 84 monitoring points in an area of 42 metres by 24 metres giving an average intensity of one instrument per 12 square metres of plan area during the period of recharge (or one per 9.7 m^2 if recharge wells are included). In three dimensions, again excluding the recharge wells, the intensity of instrumentation is one instrument per 78 m^3 of soil.

This quantity of instrumentation gave not only an intensive coverage for monitoring purposes but also provided a large amount of redundancy and overlap allowing the performance of different instrument types to be compared.

14.1 Reliability

Excluding the 12 manually read inclinometer access tubes there were some 89 individually readable instrument elements installed in the slope. Of these only three failed during the 22 month period between installation and slope collapse. One of these failures actually being caused by the site staff accidentally damaging a signal cable and subsequently destroying the instrument whilst attempting to recover the (still serviceable) instrument to expedite a full repair.

Reliability can be calculated as $\frac{89-(3\times12/22)}{89}$ % per annum

$$= 98.2 \text{ \% per annum,}$$

an extremely high figure, reflecting the care that went into instrument preparation and installation. All instruments were checked for correct working immediately before and immediately after installation, and all "loose" cable ends coiled and wrapped in polythene bags for protection. The remote location of the site and the local reputation of the landowner helped, in that no vandalism, other than by stray cattle, of the instrumentation or the gauge-house took place.

The inclinometer torpedo itself was found to be extremely reliable although there were a few minor problems with the cartridge readout unit. The torpedo itself had travelled over 17.3 kilometres before requiring attention to the nylon wheel bearings in the week prior to slope collapse.

The reliability of the data logging system was not quite so good, which considering that at the time it was a new and largely untried system is not surprising. Even so most of the problems were due to software errors and not component failure.

14.2 Piezometers

As detailed in Section 7.1 two main types of piezometer were employed at Selborne, twin-tube pneumatics and vibrating wire piezometers. Each of the recharge wells also acted as a standpipe piezometer prior to being brought "on line" as a recharge well. The vibrating wire piezometers were concentrated in the zone of expected slip surface formation not only because they could be continuously electronically monitored but also because of their expected quicker response time in low permeability soils compared with pneumatic piezometers. In effect both instrument types were found to have a rapid response to changes in pore-water conditions (see Sections 11.4 and 14.4) due to the highly fissured nature of the Gault Clay.

14.3 Post-Installation Lag Times

The installation of nine additional piezometers during a second season of fieldwork in June 1988 allowed the effects of installation and the post installation "lag time" for equilibration with the surrounding ground condition to be evaluated.

Of the nine additional piezometers seven were twin-tube pneumatics and the remaining two were vibrating wire piezometers. Plots of total head against time from installation for these piezometers are shown on Figures 14-2 and 14-3. Figure 14-4 provides the readings for the same period for piezometers P08 and V07 which were installed the previous summer. All the piezometers, with the exception of piezometers P24 and P27, take a remarkably similar amount of time to equilibrate with the surrounding pore pressure regime. The Lag Time appears to be between 55 and 60 days, the vibrating wire piezometers possibly equilibrating slightly quicker than the pneumatics, but only by a day or two.

The two exceptions both equilibrated much faster, having reached a stable condition within 20 days of installation. The reason for this is illustrated by Figure 14-5 which shows the time lag in terms of pressure heads rather than total heads. It can be seen that piezometers P24 and P27 had an initial pressure head of around 3.5 mH2O when first read compared to readings of close to zero for all the other piezometers. The "amount" of equilibration required was therefore much less than that for the other piezometers. It is interesting to note that the time taken for piezometers P24 and P30 to equilibrate is approximately equal to the time required by the other piezometers to equilibrate from 3.5 mH2O to their final pressure, i.e. the rate of equilibration is remarkably constant for all the piezometers. This suggests that the lag time is a "material" constant either related to the mass permeability of the ground or to the permeability of the piezometer filter medium, whichever represents the greater obstruction to inflow. In this case the mass permeability of the ground was typically $4x10^{-9}$ m/s (Cooper 1986)) and the permeability of the filter pocket about 10^{-3}m/s. Calculations using Hvorslev formulae (Appendix H) show response times of less than a minute for piezometers, but of over 10 hours if the permeability of the ground is the controlling factor. Apart from piezometers P24 and P27 the lag time does not appear to have been affected significantly by either depth of installation or the final pressure head to which the piezometer must equilibrate.

How the "locked in" initial pressure heads for P24 and P27 were created can only be guessed at. The site diaries report that P24 and P27 were drilled, the boreholes left open overnight then the instruments installed the following morning and the boreholes immediately grouted to ground level. All the other piezometers installed during the second season of fieldwork had a slight delay between the instrument being installed and covered with bentonite pellets and final grouting. In no case was the delay more than half a day, but may have been long enough to have some effect. The most likely affect being that the bentonite pellets absorbed water from the sand filter around the piezometer tip, despite approximately 50 litres of water having been placed down each hole after the pellets, causing the piezometer tip to possibly have become unsaturated. Yet another example of the care that must be bestowed upon the instruments during installation. If the instruments are to be expected to perform in the same manner then they must be installed in the same manner.

Figure 14-6 illustrates the lag times in terms of pressure heads for two pneumatic piezometers installed in the recharge test bed about 20 metres south of the main slope. Again it can be seen that the two instruments took just under 60 days to equilibrate, about the same time as the majority of the other piezometers. Interestingly the two piezometers, only 2.8 metres apart and installed to the same depth, never read the same. Once the test bed trials were abandoned the two piezometers were still read daily and P32 consistently read 0.3 mH2O greater than P31. This either represents a zero error within one of the instruments or a local hydraulic gradient of 0.12.

In practical terms the time lag of up to 60 days represents a considerable period. Where piezometers are being relied upon to control construction activities, care must be taken to ensure that readings have reequilibrated before the activities they are intended to monitor commence. This may cause significant delays in the programming of construction works. In many cases it is only the change in pore water pressure that is considered important, a piezometer still equilibrating will mask this change, or give the impression that the pore water pressure is still changing when in fact it is not. This could lead to delays in construction activity or possibly even the more dangerous situation whereby pore pressures are approaching a critical value but the instrumentation is misinterpreted as indicating otherwise.

14.4 Response to Changes in Pore Water Conditions

Figure 14-7 shows the response times for five increases in recharge head for V13. V13 represents one of the piezometers that showed a direct response to recharge events, a Type C response as described in Chapter 11.4.

It can be seen that the pore water pressure increase is reflected in the piezometer reading within 15 to 20 hours of the recharge head being raised. Figure 14-8 shows the response time for day 165 (15th June 1989) in more detail. On day 165 the pressure head was raised to 5.35m AGL after having been switched off for three days whilst the recharge system was being rebuilt using reinforced garden hose. In effect therefore the head was raised by about 5½ meters. This represents the largest single jump in applied head that took place. The piezometer reading took slightly longer to equilibrate than for the other events, but had stabilized within about 22 hours.

Figures 14-9 and 14-10 show the readings for VWP13 in the hours following two "switch-off" events. Figure 14-9 represents the switch-off event prior to the head rise of day 165. The head was lost when the system sprung a leak at about 6:00am on day 162 (12th June), the water supply being switched off at about 10:00am when the site staff reached site. After 50 hours the reading on VWP13 is seen to be still falling but only very slowly. The other switch-off event illustrated (Figure 14-10) is for day 135 (16th May 1989). The recharge system was switched off at 1:35pm on the 16th May for raising the recharge head from 3.0m AGL to 4.23m AGL and switched on again at 3:15pm the following day. The reading on VWP13 was still falling immediately prior to the system being switched back on, which was some 25 hours after the initial switch off event.

The slower response time to a fall in recharge head is not surprising. The reaction to a recharge rise is a stress change effect within the soil that can quickly be transmitted through the pore water. The only flow actually required being that into the piezometer tip to record the increased pressure. A fall in recharge head does not have a direct driving force but is in effect a dissipation phenomenon in that previously created excess pore pressures will have to dissipate via a process akin to consolidation. The dissipation rate dependent upon the mass permeability of the soil and the prevailing boundary conditions.

14.5 Pneumatic Piezometers Reading Negative Pressures

Early on in the project the piezometers towards the base of the slope were observed to become "negative" i.e. the pore pressures at the piezometer tips dropped to below atmospheric pressure. The process causing this fall in pore water pressure was dilation due to direct stress relief as noted by Bishop (1957) and also observed at Saxon Pit (Burland, Longworth and Moore 1977). The transducers connected to the vibrating wire piezometers could quite easily handle this "negative" reading, since the only physical quantity being measured was the frequency of vibration of the piezometer wire. The pneumatic piezometers however, which normally relied on equalizing a pore water pressure with air pressure could not be read conventionally once the pore water pressure dropped to below atmospheric pressure. A system was therefore developed in association with the instrument manufacturers to read these "negative pneumatics".

To understand how the system works it is first necessary to explain in some detail how a conventional twin-tube pneumatic piezometer is read. Conventional pneumatic piezometers are operated by equalizing the pore water pressure acting on a small diaphragm with gas released from a readout unit to a small air sac which acts on the piezometer side of the diaphragm as shown in Figure 14-11. The air sac is pressurized until the diaphragm moves ("opens") enough to allow a return flow of air to the readout unit (see Figure 14-12a). Once the return flow has been achieved the pressurizing system is removed and the diaphragm allowed to close under the action of the pore water pressure. The air pressure on the airline to the diaphragm from the readout unit is monitored and a reading taken once the diaphragm has closed. This value is the measured pore water pressure.

Since the diaphragm actually moves during the reading operation, there is obviously some pore water volume change within the piezometer tip which can lead to reading errors, especially in low permeability soils such as the Gault Clay. Therefore it is important to pressurise the diaphragm slowly during the reading operation so that the minimum amount of over-pressurization is used to "open" the diaphragm. If the operation is carried out at speed then an over-pressurisation of the diaphragm will lead to water being expelled from the piezometer tip and consequently an underestimation of the pore water pressure will be made when the diaphragm is again allowed to close.

The system developed for reading "negative" pneumatic piezometers is, in effect, the same as for reading conventional pneumatic piezometers but that the zero point of the scale is shifted. As with conventional pneumatic piezometers the negative apparatus requires the air pressure on the readout side of the piezometer to be kept lower than the pore water pressure acting on the diaphragm i.e. for the negative apparatus a vacuum greater than the negative pore water pressure has to be applied to both readout airlines to keep the diaphragm closed prior to reading (see Figure 14-12b).

Once a vacuum has been applied to both lines of the piezometer the pressure on the airline to the piezometer diaphragm from the readout is increased i.e. the vacuum lowered, until the pressure behind the diaphragm equals the pressure (or vacuum) due to the pore water pressure. At this point the diaphragm will begin to open and the vacuum in the return airline from the piezometer to the readout unit will fall (i.e. the pressure will rise as in the conventional reading system).

Readings of the "negative" pressures were achieved by having two readout units, later replaced by one readout unit and one vacuum gauge, placed in the system as shown by Figure 14-12b. Readout No.2 monitors the loss of vacuum (pressure increase) in the airline to the piezometer. Readout No.1, which was eventually replaced by a vacuum gauge, is connected to the piezometer return line and is the equivalent of the rotawink in the conventional system.

When the pressure being applied behind the diaphragm equals the pore water pressure the diaphragm will begin to open and the vacuum in the return line will be lost i.e. the vacuum reading on Readout No.1 will drop. Also it is at this point that the value of the vacuum on Readout No.2 is equivalent to the negative pore water pressure acting on the diaphragm. The reading on Readout No.2 will level off or fall slightly as the reading on Readout No.1 drops, as illustrated in Figure 14-13. It is at this point that the value shown on Readout No.2 should be taken as the measured value of the negative pore water pressure. Readings must be carried out slowly to ensure that the reading point is not missed. The value on Readout No.2 changes rapidly once the diaphragm opens and therefore close observation must be kept on this readout unit so that the required point is not missed.

When the vacuum is initially created on the piezometer lines the diaphragm will be "sucked" closed creating a suction within the piezometer tip. Time must be allowed for the suction within the tip to re-equilibrate with the surrounding soil by pore water being sucked into the piezometer tip. In low permeability soils the re-equilibration time may be quite considerable. At Selborne the piezometers were left for one month after the initial vacuum application before being read.

If full equilibration of the negative pneumatics to the applied vacuum has not occurred prior to reading then one of two errors may occur. If the piezometer tip has only partially re-equilibrated then only a partial negative pore water pressure will be measured, i.e. an overestimation of the vacuum (underestimation of the true pore water pressure) will be made. As a worst case if no, or very little, re-equilibration has taken place due to the low permeability of the soil then no force will be acting on the ground side of the diaphragm and therefore the measurements made will reflect the maximum difference in suction that can be sustained by the air sac/diaphragm combination between the piezometer lines "to" and "from" the readout unit. At Selborne this was observed during the first attempt to read the negative pneumatics when all the piezometers were discovered to have a pore water pressure of minus 4.5 mH_2O, reflecting the elasticity of the air sac as opposed to the true ground pore water pressure.

The whole process of reading "negative" pore water pressures using this system is very time consuming, in excess of half an hour per piezometer, if the reading is to be repeated for confidence, and therefore could not be (and is not recommended to be) used as a day to day measure.

In all nine full sets of negative pneumatic readings were taken from April 1988 to March 1989. Another set of readings was attempted just prior to failure during June 1989, but due to equipment failure, a leak on the vacuum gauge, and time pressures, the readings were never completed, but all piezometers with the possible exception of P21 were still recording strong negative pressure heads. Comparisons of data from the negative pneumatic piezometers read using this system and nearby vibrating wire piezometers, see Figure 14-14, indicates that the negative pneumatic apparatus gave a lower estimate (higher vacuum) of the negative pore water pressure than that indicated by the vibrating wire piezometers, the difference being between 0.2 and 0.4 mH_2O. The error is most likely to be due to the pore water volume change required during the operation of the negative pneumatic apparatus and also due to pressure losses within the system, the pressure being read at the readout unit inevitably being lower than the pressure at the diaphragm due to the compressibility / elasticity of the gasses in the system.

14.6 Inclinometers

Two types of inclinometer were used at Selborne, essentially for two different monitoring roles. The inclinometer access tubes, read manually using a torpedo, were intended to measure (small) early strains, the torpedo being able to detect displacements of as little as 0.1mm. The string inclinometers were intended to monitor the later (larger) strains with continuous monitoring through failure and collapse. The electronically logged string inclinometers worked to a lower accuracy than the manually read torpedo, reading to only 1mm, but had the advantage of being able to provide real-time rates for displacement and acceleration of the movements.

The lower reading accuracy of the string inclinometers was due to their construction, which utilised an electrical rheostat rather than the more accurate servo-accelerometers within the conventional inclinometer torpedo.

14.6.1 Fixity

All the inclinometer access tubes were seated deep into the unweathered Gault Clay to provide a "fixed length" at their base. ISRM guidelines (ISRM 1977) suggest a minimum fixed length of 3.0 metres. Figures 14-15 to 14-26 show that this fixed length was achieved for all the inclinometers at Selborne. Records for the string inclinometers, Figures 14-27 and 14-28, also show that both the string inclinometers were fixed for at least 2.0 metres at their base, except for the last reading on SI01. By the time of the last reading, the string inclinometer access tube had deformed enough to move the whole inclinometer string slightly. Therefore there appears to be movements below about 8 metres when in fact it can be shown by Inclinometer 04 (Figure 14-18) which was situated close by, that no movement took place below about 9.0 metres depth.

The fixed lengths also show the stability of the torpedo itself. Profiles for all twelve inclinometers within their stable zone show no movements greater than the reading tolerance of the torpedo, 0.1mm, confirming the regular laboratory checks made on the inclinometer and also suggesting that the small order displacements discussed later in Section 16.2 can unquestionably be taken as "real" movements.

14.6.2 Movement in relation to position of slip surface

Post failure excavations exposed three inclinometer access tubes and in each case the exposed tube was clearly aligned along the slip surface. Plate 14-1 shows Inclinometer 07, the access tube is laid flat along the slip surface and the empty grout stack is still visible towards the upper left of the plate.

The inclinometer elements showing maximum displacements over a 0.5 metre reading length are illustrated in Figure 14-29. At first sight the elements showing maximum displacement do not appear to correspond with the projected centreline slip surface profile shown on Figure 14-29. However, if allowance is made for the lateral dip of the slip surface (see Section 15.2) and the inclinometers projected onto an adjusted centreline slip surface profile, Figure 14-30, then it can be seen that the inclinometer elements showing maximum movement are in fact in very close accordance with the final slip surface position.

Plates 14-2 and 14-3 show two of the three inclinometers excavated post collapse. Both tubes are exactly aligned along the slip surface. Plate 14-2 clearly shows how there was little sympathetic displacement of the inclinometer tube below the slip surface. Above the slip surface the plastic access tube has been pulled through the grout stack to be laid flat along the slip surface, the empty grout stack still visible and vertical about four metres down slope. Inspection of the inclinometer tubes dug out during the post-failure investigations showed that some of the inclinometers had come apart during failure (or more likely during collapse) at one length of inclinometer tube above the slip surface . All the breaks were at joints, the collar being left in the grout and the lower section of tube having been pulled out. The authors view is that the more flexible access tube deformed more than the rigid collar therefore breaking the pop-rivets holding the two together. By having broken and slipped through the grout the inclinometer has actually deformed with the slope. Had the collar connection been more robust then the soil may well have "hung up" around the inclinometer tube, as may have happened around the north corner which possibly hung up around the wire extensometer weight liners (see Section 16.4).

It is also the authors opinion that the access tubes did not come apart until quite late in the deformation process, i.e. deformations will make the tubes become unreadable a long time before they come apart.

14.6.3 String Inclinometers

The string inclinometers were very successful in their operation but only partially successful in their location. Originally three inclinometer strings had been proposed to be placed at the top, middle and towards the toe of the slope, but due to cost constraints, the string inclinometers being the single most expensive instrument employed at Selborne, only two were installed. One was installed on bench four and the other 1.5 metres back from the slope crest. The upper string inclinometer was more or less on the upper graben of the slip surface and during collapse the soil actually fell away from the outer tube leaving it "hanging free" as illustrated by Plate 14-4. The lower inclinometer string was situated only just upslope of where the slip surface emerged from the slope face, therefore only two out of the eight elements were actually recording any movements. A third string inclinometer mid-way up the slope could have provided some very interesting data about the rates and sequence of movements during collapse.

14.7 Wire Extensometers

Although initial inspection of the wire extensometer results was not encouraging, closer inspection shows that their performance, especially during the early stages of collapse, was quite successful. The whole system was very prone to mechanical disruption due to problems such as the tensioning weights jamming in the drop wells, "trips" and permanent distortion of the invar wires by stray cattle (and site staff) and the plastic sheeting covering the slope occasionally breaking free and wrapping around the wires.

The detailed site diaries kept throughout the project, log all disruptions and adjustments made to the wire extensometers by the site staff from installation through to collapse. Careful inspection of results and cross-calibration with the information in the site diaries allows an initial output such as Figure 14-31 to be refined to the plot shown in Figure 14-32.

Over short periods of time where there is confidence of the "end points" the wire extensometers provide a very good record of the rates of movement.

One problem noted with the instruments especially as they approached collapse, involved tilting of the anchors. The anchors all had about one metre of rebar exposed above ground level as shown by Plate 14-5. The complete anchor and concrete block units on the slope face were observed to disturb backwards (toward slope crest) i.e. in the direction of the applied force from the invar wire. Whether this tilting is a true reflection of movements within the slope or due to cyclic loading, say due to wind effects transmitted to the invar wire, is unclear, but the phenomenon was only observed to occur on the anchors actually on the slope face.

Unfortunately the magnitude of the displacements due to tilting are likely to be of a similar magnitude to the early strains within the slope thereby possibly invalidating the exact values of the early data for the shorter length extensometers. The longer wire extensometers all remained vertical, but they did have either deeper footings or footings into more competent material. Even allowing for the error in the wire extensometers on the slope due to tilt, there are still eight reliable monitoring positions.

The transducers used for measuring the movements were circular thyristers which each had a blind spot for about 14% of their circumference. At large displacements therefore it was possible for readings to go "off the scale". To avoid the problem of the blind spot the transducers operated via a gear system such that one revolution of the transducer equalled either 0.2, 1.0 or 3.0 metres of extensometer movement. In this way some match of expected movement to instrument range could be made. The ranges and positions of the wire extensometers are illustrated in Figure 14-33. During collapse some of the wire extensometers still went off the scale and a number went right through the blind spot and came back onto the scale again.

15. POST FAILURE OBSERVATIONS

Following slope collapse on day 196 (16th July 1989) a post-failure dig out was initiated to try and gain as much knowledge about the failure as was plausible within the few remaining weeks of site possession.

The first and primary objective was to try and locate the slip surface within the slip mass. The instrumentation obviously provided a good indication of where the slip surface was likely to be found. Starting at the base of the slope a trench was dug into the slope along the line of inclinometer 10 (which had not moved significantly but had been covered by the slipmass runout) and inclinometer 07, this trench was termed Trench 1. Careful excavation revealed inclinometer 07 laid flat along the slip surface as illustrated by Plate 15-1. The slip surface was then traced from inclinometer 07 up slope as far as possible. This process took about three days with the slip surface becoming more evident with a little surface drying on the second and third days. Once located the slip surface was clearly marked and photographed, following this the trench was widened and battered back to about 1.5m above the slip surface to allow safe working (see Plate 15-2). Once battered back the slip surface was accurately surveyed and a series of 300mm diameter samples very carefully obtained by pushing sample tubes through the slip surface using the excavator bucket as shown by Plate 15-3.

Once widened to allow safe working Trench 1 intercepted the southern isolating trench and the opportunity was taken to obtain a series of six 150mm diameter push samples of the "undisturbed" material on the outside of the isolating trench for comparison with the rotary drilled cores.

Following completion of the Trench 1 investigations, another trench, Trench 2 was begun along an approximate centreline of the slope, targeting inclinometers 09 and 06 as slip surface locators. Both inclinometer tubes were again found to be laid flat along the slip surface (Plates 15-4 and 15-5). The same processes as in Trench 1 were again followed, the slip surface was fully located on both sides of the trench, marked and photographed, trench sides battered back to allow safe working, the slip surface fully surveyed and another three 300mm diameter push samples taken through the slip surface for laboratory testing at Warwick University.

15.1 Nature of Slip Surface

The nature of the slip surface can be divided into three broad categories. Towards the lower part of the slope in the less fissured zone of the highly plastic Gault Clay the slip surface was a single, highly polished and strongly striated slickensided surface. The slip surface in this region is considered to have formed along a preferred bedding plane, with no zone of shear either side of the actual slip surface.

The slip surface in the upper section of the slope which passed through the structureless solifluction deposits and across the bedding of the upper weathered Gault was much rougher. There was little or no polishing but striations were still clearly visible. The actual disturbed region was greater than in the lower section with a zone of shearing of approximately 2mm.

In between these two zones, in the area of maximum internal shearing and minimum radius of curvature, the slip surface was found to pass through a wide (20mm) zone of disturbed material. The parent material in this area was the lower weathered Gault Clay. The disturbed zone had a brown reworked clay matrix with grey relicts in the upper part and a grey matrix with grey and some brown relicts towards the base. Two 300mm diameter push samples were taken through this "gouge" zone during the post-failure dig out in Trench 2, for examination and testing at Warwick University.

These three categories are found to be in close agreement with the divisions given to compound slides by Barton (1984). A steep rearward slip, almost certainly having formed along fissure surfaces, a tight middle section and a lower translational zone of sliding approximately parallel to the bedding. Barton notes that material in the intermediate section of a compound slide, where the slip surface has a sharp radius of curvature, will have undergone intense deformation and ultimately complete remoulding of the clay. Exposures of this part of the slip surfaces are noted to be rare. The "gouge" zone observed at Selborne provides good evidence for these processes the material having been highly disturbed comprising clay relicts in a reworked clay matrix.

Plates 15-6, 15-7 and 15-8 show the nature of the slip surface in the three zones.

15.2 Slip Surface Cross Fall

The detailed survey data for the slip surface position in each trench is presented as Figure 15-1. It is clear that there was a south-north cross fall on the slip surface. To allow accurate comparisons of instrument data the slip surface and instrument elevations can be projected onto an interpolated centreline profile. Figure 15-2 shows the interpolated centreline slip surface profile, the original cut slope profile and the post collapse centreline surface profile. Instrument positions have been adjusted to a centreline profile by altering the instrument elevation by a value equivalent to the difference in projected slip surface elevation at the instrument location to that at the centreline. Figures 14-29 an 14-30 illustrate the process for the critical inclinometer elements, similar adjustments have been made for piezometer locations.

The north south variation in slip surface elevation, and the geological influences potentially causing it have been addressed by Barton and Cooper (in prep). They report on a joint survey carried out at various locations within the clay pit and within the post failure investigation trenches. They conclude that the nature of fissuring did have an effect on the shape of the slip surface, and on the direction in which the slip mass moved.

16. ANALYSIS

Throughout this Section, for ease of comparison and presentation of results, dates are referred to by a day number. Day zero represents the 1st January 1989. Slope formation therefore took place between days -500 and -460, pore pressure recharge began on day 02 and slope collapse took place on day 196.

16.1 Development of Failure

The displacement-time curves for the critical elements of the manually read inclinometer access tubes affected by the failure are presented as Figure 16-1, where critical element refers to the 0.5 metre reading length including the slip surface. This diagram shows that movements were developing immediately following slope formation. Inclinometer 09, one bench up from the slope base, experienced significant displacements (10mm) before day -300. Inclinometers 08 and 07 were moving similarly but at a slower pace. The mechanisms involved with the development of these early strains are not necessarily concerned with the ensuing slope instability, but rather with stress relief.

Unloading during slope formation will have lead to a release of in-situ stresses within the soil, the strains resulting from this relief concentrated along bedding surfaces (see Appendix J). These surfaces subsequently become the focus of movement once shear resistance has been overcome, with shear resistance being exceeded earlier along the lower surfaces than higher up in the slope. The greatest resistance to shear being in the zone of maximum curvature, where the shearing forces act at the maximum angle to the fissure surfaces. Also this zone is farthest from the slope surface and therefore will have undergone only minor stress relief due to unloading. During this period inclinometers 06 and 05, towards the middle of the slope, appear to be relatively stable, showing negligible movements for the first 400 days. Even during the early stages of recharge (Stages 1 to 3) these two inclinometers remained relatively stable, not until about day 75 did they begin to show any sizeable displacements.

Displacements were therefore first becoming apparent at the base of the slope, as would be expected (Skempton and La Rochelle 1965, De Beer 1969, Garret and Wale 1989), the base being the point of highest stress concentration, but not due to slope failure but due to relief of in-situ stresses along the low angle fissure or bedding surfaces. Strains due to this mechanism would be expected to be less further from the slope face since stress release would be greatest at the toe of the slope.

Inclinometer 04 at the crest of the slope was subject to a small local instability following the Great Storm of day -443 which affected the readings for the upper two metres or so by leaving the concrete surround to the inclinometer cover loose in the surrounding soil. The critical element at 6.0 metres depth was showing very small movements taking place from day -300 (not evident in Figure 16-18 due to the scale). Therefore early strains were also being noticed at the top of the slope.

Movements were therefore being noticed at both the top and the base of the slope with little or no movement in between, a possible indication of progressive failure as described by Bishop (1967). Bishop suggested that in a long term analysis, where effective stress parameters are operating that a progressive failure mechanism may be expected to begin at the surface, both toe and crest, and propagate inwards into the soil mass. He also notes that failure will usually take place first where normal stress is least i.e where the slip surface is emerging at a surface, but that this could be complicated by the previous stress history of the slope prior to cutting or toe erosion. The driving influence for the progressive failure mechanism operating in this case is not necessarily the same at both slope crest and toe. In this case it has previously been suggested that the early movements towards the base of the slope were more likely due to the process described by Bjerrum (1967) whereby high horizontal stresses with a reduced overburden lead to shear failure and stress relief. The movements seen in Inclinometer 04 at the slope crest are considered as more likely being due to a similar mechanism which was self limiting in extent due to a rapidly increasing vertical shear stress, or possibly due to localised high pore pressures. These very early movements can be taken as an indication of progressive failure even although the same processes are not necessarily acting at toe and crest.

Later movements, even once recharge had reached Stages 1 and 2, still showed the same pattern of displacements. Movements were still concentrated at the crest and towards the toe of the slope with very little movement in between. These movements being concentrated on the same bedding surfaces exhibiting the greatest strains due to stress relief.

These later movements which all share a common causation, can be truly considered as evidence for the operation of a progressive failure mechanism; strain softening leading to mobilized shear strengths being at varying values between a peak and a residual value, dependent upon position within the slip surface.

The observations of slope movement made here appear to contradict the findings of De Beer (1969) for overconsolidated clays in Belgium and of Garret and Wale (1985) for Gault Clay within the UK. Both these authors found slope movements began at the toe of a slide and worked progressively backwards towards the slope crest. De Beer states that the distortions along potential slip surfaces start at the toe progressing gradually upwards, the formation of a tension crack representing a considerable time lag from displacements at the toe. Tension cracks were found by De Beer only to occur in slopes greater than 2:1 (vertical:horizontal), but this is largely due to the fact that slopes cut shallower than this in his study did not rupture (collapse). From De Beer's work it can be inferred that tension cracks represent a sign of incipient collapse and not, as often considered, an initial indication of slope instability.

Closer inspection of De Beers findings shows that movements were occurring at the top of the slope and that these movements were observed to begin only a matter of days after movements were first noted at the toe. Displacements at the slope crest were far smaller than those being observed at the toe, and this is the point that De Beer is trying to make, i.e. that the *largest* displacements occur first at the toe, not that movements are not occurring at all at the slope crest. Also it must be noted that results are not given by De Beer for inclinometer installations in the upper half of the slope, data are presented only for inclinometers installed at the crest and towards the toe of the slopes. Therefore no conclusions can be drawn with regard to the order of the movements in this region. Were displacements at mid-slope moving in advance of or after movements first becoming apparent at the slope crest ?

In this respect the findings of the Selborne study are in agreement with the ideas put forward by Bishop and in broad agreement with the findings of De Beer. Noticeable movements were first occurring at the toe of the slide and later at the slope crest. The middle region of the slope only beginning to move once upper and lower portions of the slip surface had become well established.

Figure 16-2 shows the interpolated pore water pressure distributions along the eventual slip surface at day numbers 136, 158 and 188. The pore water pressure on the slip surface does not appear to increase despite there being known increases in the recharge head, but there are accelerations in the displacement of the slip mass with increases of recharge head (Figure 16-1). It appears that the increased recharge pressures are being rendered ineffective by reducing shear strengths and subsequent increasing movements along the slip surface leading to greater dilation and hence no net increase in the measured piezometric heads. Figure 16-3 shows the pressure heads for piezometers V14, V17, V18 and V19, which are all close to the slip surface, in the days leading to slope collapse. Points P and Q mark rises in the recharge head over this period. It can clearly be seen that the pressure heads fall despite the increase in applied head, the fall in pressure head being matched by an acceleration of the slip mass (Figure 16-1).

It is notable that the greatest reduction in pore water pressure takes place in piezometers V17 and V18. These two piezometers are located in the zone which, due to its geometrical position, will suffer the greatest degree of internal shearing during slope collapse. It has already been seen from inclinometers 05 and 06 (Figure 16-1) that displacements in this region did not occur until comparatively late in the failure sequence. The combination of these two effects, a high degree of shearing in a relatively short time period, would be expected to produce a large amount of dilation and consequently a large degree of pore water pressure reduction - as is the case.

The pore pressure reductions could also have possibly been due to a loss of driving head. This theory can be discounted since it is known that the piezometer readings within the recharge zone remained "high" right up until slope collapse, and also that the recharge wells examined in the post-failure digout were found to be intact despite being severely distorted (Plate 11-3).

Figure 16-4 presents the increase in the average pore pressure ratio (r_u) within the recharge zone over the last 20 days before slope collapse and the corresponding reduction in r_u along the slip surface, the two lines illustrating the direct response of pore water pressure to dilation along the slip surface.

16.2 Basal Slips

Figure 16-5a and 16-5b show the records for inclinometers 01 to 05 situated towards the upper part of the slope and behind the slope crest. It can be seen that below the main failure there were significant movements along a line extended from the eventual basal failure plane. These movements decrease in magnitude with distance from the slope face. The very small values of movement in inclinometers 01 and 02, less than 0.8 mm, although close to the reading accuracy of the torpedo are considered to be genuine as discussed in Section 14.5. The magnitude of movement decreases with increasing remoteness from the slope face suggesting that the mechanism involved in the creation of this surface is recoverable strain energy release. This energy is released as the soil expands following stress relief during slope formation, the amount of stress relief and consequently the amount of strain energy release decreasing with increasing distance from the slope face. This mechanism has been identified and discussed by Burland et al at Saxon Pit (Burland, Longworth and Moore 1977). In the Saxon Pit study block movements within the shear band were noted to extend back from the excavation face for about the same distance as the excavation depth, and total horizontal surface movements were noted at distances of up to 2½ times the excavation depth.

At Selborne horizontal displacements were only recorded at distances of up to excavation depth from the slope crest but this was largely due to there being no accurate monitoring points beyond this. Also the slope face at Selborne was a nominal 1:2 compared to the near vertical face at the Saxon Pit site. The information from the inclinometers at Selborne shows that the material was not moving as a block beyond the slope crest but that displacements were diminishing with increasing distance from the crest indicating that the material behind the slope crest was probably in what is termed the "end of region" by Palmer and Rice (1973). The "end of region" being the zone where the shear strength is decreasing from a peak to a residual value (Figure 16-6).

16.3 Shape of Failure

The effect of the recharge system has been described previously, in that accelerations of the slip mass can be directly related to rises in recharge head (Figures 16-1 and 16-4), but to what extent did the recharge system control the actual mechanism and shape of the failure? As detailed in Section 11 the recharge system was set far back and deep within the slope to try to instigate a deep seated failure - as opposed to a shallow surface failure. The shallow failure was in fact averted by covering the slope face in PVC sheeting.

Before recharge even began there had been significant movements of three inclinometers, mainly towards the base of the slope. These movements were all on what eventually became the slip surface. The effect of the recharge appears to have been a time related one in that it acted as a catalyst speeding up the natural processes operating, the geological influences having had a more prominent affect on the actual failure shape than the pore pressure regime. Whether or not the slope would actually have failed without the affect of the recharge is unknown. The early strains and associated stress distributions occurring after slope formation may have brought the slope into a state of equilibrium.

Figure 16-7 shows the centreline slip surface with the geological section superimposed and also the zone of "negative" reading piezometers outlined. The initially circular slip surface at the rear of the slope is clearly modified to a compound slide at the boundary of weathered to unweathered Gault Clay, the slip surface then follows this boundary to the slope face. The slip surface is also seen to be resting just above the zone of negative pore water pressures toward the base of the slope. The clay in this area will have had artificially high effective shear strengths until the pore pressures were able to reequilibrate. The fact that these negative pore water pressures exist is of course a geological phenomenon in that only the less fissured unweathered Gault Clay can sustain these negative pore pressures for such a length of time.

16.4 Surface Movements

Figure 16-8 shows the (surveyed) surface movement vectors for all the inclinometers and the forward wire extensometer anchors and frames for three pre-collapse and one post collapse surveys. The vectors indicate a "fan" of movement (Figure 16-9) pivoting towards the north-west corner of the slope. This corner of the slope was noted to hang up during final slope collapse (Plate 16-1).

The bedding crossfall, as shown by the lower part of the slip surface, and the pattern of jointing suggested that a north easterly movement of material was likely (site discussion with M. Barton, Summer 1988). The fanning of movement is most likely a result of the progress of movements. Movements initiating in the south east will have allowed material in the region of wire extensometer WE056 and WE051 to move south-eastward rather than directly eastward (downhill). Further east in the region of WE058 and string inclinometer SI02 material was not constrained since it was close to the emergence of the slip surface. Material in this zone was therefore free to move in the direction of the bedding and jointing i.e. north-easterly.

16.5 Influence of Side Shear Breaks

Figure 16-10 shows the profile of the side shear break's and the two surveyed slip surface sections. SS1 which is only 2.5 metres from the southern side shear break is above the lower level of the isolating trench, the slip therefore having been contained within the isolated zone illustrating that the side shear break was successful and did not significantly affect the failure pattern in this region. SS2 which is situated just north of the slope centreline coincides with the side shear break profile at about half way down the slope, but is probably too remote from the isolating trench to have been significantly affected. If allowance is made for the lateral variation of the slip surface then the likely projected slip surface position at the northern side shear break is shown as Figure 16-11. It can be seen from this figure that the slip surface is well below the side shear break. This is of course speculative and the effect of the isolating trench may have been to modify the shape of the slip surface upwards, but since it has already been shown that a basal slip was forming well in advance of side shear break construction and that the lower section of the slip surface is considered to be preferred bedding plane controlled this is thought unlikely.

The slip surface emerging below the northern side shear break will have created additional "friction" increasing the resistance to sliding , and especially resisting any acceleration in the rate of sliding. Plate 16-1 shows the slope post-collapse. It can be seen that the north west corner hung up during collapse but also that the northern end generally moved less than the southern end. The effect of the northern isolating trench not having been quite deep enough appears to have retarded the rate of failure but not, apparently, to have affected its shape.

The depth of the side shear breaks was very much geologically controlled in that they were dug as deep as possible into the hard clay as the power of the Poclain VC15 excavator would allow. It is the authors belief that the combined influence of the bedding and the northern side shear break has been to provide a fan of failure pivoting towards the north west corner (see Section 16.4). If it had been possible, then to have formed the slope normal to the bedding may have been appropriate. This would have required a detailed joint survey as either part of the feasibility studies, for which money had not been available, or for a study at the start of the project, for which time was not available. To have formed the slope normal to the bedding would also have occupied a larger part of the pit which would have interfered with commercial activities at the pit.

16.6 Janbu Analysis

Although a full numerical analysis of the slope failure is not within the scope of this thesis a number of computer analyses using a commercial slope stability programme have been carried out. The program used was "SLOPE" version 8.2 licensed by Geosolve Limited (Borin 1993). Analyses were carried out using a Janbu analysis with parallel inclined interslice forces. This method of analysis satisfies conditions of horizontal, vertical and moment equilibrium for the slipped mass as a whole. By assuming that all the interslice forces are parallel, but not necessarily horizontal, the analysis is able to calculate the inclination of the interslice forces which allows all conditions of equilibrium to be satisfied simultaneously. This method has been shown by Spencer (1967) to be more applicable to steeper slopes but can be subject to the "interlock problem" (see Appendix K). "SLOPE" prints a warning if the calculated FoS is likely to be in error due to this problem. Summaries of all the analyses and input data are presented as Appendix L.

The section analyzed is shown on Figure 16-12. Restrictions within the program allow only a maximum of 30 data points to define both the strata and slip surface profiles. In order that as many points as possible could be used to give an accurate representation of the slip surface the surface profile of the slope was reduced to the straight line shown on Figure 16-12. Soil parameters and geological profile are as given in Section 6.1.

Analyses were carried out for 11 dates ranging from day -118 to day 194 i.e. from well before recharge commenced up to just before final slope collapse. Line 'A' on Figure 16-13 represents the results of these analysis in graphical form. Note at no point does the analysis give a factor of safety (FoS) less than 1.0, that is, according to these analyses the slope never actually failed! This is largely as a result of the method used to represent pore water pressures in the "SLOPE" program. In this instance the pore water pressures have been entered as a grid of piezometric levels as shown in Figure 16-14, the pore water pressure acting on the slip surface being interpolated from the four nearest grid positions. The negative pore water pressures in the region of pore pressure grid coordinates (6,5) and (7,5) affect the analysis significantly by reducing the interpolated pore water pressure acting on the basal part of the slip surface. If, however, a Janbu analysis is carried out using a phreatic surface calculated to represent the pore water pressures known to be acting on the slip surface then the result shown by line 'B' on Figure 16-13 is produced. The difference in FoS between the two methods is simply a function of the assumptions made and the way in which the program interprets the piezometer information used to evaluate the pore pressure acting at the slip surface. The danger in using a slope stability program without an understanding of the mechanisms involved becomes clear.

Figure 16-13 also shows an apparent increase in FoS from day 180 to day 194. This is as a result of the of pore water pressure reduction that took place as failure progressed, the dissipation of pore water pressure being due to dilation as the soil sheared during failure (see Section 16.1). Also as mentioned above by day 194 the soil parameters used in the analysis are clearly inappropriate. An analysis carried out for day 194 (just prior to collapse but post what could be termed engineering failure) using residual strength parameters gave an FoS of about 0.6, shown by the dashed lines on Figure 16-13.

The calculated factors of safety, although not giving an empirical value for the status of the slope at any one time, give a good idea of the progress of the slope towards failure.

17. CONCLUSIONS

The Selborne Slope Study has provided a detailed case history showing the conditions within a slope through the transition from a stable to an unstable state, to and beyond collapse. This Thesis has outlined the organisation of the project, the instrumentation and control systems employed on the project and detailed the results and findings of the project. Information has been provided with the aim of allowing further work to be carried out using this data to evaluate the effectiveness of current slope stability analysis techniques, and also to aid in the refinement and development of new techniques. It is hoped that the project has provided field and laboratory data suitable for application in finite element research work.

Probably the most successful, and in terms of future projects a very important, aspect of the Selborne Slope Study was the recharge system. The system worked remarkably well, allowing the slopes progress from a stable to an unstable condition to be carefully controlled. The project has shown that the pore water pressures within a slope can be controlled, and consequently the rate of failure itself regulated in a cheap, relatively simple and almost infinitesimally adjustable manner.

A pore pressure "control" system such as this may well be applicable to other branches of geotechnical engineering where control of insitu (effective) stresses or known stress conditions for comparisons in instrumentation behaviour are required.

17.1 Instrumentation

It has been shown (by this project) that instrumentation can be installed to a high level of reliability, and that reliability maintained for two years. The instrumentation reliability at Selborne has been assessed as 98.2% per annum. Even during this project with an absence of other engineering works great care was required to prevent damage to the instrumentation by site activities. On a site where construction activity and not instrumentation installation is the main focus of work then the chances of instruments remaining undamaged is much reduced, contractual arrangements have been shown to help reduce the chance of damage to instrumentation by site activities (Davies et al 1989).

A need for future work on the effect of instrument installation procedures has been demonstrated. Specifications for installation do exist, for example ISRM (1977), BS5930 (1989). Research as to how well these specifications are complied with, and to what extent non-compliance affects subsequent readings is required. Inclinometer groove roll and piezometric lag times are examples of important features which should be considered. It may be that the specifications and typical industry procedures, as they exist, are suitable for the general engineering purposes to which data are put.

However, more exacting specifications may be required for certain instrument applications and possibly also for uses such as research.

Groove roll has been shown to represent a potentially significant problem. It is recommended that the effects of groove roll be considered when an instrumentation design is being formulated. If groove roll is likely to be significant in the context within which the instrumentation system is to be used, then readings of groove roll should be made following installation. To assess whether groove roll is likely to be significant there is a need to know typical values. Inclinometer access tube manufacturers and suppliers have been contacted and they note that keyway spiral is within 1° per 3m. However, they do not have, or take measurements to confirm this. A study investigating typical values of groove roll in both manufactured access tube and completed inclinometer installations would be of interest.

Piezometers have been shown by this project to react rapidly to changes in stress conditions and for there to be a good repeatability between vibrating wire piezometer and pneumatic piezometer readings. It was noted, however, that both types of piezometer required a considerable time post installation to equilibrate with the pre-existing pore pressure regime. This post installation lag time was found to be typically 55-60 days, regardless of depth of installation or eventual equilibrated pore water pressure. This represents a considerable period of time, especially in terms of standard construction activities.

The displacement measuring instruments, inclinometers, string inclinometers and wire extensometers were noted to perform extremely well but were found to be sensitive to location in terms of the usefulness of their data. For example both the string inclinometers functioned well, but were largely unused due to their positioning in relation to the eventual location of the slip surface. Standard inclinometer access tubes were in plentiful enough supply to give good data, but there is an anomaly between the horizontal to vertical reading accuracy. Lateral displacements are recorded to one tenth of a millimetre where as vertical readings are at half metre intervals. The amount of movement is therefore very accurately portrayed but its exact elevation is less clear. This study has shown that the inclinometer access tubes sheared at exactly the intersection with the slip surface. There was little or no sympathetic movement of the tube below the shear surface. A system of carefully machined collars fitting the top of the inclinometer access tube could be employed to allow readings to be taken at intervals of 0.1 metres, or less, over what is seen to be the key movement length. This would allow a more accurate definition of the potential slip surface, a drift in slip surface location of up to 0.5 metres is likely to make a significant difference to any slope stability analysis.

Another problem noted with the displacement measuring instruments, with the exception of the manually read inclinometers, was that random errors within the instrumentation were of the same order of magnitude as early creep strains. The wire extensometers and string inclinometers did not therefore really come into their own until collapse was already in progress. More careful detailing of the instrument design, gained from the experience from this project, combined with the recent advances in electronic technologies, would certainly allow newly manufactured instruments to perform to a higher resolution thereby recording smaller displacements.

The wire extensometers particularly, suffered numerous teething problems and their detailed design needs refinement before use on a future project. The basic concept, however, is sound and they did provide valuable information about rates of movement and about the order in which parts of the slope were moving during the latter stages of failure.

The use of Kevlar rather than invar wire for the wire extensometer strings may have made them more user friendly leading to less errors due to kinking of the wires and permanent distortions due to the plastic sheeting wrapping around the wires. The lighter kevlar, which is of only a slightly higher thermal expansion coefficient, would have required less weight to keep the string taut, again simplifying their construction and use.

It was noted during the final stages of slope collapse that the cables to the vibrating wire instruments snapped with an instantaneous loss of signal, where as the pneumatic piezometer lines tended to stretch. It is the author's opinion that a form of basic, cheap, logged extensometer could be developed using a simple signal cable that snapped at a certain tension or elongation. A number of short lengths of cable could be installed in slit trenches and logged to provide information on the order of movements within a slope. This type of instrument could also be useful for monitoring temporary slopes for construction or where a "green light go, red light stop" type safety system were required.

17.2 Failure Development Pattern

The development of the failure, that is the position of the slip surface, at Selborne has been shown to have been more geologically than either geometrically or hydrogeologically controlled. The increasing of the pore water pressures clearly allowed failure to take place but its position was governed by purely geological factors. This is evidenced by the fact that the lower part of the failure surface was forming well in advance of either side shear break construction or recharge commencing.

Analysis of the instrumentation results has shown that movements along the eventual slip surface were first being noted at the toe of the slope. Further movements occurred at the toe and at the crest, with movement only taking place in the mid-section just prior to failure.

This *bottom-top-middle* pattern of movement fits with the ideas of a progressive failure mechanism, as noted by Bishop (1967), having been in operation.

It is also worth noting that no tension cracks were observed at the slope crest, even 24 hours before final rupture and collapse. This compares with De Beers (1969) findings where he noted tension cracks as a sign of incipient failure on slopes of than 1 in 2 or greater.

Analysis of the piezometric records has clearly shown a decrease in measured pore water pressure around the slip surface despite increased pressures within the recharge zone. The decrease in pore pressure along the slip surface has also been shown to correspond to increased movements of the slip mass. This is believed to be clear evidence of dilatancy occurring along the developing slip surface. It was noted that the greatest reductions in pore water pressure were measured in the zone of greatest dilation potential, that is, in the tight middle section of the slip surface.

Evidence gained during the study indicated that a basal slip plane had formed extending for a considerable distance behind the slope face prior to either recharge commencing or the upper part of the slip surface forming. Displacements along the basal slip surface were noted to decrease with increasing remoteness from the slope face and as such the findings of this project are in broad agreement with the findings of the Saxon Pit study (Burland et al 1977). The basal slip at Selborne was observed to extend back from the slope face for a distance approximately equal to the slope height. It would have been interesting to excavate, sample and investigate a portion of the basal slip plane during the dig out operation following slope collapse. However, the dig out was carried out under very strict time constraints and a considerable amount of excavation and hence time would have been required to expose the basal slip plane and create a safe working platform for sampling and investigation.

Pore water pressures below the toe of the slope were observed to drop to below atmospheric pressure following slope formation. A system to read these "negative" pressures using the pneumatic piezometers was developed. It is believed that the pressures remained negative throughout the two year duration of the fieldwork.

It has been shown that the geology of the site had the overriding control on the failure pattern, with areas of the slip surface forming prior to commencement of recharge. Human intervention appears to have had a time related affect only. Whether the slope would ever actually have collapsed without the application of recharge is unclear.

Given sufficient time natural fluctuation in groundwater combined with time related softening processes may eventually have led the slope reaching a state of engineering failure .

17.3 Considerations for Future Studies

The Selborne Slope Study was generally very successful in achieving its stated aims, but as with all experimental projects given the chance to repeat the study certain changes could be made.

The side shear breaks were generally very successful in creating an isolated, two dimensional failure. However, there was still an element of rotation in the slide. Much of this rotation is considered to be related to the fissure pattern within the clay. To create as near a plane strain type failure as possible, it may have been advantageous to form the slope normal to the bedding.

In a future project the installation and reading of inclinometer access tubes prior to slope formation could be useful to enable the effects of stress relief to be established and discounted from early measurements of strains. This process would obviously require careful site control during excavation so as not to allow any excessive damage to the inclinometer tubes during excavation. It may also prove possible to install a limited number of piezometers to measure unloading effects.

The string inclinometers were obviously a success in their operation, but not so successful in their location. A similar project in the future would gain greatly from having three such instruments. This would have obvious cost implications, string inclinometers being the single most expensive instrument employed on the project. Cost savings could be gained by replacing non-vital vibrating wire piezometers with pneumatic piezometers. The pneumatic piezometers were shown by this project to perform almost as well as the vibrating wire piezometers at a proportion of the cost. The pneumatic piezometers also had the advantage that the pneumatic lines connecting the piezometer to the gauge house remained operable at quite major distortions

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Overall the project was a great success. It has shown that to function to its full potential a geotechnical instrumentation system requires a considerable amount of understanding with much day to day care and attention. This attention however, is rewarded by the provision of data that can accurately describe the pore pressure and displacement regime within the ground at any instant in time. The instrumentation systems used at Selborne were shown to be sensitive to relatively subtle changes of parameters and to react rapidly. The system was successful in its dual aim of acting as control mechanism for the recharge and to monitor and record the slopes progress to failure.

The post failure excavations and analysis of instrumentation data have established that a progressive failure mechanism was in operation. Failure first occurred at the toe of the slide followed by the crest and finally the mid-section. The process of dilation occurring along the developing slip surface has been clearly illustrated. The project has shown that the state of stress within a slope can be established by instrumentation data in association with suitable laboratory testing. This should allow an accurate portrayal of the status of a slope on its path to failure. Current analysis techniques are not sufficiently developed to allow this portrayal. It is hoped that the body of data gathered here will aid in the development of such analysis techniques.

ACKNOWLEDGEMENTS

The work was funded by the Science and Engineering Research Council (S.E.R.C.).

Thanks must go to Colonel J A de Benham Crosswell for generously providing the research site and allowing our use of it. Thanks must also go to the Colonel and his staff at the Selborne Brick and Tile Company for their day to day help with the project and loan of their workshops and various plant when needed.

Many thanks are extended to the project site technicians, and my colleagues during the long winter months on site, Tim Walton and later Mark Machin. The Technicians within the Department of Civil Engineering also provided a great amount of assistance, especially Steve Wake who acted as site technician during the final six weeks of the project, and Ken Yates for all his technical advise and help in organising the project. Grateful thanks are also extended to Ross Sandman and the rest of the Kingston University drilling crew for getting the project off to a good start, and later for the recovery of cores and installation of further instrumentation.

Academically thanks must go to Professor Bromhead of Kingston University, Dr Petley of Warwick University and Dr Barton for advice and discussion of various technical issues, both on site and subsequently during the data analysis. Mention must also go too to Dr Clark of the Institute of Irrigation Studies at Southampton University, who gave advice relating to spreadsheet systems for data analysis and to Rob Weeks, Chris Sweet and especially Richard Gillard, all of Geotechnical Instruments Ltd for their hard work relating to the automated logging system and their rapid response to other instrumentation problems as they arose.

Others who I would like to thank are Mike Riley and John Cross from the Department of Civil Engineering, who provided the survey; the S.E.R.C. steering committee of Professor Richard Chandler, Dr Brian Simpson and Dr Christine Cooling; and the S.E.R.C. Geotechnics Committee who provided the funds.

Finally I extend my warmest thanks to Dr Mike Cooper, who as Project Leader and PhD supervisor provided help and support throughout the project, and subsequently during data analysis and Thesis preparation.

