Observational Approach and Instrumentation for Construction on Compressible Soils

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This paper describes the observational approach for construction on compressible soil. This approach consists of using observations and measurements to evaluate the performance of structures, both existing and under construction, for the purpose of deciding on corrective measures or improving design and construction of future structures. The paper contains a description of simple, practical instrumentation which provides the necessary quantitative observations. Observations and measurements, methods of recording, and typical interpretations are illustrated by case histories.

•THE OBSERVATIONAL approach or learn-as-you-go procedure, as it was often referred to by Terzaghi, consists of observing and measuring in the field the effects of successive construction steps on the surrounding soils and structures. The uncertainties usually involved in construction on compressible soils are often overcome by conservative design, but the added expense of being conservative does not always guarantee the success of the project. The observational approach often provides a more satisfactory answer.

The observational approach to engineering problems is possible because, in most instances, unsatisfactory performance or failure does not suddenly occur but is preceded by signs that can be recognized. By careful observation using adequate instrumentation and by competent interpretation of the results obtained, it is possible to predict the behavior of the structure being constructed, and the effect of construction on the adjacent structures. If necessary, changes may be made in the construction procedures or in the design of the structure, as construction proceeds, depending on the results of the observations.

Field measurements of full-scale structures also provide data which can be used directly in the design of other structures with similar soil conditions. The observational approach is, therefore, of great benefit when a series of structures is built in stages at the same location, making possible improvement in the design based on prior experience.

TYPE OF OBSERVATIONS

The most frequent type of observation is the measurement of horizontal and vertical movements. The movements occur in structures, at the ground surface, or at various depths in the subsoil. The observations may consist of measuring the displacement of reference points or of recording the formation and width of cracks. The stresses or forces in the soil mass such as those occurring beneath a footing or at the back of a retaining wall can also be measured, but such measurements are less frequent because this type of measurement is somewhat difficult.

The elevation of the water table and the porewater pressure in the soil are often important observations because variations in the position of the water table and variations of porewater pressure with time greatly affect the stability and settlement of structures on compressible soils.

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The field observations have one characteristic in common. To be usable in the observational approach, the successive values of the variable must be measured both as a function of time and as a function of the factors which may influence the variable. In this manner, the trend of the observations can be established and extrapolation becomes possible.

Observations can also consist of evaluating or measuring the engineering properties of subsurface materials during construction, at which time many measurements are possible at a reasonable cost. Although this type of observation is a part of the observational approach, the methods employed for determining the engineering properties of soils in situ are not discussed in this paper.

INSTRUMENTATION AND OBSERVATION PROGRAM

An instrumentation program consists of three phases: (a) installation of instrumentation, (b) observations and readings, and (c) interpretation. Because all the phases are equally important, every effort should be made when planning an instrumentation program to allot sufficient time and money for the satisfactory completion of the latter phases.

The instrumentation and observation program should be designed by an engineer having considerable experience in this field. The engineer should have sufficient authority so that he can carry out the field program with the cooperation of all the parties concerned. It is essential that the main objective of the instrumentation program be well defined and understood in advance.

The proportion of field instruments which become inoperative is usually high and it is advisable to use a greater number of instruments than would otherwise be necessary. Although initial readings are taken on all the instruments, regular readings may be limited to a selected number of instruments. The instrumentation should be installed at locations determined by a compromise between adequacy of data obtained, protection from construction activities, and ease of reading.

The recording of field readings should be facilitated by the use of adequate surveying equipment, convenient measuring scales, and clear and complete data sheets. The use of a mixture of units such as feet, inches, centimeters, and non-decimal fractions should not be permitted. The numbering system of the instruments should not be changed during the progress of the work.

Every effort should be made to obtain reliable zero readings, and there should be no doubt about the validity of calibration charts to which all subsequent readings are referred. If the instruments can be retrieved at the end of the program, the zero readings and calibrations should be checked. A field reading generally cannot be checked when an error is discovered at the time the results are interpreted. This lack of control should be compensated by taking the same measurement several times.

The results of the field observations should be assembled, as soon as they become available, in easily readable tables which exclude intermediate calculations. The data should be plotted to a carefully chosen scale to permit distinction between random variations and trends in measurements. It is often useful to compare a plot of the main variable vs time, with a temperature chart, river level chart, or other secondary variables plotted vs time.

INSTRUMENTATION

The field instrumentation should be as simple as possible because complicated instruments often become inoperative. Simplicity should be preferred to accuracy because in foundation engineering it is generally only necessary to obtain answers with an accuracy of ± 15 percent. The effects of climate and weather should be taken into account in choosing instrumentation, as perfect waterproofing and protection from freezing are very difficult to achieve. The instrumentation should be designed to provide readings which are not affected by temperature, because the fluctuation of outdoor temperatures is commonly rapid and erratic. Instruments should be designed for a well-defined purpose. Multipurpose instruments should not be used when the main purpose of the instrument is compromised.

The following paragraphs describe a list of instruments which fulfill the demands of most instrumentation programs. More detailed descriptions of the instruments are available in the technical literature.

Bench Marks and Reference Points

Permanent bench marks of known reliability are an important requirement when measuring settlements or heaves. In many instances special installations are required. At least two permanent bench marks should be provided as there is always the possibility that one may be destroyed. If the permanent bench marks are some distance from the site, temporary bench marks should be installed at more readily accessible locations. The relationship between the temporary bench marks and the permanent bench marks should be determined, from time to time, to assure the continued reliability of the temporary bench marks. Many types of bench marks are described in the references.

Reference points, ranging from bronze screws set in masonry to simple scratch marks, may be placed on structures. The type of reference point used depends on the type of measurement and the duration over which the measurements are to be carried out, a more elaborate and better protected reference point generally being required for measurements extending over a period of years.

Surface Settlement and Lateral Movement Rod

Many types of reference points may be installed to permit the measurement of settlement, heave or horizontal movement using ordinary surveying methods. The surface settlement and horizontal movement rod shown in Figure 1 has been found convenient for installation at the surface of embankments. The measurements can be made to an accuracy of 0.01 ft.

