INSTRUMENTATION TO MONITOR BUILDING DAMAGE FROM EXCAVATION INDUCED GROUND MOVEMENT

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ABSTRACT

Geotechnical engineers have long relied on instrumentation to monitor the performance of soils and structures both during and after construction. The diverse technology available for taking these measurements is manifested in a range of commercially available products that have been proven reliable in the field. The most difficult decision for the field engineer is often exactly which instruments are best suited for a given application. This thesis gives a summary of the theory behind structural damage due to ground movements, and an overview of the technologies available for measuring deformation and damage. Some consideration is given to each instrument's precision and the advantages and limitations of their use, and interesting uses of these technologies are noted. Finally, a comprehensive instrumentation scheme for the long-term monitoring of a high-performance academic building is proposed. This system is geared toward total and differential settlement measurement, and can be easily incorporated into the overall "smart systems" that are proposed for the self-monitoring of the conceptual structure. The proposed scheme is one method of preparing a building for performance monitoring during subsequent adjacent construction activities.

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To the MEng class of 1998 – thanks everyone. For the weekend in Vermont and every week following that. It was amazing to work with such a concentration of talent, and I'm sure I will never by so lucky to have such an experience again. Of course I know that I'll never be going for chickens again – and I'll miss that. Most of all, thanks for taking seriously the ideas of a senile OLD MAN. I'll think of you all often after leaving here – so long as my memory serves me...

Mom and Dad, thank you for all of your support over the years. Without your help, I would have never made it to the place I am today. How does one express such thanks? I love you.

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1 INTRODUCTION

Long recognized as a major contributor to structural damage, ground movements have many causal factors. Ground movements can be categorized into those that occur due to or during construction, and those that happen after construction, also referred to as long-term ground movements.

1.1 Construction Related Movements

Many different construction activities can induce ground movements in the surrounding soil. Excavations and tunneling, in particular can cause movements, even when they are occurring far beneath the ground. Profiles of ground movement due to tunneling have been reported in the literature by Peck (1969), Attewell (1977), and others, and the magnitude and effect of these movements have been examined by New and O'Reilly (1991).

Other construction activities can also induce ground movements. In particular, construction dewatering activities intended to create a dry working area can cause ground subsidence. This is due to an increase in effective stresses in the soil that causes consolidation settlement. Hutchinson (1964) reports such settlements for an excavation on Oslo, Norway. In addition to changes in effective stresses, complications in dewatering can cause ground movements. During the construction of the Sears Tower, pumping activities removed some sand and silt with the water, resulting in ground movements up to 25 cm surrounding the excavation (O'Rourke, 1981).

Another factor causing settlement in a soil mass is increased overburden due to construction. Building dead loads cause initial settlement in soils provided that the additional loading from construction is greater than the weight of the soil excavated during construction. Ueshita, et al., (1975) document the heave and settlement caused by construction for a fifteen-story building founded on clay and sand in Nagoya, Japan. A maximum vertical displacement (heave) of 22 mm in the soil was recorded at the end of excavation, prior to placement of the structure. During construction, the building settled 28 mm, for a net settlement of 6 mm.

1.2 Long Term Movements

Heavy structures often cause long term consolidation settlements in their underlying soils. This is due to a dissipation of excess pore pressures in low permeability soils over time that results in a net increase in effective stresses in the soil. Horn and Lambe (1964) document structures on the MIT campus that have shown long term settlements up to 8 inches. Andersen and Clausen (1975) show the settlement history of a large storage building in Oslo, Norway. The six-story structure, founded on a normally consolidated clay deposit, has settled up to 70 cm over 50 years.

Other long term ground movements can be caused by a regional lowering of the groundwater table due to pumping. For example, in Mexico City, subsidence greater than 5 m has occurred due to pumping for municipal water supplies (Zeevaert, 1972).

Alternatively, soil heave can result from an increase in the elevation of the water table. Cheney and Burford (1974) document structural damage to office buildings caused by the removal of trees. Trees can create high suction pressures in soil and hence draw down the water table. When they are removed, the water table can rise, causing soil heave.

1.3 Scope

The focus of this thesis is ground movements that are caused by or associated with excavations. Excavations induce movements in the surrounding soil that may be particularly troublesome in an urban environment, where the construction site often abuts high value structures. Careful design of lateral earth support systems (wall and bracing) and restrictions on groundwater pumping are critical in controlling both horizontal and vertical soil movements for a given project (Clough & O'Rourke, 1990).

One major problem in predicting and controlling ground movements due to excavations is uncertainties in site characterization concerning both the stratigraphy and engineering properties of pertinent soil and rock strata. The latter are usually estimated from combinations of laboratory and in situ measurements.

Given the uncertainties in site characterizations, engineers are always in the situation where prediction and/or control of ground movements is difficult, at best. Therefore, engineers seek construction methods and monitoring data to minimize risk (after Casagrande, 1965) and also to help refine designs by the observational method (Peck, 1969). This thesis reviews the equipment that is commonly used to monitor excavation performance.

The proper selection of in situ monitoring equipment is not a straightforward task. There are many technologies available to measure key parameters such as ground deformations, pore pressures and lateral earth pressures, among which there are numerous designs for each category of instrument (such as piezometers). Each construction project is likely to require its own unique instrumentation scheme depending on the scope of the project, surface and subsurface conditions, the nature adjacent facilities and the owner's aversion to risk.

It is the intent of this project to examine some of the more common technologies used to monitor surface and subsurface deformations and stresses. The design and use of different types of instrumentation will be examined, and some consideration will be given to the capabilities and limitations of these devices. Examples are quoted for recently published or interesting project implementations of these devices, and issues relating to data collection are discussed.

This document is organized as follows. Chapter 2 presents a background on available empirical and theoretical studies on building damage due to ground movements. The chapter follows a chronology of the methods proposed to measure and predict potential damage. Chapter 3 compares the different types of instrumentation that are currently available. Chapter 4 proposes an instrumentation scheme for construction of a new Civil and Environmental Engineering building on the MIT campus.

2 BACKGROUND

The most comprehensive studies linking self-weight settlements of buildings to structural damage were carried out in the 1950's by Skempton and MacDonald (1956) and Polshin and Tokar (1957). These studies show that damage is most often caused by differential settlements rather than absolute settlements. More recently, similar empirical studies by Boscardin and Cording (1989) and Boone (1996) have linked structural damage to ground movements induced by excavations and tunneling activities.. This chapter reviews those empirical findings and summarizes simple theoretical models (after Burland and Wroth, 1975) that have been proposed for predicting the occurrence of structural damage.

2.1 Building Damage Due to Self-Weight Settlements

Skempton and MacDonald (1956), in one of the earliest studies of its kind, examined data on 98 buildings, both load bearing wall structures and steel or concrete frame buildings. Of these buildings, 40 were damaged by settlement and 58 were not. Skempton and MacDonald calculated values of greatest differential settlement, Δ, and angular distortion, δ/l for these buildings (see Figure 2.1) and developed limiting values of δ/l for different structural systems. The differential settlement is defined as the greatest vertical distance between two points on the foundation of a structure that has settled, while the angular distortion, δ/l, is the difference in elevation between two points, divided by the distance between those points.

Data from Skempton and MacDonald's work suggest that the limiting value of angular distortion is 1/300. Angular distortion, greater than 1/300 produced visible cracking in the majority of buildings studied, regardless of whether it was a load bearing or a frame structure (see Figure 2.4).

Other key findings by Skempton and MacDonald include limiting values of δ/l for structures, and a relationship between maximum settlement, ρ_{max} and δ/l for structures founded on sands and clays. Charts showing these relations for raft foundations and isolated footings are shown in Figure 2.5.

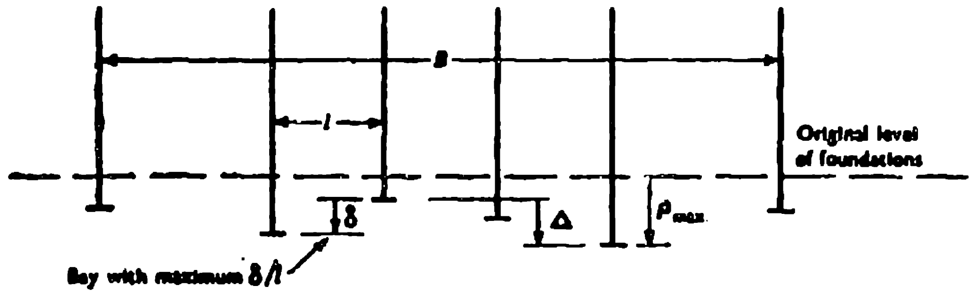
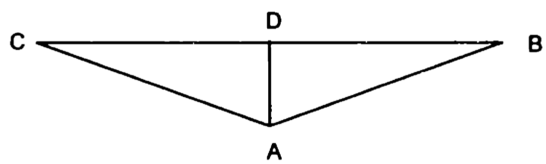


Figure 2.1 – Diagram illustrating the definitions of maximum angular distortion, δ/l, maximum settlement, ρ_{max}, and greatest differential settlement, Δ, for a building with no tilt (Skempton and MacDonald, 1956).

Grant, Christian and Vanmarcke (1974) revisited and evaluated the work of Skempton and MacDonald to include consideration for the value of maximum curvature*, X, illustrated in Figure 2.2. This factor was suggested by Horn and Lambe (1964) as another potential indicator of damage. X is a measure of the maximum settlement between two reference points divided by the distance between the points, later termed *relative deflection* (e.g., Burland and Wroth, 1974).



Maximum curvature, X = AD/CB

Figure 2.2 – Maximum curvature, X, as defined by Grant, et al. (1974).

Grant, et al. (1974) added another 95 structures to the database developed by Skempton and MacDonald, and included such variables as settlement time. The authors found that a lack of data prevented calculation of X for many structures, and thus no new correlation could be developed for damage as a function of this factor. Rate of settlement, on the other hand, was

* Grant, et al. (1974) chose the term Δ to represent maximum curvature. However, to avoid confusing this Δ with Skempton and MacDonald's *differential settlement* Δ, the symbol X was chosen to represent maximum curvature in this paper.

found to be a factor in structural damage. In addition, the authors explored relations between maximum differential settlement, maximum net slope (δ/L), and maximum settlement for different foundation schemes and different subsurface soil conditions. δ/L in this case is the net slope of the line tangent to the settlement profile, illustrated in Figure 2.3.

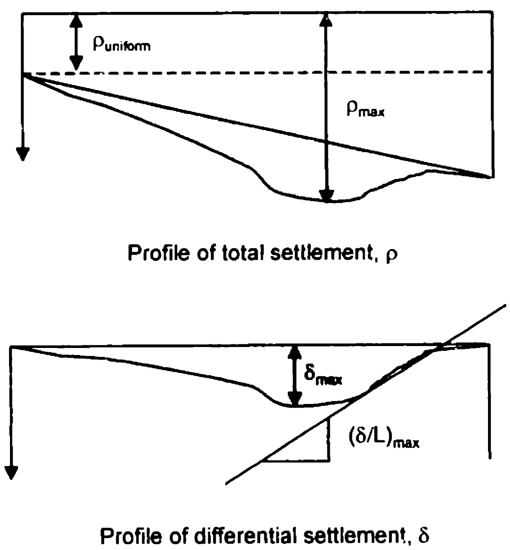


Figure 2.3 – Definition of δ/L and settlement according to Grant, et al. (1974).

Thus, important conclusions from Grant, et al., were:

1. δ/L calculations provide limiting settlement values that are equally useful compared to the X calculation. Furthermore, maximum curvature (X) calculations are more data intensive because they require the knowledge of the exact location of damage within a structure.

2. For rafts, rigidity of the foundation is an important factor in the determination of limiting net slope, δ/L.

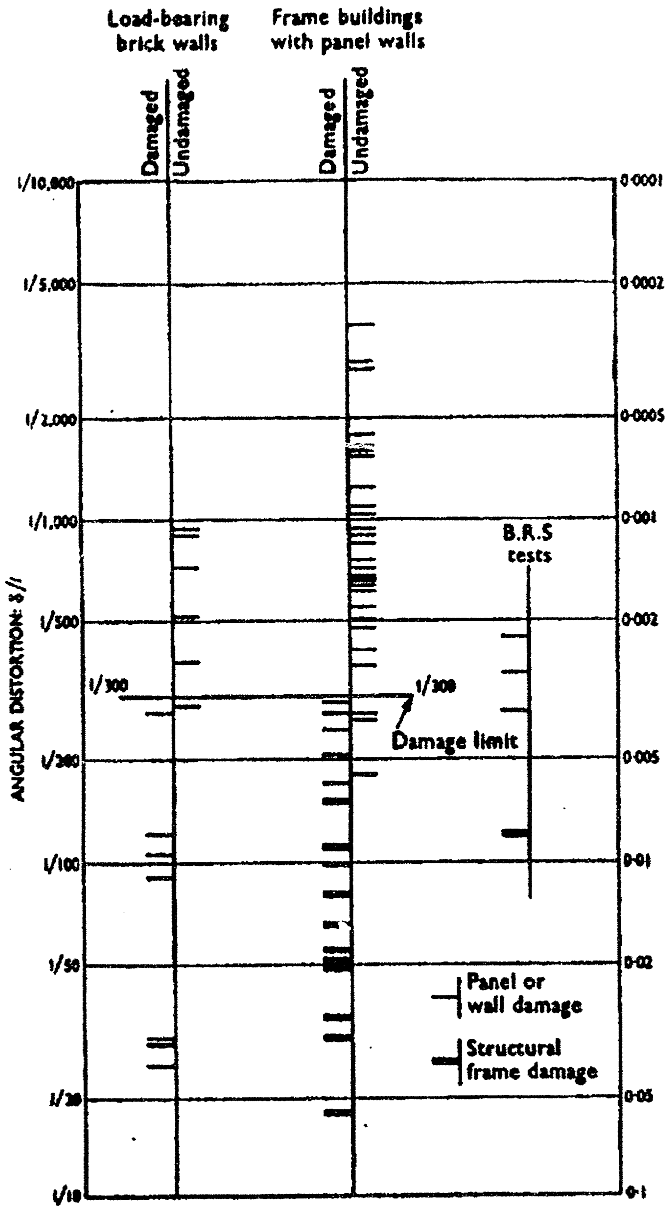


Figure 2.4 – Field evidence of damage related to angular distortion (Skempton and MacDonald, 1956).

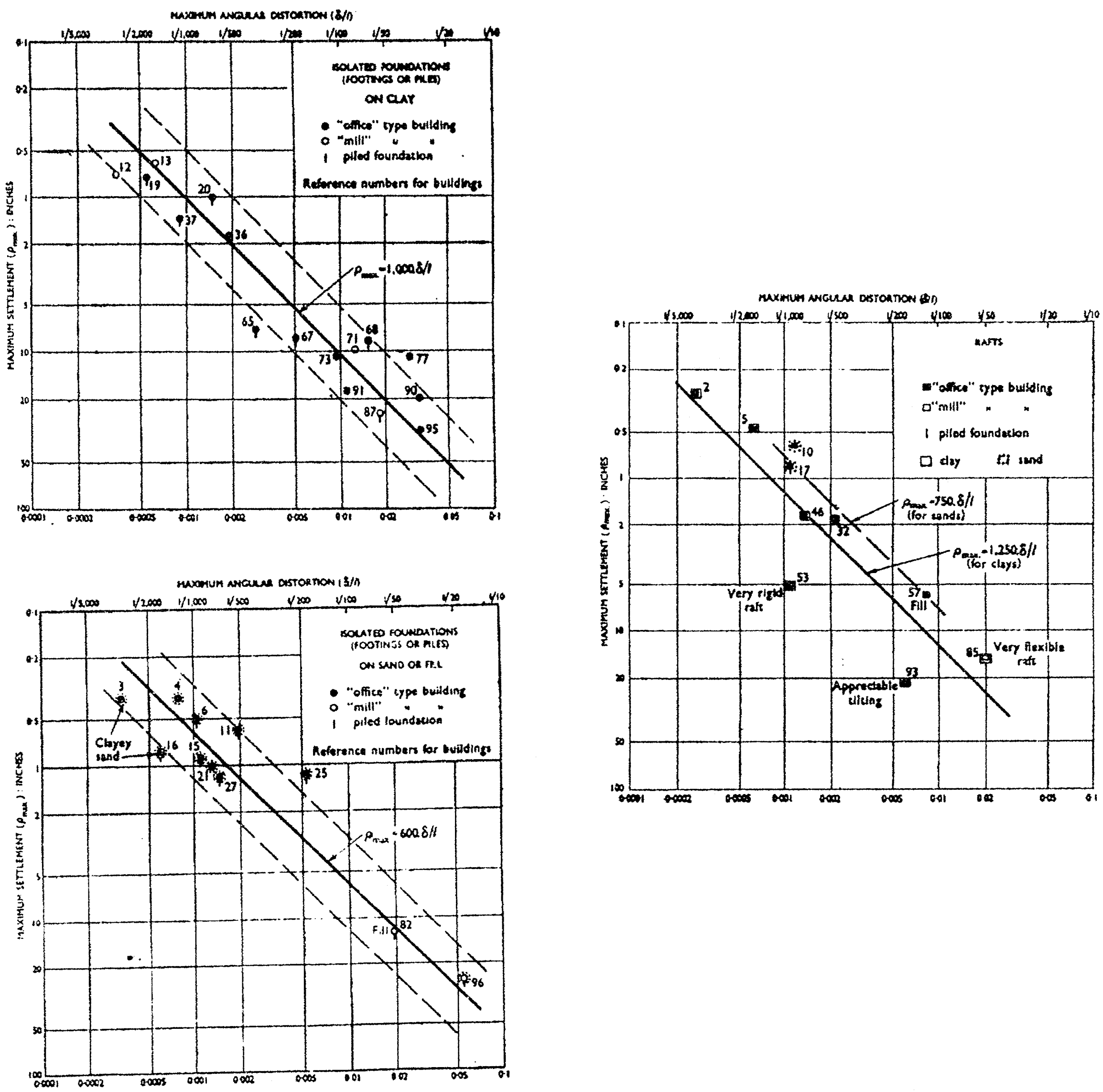


Figure 2.5 – Relationships between maximum angular distortion and maximum settlement for various foundation types on clays and sands (Skempton and MacDonald, 1956).

2.2 Structural Damage Due to Excavations

Boscardin and Cording (1989) compiled a similar database of structural damage and ground movements associated with braced excavations and tunneling projects. Much of their database was established from projects during the construction of the metro in Washington D. C. and includes data on a number of structures that were instrumented as part of the study. This differed from earlier studies in that much of the work previously done was on structures deforming under their own weight. In contrast, ground movements around excavations include significant lateral and vertical deformations. Factors examined included the length of the building, and its location relative to the subsidence. The authors explored angular distortion, β, and horizontal strain ε_h, as parameters to correlate with building response. Figure 2.6 shows the definition of β which corresponds to the rotation of the straight line connecting two reference points minus any rigid body tilt, and is also referred to as *relative rotation* (Burland and Wroth, 1974). Horizontal strain, ε_h, is the average strain due to relative horizontal movement between two reference points. One of the results of Boscardin and Cording's study was a bullseye type chart (Figure 2.7) relating horizontal strain, angular distortion and the expected damage experienced by the structure.

