Chapter 10

Basic field instrumentation for site investigation

INTRODUCTION

Soil instrumentation is a complex and rapidly evolving field of study, and has been covered in detail by many authors. Among the more comprehensive accounts are those by Hanna (1985) and Dunnicliff with Green (1988); in contrast this chapter deals only with those types of instrument which are either in common use or are thought to be relatively easy to use during site investigations.

The amount of instrumentation used in site investigation depends on the type of investigation being carried out. In practice the amount of instrumentation used in routine pre-design site investigation is very limited and normally consists only of pore water pressure measuring devices. In the case of investigations for deep excavations in rocks, measurements of *in situ* stress are also made.

In contrast to this, trial construction, the investigation of the safety of existing works, and the investigation of failures to allow the design of remedial works all typically involve considerable and quite variable instrumentation. The main parameters which may require measurement are displacement, strain, stress and force; pressure in the form of pore water pressure will be the most frequent measurement because of the relative importance of this parameter in geotechnical design.

USES OF INSTRUMENTATION

Site investigation carried out before design will always require the determination of pore water pressures. As a very minimum, the groundwater level and its seasonal variations should be determined, because this information is vital in assessing the geotechnical information provided by boring and testing, and more importantly because groundwater conditions play a very significant part in choosing foundation types, their levels, and the precautions necessary during their construction. Despite this, it is rare to see a site investigation report which not only has an adequate number of measuring points, but also has records made over a sufficiently long period to ensure that seasonal fluctuations, artesian pressures, underdrainage, and tidal variations are detected. The importance of good groundwater information to designer and contractor is hard to overemphasize.

Pre-construction trials are carried out relatively infrequently because of their cost. They may be carried out purely for research, or to provide design information which cannot accurately be obtained by less expensive techniques and which will have a significant effect on the cost of a proposed structure. An example of this type of study is the trial embankment, where a section of earth-fill may be placed to provide information on the suitability of the proposed soil as fill, to provide method specifications for handling and compacting available materials, to check on the stability of a proposed embankment geometry, or to determine the probable amount and rate of settlement. Depending on the reason for construction, observations of a trial bank may vary from visual records of plant performance, and density measurements, to a complex layout involving the measurement of pore water pressure, settlement, lateral displacement and earth pressure. In theory such a system has a better chance of surviving the construction process than if it is installed in the actual works, because the construction plant is controlled by the engineers who install the instrumentation; in practice it is still necessary to install considerably more instrumentation than is strictly necessary to provide the relevant data.

Instrumentation placed to monitor performance during and after construction of the works may once again be for the purposes of research, or for more straightforward economic reasons. Work by the Building Research Establishment in England has shown the importance of instrumenting full-scale structures in developing an understanding of the mechanisms involved, particularly in earth dams and large diameter bored piles, deep excavations in stiff clays and settlements of structures (Cooke and Price 1973; Penman and Charles 1975; Burland and Hancock 1977; Penman 1978).

On the other hand, instrumentation may be used to allow less costly construction without the danger of failure, as proposed in the 'observational method' (Peck 1969) and it has also often been used to check the stability of embankments on soft ground during construction. These latter applications require the regular and rapid reading and processing of data from the instruments. Failure of site staff to appreciate the importance of this has been the cause of failures.

Whereas instrumentation used to monitor the performance of a civil engineering structure during and after construction may reasonably be specifically placed to obtain certain key measurements, post-failure instrumentation has inbuilt dangers. In order to determine the cause of a failure and the parameters necessary for the design of remedial measures some assumptions concerning the failure mechanism will be necessary. These pre-conceived ideas may well prove wrong and therefore it will be wise to make a generous allowance for a wide spread of instruments.

REQUIREMENTS FOR INSTRUMENTATION

The primary requirement of any instrument is that it should be capable of determining a required parameter, such as water pressure, or displacement, without leading to a change in that parameter as a result of the presence of the instrument in the soil. This aspect of instrument performance will be discussed in detail in later sections of this chapter.

In addition, since most soil instruments will be placed in an hostile environment, it is important that they should be robust and reliable. Most instrumentation cannot be recovered from the ground if it fails, and it will often be abused during installation or during construction of the works.

Even where instrumentation is as simple, reliable and robust as possible, a proportion must be expected to fail to work, or to be destroyed by construction plant or vandals.

It is necessary that any instrumentation should be sufficiently duplicated and plentiful to allow for losses, and it is therefore helpful if those instruments which are most at risk are cheap.

Obviously, any instrumentation which is installed must be capable of measuring relevant properties. Relevance requires sufficient accuracy, correct positioning, and a suitable speed of instrument response to changes occurring in the soil.

PORE WATER PRESSURE AND GROUNDWATER LEVEL MEASUREMENT

This is the most common form of *in situ* measurement, and fortunately only one measurement is required at any point to define the regime. Quite simple devices are often used to determine water pressure in the ground, but these devices are unsuitable under many conditions. Hanna (1973) has defined the requirements of any piezometer as:

1. to record accurately the pore pressures in the ground;
2. to cause as little interference to the natural soil as possible;

3. to be able to respond quickly to changes in groundwater conditions;
4. to be rugged and remain stable for long periods of time; and
5. to be able to read continuously or intermittently if required.

Not all of these requirements are necessary for every piezometer installation and clearly the speed of response is a matter of relativity. Hvorslev (1951) investigated the response time in piezometers and showed that because some flow of water from the soil into the piezometer system is required for any piezometer to record pressure changes, and because the soil surrounding the piezometer presents a resistance to flow, a time lag must exist between the groundwater pressure changes and the recording of that pressure change by the piezometer. This problem is discussed in Chapter 9, but in simple terms the hydrostatic time lag is proportional to the volume of water that must flow into the piezometer for a given pressure change, and in addition it is inversely proportional to the permeability of the soil surrounding the piezometer tip.

Standpipes and standpipe (Casagrande) piezometers

The simplest form of pore pressure measuring device is the observation well or standpipe. This consists of an open-ended tube which is perforated near the base, and is inserted in a borehole. The space between the tube perforations and the wall of the borehole is normally packed with sand or fine gravel, and the top of the hole is then sealed with well tamped puddle clay or concrete to prevent the ingress of surface water (Fig. 10.1).