Foundation or Embankment Settlement Plate

The foundation or embankment settlement plate shown in Figure 2a provides a means of measuring the settlement at a point in a foundation or embankment as fill material is placed. A base plate is placed on the foundation or embankment at a specific elevation and, as the fill is constructed, steel pipe sections are progressively extended vertically. Elevations of the pipe sections are measured before and after each additional section is placed. Because it is necessary to add successive lengths of pipe and because of the necessity of successive surveys when adding the pipe sections, the accuracy of the settlement value is rarely greater than 0.1 ft.

If the lower end of the pipe is perforated it provides a means of measuring water levels in the embankment. If reference pipes of two different diameters are provided, so that one pipe can telescope over the other, the settlement of plates at two different levels in the embankment may be measured by one installation, as shown in Figure 2b.

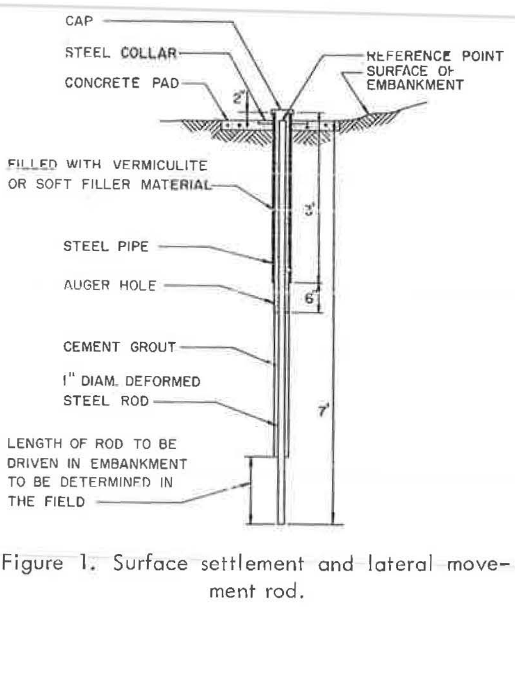


Figure 1. Surface settlement and lateral movement rod.

Water Level Settlement Gage

The foundation or embankment settlement plate described in the preceding section requires that pipes be extended vertically through the fill at the location of the plate, interfering with construction of the

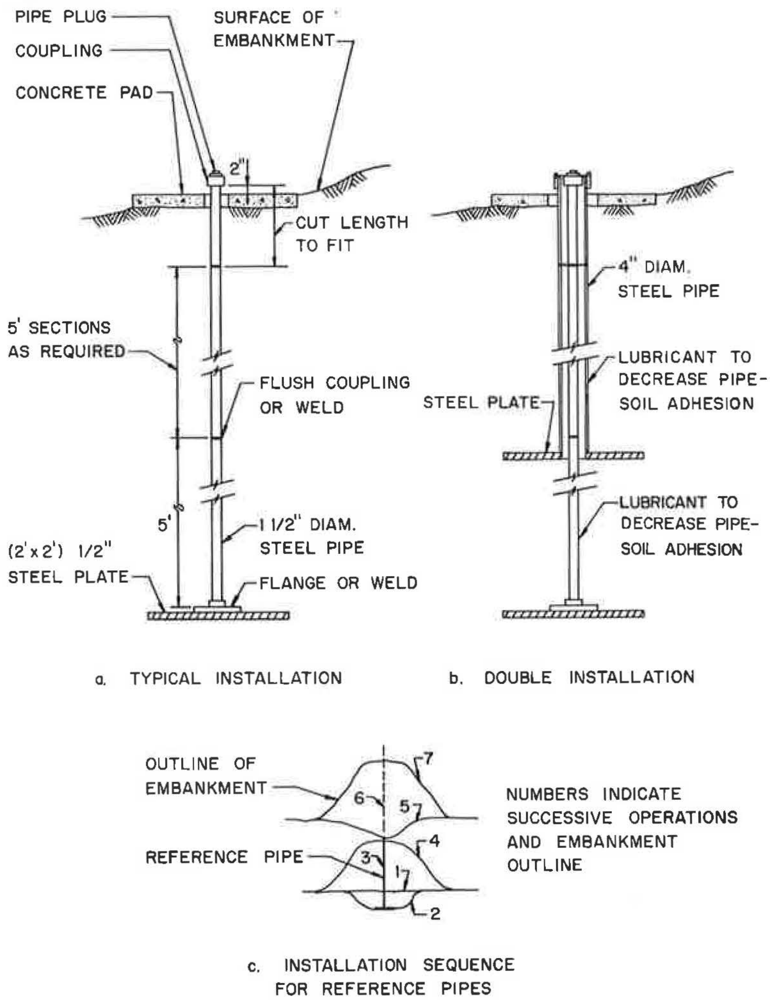


Figure 2. Foundation or embankment settlement plate.

fill. The water level gage shown in Figure 3 can be used to measure the settlement within an embankment with respect to a point located at the same elevation outside of the embankment. The device can also be used to measure the settlement of the back of a retaining wall as shown in the figure. The accuracy of the measurement is rarely greater than 0.05 ft.

Device for Measuring Heave of Bottom of Excavation

The heave of the bottom of an excavation can be measured by installing the heave point shown in Figure 4a within the soil mass prior to excavation. The heave point is attached to rods and pushed into the soil until it has reached the elevation at which heave is to be measured. The rods are then disconnected from the heave point and withdrawn. Alternatively, if the upper soil strata are too hard, the heave point is lowered to the bottom of a boring which penetrates the hard strata, and the heave point is pushed through the softer soils until it reaches the desired elevation as shown in Figure 4b. The elevation of the top of the point is recorded at the time of installation and again at the time the point is retrieved on completion of the excavation. The accuracy of the measurements is rarely greater than 0.05 ft.