Further information relating to this project, and a detailed set of data in electronic format is available from either Dr Cooper or myself, who can be contacted care of Dr Cooper at:

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PLATES

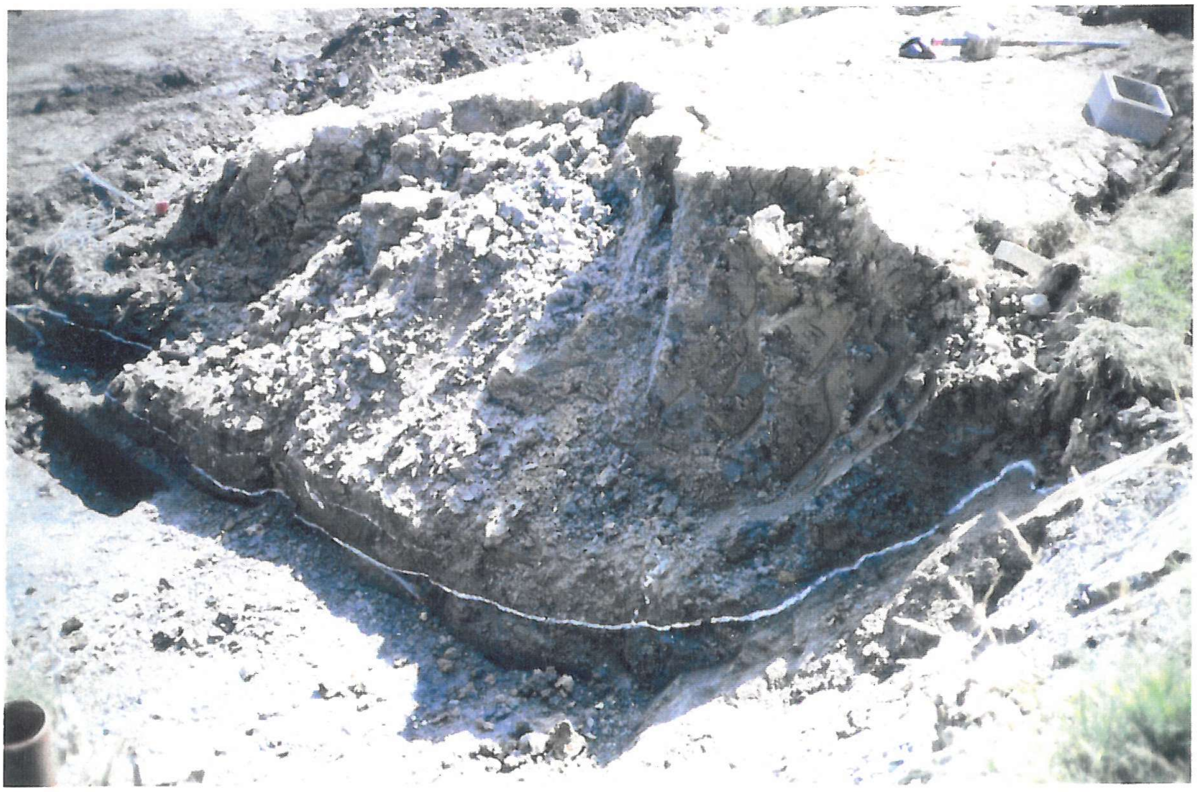
Plate 5-1. Selborne cut slope prior to failure

Plate 5-2. 40 gallon tank on gauge house roof.

Plate 5-3. Post-failure investigation trenches.

Plate 5-4. Highlighted slip surface Trench 1.

Plate 5-5. Highlighted slip surface Trench 2.



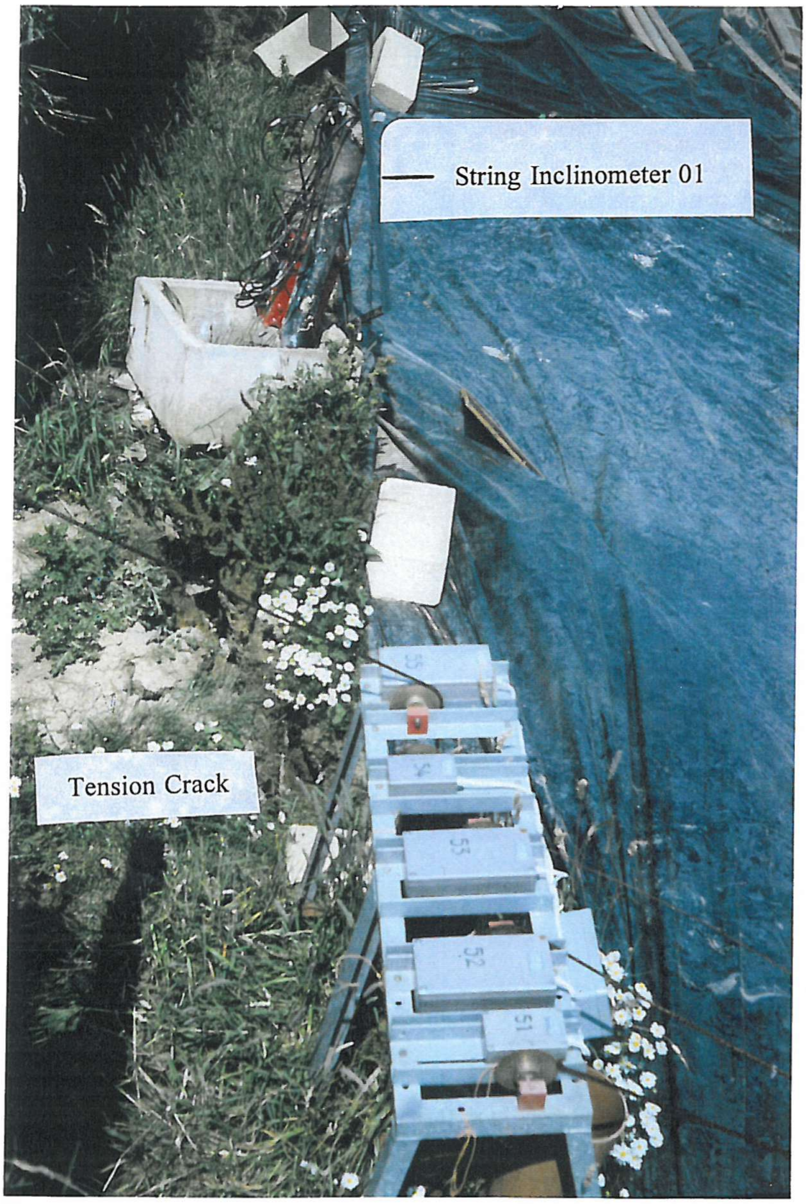


Plate 7-1. Slip surface emerging in front of String Inclinometer 01.

Plate 9-1. Piezometer installation. Measured quantity of saturated sand being placed in borehole to provide piezometer filter "pocket". Sand washed in with bucket of water to clean borehole walls.

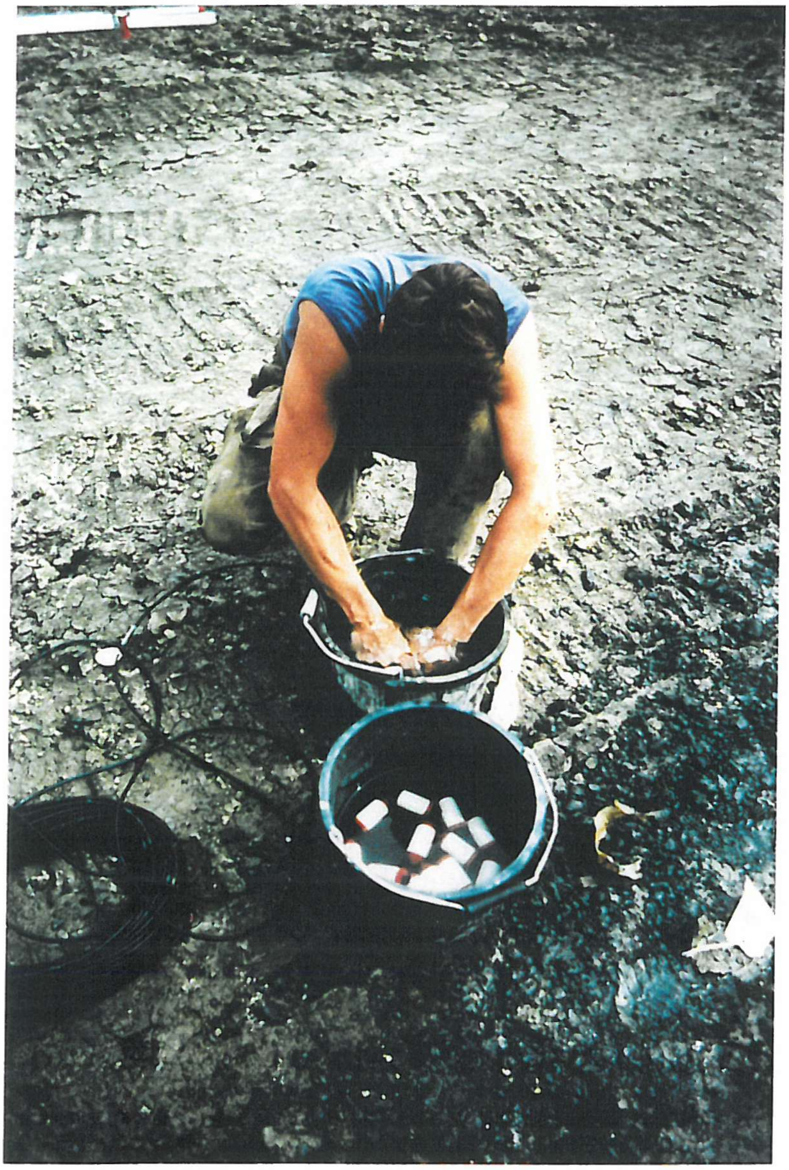


Plate 9-2. Saturated piezometer tip being assembled under water immediately prior to installation.

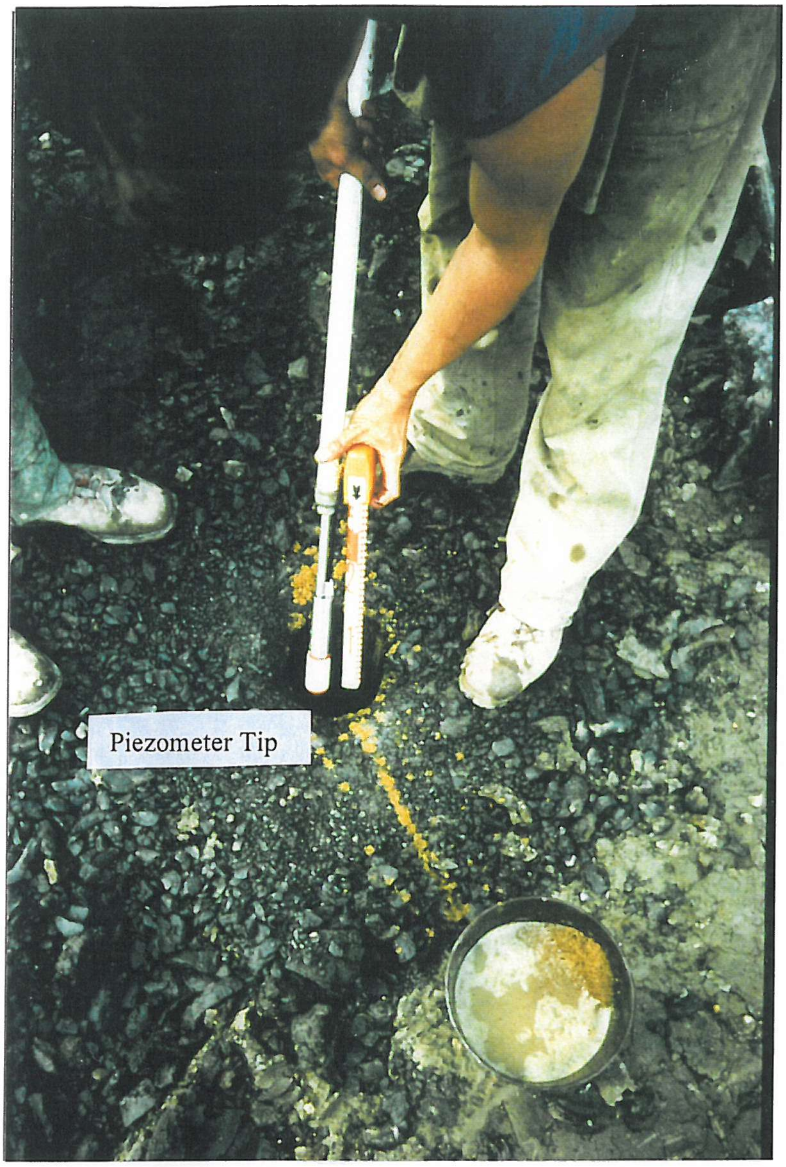


Plate 9-3. Piezometer attached to placing tool and placing rods.

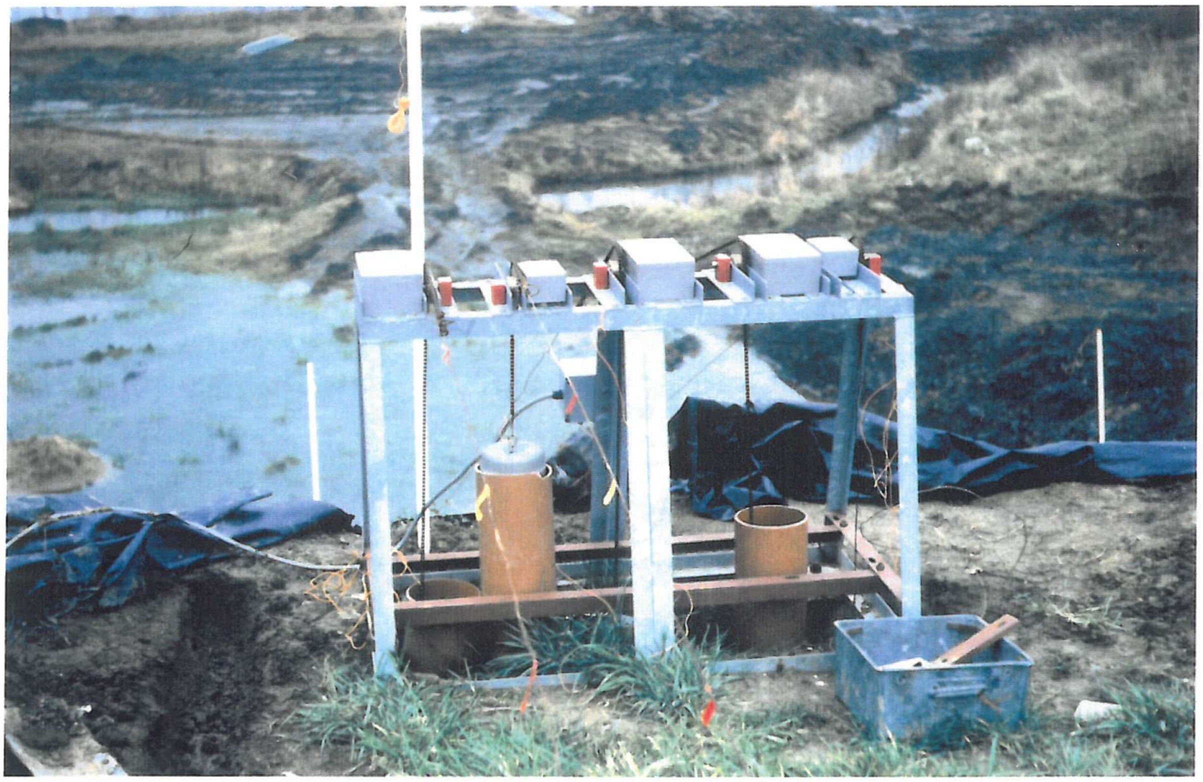
Plate 9-4. Saturated sand being placed over the in place piezometer to complete the "sand pocket" before removal of the placing tool and rods.

Plate 9-5. Slope profile following first seasons fieldwork.

Plate 9-6. Drilling and slope formation.

Plate 9-7. Drilling and slope formation.

Plate 9-8. Wire extensometer frame - Grid line 3/4.



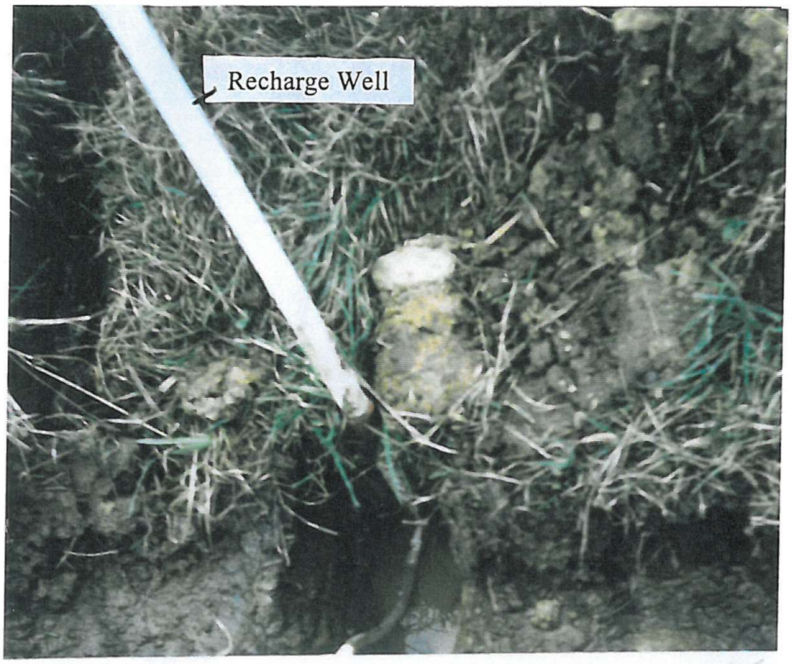
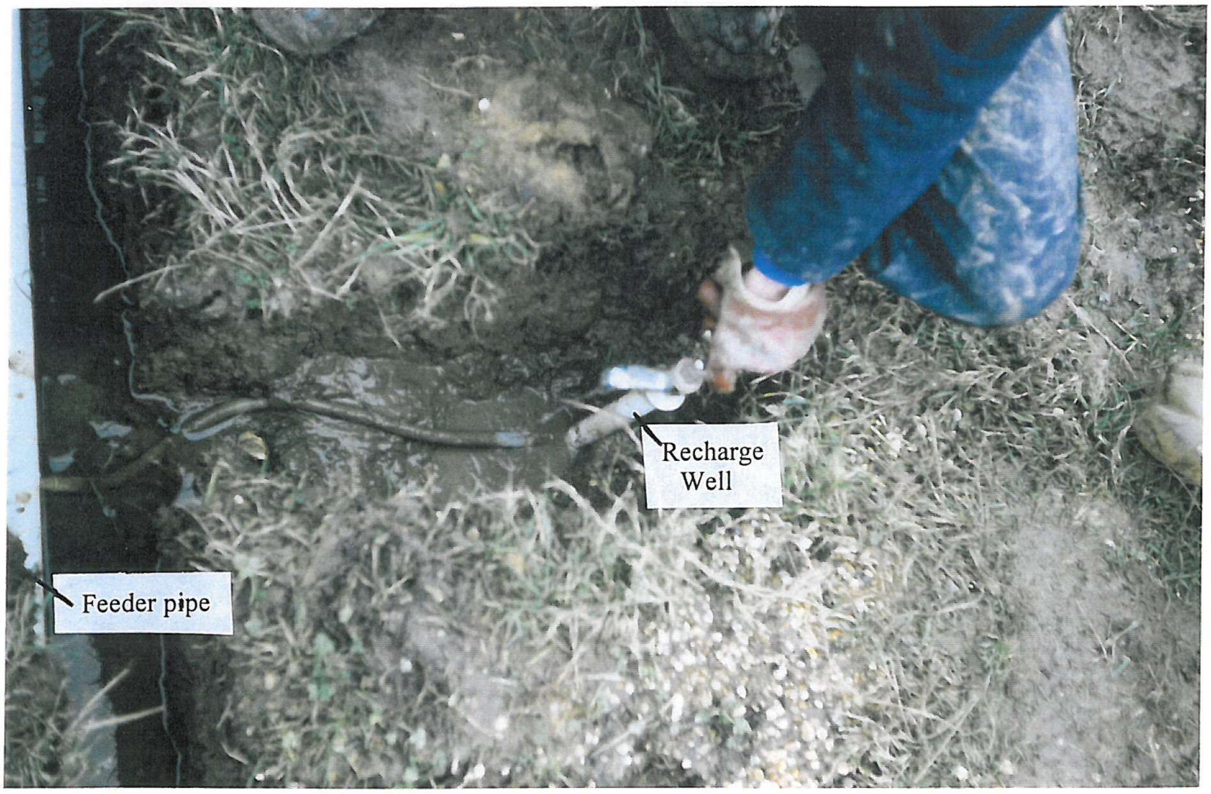


Plate 11-1. Waste pipe to recharge well connection details.

(Also see Figure 11-4)



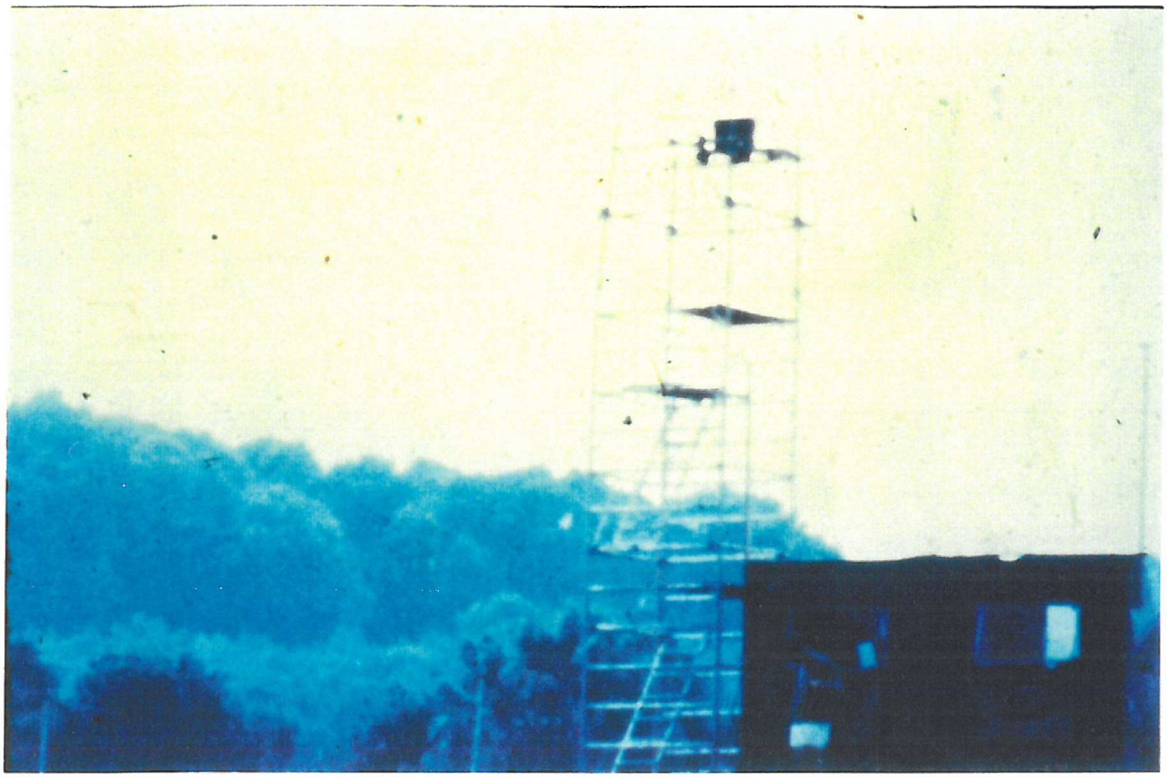
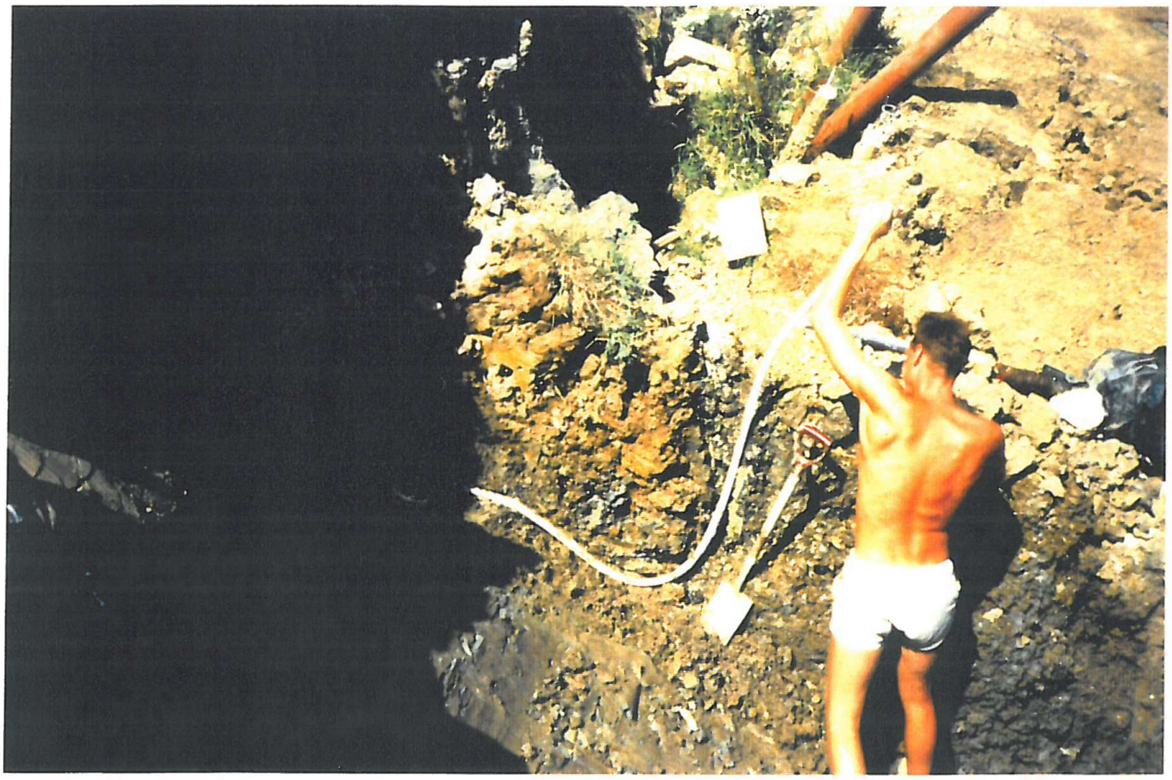


Plate 11-2. Scaffold tower for recharge works.

Plate 11-3. Deformed recharge well as exposed during the post-failure dig out. Note no rupture of the pvc piping or connections took place.



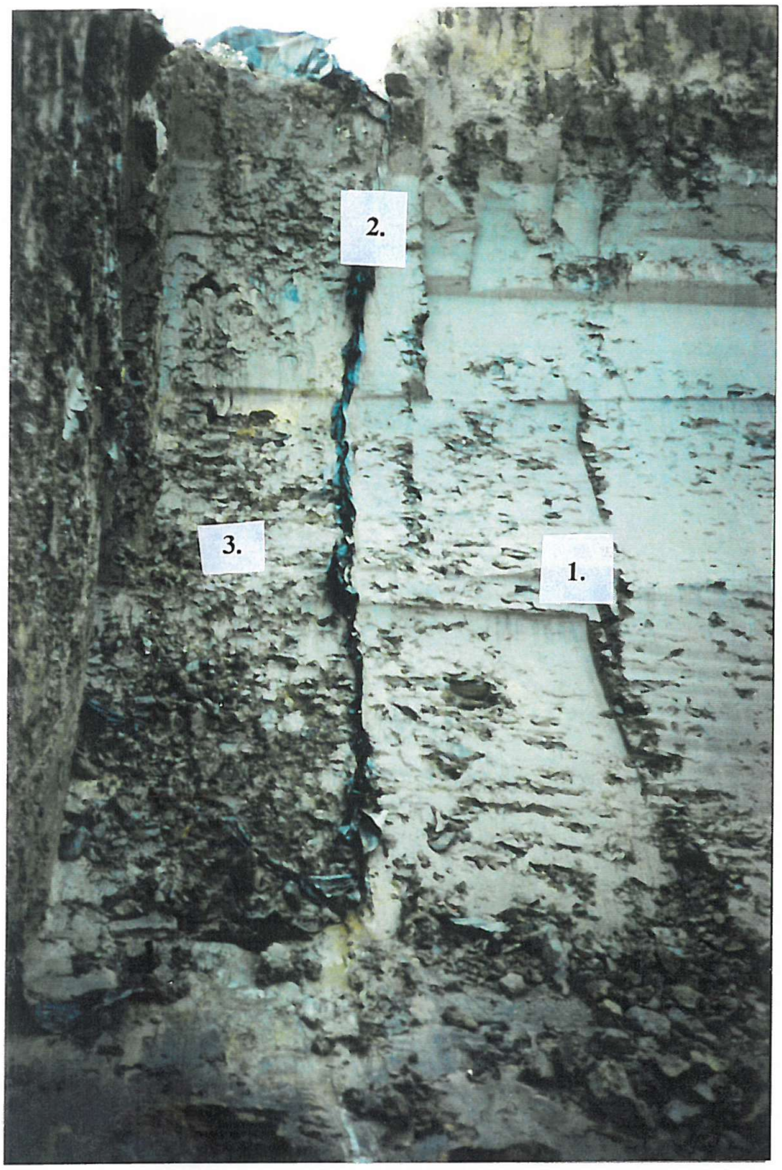


Plate 12-1. Southern side shear break exposed during the post-failure dig out.

1. Slope
2. Double layer of plastic sheeting
3. Backfill to side shear break

Plate 14-1. Inclinometer 07 post failure. Note the access tube laid flat along the slip surface and the empty grout stack still visible towards the upper right of the plate.

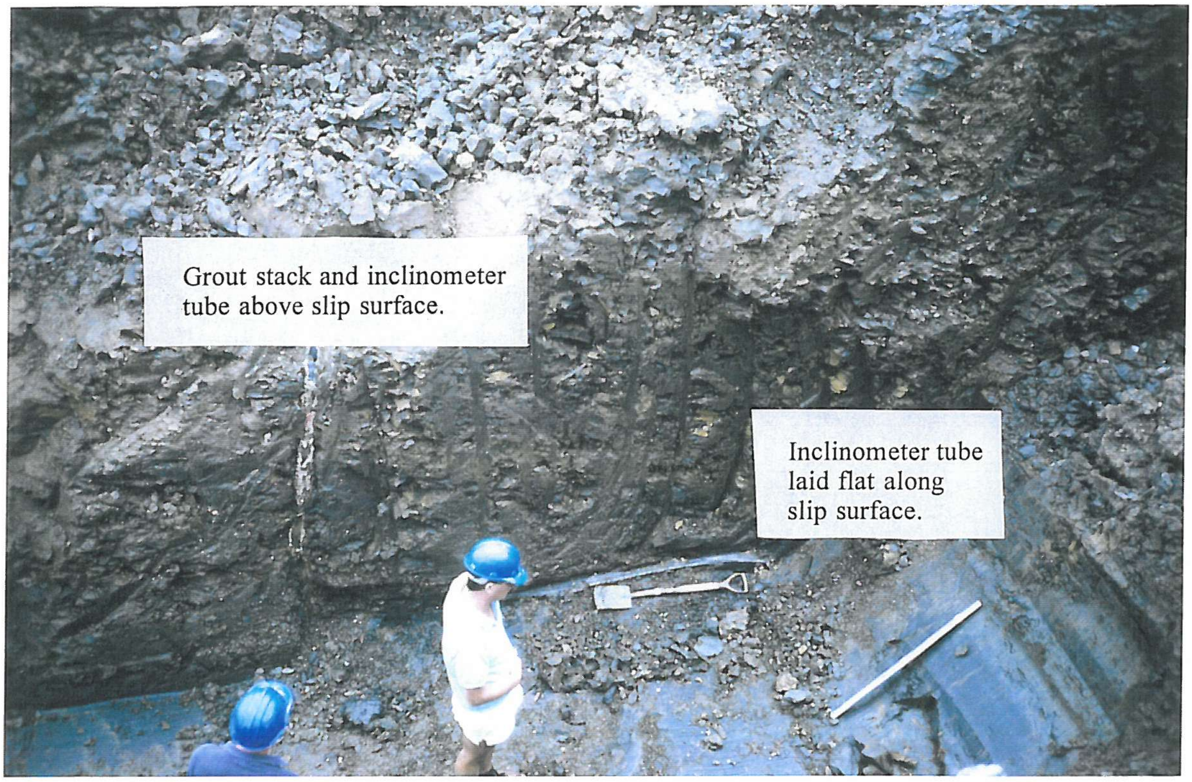
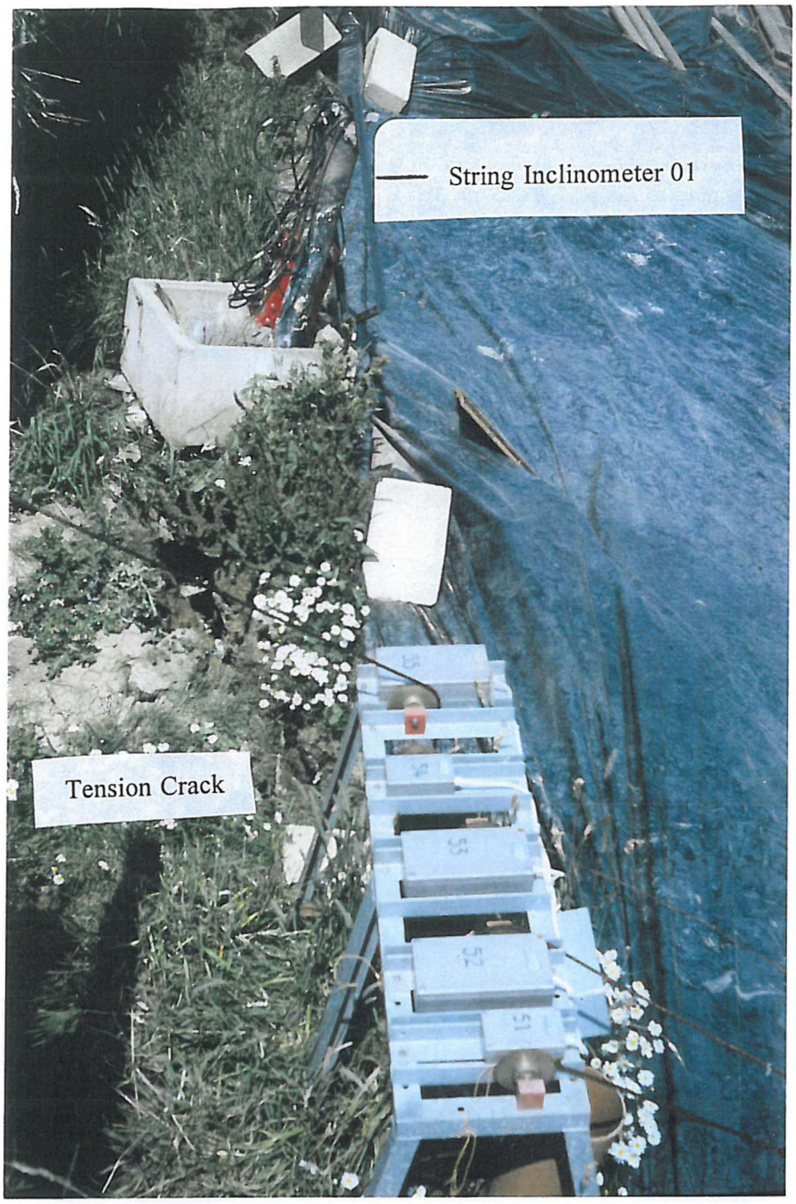


Plate 14-2. Inclinometer 09 post failure.

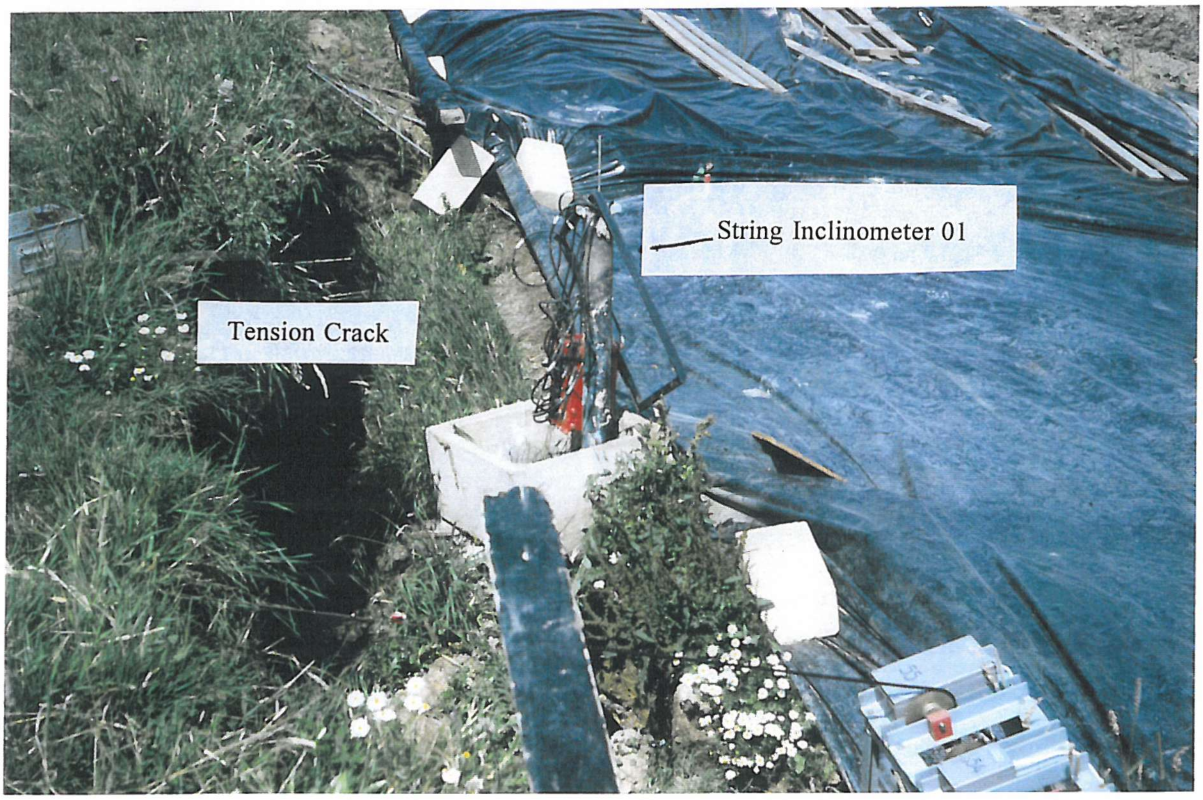
Plate 14-3. Close up of Inclinometer 07 post failure.



a) cica 2:00pm

Plate 14-4 Slip surface adjacent to String Inclinometer 01. 16 July 1989 (day 196).

b) cica 4:00pm



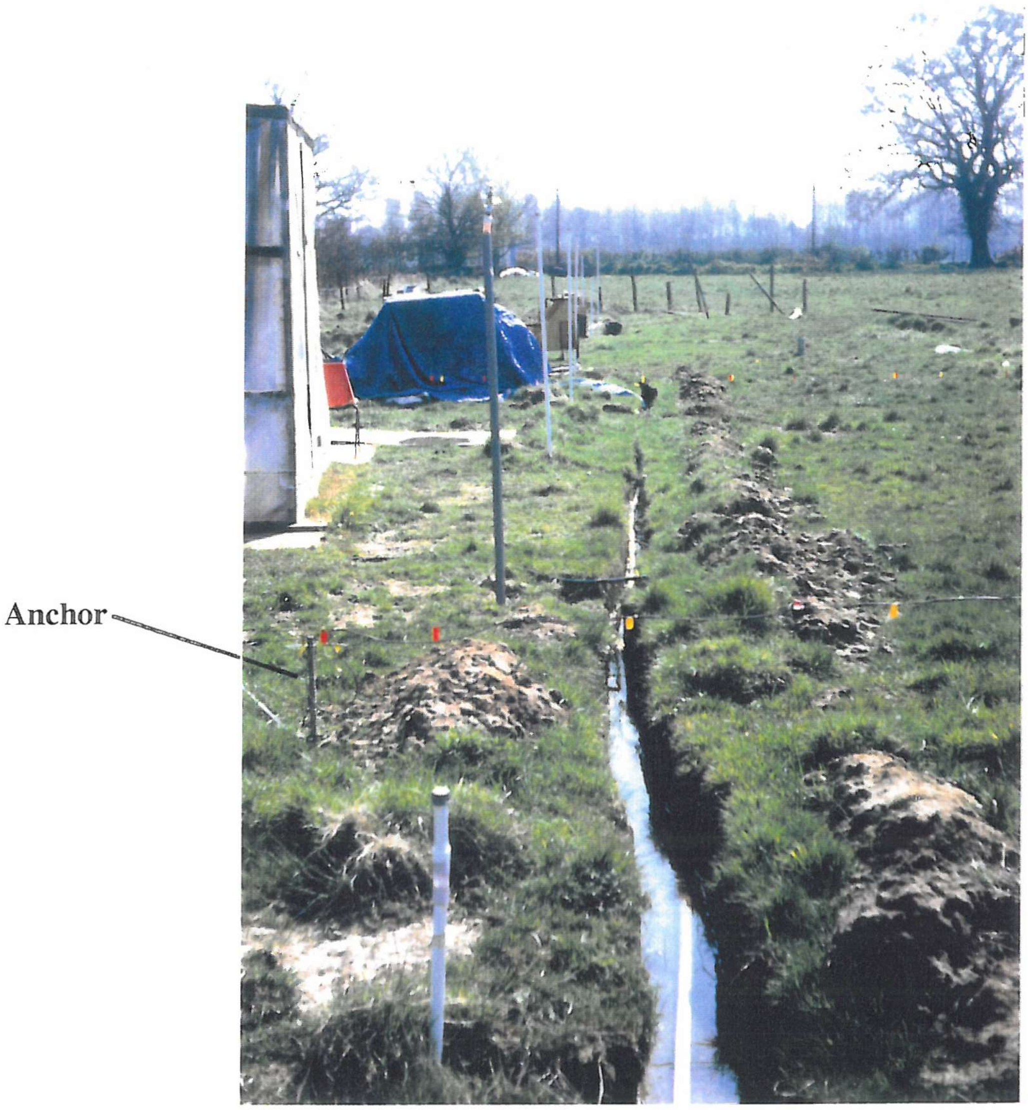


Plate 14-5. Wire extensometer anchor.
Anchor is formed from 8mm reinforcement
bar in a concrete socket.

Plate 15-1. Inclinometer 07 laid flat along slip surface.

Plate 15-2. Inspection trench widened and battered back.

Plate 15-3. Sampling of the slip surface using 300mm dia. push samples.

Plate 15-4. Inclinometer 09.

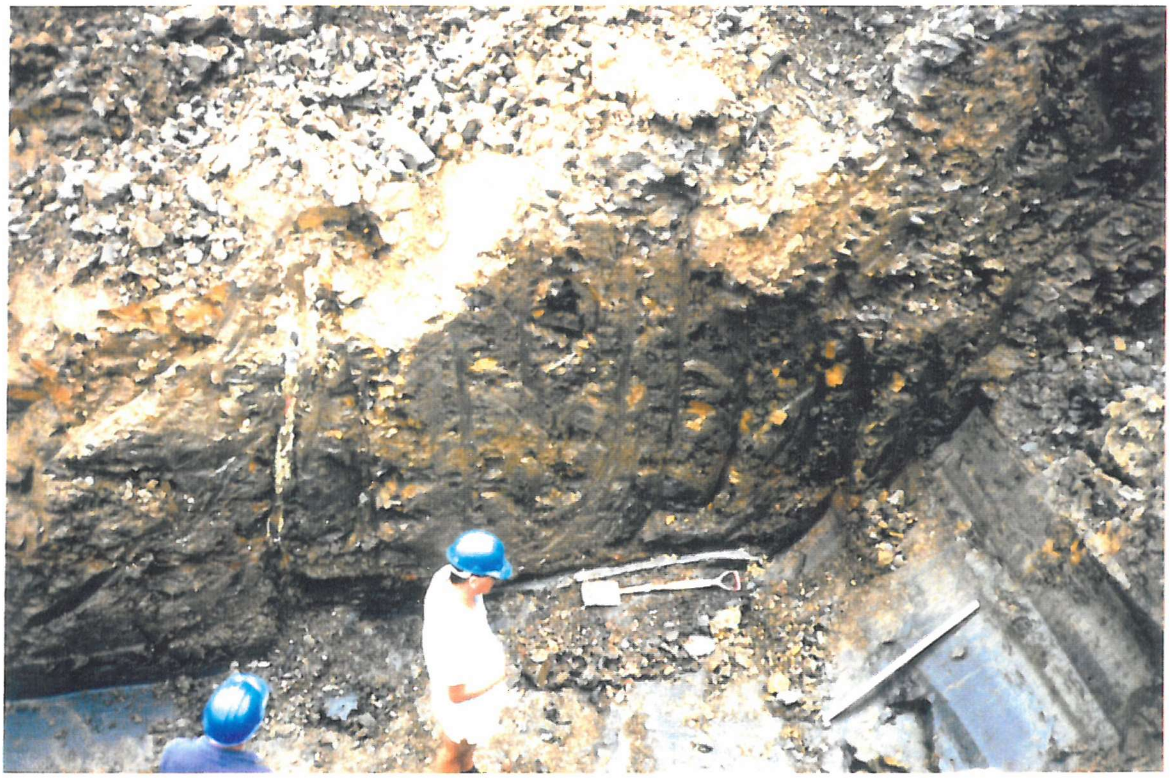


Plate 15-5. Grout stack, Inclinometer 06.

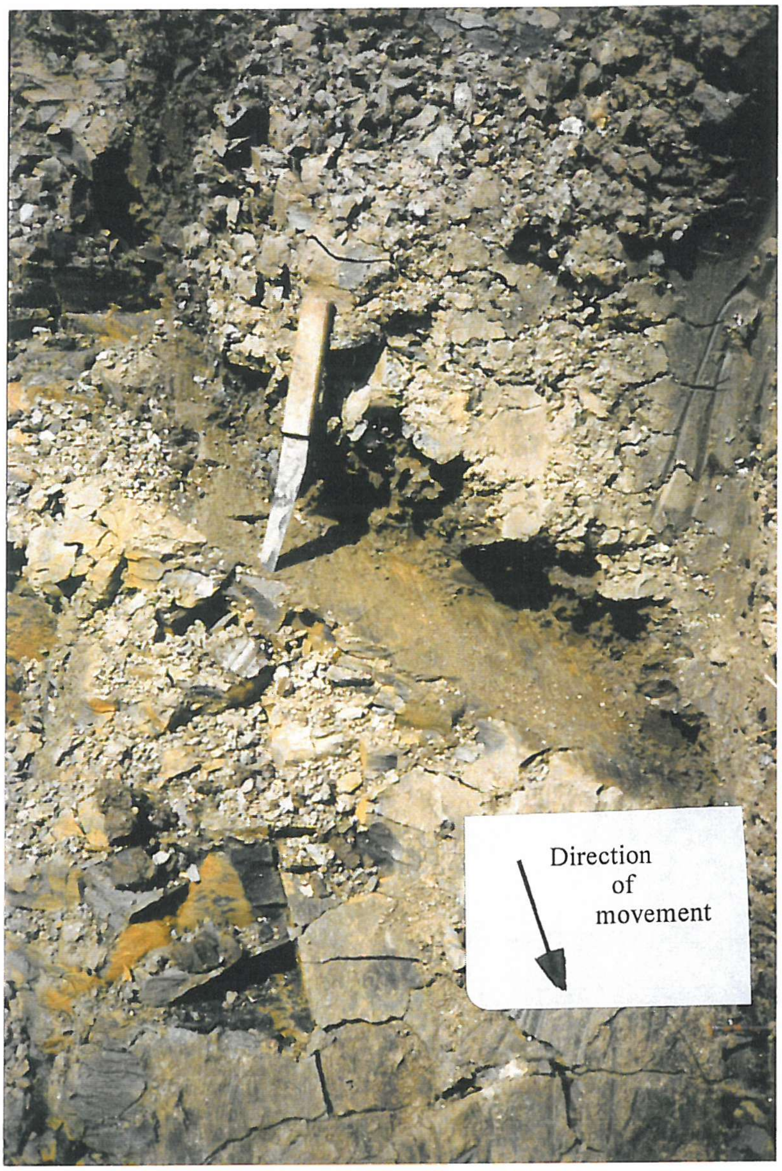


Plate 15-6. Slip surface in "mid" region.

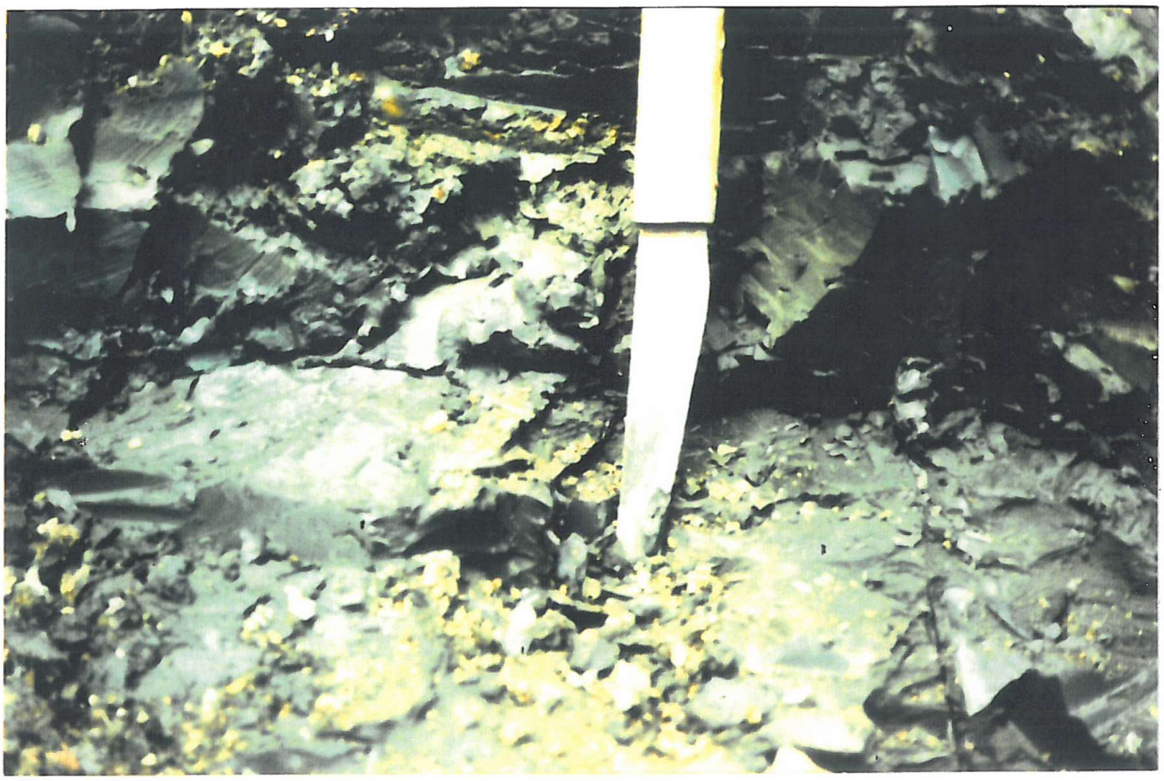


Plate 15-7. Slip surface between mid-zone and basal slip.

Plate 15-8. Exposed basal slip surface. Note the shiny surface and striations.