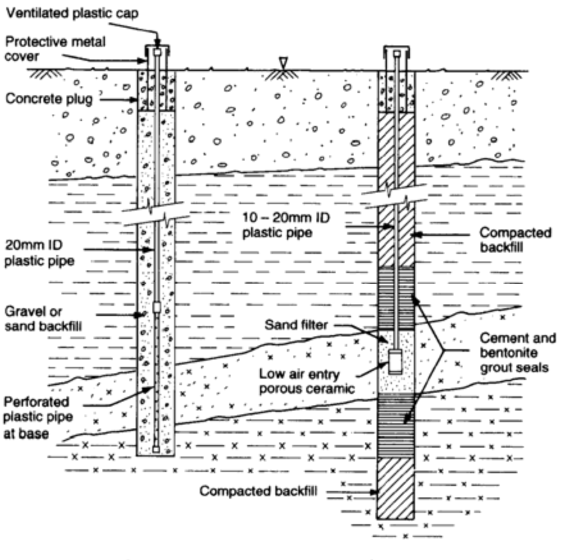


Fig. 10.1 Standpipe and standpipe (or Casagrande) piezometer.

Measurements of water level in the standpipe are made by lowering an electrical ‘dipmeter’ down the open standpipe. The standpipe is plastic, and typically 10—20mm dia., and the dipmeter normally consists of a coaxial or twin cable connected at the surface to a battery and some device to detect closure of the electrical circuit. This may consist either of a milliammeter or an oscillator, giving either a visual or a audible signal when the water level is met. The base of the coaxial cable, which is lowered down the standpipe, is covered with a metal probe so designed that the electrical circuit will not be closed by stray water clinging to either the cable or the inside of the standpipe.

The standpipe is very simple to install, but it unfortunately suffers from considerable disadvantages. First, no attempt is made to measure pore water pressure at a particular level, and it is therefore assumed that a simple groundwater regime exists, with no upward or downward flow between strata of differing permeability. When seepage between adjacent strata occurs, for example where a perched water table exists in granular soil above a clay deposit, the water level in the standpipe will be meaningless. A second major disadvantage is due to the considerable length of time required for equalization of the level of water in the standpipe with that in the ground, in soils of lower permeability.

To overcome the uncertainties connected with the standpipe the most common practice is to attempt to determine the water pressure over a limited depth, by sealing off a section of the borehole. The system commonly used is termed a ‘standpipe piezometer’, and consists of a porous tip (sometimes referred to as a ‘porous pot’ or ‘well point’) embedded in sand or gravel at the level of pressure measurement and connected to a plastic tube of 10—20mm dia. which extends to the ground surface (Fig. 10.1). The sand filter is sealed above and below with grout, which typically consists of tremied cement and bentonite pellets or hand formed bentonite balls. The most common tips in use are normally the ceramic type developed by Casagrande, or more modern porous plastic equivalents, both typically having an average pore size of 50—60 tm and a low air entry resistance. The top of the tubing should be protected at ground level by a lockable cover, preferably of steel, set in concrete; vandalism accounts for most of the failures of standpipe piezometers.

The standpipe piezometer accounts for the majority of piezometer installations in site investigations, primarily because it is relatively simple to install. There are, however, a variety of precautions that must be taken if a really good piezometer installation is to be made. First, if the seals are to be effective then backfill to the borehole should be well tamped down. Stiff clays which are thrown back into the borehole without compaction are not only highly permeable, but may well settle with time and endanger the continuity of bentonite seals above them. During the installation of the lower bentonite seal it will be wise to keep the borehole cased to below the level of the bottom of the sand filter, or typically about 0.45 m below the proposed tip level. The bentonite and cement should be mixed in 1:1 proportions in a motorized grout mixer, and the mix should be as stiff as is compatible with tremie pipe placement at the base of the hole. Typically the cement/bentonite seal would be 2 m long. Vaughan (1969) has examined the problems of sealing piezometers installed in boreholes, where the grout seal extended up to ground level, and concludes that for ‘a typical installation’ the permeability of the seal can be significantly higher than that of the soil surrounding the piezometer tip without serious errors arising. This illustrates the value of back- filling the entire hole with grout, rather than using relatively short seals.

Some attempt should be made to prevent the upper surface of the wet cement! bentonite seal from contaminating the sand filter; this can be achieved by either dropping hand-made cement/bentonite balls of a stiff consistency down the hole, or by using bentonite pellets. Of course bentonite pellets could be used for the entire seal, but this will normally be expensive for the rather large (200mm dia.) borehole typically used in the UK. The upper surface of the seal should be tamped with a metal disc attached to light rods, in order to form a horizontal surface, compact the bentonite balls or pellets, and measure the position of the top of the seal.

Any water in the base of the hole, at the sand filter position, will inevitably be badly contaminated with bentonite. If the piezometer is to be used for permeability measurement, then the water should be changed before the sand filter is placed. In order to achieve this without damaging the bentonite seal it is possible to provide protection by dropping a permeable sack (nylon stocking) of sand on to the top of the seal, and tamping this in place. A shell can then be gently used to remove the contaminated water. Once this is done further sand can be dropped from the top of the hole until the level of the porous pot is reached. Once the pot is positioned, the casing can be pulled back to the level of the top of the sand filter, and further sand placed until the correct level is reached. When a hole is full of water, the sand may take some minutes to sediment.

The top seal can be formed in the same way as the bottom bentonite seal, with the contact between the sand and bentonite being made up from bentonite balls or pellets. Backfill above the seal should be gently compacted.

In order to speed equalization between groundwater levels and the level in the standpipe, it is important to ensure that the sand filter is saturated when placed. In addition, once the bentonite/cement seal has had time to stiffen up, the piezometer standpipe tube should be topped up with water. If these precautions are not taken, equalization may take several months (Fig. 10.2), in soils of low permeability. After topping up the standpipe, readings of water level should be taken at intervals of a few days in order that the equalization of the piezometer can be assessed.