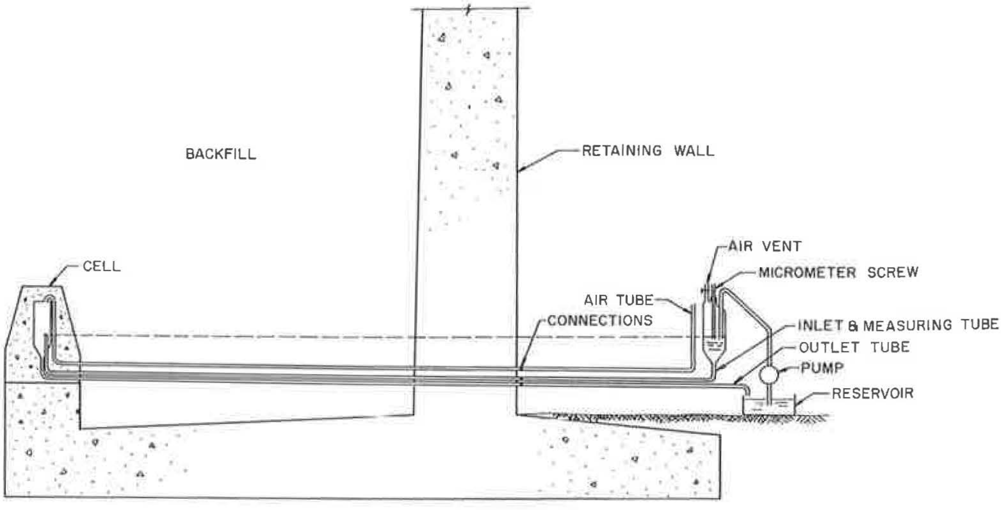


Figure 3. Water level settlement gage.

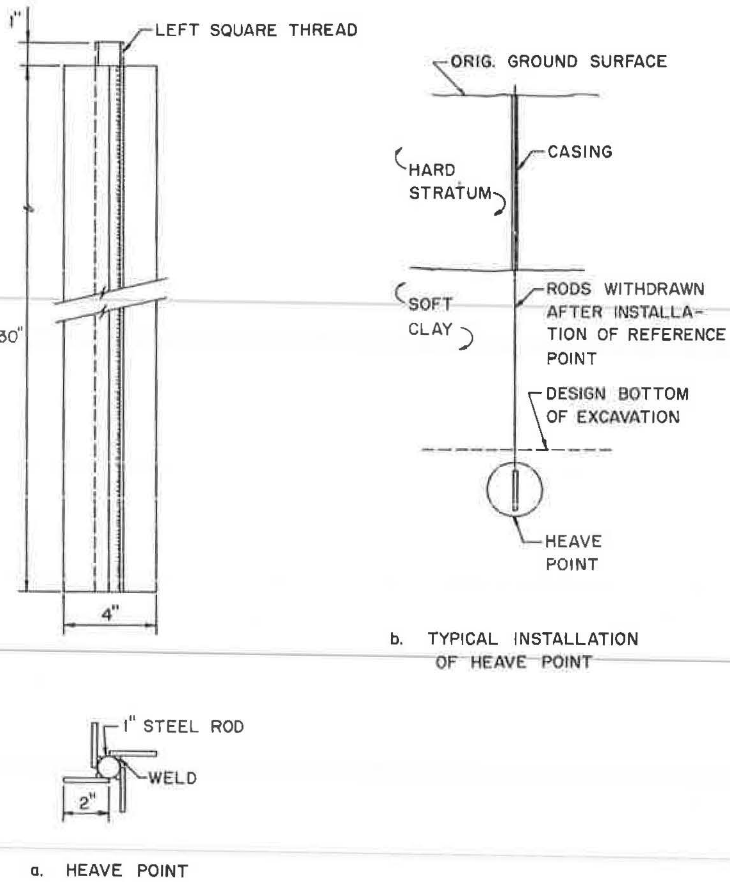


Figure 4. Device for measuring heave.

Wilson Slope Indicator

The Wilson slope indicator is an instrument for measuring deflections at depth. The instrument consists of a tiltmeter, a control box, an electric cable, and a 3-in. grooved aluminum casing which is installed in a 5-in. boring. The tiltmeter consists of a pendulum which, when activated by an electrical current, moves a conductor against a resistance and subdivides it into two resistances forming one-half of a Wheatstone bridge. The other half of the bridge is located in the control box. The pendulum is enclosed in a cylinder about 2. 5 in. OD and about 15 in. long. The tiltmeter cylinder is lowered into the aluminum casing by means of the electric cable and is kept in the constant azimuth of one set of grooves. Because the casing contains four grooves at right angles, the angle between the vertical and the casing can be measured in two rectangular directions. By integration of the readings, which are made at several depths, at different times, the successive deflections of the casing can be calculated. If the casing is fixed to a structural member, the bending moments in the member can be calculated from the deflection of the member, taking into account the structural properties of the member. The maximum angular deviation that the tiltmeter can measure is about 8 deg from the vertical. The accuracy of the readings is about 0. 001 radians which corresponds to about 1 in. in 100 ft. However, under favorable conditions displacements as small as 0. 1 in. may be measured.

Piezometers

In relatively pervious material with a coefficient of permeability $k>10^{-3}$ cm/sec, a wellpoint piezometer of the type shown in Figure 5a may be employed. A typical installation of such a piezometer is shown in Figure 5c.

In fine-grained soil with $k<10^{-3}$ cm/sec, a porous cylinder is used in place of the wellpoint. This cylinder may consist of plastic which is or is not surrounded by a sand filter. The porous cylinder has a lead consisting of plastic tubing extending to the ground surface. A typical plastic porous cylinder piezometer is shown in Figure 5b.