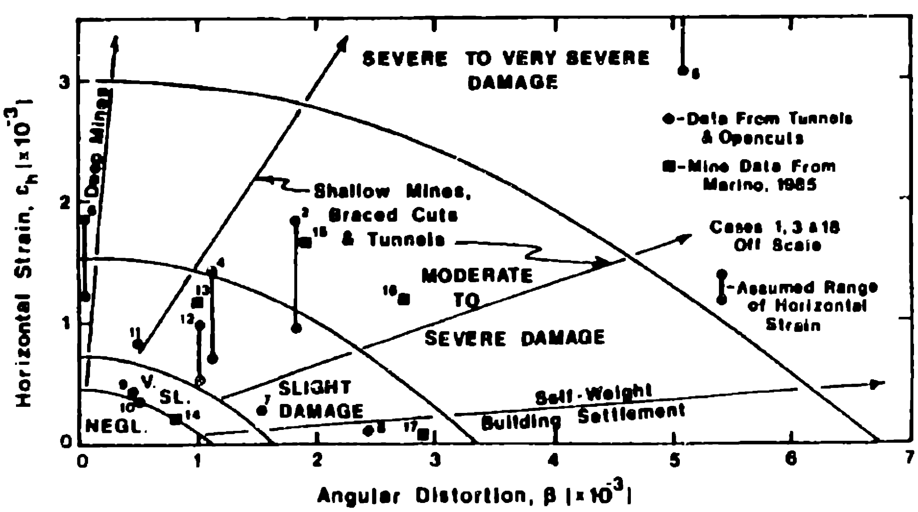
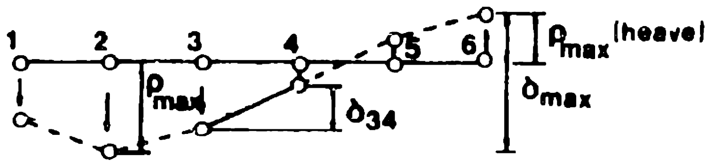
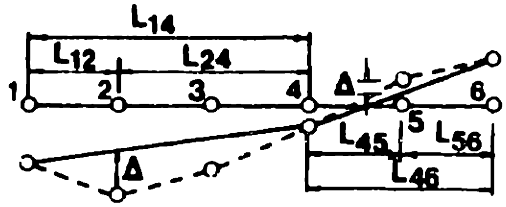


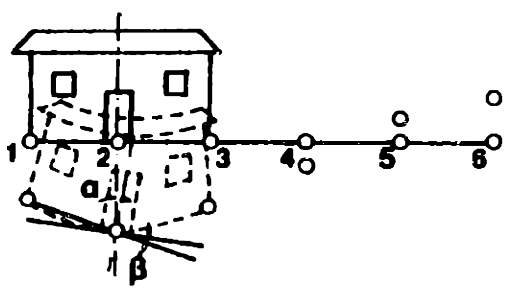
Figure 2.7 – Relationship of damage to angular distortion and horizontal extension strain (Boscardin and Cording, 1989).



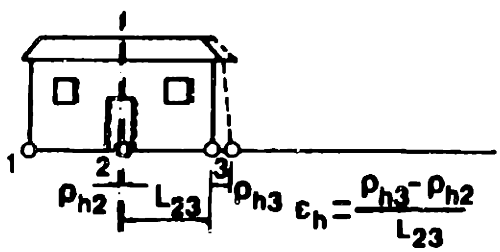
a) Settlement and Differential Settlement



b) Relative Deflection and Deflection Ratio



c) Tilt and Angular Distortion (Relative Rotation)



d) Horizontal Displacement and Horizontal Strain

Figure 2.6 – Building and ground movement parameters (Boscardin and Cording, 1989).

Conclusions from this work include:

1. Buildings adjacent to excavations are more likely to experience damage than those which undergo similar settlements solely due to their self weight.

2. The occurrence of lateral strains in the underlying soil reduces the structural tolerance with regard to differential settlements.

3. Angular distortion, β, is an appropriate parameter to correlate with measured structural damage.

4. The potential for damage can be assessed from estimates of three key parameters: 1) ε_{crit}, 2) L/H, and 3) potential ground movement adjacent to the structure.

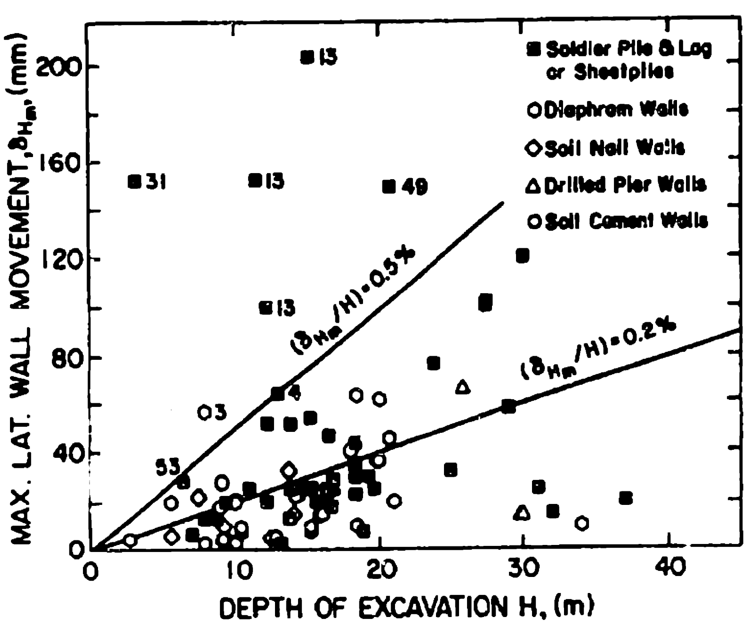


Figure 2.8 – Observed maximum lateral wall movements for insitu walls in stiff clays and sands (Clough and O'Rourke, 1990).

The last of these three parameters is best estimated from empirical data complied by Clough and O'Rourke (1990). The authors interpret movement patterns adjacent to deep excavations for a small number of classes of ground profiles. They examined bracing movements due only to excavation-induced subsidence. The authors filtered out factors such as water table fluctuations and foundation construction within the excavation in order to remove the bias that these factors add to the analysis of excavation support movement. Profiles were developed for different soil types using inclinometer and settlement measurement data. An example of this data for stiff soil is shown in Figure 2.8. They found that in stiff clays, the maximum lateral movement of excavation bracing system, regardless of its stiffness, is 0.2% to 0.3% of the depth of the excavation. In soft clays, lateral movement is a function of system stiffness and ranges from 0.5% to 2% of the depth of the excavation, depending on the factor of safety against basal heave. Soft clay results are shown in Figure 2.1. In stiff clays the settlement profile is triangular in shape, whereas in soft clays, it is trapezoidal in shape. These conclusions led to the profiles shown in Figure 2.2. Finally, for overconsolidated soils with high insitu lateral stresses, movements will penetrate farther from the wall than in other soils - potentially leading to movements within the tieback anchors.

2.3 Theoretical Models of Structural Damage

Most theoretical models that link ground deformations to structural damage are based on a framework proposed by Burland and Wroth (1974). These authors suggest that as a first approximation, the structure can be represented by an equivalent deep beam model. According to this model, the structure is then replaced by a beam of length, L, and height, H, with equivalent elastic properties E and G, that characterize the bending and shear response of the whole structure. They solve the deflection ratio, Δ/L of a centrally loaded beam (based

on Timoshenko, 1957) due to shear and bending, as functions of a), the critical extreme fiber strain ($\varepsilon_{b\,max}$) and b) critical diagonal strain ($\varepsilon_{d\,max}$). For the case where the neutral axis lies at the mid-height of the beam, the results are as follows:

$$\frac{\Delta}{L} = \left\{ 0.167\frac{L}{H} + 0.65\frac{H}{L} \right\} \varepsilon_{b\max} \qquad (2.1)$$

$$\frac{\Delta}{L} = \left\{0.25\frac{L^2}{H^2} + 1\right\}\varepsilon_{d\,max} \quad (2.2)$$

Critical strain in this analysis does not refer to strain that causes structural damage in building elements. Rather, it refers to a limiting strain at the onset of visible cracking. Mainstone and Weeks (1970) and Mainstone (1971) report ε_{crit} ranging from 0.081% to 0.137% for brick infilled panel walls, whereas Polshin and Tokar (1957) report visible cracking in brick walls at a tensile strain of 0.05%. Burland and Wroth (1974) choose a limiting value of ε_{crit} equal to 0.075%.

This work led to charts relating $\Delta/L\ \varepsilon_{crit}$ and L/H for combined bending and shear, one of which is shown in Figure 2.11. This figure shows that for L/H <1.2, diagonal strains are limiting, while for L/H > 1.2, bending is critical. Burland and Wroth's theory suggests that there is a link between L/H and the onset of visible cracking.

Equations (2.1) and (2.2) were derived assuming isotropic elastic properties for the beam model, with a Poisson's ratio, $\nu = 0.3$. Hence, $E/G = 2(1+\nu) = 2.6$ in this derivation. Burland and Wroth also examined the effect of beam stiffness on the relation between $\Delta/L\varepsilon_{crit}$ and L/H by varying values of E/G (Figure 2.12).

When the neutral axis is moved to the lower extreme fiber of the beam, considered to be more realistic behavior of a stiff raft foundation, the equations reduce to:

$$\frac{\Delta}{L} = \left\{0.083\frac{L}{H} + 1.3\frac{H}{L}\right\}\varepsilon_{b\,max} \quad (2.3)$$

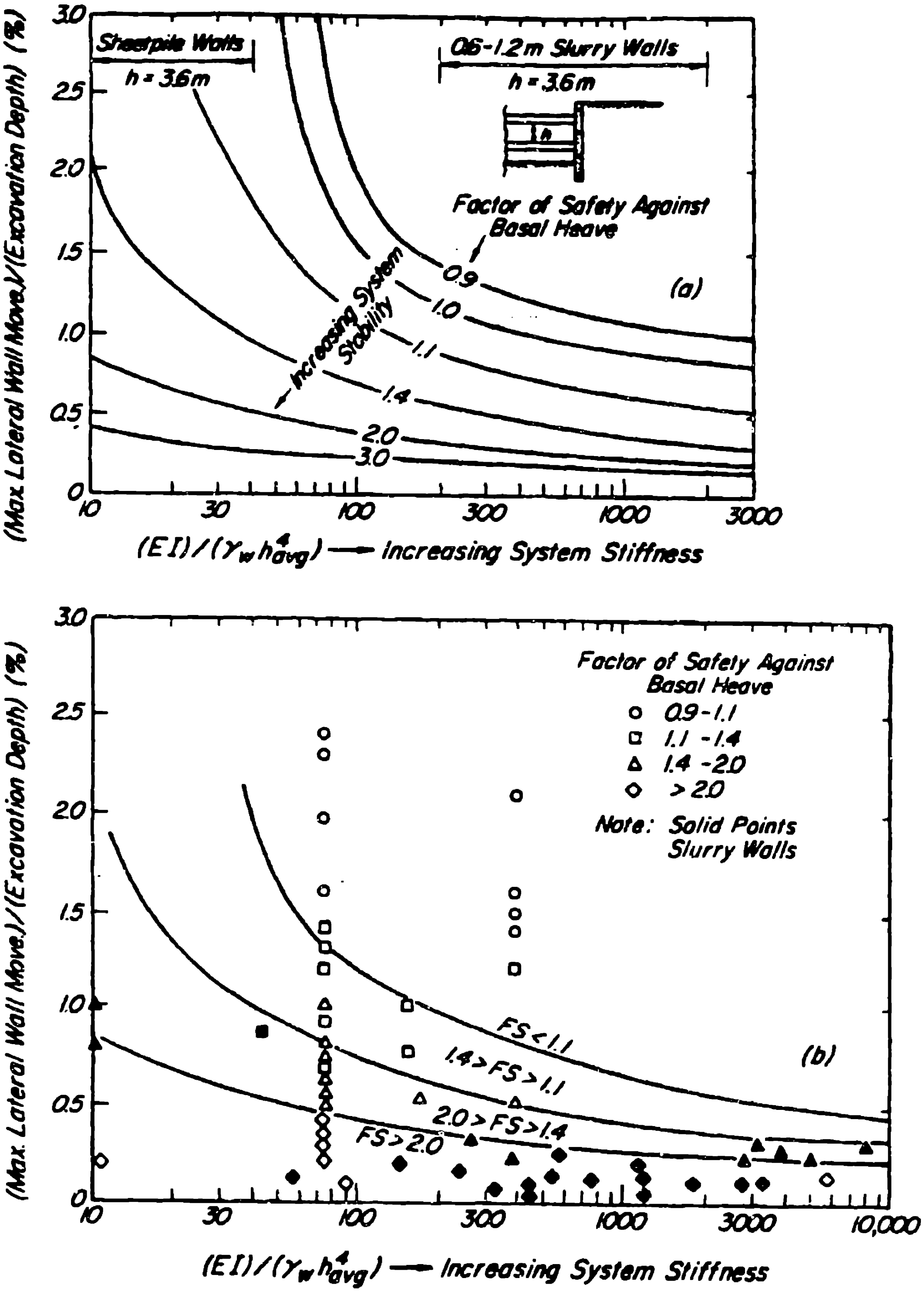


Figure 2.1 – Relation among maximum lateral wall movement, system stiffness, and factor of safety against basal heave for cuts in plastic clay: *(a)* calculated by finite-element solutions; *(b)* comparison with field measurements (Terzaghi, et al., 1996).

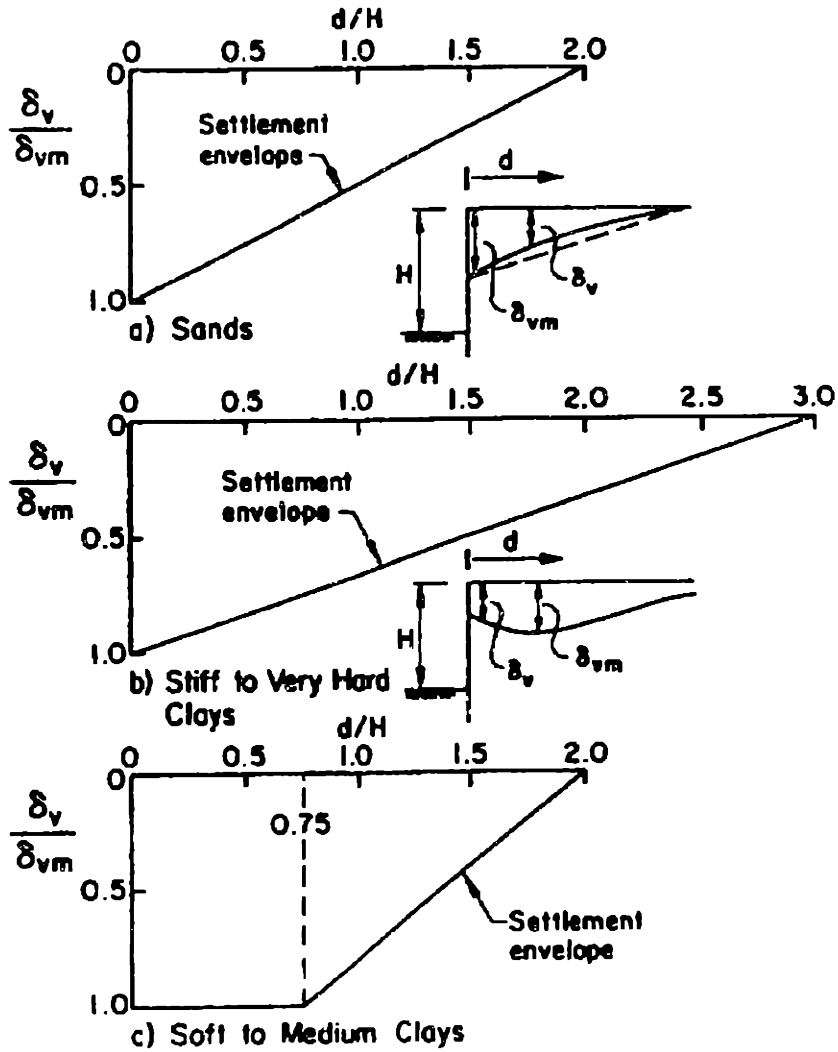


Figure 2.2 – Dimensionless settlement profiles for estimating the distribution of settlement adjacent to excavations in different soil types (Clough and O'Rourke, 1990).

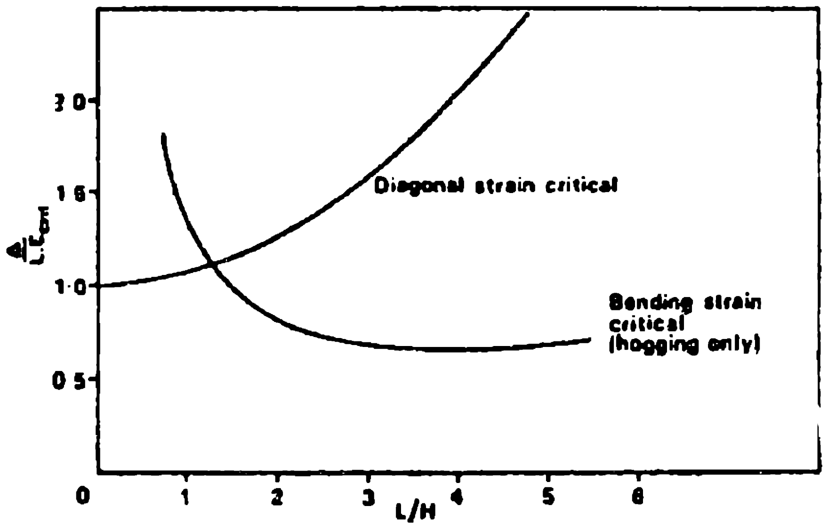


Figure 2.11 – Relationship between $\Delta/L\varepsilon_{crit}$ and L/H for rectangular beam deflecting due to combined bending and shear – neutral axis at one edge (Burland and Wroth, 1974).

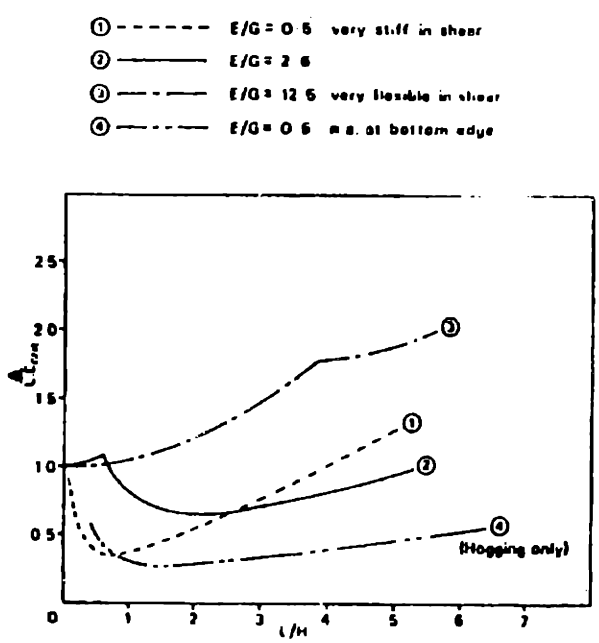


Figure 2.12 – The influence of E/G on the relationship between $\Delta/L\varepsilon_{crit}$ and L/H for rectangular beams (Burland and Wroth, 1974).

$$\frac{\Delta}{L} = \left\{ 0.064 \frac{L^2}{H^2} + 1 \right\} \varepsilon_{d\max} \quad (2.4)$$

Equation (2.3) is only valid for foundation heave leading to a deformation profile that is concave downward (hogging) since $\varepsilon_{b\ max}$ would be zero for a concave upward profile (sagging).

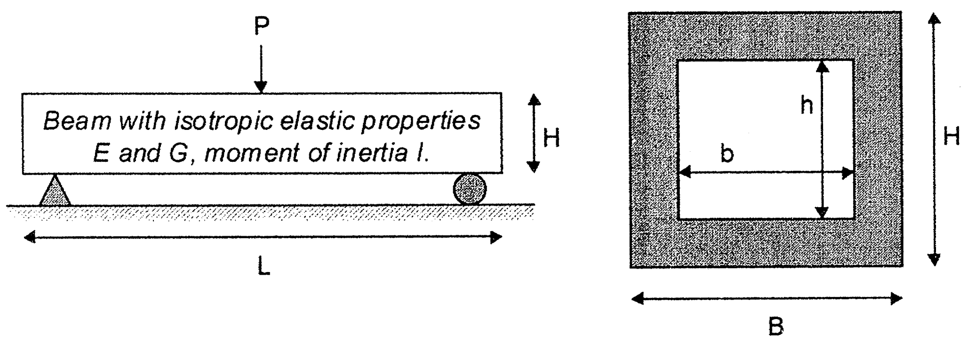
While this is an understandably simplified analysis of structural behavior, it is not a complicated task to rederive these relations using Timoshenko's equations (variables defined in Figure 2.13a), assuming that a building is more accurately modeled as a hollow beam. Burland and Wroth's relations were developed using the equation for a centrally loaded beam flexing in both shear and bending (Timoshenko, 1957).

$$\Delta = \frac{PL^3}{48EI} \left\{ 1 + \frac{18I}{L^2 H} \frac{E}{G} \right\} \quad (2.5)$$

For this analysis, the hollow beam is assumed to have dimensions such that the thickness of the walls, the floor and the roof is one twelfth of the width and height of the exterior of the structure. The only variable that changes in Equation (2.5) is the moment of inertia, I. A new value for I is calculated for the hollow beam by subtracting the moment of inertia of the open area from the moment of inertia of a solid beam (Figure 2.13b). Based on the assumed dimensions of the hollow beam, the moment of inertia in Timoshenko's relations is calculated as approximately one half that of the solid beam. For the neutral axis in the center of the beam, Equations (2.1) and (2.2) become:

$$\frac{\Delta}{L} = \left\{ 0.083 \frac{L}{H} + 0.325 \frac{H}{L} \right\} \varepsilon_{b\max} \tag{2.6}$$

$$\frac{\Delta}{L} = \left\{ 0.5 \frac{L^2}{H^2} + 1 \right\} \varepsilon_{d\max} \tag{2.7}$$



$$\Delta = \frac{PL^3}{48EI} \left\{ 1 + \frac{18I}{L^2 H} \frac{E}{G} \right\}$$

a. definition of variables in Timoshenko's (1957) deflection equation for a beam flexing in combined shear and bending.

$$I = \frac{BH^3}{12} - \frac{bh^3}{12}$$

b. calculation of moment of inertia, I, for a hollow beam.