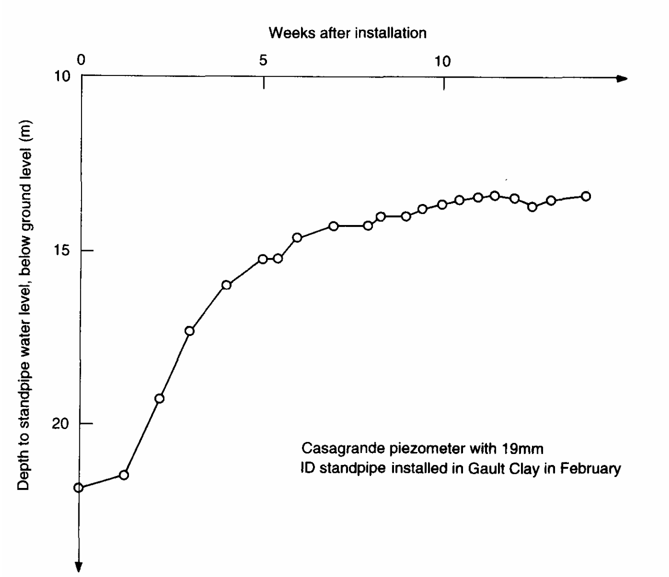


Fig. 10.2 Extreme example of equalization in a Casagrande standpipe piezometer.

The standpipe and Casagrande or standpipe piezometer are the two devices in most common use in site investigation but they are not suitable for some applications, particularly where it is not possible to read the water level in the standpipe from directly above, or where pore water pressure responses to relatively rapid load changes must be measured. Because of the method of recording the water pressure (i.e. by groundwater filling a plastic tube) the response of the system to changes in groundwater pressure is slow. The two most common piezometers used in these circumstances are the closed hydraulic piezometer, and the pneumatic piezometer.

Pneumatic piezometers

The pneumatic piezometer tip typically consists of a ceramic porous stone, behind which is mounted an air activated pressure cell (Fig. 10.3). The tip is connected to instruments at the surface via twin nylon tubes, and these are connected in turn to a flow indicator and a compressed air supply and pressure measuring apparatus. When pore water pressures are to be read, air or nitrogen is admitted to one line, but is prevented from flowing up the other line by a blocking diaphragm in the tip.

When the air pressure reaches the pore water pressure, the diaphragm is forced away from the inlet and outlet tubes in the tip: air returns up the vent line and a visible signal of this is given by air bubbles in the air flow indicator. When the return air ceases to flow, the pressure in the feed line is equal to the pore water pressure. It should be pointed out that the use of compressed air can affect the performance of the piezometer due to moisture carried by the air being introduced behind the diaphragm. In order to avoid this happening the use of bottled nitrogen is preferred, particularly in situations where the piezometer is expected to be in use over a period of several years. Compressed air may be used without seriously affecting the performance of the device for short-term usage. Any moisture that does build up in the system can be removed by periodically flushing with nitrogen.

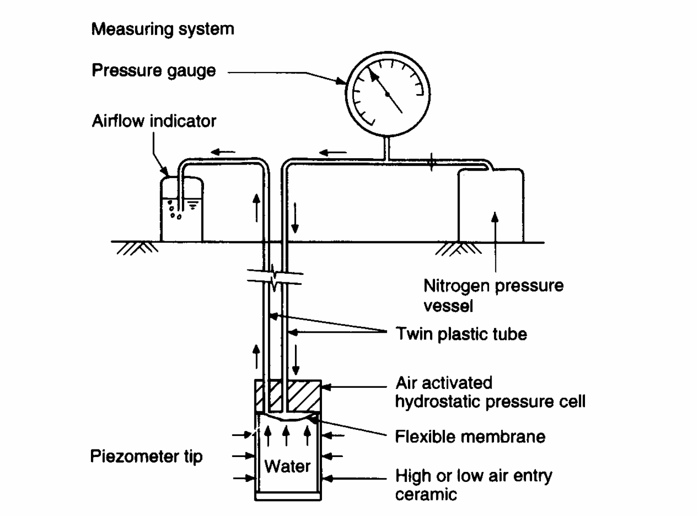


Fig. 10.3 Pneumatic piezometer equipment.

Typically, the amount of water displaced by the diaphragm is very small (<0.1cc) and the total fluid volume of the tip is low, and therefore the time required for equalization between the groundwater pressure and the air line pressure is very small. In addition the piezometer has the advantages that:

1. the reading point (at ground level) need not be directly above the tip, and differences in level between tip and reading point are of no consequence;
2. there are no freezing problems where tubes must pass close to the ground surface, because they do not contain water;
3. the system has good accuracy, with readings of pressure possible to ± 1 kN/m^2, and a system accuracy generally equal to, or better than, ±2 kN/m^2;
4. the system is simple to install and relatively easy to use; and
5. plastic tubes used to connect the tip to the surface can be of relatively cheap nylon, since only relatively low permeability is required.

The pneumatic piezometer is considerably more expensive than a simple standpipe piezometer, with the parts required for installation costing about twice as much, and the system requiring a more sophisticated readout unit. Once the cost of installation is included, however, the cost differential will be very much less significant. Despite the many advantages of the pneumatic piezometer discussed above, the system does have some disadvantages. It has been suggested, for example, that in the long term some pneumatic piezometers become unreliable, whilst because there are no fluid connections to the soil it is not possible to use this type of piezometer for *in situ* permeability determination. Further problems arise because of the mode of operation of the system: clearly it is not possible to measure negative pressures, and in addition the fluid chamber inside the piezometer cannot be de-aired once the device is installed. These two features make the application of pneumatic piezometers to earth-fill problems undesirable.