The water level in piezometers may be determined by means of an electrical probe. This probe basically consists of a thin double-conductor electric cable. The conductors

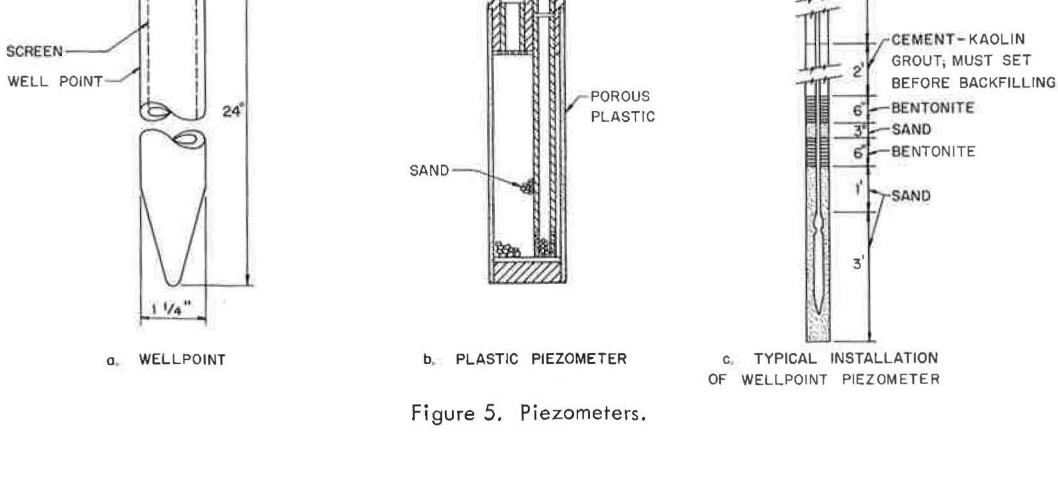


Figure 5. Piezometers.

at one end of the cable are connected to an ohmmeter and battery; the other end of the cable is introduced into the piezometer tube. When the conductor at the lower end of the cable comes in contact with the water, a sharp variation in resistance is indicated on the ohmmeter. If the piezometric level is higher than the top of the lead tube it may be measured by a Bourdon gage.

Because it is not possible to push an electric cable into the lead tube of a piezometer over great horizontal distances, another type of reading device may be used. The outlet of a small diameter air tube is connected to the lead tube at a predetermined elevation which must be below the lowest expected piezometric level. The vertical distance between the outlet of the air tube and the piezometric level is calculated from the air pressure which must be applied to the inlet of the air tube in order to produce a small air flow. The inlet of the air tube can be located in any convenient place outside the construction area. A piezometer equipped with an air tube is referred to as an air-activated or bubbler piezometer, which is described in the references.

Strain Gages

Several types of strain gages can be used to measure the strain occurring in a structural member under load. Some of the instruments available include the Whittemore mechanical gage, the Baldwin SR-4 electrical gage, and the vibrating wire gage.

The Whittemore gage consists of a frame formed of two parallel bars connected near their ends by spring fulcrum plates which prevent motion of one bar relative to the other except in a longitudinal direction. A steel point is attached to one side of each bar. The steel points are inserted in small holes drilled in the member. The gage is 10 in. long and the relative motion of the bars is measured by a dial micrometer graduated to read 10^{-4} in.; thus, unit strains as low as 10^{-5} can be measured with the gage. Although the Whittemore gage is accurate enough for foundation engineering, a very meticulous operator is required and readings are time consuming.

Baldwin SR-4 strain gages operate on the principle that the electrical resistance of wires attached to a structural member is related to the change in length of the wires as a result of strain in the member. These gages must be waterproofed and extreme precautions must be taken to keep the resistance of the electrical connections constant. Baldwin SR-4 strain gages can be incorporated in a load cell as shown in Figure 6. This type of load cell has been successfully employed for a period of nearly a year as outlined in the case history given later.

Vibrating wire strain gages typically consist of a steel tube with circular steel plates attached to each end of the tube. An axial pretensioned wire is stretched between the center of the plates, and an electromagnet within the instrument plucks the wires and picks up its vibrations. Strains, due to the load to be measured, are transmitted to the tube and to the wire altering the frequency of the wire. The gage is read by tuning a similar wire in a receiver to the same frequency as the wire in the instrument by observing traces on a cathode ray tube. The vibrating wire gage has the advantage that changes in the properties of the electrical circuit do not alter the frequency of vibration of the wire. However, the gage is not yet commonly employed.

Earth Pressure Cells

The earth pressure at the interface of a structure and soil, and the earth pressure within a soil mass, may be measured with earth pressure cells. However, such measurements are difficult and are usually limited to special projects.

The Carlson stress meter, which is described in the references, is solid and sufficiently sensitive and reliable provided it is properly installed. However, in common with electrical gages, difficulties often arise in measurement due to changes in the resistance of electrical connections. Vibrating wire pressure cells which eliminate this problem, are to be preferred but are expensive.