Figure 2.13 – Definition of variables used to model beam deflection.

Figure 2.15 is a plot of the effect of changing the beam geometry from solid to hollow. It compares Burland and Wroth's original $\Delta/L\varepsilon_{crit}$ v. L/H graph with the newly derived equations (2.6) and (2.7). This result shows that changing the beam model from solid to hollow will reduce the L/H value at which the critical strain changes from diagonal to bending. However, changing the beam model has little noticeable effect on the value of $\Delta/L\varepsilon_{crit}$ where this change occurs.

Boscardin and Cording (1989) extended Burland and Wroth's work to include a component of horizontal strain, ε_h in the calculation of ε_{crit}. Horizontal strains are likely to occur in an area where there is an open excavation or tunneling activity. The authors show that Burland and Wroth's equations can be modified to include horizontal strain by substituting for $\varepsilon_{b\ max}$ and $\varepsilon_{d\ max}$ as follows, where θ_{max} is the maximum angle of the tensile strain, taken from the horizontal:

$$\varepsilon_{b\max} = \varepsilon_{crit} - \varepsilon_h \tag{2.8}$$

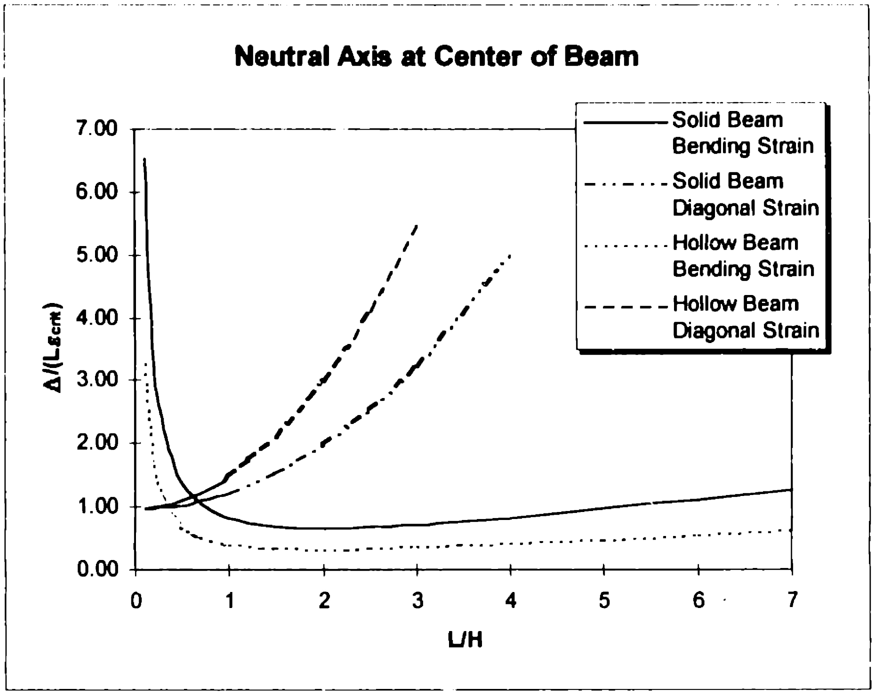


Figure 2.15 - Relationship between $\Delta/L\varepsilon_{crit}$ and L/H for solid and hollow rectangular beams deflecting due to bending and shear - neutral axis in the middle of the beam.

$$\varepsilon_{d\max} = \left\{ \frac{\varepsilon_{crit} - \varepsilon_h \cos^2\theta_{\max}}{2\cos\theta_{\max}\sin\theta_{\max}} \right\} \tag{2.9}$$

Thus, Boscardin and Cording's analysis can be applied to structures that experience damage due to settlement caused by self- weight *as well as* structures damaged by ground movement due to adjacent construction. Damage caused by horizontal strains as they relate to the length of a structure is shown in Figure 2.16.

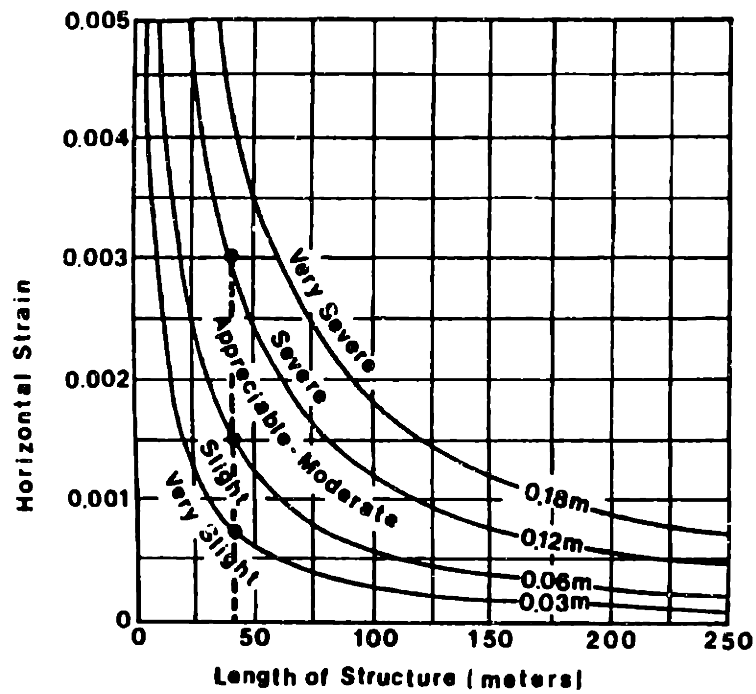


Figure 2.16 – Relationship of damage to length of structure and horizontal ground strain (National Coal Board, 1975).

Recently Boone (1996) examined a number of factors beyond simple angular distortion in order to predict potential structural damage from ground movement. Boone developed a methodology for calculating potential tensile strain experienced by structural members and related this strain to anticipated crack widths. The analysis considers bending strains, elongation of building elements, and lateral ground movements. It assumes that crack width is an appropriate indicator of damage severity. By correlating these crack widths with empirical observations on damaged buildings, Boone created a bullseye chart (Figure 2.17) to predict damage as a function of the estimated crack width due to total tensile strain and the estimated crack width due to the maximum principal tensile strain. The significant results of this work are a methodology for approximating crack width, and a table to use this approximation for estimating damage to typical buildings. Figure 2.18 shows Boone's version of the table developed by Burland, et al., (1977) that relates crack width and damage.

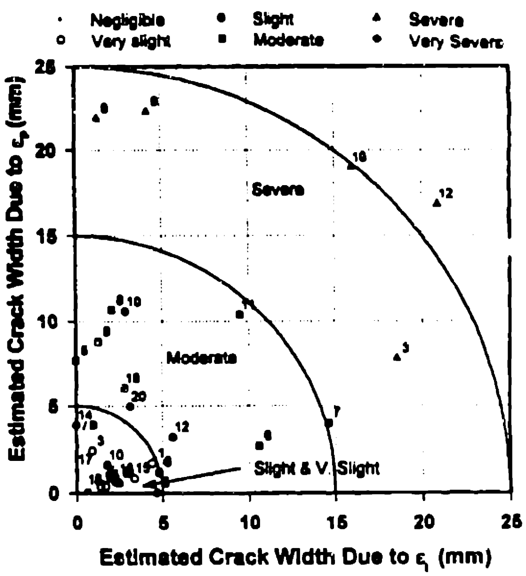


Figure 2.17 – Estimated crack widths and reported damage (Boone,1996).

Damage category (1)	Description of typical damage (2)	Approximate crack width (3)
Negligible (0)	Hairline cracks.	<0.1 mm
Very slight (1)	Very slight damage includes fine cracks that can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external brickwork visible on close inspection.	1 mm
Slight (2)	Slight damage includes cracks that can be easily filled and redecoration would probably be required; several slight fractures may appear showing the inside of the building; cracks that are visible externally and some repointing may be required; and doors and windows may stick.	5 mm
Moderate (3)	Moderate damage includes cracks that require some opening up and can be patched by a mason; recurrent cracks that can be masked by suitable linings; repointing of external brickwork and possibly a small amount of brickwork replacement may be required; doors and windows stick; service pipes may fracture; and weather-tightness is often impaired.	5 to 15 mm or a number of cracks >3 mm
Severe (4)	Severe damage includes large cracks requiring extensive repair work involving breaking-out and replacing sections of walls (especially over doors and windows); distorted windows and door frames; noticeably sloping floors; leaning or bulging walls; some loss of bearing in beams; and disrupted service pipes.	15 to 25 mm but also depends on number of cracks
Very severe (5)	Very severe damage often requires a major repair job involving partial or complete rebuilding; beams lose bearing; walls lean and require shoring; windows are broken with distortion; and there is danger of structural instability.	Usually >25 mm but also depends on number of cracks

Figure 2.18 – Severity of cracking damage (after Burland, et al., 1977).

3 INSTRUMENTATION

This chapter describes the types of instrumentation that are used to monitor building and ground movements. Sections 3.1 and 3.2 consider instrumentation in two categories, 1) devices that are placed in the ground to measure movements and 2) those that are attached to the structure to measure displacements and/or loads. Some types of instruments do not fit neatly into either category. For instance, load cells can be used to measure the lateral earth pressure on a structural interface, the tip pressure at the base of an end-bearing pile, or to register the tensile load in a tieback anchor. As another example, tiltmeters may be used both in the ground and on a structure. They may be placed in a hole in the ground to monitor slope movements, or may be placed between a stable benchmark and a structure to measure foundation movement. Similarly, pendulums may be used on a structure to monitor building movements or in a borehole to measure ground movements

The discussion of each instrument includes an overview of precision and resolution, common uses, advantages and disadvantages and some considerations for automated data collection. Where appropriate, recently published interesting applications of these instruments are noted.

Section 3.3 discusses data acquisition systems applicable to geotechnical instrumentation. Different types of systems will be investigated, and their application as part of a geotechnical monitoring program will be explored.

3.1 Instrumentation in the Soil

There are many types of instruments that are useful for generating data on the geotechnical conditions at a construction site. Designs can usually be divided into mechanical systems and electrical systems. Electrical instruments usually work on one of two principles, electrical resistance or vibrating wire. Reliability is a major concern for instrumentation, especially when it is to be used for long term monitoring of potential damage to a structure. Hanna

(1985) reports that the Telemac Company recorded the long term reliability of 3346 extensometers in their use. Of these, 5% broke down in the first year and 18% after 30 years. An average 15% failure rate for instrumentation is expected over a 30 year period. An interesting long term accuracy study is mentioned by Burland (1995). In this study, biaxial tiltmeters were part of a redundant monitoring scheme that was used over a number of years to monitor movements in the Tower of Pisa. The author compares tiltmeter data to movements that were measured during the same period using precision leveling, and reports that of the four tiltmeters installed, one showed continued drift throughout the monitoring period, while two others showed excessive temperature dependence in at least one of their measurement axes. Without the leveling data, inaccurate movements of the structure would have been recorded using only the tiltmeters. This finding suggests that there is a benefit to redundancy in instrumentation for a geotechnical monitoring program. More specifically, it is beneficial to check the data generated by one type of measuring device with similar data measured by a different type of instrument.

3.1.1 Inclinometers

Inclinometers are used to monitor lateral ground deformations. Frequently they are used to monitor slope movements. They fall into two categories, 1) probe inclinometers and 2) fixed-in-place inclinometers. All inclinometers require the installation of a tube, known as a casing, within a borehole in the soil, or through the embedded structural wall on the structure being monitored. In structural monitoring applications, the casing is generally easy to cast into a diaphragm wall, but it is much harder to attach inclinometers to piles or sheet pile walls. In the latter installation, the casing can be welded along the length of the pile, or can be grouted in a pipe that has been welded to the pile. Alternatively, a hole can be drilled in the pipe pile and the casing installed through the base of the pile (Dunnicliff, 1988). The casing remains in place throughout the measurement period. These are the two main types of inclinometer:

i) **Probe Inclinometers:** Probe inclinometers, Figure 3.1, use an instrumented 'torpedo' to measure lateral deformations. This torpedo contains internal tiltmeters or accelerometers that continuously measure the angle of the torpedo as it is lowered into the tube. Torpedo position is maintained by wheels that run in a groove, or keyway, that is cast into the tube. The collected data allows calculation of the horizontal displacement of the tube and a continuous profile of deformation can be generated for a discrete time interval.

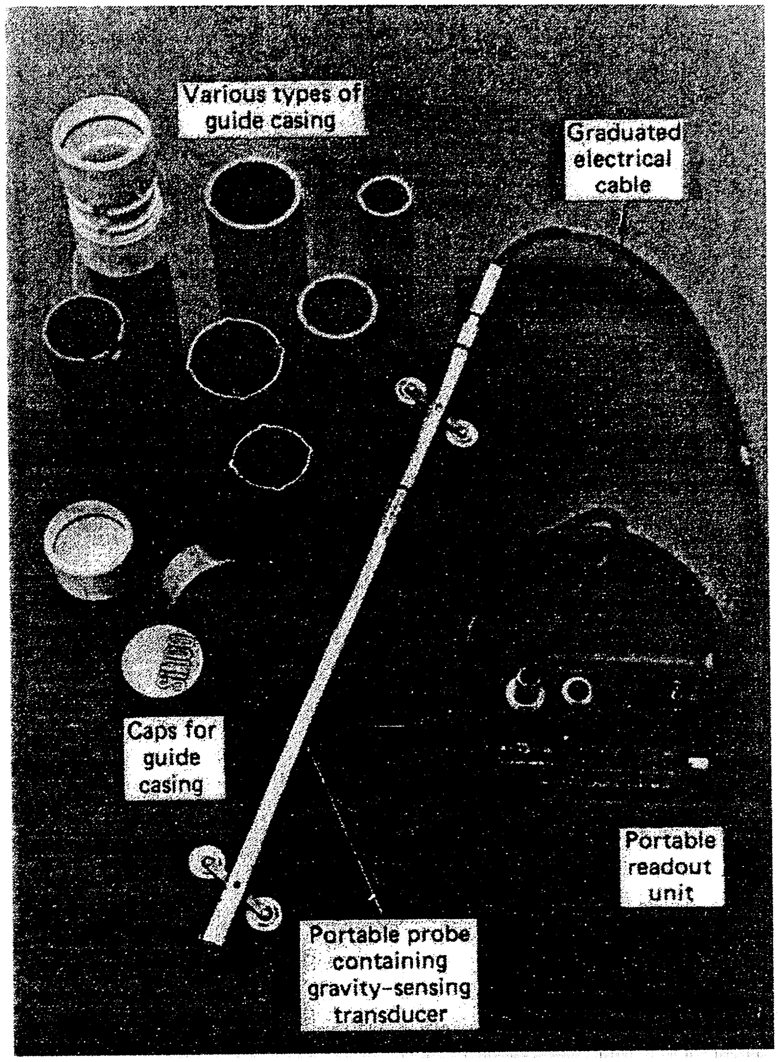


Figure 3.1 – Slope Indicator Company Digitilt® Inclinometer.

ii) **Fixed-in-Place Inclinometers:** Fixed-in-place inclinometers do not require the use of a torpedo to generate data. The design consists of a tube that contains vibrating wire transducers, electrolytic level sensors, or accelerometer transducers located at intervals along its length. These sensors measure individual deflection at the elevation of their location. This type of inclinometer can generate continuous data on deflection within specific layers of soil. An example of a fixed in place inclinometer is shown in Figure 3.2.

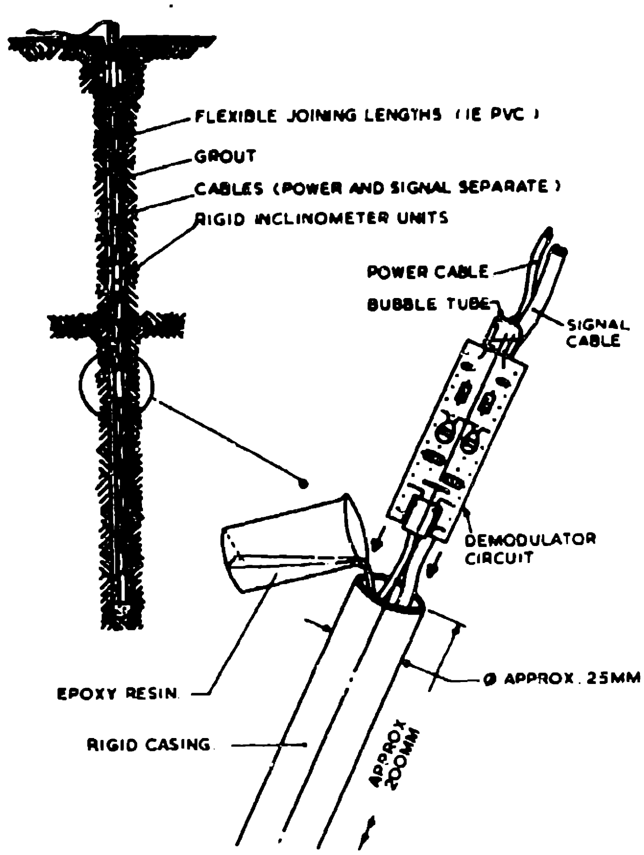


Figure 3.2 – Fixed in place Inclinometer (Enever et al., 1977).

Inclinometer casings may be made of several different types of material, including pultruded fiberglass, aluminum alloy, polyvinyl chloride (PVC) or acrylonitrile/butadiene/styrene (ABS) plastic. The material must be flexible enough to allow the casing to deform with the soil, yet strong enough to maintain a circular profile to allow for movement of the torpedo. ABS is preferred over PVC in many cases since PVC tends to become more brittle (Dunnicliff, 1988). On the other hand, plastics must be protected from extreme temperatures, as heat can cause warping in the keyway. Special attention should also be paid during the coupling process to prevent soil intrusion and to safeguard against the creation of a spiral keyway in the tube.

Inclinometer precision varies depending on the specific sensor technology used in the torpedo or in the fixed-in-place casing. Dunnicliff (1988) reports values of precision that range from ±0.02 in. to ±1 in. in 100 feet for different technologies. This variability is a function of a number of factors, including but not limited to the inclination of the casing, the physical condition of the torpedo, the alignment of the casing, and the borehole backfilling procedure. If the casing is not well grouted in the borehole, voids between the casing and the surrounding soil will lead to errors in deformation measurements. Another important consideration in inclinometer design is the location of the zero position of the top of the casing, a task that can be simplified by extending the casing to a fixed datum.

Data logging can be automated for a probe inclinometer. However, an operator is usually required to conduct the torpedo transit. Thus remote logging does not make sense for this type of monitoring device. In contrast, the fixed-in-place inclinometer lends itself directly to the types of automated data acquisition systems that are discussed in detail in Section 3.3.

Advantages & Disadvantages

One of the limitations of inclinometers is that they only measure horizontal ground deformations. Probe inclinometers generate a continuous deformation profile with depth, while fixed-in-place designs provide information only at the location of the internal sensors. Hence the deformation profile is a coarse approximation of the continuous profile offered by the torpedo system.

Probe inclinometer data is obtained at discrete times that are usually specified in contract documents. In contrast, automated data collection for fixed-in-place designs can assure continuous data logging throughout construction and hence provide a more immediate alert of potential problems caused by accelerating lateral deflection.