The hydraulic piezometer

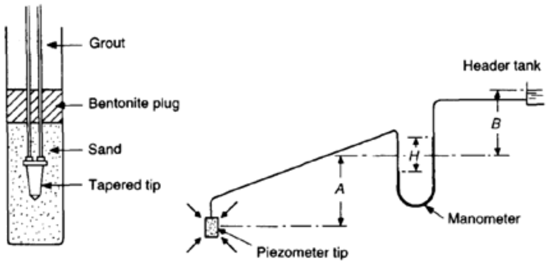
The closed-hydraulic piezometer was developed at the Building Research Station (Penman 1956) and is widely used in unsaturated rolled earth-fill applications. When relatively dry earth-fill is compacted (for example in earth dams) then the pore space will contain both air and water. Because of surface tension effects in the bubbles of air, the pore air pressure will be considerably higher than the pore water pressure.

If the pore size of a piezometer ceramic is large, then both air and water can freely pass into the chamber within the tip. If small pores are used in the ceramic, however, water will pass through the ceramic but the air will require a considerable pressure differential before it can penetrate the pores. This pressure differential between the inside and outside of the ceramic, at which air will pass through, is termed the air entry value, If a low air entry value ceramic is used then the pore air pressure will be recorded (Bishop and Vaughan 1962), but when a high air entry ceramic is installed, it will be possible to record pore water pressure, so long as the difference between the pore air pressure and the pore water pressure does not exceed the air entry value. Typical values for two commercially available ceramics are given in Table 10.1.

Table 10.1 Typical values for two commercially available ceramics

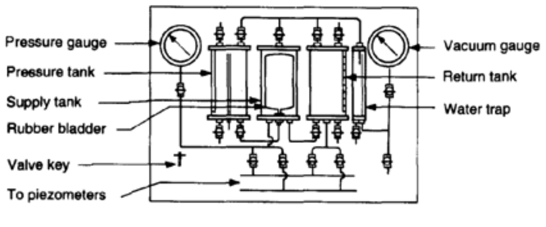
Property	Low air entry ceramic	High air entry ceramic
Average pore size (μm)	60	1
Coefficient of permeability, K(m/s)	3x 10^{-4}	2x 10^{-8}
Air entry value (kN/m^2)	5	100

When using the closed hydraulic piezometer, measurements of water pressure are made at a point which is remote from the piezometer tip. The tubing connecting the tip to the measuring device must be filled with a relatively incompressible fluid; to achieve this two tubes connect the tip to the measurement point at ground surface and these are flushed with de-aired water before taking readings. Figure 10.4 shows a typical twin-tube piezometer tip designed for installation in a preformed hole of the same size in compacted fill, together with a diagram illustrating the method of measuring the pore pressure and de-airing unit.



(a) Piezometer tip

(b) Header tank with mercury manometer for pressure measurement



(c) Panel mounted de-airing unit

Fig. 10.4 Twin tube hydraulic piezometer equipment.

Measurement of pressure at ground surface may be carried out either by mercury manometer or pressure transducer. Pressure measurements may be relative to atmosphere or to some other constant pressure. This may be provided by a water header tank or by a mercury constant head back pressure unit similar to that described by Bishop and Henkel (1962). The pore pressure at the piezometer tip is inferred from the difference in level between the tip and the measuring point.

The twin-tube hydraulic piezometer has the advantages of being relatively simple and inexpensive.

Although the system must be de-aired, the frequency with which this must be done is reduced by using nylon tubing coated with polythene. Because the piezometer has two tubes, two independent readings can be made from the same tip in order to provide a check. The response time of the system is generally low, but depends on the quality of de-aired water within it, the type of pressure measuring device, and the size and length of leads connecting the tip and pressure measuring device. Provided the system is filled with good quality de-aired water the response time has been shown by Penman (1961) to be largely dependent upon the tubing connecting the tip to the pressure measuring device. This is due to volume changes in the tubing in response to a pressure increase and is therefore related to the type of tubing, the length of tubing, and the restraint offered by the soil when it is buried. Polythene tubing suffers relatively large volume changes for a given pressure increment, (about $0.198\text{mm}^3/\text{kNm}^2$ per m run), whereas the expansibility of nylon tubing is much less (about $0.048\text{mm}^3/\text{kNm}^2$ per m run for nylon 66 tubing). The disadvantage of using nylon tubing however is that it will allow the diffusion of water through the walls and hence may affect the response time. The rate of diffusion of water through the walls of buried tubing is not known although it is thought to be small (Penman 1961). A suitable compromise is the use of a composite tubing comprising nylon walls with an outer skin of polythene to prevent any diffusion. Tests by Penman (1961) have shown that the effect of introducing 300 m of polythene tubing between the tip and the pressure measuring device is to increase the response time by a factor of 50. This can be reduced to a factor of about 15 by the use of nylon tubing. The effect of burying the tubing may only reduce the response time by about 25%. It should be pointed out that response times can be further increased if the pressure measuring device is not very rigid. The use of an electrical pressure transducer such as that described by Margason *et al.* (1968) allows response times of less than 5s to be achieved for the pressure measuring device alone.

In view of the factors mentioned above the response time of the system should be calculated. Methods of calculating response times have been given by Hvorslev (1951) and Gibson (1963). Hvorslev’s method ignores the compressibility of the soil skeleton and hence it is only suitable for coarse-grained soils. Penman (1961) has shown that this method may lead to appreciable errors when attempting to estimate the pore pressure in a clay soil from piezometer readings taken long before equilibrium has been established. Gibson’s method takes into account both the compressibility and the permeability of the soil and is therefore suitable for clays and coarse-grained soils.

Problems may arise when de-airing twin-tube piezometers if the pressure at the tip is changed. To avoid the effects of hydraulic fracture, or excessive swelling or consolidation, the pressures applied to the two tubes should have an average value equal to the pressure at the tip. If the gauge house is much higher than the piezometric level then water in the tubes will be in tension; under extreme conditions cavitation will occur. Where the tubes pass close to the ground surface, problems of freezing can be avoided by using an ‘antifreeze’ mixture. One marketed by Geonor consists of 5.51 of glycerine, 5.51 of alcohol, 4 ml of concentrated sulphuric acid, and 101 of water.