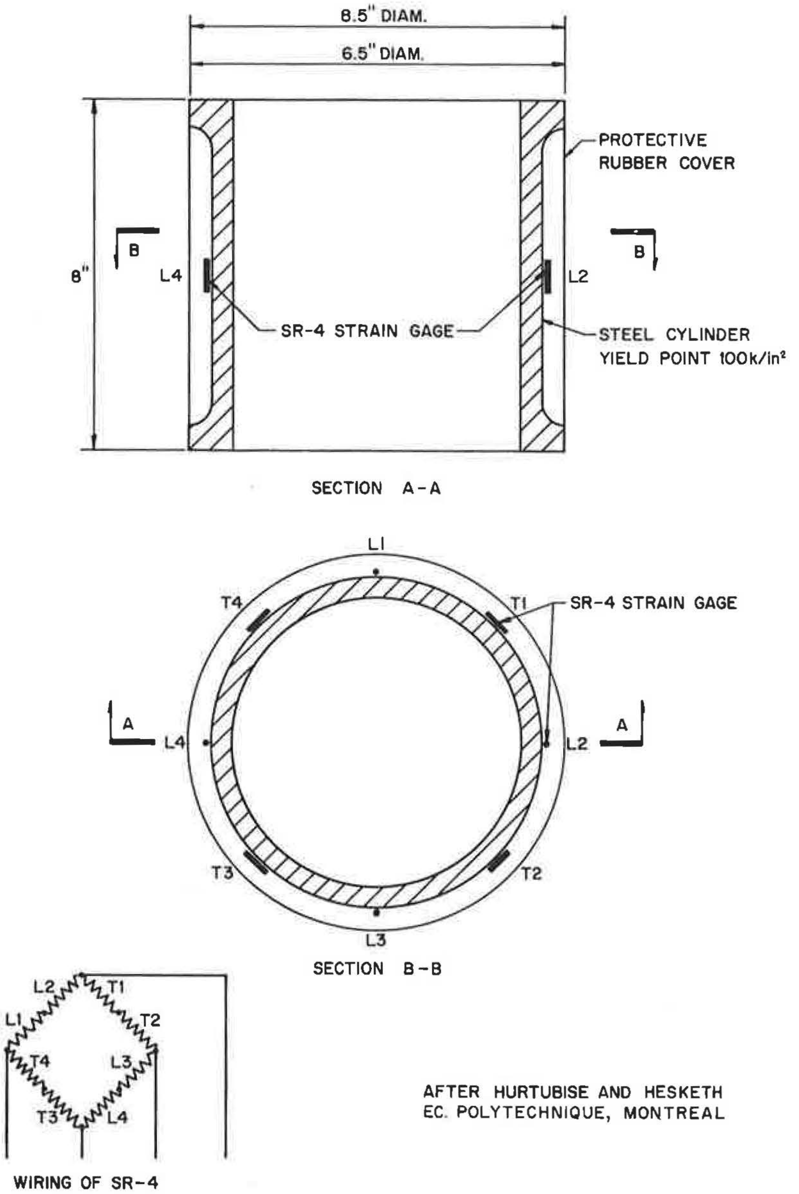


Figure 6. Load cell.

CASE HISTORIES

Observation of an Anchored Wall in Montreal

This case history describes the observational techniques and instrumentation employed in the construction of the side walls of a deep excavation for the Berri-DeMontigny subway in Montreal.

The excavation through strata of sand, silt, and very dense till was extended to a depth of about 60 ft into shale (Fig. 7). The excavation was supported by a wall consisting of 30-in. diameter reinforced concrete drilled piers at 10-ft centers with a gunite membrane between the piers. The piers were tied back with steel cables anchored into the shale outside the excavated area. The excavation was close to streets, utilities, and multistory buildings founded on shallow spread footings.

The instrumentation consisted of reference points on the surrounding buildings and on the top of the piers (Detail A, Fig. 7) and of load cells in selected tie-backs. The load cells, which incorporated Baldwin SR-4 strain gages (Fig. 6), were designed for a maximum load of 400 kips. Calibration readings taken before and after use indicated that the drift in the zero readings was no greater than five kips.

Because of the large size of the excavation and the number of reference points, it was impractical to survey all of them at frequent intervals. However, all the reference points were surveyed two or three times after they were installed to obtain a good set of initial readings. Then, regular observations were limited to selected areas, the frequency of the readings at any one location being dependent on the trend indicated by

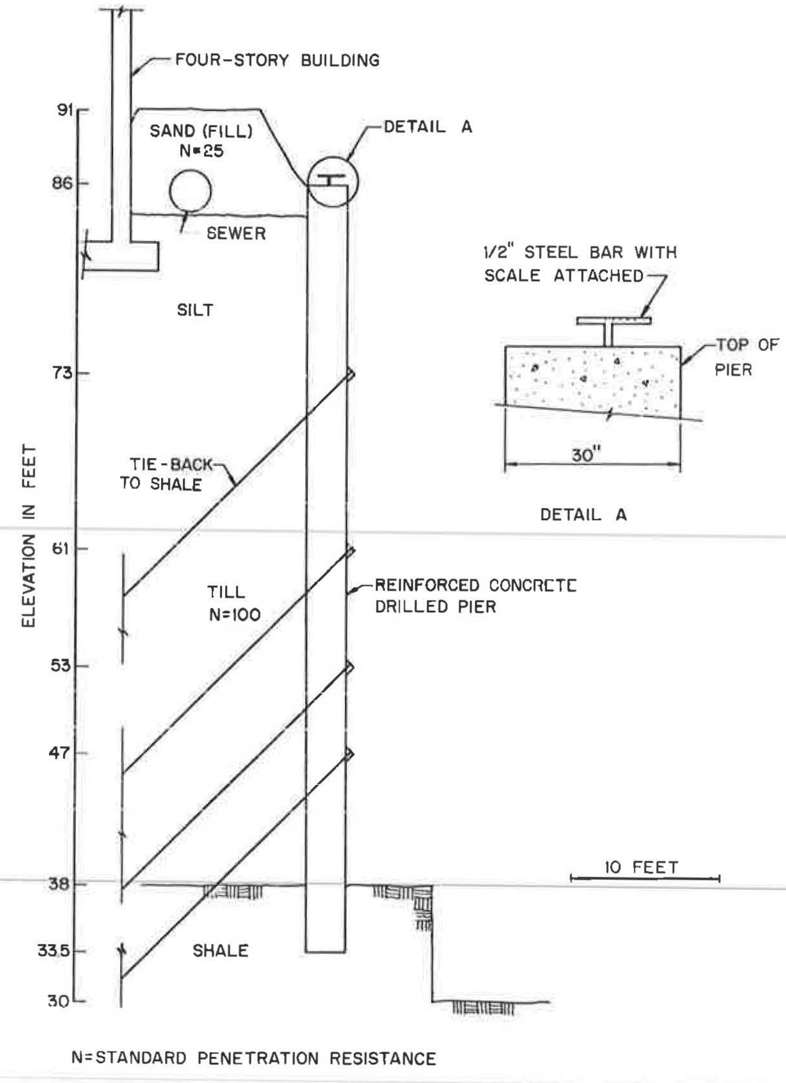
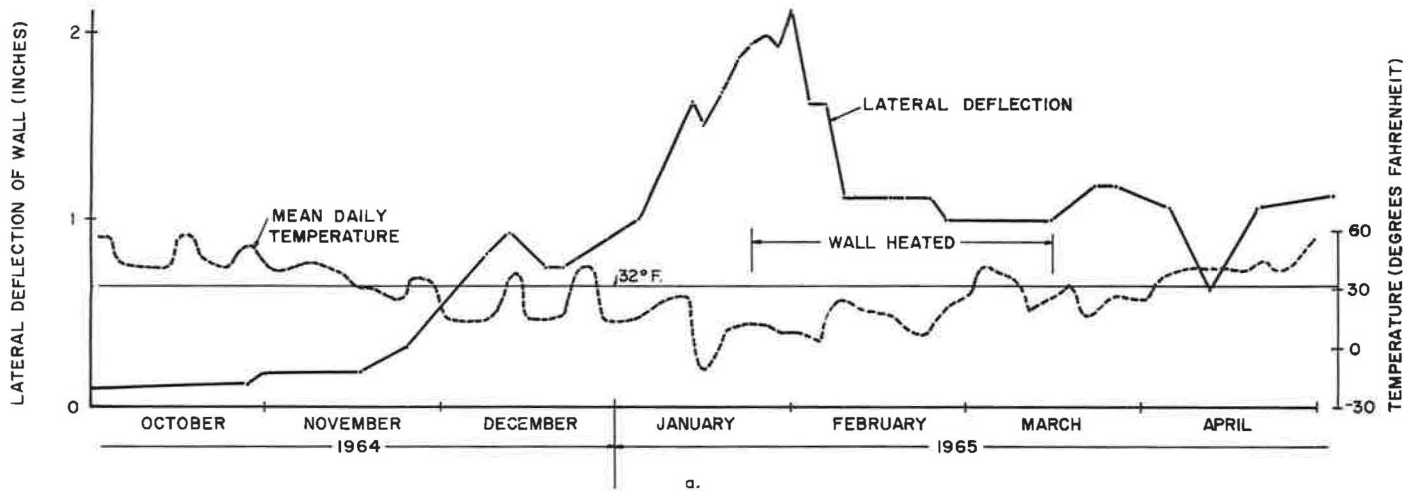
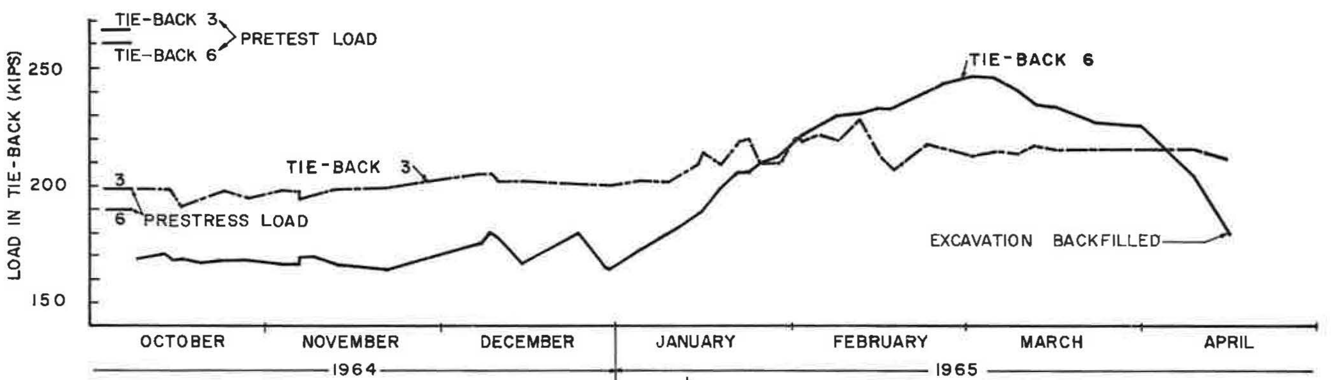