Interesting Applications

Inclinometers are usually an important component of geotechnical monitoring programs for excavations and tunnels. In Boston, recent urban excavations have relied on inclinometer data to measure ground movements during construction. Decisions on excavation bracing benefit from these data in that designs can be less conservative in the presence of real-time deformation information. Furthermore, deformation data may be necessary to appease abutters worried about potential damage to their property. Erikson, et al., (1992) report on a number of Boston excavations that used inclinometers as part of the geotechnical monitoring scheme:

Five Hundred Boylston Street required a 35- to 40-foot deep excavation adjacent to an historic church that had been previously damaged by nearby construction. Extensive monitoring was required (Figure 3.3) to satisfy the owners of the church that the new construction would not adversely affect their property. As part of the monitoring program, eight inclinometers were employed to measure the deformation of the excavation support system, which consisted of reinforced concrete diaphragm walls and steel sheet piling. Without inclinometer data it would have been difficult to predict, and more importantly prevent impending damage. In the view of the authors of this study, inclinometers were the most important source of geotechnical data during construction (Erikson, et al., 1992).

3.1.2 Extensometers

Extensometers are widely used in construction projects; three types are discussed in this section, 1) surface extensometers, 2) probe extensometers and 3) fixed-in-place extensometers. Surface extensometers are also known as *crack gages* and *convergence gages*. These devices usually consist of a set of pins connected by a tensioned wire, a steel rule or a tape. Changes in distance between the pins may be measured on the tape or wire with a scale or calipers.

Figure 3.4 is an example of a tape extensometer.

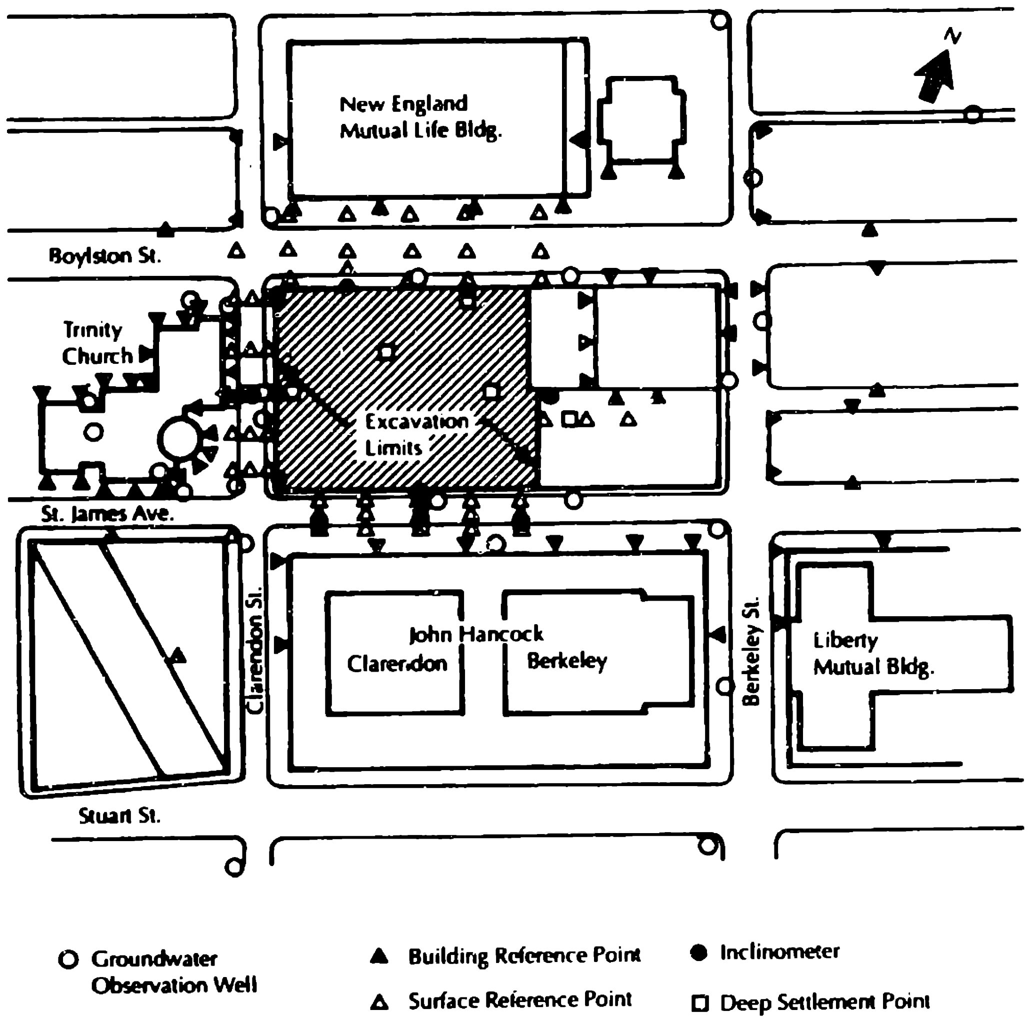


Figure 3.3 – Instrumentation location plan for 500 Boylston Street (Erikson, et al., 1992).

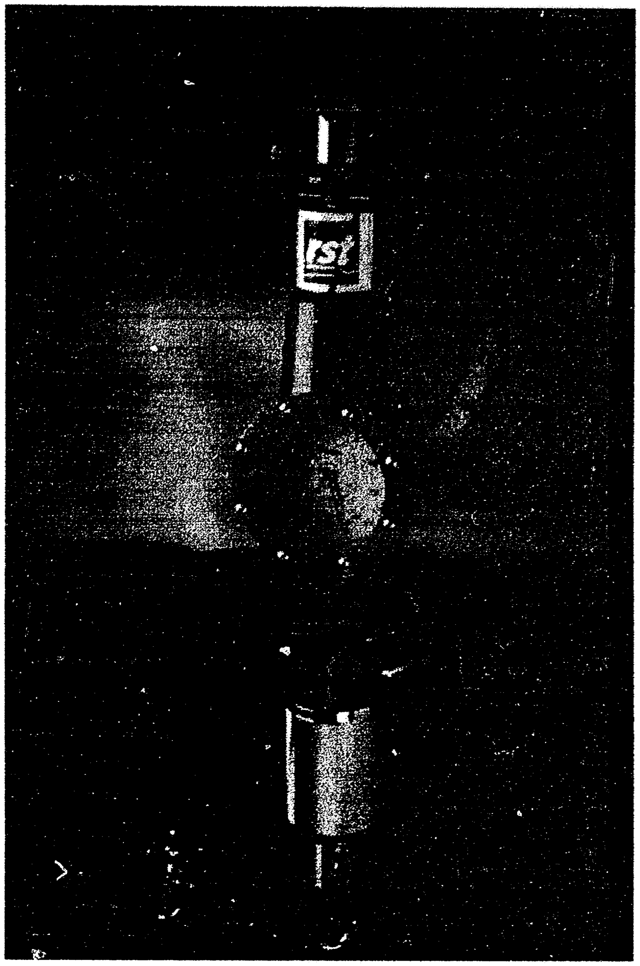


Figure 3.4 – RST Instruments tape extensometer.

Probe extensometers work on a principle similar to the inclinometer mentioned in Section 3.1.1. The casing into which the probe is inserted is outfitted with magnets along its length. As the probe passes these magnets, the distance between them is measured. Probe extensometers may be installed horizontally or vertically. The probe can record the location of the sensors using a frequency-displacement induction coil, a current-displacement induction coil or a magnet and reed switch.

Fixed-in-place extensometers are similar to fixed-in-place inclinometers. A probe is not required to read data. Measurement methodology varies based on the type of extensometer. For instance, some units consist of a plate anchored at a stable depth. A riser pipe is attached to the plate, and projects upward out of the soil. The vertical movement of this pipe is measured using precision leveling. In another system, linear displacement transducers are used to provide an output signal that is a function of the extension of the gage. Still other extensometers use vibrating wire or electrical resistance strain gages (Figure 3.5).

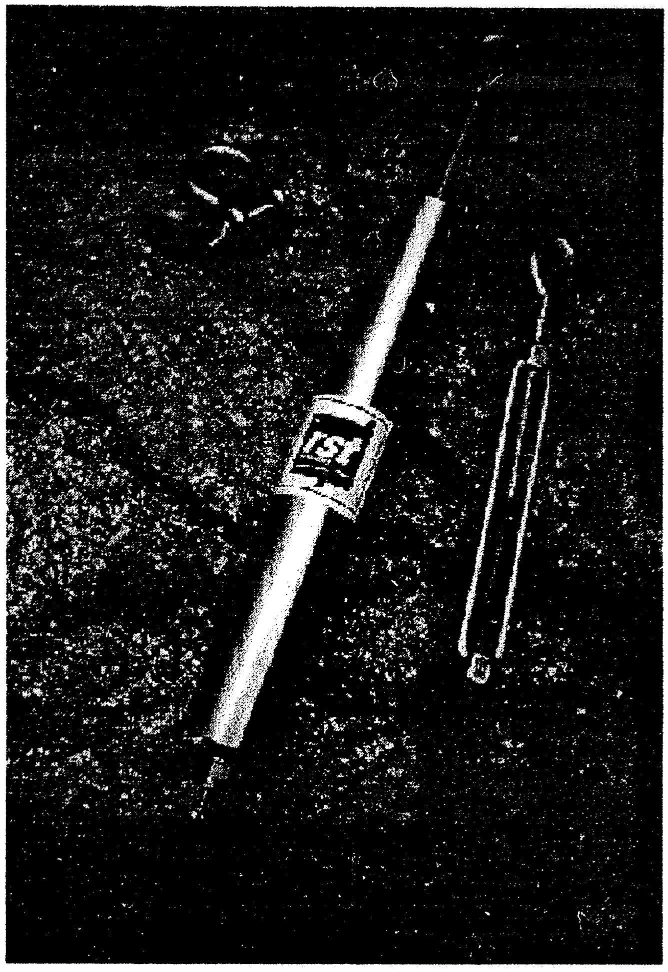


Figure 3.5 – RST Instruments vibrating wire extensometer.

Surface extensometers usually have a limited span, but can be precise to ±0.005 in. for the pin setups. (Dunnicliff, 1988) Probe extensometers vary in resolution depending on the technology used in the probe. For instance, current displacement probes have precision to ±0.02 in., while frequency displacement gages are precise to ±0.001 in. Magnet and reed switched probes are accurate to ±0.02 in. for vertical installations and ±0.1 in. for horizontal installations. Precision of the fixed-in-place type extensometer is a function of the measurement technology. For a discussion of precision in leveling, see Section 3.2.1. For various strain gages, see Section 3.2.4.

Systems that produce an electrical output signal easily lend themselves to automated data collection. Methods that rely on leveling and probe readings, on the other hand, are not as easily incorporated into such a scheme. Since the category extensometers encompasses such a broad range of instrumentation, it is not feasible to make generalizations about the best data collection system for this family. Decisions will need to be made on a case by case basis, subject to some of the considerations discussed in Section 3.3.

Advantages & Disadvantages

As mentioned before, surface extensometers have a limited range of measurement. However, they are very inexpensive, especially when compared to borehole installations for probe and fixed-in-place equipment. Furthermore, the wire type of gage may be fitted with a trigger alarm to warn when excessive deformation has taken place.

Different types of probe extensometers have unique advantages and limitations. For instance, in each case compaction of the soil surrounding the installation is difficult. This is because large compaction equipment may not be used without a good chance of damaging the casing. However, this equipment is available in many versions for all different site conditions. Another advantage is that the frequency-displacement type probe is highly accurate. The advantages and limitations of probe inclinometers, discussed in Section 3.1.1, also apply to probe extensometers.

The advantages and limitations of fixed-in-place extensometers are similar to those mentioned for leveling methods and for strain gages.

Interesting Applications

Schrijver and Hemerijckx (1991) report on a tunneling project under a river in an historic section of Antwerp, Belgium that employed the use of extensometers to measure vertical displacements. A total of 11 instruments were used, 7 to measure the settlement of a quay wall along the river and 4 to measure deformations of near historic structures. Figure 3.6 shows a cross section of the project area.

The tunnels were constructed in sand and clay as part of a transit system connecting the residential part of the city to the old town center. Extensometers showed that settlements in the vicinity of the quay wall were 3 mm, while settlements near the historic buildings were 2 mm. Angular distortion due to differential settlement was reported to be well below the allowable limit of 1/500 to prevent damage to these old buildings. Results of extensometer monitoring are shown in Figure 3.7.

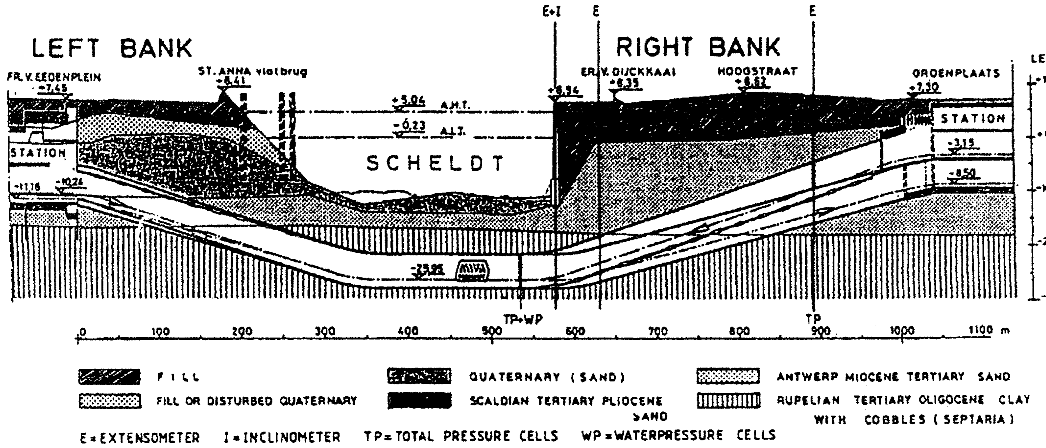


Figure 3.6 – Geological profile of Antwerp project (Schrijver and Hemerijckx, 1991).

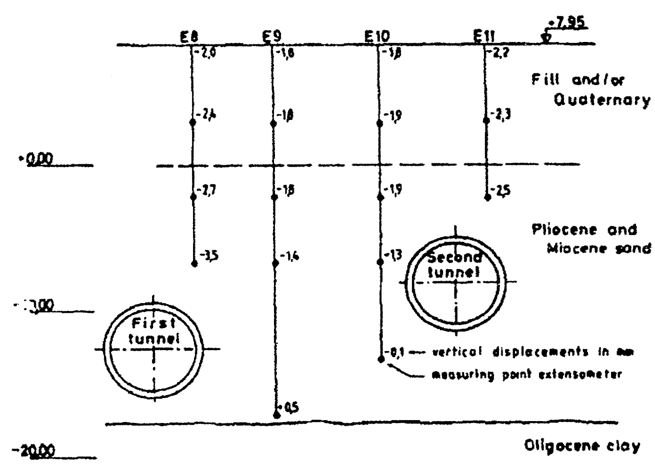


Figure 3.7 – Results of extensometer measurements in Antwerp, Belgium (Schrijver and Hemerijckx, 1991).

3.1.3 Combined Inclinometer/Extensometer

A new development in instrumentation combines the function of the inclinometer and the extensometer in the same casing. These units may be used to monitor 3-dimensional deformations in either a horizontal or vertical plane. The design consists of a torpedo probe that contains internal sensors for tilt and sensors for position. The casing is similar to that of an inclinometer, except it also contains external magnets at discrete intervals* that allow the probe to measure horizontal or vertical position (Figure 3.8).

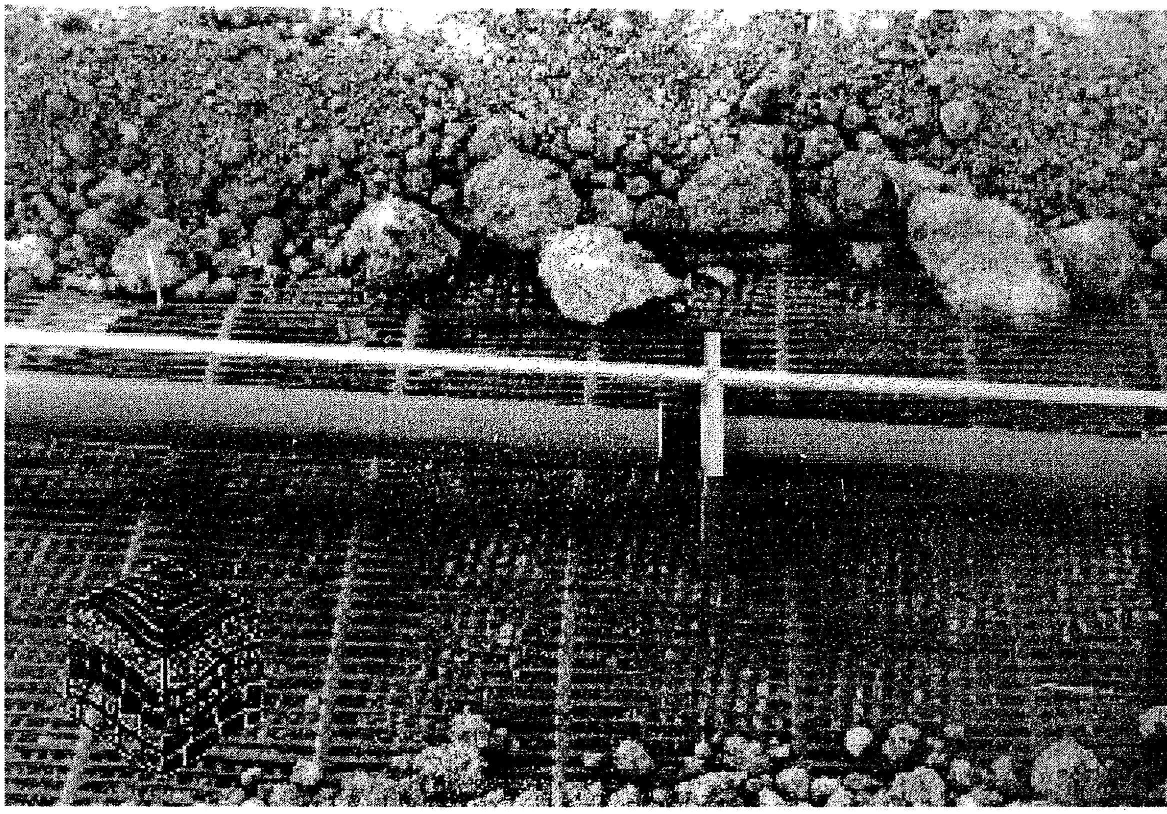


Figure 3.8 – View of a combined inclinometer/probe extensometer casing showing external magnet (GeoTesting Express, 1996).

Precision of these instruments has been quoted as ±0.2 mm for 3D deformation measurement at 1 m intervals (Pagani, 1998), and 1/16" over 100 feet for vertical profiling at 2 foot intervals (GeoTesting Express, 1996).

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* The INCREX unit available from Pagani uses magnets set 1 m apart (Pagani, 1998).

Interesting Applications

GeoTesting Express (1996) reports that in East Bridgewater, Massachusetts, a horizontal inclinometer/extensometer was installed six inches beneath a landfill liner to monitor deformations in the liner during operation. The casing was combined with a second pipe allowing a pulley and cord to be attached to the probe. (Figure 3.9). This arrangement makes it possible to run the probe back and forth in the casing to make measurements.