DISPLACEMENT MEASUREMENT

Measurements of displacement may be made relative to time, and to some datum remote from the point of measurement. A straightforward method of monitoring absolute displacement is to use conventional surveying techniques: the type of datum required for such a scheme will depend upon the accuracy to which measurements must be made. If only low levels of accuracy are required then a pre-existing datum such as an Ordnance Survey Bench Mark might be satisfactory, but in most applications it will be necessary to construct a more suitable datum. This may consist of a metal point installed on a suitably rigid structure, or it may consist of a surface monument. The simplest type of surface monument can be made by concreting a steel bar and anchorage plate into a hole in the ground. Bearing in mind that many soils are subject to seasonal movement, even in the temperate climate of the British Isles, the depth of the concrete block below ground level is important. For example, it would be unwise to found the block at less than 2.Om in London clay, but in gravel a much shallower depth might be suitable.

Accurate measurements of absolute displacement are very difficult. Green and Cocksedge (1975) and Green (1975) report some of the problems in monitoring New Zealand House in London. In this case a benchmark on a bank was used. Despite the fact that this structure had been built many years before, it was later found to be moving.

When high accuracy is obtained, the regional movements of the Earth's crust are detectable (Wilson and Grace 1942; Green 1975). In London this may amount to about 3 mm per annum.

One method of reducing the problems associated with moving benchmarks is to use a purpose built benchmark which incorporates a steel bar driven to bedrock. Since this bar will typically be rather long it must be laterally supported at about 3 m intervals. Figure 10.5 shows a suitable layout. The fluid filling around the steel bar is used to ensure that the bar is maintained at ground temperature, and is not subjected to the fluctuating ambient temperatures that an air environment would have.

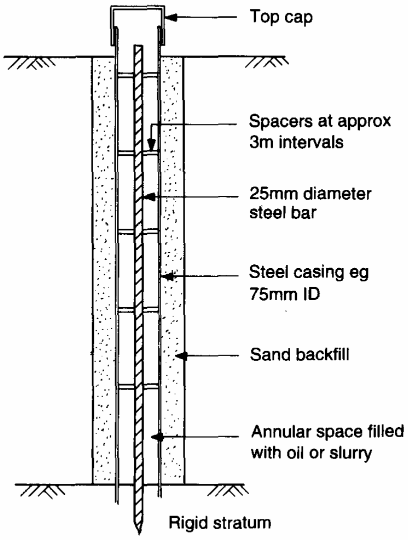
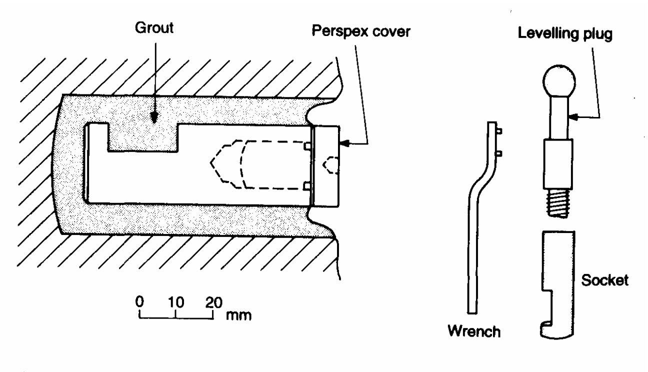


Fig. 10.5 Bench mark driven to bedrock.

When installing datum points it is important to secure them against vandalism. Even when this has been done, it is advisable to install several datum points. All of these should be outside the zone of influence of the structure being monitored.

Many soil instrumentation problems do not require the measurement of displacement in all three dimensions. For example, when monitoring a multistorey building it will normally be sufficient to measure settlement (i.e. movement only in the vertical direction). Displacement measurements can be conveniently split into three groups: vertical movements, horizontal movements and relative movements.

Vertical movements can be monitored whether or not the measurement point is accessible. If the measurement point is readily accessible, then conventional surveying apparatus may be sufficient provided an accuracy no better than ±5 mm is required. Better accuracy will require the use of a precise level, an Invar staff, and accurately machined reference points. Cheney (1974) details a type of levelling station developed at the Building Research Establishment which has been used with considerable success (Fig. 10.6). The system consists of a socket which is grouted into a purpose-made hole in the structure. The socket is threaded to accept a levelling plug which ensures radial positioning to ±0.03 mm. When not in use the socket is protected by a perspex cover: it is inconspicuous and virtually indestructible. Under favourable conditions it is possible to obtain levels to an accuracy of about ±0.5 mm.

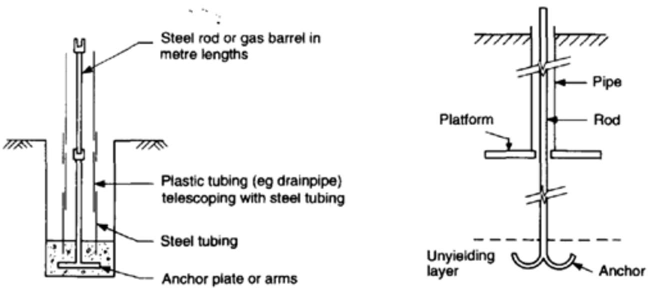


(a) Section of a BRE levelling station grouted into a prepared hole

(b) Components of the BRE levelling station

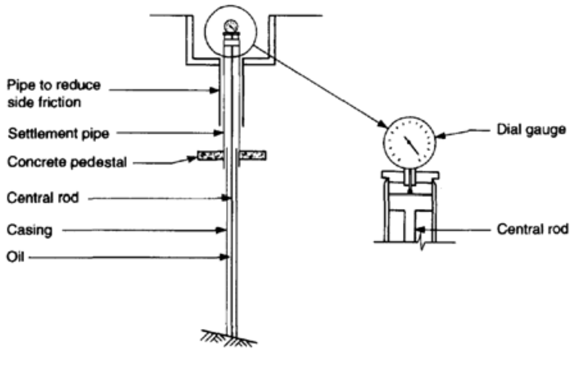
Fig. 10.6 BRE levelling station (after Cheney 1974).