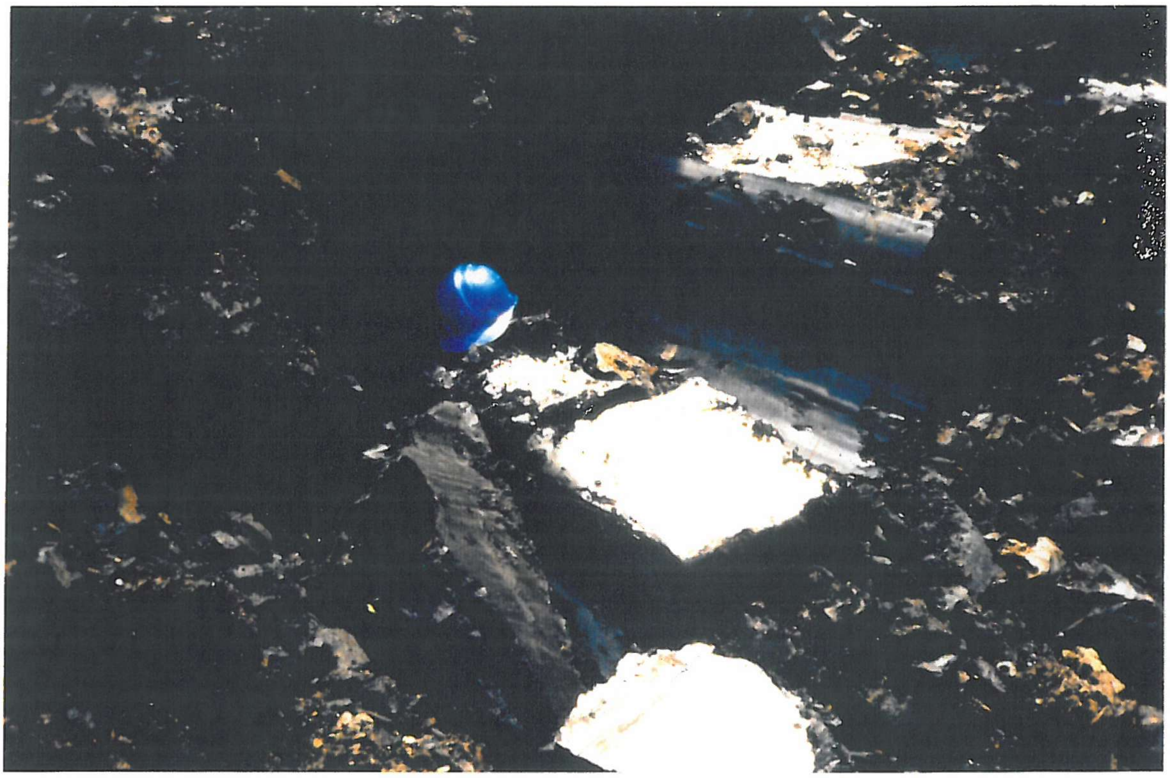


Plate 16-1. Slope post collapse, Day 197

FIGURES

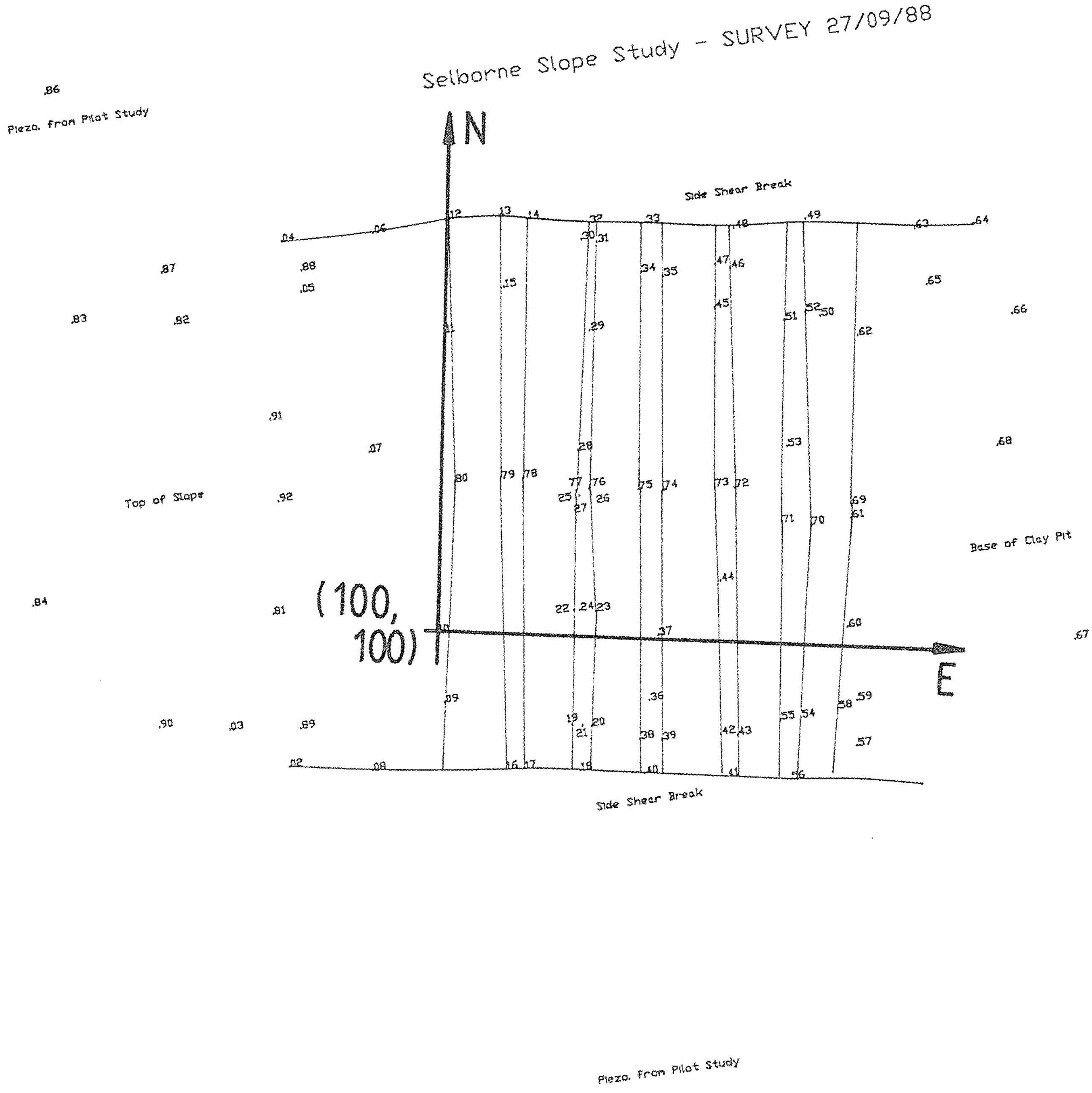


Figure (ii) Local survey coordinate system

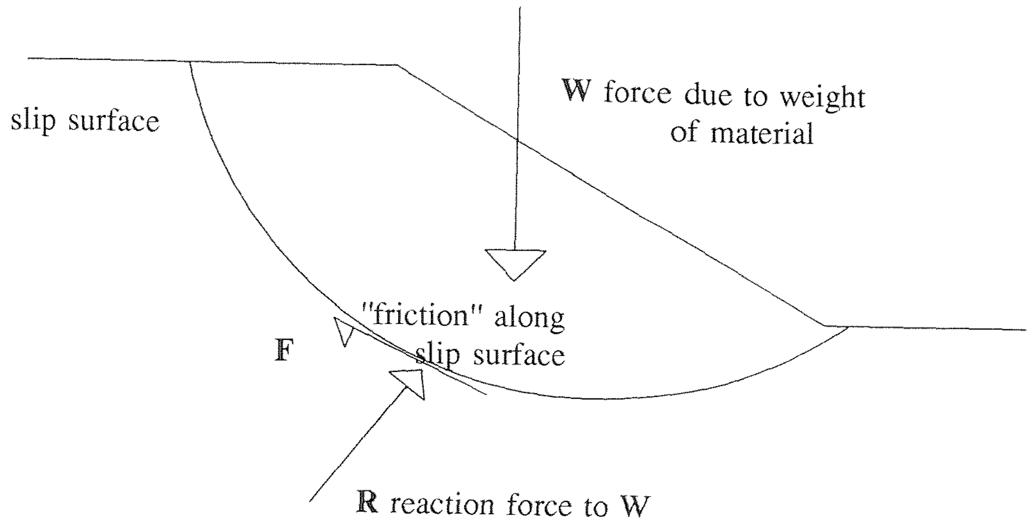


Figure 2-1 Simplified diagram of forces acting on a slope

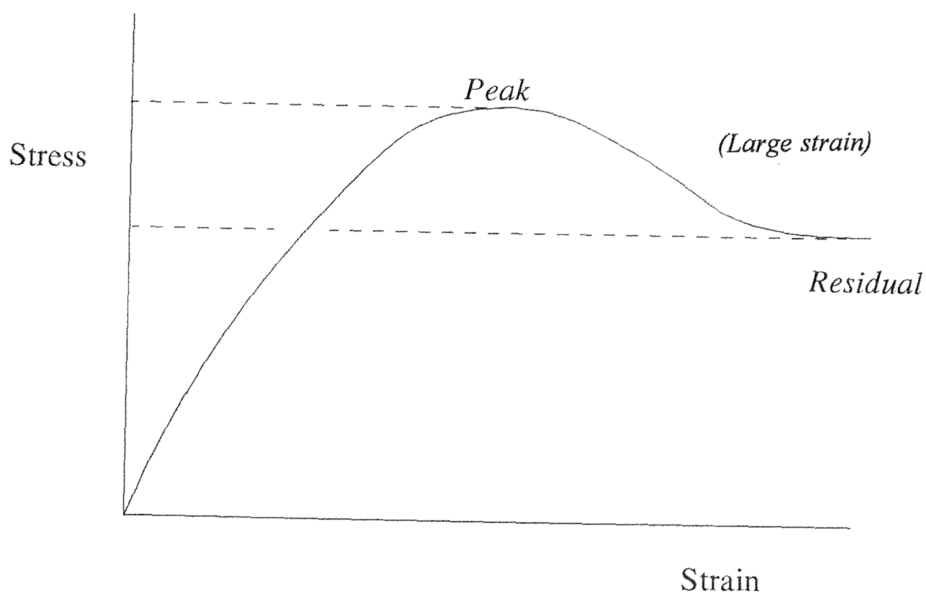


Figure 2-2 Stress-strain relationship for a brittle clay

Figure 2-3 shows how in a steep slope or high cutting a point, O, will follow the stress path O to U1 to D1 to failure. It can be seen that only a small amount of pore pressure recovery is required for the point to reach the limit state line and hence failure. The effects of creep deformations in this case could rapidly reduce the length of U1 D1 so that a relatively small increase in pore pressure, say due to unusually high rainfall, could cause failure.

For a shallow slope the stress path to failure for a point O is O to U2 to D2. Hence it can be seen that even allowing for a large amount of creep deformation the pore pressure change required to cause failure in a shallow slope remains quite large.

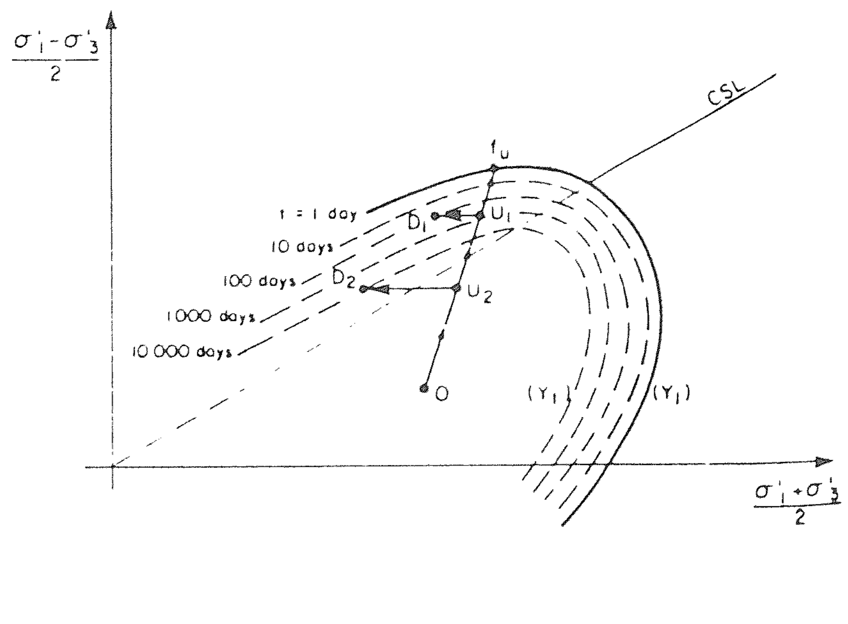


**Figure 2-3 Time dependent limit states.
(After Tevanas and Leroueil 1981)**

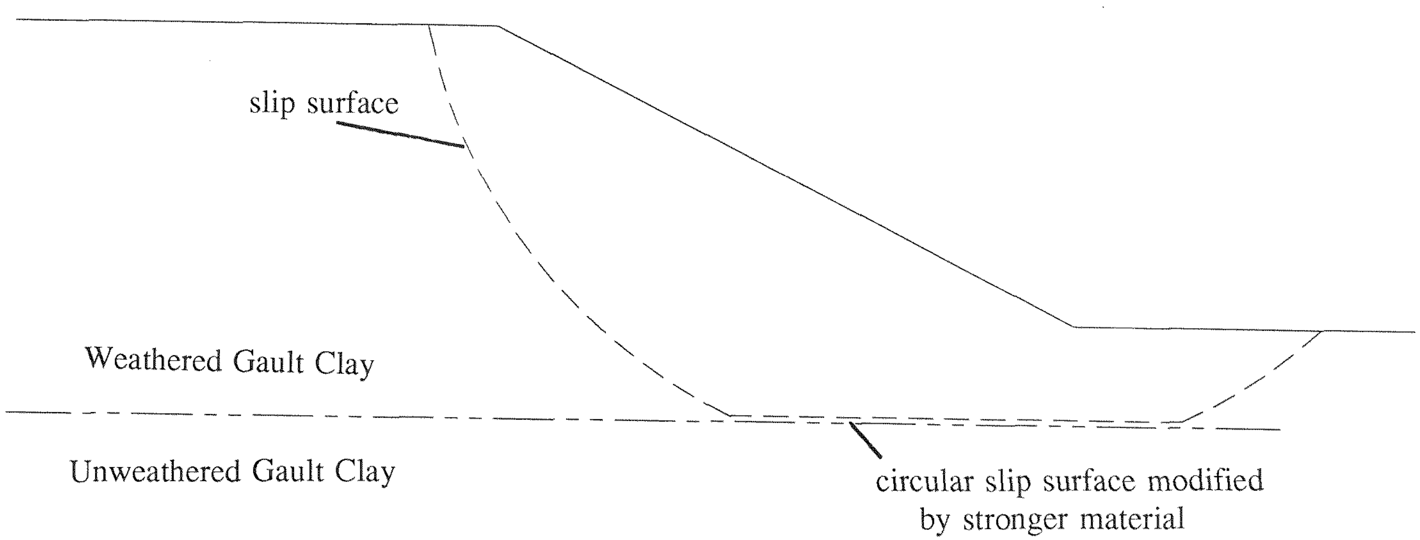
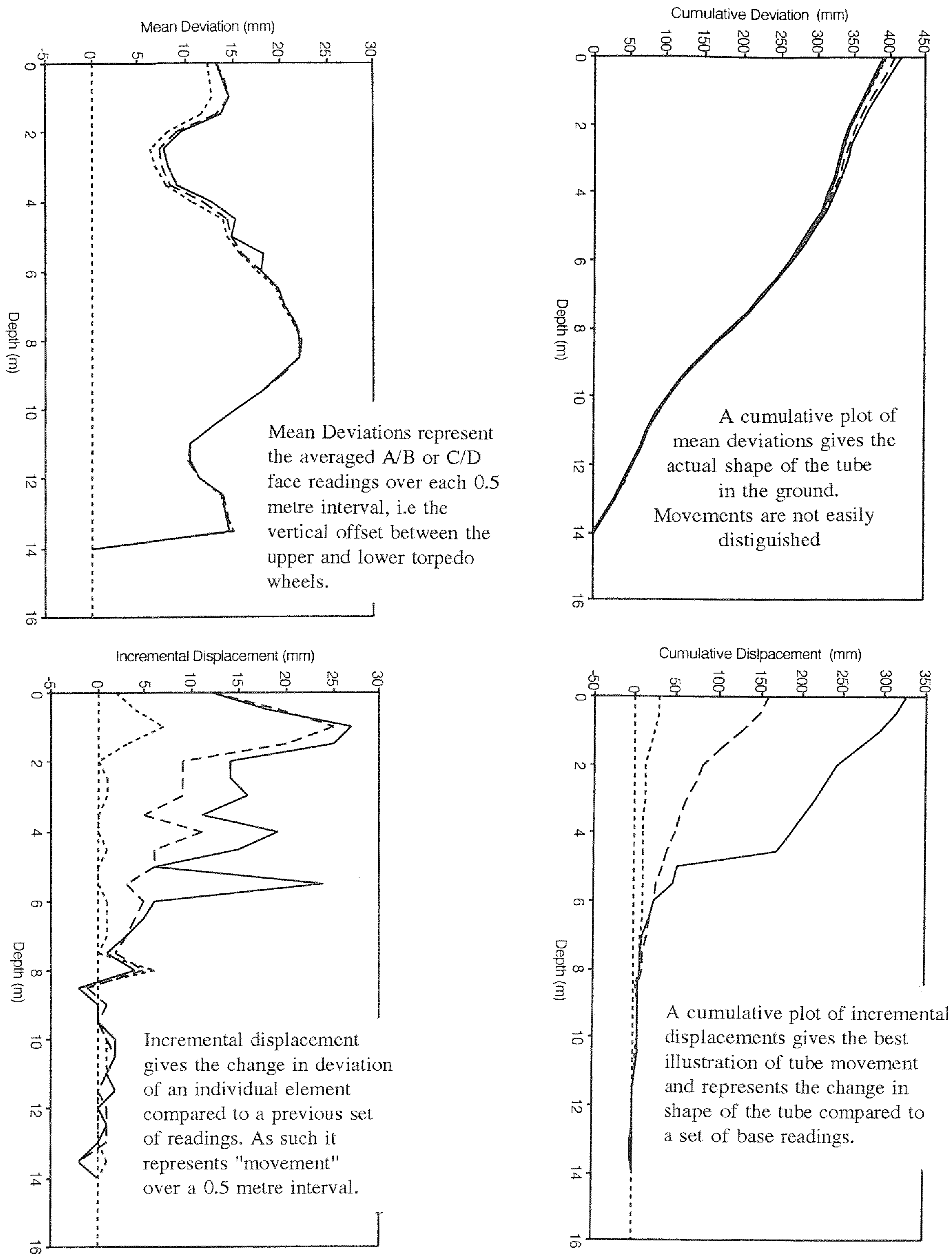
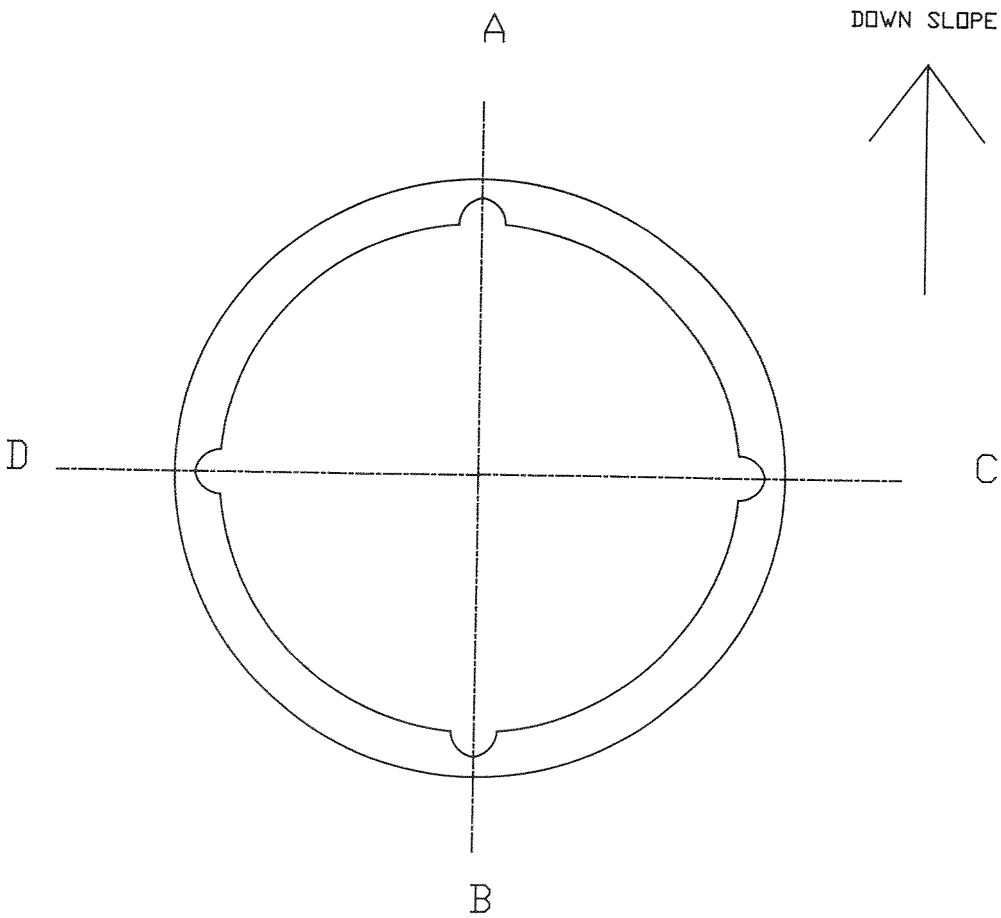


Figure 3-1 Compound failure in Gault Clay

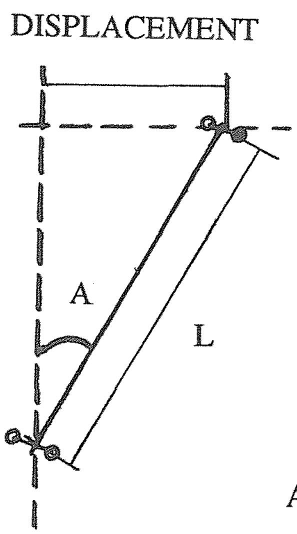


Example used Inclinometer 05. Reading dates: 5 May 1988, 20 March 1989, 28 June 1989. Base date: 28 November 1987.

Figure 4-1 Typical inclinometer displacement profiles



4-2 Inclinometer access tube keyway notation



DISPLACEMENT = L Sin A,

where L is the length of the inclinometer torpedo.

Similarly,

APPARENT DISPLACEMENT = L Sin A'

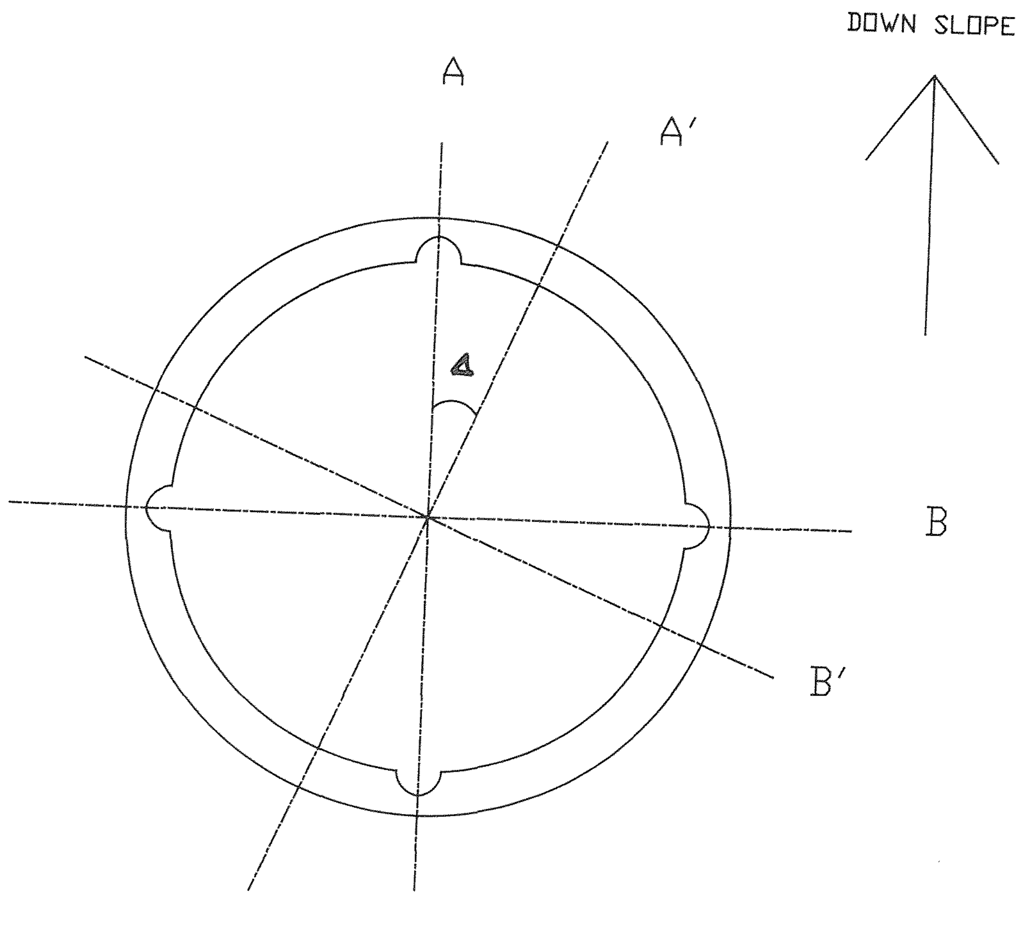


Figure 4-3 Groove roll rotation

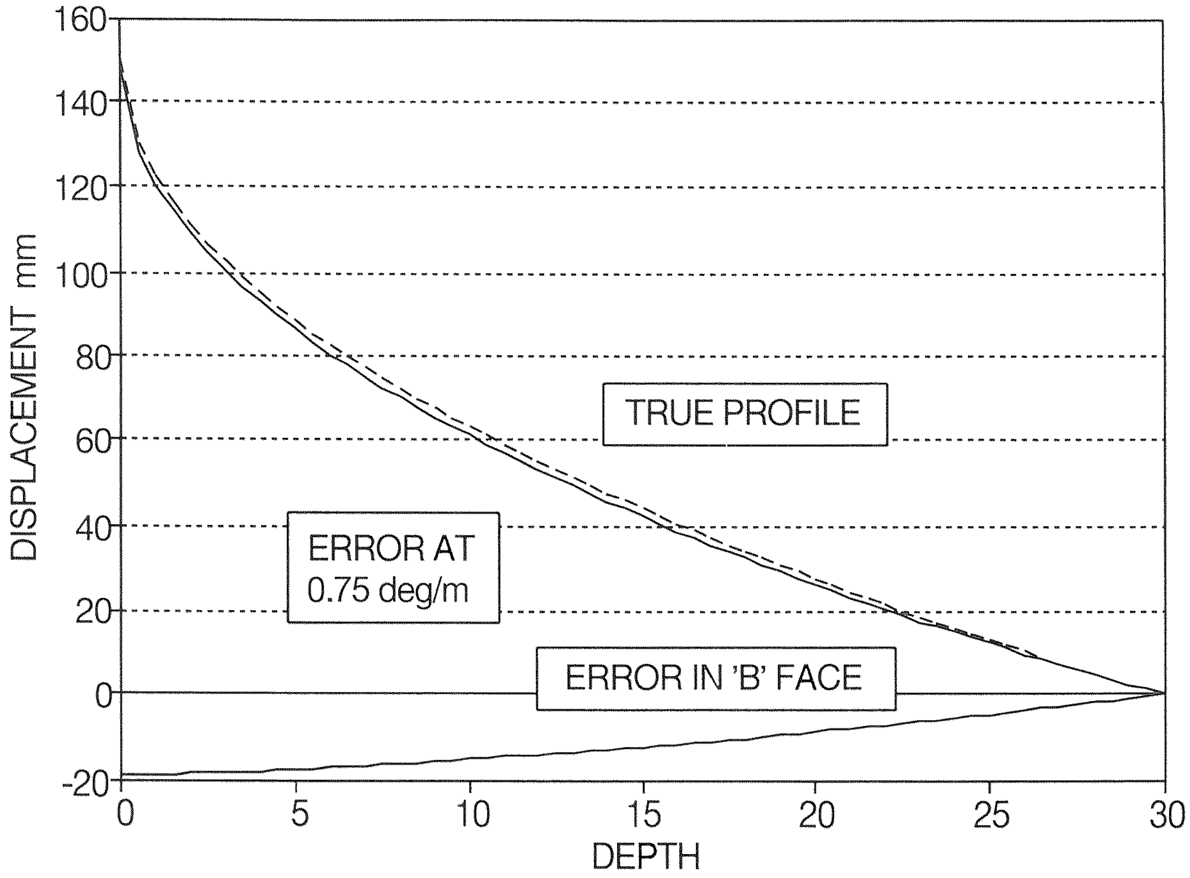


Figure 4-4 Errors due to keyway spiralling

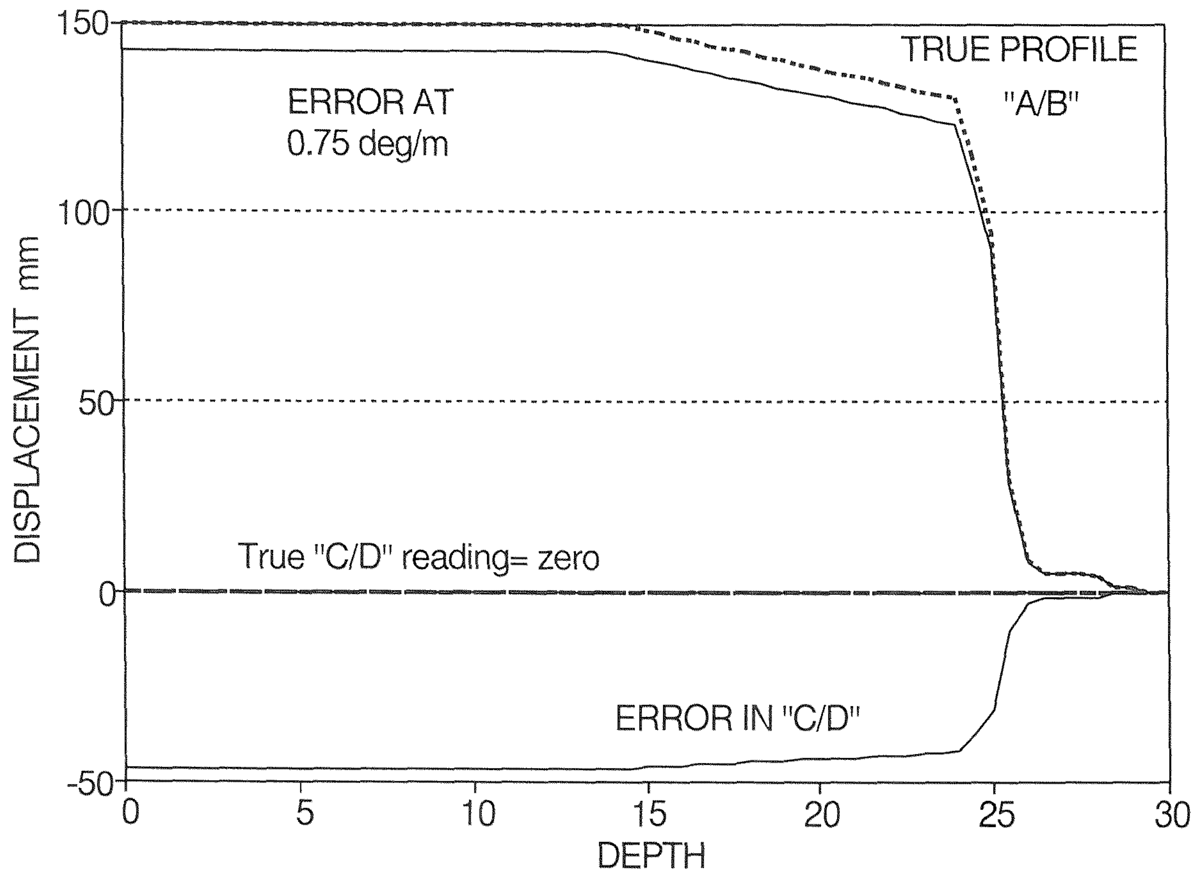


Figure 4-5 Errors due to keyway spiralling (movement at 25m depth)

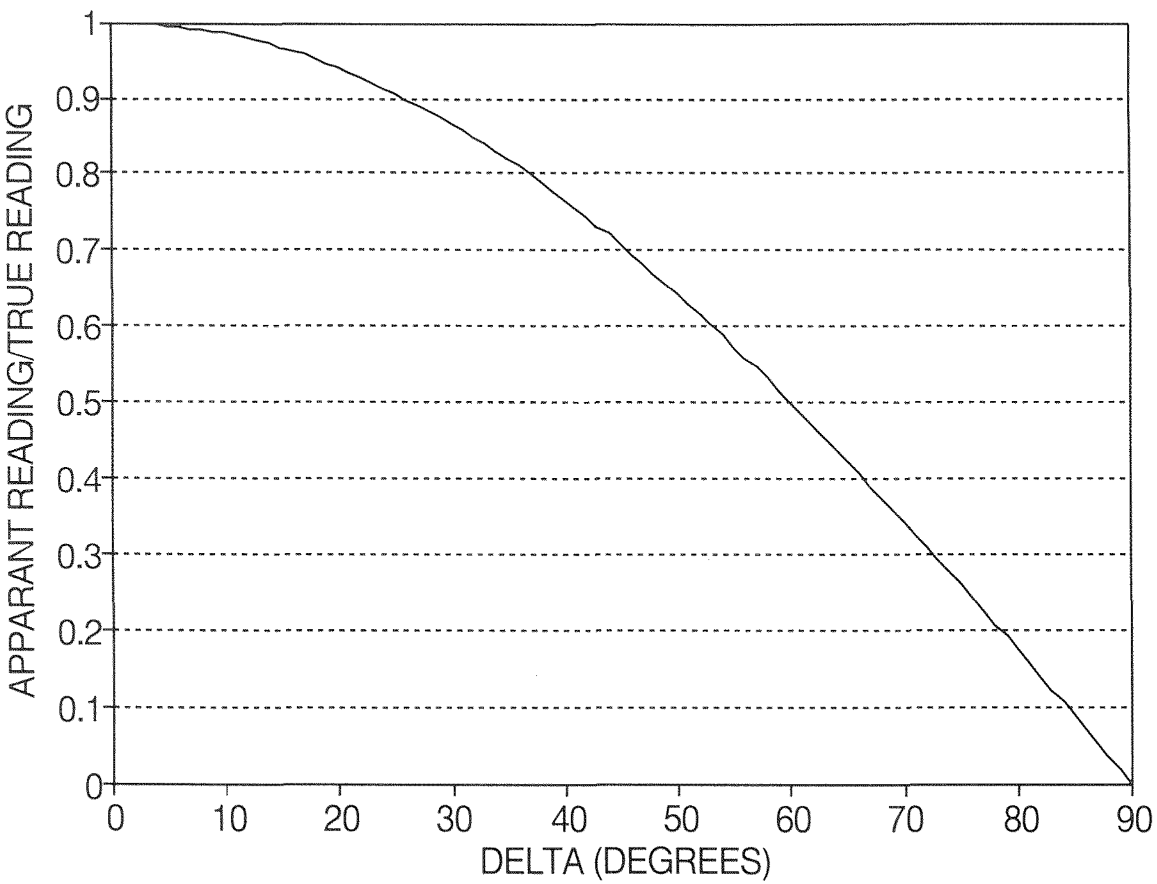
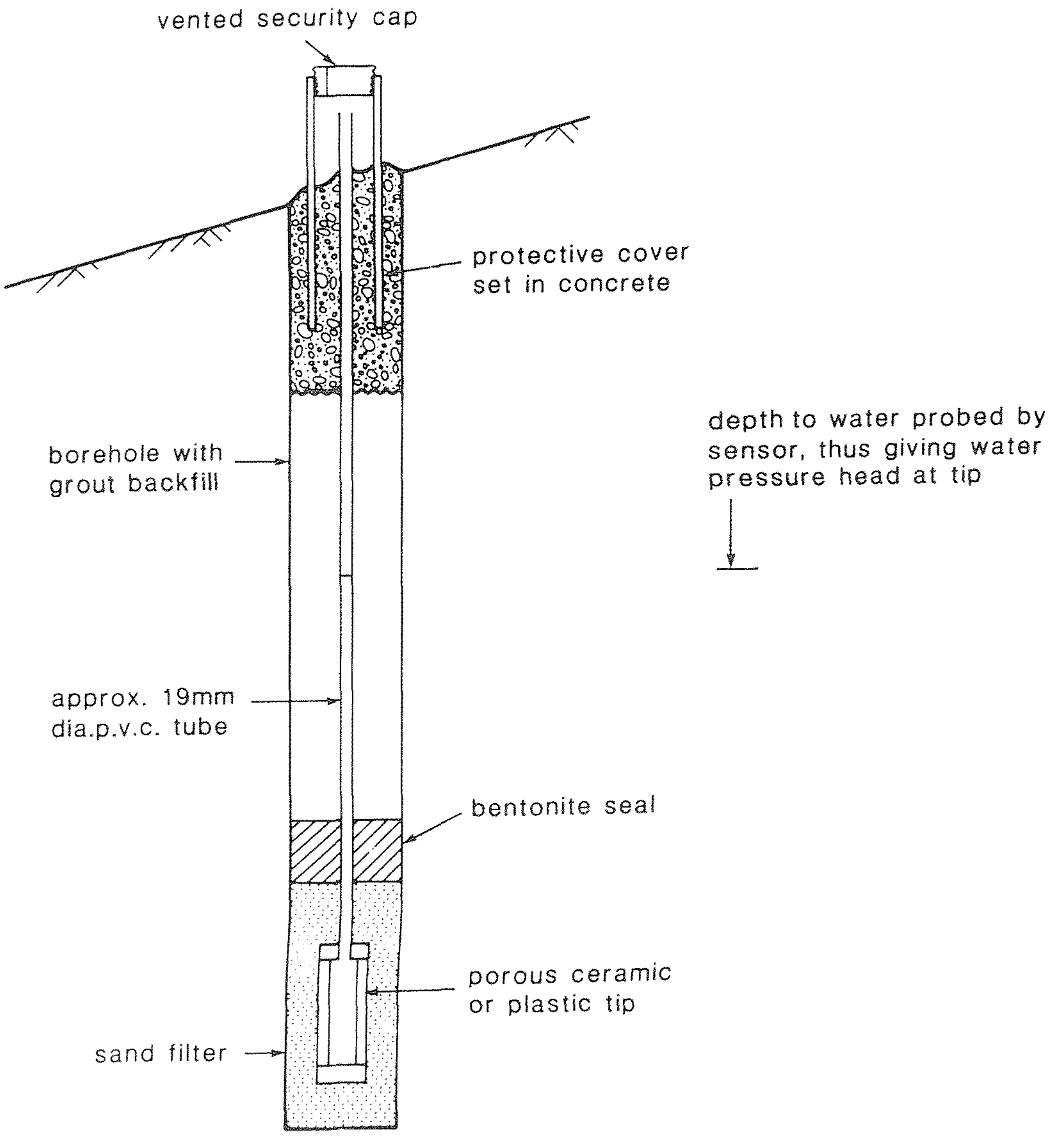
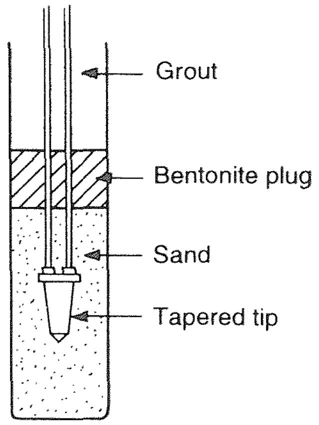


Figure 4-6 Ratio of apparent to true reading versus groove roll (Δ)

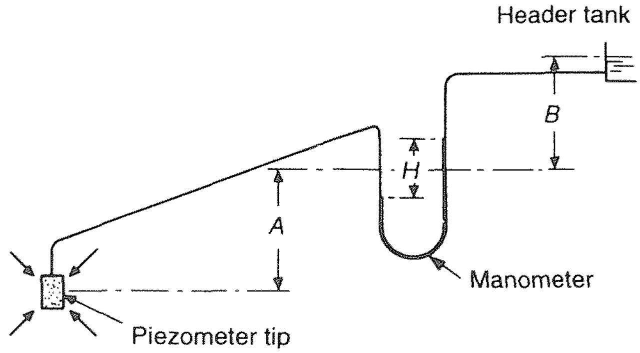


(Illustration from Bromhead : The Stability of Slopes, 2nd Ed.1992)

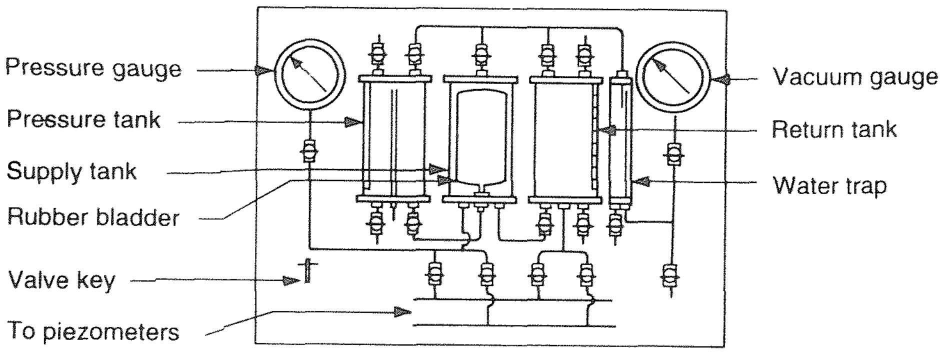
Figure 4-7 Standpipe piezometer



(a) Piezometer tip



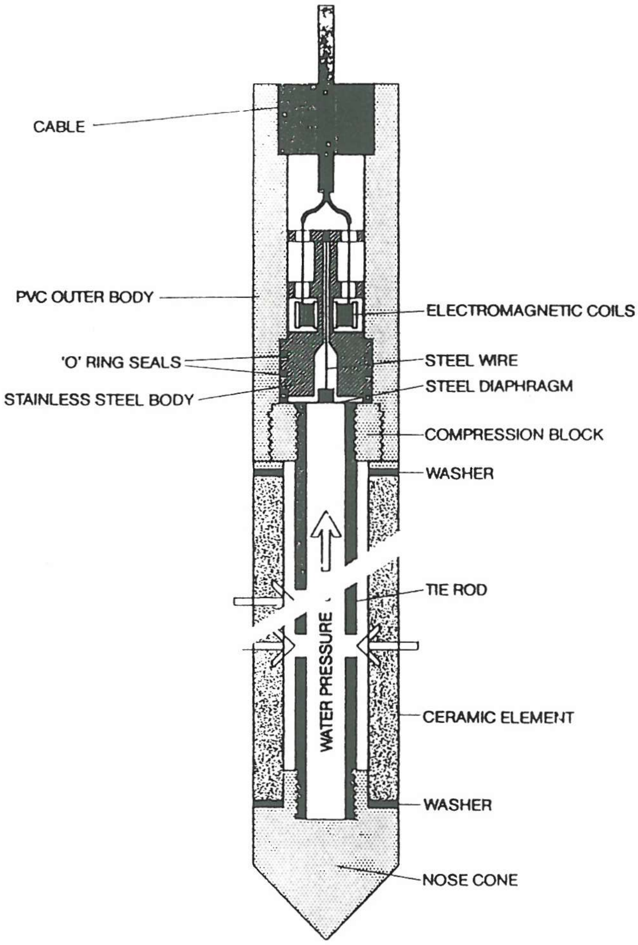
(b) Header tank with mercury manometer for pressure measurement



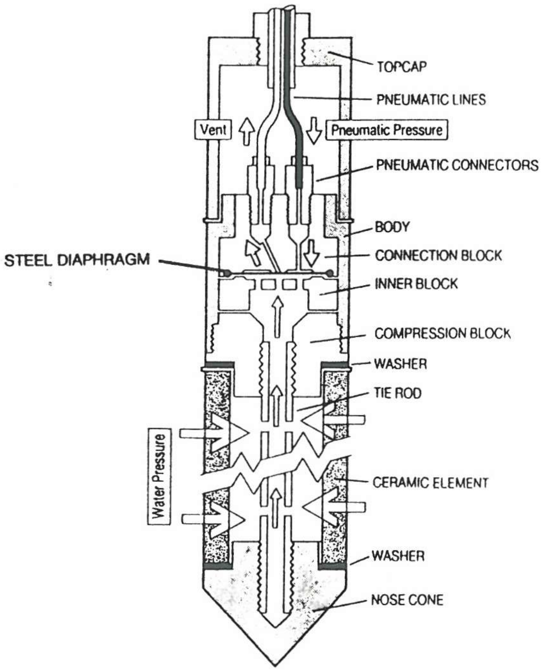
(c) Panel mounted de-airing unit

(Illustration from Clayton, Matthews and Simons : Site Investigation, 2nd Ed. 1995)

Figure 4-8 Twin tube hydraulic piezometer



a) Vibrating Wire Piezometer



b) Pneumatic Piezometer

(Based on drawings provided by Geotechnical Instruments Ltd)

Figure 4-9 Diaphragm piezometers

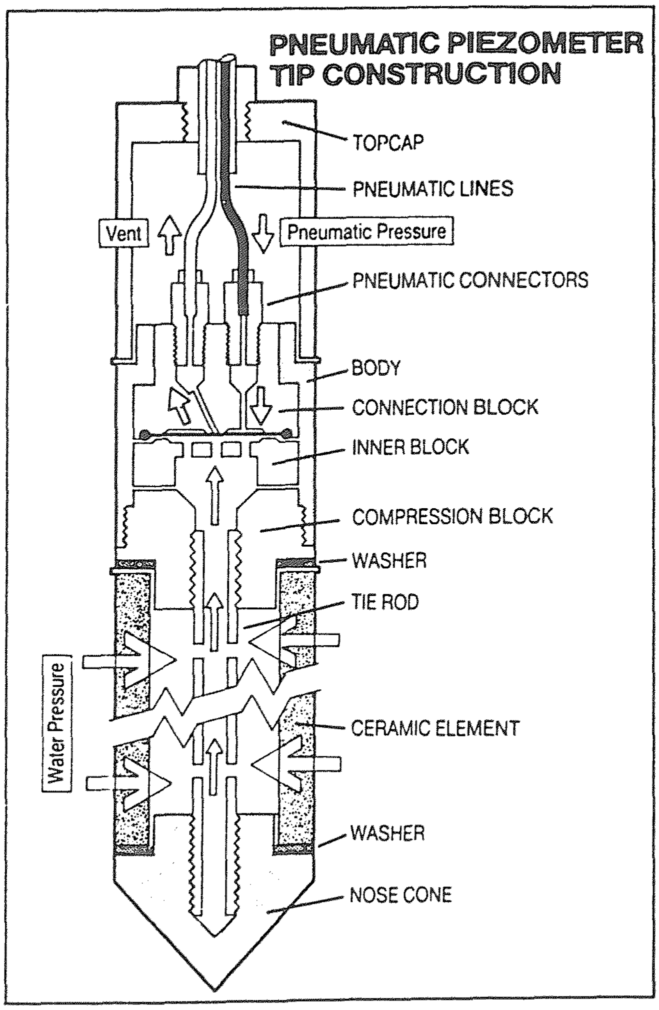


Figure 4-10 Operation of conventional pneumatic piezometer

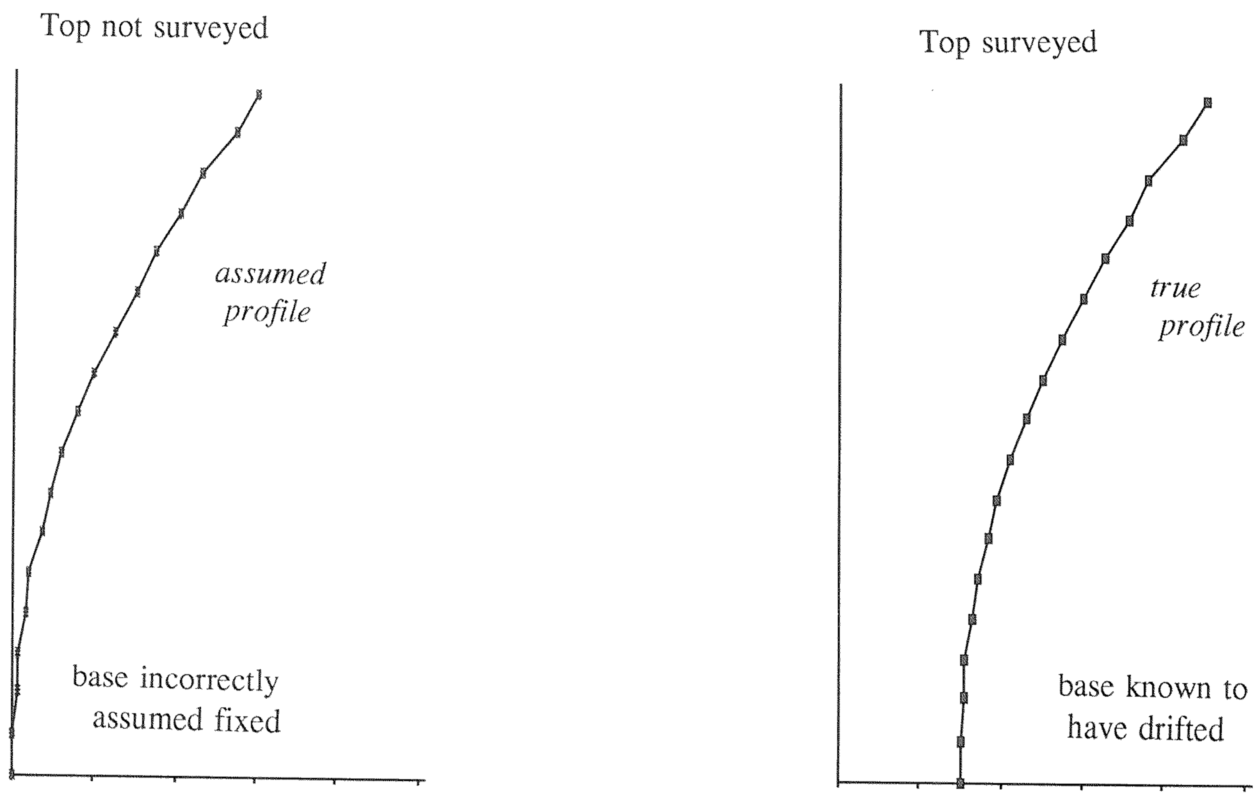
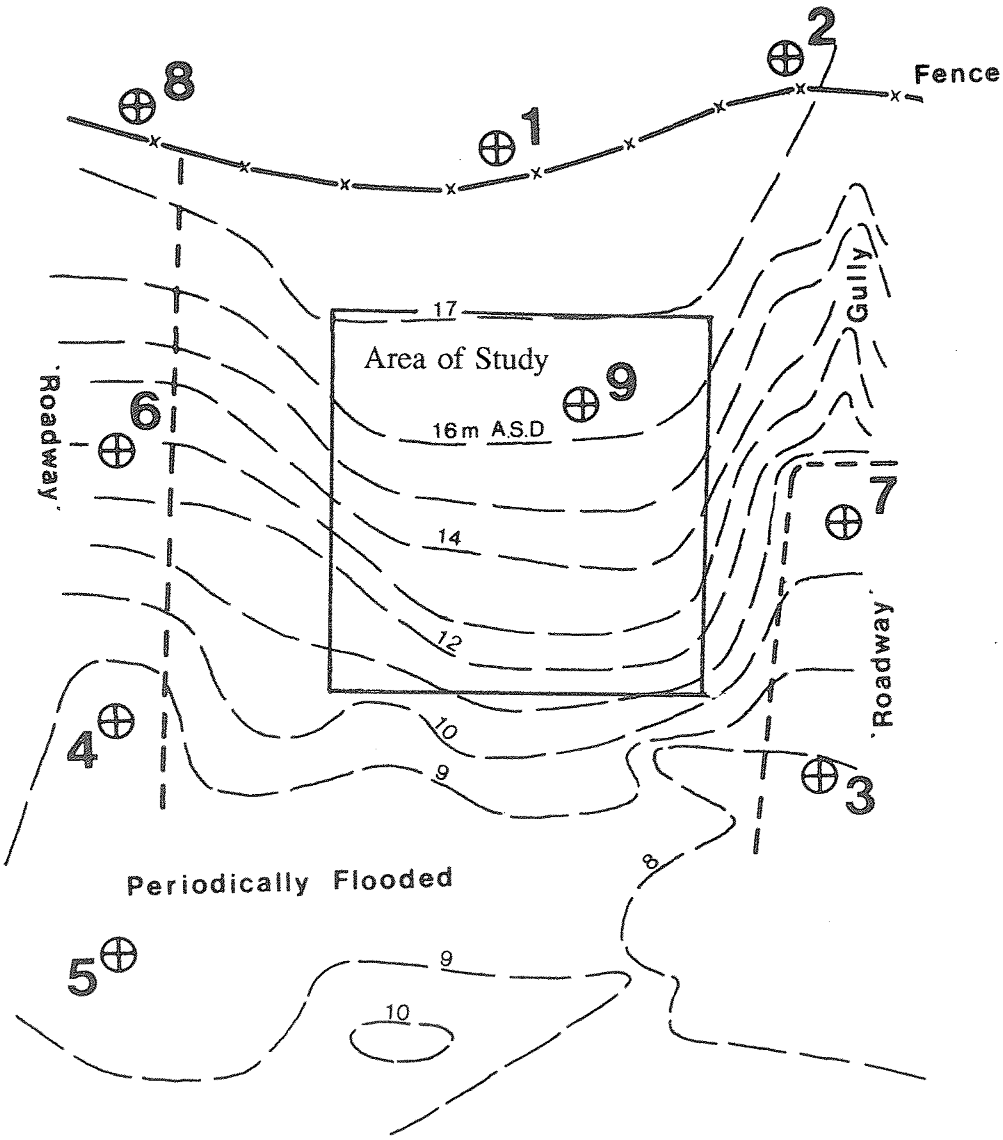


Figure 4-11 Possible inclinometric errors due to non-fixity

Figure 5-1 Site location plan



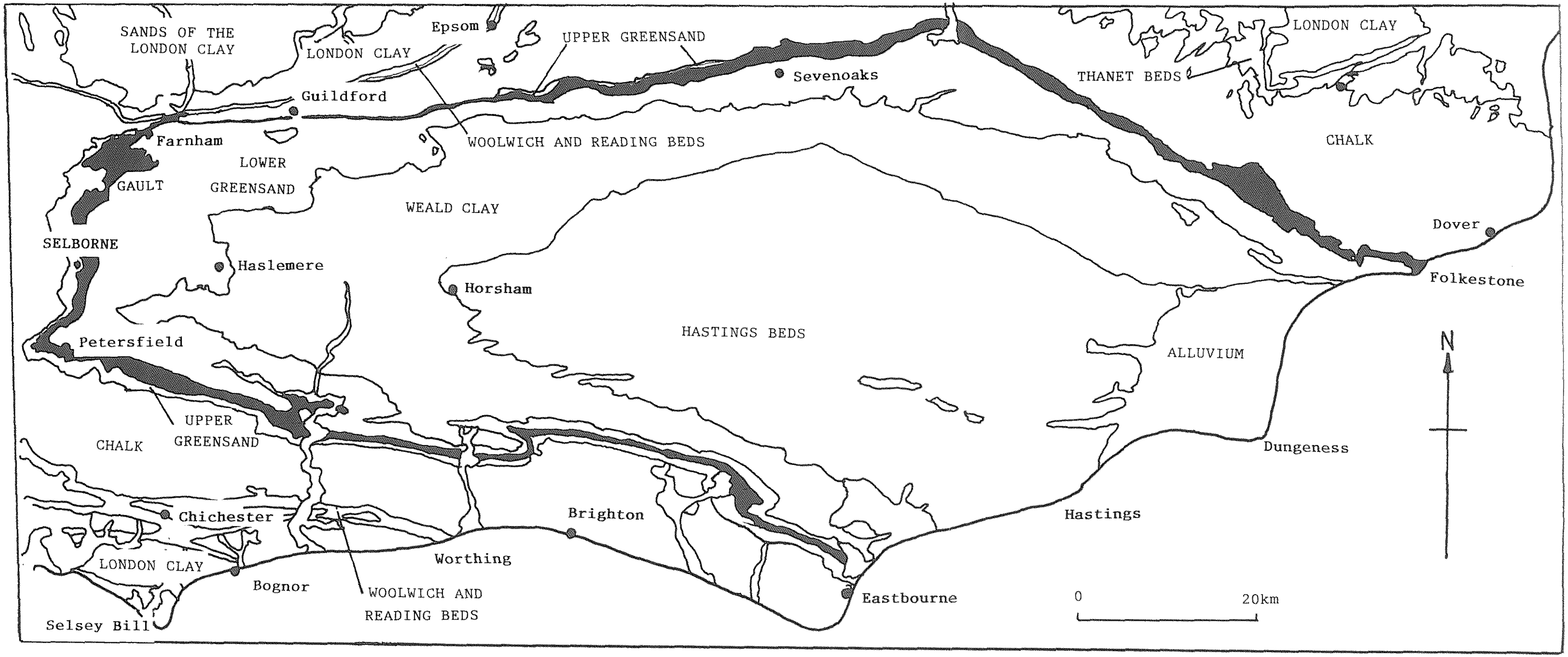
Information based on Feasibility Study Survey.