Figure 7. Berri-DeMontigny subway station, Montreal: typical section of anchored wall.



a.



b.

Figure 8. Berri-DeMontigny subway station, Montreal: observations on anchored wall.

previous measurements at the location. If abnormal movement had occurred in areas not surveyed regularly, the availability of the reference points and the initial readings would have permitted a measurement of the total movement.

When the tie-backs were installed they were pretested to a load at least equal to the design load. The load was then released to a prestress load equal to about 75 percent of the design load. As the excavation was continued the load in the tie-backs increased but not to the extent anticipated in the original design, and it was possible to reduce the number of tie-backs at some locations by as much as 30 percent.

The results shown in Figure 8 are typical of the observations which were made on the wall (Fig. 7) during the winter of 1964-1965, when the depth of the excavation did not change. The deflection of the wall in the direction of the excavation was not significant until the middle of November 1964 when freezing temperatures first began to occur (Fig. 8a). In early December, overflow and leakage from a sewer saturated the silt behind the wall. The load in the tie-backs and the deflection of the wall increased sharply, probably due to freezing of the material behind the wall. Although the wall moved back slightly during a period of mild temperatures in December, the deflection continually increased during prolonged freezing temperatures. By the middle of January the total deflection was of the order of 2 in. Further movement of the wall was considered undesirable because settlement of the adjacent building was likely to occur. The lateral movement was attributed to freezing of the material behind the wall, and heating of the wall was adopted as a remedial measure.

An enclosure consisting of a timber framework and a plastic covering was installed around the wall adjacent to the building. The temperature within the enclosure was maintained at about 36 F. The wall moved back soon after heating began and the total deflection remained at about 1 in.

The loads in two typical tie-backs during this period are shown in Figure 8b. The load in tie-back 3, located within the heated portion of the wall, did not increase any further once heating was begun. The load measured in tie-back 6, in an adjacent unheated wall, increased during the entire cold period and did not decrease until the weather became mild in the spring.

At this site, the observations indicated that (a) the design assumptions as to the magnitude of the earth pressure were conservative—several tie-backs could be safely omitted; and (b) prolonged freezing temperatures caused an increase of the load in the tie-backs and of the deflection of the wall. When the maximum permissible deflection of the wall had been reached, heating of the wall was effective in preventing further deflection.

Observation of a Braced Excavation in Chicago

This case history describes the observational techniques and instrumentation employed by the Chicago Park District in the construction of a braced excavation for an underground garage.

A sheet pile wall was driven around the periphery of the site through sand fill and soft clay to a layer of stiff clay (Fig. 10b). The foundation of the garage consisted of a slab constructed on the soft clay; such a solution proved feasible as the weight of the garage was less than the weight of the excavated material.

Because of the proximity of major buildings, including the Chicago Orchestra Hall and the Chicago Art Institute, the construction of the garage was to be undertaken in such a manner that the movements of the surrounding area would be minimal. Provisions were made to obtain readings of deflection of the sheet pile wall using the Wilson slope indicator during the construction period. The instrumentation also included reference points on adjacent buildings and on the sheet pile wall. Heave measuring devices were installed at the foundation level of the proposed garage.