When the measurement point is accessible only from directly above, more sophisticated methods are required. The simplest form of instrument is the rod settlement gauge, which may consist of a plate fixed at the desired measuring point and coupled to a rod extending through telescopic tubing to ground level (Fig. 10.7). Movements of the plate are determined from measurements of the level of the top of the rod made with conventional surveying equipment. Measurements relative to a datum can be made using a settlement platform connected to a rigid pipe, inside of which a rod extends down to an unyielding layer (Fig. 10.7). A more precise version of this instrument is detailed by Bjerrum *et al.* (1965). Simple rod settlement gauges have frequently been used to monitor the settlements of embankments during and after construction. In this application they are very vulnerable to destruction by construction plant.



(a) Simple rod settlement gauge

(b) Rod settlement gauge for relative movement

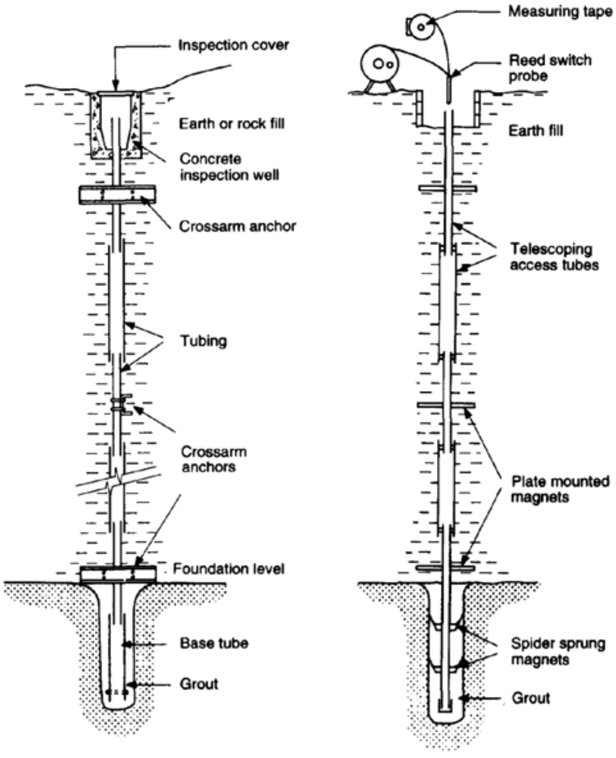


(c) Precise settlement gauge and benchmark

Fig. 10.7 Rod settlement gauges (from Bjerrum *et al.* 1965; Dunnicliff 1971; Hanna, 1973).

The US Bureau of Reclamation Settlement Gauge and the Building Research Establishment Magnet

Extensometer are both devices which can provide vertical displacements for a number of points located above and below each other. The USB settlement gauge (Fig. 10.8) consists of alternating lengths of large and small diameter telescopic tubing which are installed in fill. The small diameter tubes are anchored to the soil by cross-arms which are fixed to the tubes with ‘U’ bolts. Measurement of the level of the bottom of each small diameter tube is made by lowering a probe on a steel tape inside the tubes. Pawls extending outwards from the side of the tube allow the location of the bottom of each small diameter tube and the distance to the top of the tubing can then be read from the tape. A special base tube contains a retractor pin.



(a) USBR Settlement Gauge (b) BRE Magnet Extensometer

Fig. 10.8 USBR settlement gauge and BRE magnet extensometer.

When the probe reaches the bottom of the hole and strikes this pin it telescopes and automatically retracts the pawls. The probe can then be withdrawn, and the top of the tubing levelled to deduce absolute settlements.

The BRE magnet extensometer (Burland *et al.* 1972; Marsland and Quartermain 1974) uses toroidal magnets installed on the side of a borehole to provide markers. A plastic tube is grouted in the centre of the borehole to allow access for a probe which uses reed switches to locate the magnets. When the reed switch is closed by the magnet it closes an electrical circuit and can be used to operate an oscillator. Figure 10.8 shows two methods of installing the device. In fill a similar layout to the USBR gauge can be used. The fixing of the magnets to the plates will give a much more reliable system than can be achieved when the system is used in a borehole. Here the magnets must be pushed down the borehole over the central tube, and they are supported on spider springs. This arrangement means that the magnets may tilt or drop during grouting, and that they may subsequently move if the grout moves. Possible sources of error are that the reed switch may become magnetized by accident and subsequently operate in a different position, that an uncentred probe gives erratic readings, and that the magnetic field is not symmetrical. This last problem can perhaps be overcome by always inserting the probe in the same orientation.

Where the measuring point is not even accessible in plan, fluid settlement gauges or electrolevel trains are necessary. Hydraulic settlement gauges rely on a cell containing an overflow pipe (Fig. 10.9).

Water is passed down a line to the cell until a constant level on a measuring tube remote from the cell indicates complete filling of the tube. The level of the water in the tube is assumed to be that of the overflow in the cell. This type of instrument has a repeatability in the range ±5- ±l0mm, (Penman and Mitchell 1970), but it can be difficult to use if certain precautions are not observed. First, air bubbles in the water line should be removed by flushing thoroughly with de-aired water. If the water line has an internal diameter greater than 2.5-4.0 mm, it may not be possible to flush out the air because the water may simply by-pass any bubbles. Secondly, care should be taken when laying the pipes to the cell to avoid placing tubing above the cell level. In addition, blockages in either the drain or air pipes will cause inaccurate readings.

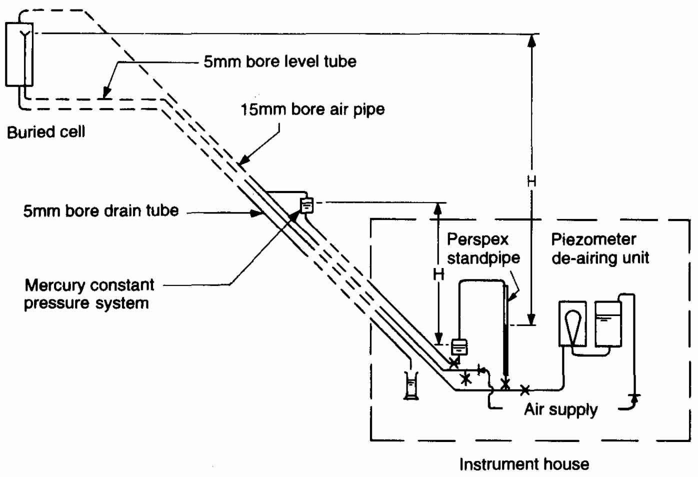


Fig. 10.9 Hydraulic overflow settlement gauge (from Penman 1972).