Scale 1:500.

Levels to Feasibility Study site datum.
17m ASD equivalent to 86.9m AOD.

⊕1 Piezometer from feasibility study.

Figure 5-2 Pre-existing site topography (from Copper 1986)



(Adapted from British Regional Geology, The Wealden District. Fourth Edition)

Figure 6-1 Geology of the Wealden District

Cretaceous Formations Present in the Wealden District.

UPPER-CRETACEOUS	CHALK	• Upper Chalk • Middle Chalk • Lower Chalk
LOWER-CRETACEOUS	UPPER GREENSAND	
	GAULT	• Upper Gault • Lower Gault
	LOWER GREENSAND	• Folkestone Beds • Sandgate Beds • Bargate Beds • Hythe Beds • Atherfield Beds
	WEALDEN BEDS	• Weald Clay • Hastings Beds

(Based on information from British Regional Geology, The Wealden District. Fourth Edition)

Figure 6-2 Sequence of geology

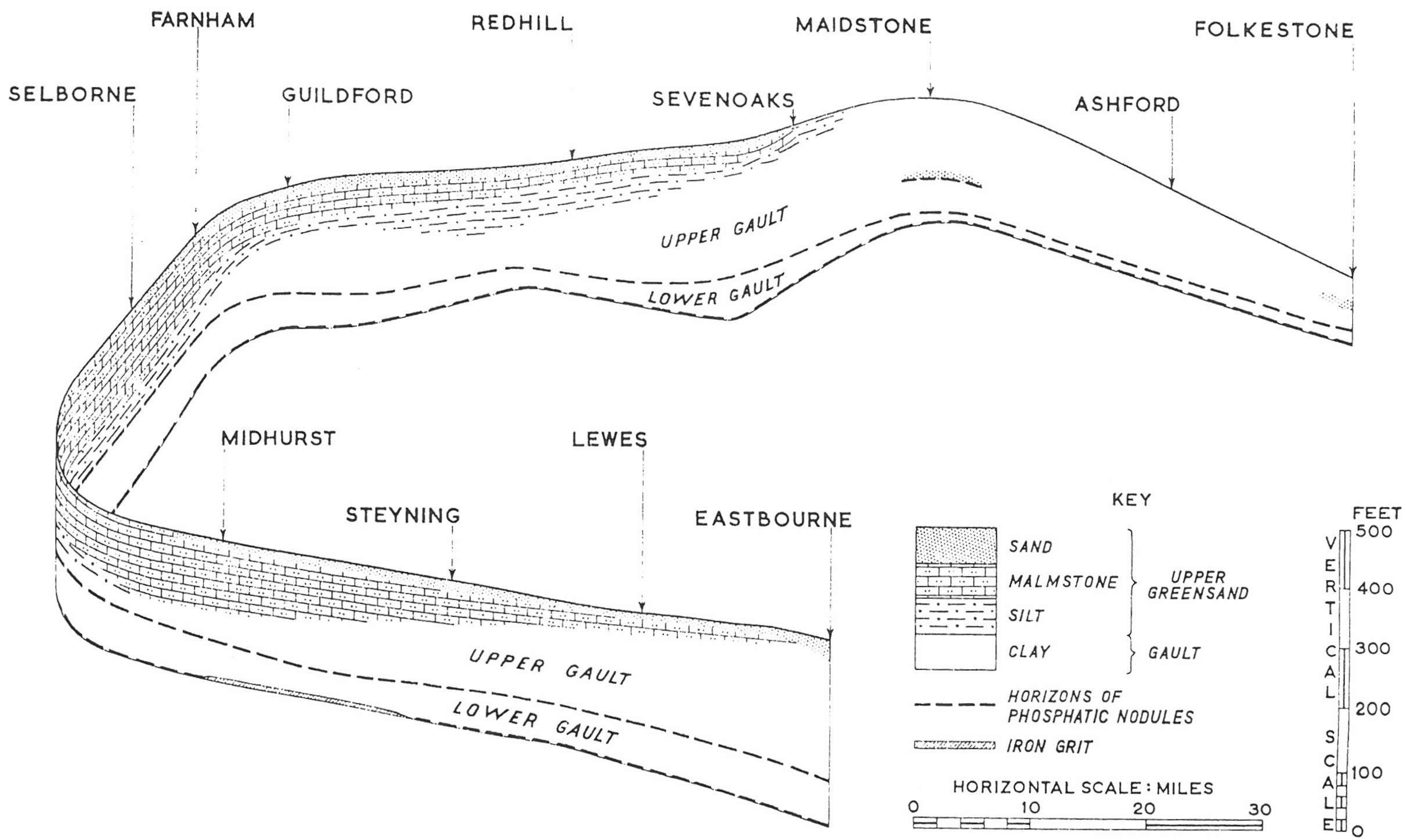


Figure 6-3 Ribbon diagram showing the relationship of the Gault to the Upper Greensand in the Wealden District.
(from British Regional Geology, The Wealden District. Fourth Edition)

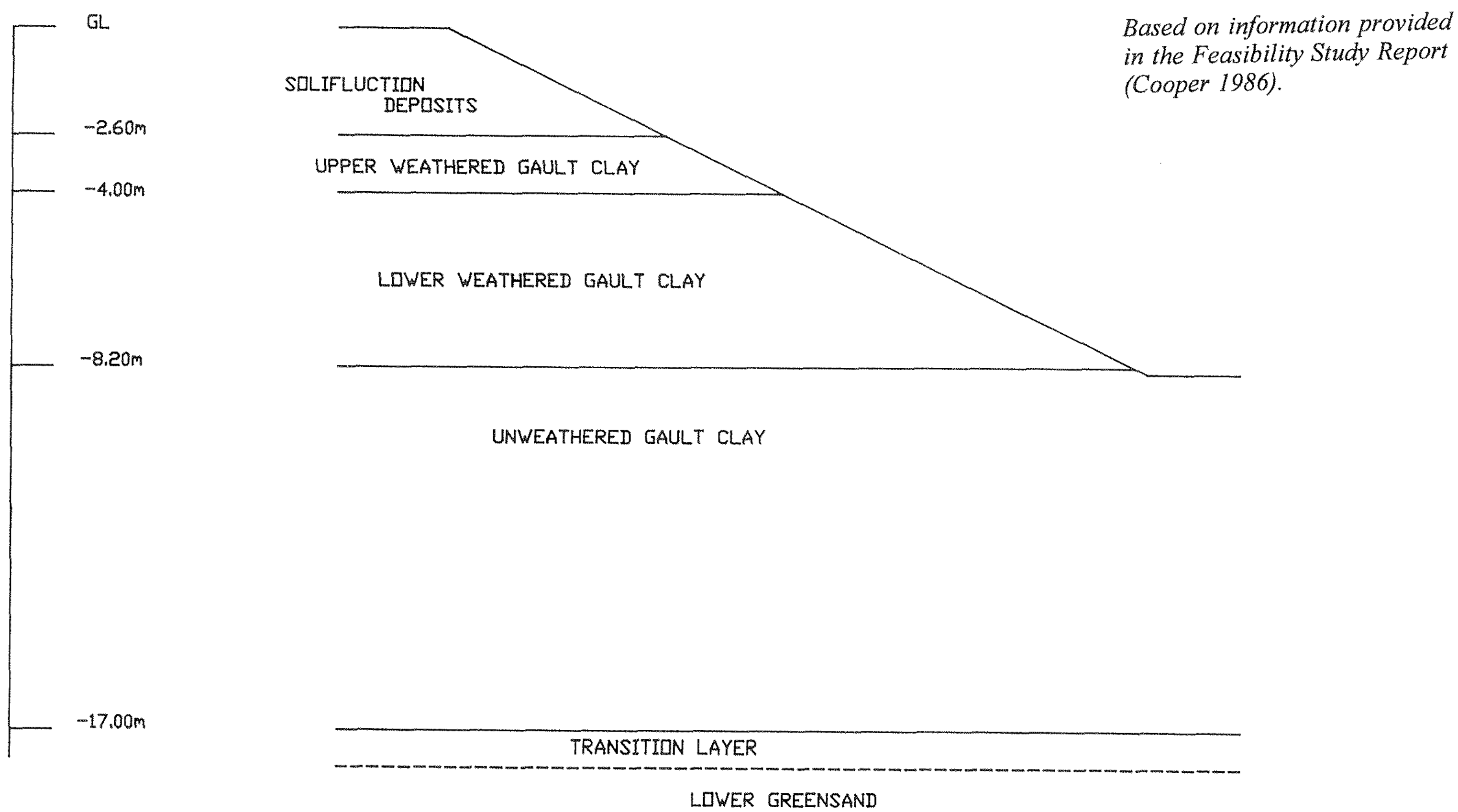


Figure 6-4 General centreline soil profile

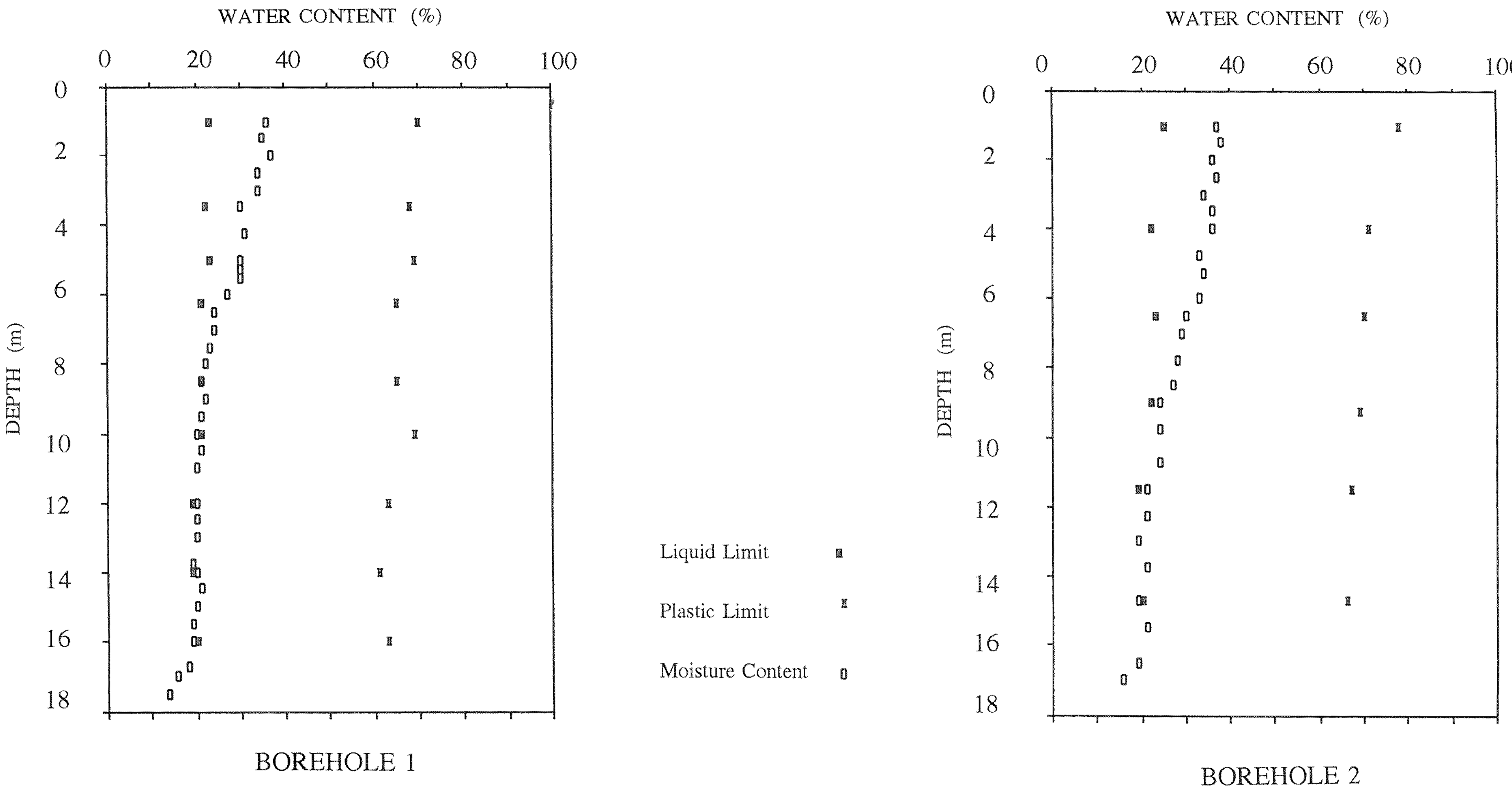


Figure 6-5 Index properties for rotary-cored holes 1 and 2

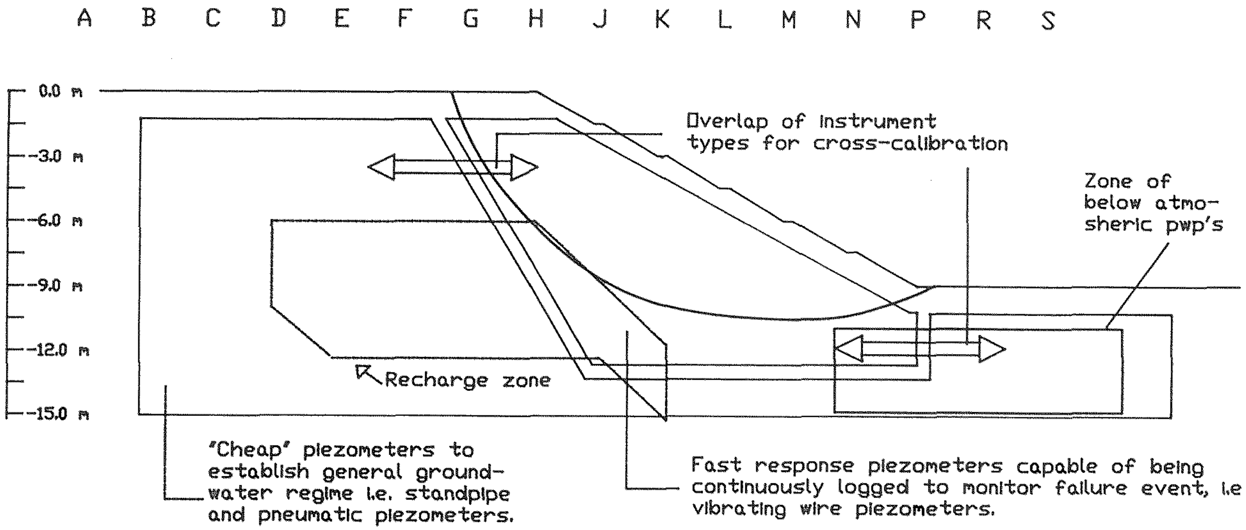


Figure 7-1 Piezometer layout theory

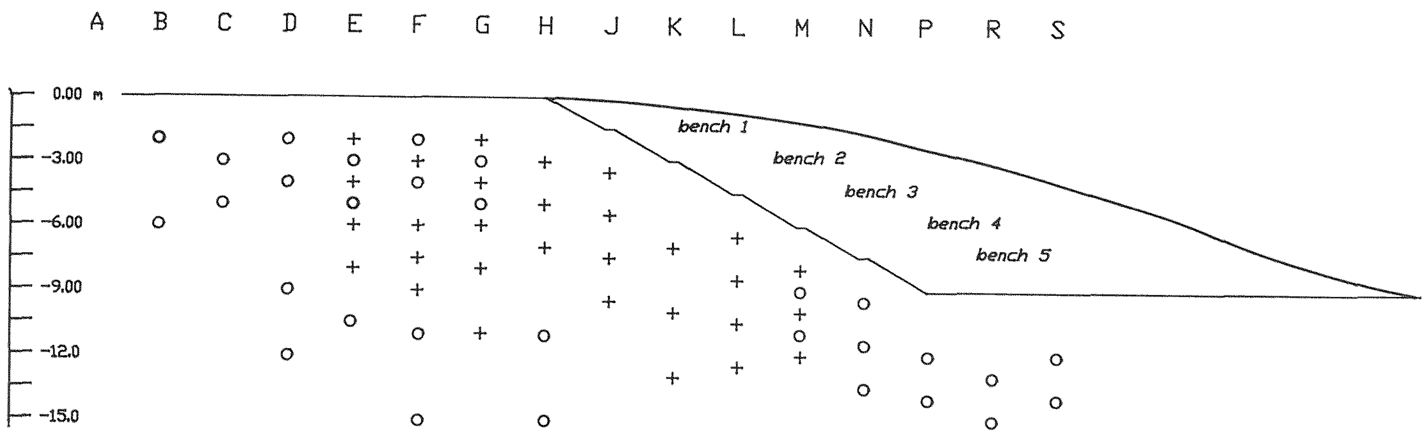


Figure 7-2 Piezometer layout design

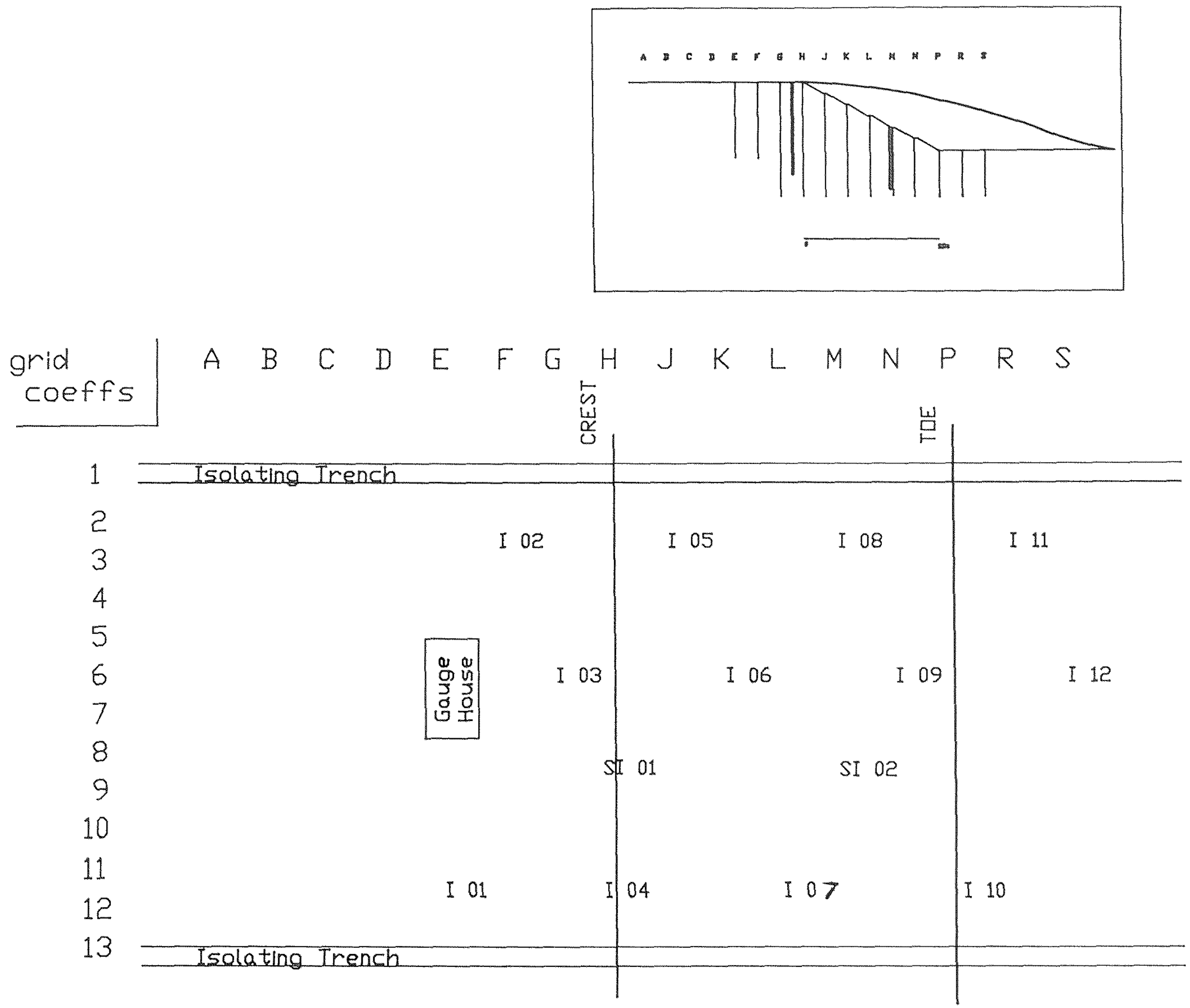
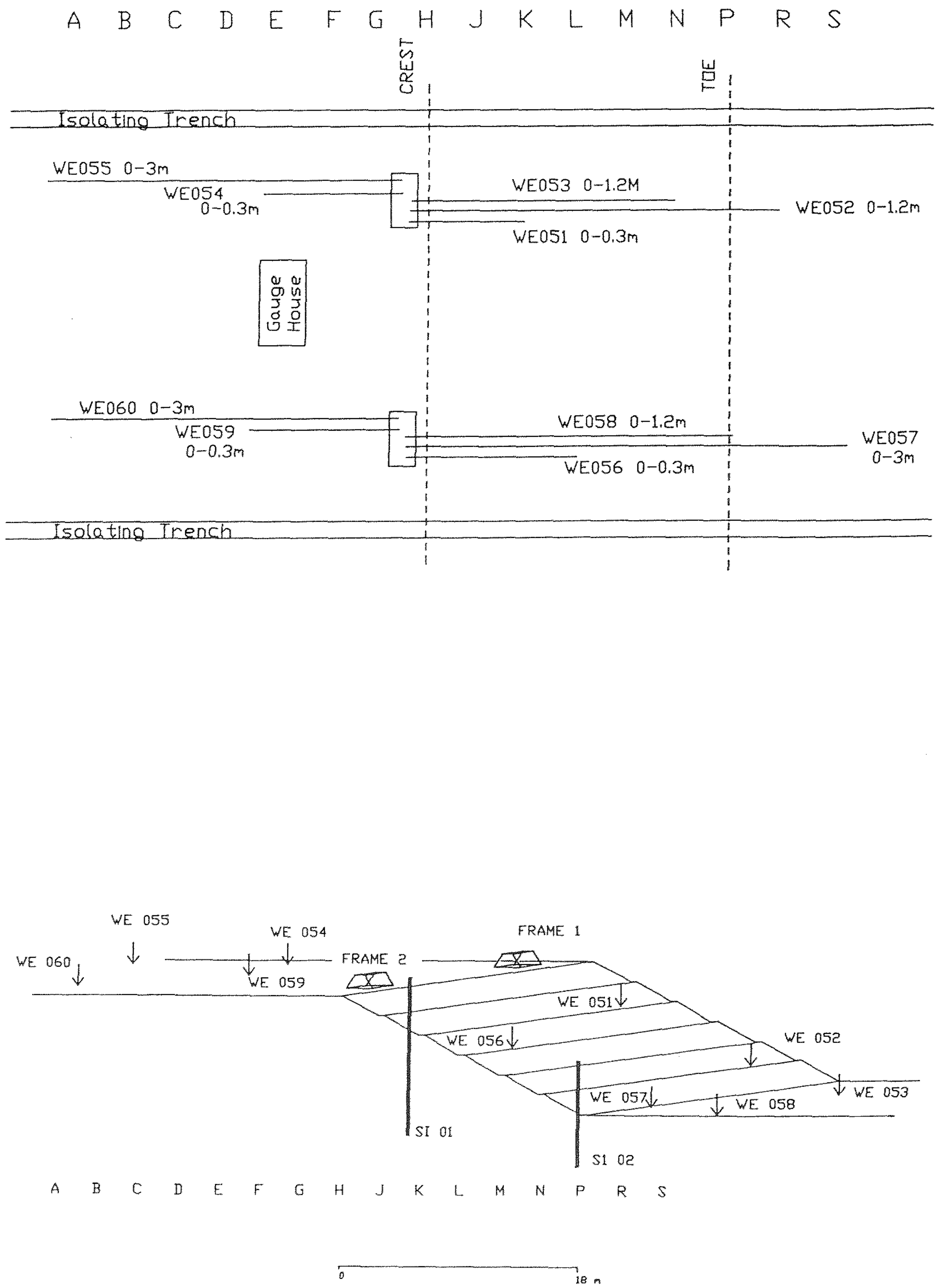


Figure 7-3 Inclinometer layout design



7-4 Wire extensometer and string inclinometer layout design

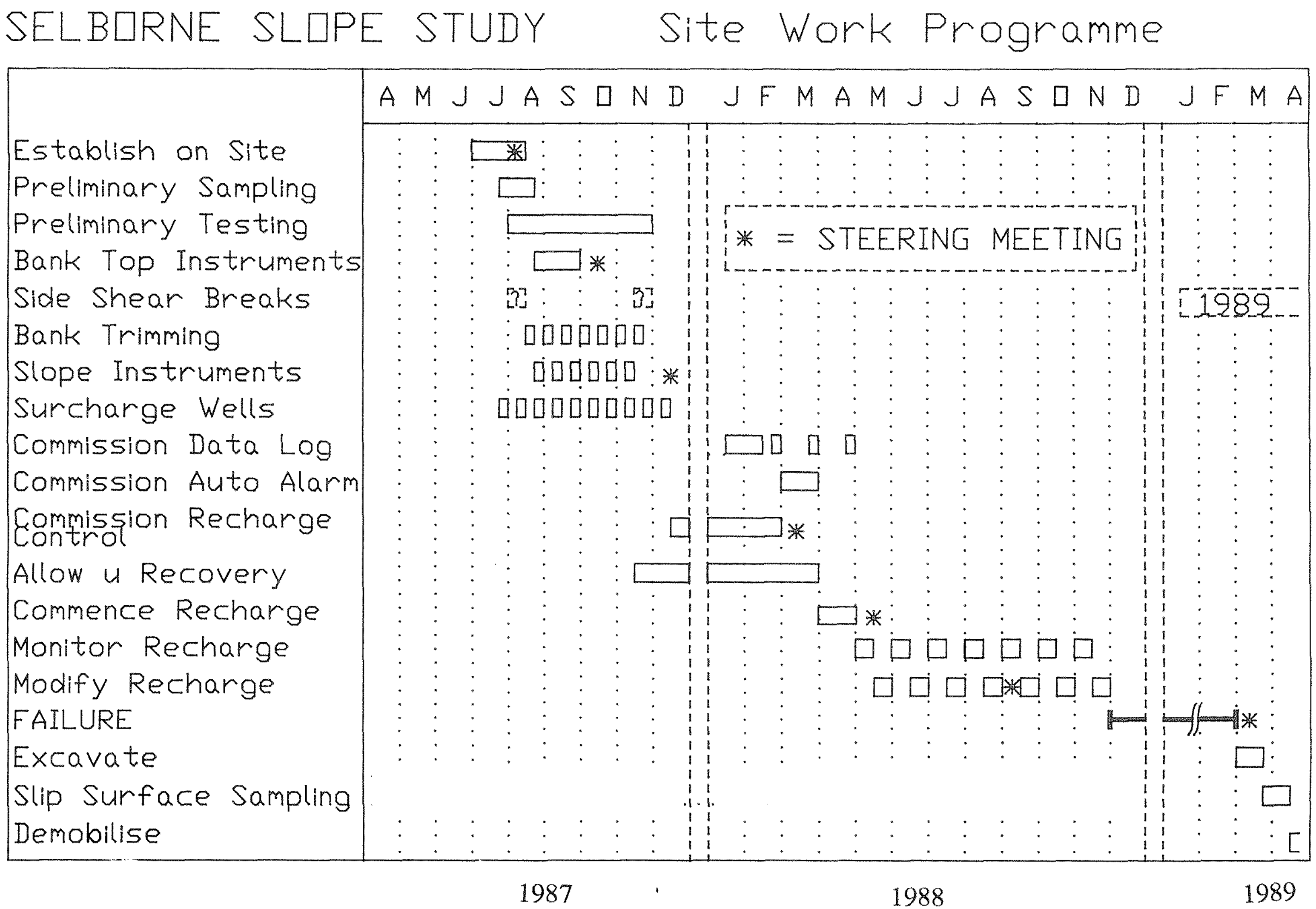


Figure 7-5 Intended Program of works

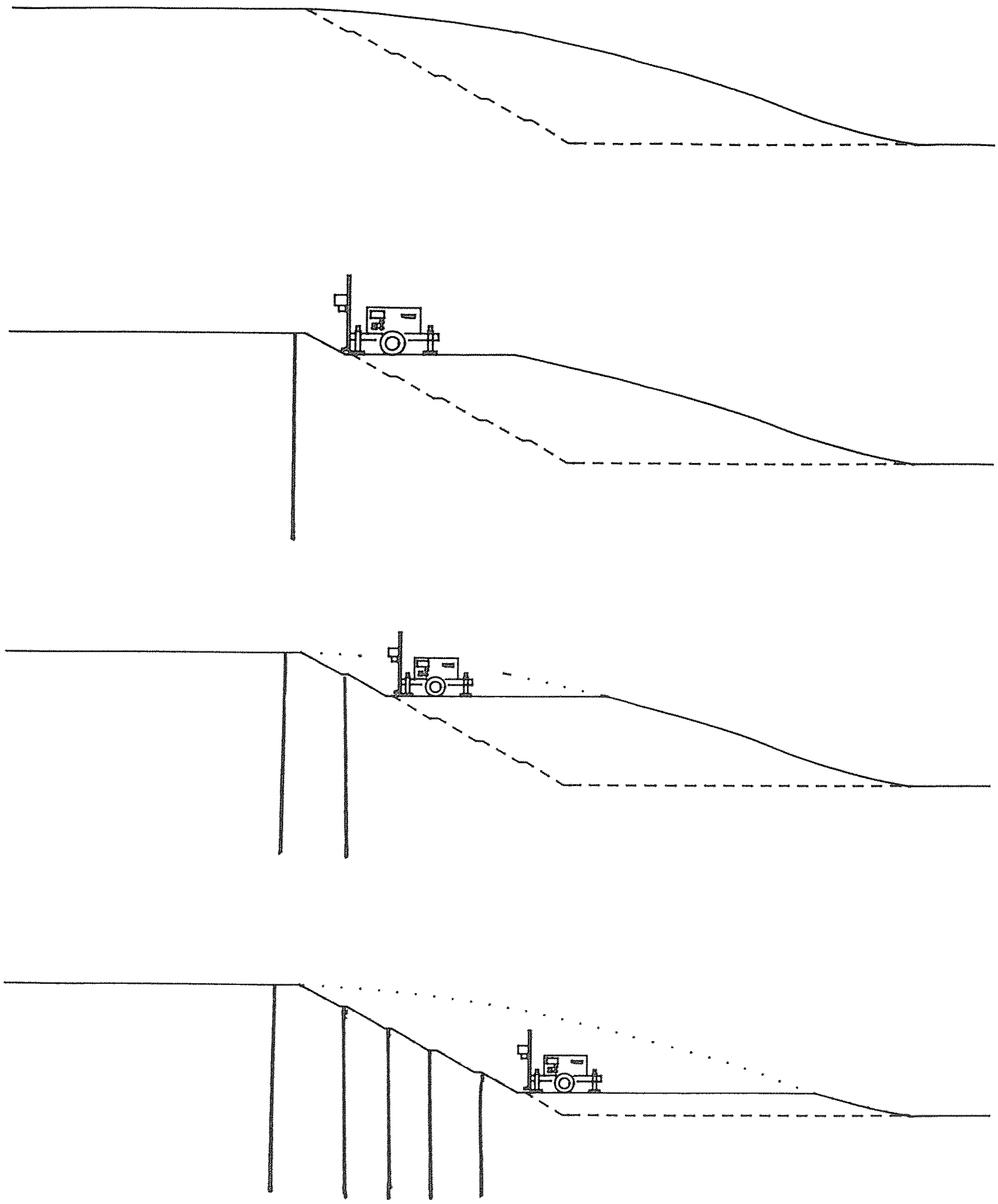
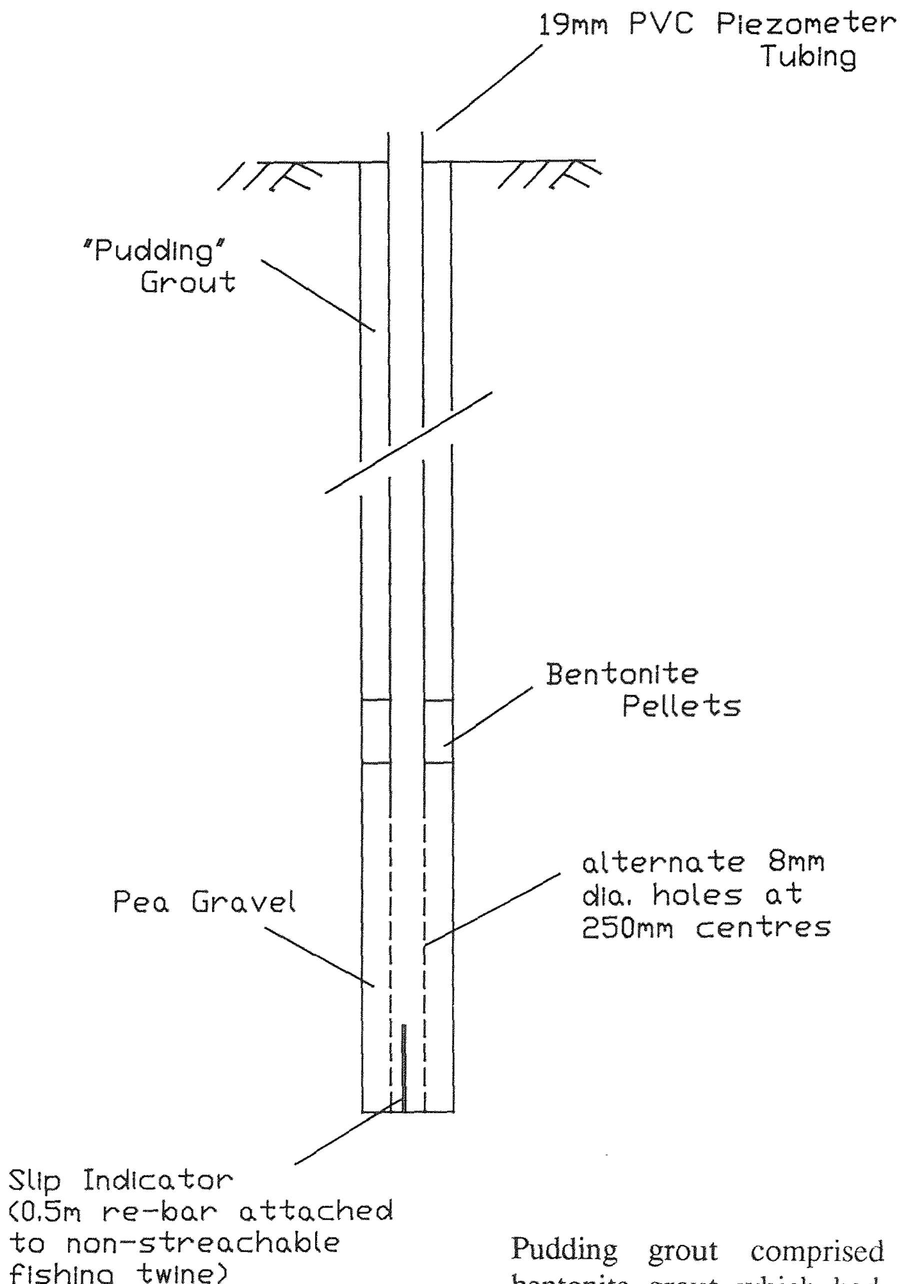


Figure 9-1 Slope formation and instrument installation procedure



Pudding grout comprised a cement bentonite grout which had added to it clay arisings produced by the instrumentation borehole drilling. The continuous flight auger drilling technique used produced small, smooth pellet like clay arisings.

Figure 9-2 Recharge well/slip indicator design

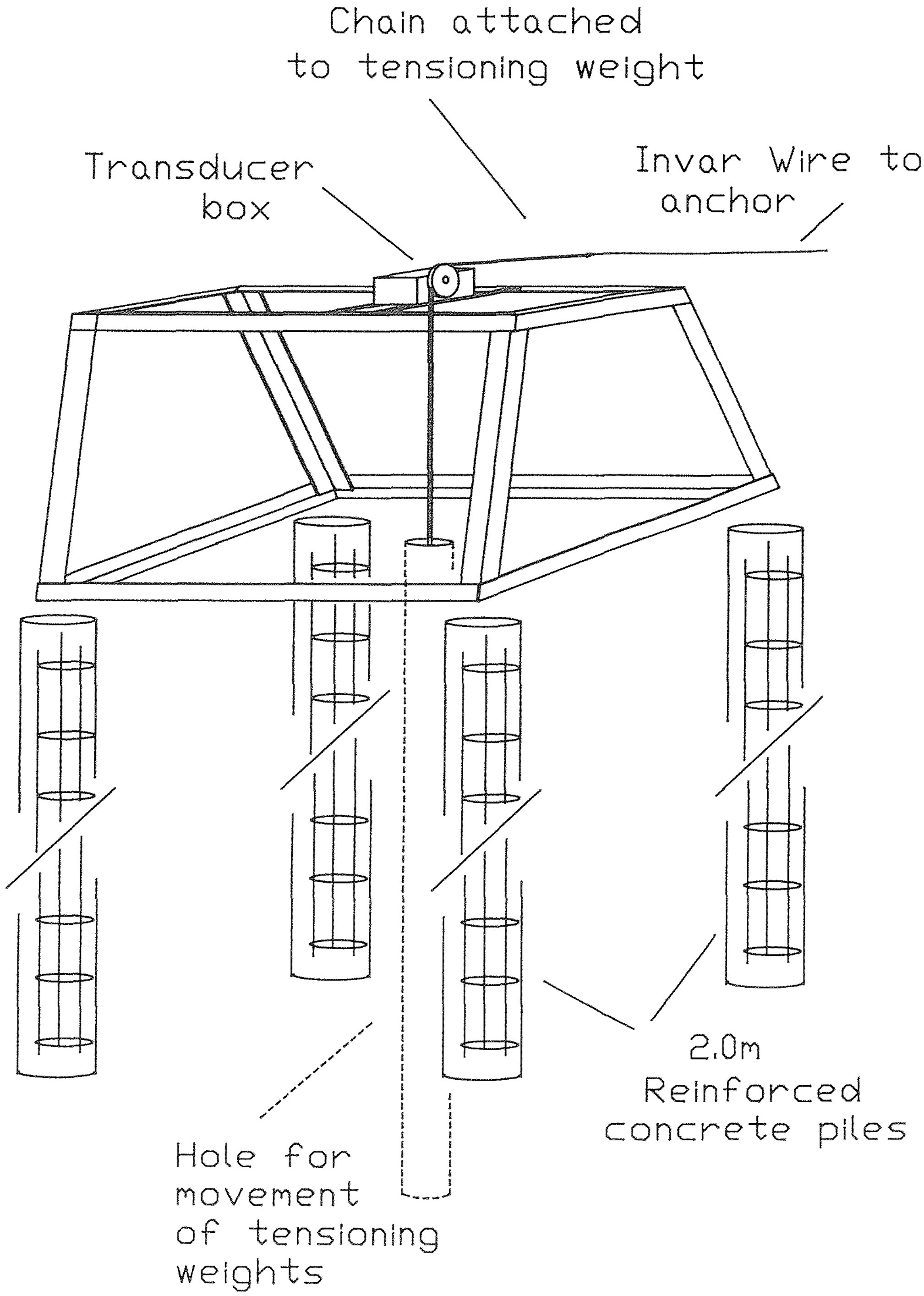
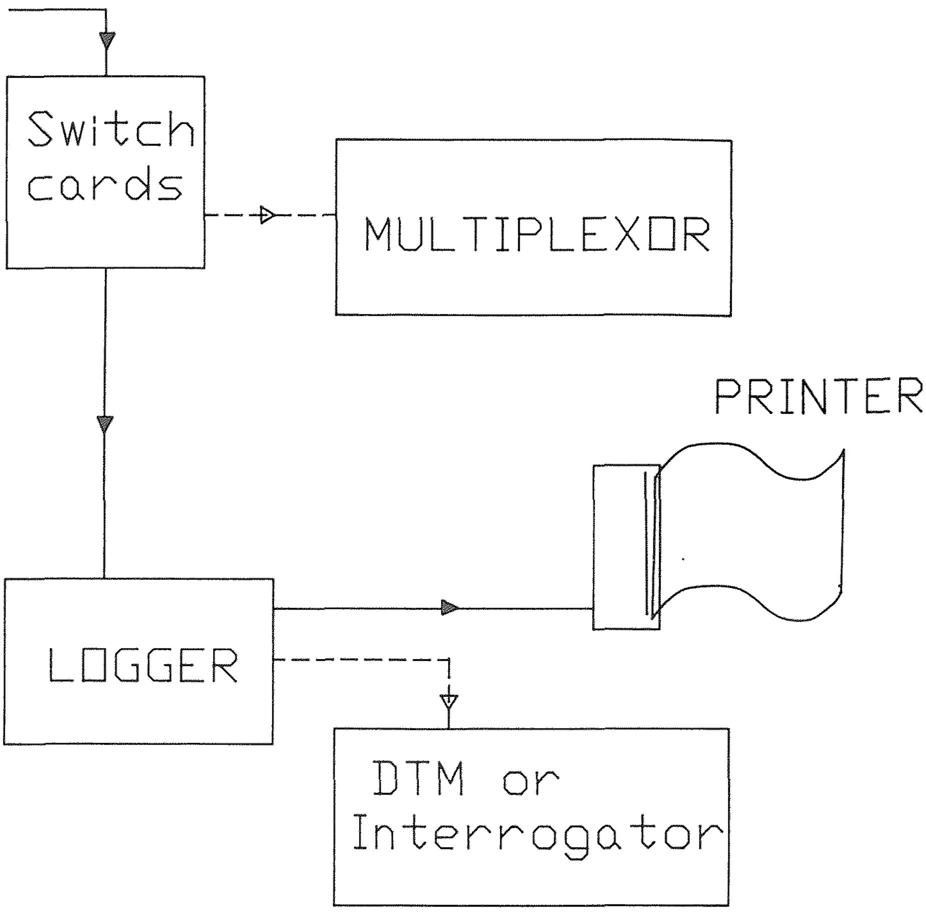


Figure 9-3 Wire extensometer frame installation details



(DTM = Data Transfer Module)

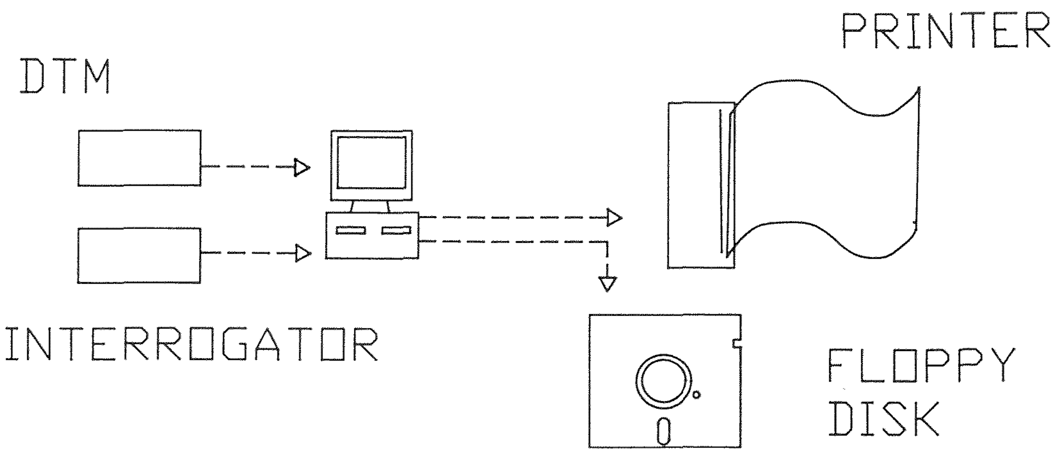
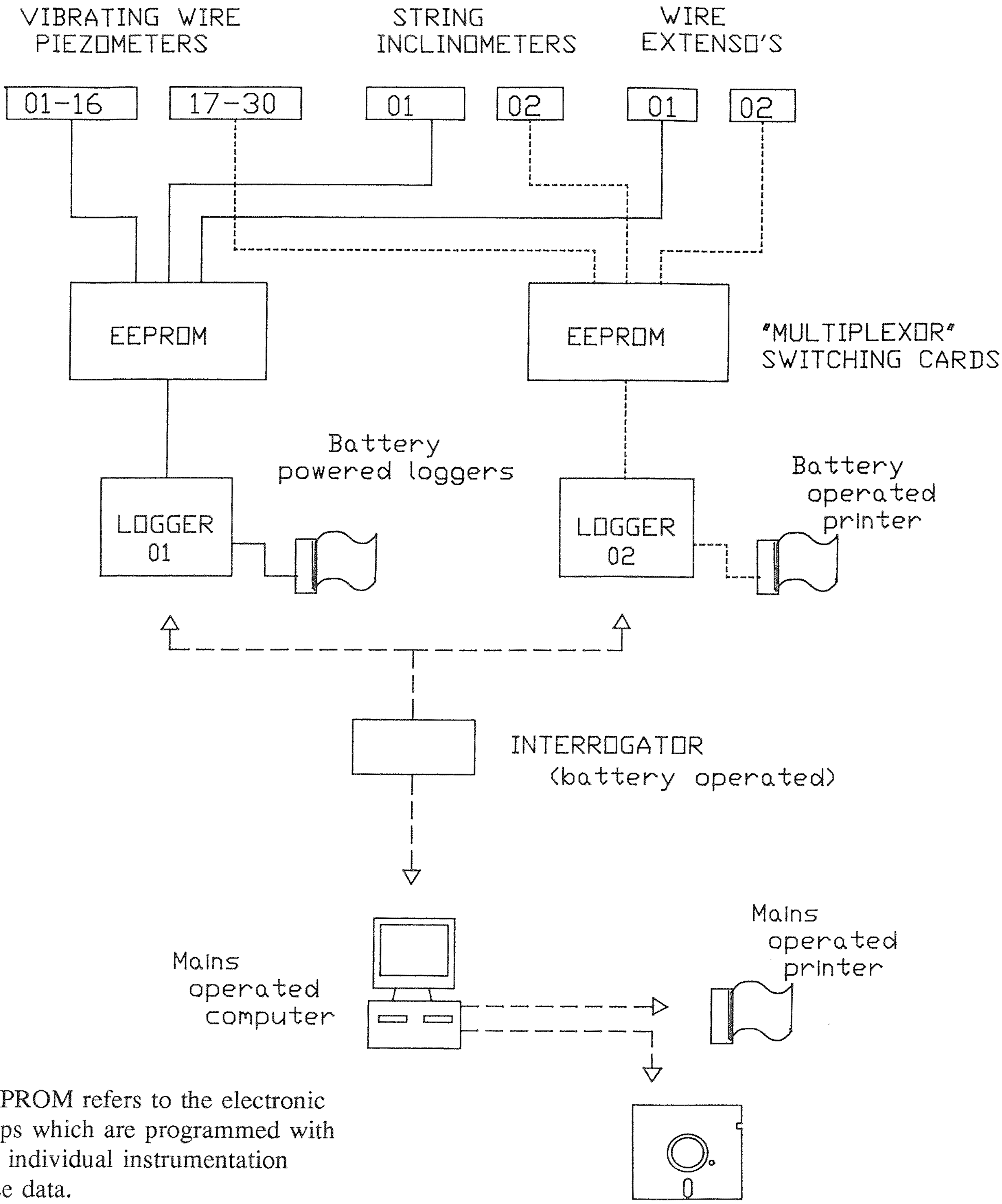


Figure 10-1 General logging procedure



EEPROM refers to the electronic chips which are programmed with the individual instrumentation base data.

Figure 10-2 Instrument logging paths

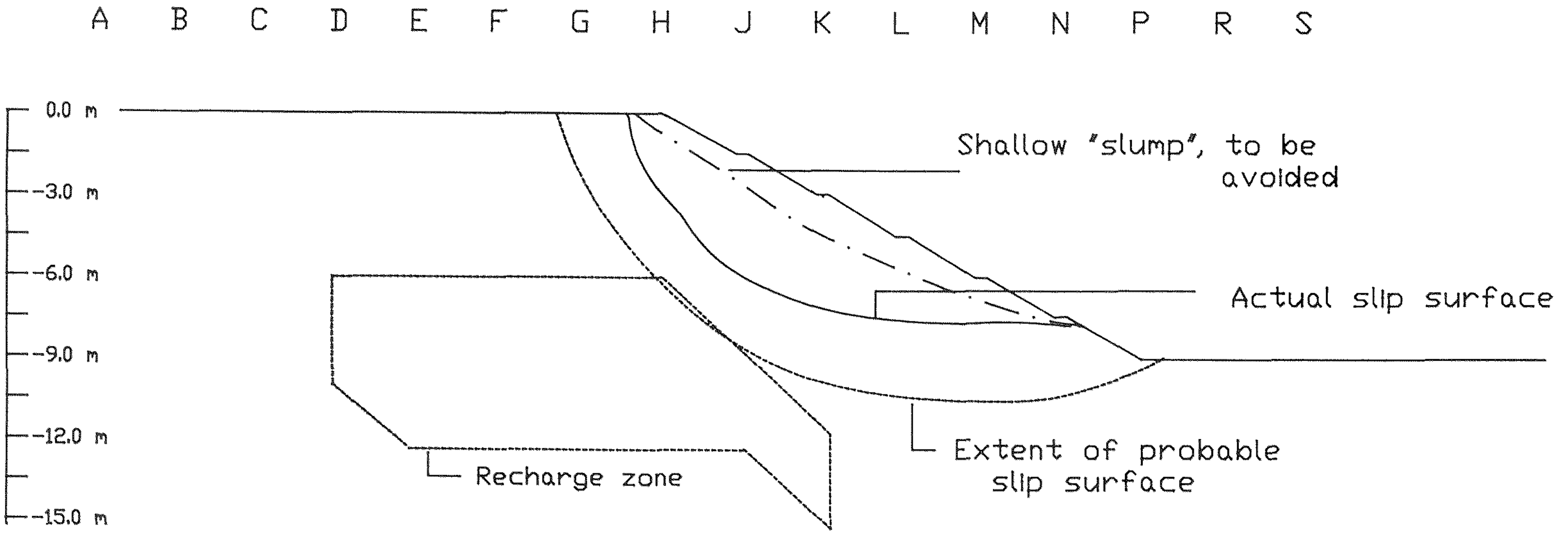


Figure 11-1 Intended mode of failure

Pneumatic piezometers installed to 8.0 metres depth.
Standpipe-piezometer installed to 8.0 metres depth (0.5 metre filter length).
Recharge well response lengths 6.0-10.0 metres depth.