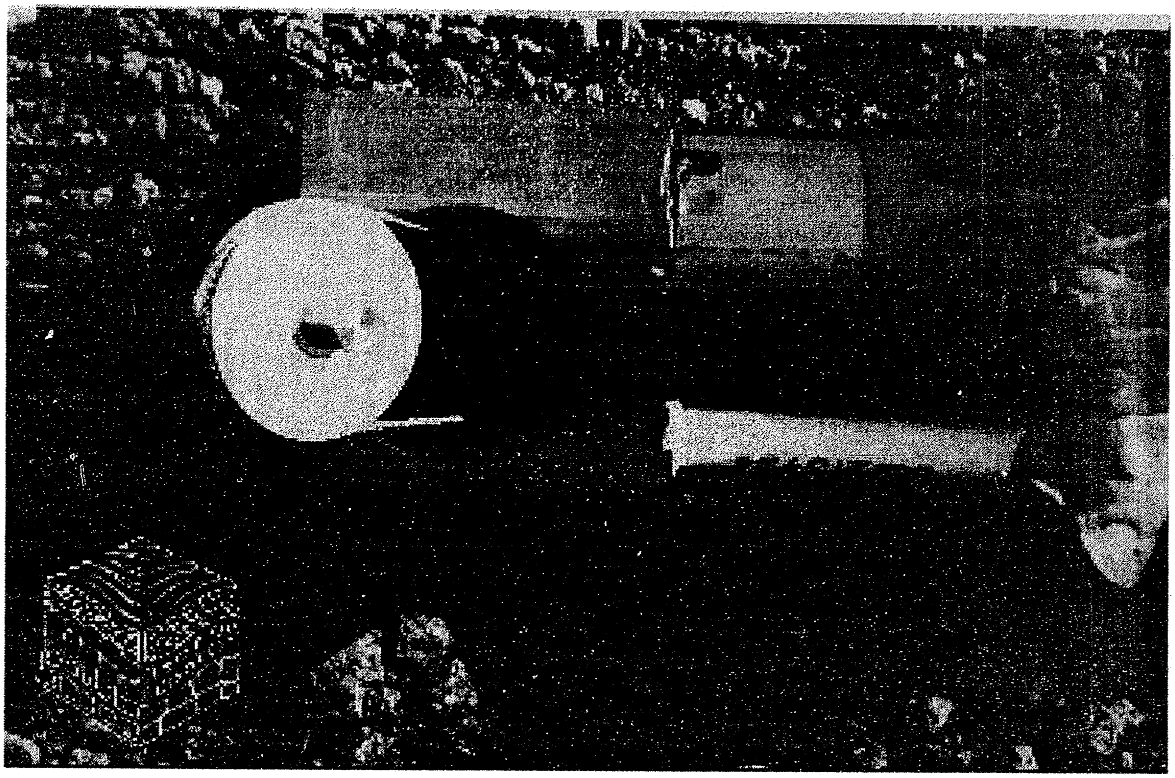


Figure 3.9 – Horizontal inclinometer/extensometer showing pulley arrangement (GeoTesting Express, 1996).

The probe in this case has an audible tone to indicate magnet position in the casing. When a beep is heard during testing, a steel tape is used to measure horizontal position of the probe, with an accuracy of ±0.01 in over 100 ft. Results of the testing program are shown in Figure 3.7.

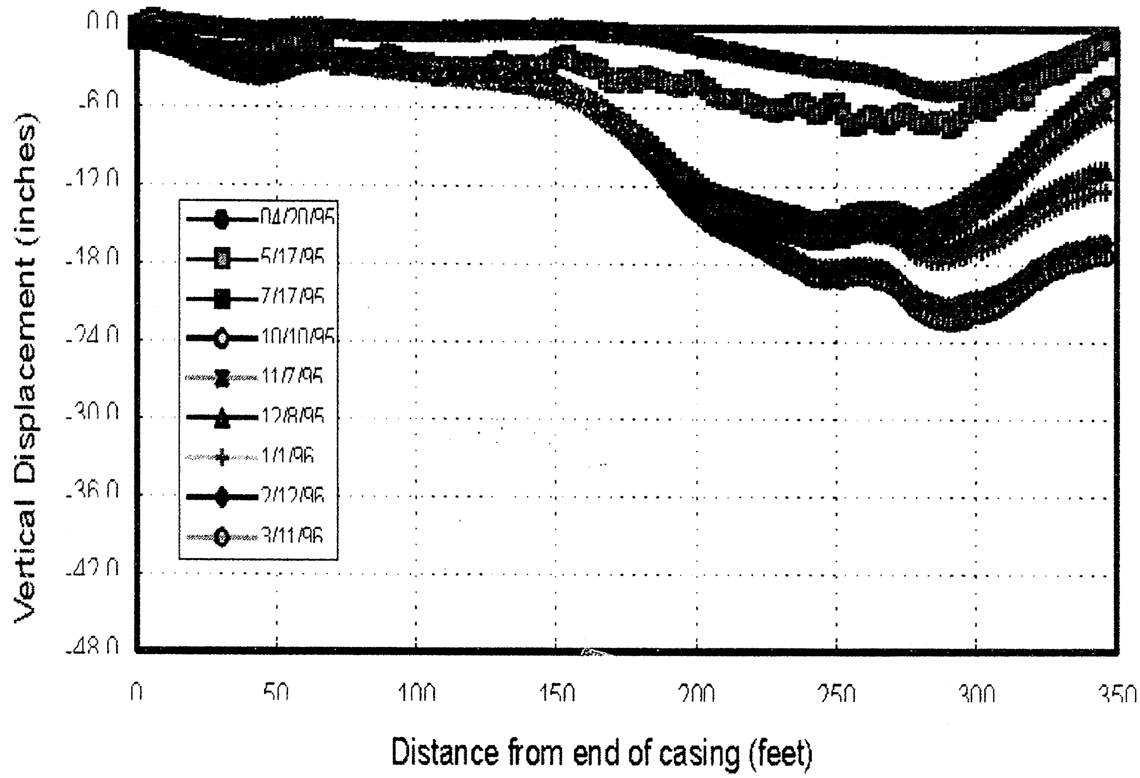


Figure 3.10 – Horizontal inclinometer/probe extensometer data from East Bridgewater, Massachusetts (GeoTesting Express, 1996).

3.1.4 Liquid Levels

The design and use of water levels for measuring settlement between two points was discussed originally by Aldrich (1951). Data collection for the study, on the differential settlements of the Hayden Library on the campus of MIT, was performed using water levels (or aqualevels) to measure differential settlements in basement columns. Modern applications employ liquid levels to measure changes in elevation in the ground. Other terms for this type of instrumentation include *hose level* and *hydraulic pneumatic level* gages.

The liquid level works on the principle that two connected reservoirs will naturally seek the same pressure when they are connected. This pressure is manifested in the height of the water column in the reservoir. For a zero flow situation, two reservoirs placed at different locations may be used to directly measure the difference in elevation between the points (Figure 3.11).

Two water columns are connected by a hose with shut-off valves at either end. The columns are attached to frames that provide a means for hanging the columns from reference pins. The frames also provide a means for leveling the system once it is hung. A precision scale is provided for measuring the height of the water in the column.

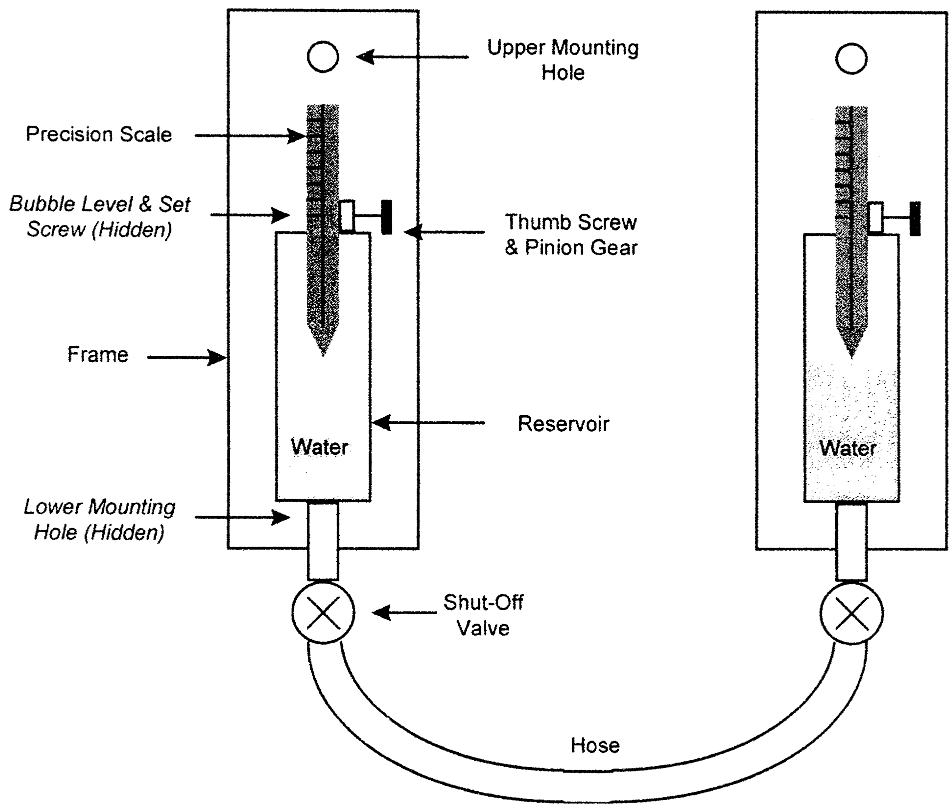


Figure 3.11 – Principle of the aqualevel.

This MIT system was used by attaching each level to a pin on a basement column in the library. After each frame was leveled, the valves were opened and the water allowed to equilibrate. The resulting water heights were measured on the precision scale by moving it up and down with the thumb screw until the point just touched the meniscus of the water.

Each scale is accurate to 0.01 in. After the first reading, the position of the levels is reversed and another reading taken. The two readings are averaged, for an overall accuracy of 0.02in. The difference in the readings between the two reservoirs is a direct measure of the relative settlement between the two points.

Modern designs of the liquid level are available, with much higher precision. The sensitivity of a model offered by Geokon, Inc. is 0.025% of the range, which is 0.0025 inches for a model that can measure differential settlements up to 10 inches.

Clearly, the use of the simple water level described previously is labor intensive. Automation of the data collection for this system is not practical. However, modern designs of liquid level gages may be automated.

Dunnicliff (1988) describes many types of liquid levels that incorporate pressure transducers and alternative fluids like mercury. Units may be designed to measure elevation differences between two points (Figure 3.12), between more than two points, or continuously over an extended length. These systems may be set up with a central readout point, or with the readout above or below the measurement point. Data acquisition units are widely available for recording pressure transducer data. A general discussion of data acquisition as it applies to geotechnical instrumentation may be found in Section 3.3 of this chapter.

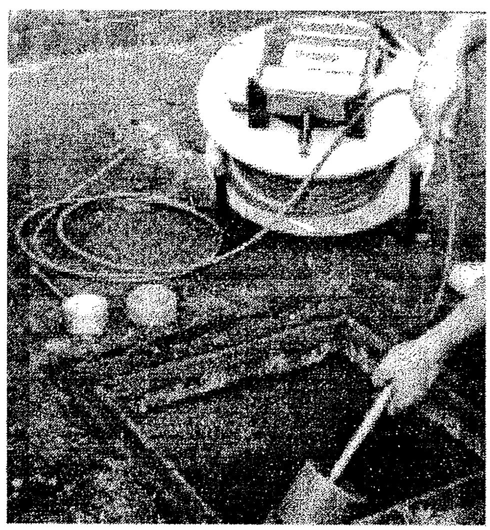


Figure 3.12 – Geotechnical Instruments Ltd. liquid level.

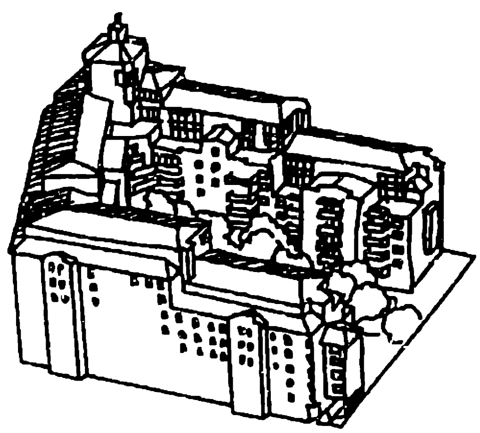
Advantages & Disadvantages

The advantage of the liquid level is that the design is simple. Modern units can be highly accurate, with precision to ±0.004 in. (Dunnicliff). However, precision is affected by surface tension forces and entrained gas in the fluid, and temperature fluctuations that lead to fluid density changes. Proper hose diameter is critical to reduce the effect of surface tension, and some systems need to be thoroughly de-aired to prevent errors contributed by fluid discontinuities.

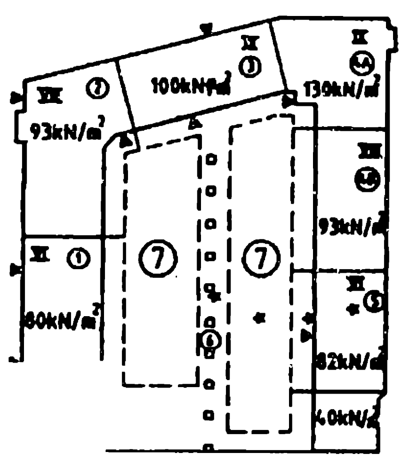
Interesting Applications

Although no details are provided about the design, Dunnicliff, et al., (1996) mention the use of a unique liquid level gage for monitoring underground construction as part of the Central Artery Tunnel project in downtown Boston. In this particular project, a highway tunnel will be placed over an existing subway tunnel within an underwater excavation. The subway will remain in operation throughout construction, which requires dredging in the channel above the subway tunnel. Dredging will reduce the overburden stresses on the tunnel and increase the chance of that the subway tunnel will float up into the channel. In addition to the liquid level, strain gages, extensometers and tiltmeters will be employed to monitor the project.

Another recent use of a liquid level was described by Svensson (1991). In this project, the measured settlements of an apartment complex built on marine clay in Sweden were compared to values predicted from elastic theory. The 9 story structure was constructed on a piled raft foundation in an overconsolidated deposit (Figure 3.13).



The Building



Total building load

Figure 3.13 – Building and loading described by Svensson (1991).

Settlement up to 6 cm, which was less than calculated using elastic solutions, was measured with the liquid level and leveling stations. The author attributed this discrepancy to the fact that the calculated value included settlement within the construction induced heave. According to Svensson, some heave is actually "locked-in" when construction occurs shortly after excavation. Ground settlement profiles were plotted for measurements taken on January 26, 1987, October 22, 1987 and April 24, 1990 (data sets ①, ②, and ③ in Figure 3.14), for three liquid levels, numbered 3, 4 and 5 (depicted by a ☆ on the plan view in Figure 3.14). The lower curves in Figure 3.14 show a comparison of liquid level data with leveling stations numbered 10 and 12. The results show that ground movements measured using liquid levels exceeded building movements (measured at leveling stations) by 15 to 25 mm after 2 years.

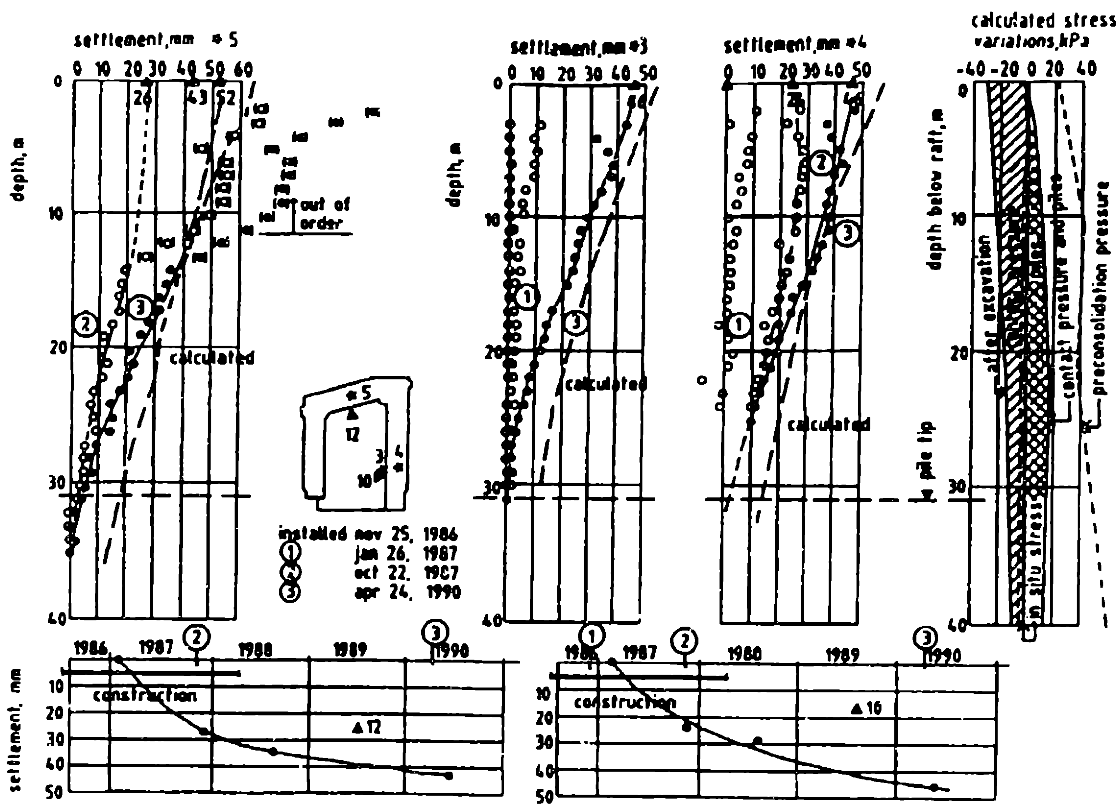


Figure 3.14 – Settlement distribution with depth as measured using liquid levels (Svensson (1991).

3.1.5 Piezometers

There are a wide range of piezometer designs available for measuring pore water pressures in the ground. Piezometers should be part of any geotechnical investigation program where groundwater conditions (flows, pressures, etc.) must be monitored. They are the most commonly used devices to provide this type of data*. What follows is a condensed examination of some of the more common piezometer designs available.

Most piezometers can be categorized in two groups. The first group of devices have no diaphragm separating the pressure measurement from the pore water. Open standpipe and twin-tube hydraulic piezometers are in this category. The second group have a diaphragm, and include devices such as pneumatic, electrical resistance and vibrating wire strain gage piezometers. Piezometers differ from observation wells in that they measure pore water pressure at a distinct elevation in the soil, while observation wells are open boreholes used primarily to determine seasonal (and other) fluctuations of the water table. Many engineers often erroneously assume that pore water pressures can be estimated reliably by hydrostatic assumptions for a known water table.

All piezometers are sealed within the borehole in which they are placed. This ensures that the measured pore pressure is representative of the layer in which the piezometer is installed. For example, the tip of the piezometer can be embedded in a sand pocket at the base of the borehole. Bentonite is usually the material of choice for sealing, and grout is placed above the bentonite. The only major differences between the instruments mentioned are the types of sensors used to measure pressure and the data collection system. The next section will present the different sensor designs. Figure 3.15 shows vibrating wire type piezometers available from two manufacturers. Key features in this figure are the porous stone, which acts as a filter to separate the pore water and the soil, the diaphragm, (or pressure membrane) which is the physical barrier that deflects under pressure, and the vibrating wire strain gage.

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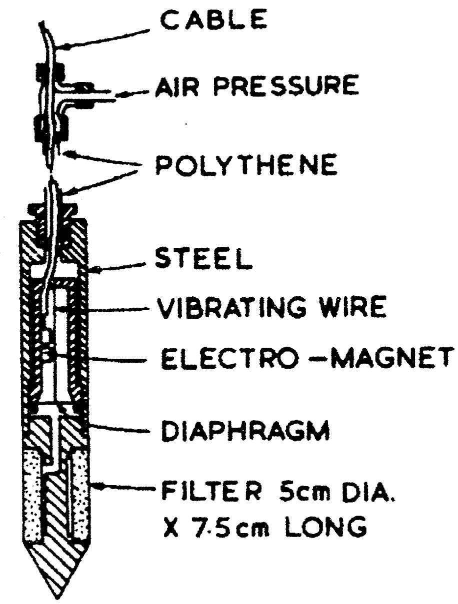
* A developing system known as TDR, with capabilities of pore water pressure measurement, is discussed in Section 3.2.7.

The strain gage measures deflection in the diaphragm, which can then be converted into a pressure reading.