The mercury-filled settlement gauge (Irwin 1967) is a more sophisticated version of this device, as can be seen in Fig. 10.10. The vertical distance between the cell in the ground or structure and the top of the mercury in the left-hand tube of the indicator unit is obtained by measuring the difference between mercury manometer levels at the same pressure. The mercury in nylon tube A (Fig. 10.10) can be moved by air pressure so that an electrical circuit is completed between the stainless-steel couplings at C and D. Gas is pumped into the Tee-piece coupling at F until the electrical circuit is broken at C, and the pressure is then slowly released until the contact is remade. At this point the difference in mercury manometer levels is recorded. Because of Health and Safety legislation the use of mercury is now avoided wherever possible.

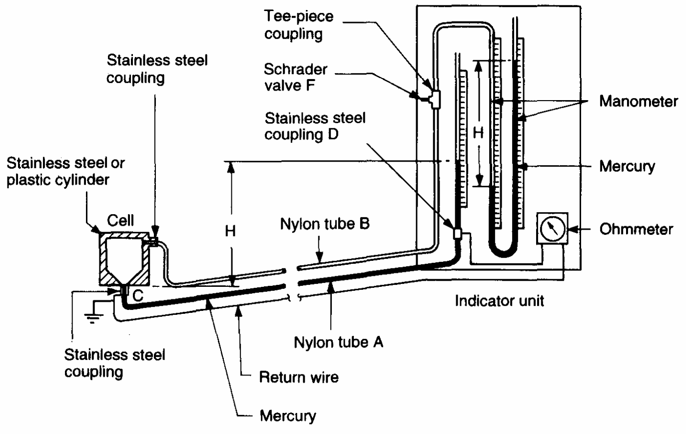


Fig. 10.10 Mercury-filled settlement gauge (from Irwin 1967).

The use of air in the system can introduce water vapour and encourage oxidation of the mercury. Bottled nitrogen overcomes these problems. Other difficulties may occur if the mercury column breaks up, or water gets into the system. The mercury may break up as a result of the surface texture of the nylon tubing; nylon 66 has been found satisfactory. When water enters the system the mercury must be blown out, and the tubing flushed through with alcohol.

The accuracy of the mercury-filled settlement gauge is about ±2.5 mm, but can often be less than ± 1 mm. If there is a large difference between the cell and gauge house temperatures, then a density correction must be made for the mercury levels.

The hydrostatic profile gauge (Bergdahl and Broms 1967) provides a method of measuring a settlement profile, for example, for an embankment cross-section. An access tube is buried at the desired level, and anchored to a concrete pad at each end (Fig. 10.11). A nylon draw cord is placed through the tube, by blowing a piece of rag tied to one end, using a compressor. Protective caps should be arranged to secure the cord and tubes from vandalism. Since construction workers often require rope, protective measures should be substantial. The measuring apparatus consists of a digital or analogue pressure transducer readout box, and a probe and tube connected to a tube drum. In its original form the probe contains a flexible bladder which is filled with antifreeze mixture and connected via an antifreeze filled tube to a pressure transducer in the drum. A second tube equalizes the pressure around the outside of the bladder in the probe, with that in the drum. Changes in level of the probe produce changes of pressure at the transducer. The equipment is used by placing the drum on the concrete pad, and drawing the probe through the tube while stopping to make measurements at known distances from the end of the tube.

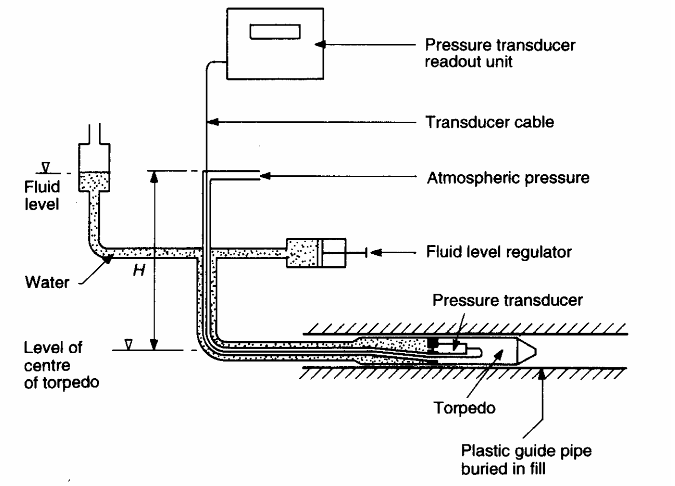


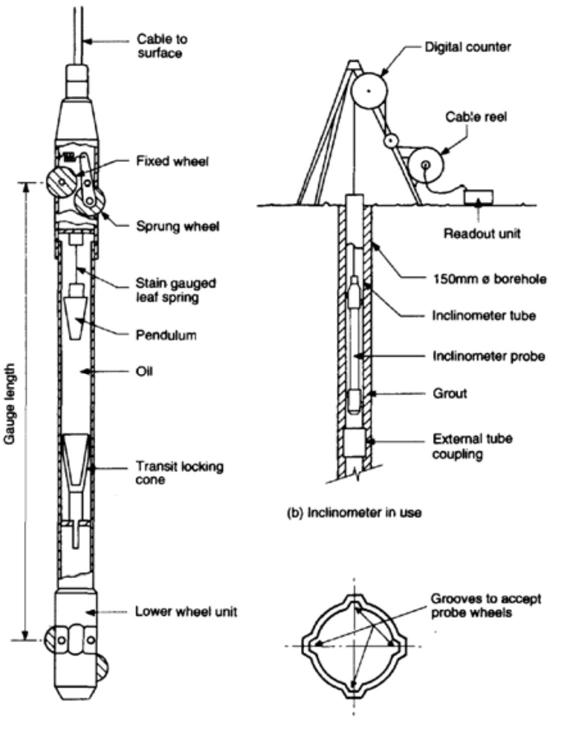
Fig. 10.11 Improved hydrostatic profile gauge (based on Bergdahl and Broms 1967; and Borros AB).