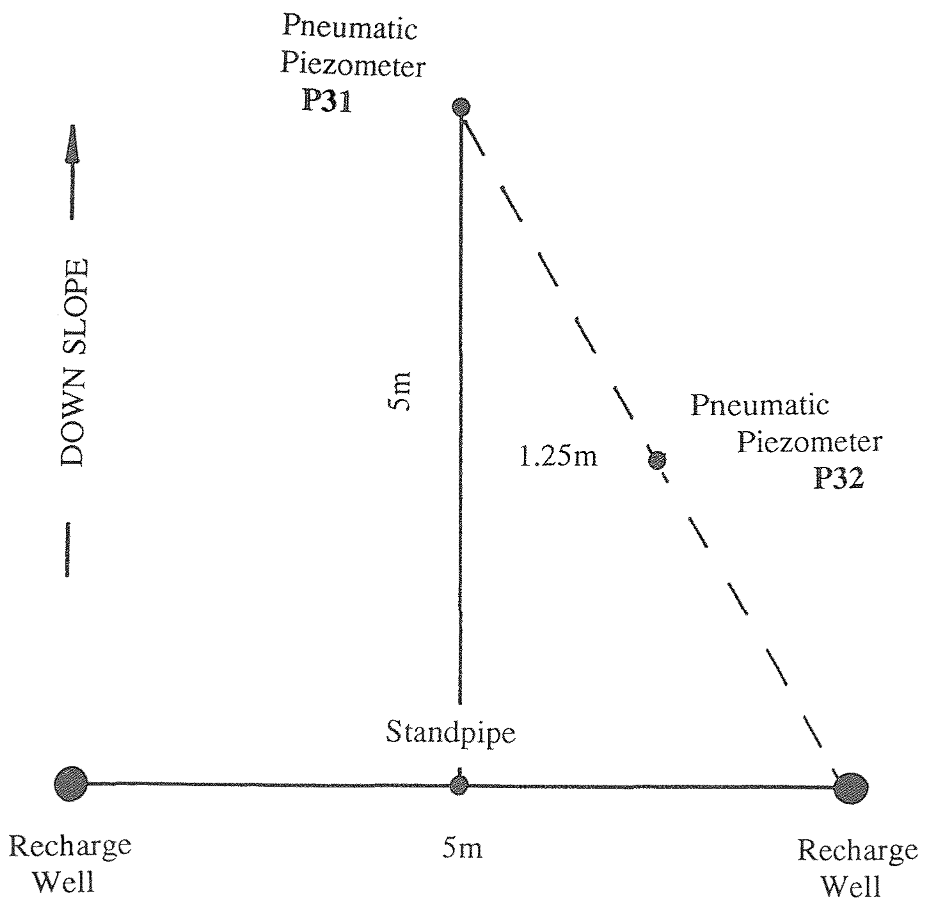


Figure 11-2 Recharge test bed

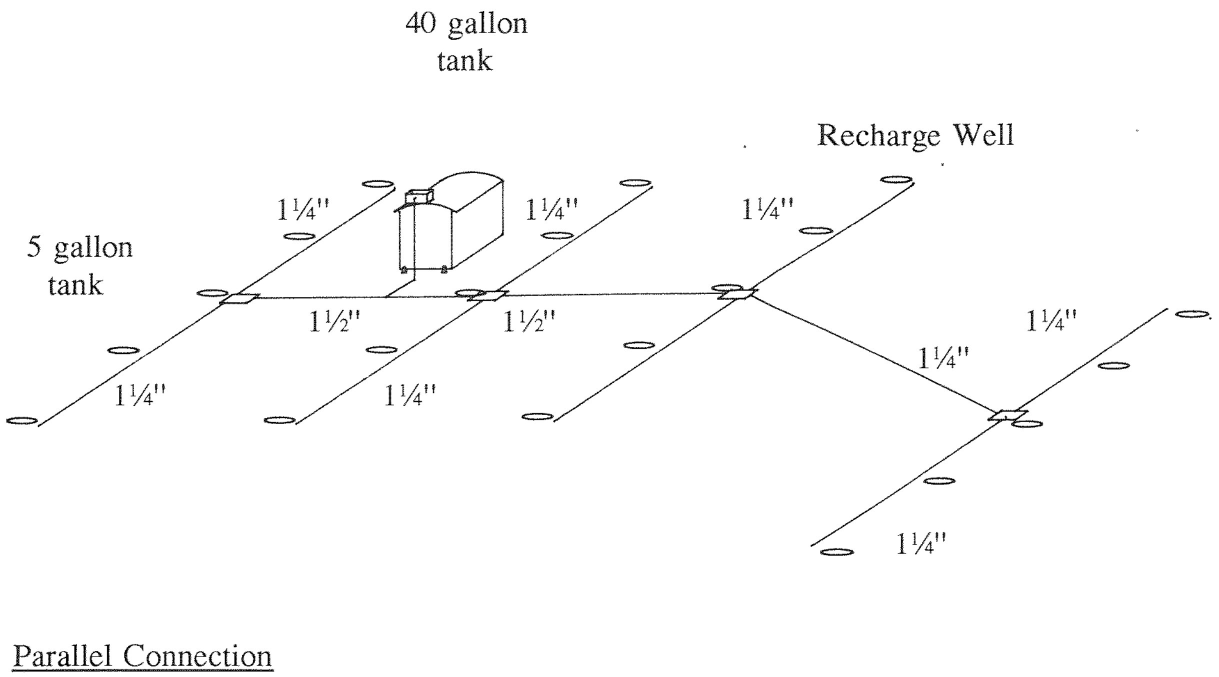


Figure 11-3 Sketchmatic view of recharge system

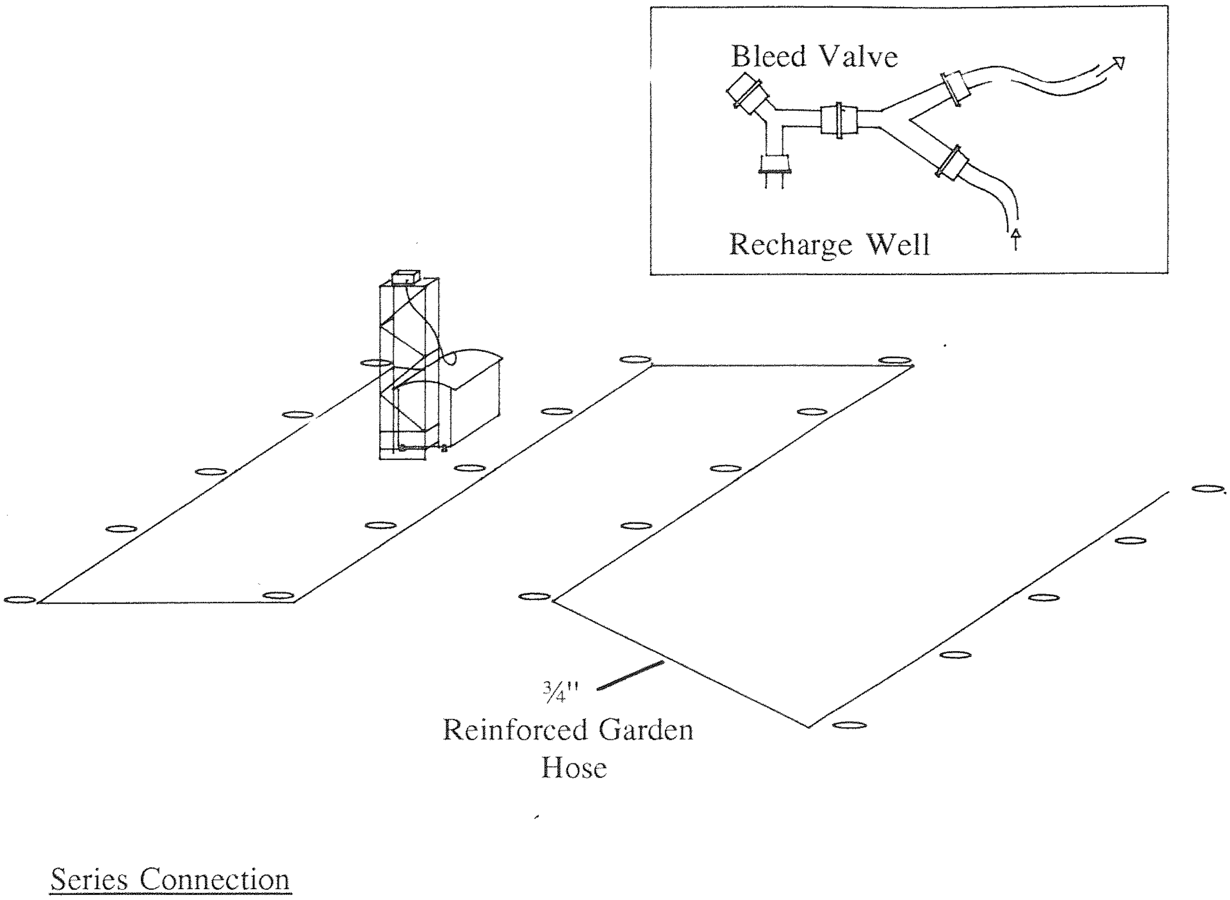
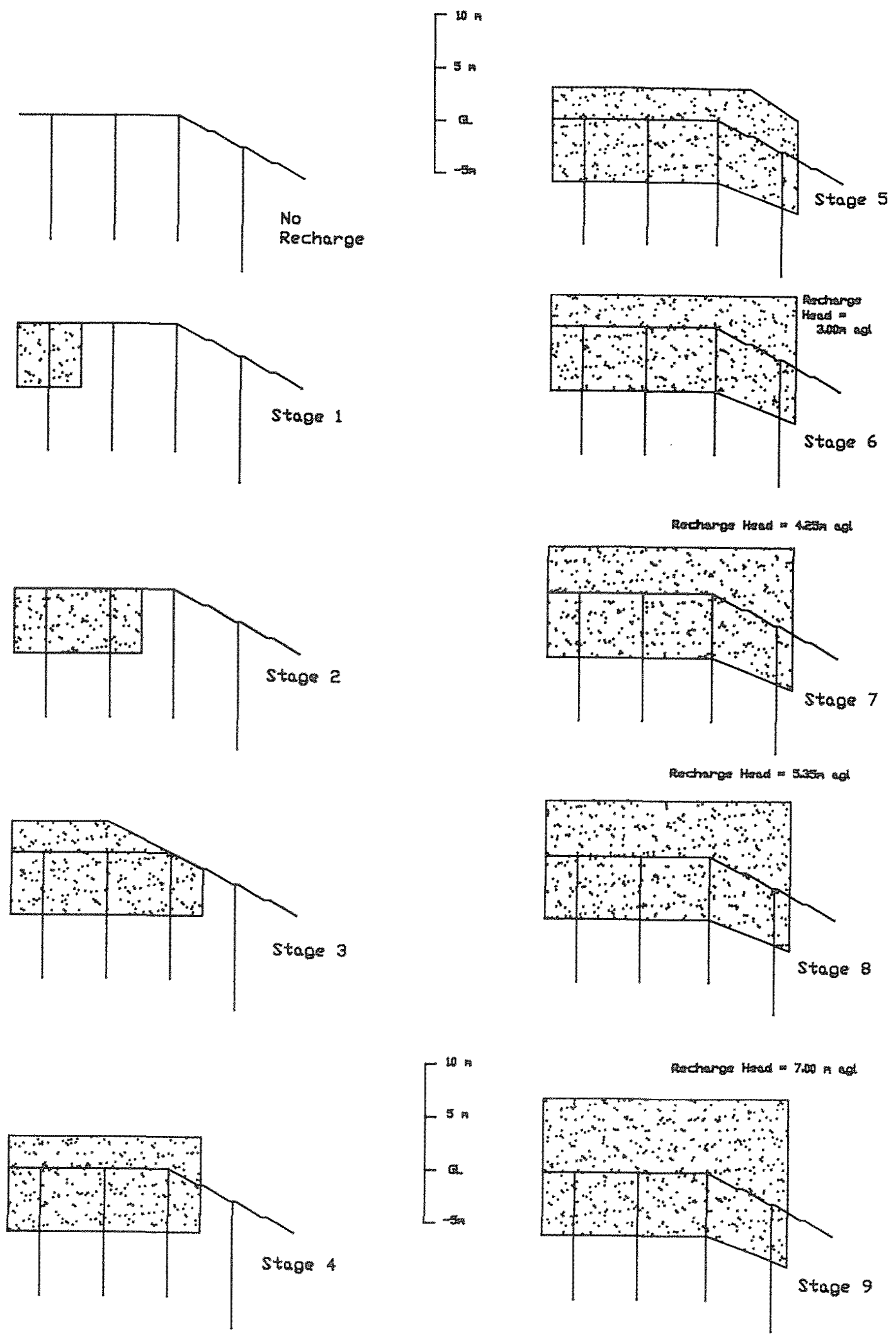


Figure 11-4 Sketchmatic view of revised recharge system



Shaded area indicates intended extent and applied head for recharge.

Figure 11-5 Summary of recharge events - the nine steps to failure

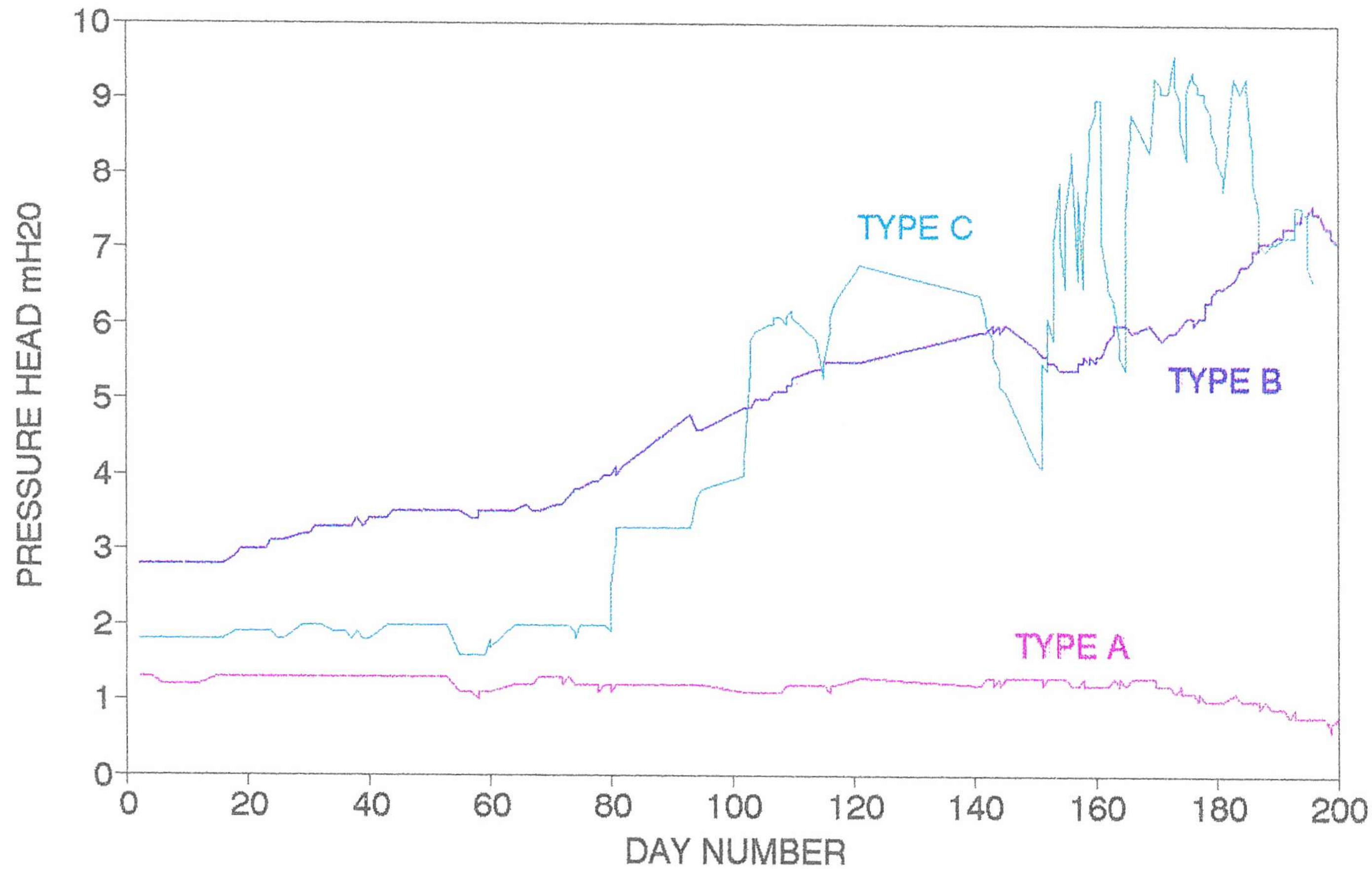


Figure 11-6 A,B & C type piezometer responses to recharge

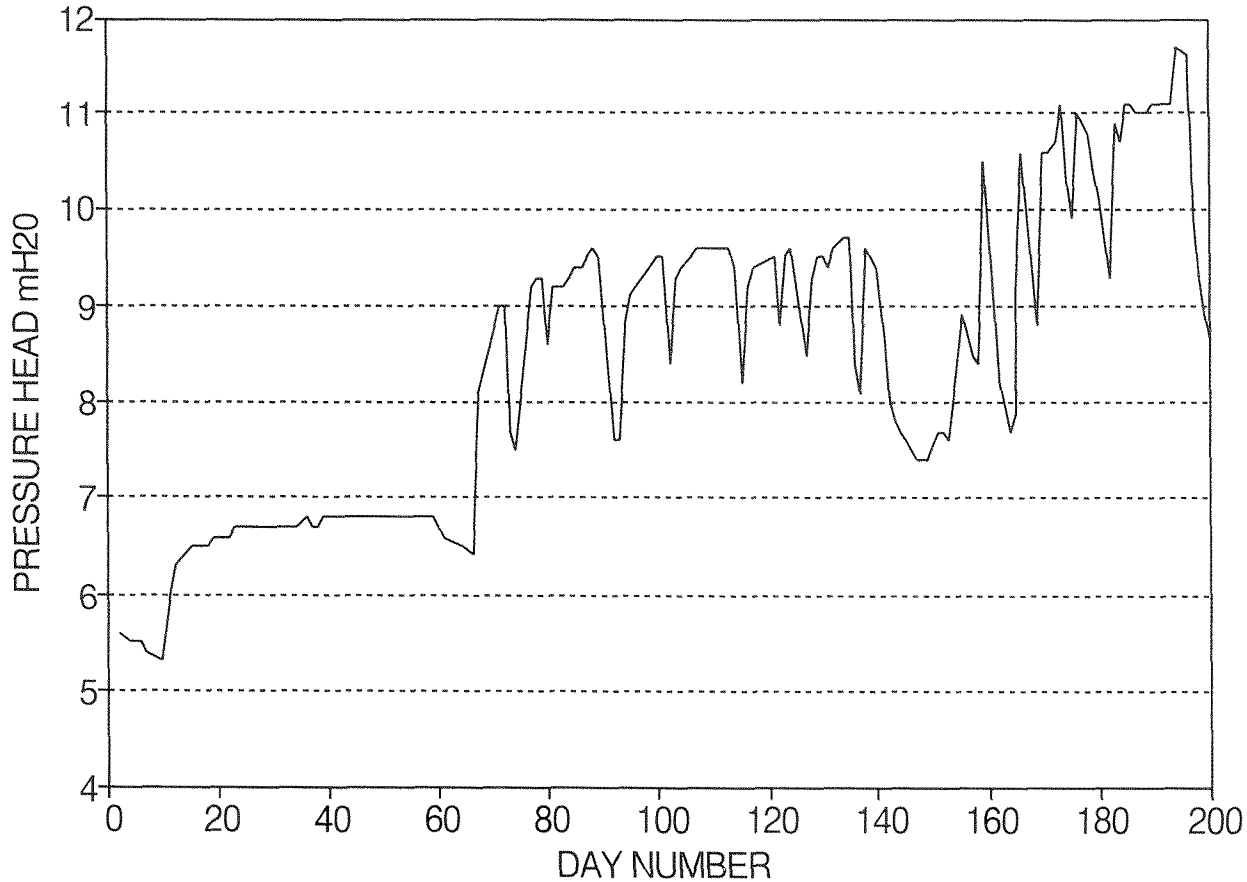


Figure 11-7a PP24 January to August 1989 (Days zero to 200)

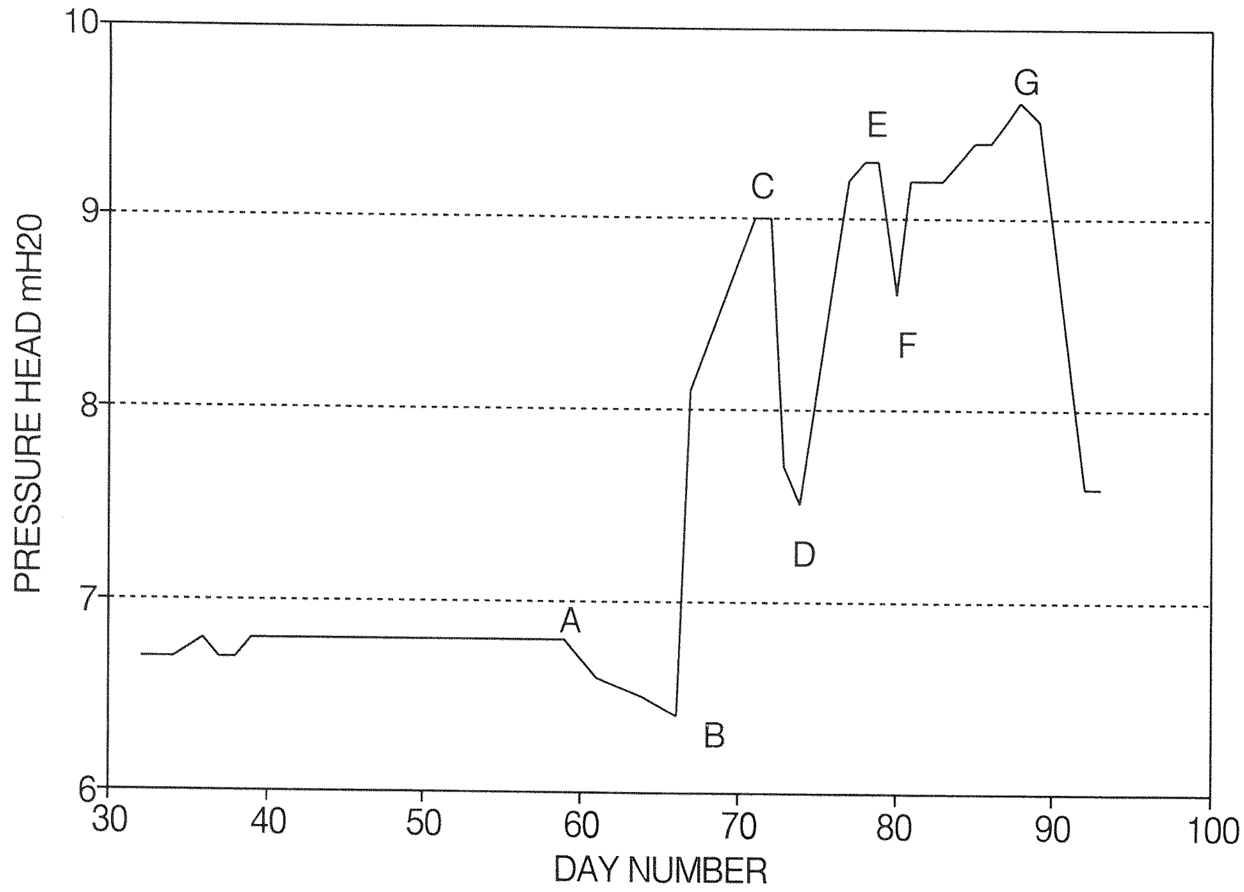


Figure 11-7b PP24 February to April 1989 (Days 30 to 90)

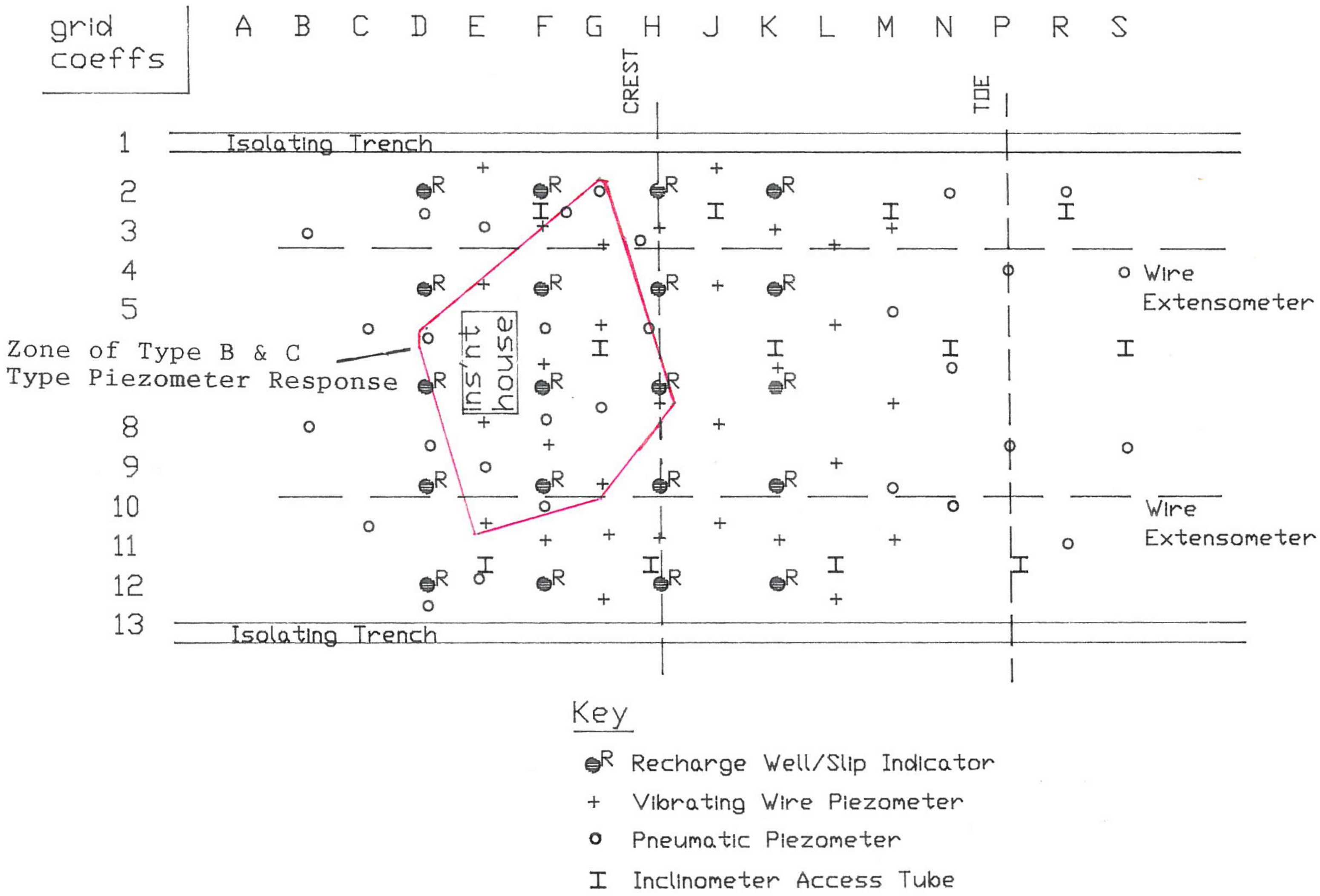


Figure 11-8 Location of B & C type response piezometers - Plan

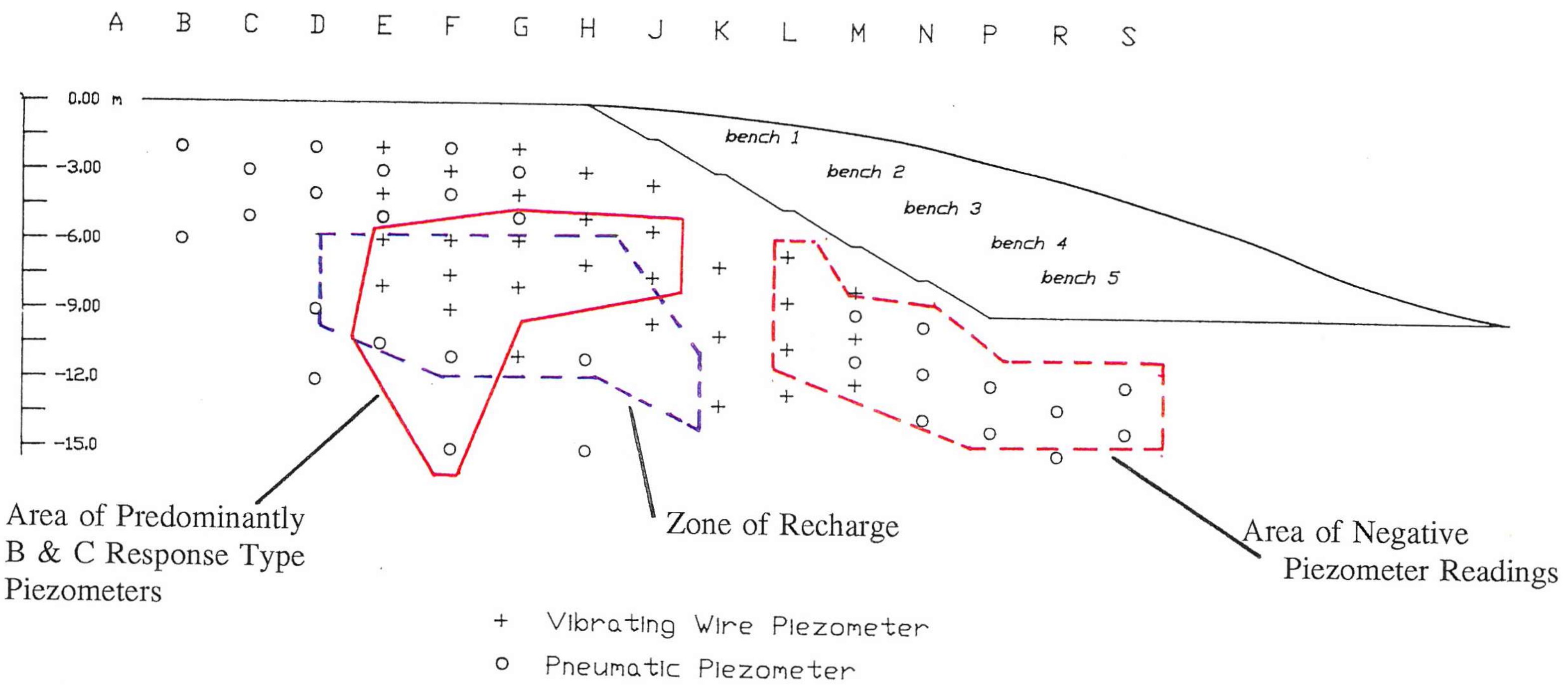


Figure 11-9 Location of B & C response type piezometers - Section

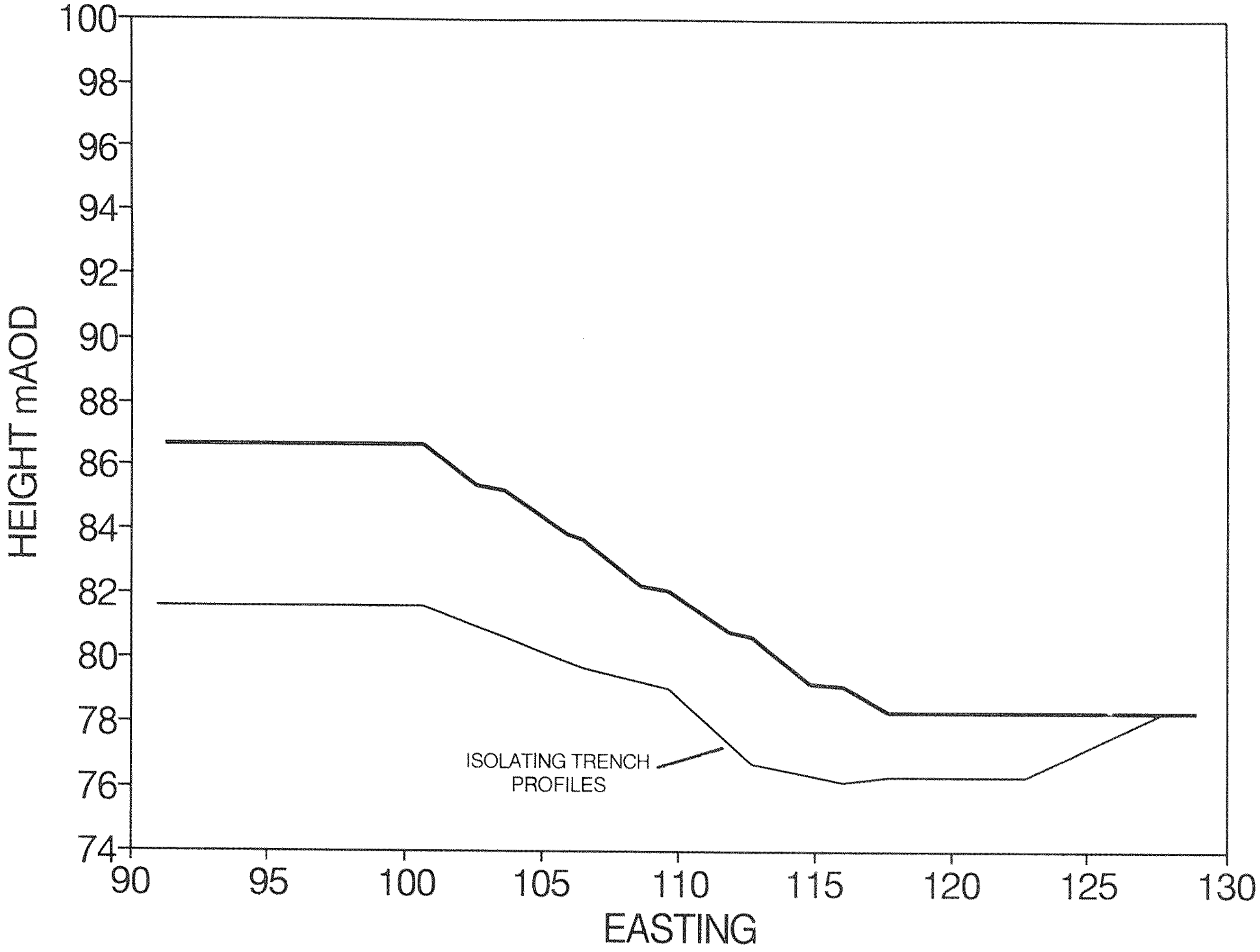


Figure 12-1 Isolating trench profiles

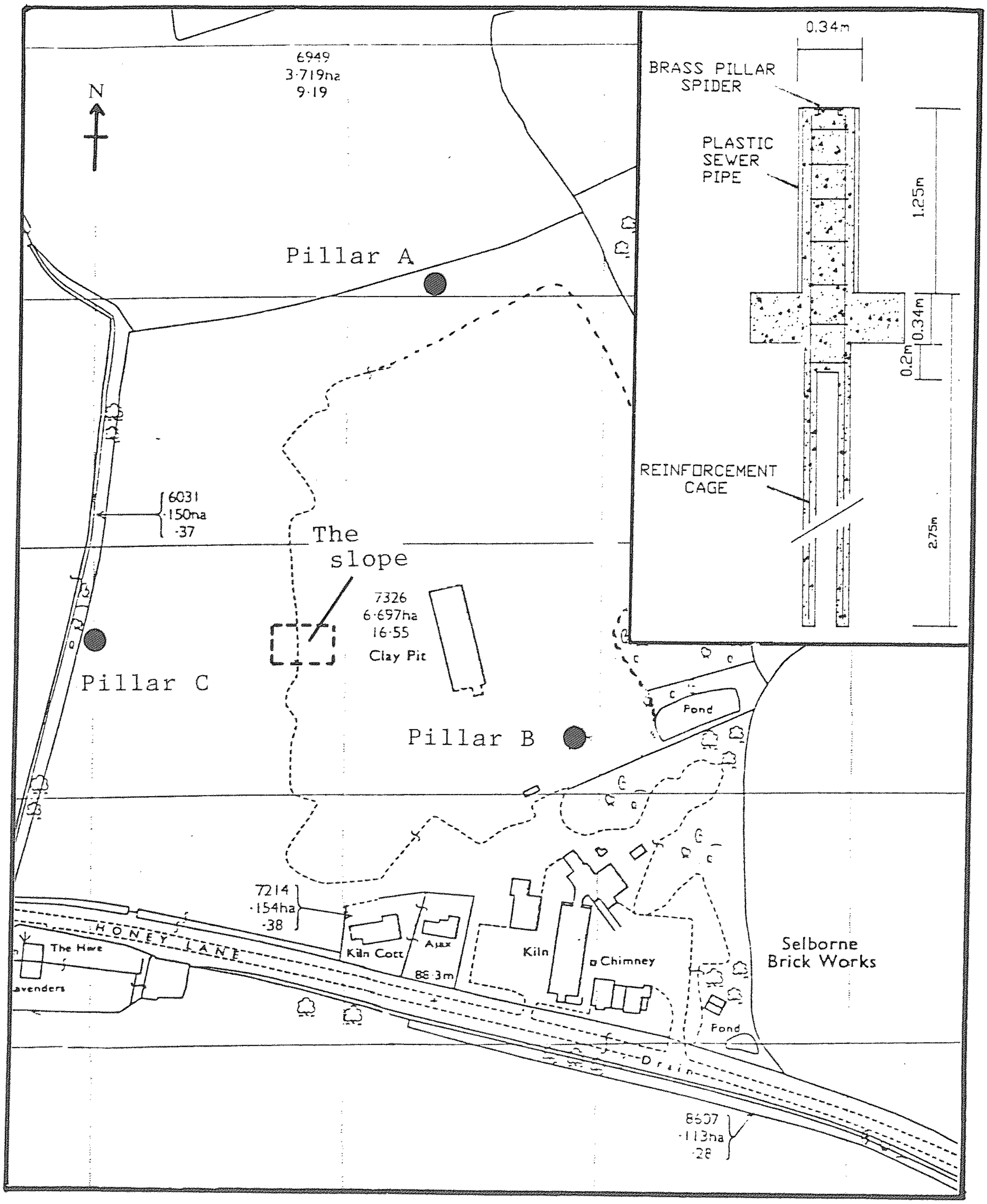


Figure 13-1 Location of survey reference pillars

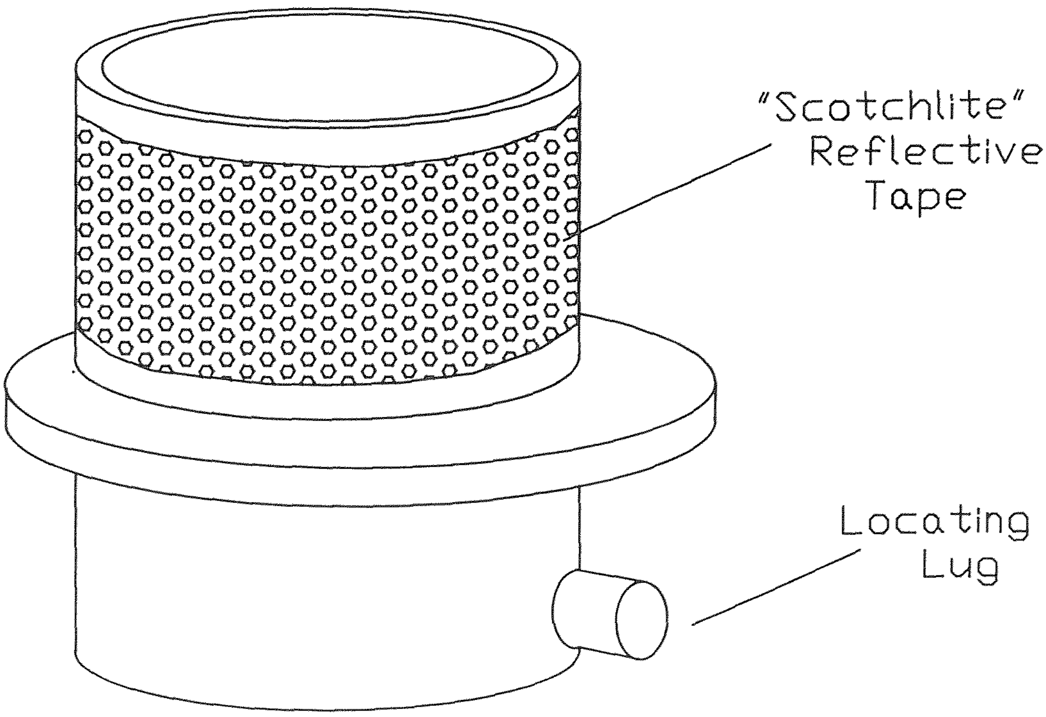


Figure 13-2 Reflective inclinometer top targets

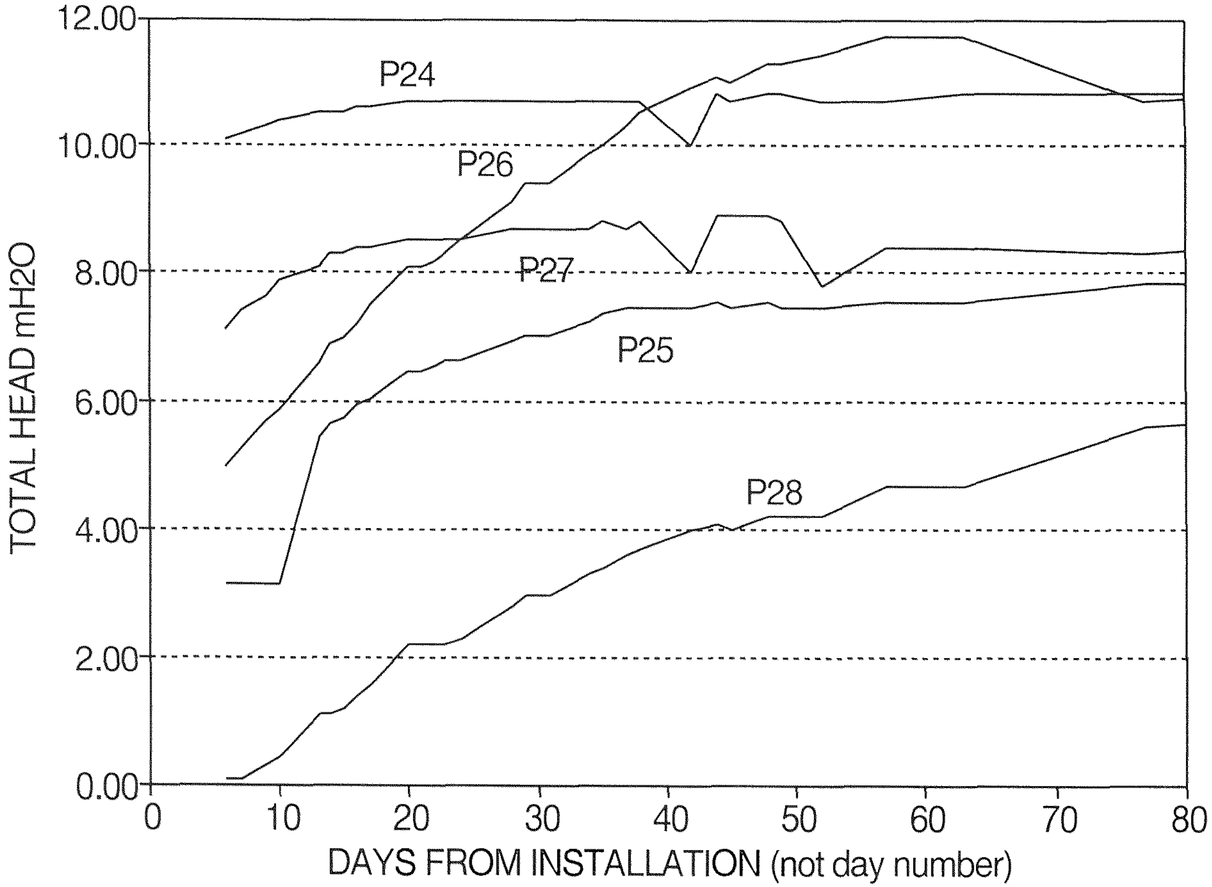


Figure 14-2 Installation lag times, total heads.

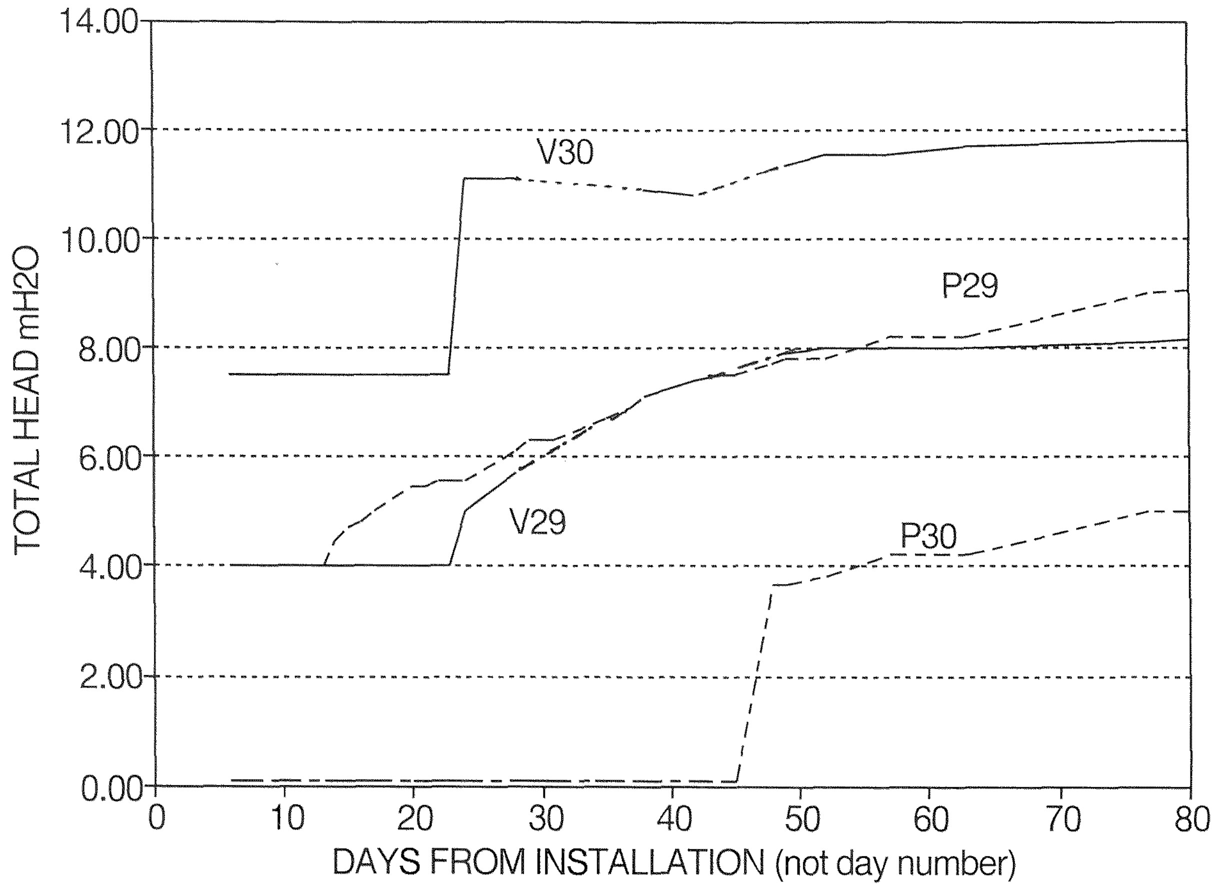


Figure 14-3 Installation lag times, total heads.