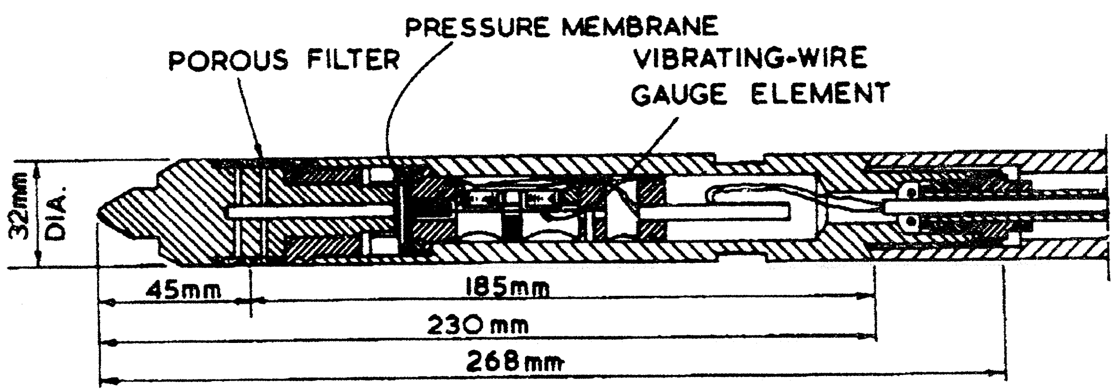
Depending on the type of sensor, data collection from the piezometer will vary. Open standpipe piezometers may use a capillary or acoustical reader as shown in Figure 3.16, while twin tube piezometers use dual horizontal tubes with Bourdon pressure gages for each tube. The twin tube unit is useful for long term monitoring of pore pressures in dams and embankments. As long as the Bourdon gages show pressures that differ only by their elevation, the piezometer may be assumed to be sufficiently de-aired to eliminate this source of error. Bourdon gages are accurate to ±0.25 to ±1% of the maximum reading over their specified range (Hanna, 1985).

Pneumatic piezometers require a gas supply line to maintain pressure in the sensor. A gage is connected to the inlet line and records the line pressure required to compensate for pore water pressure on the diaphragm. If pore pressure drops, the diaphragm deflects and opens the outlet line in the instrument. This vents gas, releasing pressure until the diaphragm shuts. Thus, a continuous reading of pore pressure is available on the inlet line gage. Literature from Geotechnical Products (1998) lists the range of pneumatic piezometers as up to 200 m of head. Accuracy for these systems is 0.5% ±0.2 m of head, with a resolution of 0.1 m of head.

Vibrating wire and electrical resistance piezometers use strain gages attached to the diaphragm. Pore pressure on the diaphragm changes the resistance properties of the strain gage, or the tension in the vibrating wire. This change in the sensor may be directly converted into a pressure reading. Product literature from Geokon (1998) lists standard ranges from 0.5 to 5000 psi for their line of vibrating wire piezometers. Fluctuations as small as 0.02 inches can be measured with these instruments. Accuracy is better than 0.5% of the full scale of these instruments (Geonor, 1998).



BRS-type vibrating-wire piezometer



Geonor vibrating-wire type borehole piezometer

Figure 3.15 – Details of vibrating wire piezometers (Hanna, 1985).

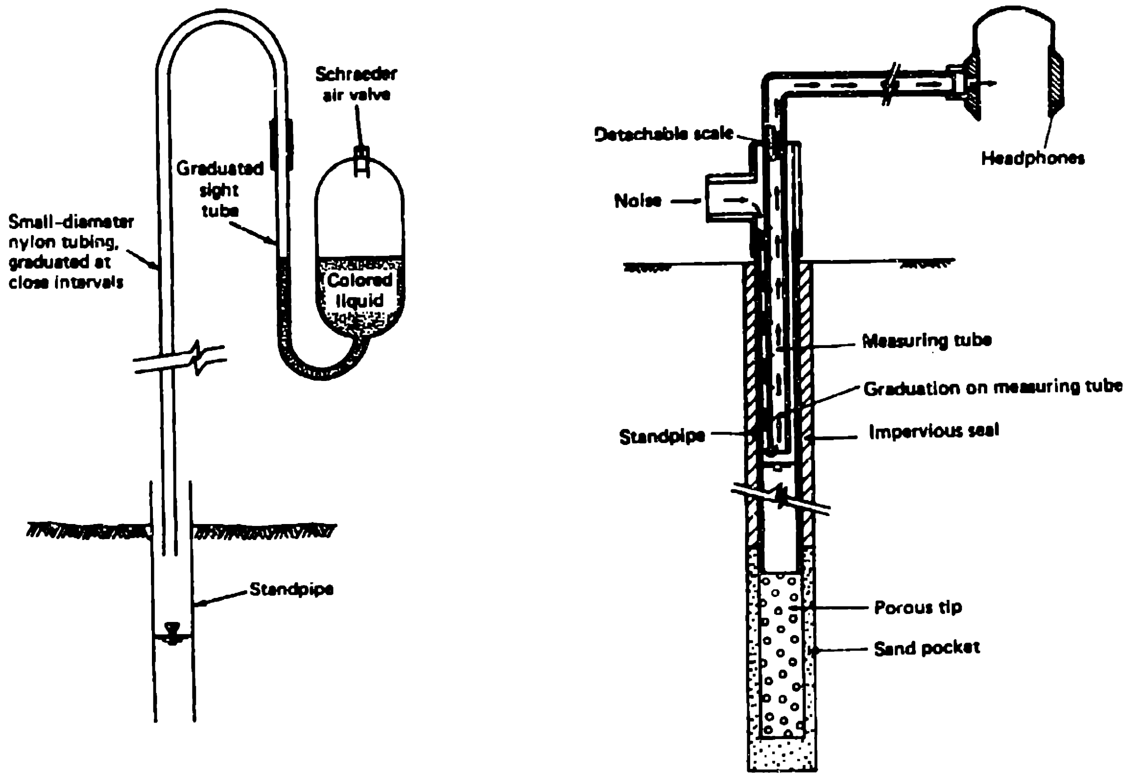


Figure 3.16 – Capillary and audio readers for open standpipe piezometers (Dunnicliff, 1988).

Open standpipe piezometers may be attached to a bubbler system that records pressure data on a strip chart. In this system, a pressure transducer is placed in the bottom of the standpipe, and a small stream of gas is passed down to the transducer through a polyethylene tube at a rate of several bubbles per minute. The pressure of the gas, which is equal to the height of water in the standpipe, is registered by a chart recorder. Details of this setup are provided by Hanna (1985). Pressure readings may be converted to electrical signals using a transducer, which are directly compatible with automated data collection.

Advantages & Disadvantages

Open standpipes have two disadvantages: 1) the water may freeze in winter. This can be avoided by adding a layer of ethylene glycol (antifreeze) in the top of the standpipe; 2) there is a long time lag in the response time of the piezometer in response to changes in water pressure, up to 10 days for 90% equalization in silt with a hydraulic conductivity of 10^{-6} cm/s

(Hvorslev, 1951). These piezometers can be reliable, however, and can be easily converted to a diaphragm type unit. Standpipes also allow for groundwater sampling at the location of the instrument.

Twin tube hydraulic piezometers require thorough de-airing to ensure accurate readings. The tubes may not be placed substantially above the piezometric elevation and regular flushing of the system may be required. These instruments are well suited for long-term monitoring, are reliable and have no moving parts (Dunnicliff 1988).

Pneumatic piezometers are capable of measuring negative pore pressures by applying vacuum to the outlet line. The time lag is short and the parts of the system that require calibration are readily accessible. However, they require a pressure source, usually gas, which will be an additional expense when integrating these instruments into a monitoring scheme consisting of mostly electric devices. Furthermore, gas flow rates decrease with increasing length in supply lines, and the supply system must be carefully connected to the instrument to ensure that there are no gas leaks.

Vibrating wire and electrical resistance gages have fast response times which makes them suitable for measuring dynamic pore pressure changes, such as would be associated with pile driving and earthquakes. Furthermore, they are easy to read (Geotechnical Products, 1998; Dunnicliff, 1988). There are no freezing problems associated with this equipment and they may be used to measure negative pore pressures. The connections in these units, however, can be sensitive to moisture and electrical resistance units have low electrical outputs (Dunnicliff, 1988). Also, for long term monitoring applications, drift and calibration associated with electronic instrumentation make them less attractive than twin-tube hydraulic piezometers.

Interesting Applications

The US Geological Survey (USGS) has incorporated piezometers in a real-time slope stability monitoring project in California. Near Placerville, a landslide occurred in the Sierra Nevada mountains in January, 1997, that closed US Route 50 and briefly dammed a fork of the American River. After the clean up, the USGS moved to install a monitoring network in areas considered to be active landslide zones in the region. The system consists of gages that take measurements in real-time and transmit data to Caltrans via the USGS computer network. Figure 3.17 shows a schematic of the system, which includes a rain gage, geophones, slope movement sensors and piezometers (Reid & LaHusen, 1998).

Data from this system is continuously available over the Internet and should provide engineers with an early warning of impending slope failure. An sample of the deep piezometer data is shown in Figure 3.18.

3.1.6 Earth Pressure Cells

Earth pressure cells are used for measuring total pressure at selected locations and orientations within the soil. When combined with piezometer data, effective stresses in the area of the cell can be calculated. For reasons described in the following text, these devices are best deployed as part of a measurement scheme that allows for independent data verification from other measurement devices.

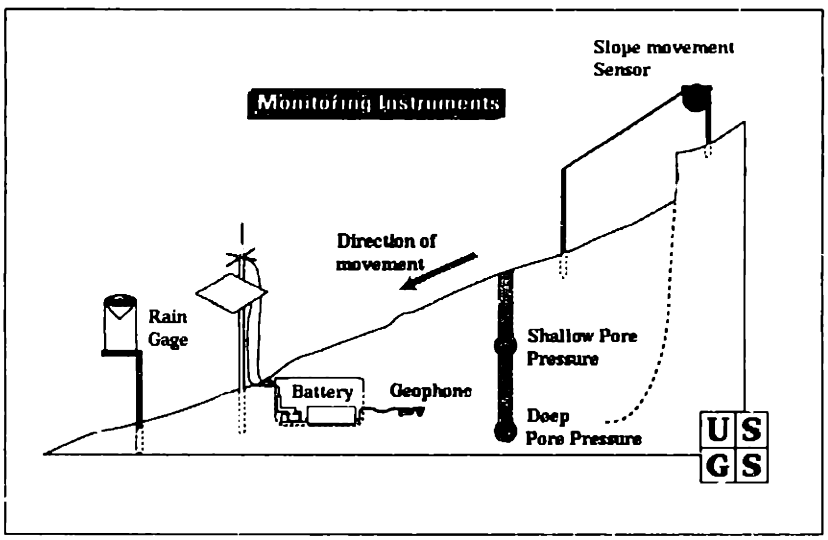


Figure 3.17 – Slope stability monitoring scheme developed by the USGS (Reid & LaHusen, 1998).

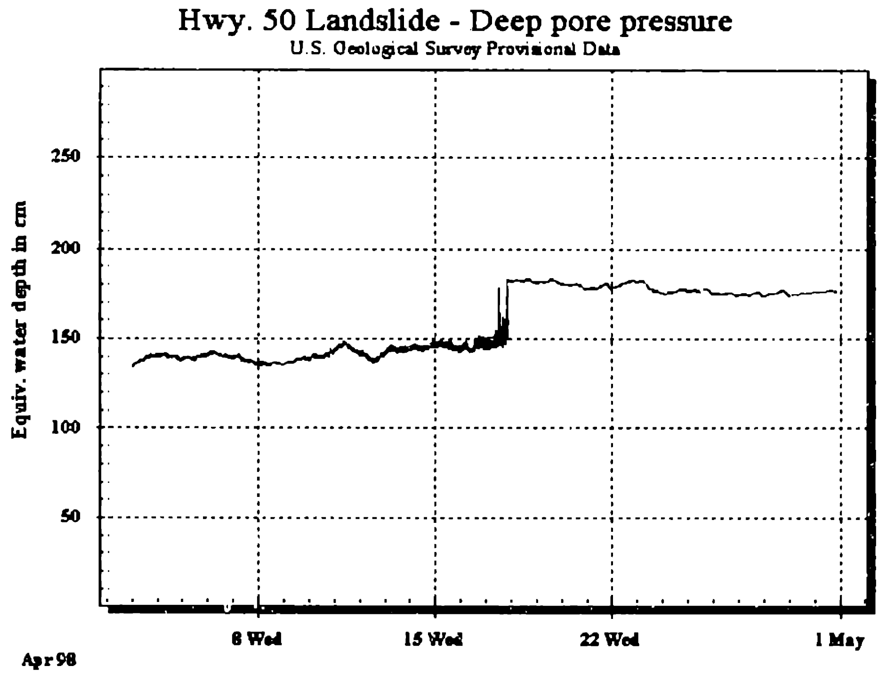


Figure 3.18 – Pore pressure data developed by USGS system (Reid & LaHusen, 1998).

Two types of embedment earth pressure cell are available, 1) diaphragm cells and 2) hydraulic cells. Both designs incorporate a flat plate or disc that is placed in direct contact with the soil. Diaphragm cells have a hollow interior that contains strain gages on one or both interior faces. The instrumented face is considered the "active" face in an earth pressure cell. As pressure changes on the contact face, the face will deflect. This deflection can be measured by the strain gages. Hydraulic designs contain a de-aired fluid, either water, oil or glycol, in a reservoir between the faces. Pressure changes on the face induce pressure changes in the reservoir fluid. The reservoir is connected to a pressure transducer that converts the change in fluid pressure to an electrical signal that may be read by a data acquisition unit. If there is one active face in a hydraulic cell, it is usually thinner than the non-active face.

Figure 3.19 is an example of an earth pressure cell.

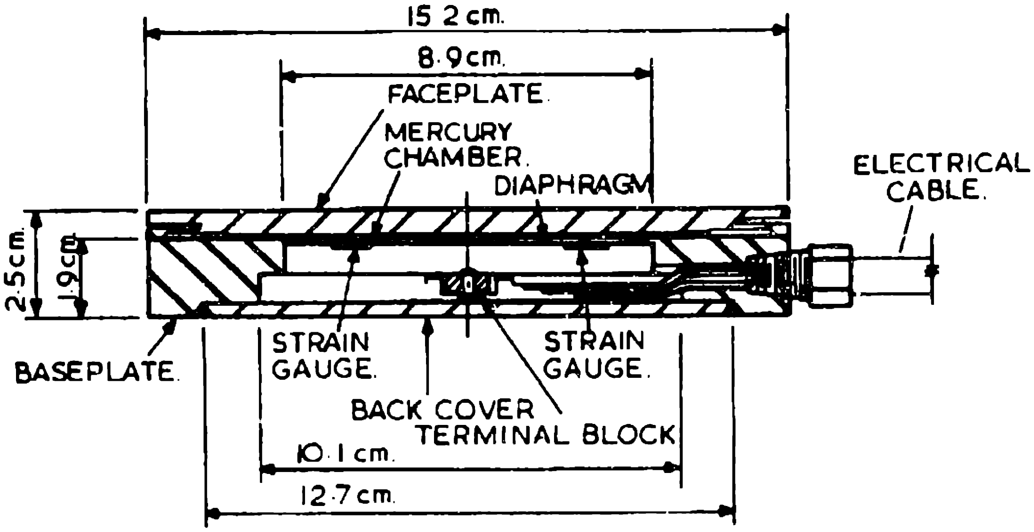


Figure 3.19 – WES soil pressure cell (Hanna, 1985).

Contact earth pressure cells are similar in design to embedment cells. However, it is more likely that cells with two active faces will be employed, as the outer face of a contact cell will be subjected to uneven loading. Generally, the data from the inner face are considered to be the most representative of the interaction between the soil and the structure. Contact cells do not suffer from the installation challenges of embedment cells.

As mentioned previously, embedment type devices are placed in the earth while contact type devices are placed at the interface of the foundation and the soil. Data from embedment earth pressure cells are often considered with caution as the installation process disturbs the surrounding soil and changes the insitu properties in the area immediately surrounding the cell. Additionally, it is difficult to determine if the soil has made complete contact with the face of the cell. Further uncertainty occurs due to arching of soil stresses that is expected around rigid objects, such as pressure transducers, embedded in soil. Embedment pressure cells with two active faces have the advantage of generating data from both faces that may be compared. If the data are similar, this increases the confidence in the quality of the installation.

Based on all of the challenges outlined in this section, it is difficult to generalize about the precision of these instruments. Pagani (1998) reports a measurement capacity of 100 kPa to 50 MPa for their vibrating wire and electrical resistance gage earth pressure cells.

As with other instruments that generate an electric signal, the earth pressure cell lends itself to an automated data collection system. More detail on methods of collecting these data will be presented in Section 3.3.

Advantages & Disadvantages

In addition to problems with disturbing the state of stress during installation, the embedment pressure cell must be carefully chosen to limit errors in measurement. For instance, the cell must be large enough so that it measures a representative average state of stress in the soil, regardless of any local non-uniformities adjacent to the cell. Furthermore, the stiffness of the cell must be properly chosen so that it accurately responds to changes in the stress in the soil. As an example, a thin walled cell will generate erroneous results in a stiff soil. As another example, a rock field containing cobbles larger than the cell itself would not be an ideal location to use an earth pressure cell since measurements would only reflect the state of stress at the contact point with one particular particle.

Contact earth pressure cells are sensitive to temperature effects. In addition, when installed in concrete, the liquid in the cell will expand when the concrete cures. For this reason, the volume of fluid in hydraulic contact earth pressure cells should be minimized.

In any case, data from earth pressure cells should be compared to other geotechnical data from devices mentioned throughout this report in order to validate the measurements.

Interesting Applications

An interesting application of earth pressure cells has been discussed by Lings, et al. (1991). This paper examined the construction of a hotel and underground garage in the stiff fissured clay of Cambridge, England. The building was constructed in a 10 meter excavation using the top-down method. Extensive instrumentation was used to monitor the performance of the excavation.

A total of seven 150 mm x 250 mm mercury filled Glötzl earth pressure cells were placed along the depth of a diaphragm wall. Four of the cells were installed between the wall and the exterior of the excavation and three of the cells between the embedded section and the interior of the excavation. Emplacement consisted of jacking the cells against the reinforcing cage prior to pouring concrete into the slurry wall. The authors concluded that these instruments are capable of measuring a reduction in lateral earth stress that occurs with the installation of diaphragm walls. According to their analysis, 6 of the 7 devices measured within 30 kPa of the expected pressures computed from the measured prop loads (Lings, et al., 1991). Figure 3.20 is a plot of the lateral stresses measured by the earth pressure cells. The top-down construction method consisted of several stages. Stage IV in Figure 3.20 corresponds to pouring the slab for the top floor of the garage. Stage XII was the final excavation to Level 1 in the garage, at a depth of 10 m.

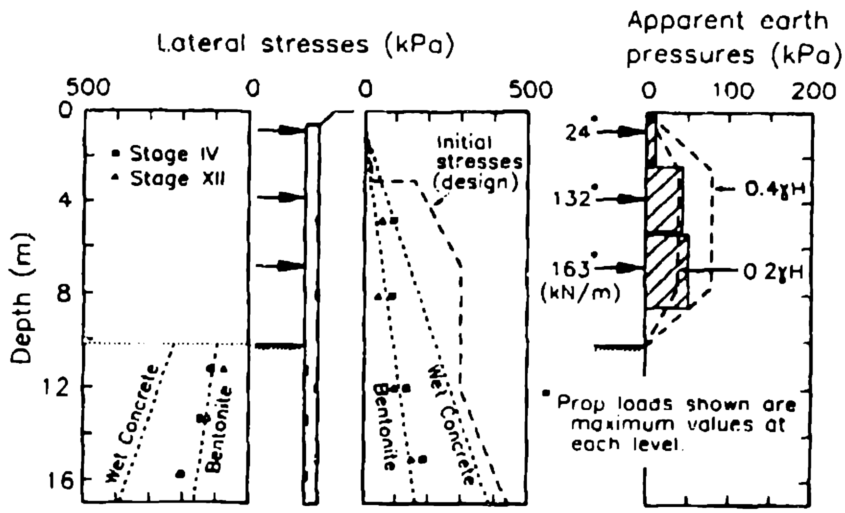


Figure 3.20 – Lateral stresses on an instrumented diaphragm wall (Lings, et al., 1991).

3.2 Instrumentation on the Structure

3.2.1 Leveling Stations

As one of the oldest systems for measuring structural movement, leveling stations and benchmarks are essential components of surveying. Combined with precision scales and sighting equipment, leveling can provide highly accurate measurements that not only provide directly useful data, but that have also been used to test the validity of data from electronic measuring devices. Surveying methods can also be used to provide a reference datum for geotechnical data generated by other measurement methods.