The construction of the garage was done in stages as shown in Figure 9. The successive deflections of the sheet pile wall at one typical location, measured using the Wilson slope indicator, are shown in Figure 10a. These deflections have been plotted assuming that the bottom of the wall did not move. From October 24, 1963, when the slope indicator was installed, to March 26, 1964, when excavation was started in this

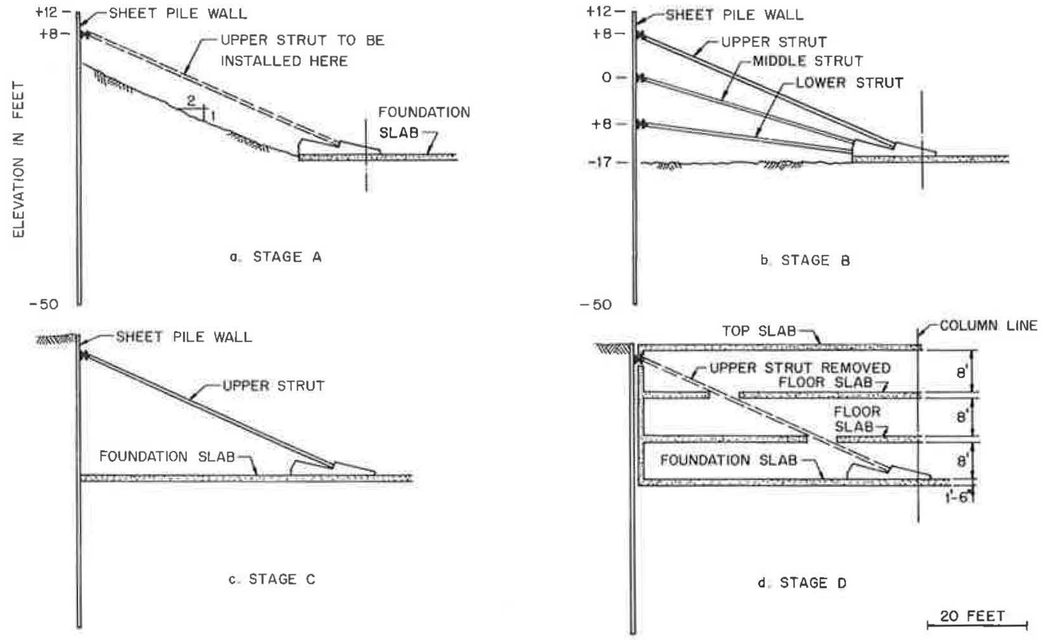


Figure 9. Underground garage, Chicago: construction stages.

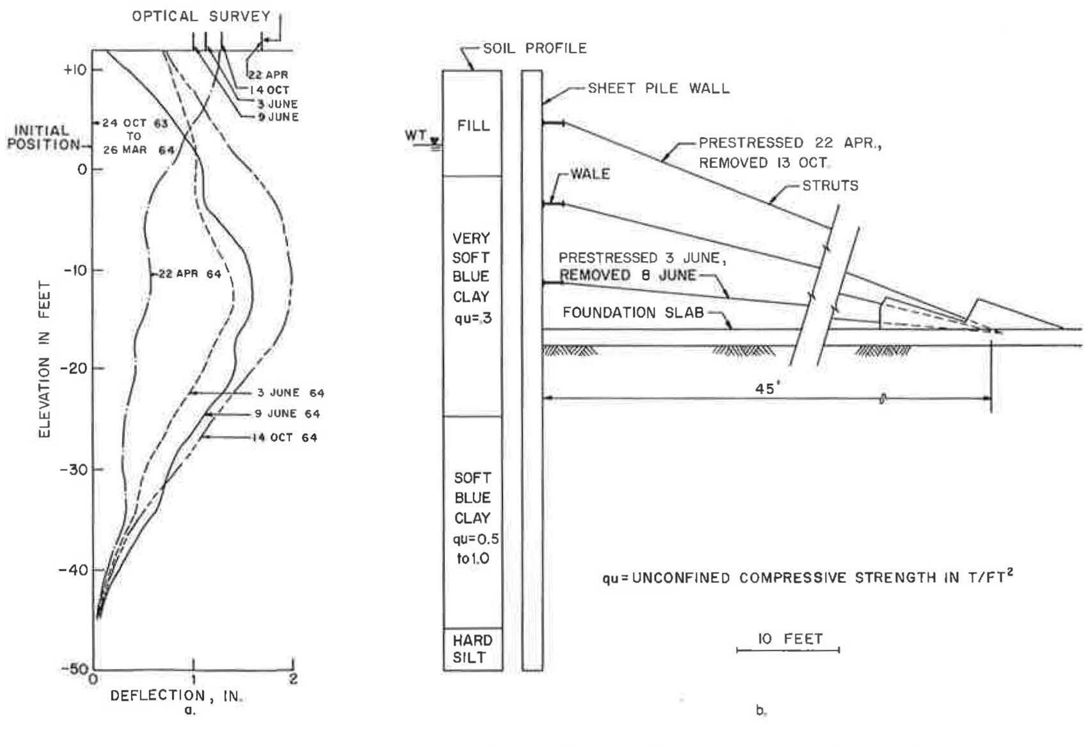


Figure 10. Underground garage, Chicago: soil conditions and observations on braced sheet pile wall.

area, the sheet pile wall remained vertical, as shown by the slope indicator readings. Initially, only the material in the center of the garage was excavated and a wedge of soil was allowed to remain against the sheet pile wall. The foundation slab in the center of the garage was constructed and the upper strut installed and prestressed (Stage A in Fig. 9a). Just before the upper strut was prestressed the wall had deflected as shown by the curve for April 22, 1964, in Figure 10a. The soil against the sheet pile wall was excavated in sections and the middle and lower struts were installed and prestressed (Stage B, Fig. 9b). At this stage, the deflections of the wall were as shown by the curve for June 3, 1964, (Fig. 10a). The foundation slab was extended to and poured in contact with the sheet pile wall. As soon as the strength of the foundation slab was sufficient to provide an effective support for the wall, the lower and middle struts were removed, which greatly facilitated construction (Stage C, Fig. 9c). After removal of the lower strut and just prior to the removal of the middle strut, the deflections were as shown by the curve June 9, 1964, in Figure 10a. The floors of the garage were constructed leaving openings for the upper struts and these struts were removed after the top slab was installed (Stage D, Fig. 9d). The deflections of the wall at this stage were as shown by the curve for October 14, 1964, in Figure 10a. Subsequent observations of the wall showed that the deflection remained constant.