This apparatus is easy to use, but it is not as accurate as devices previously mentioned, being capable of an overall settlement or heave accuracy of only ±10 mm. In addition, the tube must be installed so that it does not give a difference in level between the probe and the readout of more than about 4 m at any time during the life of the installation.

Horizontal movements can also be monitored whether the measurement point is readily accessible or not, and the simplest form of horizontal movement can be made using surface monuments and steel or Invar tapes.

Where access is not available, and the object of the instrumentation is simply to detect the level at which horizontal movement is occurring, for example, on the shear surface of a slope failure, then the slip indicator may be useful. In its simplest form the slip indicator consists of an Alkathene tube placed in the ground inside a hollow metal tube, which is subsequently removed. A short length of rod on the end of a nylon rope is passed down the tube and left at the base of the hole. When a shear surface develops and distorts the Alkathene tube the rod may be pulled upwards until it jams in the tube, thus locating the position of the slip. Another rod is then passed down the tube from the surface and should indicate the same position for the kink in the tube provided only one shear surface has developed. Lines of indicators can give the profile of a slip, provided that the movements of the soil are large enough.

If relatively small movements are expected then a slope indicator or inclinometer should be used to detect horizontal movement. A grooved guide tube is grouted into a borehole of 100—150mm dia. Measurements are made by lowering a probe down the guide tube, and making readings typically every 0.5 m. The probe is connected to a readout box, and the system operates by detecting the orientation of the probe with respect to the vertical in one plane. Figure 10.12 shows that the probe is arranged to run down two diametrically opposite grooves, and the measured inclination is that of the groove in which the two fixed wheels run. Measurements are normally made at vertical intervals equal to the fixed wheel spacing, and the deflected shape of the guide tube is approximated by a series of short straight lines.



(b) Inclinometer in use

(a) Inclinometer probe

(c) Cross-section through tube

Fig. 10.12 Inclinometer equipment (Soil Instruments Ltd).

Various types of sensors have been used in inclinometers. These include strain- gauged cantilevers, pendulums attached to rotary electrical potentiometers, vibrating wire apparatus, force-balance accelerometers and, more recently, electrolevels. The readout units associated with each of these sensors have been arranged to give a variety of display such as the angle of inclination, the sine of the angle of inclination, the relative displacement of the two ends of the probe, and the sum of the displacements as the probe is moved from the bottom of a guide tube to the top.

Typical quoted performance figures for inclinometers indicate an inclination range of $\pm 30^{\circ}$, with a sensitivity of 0.05—0.10mm over the 0.5 m gauge length. These instruments will not perform if the tube becomes sharply bent: the Slope Indicator Company quotes a minimum radius of curvature negotiable by the ‘Digitilt’ probe as 3m. Green (1973) and Murray and Irwin (1970) have carried out full-scale trials under laboratory conditions by lashing the guide tube to frames and then deforming it. Green found errors of less than 15mm in a 24 m length of guide tube for both a Slope Indicator Company instrument and a Soil Instruments Mark 1 inclinometer. Murray and Irwin tested a Soil Instruments Mark 1 inclinometer in 6 m of guide tube and found errors of up to 7 mm for a maximum horizontal displacement of 150 mm.

The figures quoted above must represent much better precision than can be obtained in the field. Here a variety of problems may arise which are difficult to detect and may be insoluble. First, the object of the inclinometer is to measure the displacements of the surrounding soil. If the casing is too stiff then soft soil will move around it, and soil deflections will not be correctly recorded. Conversely, if the casing is too weak it may be damaged during installation, or be seriously distorted by *in situ* earth pressures and movements. When using the relatively flexible plastic guide tube, Green (1973) reported 23° of twist in 24 m of guide tube. This can be overcome to a great extent by using aluminium guide tube. On the other hand, aluminium guide tube may become corroded, and is less suitable for long-term readings unless it is carefully protected using epoxy/bitumastic paint. Other problems can arise if dirt enters the guide tube and prevents the probe wheels from bearing on the casing, and if the probe is not precisely positioned at the same levels for each set of readings.

Relative movements are often required when investigating cracking and other signs of distress in structures. The Demec gauge is a well-tried device (Fig. 10.13) which can give reliable and valuable measurements of relative movement over a short gauge length. For example, the movement of a crack may be measured by placing a stud on each side of the crack using a standard length bar with two fixed points. The studs are normally glued with epoxy resin into small depressions in the surface of the structure which are chiselled out for that purpose. Once the studs are fixed and the glue is set, the two points of the Demec gauge are carefully placed into the studs and a zero reading is recorded. The Demec gauge is then placed on a standard Invar bar and a standard gauge length is recorded to check that the instrument is functioning properly. Readings taken over the crack at regular intervals will give the relative movement.

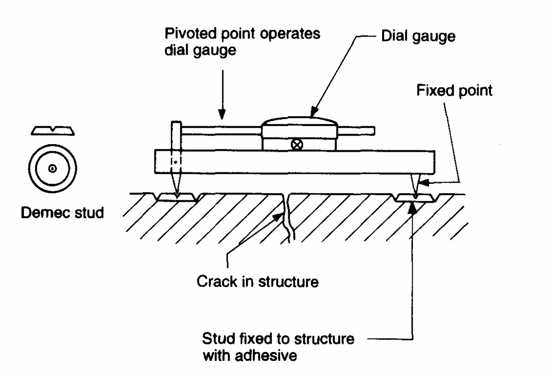


Fig. 10.13 Demec gauge.

Although the Demec gauge can give very good information it is important that a single careful operator is used on all gauge readings for a particular project. Interoperator errors can be significant.

OTHER MEASUREMENTS

At the beginning of this chapter it was noted that instrumentation exists to give data on a wide variety of parameters which have not been discussed above. These include total stress, shear stress, force and strain. Since these devices are but infrequently used in site investigations they will not be discussed. The reader is referred to Hanna (1985) and Dunnicliff with Green (1988) for further information.