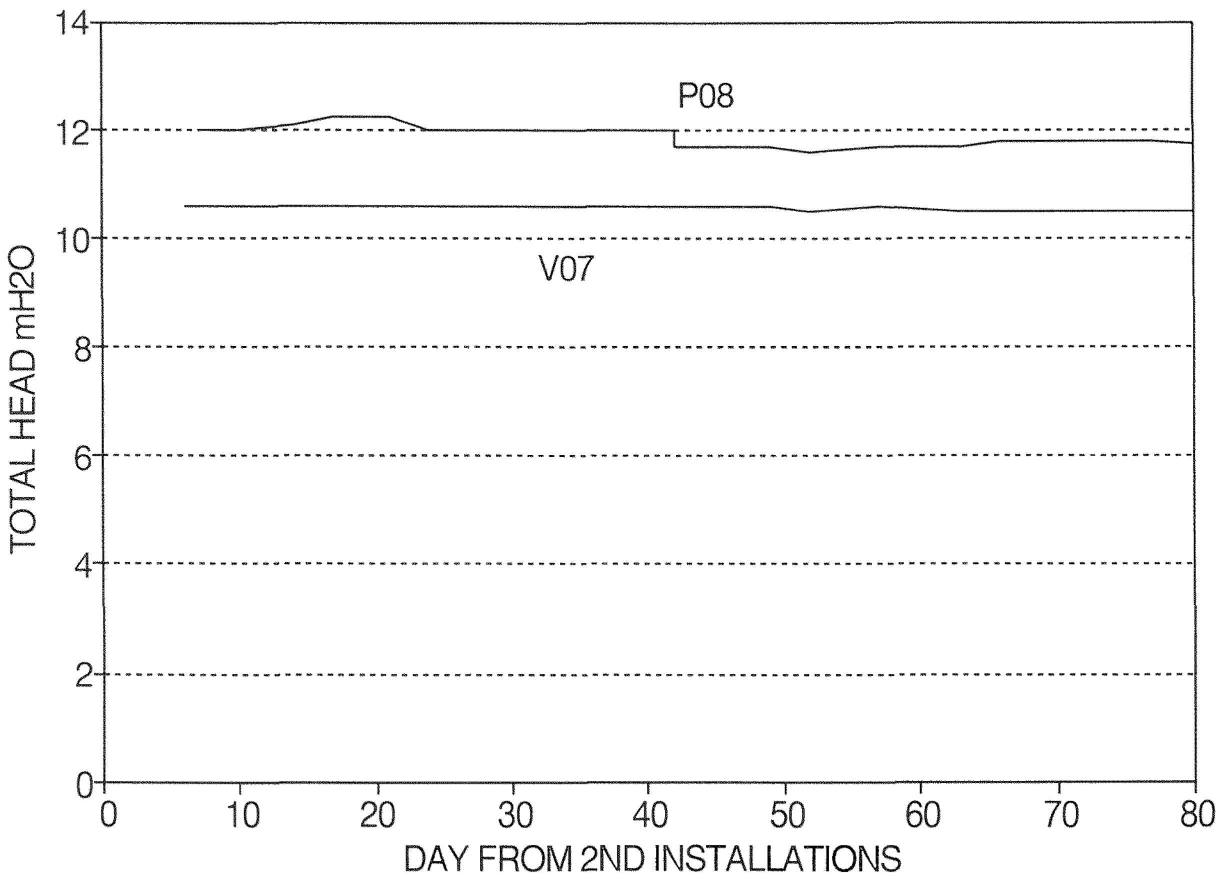


Figure 14-4 Overlay to 14-2 & 14-3 showing total heads for P08 & V07

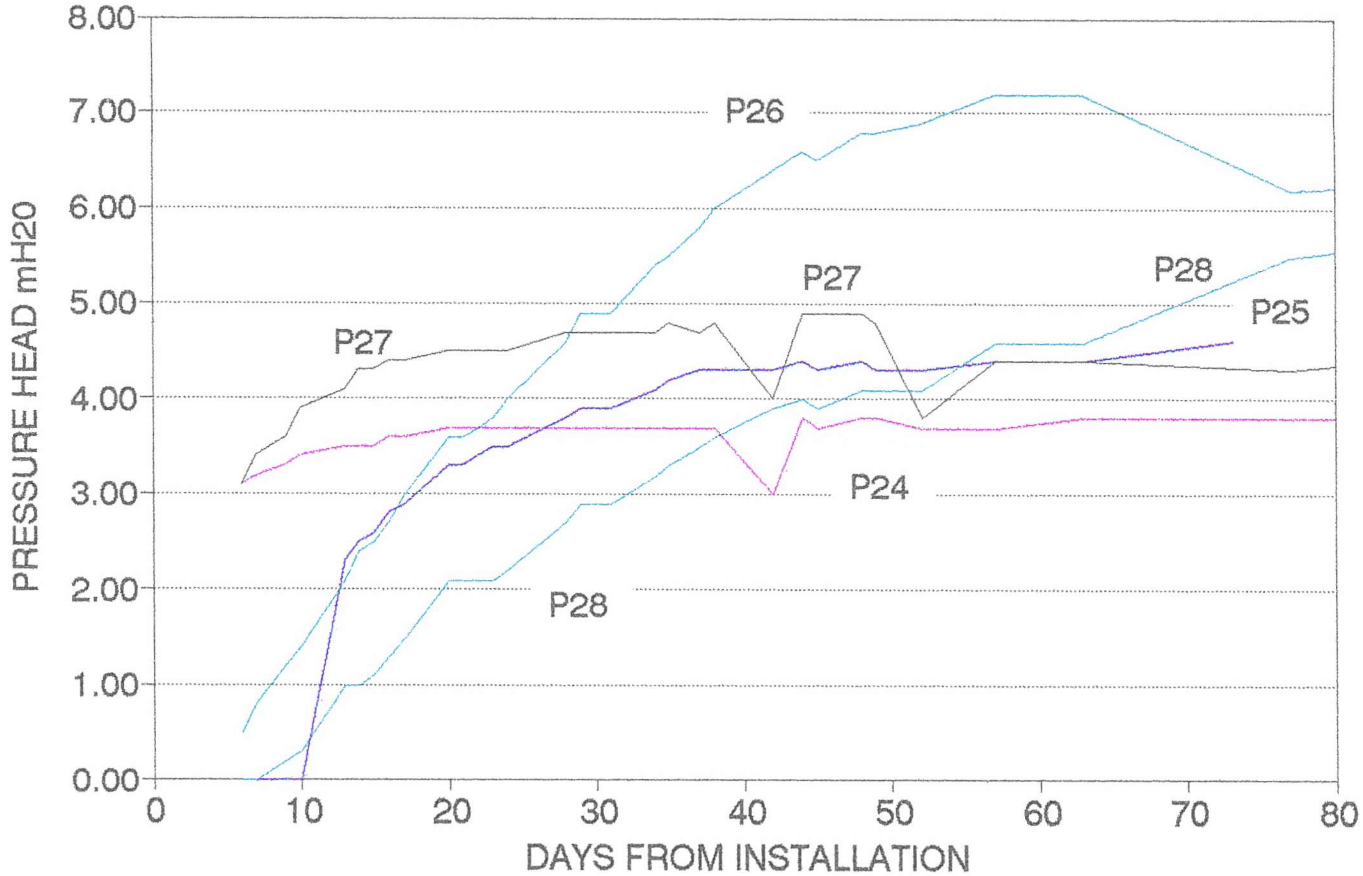


Figure 14-5 Time lag in terms of pressure heads

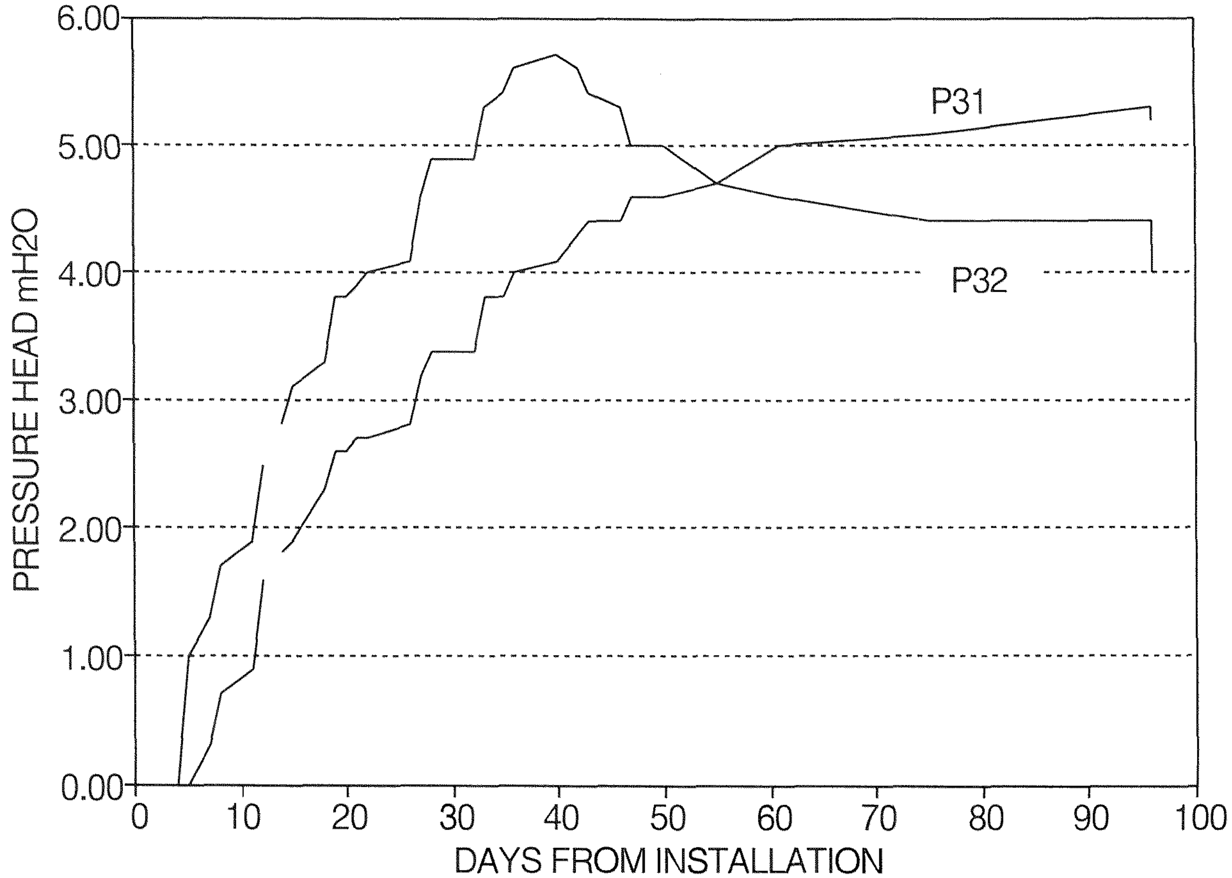


Figure 14-6 Lag times - Recharge test bed

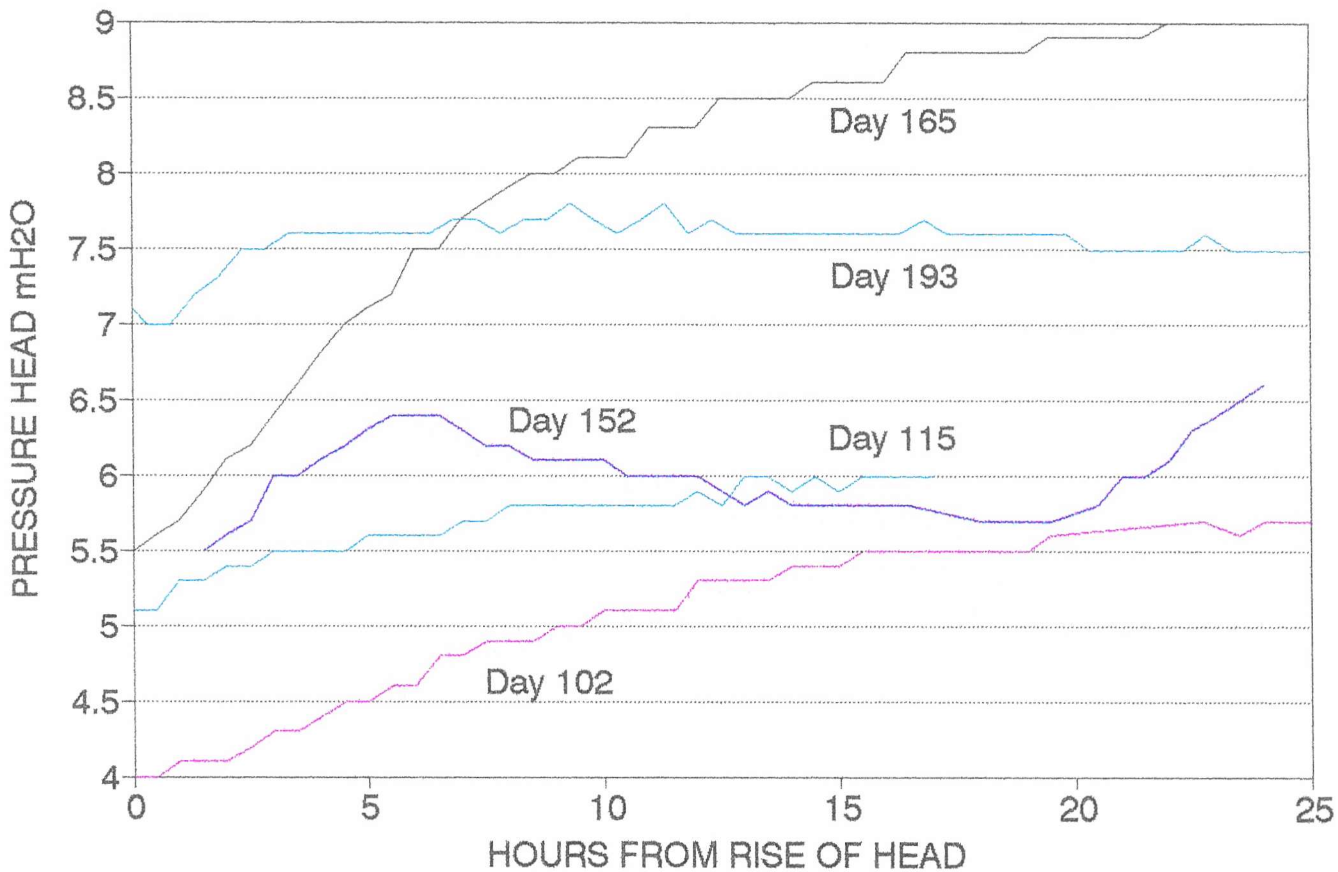


Figure 14-7 V13 response to recharge event

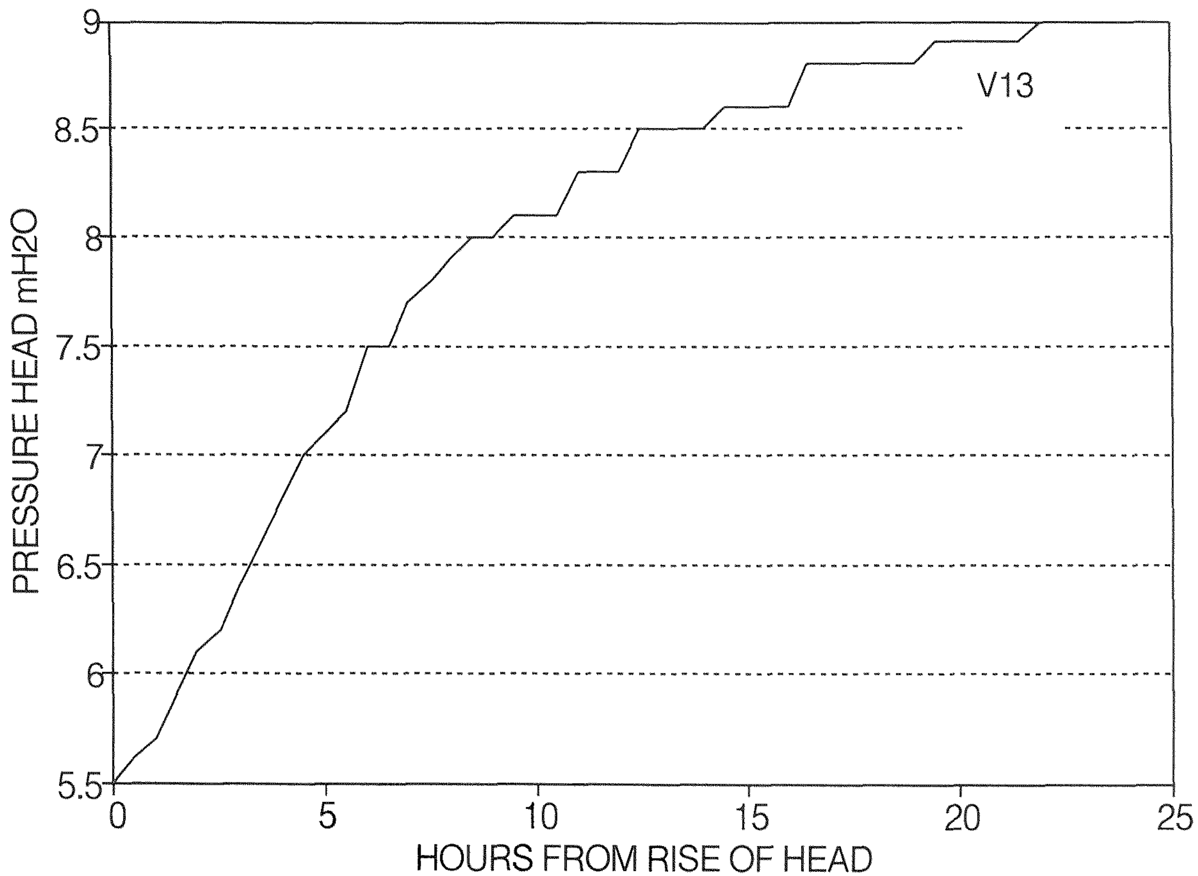


Figure 14-8 V13 day 165

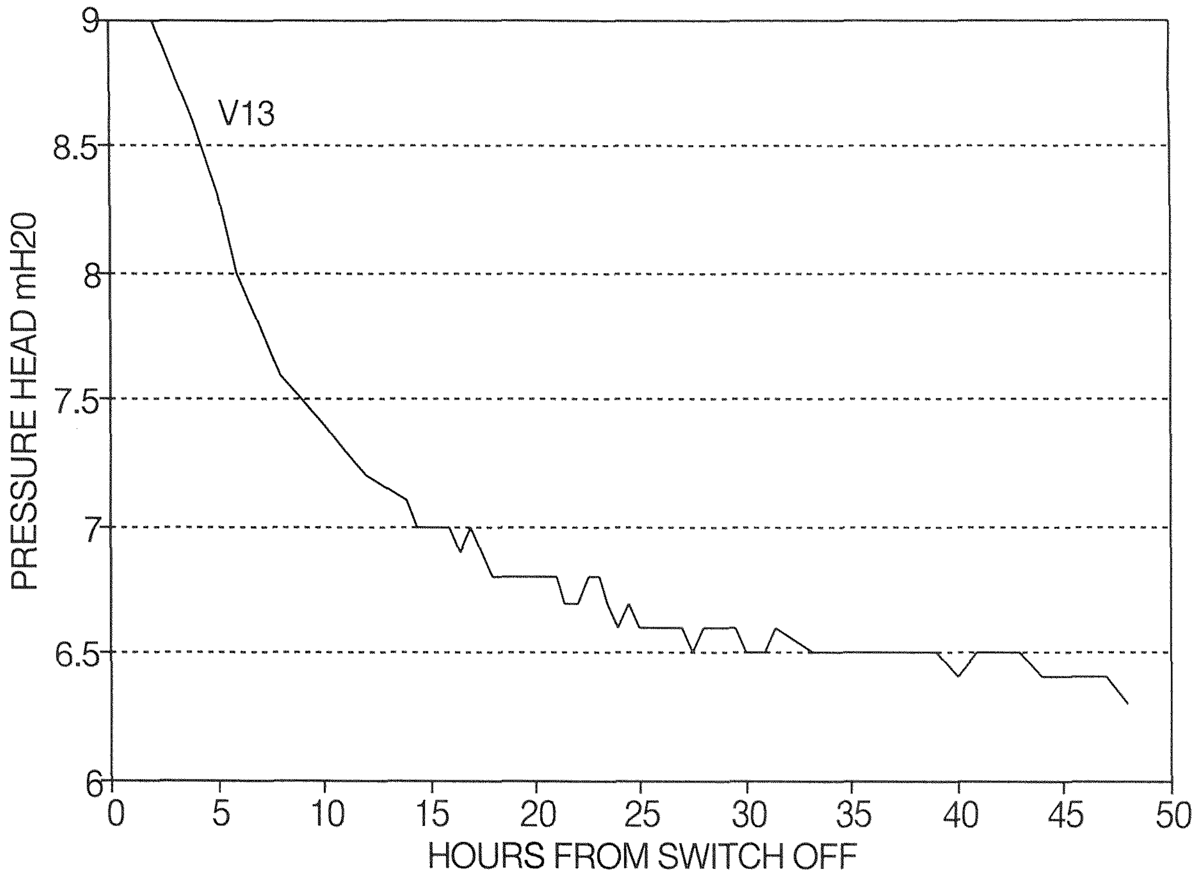


Figure 14-9 V13 Switch off event - day 162

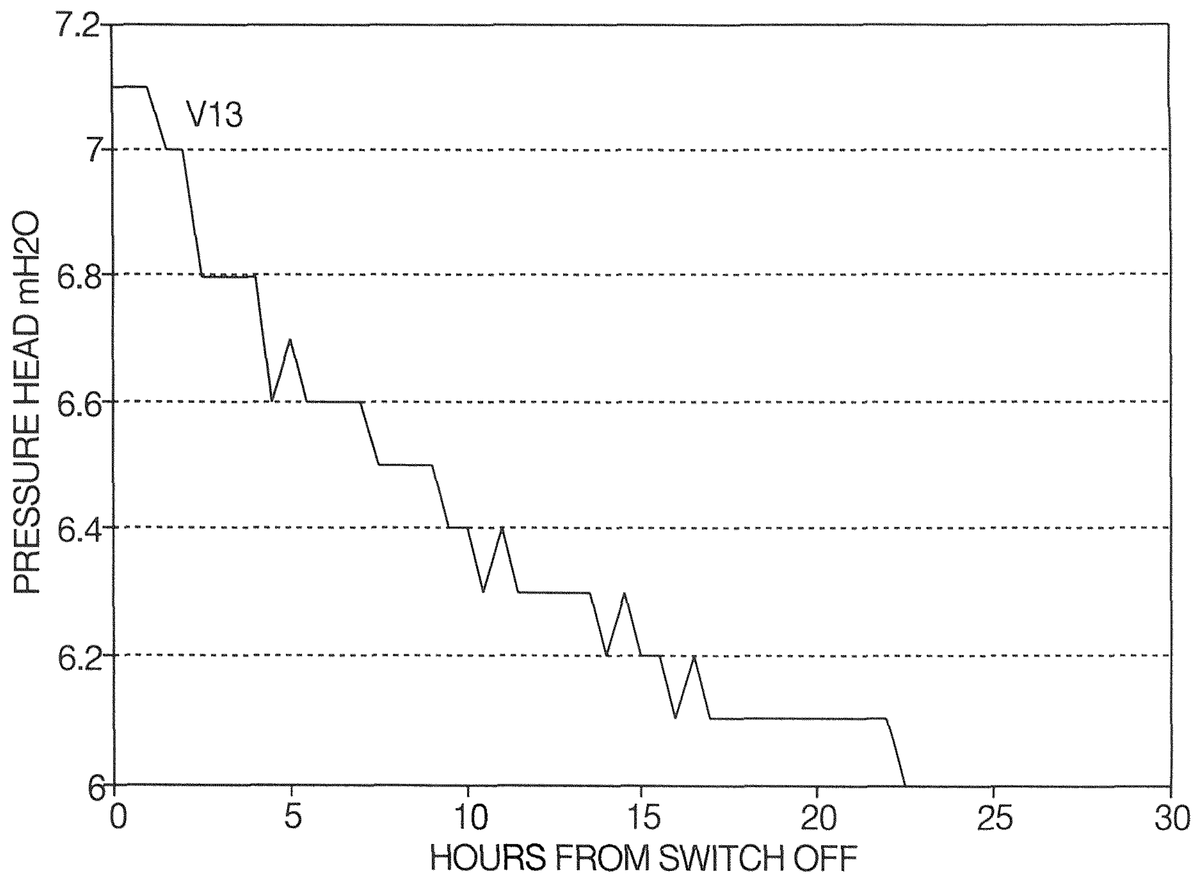


Figure 14-10 V13 Switch off event - day 135

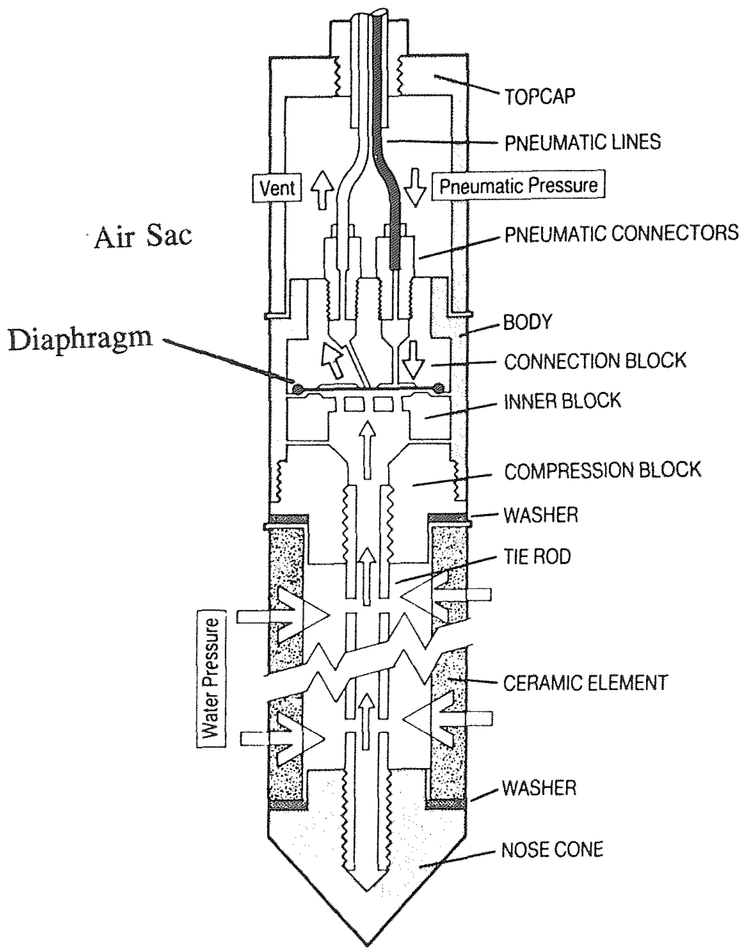


Figure 14-11 Conventional pneumatic piezometer

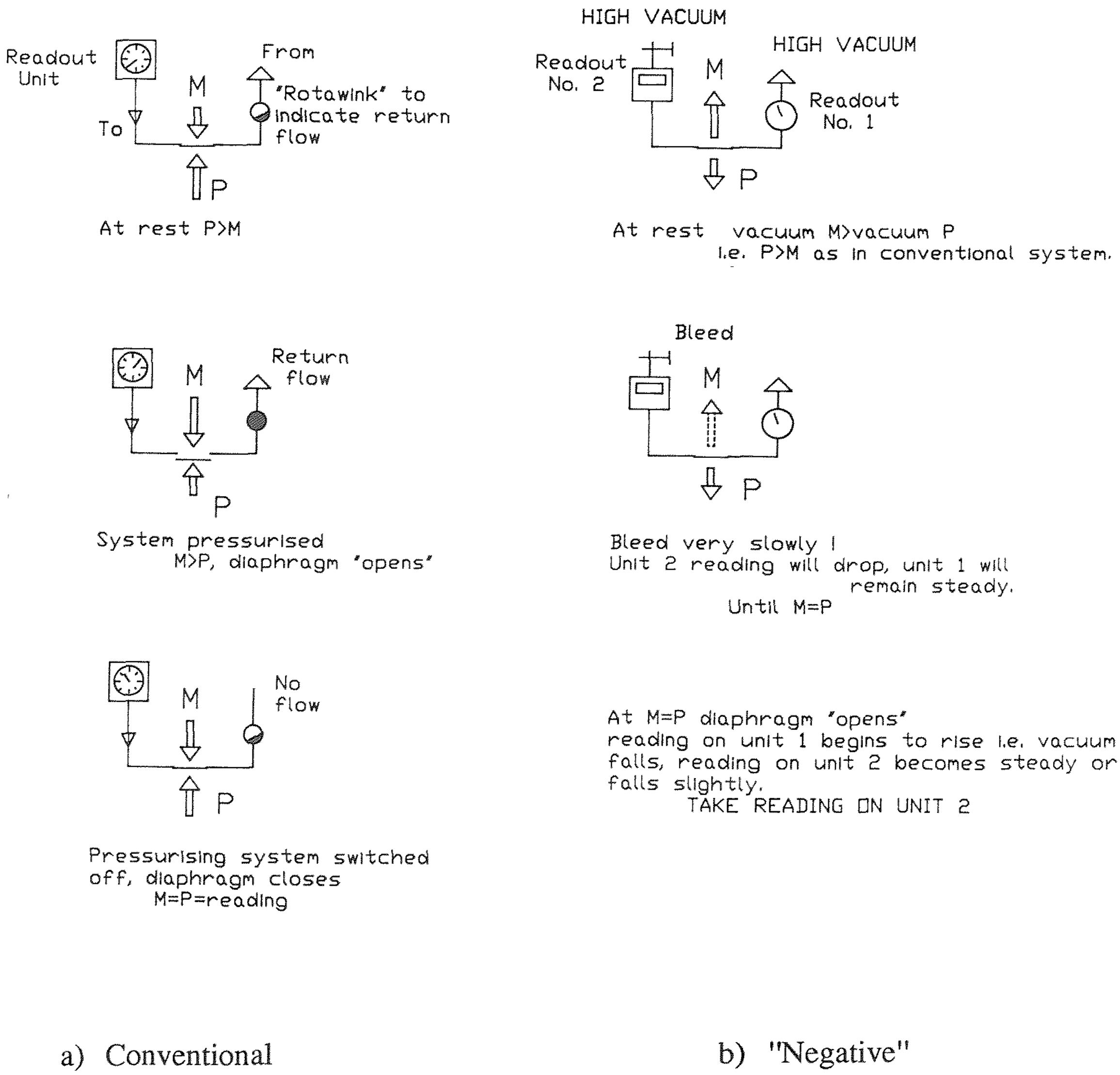


Figure 14-12 Operation of conventional and "negative" pneumatic piezometer systems

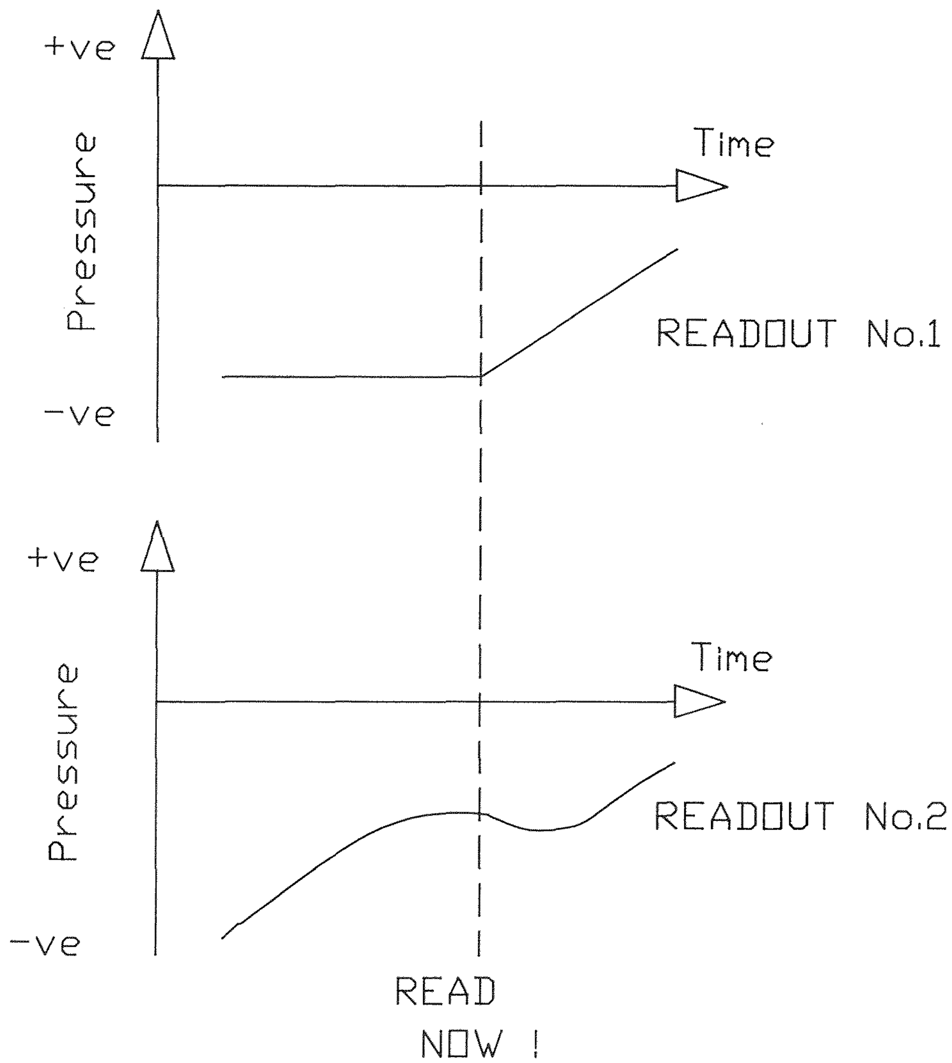


Figure 14-13 "Negative" pneumatic readout monitoring

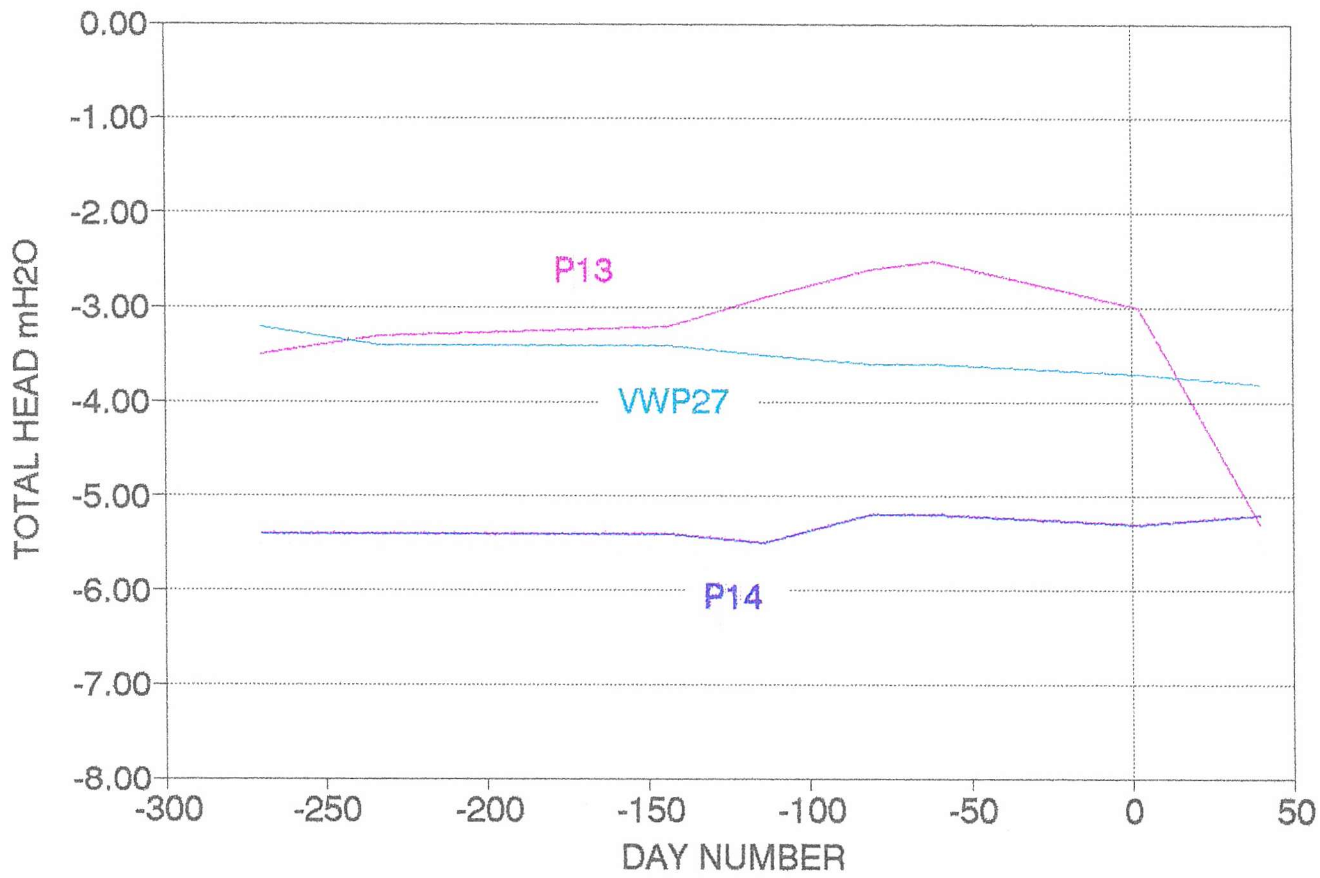
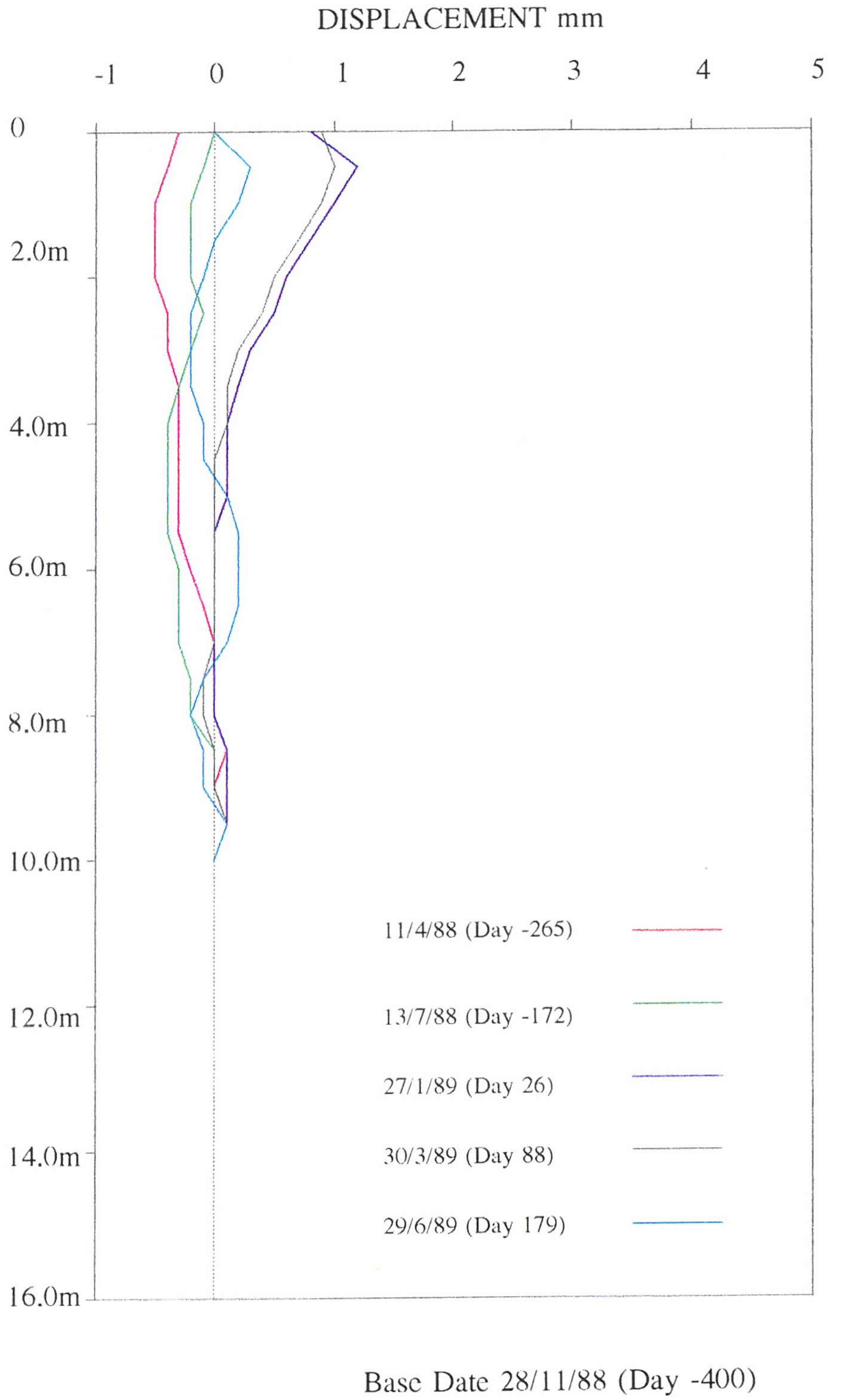
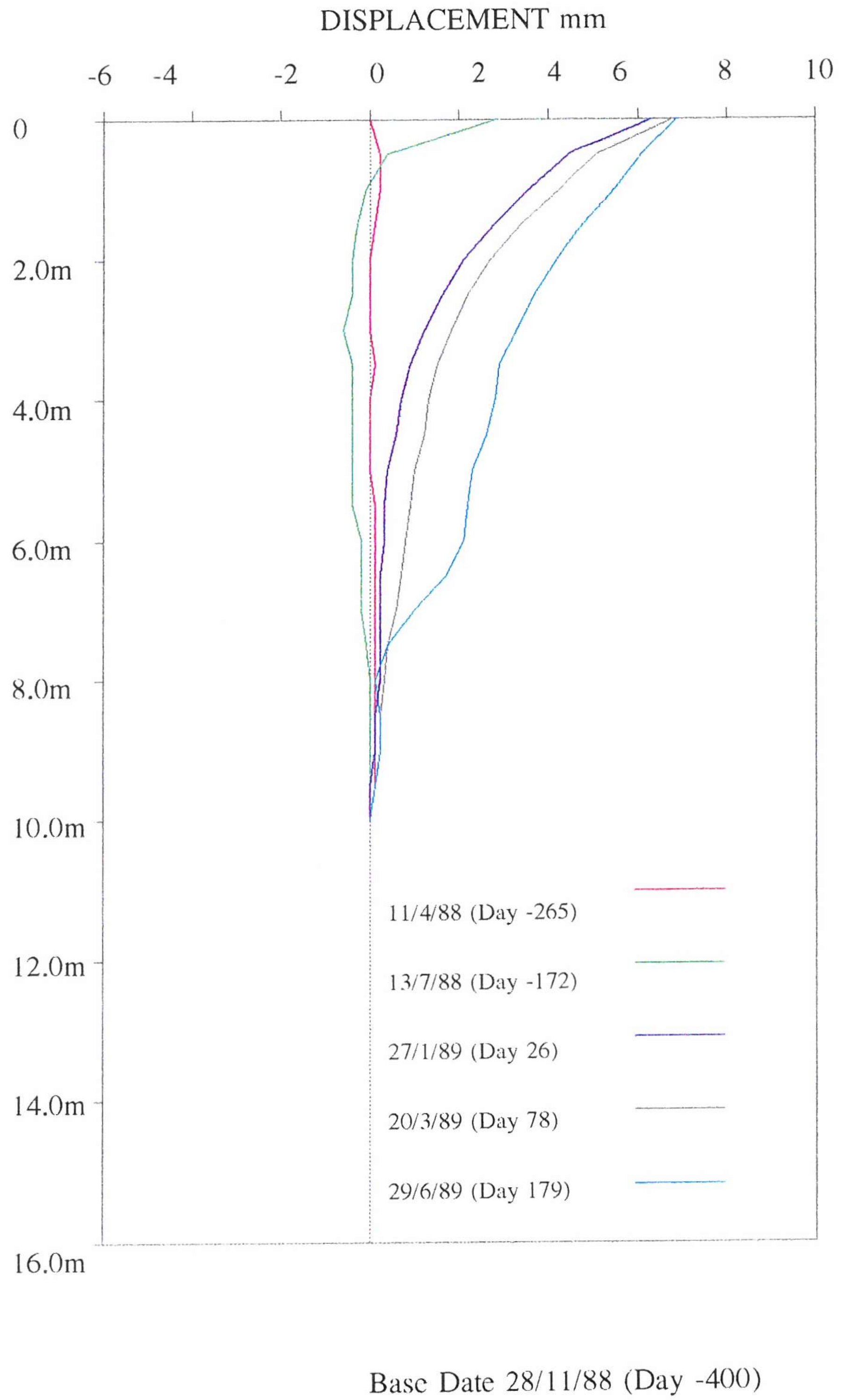


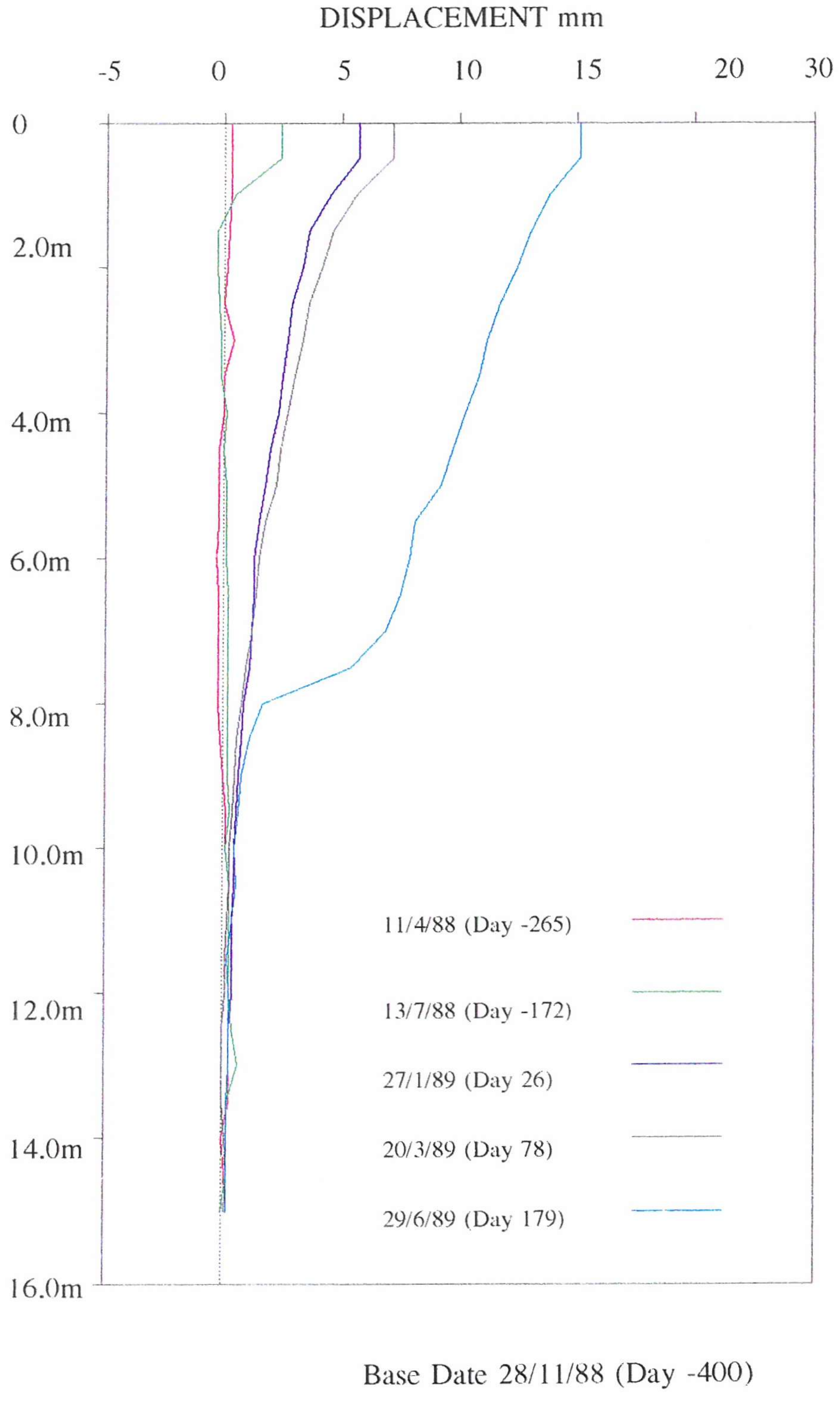
Figure 14-14 "Negative" piezometer readings



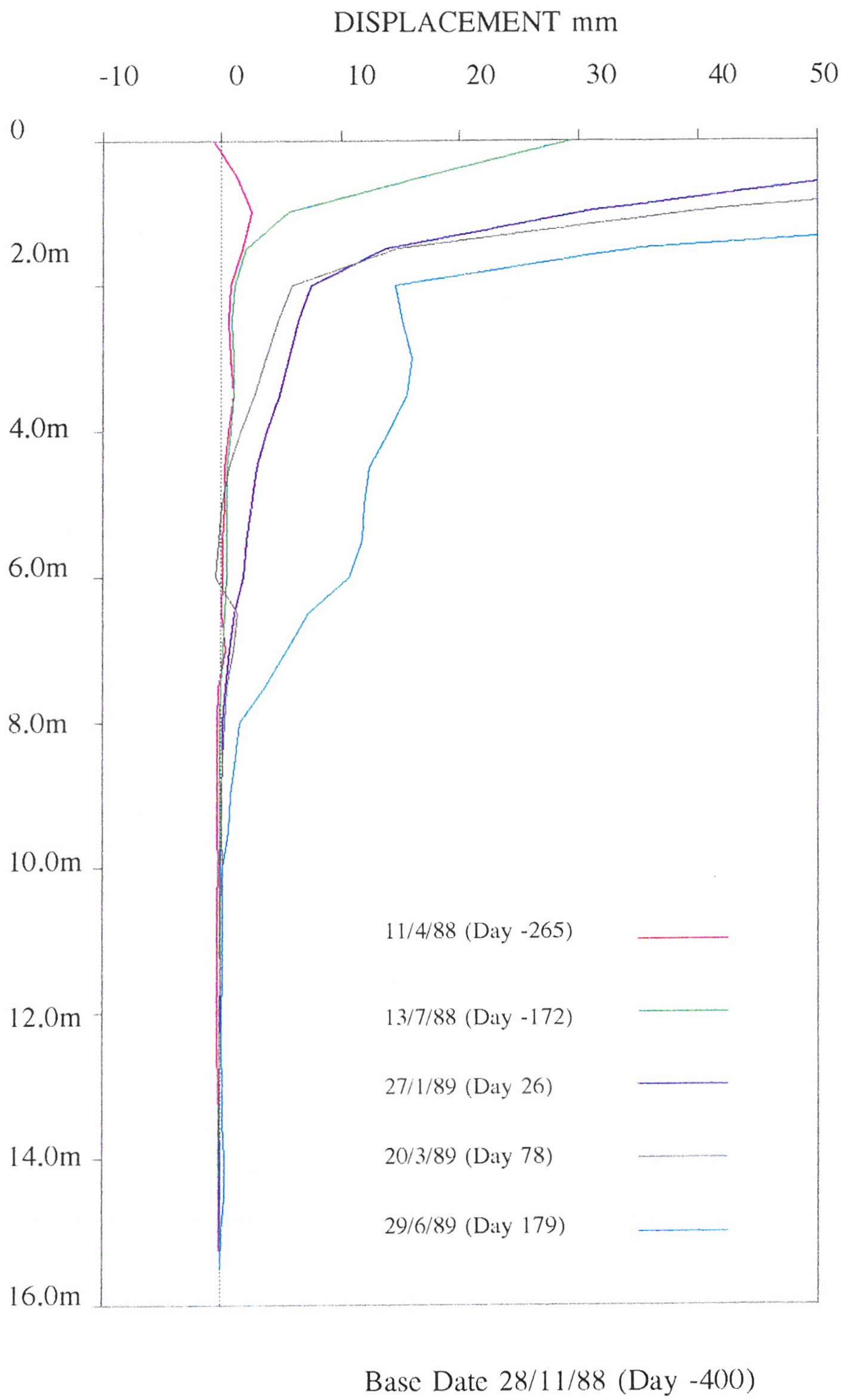
14-15 Inclinometer 01 displacement profiles



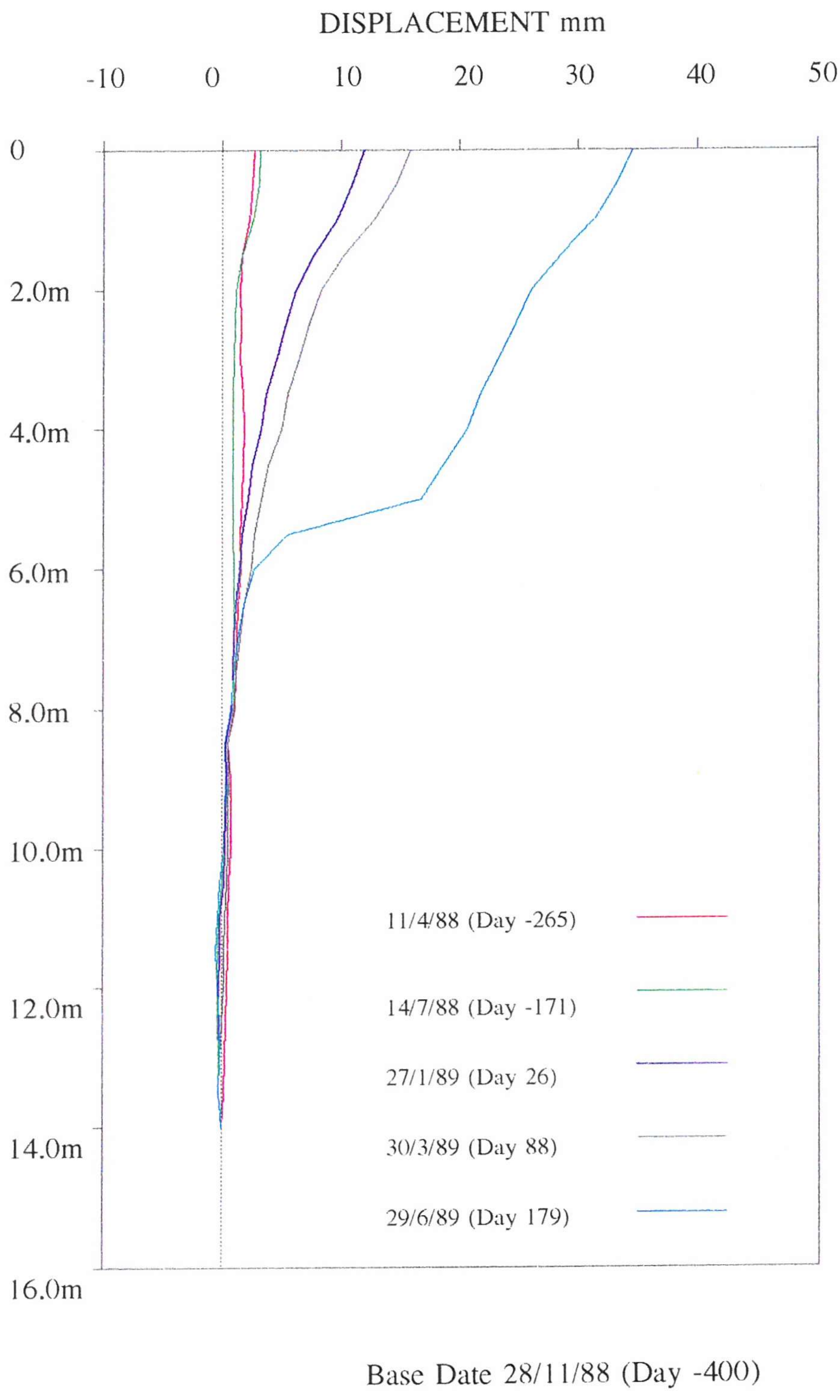
14-16 Inclinometer 02 displacement profiles



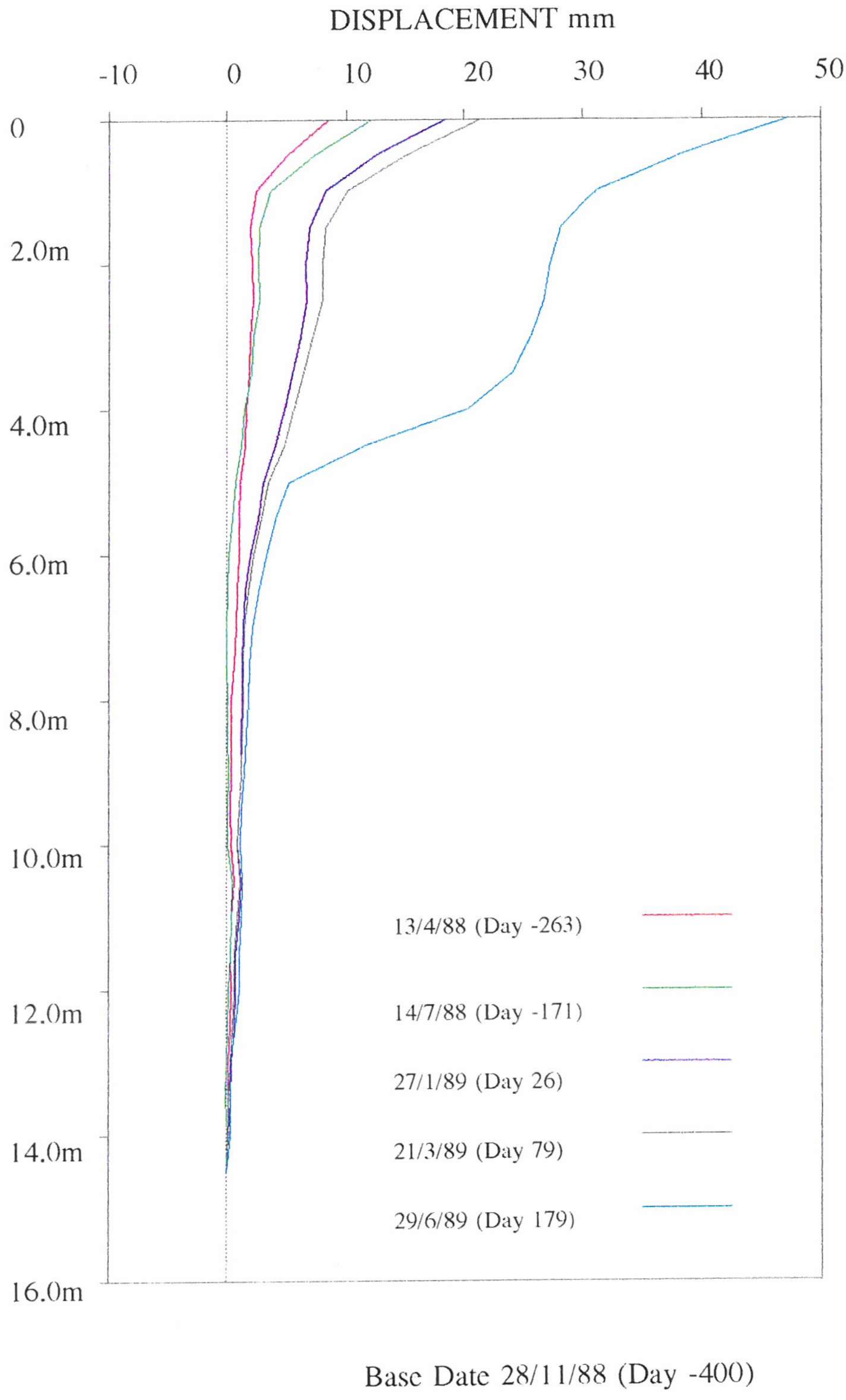
14-17 Inclinometer 03 displacement profiles