Leveling stations are generally installed in the foundation of the structure in question. They may be installed during construction or in an existing building by grouting in place. The station consists of a threaded plug that is protected by a plastic bung when not in use. During leveling, the bung is removed, and a brass or stainless steel machined pin is screwed into the threads. The pin generally has a rounded head, upon which the leveling instrument is placed.

Benchmarks or monuments are usually used as reference points at some distance from the structure. These are stable points that may be used for datum, and provide a base for an engineer's level, a laser level, an electronic distance measuring device (EDM) or a theolodite. Measurements are taken from benchmarks to leveling stations, and through the use of triangulation and a well-plumbed reference rod, horizontal and vertical movements of a structure relative to the benchmark may be computed.

Many types of surveying equipment are available to make settlement measurements on structures. Steel tape may be used to directly measure distances between stable benchmarks and leveling pins. Where more accurate measurements may be required, a theolodite may be used to site the reference rod from the benchmark. Similarly, automatic levels (engineers' levels) and laser levels may be used to mark the reference rod. For even greater accuracy and quicker measurements, the EDM has become a common surveying device. Steel tape is still used for distances up to 60 meters - and in special cases even further. At this length, accuracy can range from ±0.2 mm to ±10 mm depending on the level of care taken by the surveyor. (Hanna, 1985; Dunnicliff, 1988) A theolodite is precise to 1 arc-second if multiple sightings are performed - up to 8 positions. Automatic levels are accurate to ±0.03 seconds of arc (±0.05 mm in 30 m). Lasers are used up to 300 meters, with accuracy of ±3 mm over this distance. (Dunnicliff, 1988) EDM has a range of up to 5 km, with precision to ±5 mm plus 5 parts per million of the distance measured, the latter attributed to random error introduced by the electronics in the system. (Hanna, 1955)

All of these methods rely on a competent surveying crew. While some of the electronic measuring devices require a smaller crew than in the past, there is still no practical method for completely automating the leveling process.

Advantages & Disadvantages

Generally, electronic surveying methods have more advantages over the theolodite and steel tape. The tape is the least accurate method for measurement over large distances. It requires a clear, relatively flat line of sight between the measurement points. The theolodite requires multiple sightings for accurate measurements and can take up to ½ hour per measurement. Readings are usually done at night with the theolodite when the heat of the day will not refract sight lines. Lasers, automatic levels and EDM are all affected by atmospheric disturbance. Lasers in particular are deflected by air turbulence. EDM measurements are dependent on the density of the air through which they travel. Therefore, compensation must be made for temperature, humidity and barometric pressure. Fortunately, modern EDM devices include onboard sensors and processors to correct for these factors. While using a theolodite might take more than 30 minutes to make a single measurement, EDM can take 5 minutes for the same measurement (Dunnicliff, 1988).

Interesting Applications

Clayton, et al. (1991) report that precision leveling was used as part of an overall construction scheme that was based on a modified version of Peck's observational method. In central London, the redevelopment of the Grand Buildings, over the Jubilee Line (Figure 3.21), was a major concern for engineers. Demolition on the site would unload the soil above the subway line, subjecting it to potential heave that was limited by the owners to 15 mm. Geotechnical engineers had minimal data on the soil properties at the site.

The approach was to assume conservative linear elastic soil properties and to calculate expected heave. Precision leveling was used to measure actual ground deformations. Measurements were compared to the expected values, and construction adjusted according to the discrepancies.

Figure 3.22 shows a comparison of the measured and predicted heaves at three stages during construction.

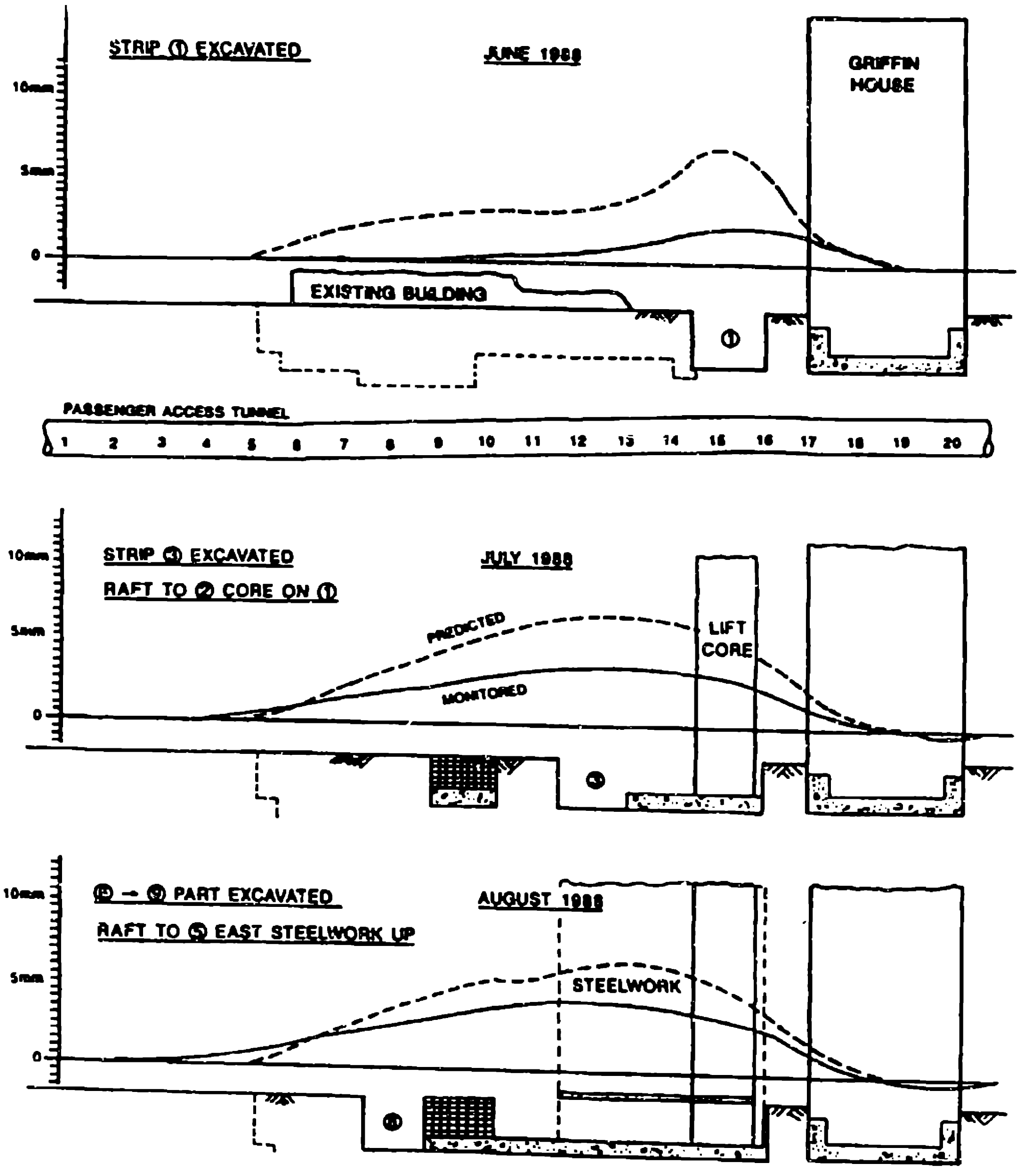


Figure 3.22 – Comparison of predicted and measured heaves during demolition, excavation and construction of the Grand Building (Clayton, et al., 1991).

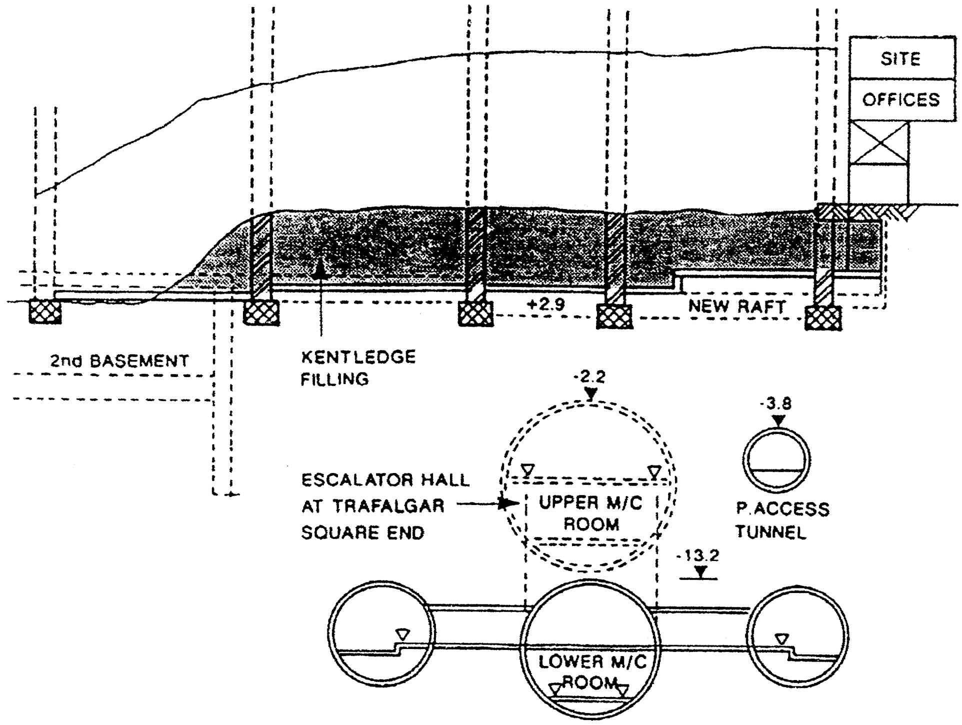


Figure 3.21 – North-South section of construction over the Jubilee Line (Clayton, et al., 1991).

Repeatability of these measurements was 0.2 mm, and the data allowed less conservative construction practices to be used. After completion of the project, the authors recommended stiffer soil parameters be used in the future to describe the behavior of this fissured clay.

3.2.2 Pendulums

A pendulum, also known as a *plumb line*, consists of a weight suspended on a long line. This line is attached to a high point in the structure and a measurement scale is placed along the line. In some systems, the weight is suspended in a fluid system to damp movements. The weight also keeps tension on the line. Inverted systems place the lower part of the line in a fixed medium while the upper part is mobile (Figure 3.23).

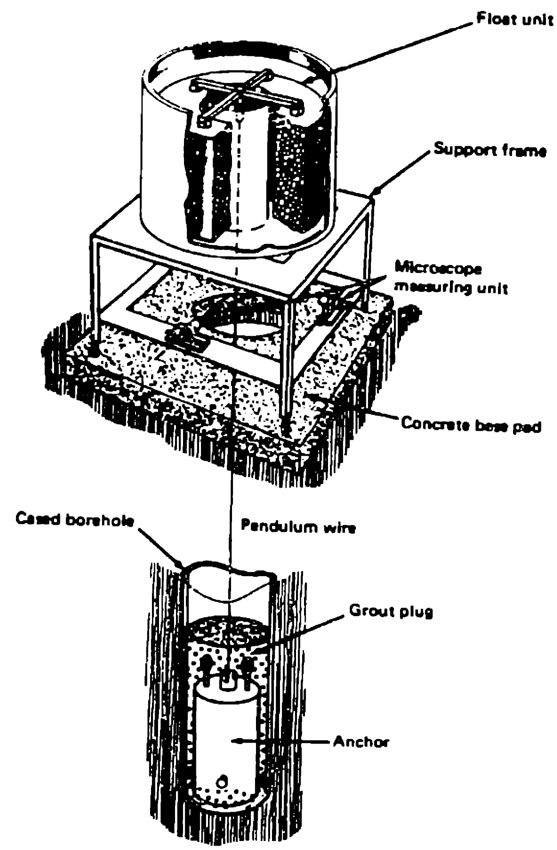


Figure 3.23 – Inverted pendulum from Soil Instruments Ltd. (Hanna, 1985).

These measurement devices are simple to use. As the structure or ground moves, gravity forces acting on the weight will move the line relative to its initial position. The scale, fixed in place, is used to read the location of the line on a regular basis. Plumb lines can produce highly accurate measurements. If a T-square scale is used to measure the location of the line, accuracy to ±0.5mm can be obtained. A vernier microscope can provide even more accurate measurements to ±0.03mm.

Optical readout systems allow for automated data collection of pendulum locations. In one commercially available system, light emitting diodes are used to sense the shadow of the plumb line. These images are projected onto a linear photodiode array that is scanned by a microprocessor (Dunnicliff 1988).

Advantages & Disadvantages

Pendulums are highly accurate and stable over the long term. However, measurements are taken at only a few points along the line, therefore a discontinuous profile of movement is generated (Hanna, 1985). Another limitation is that pendulums require a nearly vertical duct or borehole to accommodate the line.

Interesting Applications

The use of a pendulum for monitoring the movement of the Tower of Pisa is described by Burland (1995) as part of redundant instrumentation scheme. The first system used a plumb line that was installed in the top of the tower in 1934. Originally read by microscope, an update to this system employed television cameras for remote data collection. To automate the data collection process, modern devices known as a telecoordinometer (see Figure 3.24) was installed at three heights in the tower. A new plumb line passes thorough these devices. To take measurements, photocells in the telecoordinometer move back and forth on a carriage in the measurement area of the device. As the photocells pass the plumb line, the light is blocked, and the position of the plumb line recorded. Measurements are taken at intervals throughout the day, thus tracking the movement of the structure.

3.2.3 Accelerometers

Accelerometers are devices used to measure the dynamic behavior of a structure. They are frequently employed to measure earthquake response or to measure the reactions of offshore structures under wave loading. Accelerometers are designed to measure linear (one direction), biaxial, or angular (Figure 3.25) acceleration. They consist of a suspended mass that is sensitive to changes in gravitational forces. As the mass tries to move, its motion is resisted by a servo motor. The torque required by the motor to hold the mass in balance is then measured and converted into an acceleration.

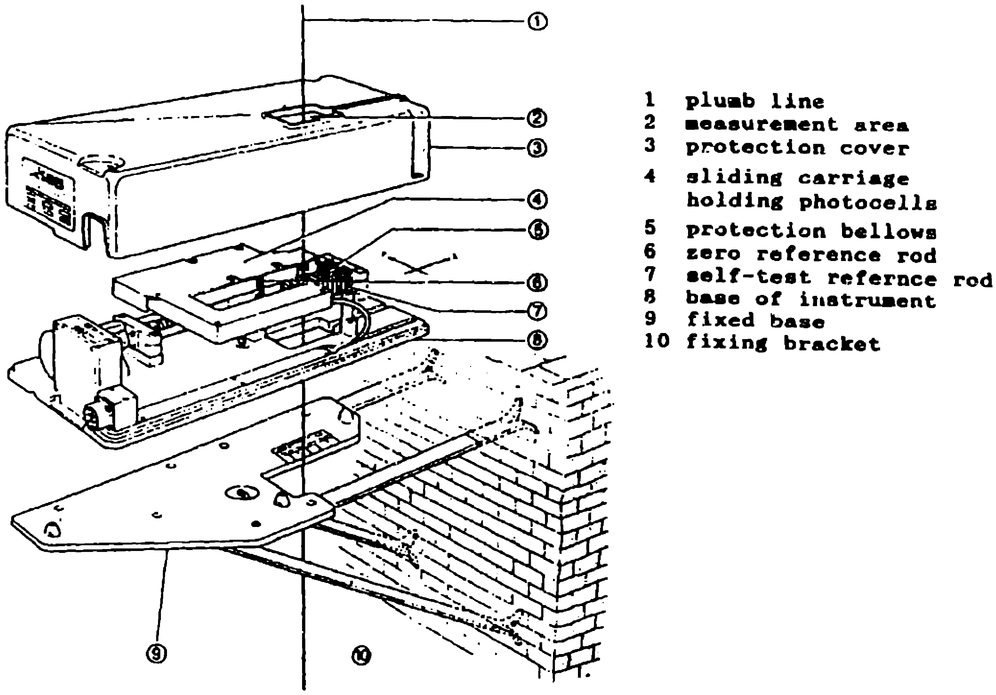


Figure 3.24 – Details of the telecoordinometer (Burland, 1995).

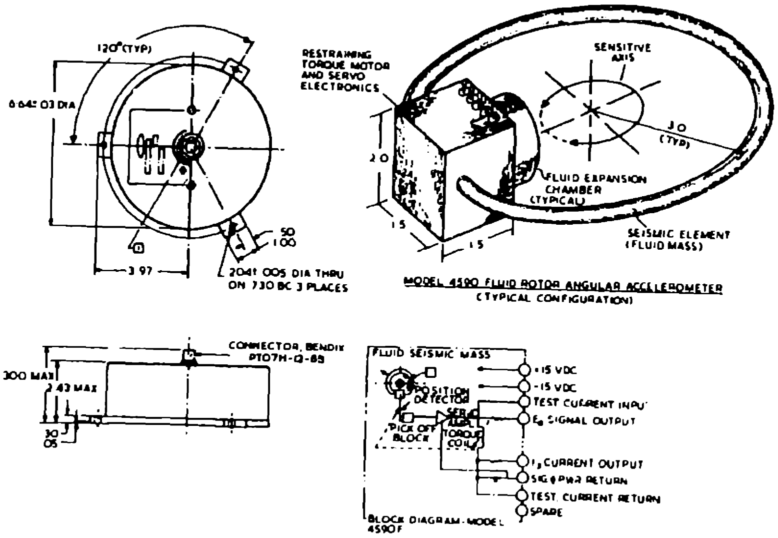


Figure 3.25 – Angular accelerometer (DiBiaglo, et al., 1981).

These devices are the main components of inclinometers (probe and fixed-in-place) and tiltmeters. They must be positioned properly, as they have a specific sensitive axis. Angular accelerometers can measure to ±0.1 rad/sec^2, and have a typical resolution of 0.001% of their range. Linear and biaxial accelerometers can measure changes as small as 0.000001g and have a range of 2 to 20 g (Hanna, 1985).

While accelerometers, like all electrical devices, may be read with a simple DC voltmeter, it would be more practical to incorporate their output into an automated data collection system. Since many of these gages are part of a larger instrument package, data collection will be a function of the system designed for the parent device.

Interesting Applications

One particularly interesting application of accelerometer technology is to enhance the data quality from global positioning satellite (GPS) monitoring systems. GPS systems in particular are discussed in Section 3.2.8. Accelerometers have been shown to reduce multipath errors in GPS readings, while GPS can correct for accelerometer drift. Thus, the two technologies work well in tandem to provide deformation data that is much more precise than would be available using one technology alone (Duff & Nelson, 1997).

3.2.4 Strain Gages

Strain gages may be designed for surface mounting onto structural members, or for embedment within them. Two different types of strain gage are available, 1) electrical resistance and 2) the vibrating wire. Electrical resistance strain gages may be weldable or bonded type units - descriptions that refer to the method of installation. These gages consist of a filament or etched foil (Figure 3.26) gage and a mounting flange. The vibrating wire gage consists of a wire stretched between two posts, a pretensioning device, a magnetic coil for "plucking" the wire, a quartz oscillator (transducer) to measure the frequency of the wire, and a temperature correction assembly (thermistor). Typical gage lengths for structural applications ranges from 2 to 10 inches. Small strain gages (sizes down to 0.125 in and smaller) are used to measure deflections within instruments like earth pressure cells (Section 3.1.6). Federal Highway Administration guidelines (1996) recommend a strain gage lengths of 0.25 in. for concrete embedment applications.

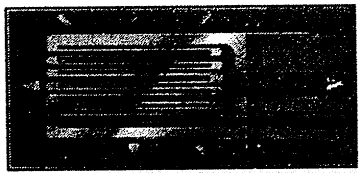


Figure 3.26 – Micro Measurements etched foil strain gage.

Strain gages are either bonded to the exterior of structural members or cast within them - for instance on rebar in a concrete beam. Figure 3.27 is an example of a weldable strain gage. When the element undergoes strain, the natural frequency of the vibrating wire, or the electrical resistance of the filament changes. By measuring the new properties of the gage, strain may be calculated within the element. Measurement of strain is of interest because increased strain directly leads to cosmetic and structural defects such as crack development.