The result of the optical survey shown at the top of Figure 10a confirms the trend of movement of the sheet pile wall during the various construction phases. However, the optical survey data show greater movement of the top of the wall than the slope indicator readings. Better agreement can be obtained between the optical survey and the slope indicator readings by assuming that the bottom of the sheet pile wall moved about ½ in. between March 26 and April 22, 1964.

At this site, the observations confirmed the predicted behavior of the wall and established that the deflections were minor and would not affect the adjacent buildings. The slope indicator readings showed that deflection of the sheet pile wall extended below the bottom of the excavation at all construction stages. Although such deflections are unavoidable, they are at a minimum when the struts are placed at a close vertical spacing. The removal of the intermediate struts after pouring the foundation slab caused only small deflections which indicate arching of the soil between the upper strut and the foundation slab. The heave of the foundation was small, on the order of ½ in. Therefore, recompression of the clay due to loading from the garage was small.

Observation of Groundwater Table at an Industrial Site

This case history illustrates the use of instrumentation in determining the groundwater table at an industrial site.

The site was located on a river terrace and the soils consisted of alluvial deposits. The soil stratification was irregular, the materials consisting of sand and gravel with variable amounts of silt. Most of the foundations consisted of spread footings constructed at a wide range of depths below the original ground surface, depending on grading requirements in various areas of the site. It was also desirable to establish the footings above the water table to avoid disturbance of the granular materials and to facilitate construction. Instrumentation was therefore installed to determine the groundwater table.

The instrumentation consisted of piezometers installed in selected exploratory borings. The tips of the piezometers were placed at varying depths and were surrounded by a sand filter. The upper portion of each boring was filled with cement grout to prevent ingress of runoff water. The water table as shown by the water level in the piezometers was observed at frequent intervals.

The position of the water table as measured in the piezometers was confirmed when excavations were made for the footings, with the exception of one particular area where the water table was higher than anticipated. The soil stratification in this area consisted of loose silty sand from the ground surface to a depth of 10 ft, medium-dense gravelly silty sand from 10 to 18 ft, and very dense sand and gravel below 18 ft, (Fig. 11). The tip of the piezometer was located in the dense sand and gravel at a depth of 30 ft and the water level measured in the piezometers was at a depth of 15 ft. However,

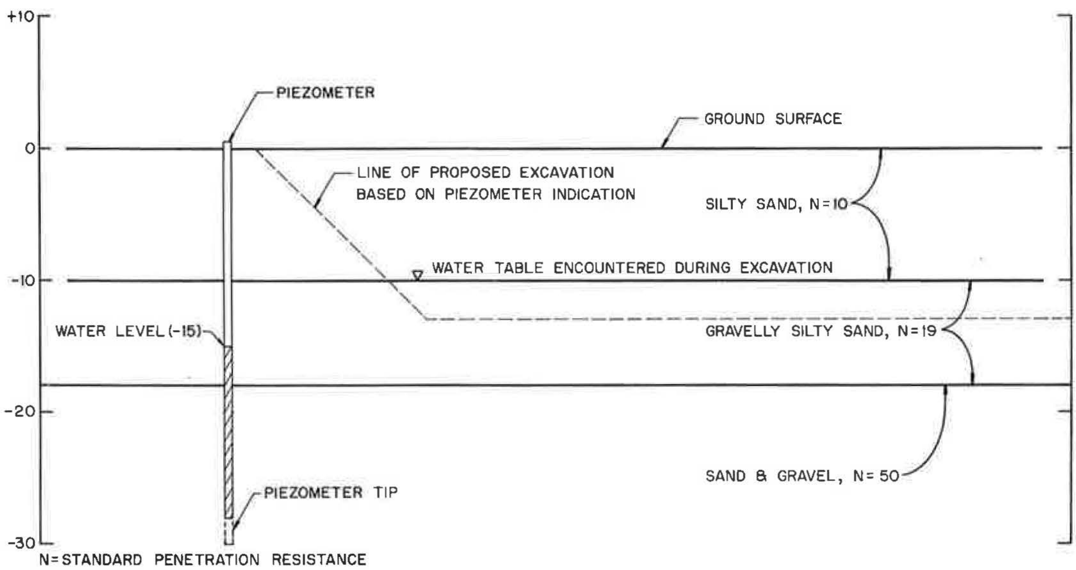


Figure 11. Industrial site: soil conditions and water table observations.

on excavation the water table was encountered 5 ft higher than anticipated on the basis of the piezometer readings. The relevance of the piezometer readings was therefore re-examined.

The relatively impervious nature of the gravelly silty sand stratum had not been recognized, in that this stratum was impervious enough to permit the development of a perched water table at a depth of 10 ft. This perched water table was not disclosed by the piezometer because the tip of the piezometer was located in the underlying pervious sand and gravel. The water table as implied by measurements of water levels in piezometers must, therefore, be critically interpreted in conjunction with the soil profile.

CONCLUSIONS

Because it is frequently impossible to predict the behavior of compressible soils in advance, engineers and contractors must often resort to radical and expensive solutions or accept an unknown risk. The observational approach reduces the number of cases in which such alternatives are necessary because it is possible to learn as you go on the basis of field observations. The observational approach also provides useful data for the design of new structures in similar soil conditions.

The observational approach requires adequate instrumentation and competent recording and interpretation of the results. Good organization of the field program is a prerequisite and cooperation among the owner, engineer, and contractor is essential.

The observational approach is widely accepted on projects such as large earth dams. There is no reason why it should not be applied to smaller projects.

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