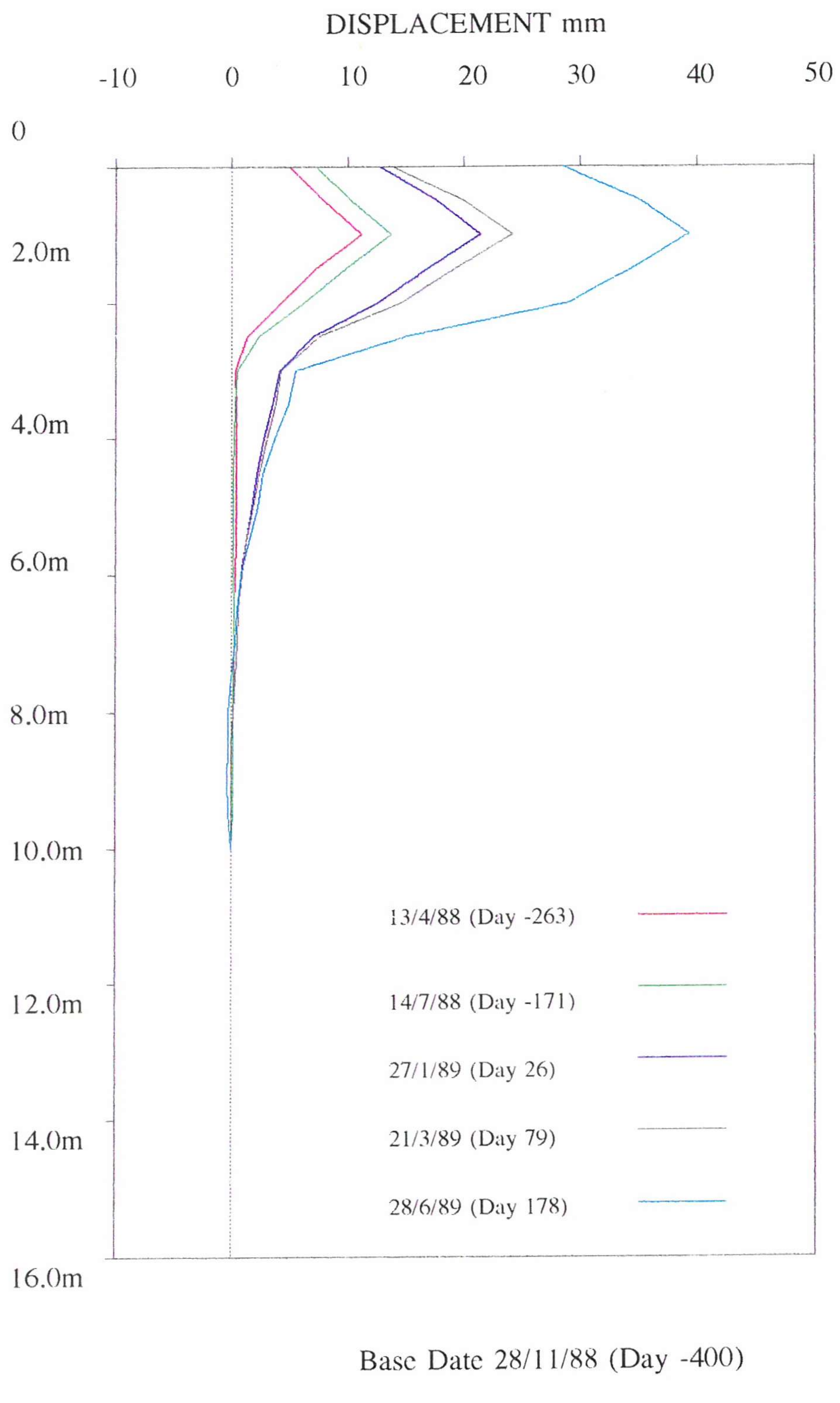
14-18 Inclinometer 04 displacement profiles



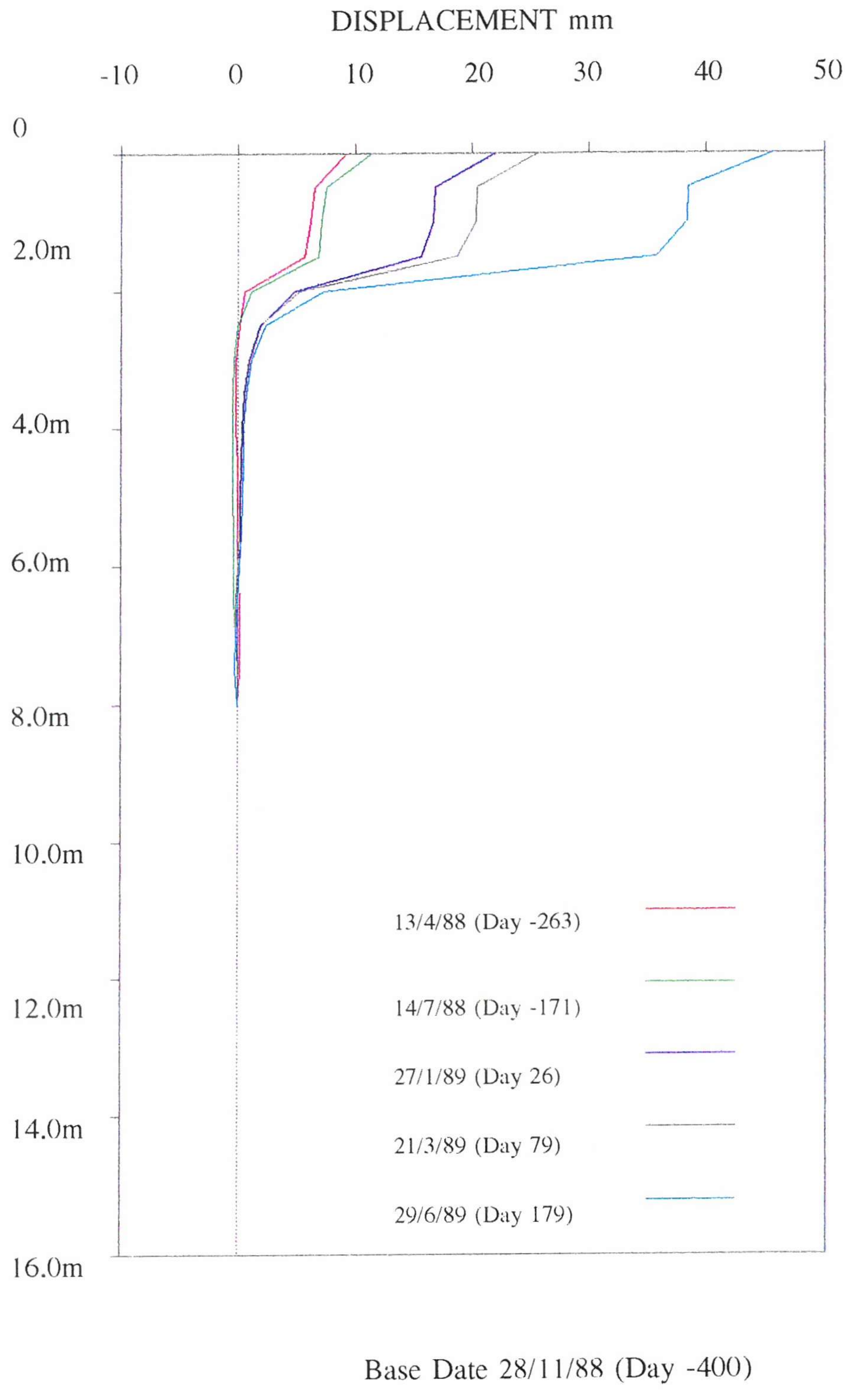
14-19 Inclinometer 05 displacement profiles



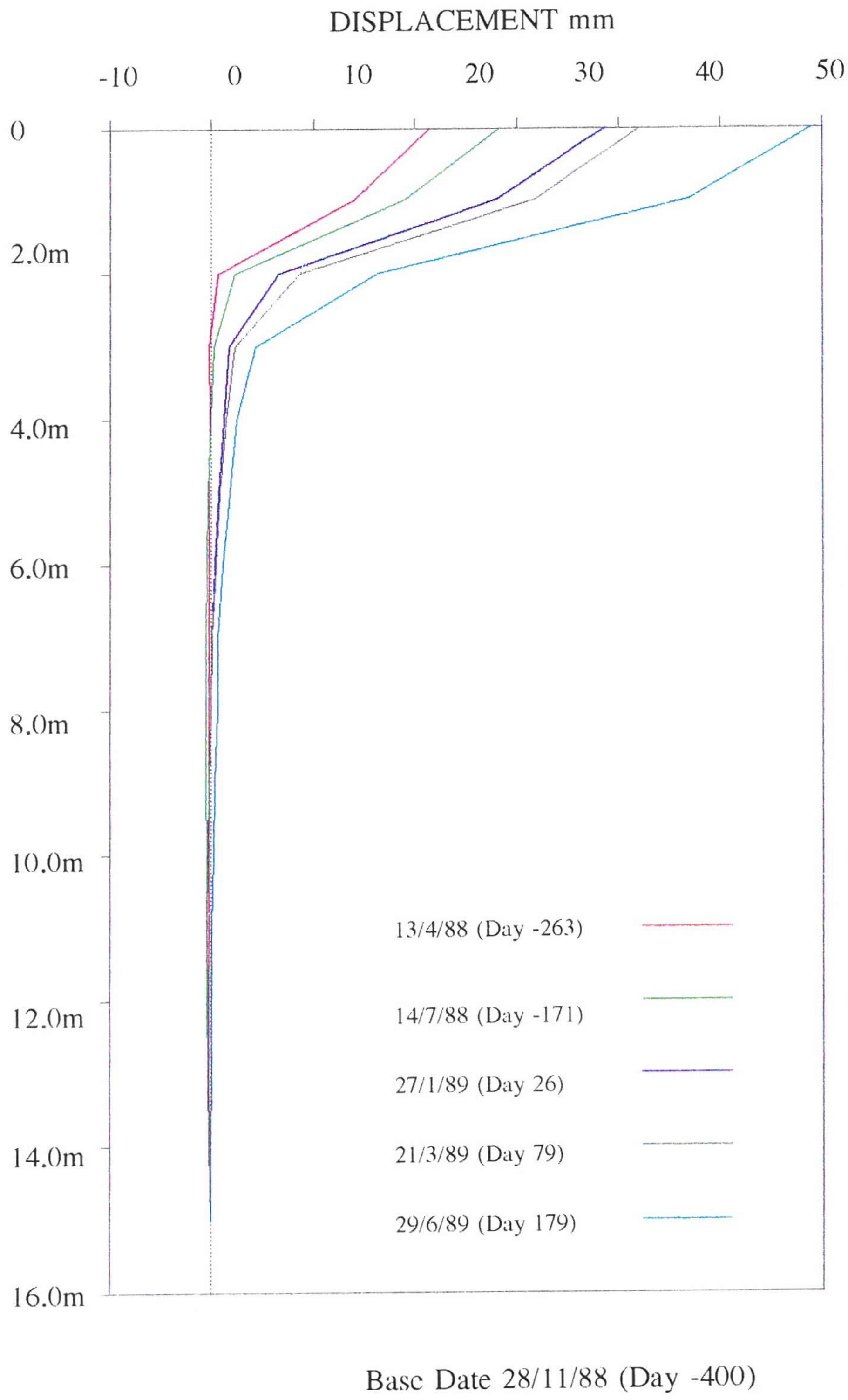
14-20 Inclinometer 06 displacement profiles



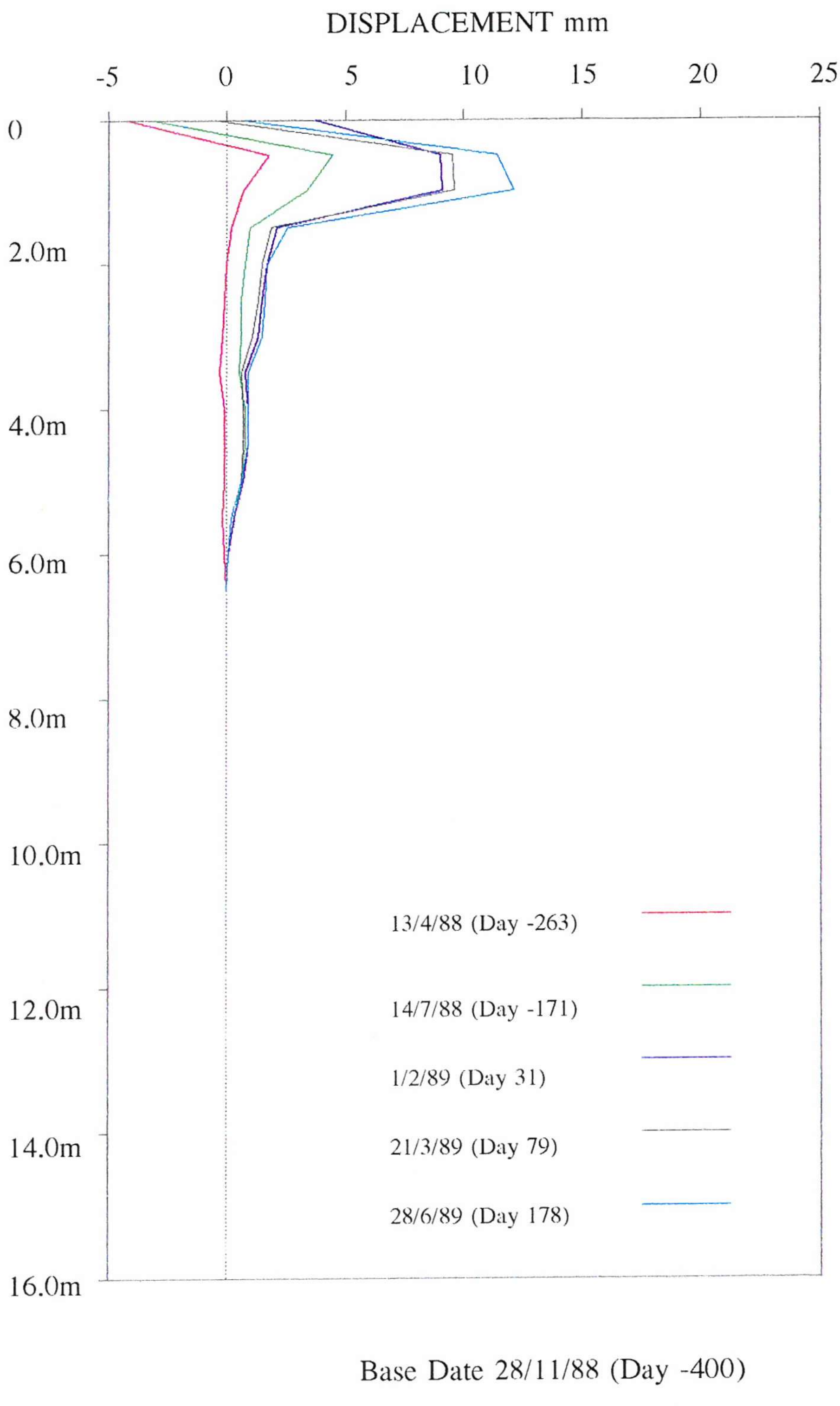
14-21 Inclinometer 07 displacement profiles



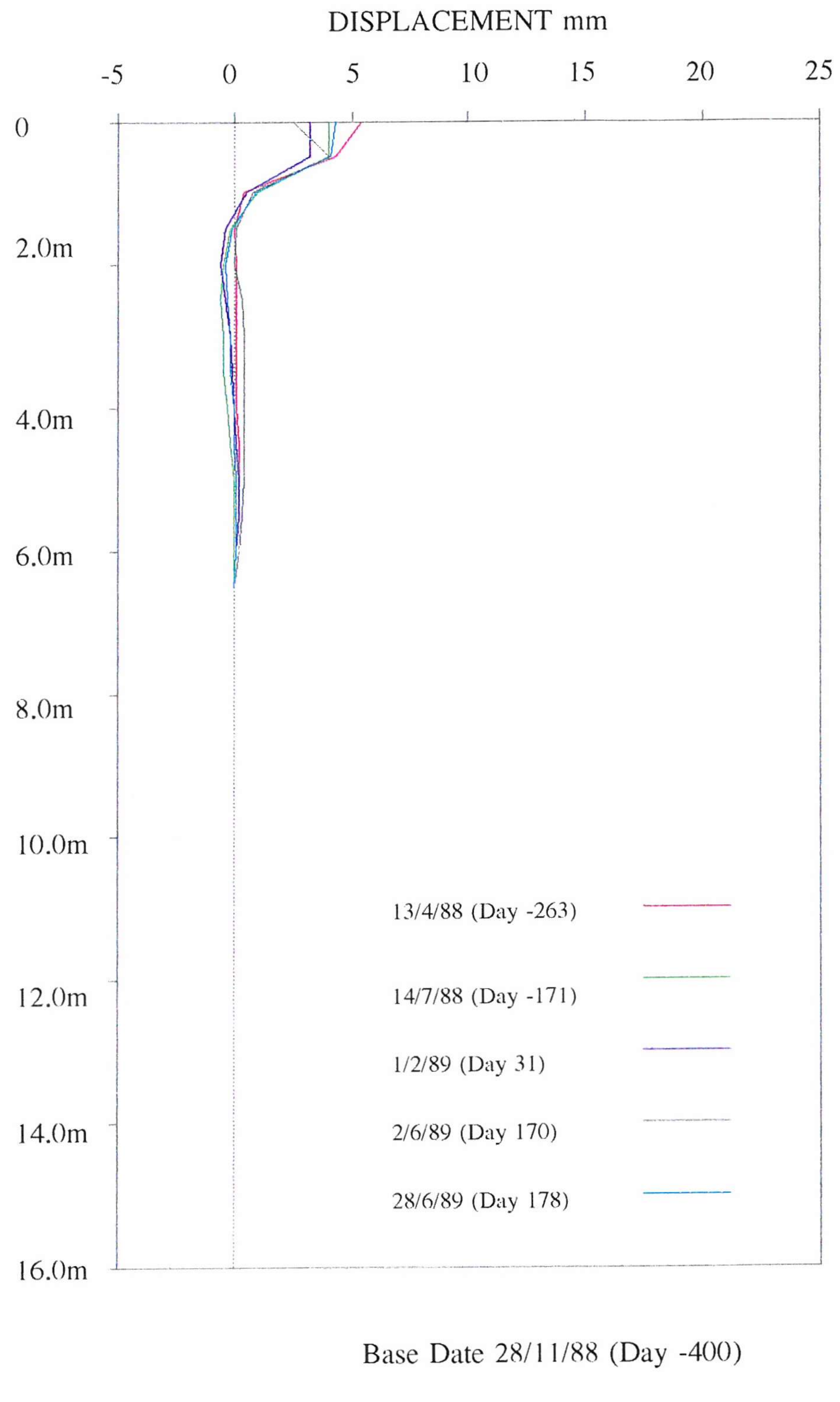
14-22 Inclinometer 08 displacement profiles



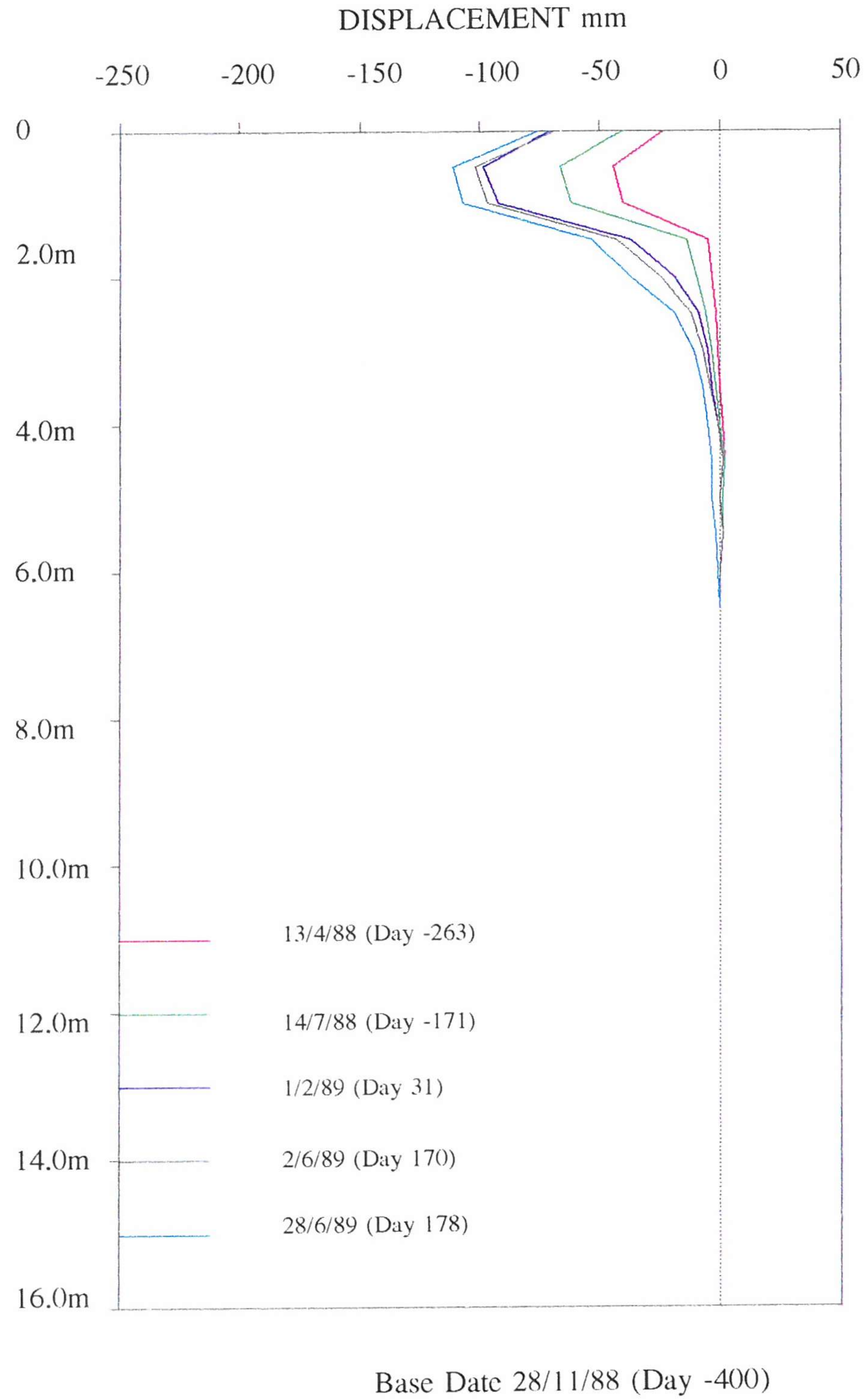
14-23 Inclinometer 09 displacement profiles



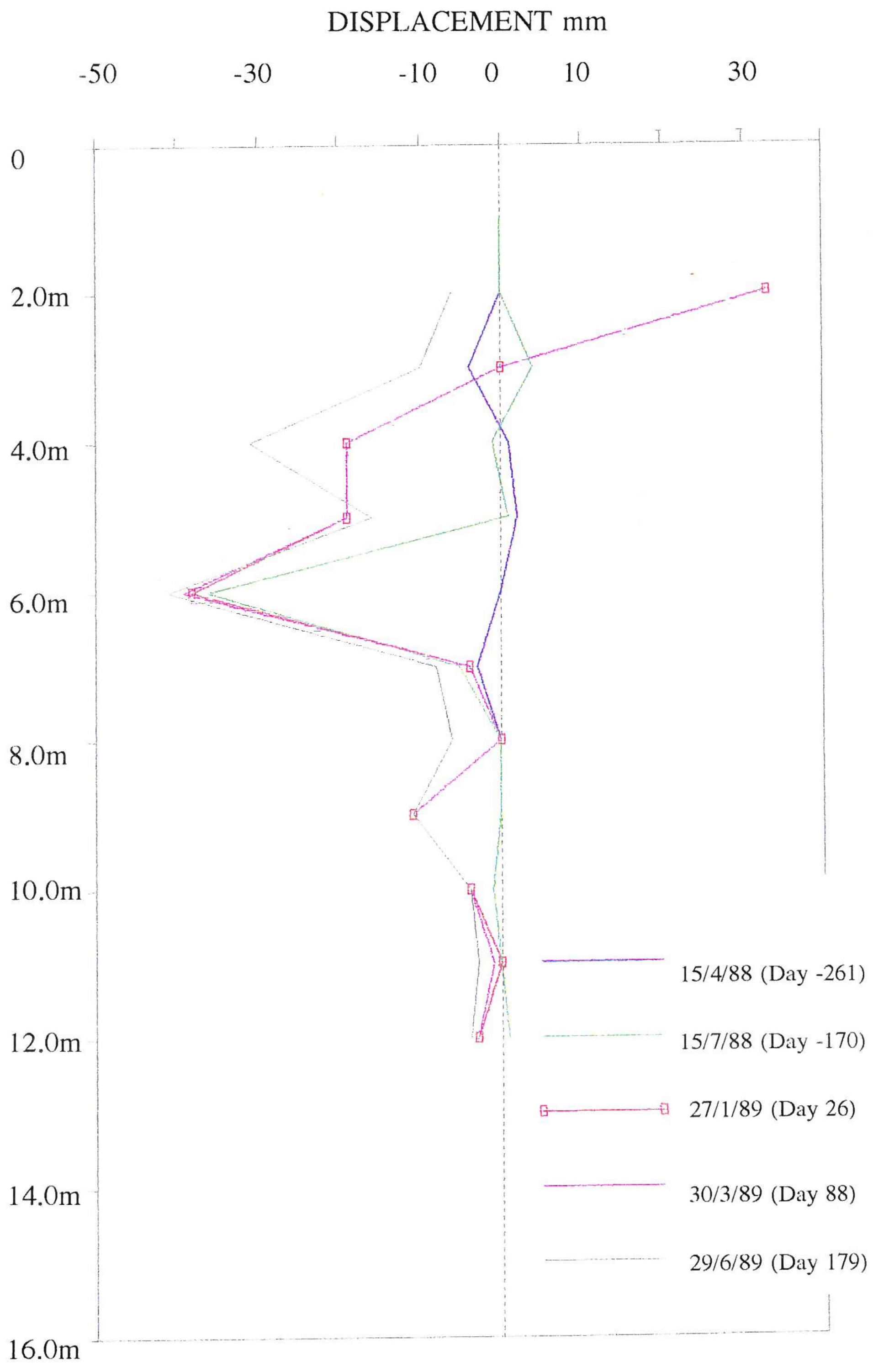
14-24 Inclinometer 10 displacement profiles



14-25 Inclinometer 11 displacement profiles

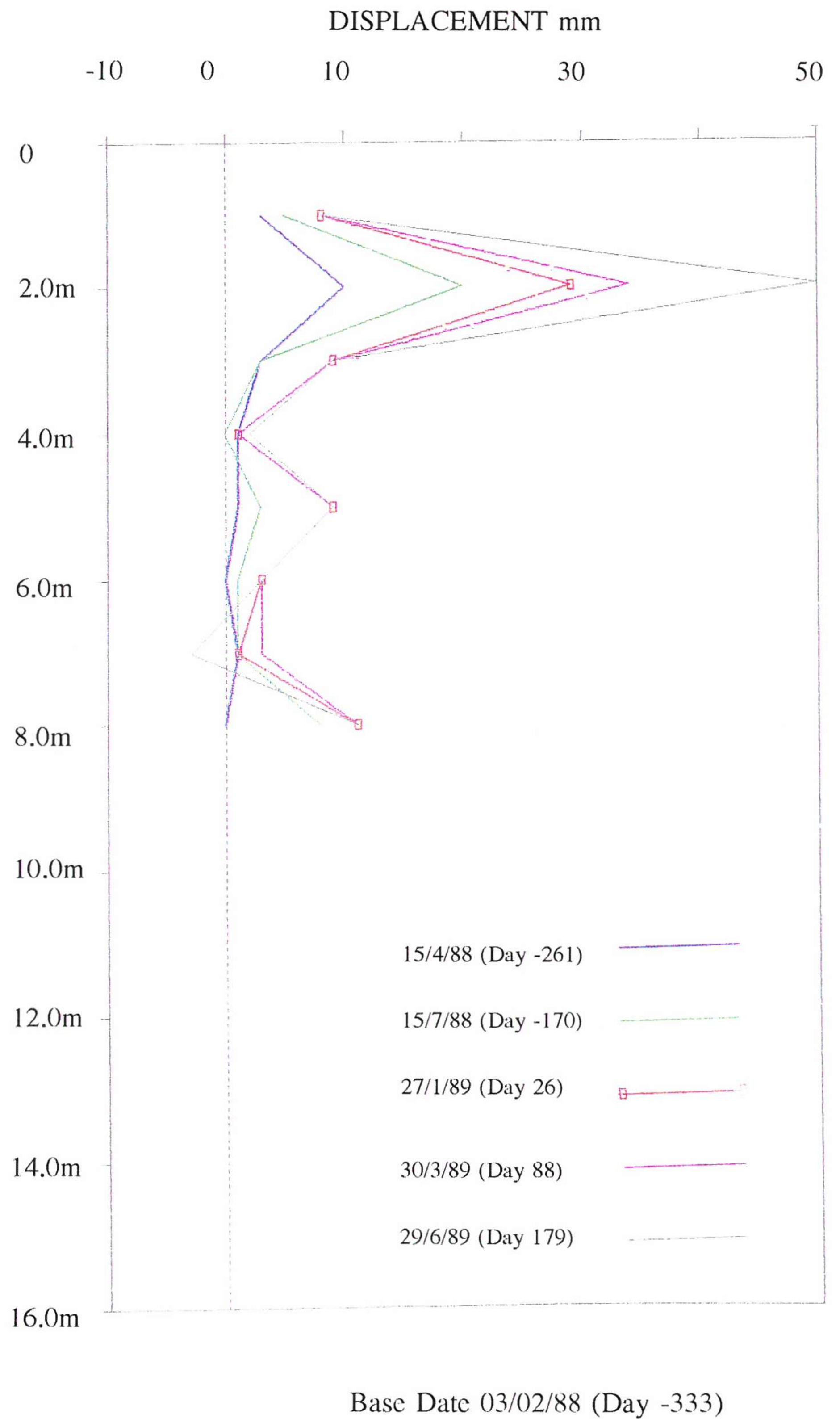


14-26 Inclinometer 12 displacement profiles



Base Date 03/02/88 (Day -333)

14-27 String Inclinometer 01 displacement profiles



14-28 String Inclinometer 02 displacement profiles

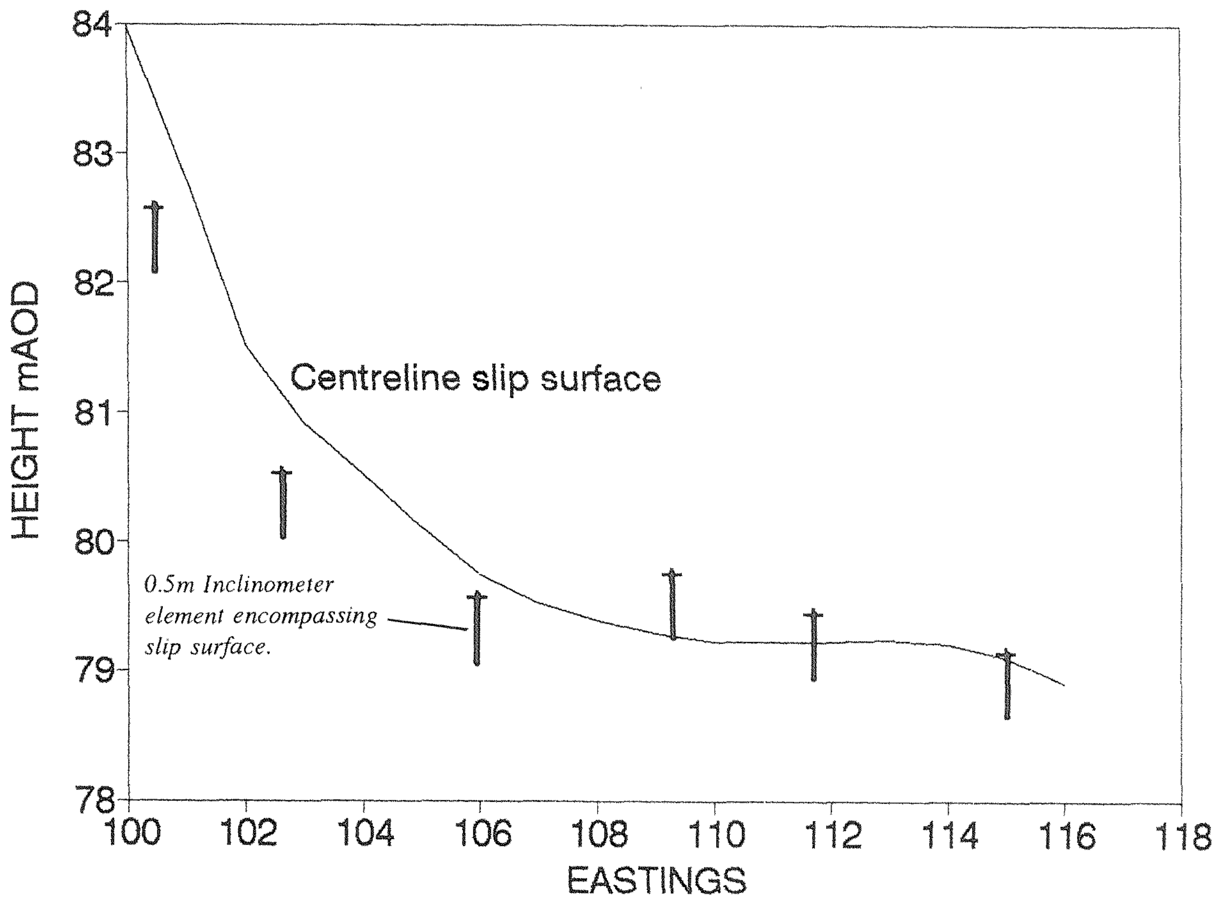


Figure 14-29 Slip surface to critical inclinometer element correlation

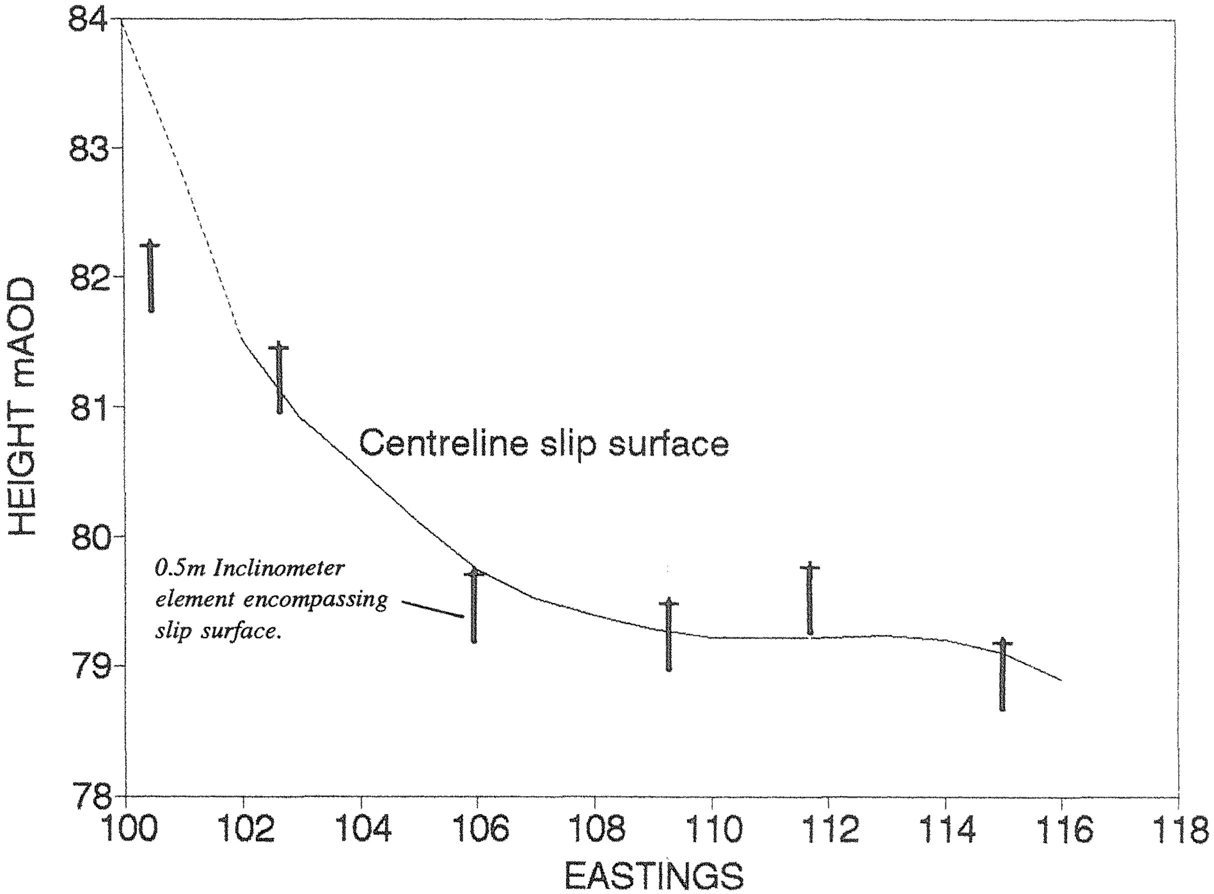


Figure 14-30 Slip surface to adjusted critical inclinometer element correlation

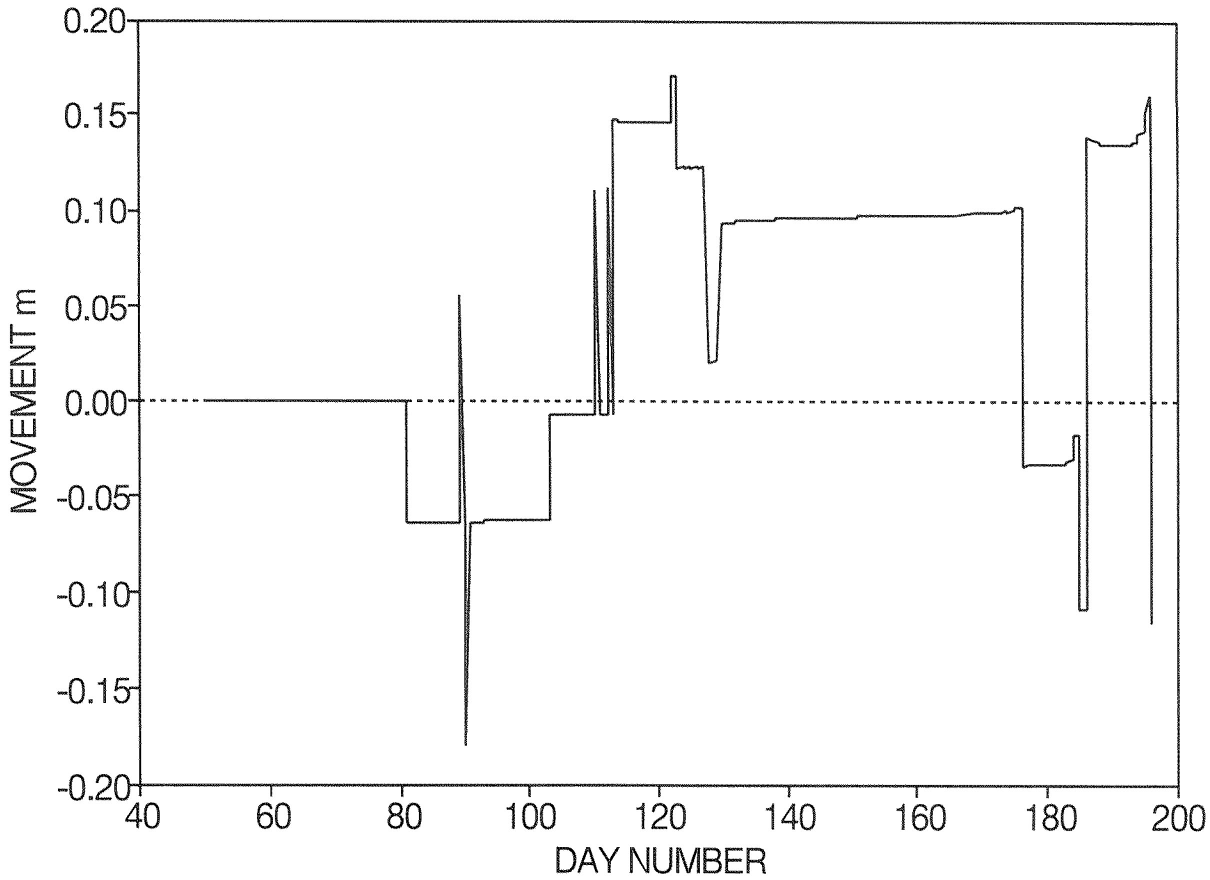


Figure 14-31 Example of unrefined wire extensometer output

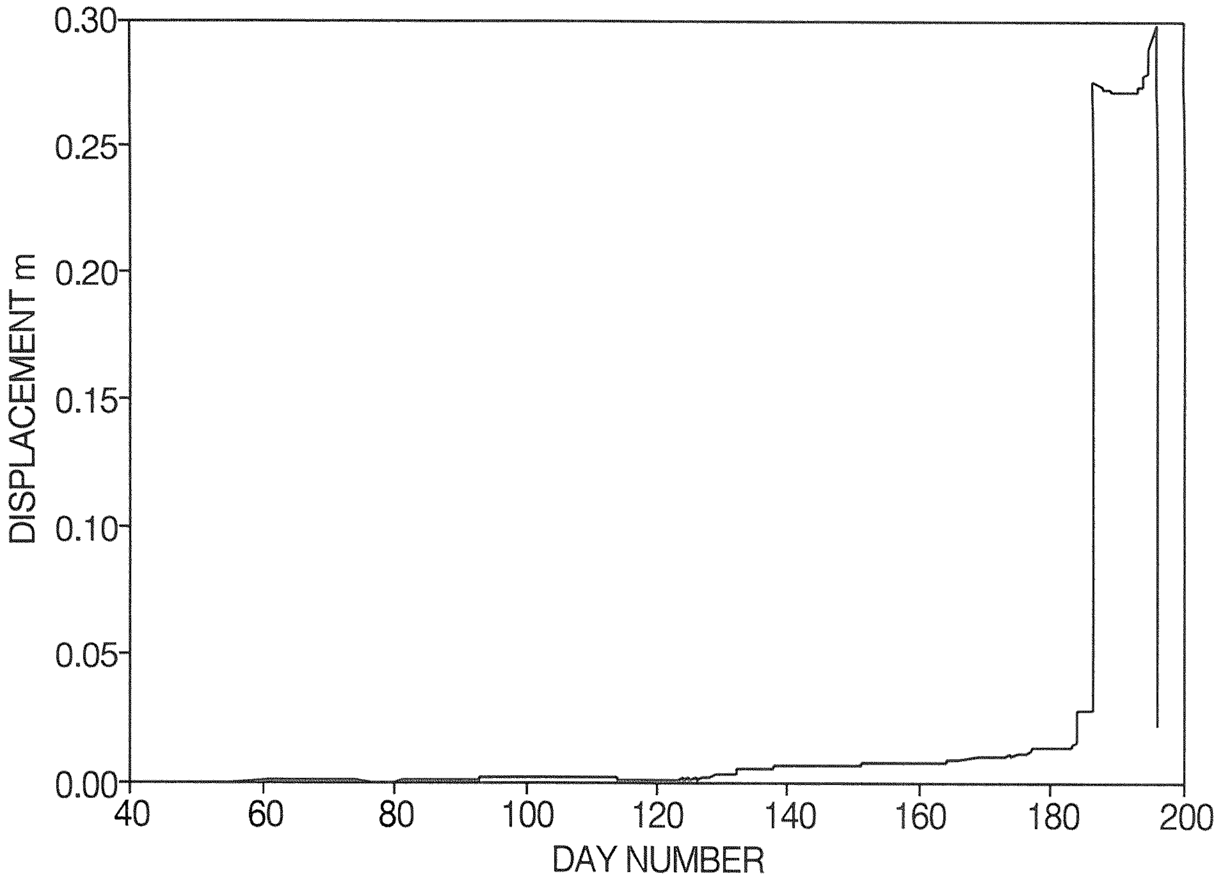
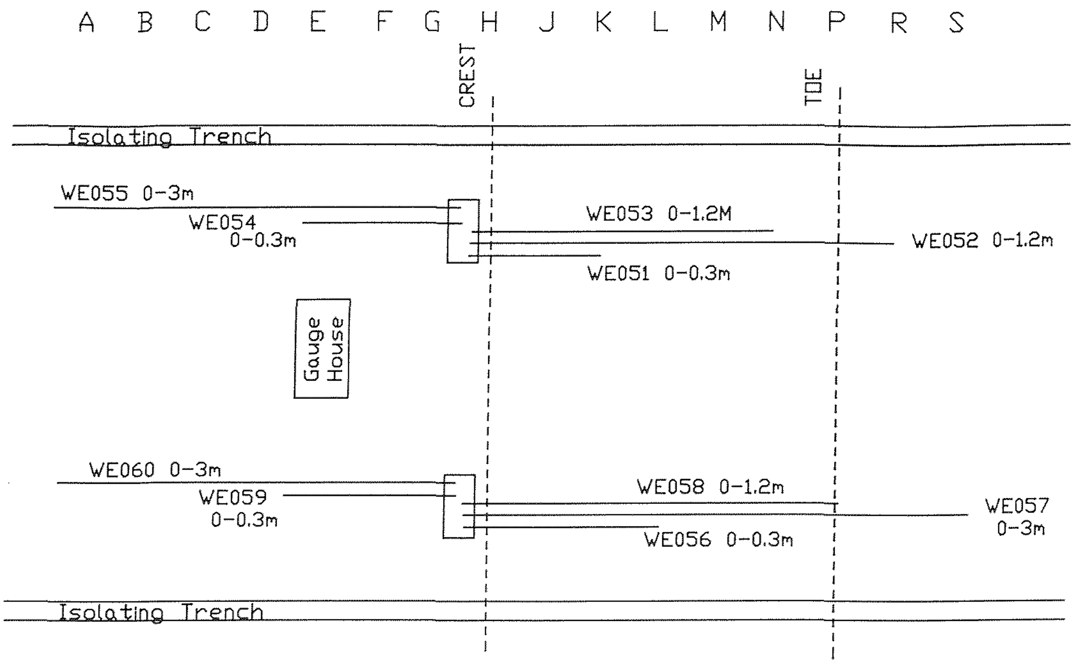


Figure 14-32 Example of refined wire extensometer output



Note: Instrument number is followed by range.
e.g. WE055 0-3.0 refers to Wire extensometer 055, intended monitoring range upto 3.0 metres.

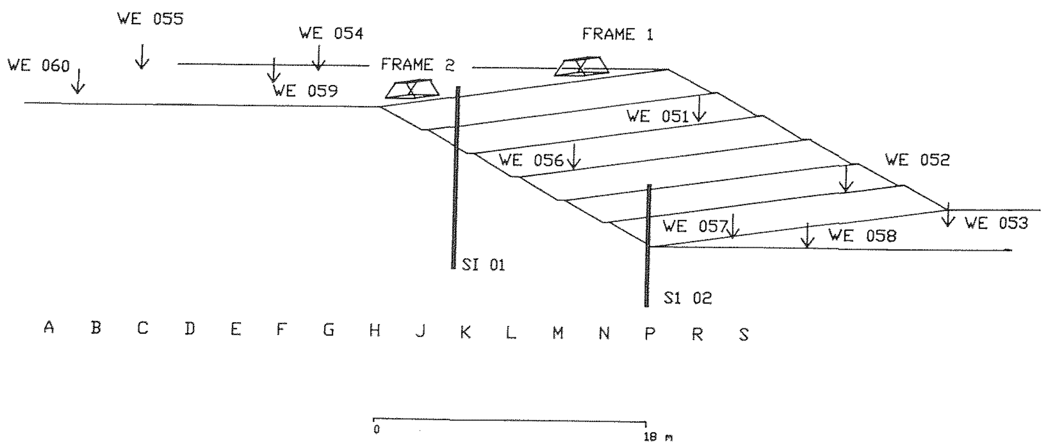


Figure 14-33 Wire extensometer ranges and anchor positions

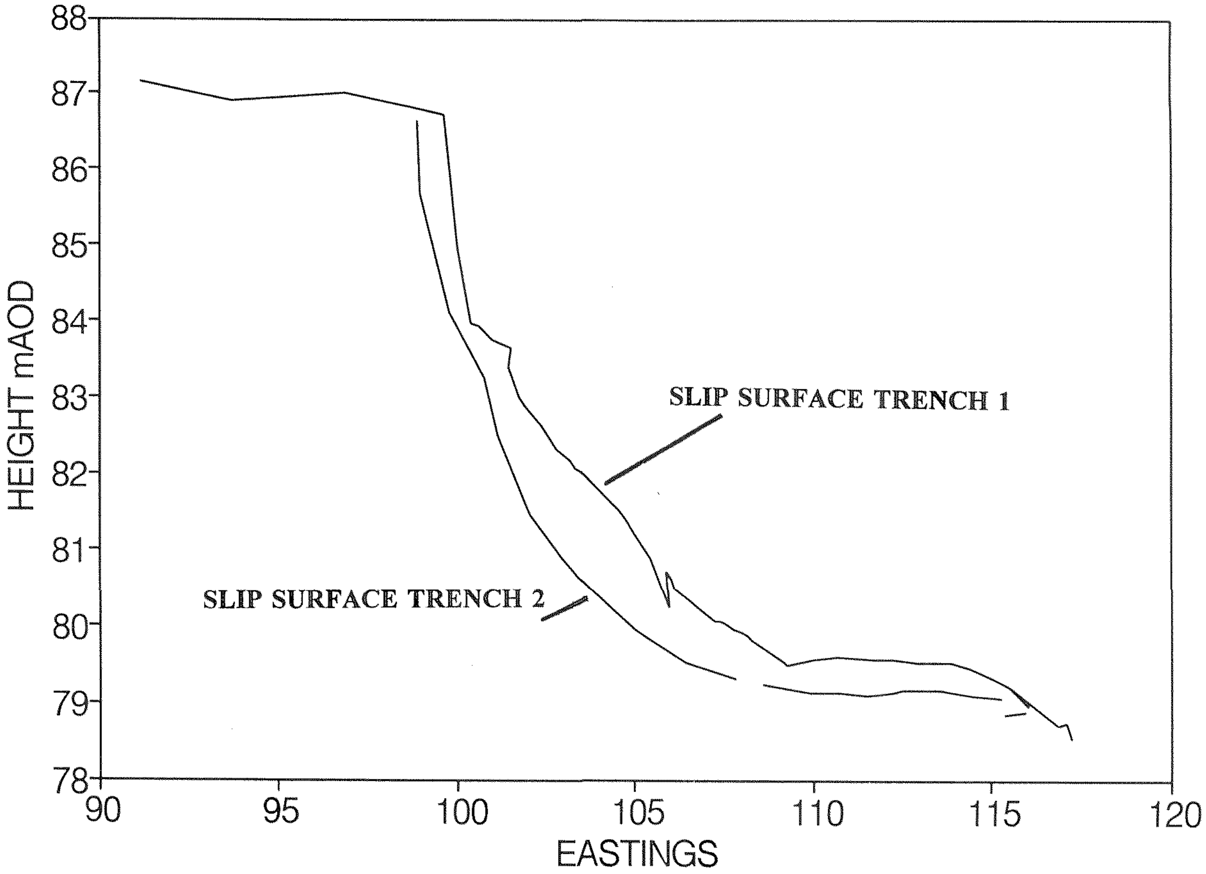


Figure 15-1 Surveyed slip surface locations

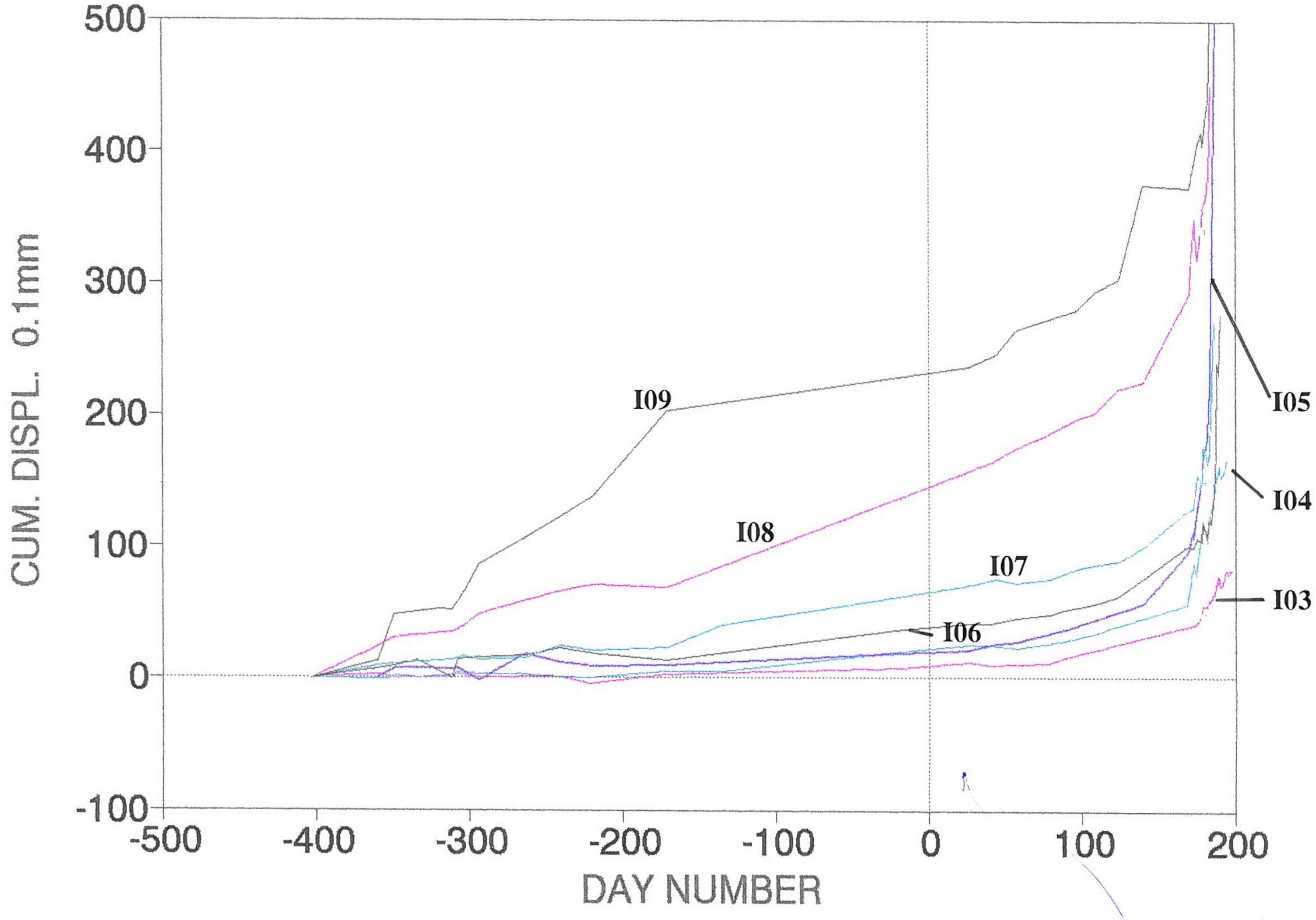


Figure 16-1 Time displacement curves for key inclinometer elements

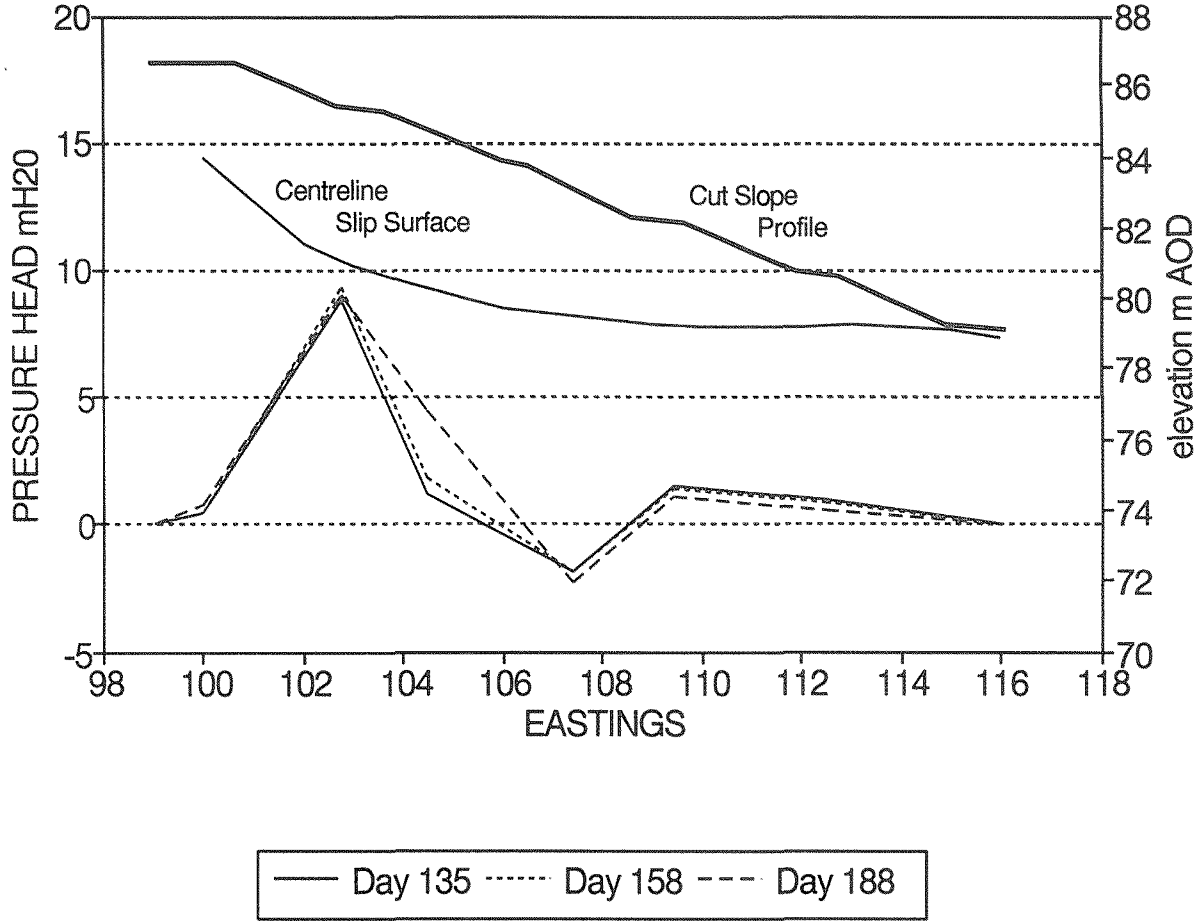


Figure 16-2 Pore pressure distributions along the slip surface at day numbers 136,158 and 188

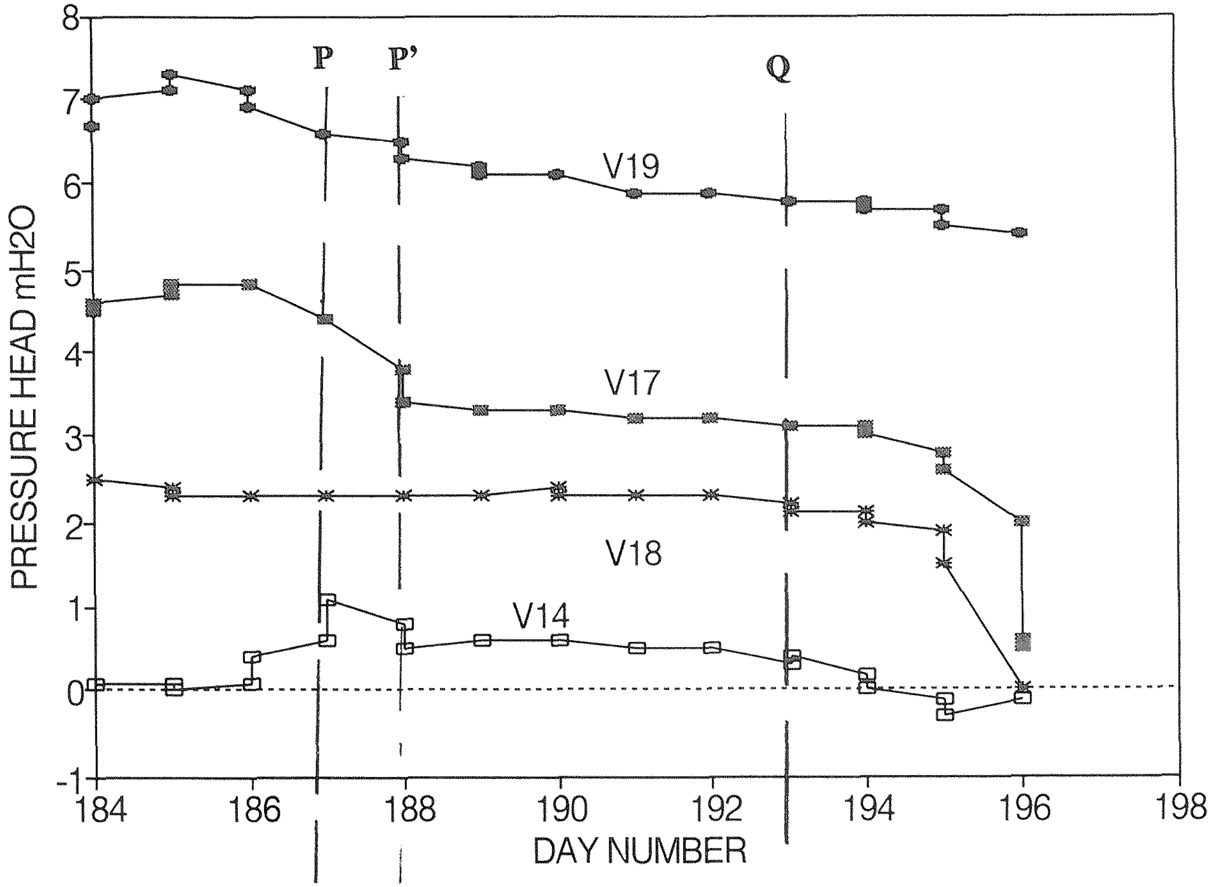


Figure 16-3 Piezometers V14,V17,V18 and V19 day number 185 to collapse

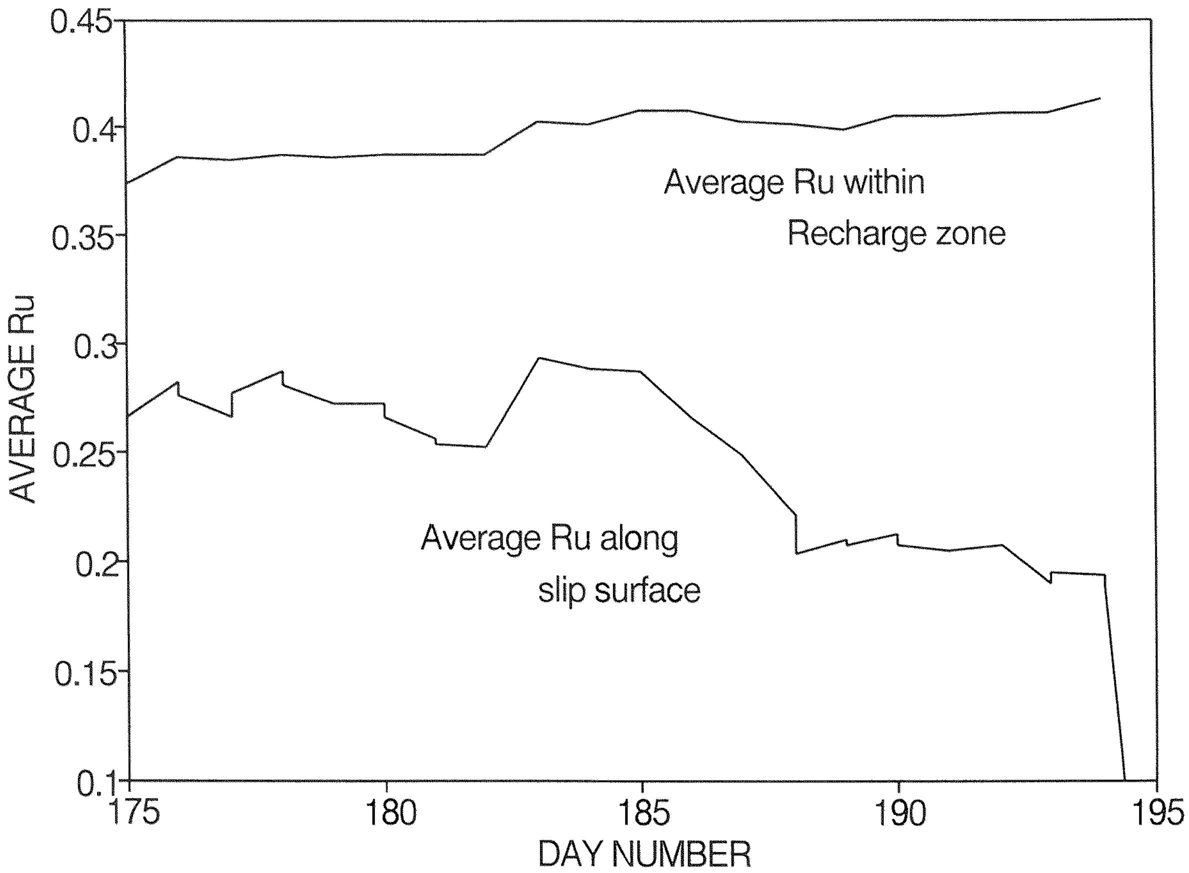


Figure 16-4 Slip surface and recharge zone r_u's in the twenty days prior to slope collapse

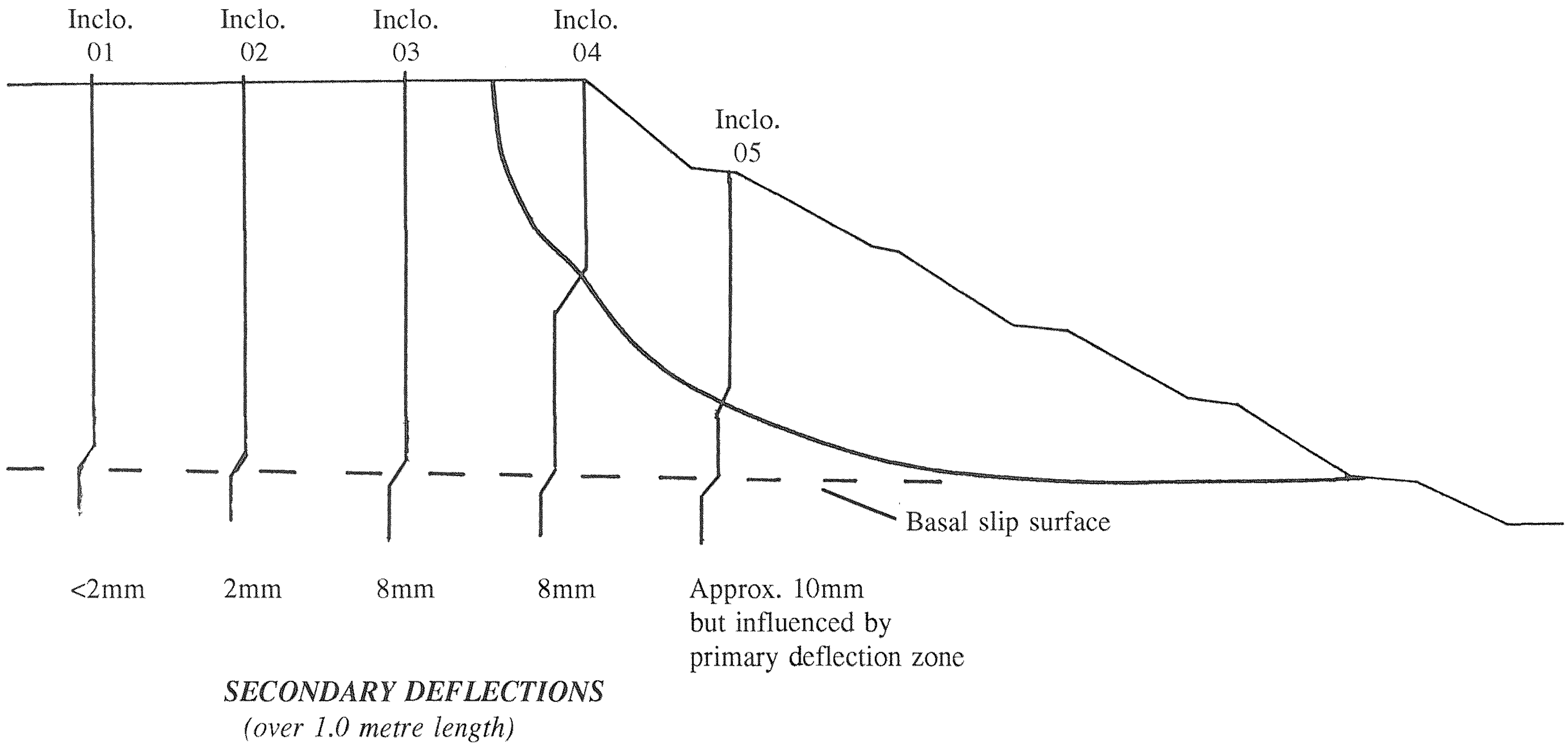
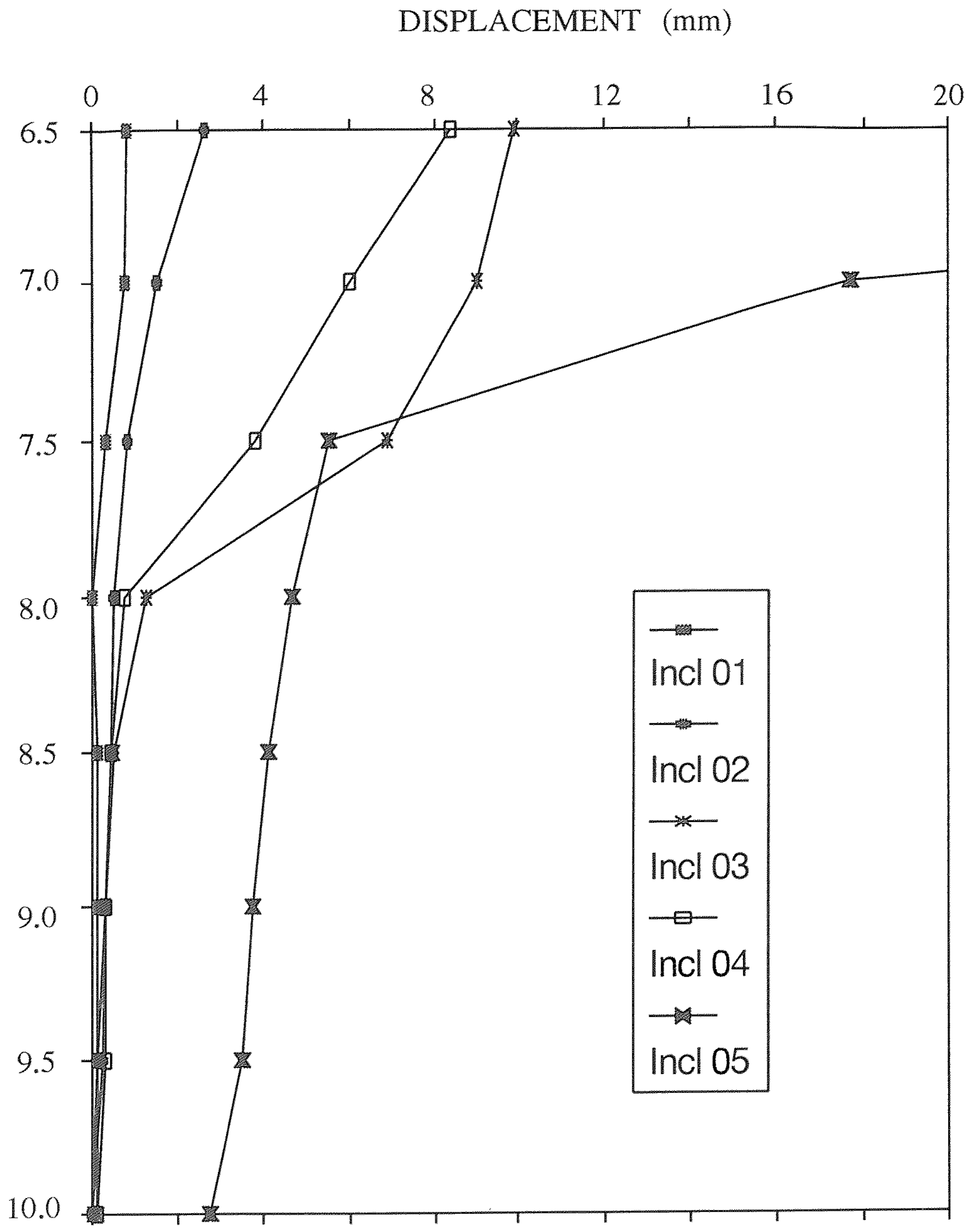


Figure 16-5a Secondary displacements. Inclinometers 01 to 05.



Readings for 10 July 1989 (Day 190)

Figure 16-5b Displacement profiles for Inclinometers 01 to 05, 6.5 to 10.0 metres below slope crest.

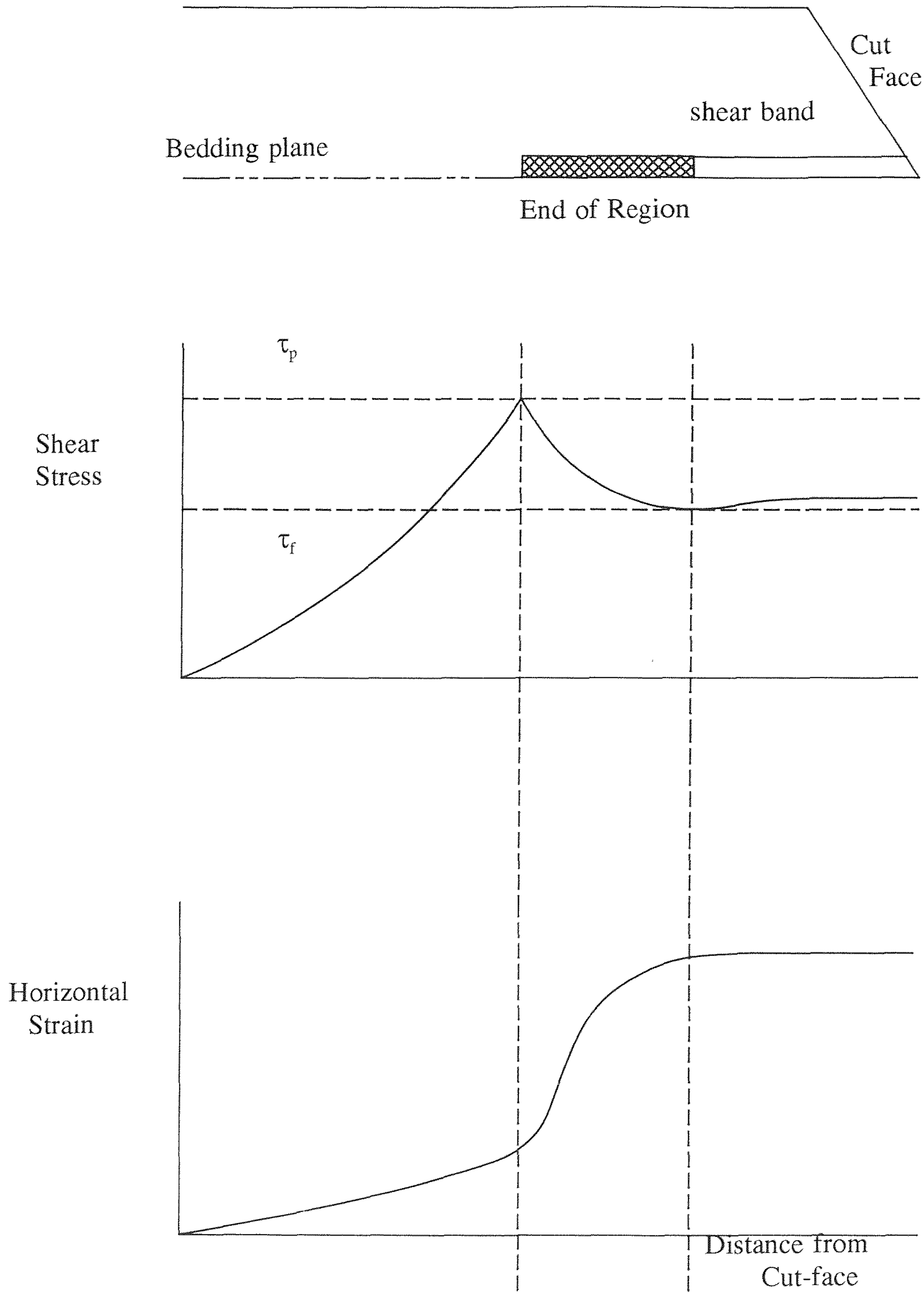


Figure 16-6 "End of Region" Zone; shear stresses decreasing from peak to residual values

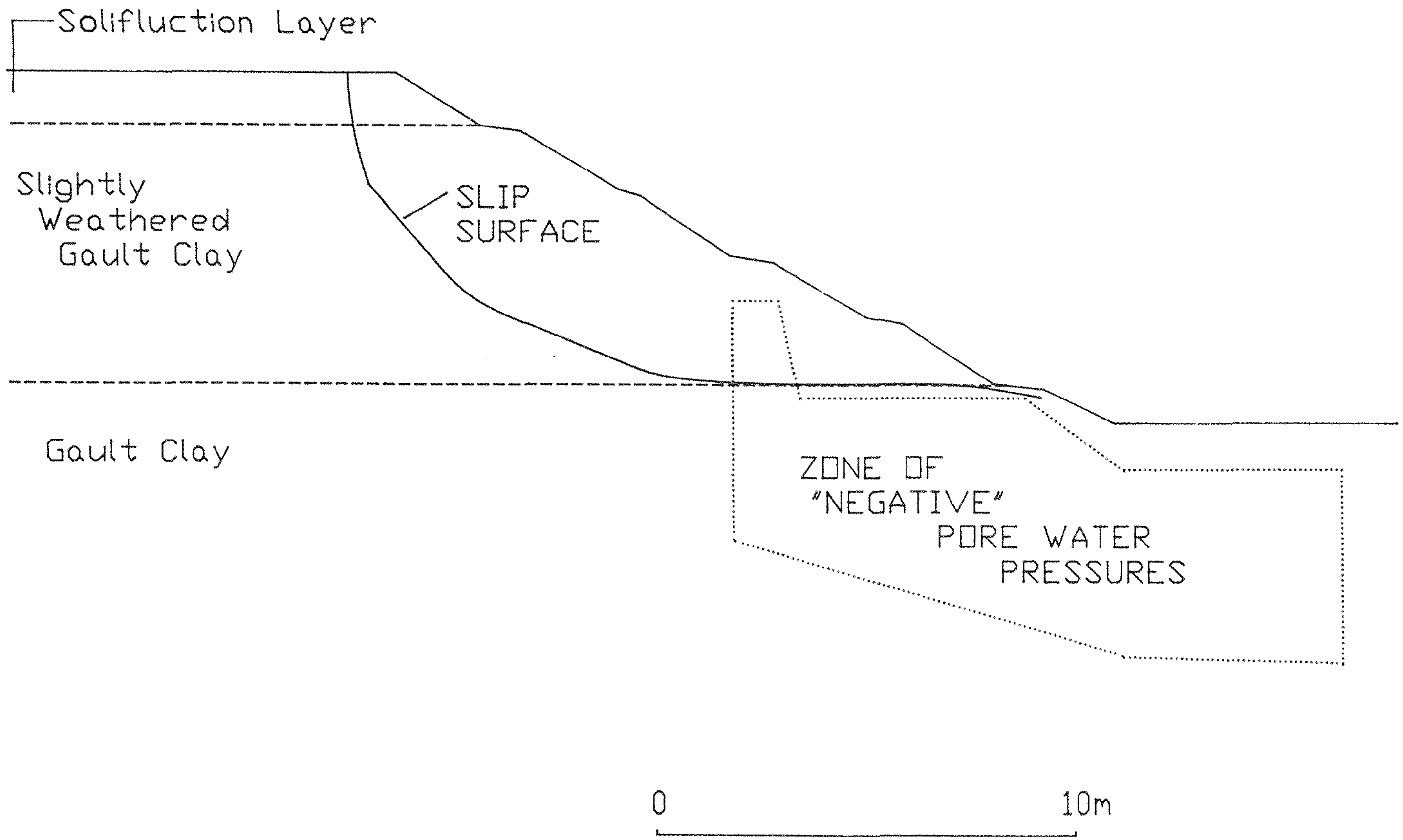
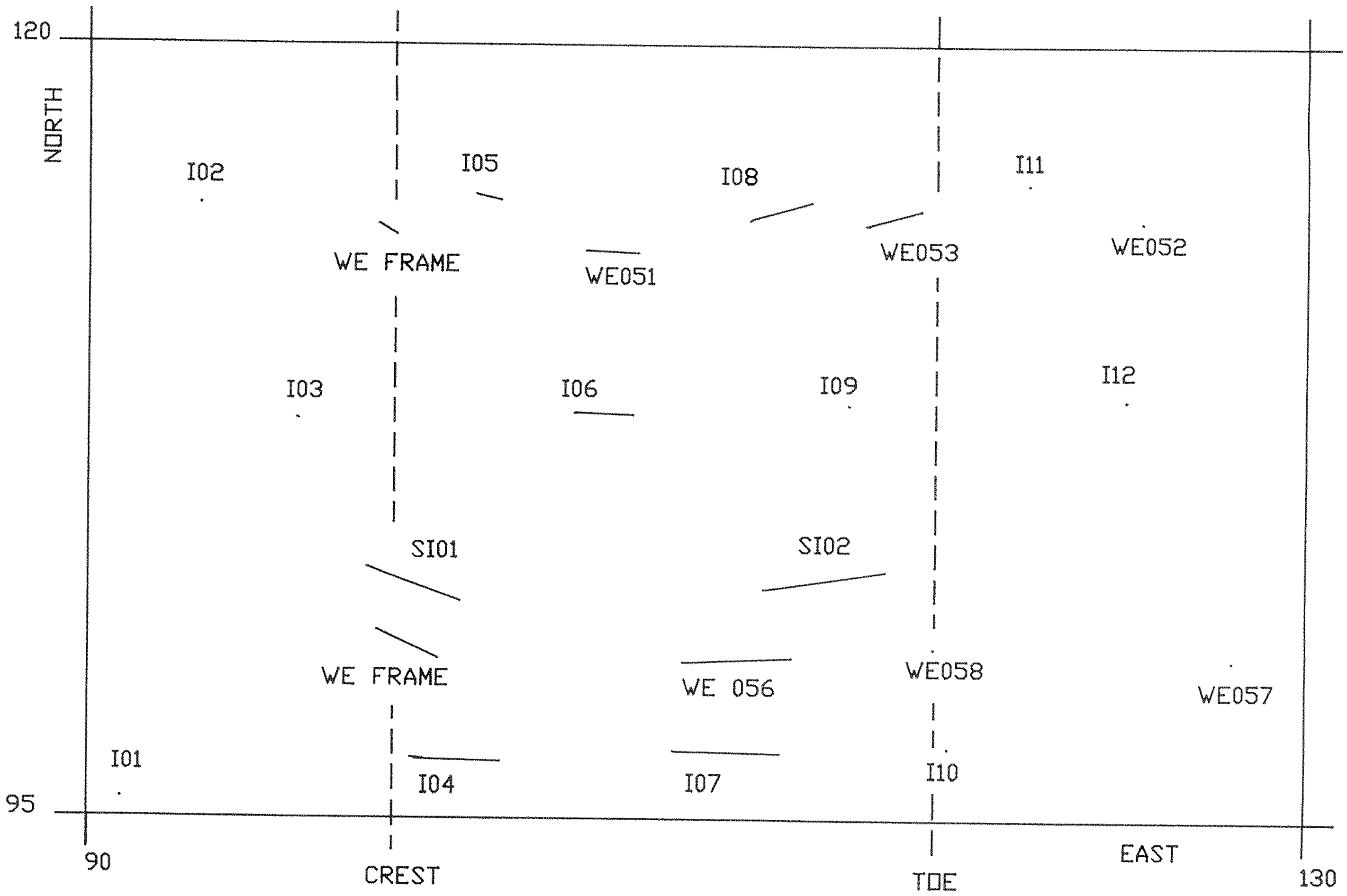


Figure 16-7 Geological section showing the position of the interpolated centreline slip surface



Note:

Vectors are to true scale.

Figure 16-8 Surface movement vectors (from survey results)

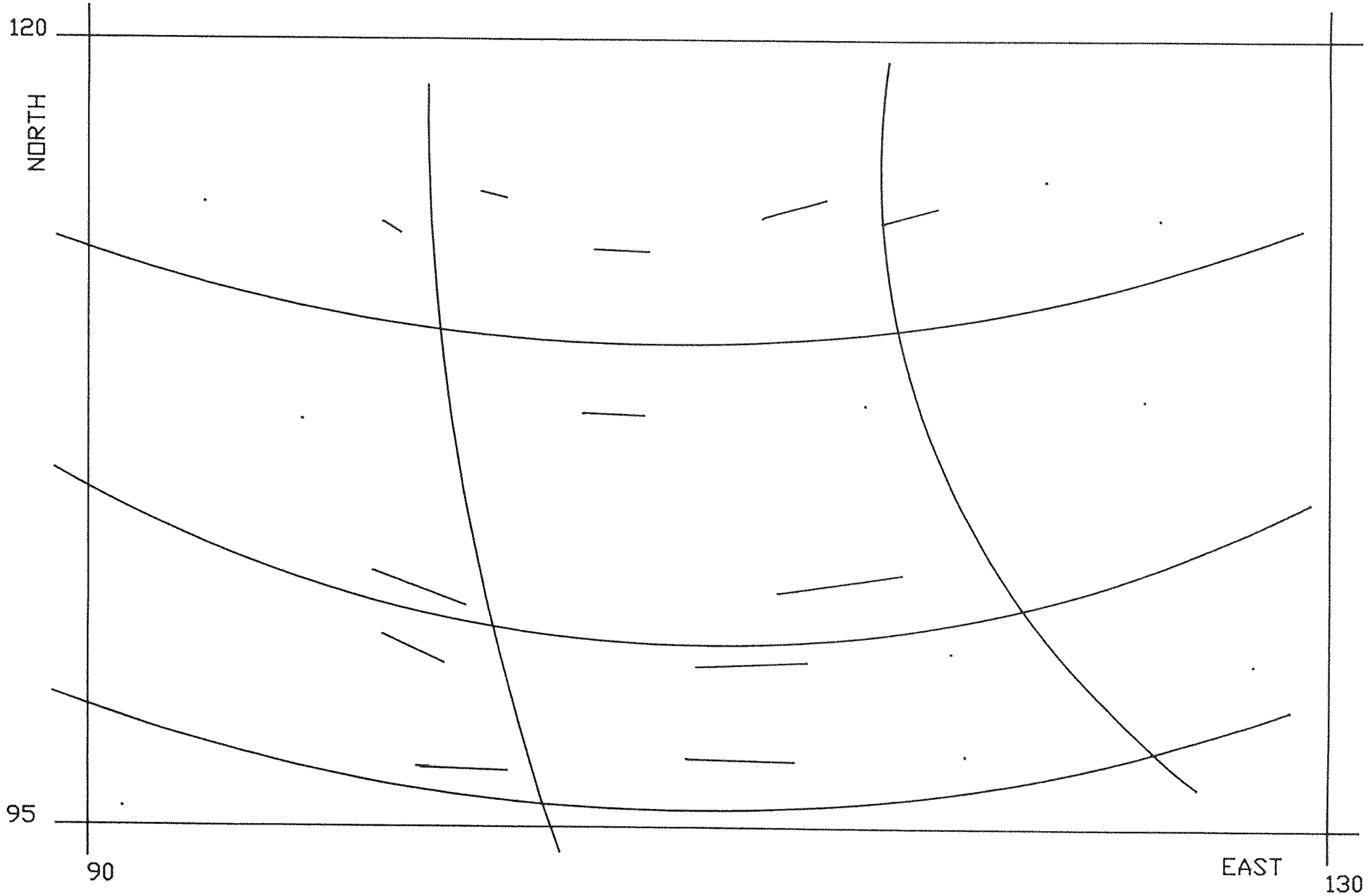


Figure 16-9 Fan of movement

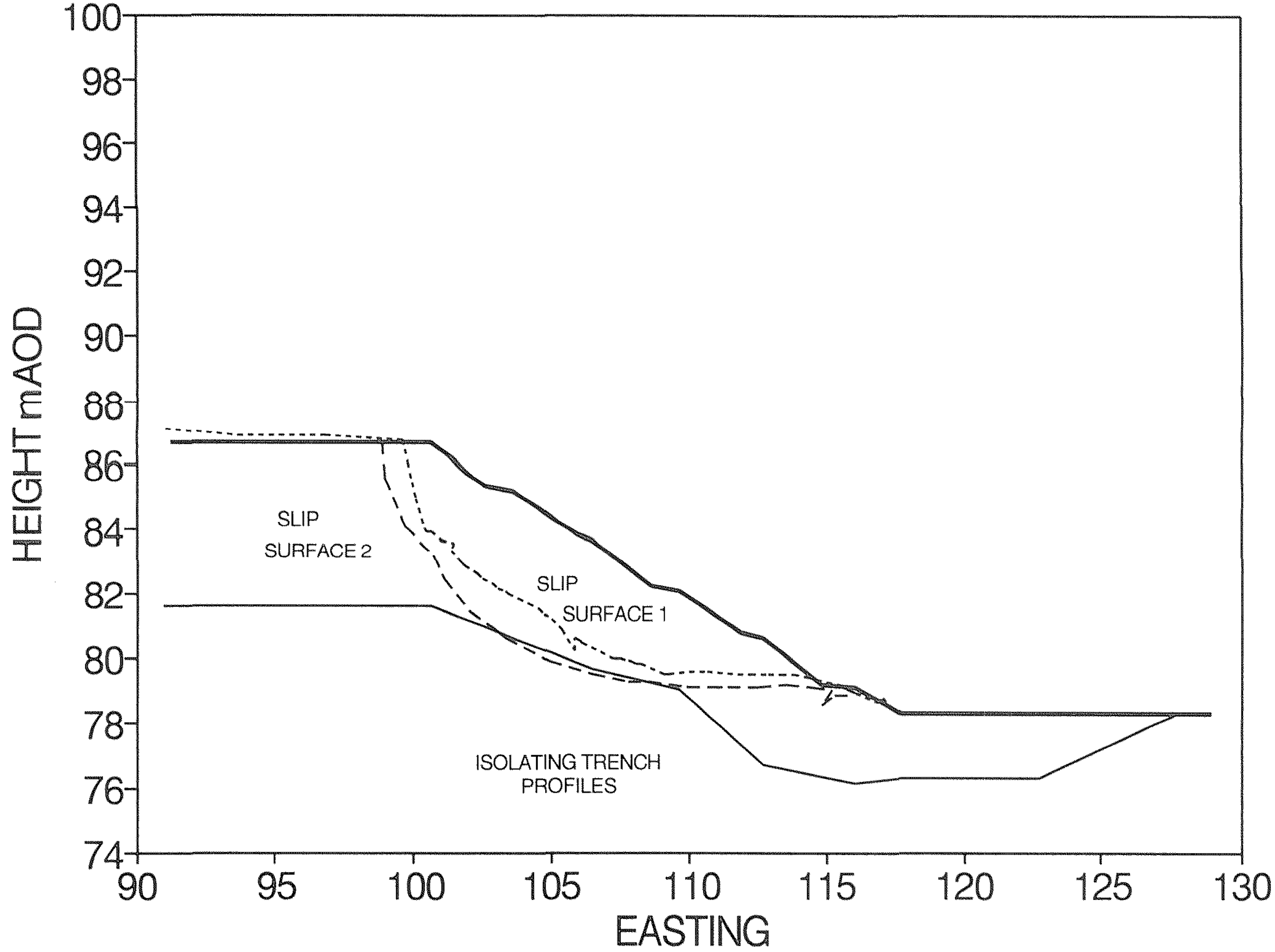


Figure 16-10 Side shear break and slip surface profiles

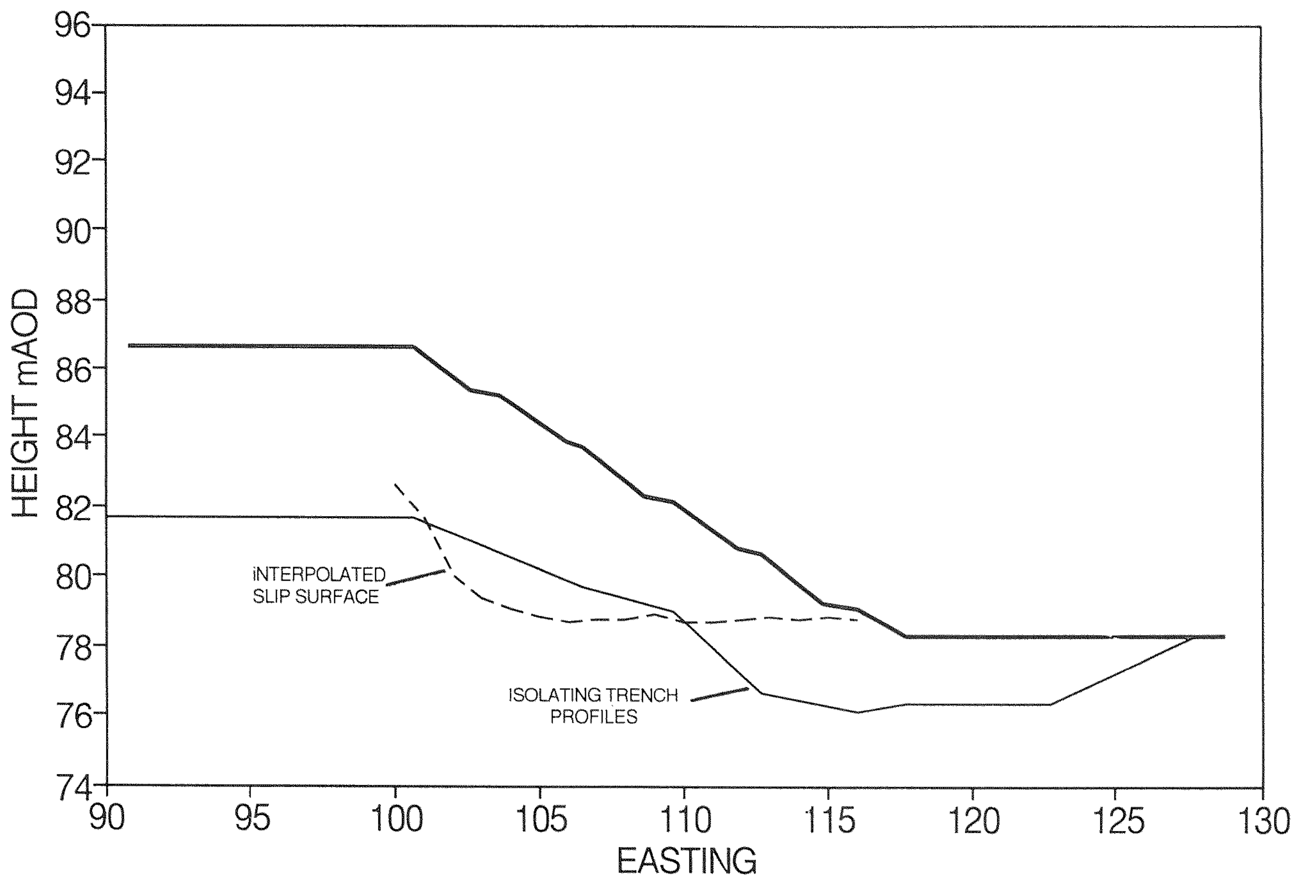


Figure 16-11 Projected slip surface profile at northern side shear break

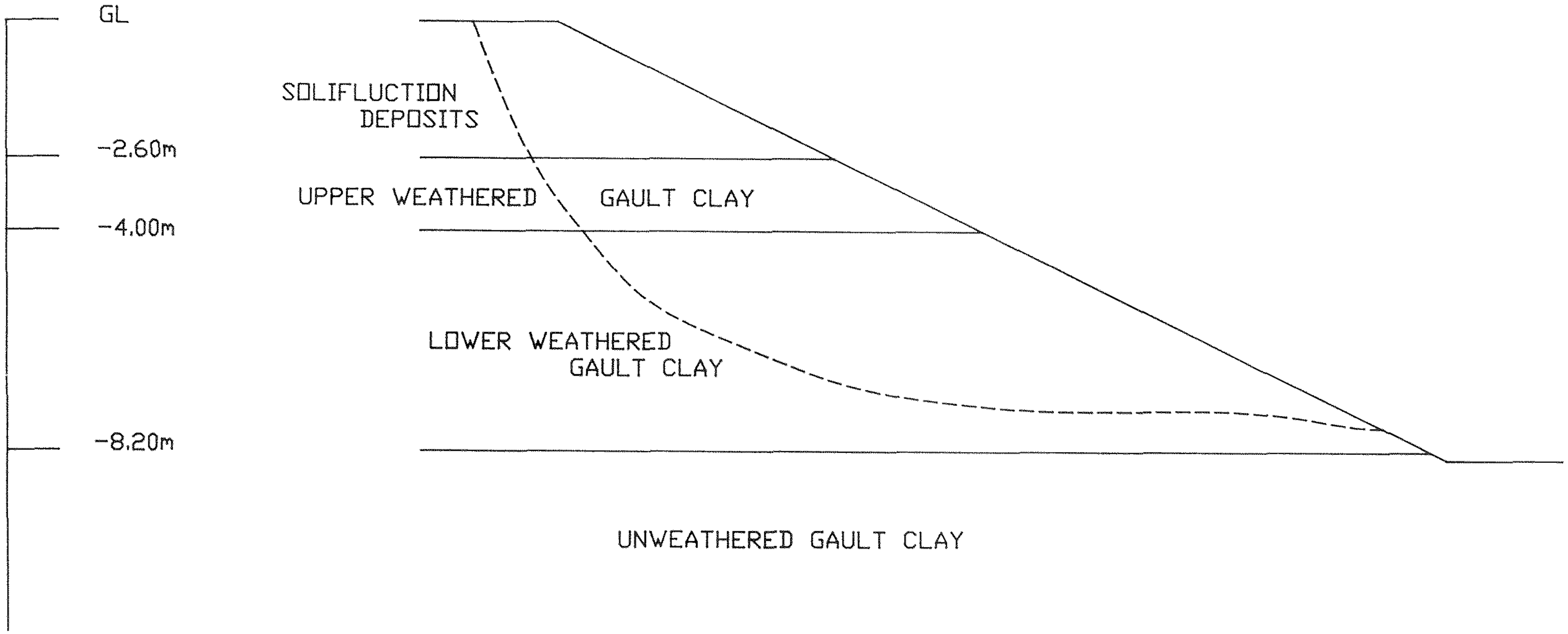


Figure 16-12 Section for SLOPE analysis

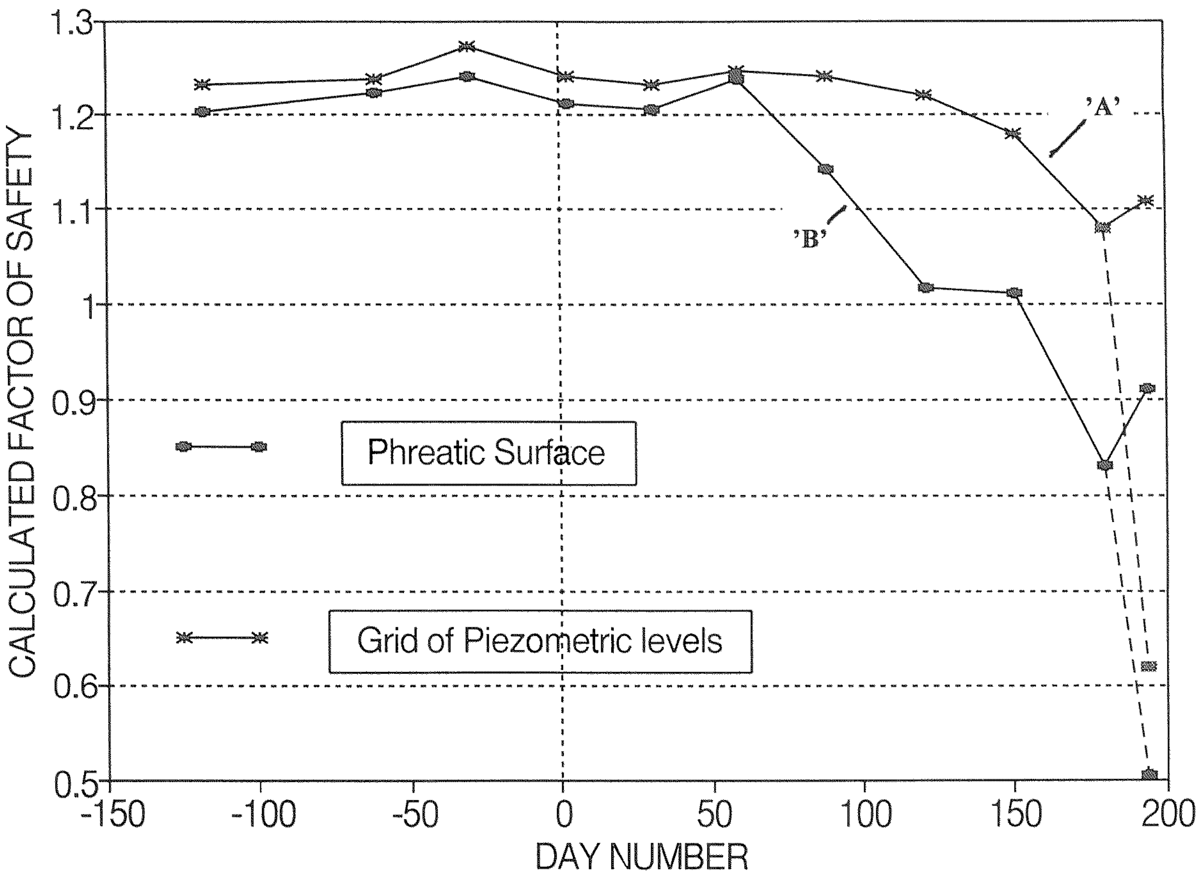


Figure 16-13 Factors of safety from Janbu analysis

The grid positions are those input to the computer program SLOPE (Borin 1993) in order to represent the pore pressure regime at the site. The pore pressures at each grid position were interpolated from the field data.

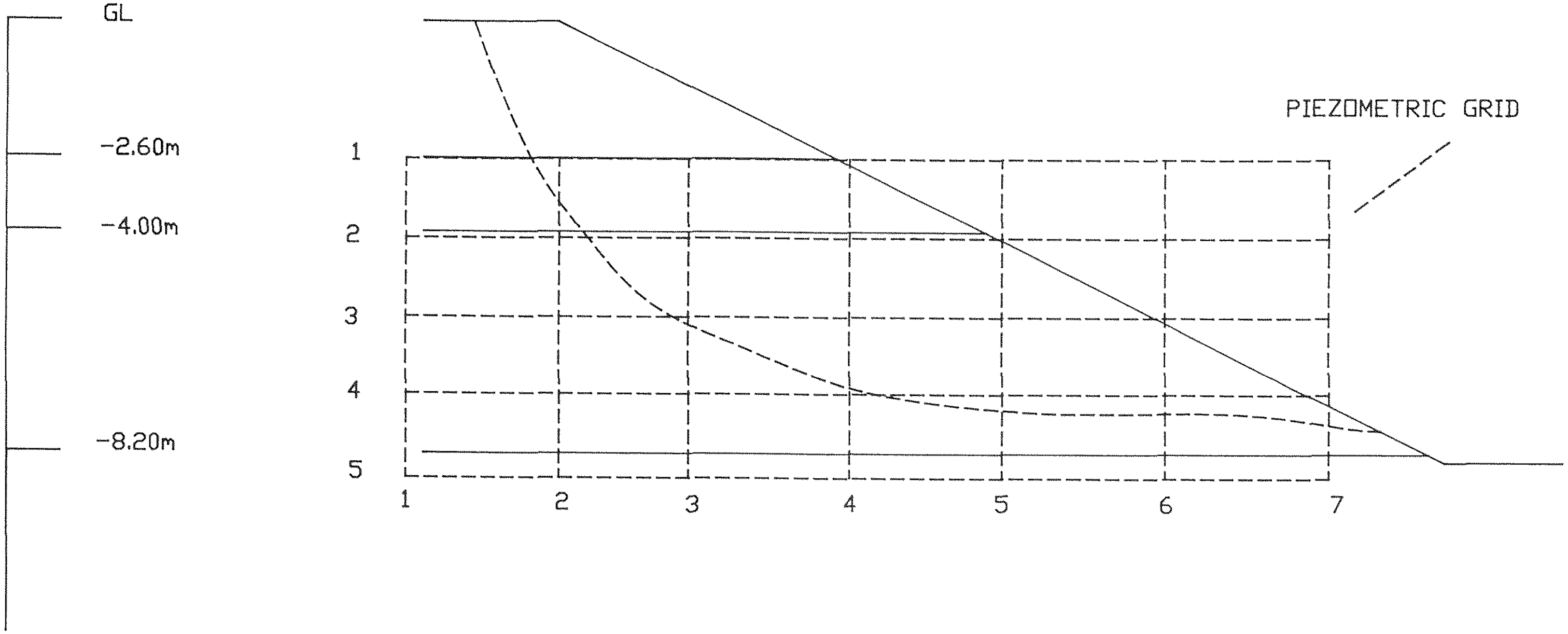


Figure 16-14 Piezometric grid for SLOPE analysis