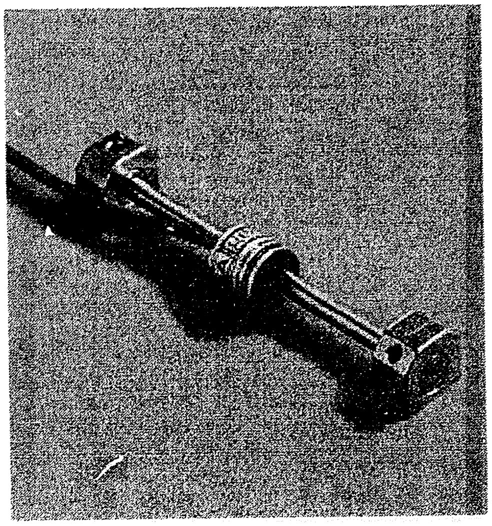


Figure 3.27 – Geokon VSM 4000 vibrating wire strain gage (Geokon, 1998).

Precision is dependent on installation and environment. High quality installation can lead to accuracy of ±1 microstrain, with a range of 3000 microstrains for vibrating wire type gages - a rage equivalent to a change in stress of 90,000 psi in steel (Dunnicliff, 1988). Typical accuracy is ±5 microstrains for bonded type gages, although accuracy ranges from 1 to 100 microstrains for all electrical resistance type gages. The range of electrical resistance strain gages is 20,000 microstrains.

As mentioned previously, environment is an important factor in gage performance. The gages must be waterproof and can be sensitive to temperature extremes.

In general, strain gages are hard wired to a control/data acquisition system. The only types of gages that may do require automation of data acquisition systems are mechanical type strain gages with dial indicators, devices that are available but have not been discussed in this section*.

Advantages & Disadvantages

There is no other type of instrumentation for generating the type of data measured by a strain gage. Hence, there are no disadvantages to this family of instruments as a whole. However, there are certain advantages and disadvantages to the different types of strain gages. For instance, electrical resistance gages have a much larger range of measurement then vibrating wire type gages. Vibrating wire type gages must be initially set with the proper wire tension to measure compressive or tensile strains over the range of interest. Long term creep of the tensioned wire can change the natural frequency of the gage, a phenomenon known as *zero drift*. Additionally, they cannot be used to measure high frequency dynamic strains. The advantage of this type of instrument over electrical resistance type gages is that the data generated is a frequency, which may be transmitted over much greater distances than a resistance reading. Electrical resistance gages are sensitive to temperature, moisture, the quality of installation, and have very low electrical output. Also, the lead wires can affect the measured resistance of the gage.

Interesting Applications

Strain gages are used extensively in structural monitoring applications. They may be bonded directly to steel elements to measure loading, or can be incorporated in load sensing bolts (Geokon, 1998) and in concrete reinforcing steel. In the latter application, specialized instrumentation must be employed that can withstand the rigors of immersion in curing concrete. Carlson/RST is one company that produces robust strain gages for long term strain monitoring in concrete (see Figure 3.28 for a photograph of this installation). Choosing an embedded gage requires advanced planning so that it can be properly placed before

* Mechanical strain gages are still used in construction applications as back-ups to automated systems.

concrete is poured. Furthermore, failed gages embedded in concrete cannot be replaced without extensive effort. In spite of these limitations, strain gages show promise in a variety of measuring environments.

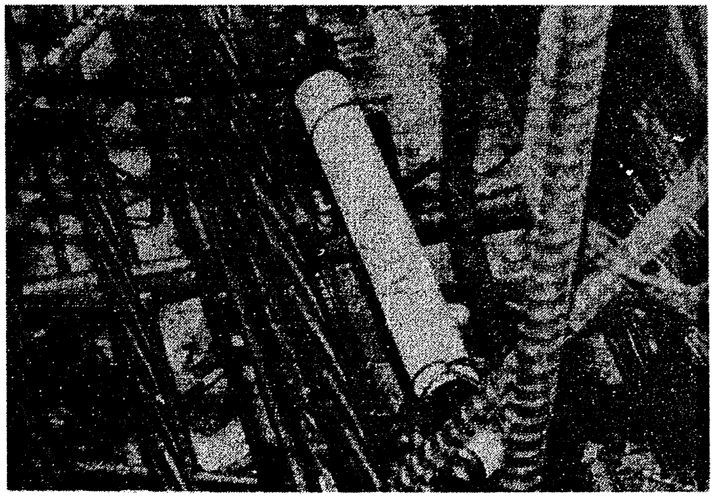


Figure 3.28 – Carlson/RST strain gage prior to embedment in concrete for bridge deck monitoring (Federal Highway Administration, 1996).

3.2.5 Tiltmeters

Although tiltmeters are considered under the heading of "instrumentation on the structure", they are equally useful when placed in the ground to monitor slope stability and deformation. Initially designed as elements of servo guidance systems for rockets and missiles, tiltmeters have recently seen increasing use in geotechnical and structural applications.

One tiltmeter design (Figure 3.29) works on the relatively simple concept of the bubble level. Electrodes are placed at various points in an electrolytic fluid that is encased in a tube. The fluid has an air bubble that moves when the meter is subjected to tiny displacements. This air bubble changes the conductivity of the fluid system and thus changes the resistance that can be measured across the electrodes. These small differences in voltage are then translated into tilt measurements (Egan and Holzhausen, 1991). Other tiltmeter designs use internal accelerometers (Section3.2.3) to make measurements of tilt. Uniaxial and biaxial devices are available, the latter having the capability to measure tilt simultaneously on two perpendicular axes.

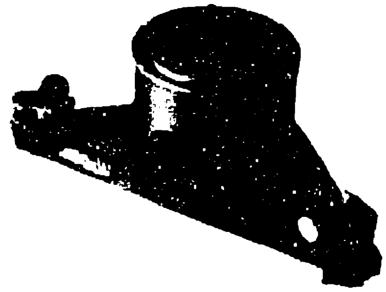


Figure 3.29 -- Applied Geomechanics 500 series tiltmeter.

Tiltmeters have seen use in many geotechnical applications. When placed in underground vaults, they have been used to monitor ground movements. Buckley, et al. (1998) report that a network of tiltmeters was deployed as early as 1973 for research on an earthquake early warning system. More recently, they have been used in attempts to determine the type of soil behavior responsible for ground deformation, and the rate at which this deformation is occurring (Tofani & Horath, 1990). In this application, the tiltmeter is placed on a ceramic plate underlain by silica sand and undisturbed fill in a vault underground (Figure 3.30). The vault has an insulated, water-tight cover and a PVC tube to permit hard-wiring to the data acquisition system. The ceramic plate provides a thermally stable base for the tiltmeter, and the sand allows for sufficient leveling of the base within the tolerance specified by the manufacturer.

Tiltmeters may also be directly connected to structural elements in a building. So used, they can measure deflections and allow for continuous monitoring of the behavior of the structure under loading. Tiltmeters can be used to measure bridge deflections and have been employed to help prioritize structural repairs. In one case, a series of tiltmeters was deployed to continuously monitor the long-term heave of a foundation under a nuclear power plant in Catalonia, Spain (AGI, 1997).

Accuracy is one of the strengths of the tiltmeter. The product literature lists typical values of 0.006 micro radians for precision. For perspective, one micro radian is the angular equivalent of 1 inch of tilt in 16 miles (Buckley, et al., 1998). These instruments have a range from 1250 to 12,500 micro radians when coupled with automated data acquisition systems (Tofani & Horath, 1990). When combined with inclinometers, these instruments offer complementary capabilities for a comprehensive ground or structural deformation measurement program.

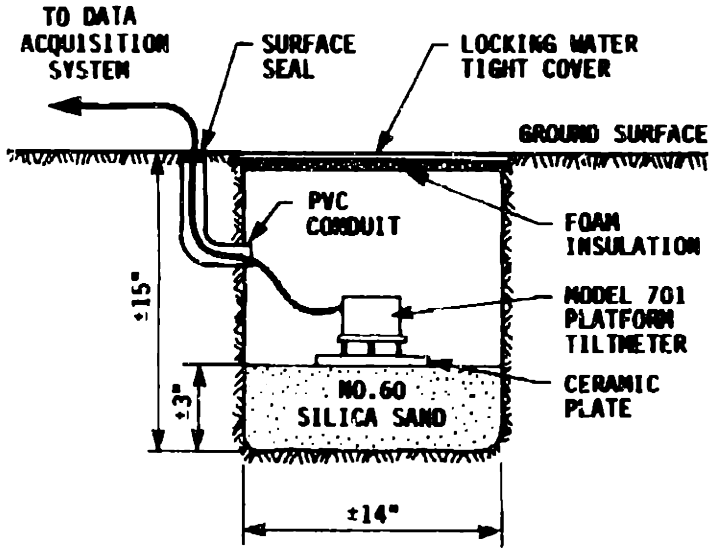


Figure 3.30 – In-ground tiltmeter setup (Tofani & Horath, 1990).

Data from tiltmeters are compatible with automated collection systems. Due to their precision, these units can generate vast amounts of data in a short period of time, and thus require some sort of automated storage system when they are employed for continuous monitoring. A description of data collection systems may be found in Section 3.3.

Advantages & Disadvantages

Tiltmeters have amazing precision. They have the capability of real-time monitoring and early-warning of structural movement and damage. Additionally, they offer very quick resolution of data. However, they are best employed in groups for long-term monitoring schemes. As part of a regular maintenance program, some of these instruments will need to be switched out as equipment failures occur. Also, due to their precision, tiltmeters are sensitive to environmental factors such as temperature fluctuations and water infiltration. Vehicular traffic and regular seismic movements unrelated to ground deformation can introduce noise into the data. And the precision and quick resolution generate large amounts of data.

Interesting Applications

Burland (1995) reports on the use of tiltmeters (referred to as electrolevels or electrolytic inclinometers in his report) for monitoring the foundation movements of the Tower of Pisa (Figure 3.31). As one element of a measurement scheme, 4 biaxial tiltmeters were installed around the base of the tower (Figure 3.32).

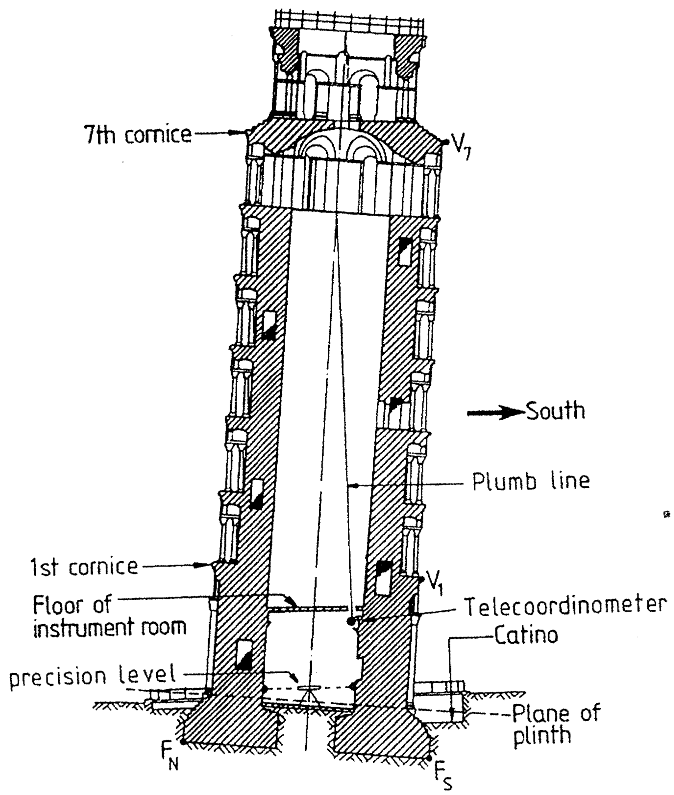


Figure 3.31 – Cross section through Tower of Pisa (Burland, 1995).

The advantage of this program, which included precision leveling, was that performance of the instruments could be compared to independent methods of measuring settlement. Results showed that only two of the four tiltmeters performed adequately. One of the other instruments showed significant drift throughout the program, and another showed excessive temperature dependence (Figure 3.33 and Figure 3.34).

Interestingly, these instruments were the first devices that were precise enough to measure the tilt in the tower induced by daily temperature variations. Burland concludes that tiltmeters are useful for real-time measurements of short term foundation movements.

However, in temperature sensitive applications and for long-term monitoring, redundancy with other types of devices is necessary to ensure accurate data collection.

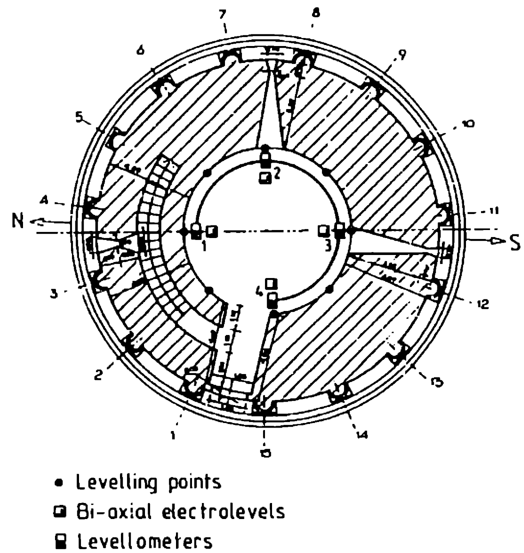


Figure 3.32 – Plan section of the entrance chamber of the Tower of Pisa showing the location of instruments (Burland, 1995).

3.2.6 Load Cells

There are several types of load cells available on the market. Major types include hydraulic, mechanical, electrical resistance, vibrating wire and optical load cells. Mechanical designs directly measure stain using a dial indicator. Electrical resistance and vibrating wire types are essentially strain gages mounted on the periphery of a steel or aluminum cylinder (Figure 3.35). Load may be calculated knowing the properties of the cylinder and measuring the strain. Some of these devices may not be well waterproofed and as such are not suitable for geotechnical applications. For other considerations of strain gage design, see Section 3.2.4

Optical load cells are not widely used outside of Europe. Basically consisting of a glass disk in a steel cylinder, this type of load cell is read by viewing the glass disk while passing polarized light through it. Under load, the shape of the disk changes, and the resulting light pattern also changes (Hanna, 1985). Because they are not widely used, they are not considered here in detail.

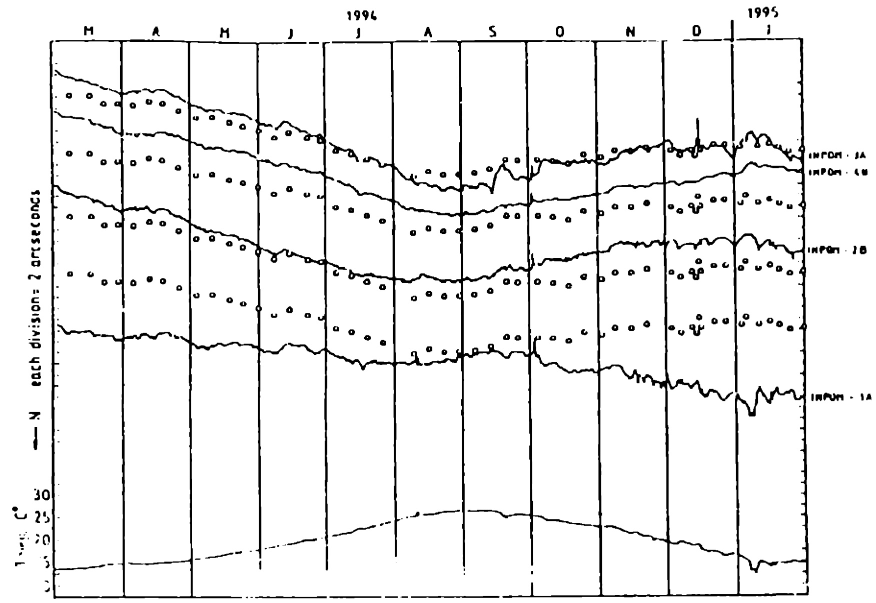


Figure 3.33 – Comparison between electrolevels and precision leveling for north-south rotations from March 1994 to January 1995 (Burland, 1995).

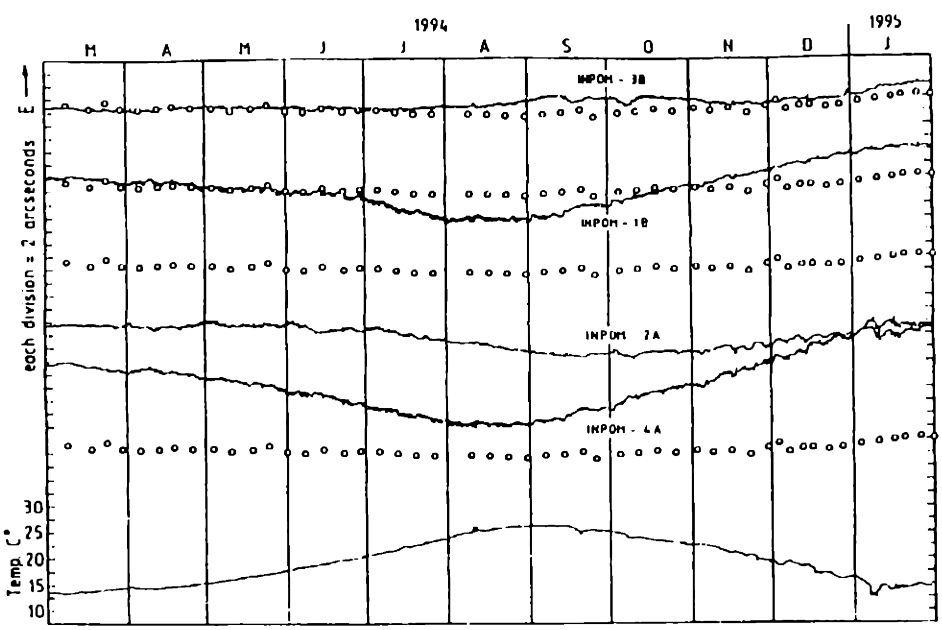


Figure 3.34 – Comparison between electrolevels and precision leveling for east-west rotations from March 1994 to January 1995 (Burland, 1995).

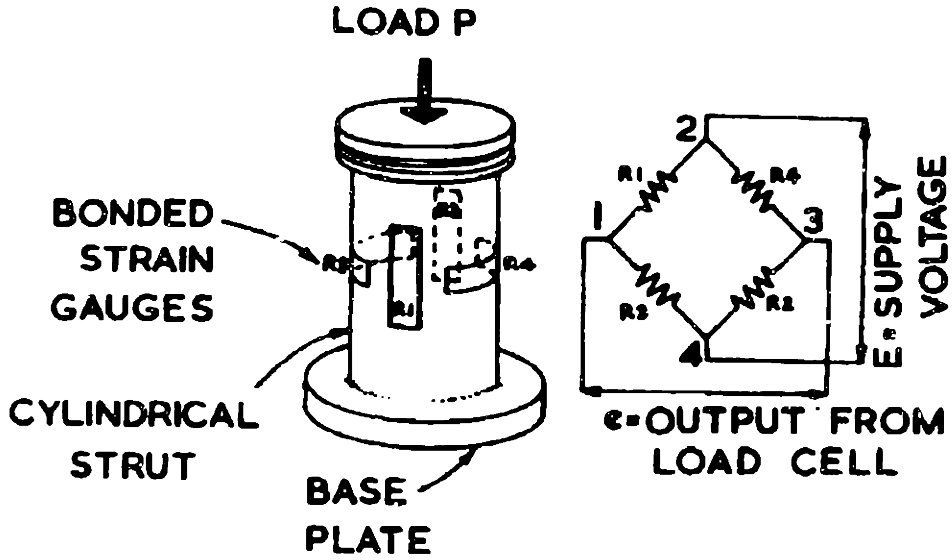


Figure 3.35 – Cylindrical load cell and circuit design (Hanna, 1985).

Hydraulic load cells consist of a hydraulic reservoir connected to a pressure transducer and bounded by steel plates. As load is applied to the plates, the hydraulic fluid is placed under pressure that is directly read by the transducer and converted into load. (Dunnicliff, 1988)

Load cells are frequently designed with a hollow core (Figure 3.36). This allows structural elements like tiebacks to be run through the cell so that a direct measurement of lock-off load may be made. Other cells may be placed between a strut and bearing surface to measure the strut load. When properly mounted, most load cells have an accuracy ranging from ±2 to 10% of their range. A typical load cell installation for a tieback anchor is shown Figure 3.37.

Load cells with pressure transducers and strain gages are compatible with automated data collection systems. Mechanical load cells may be connected to a digital output system instead of a dial indicator, and as such are also compatible with automated data collection. Optical load cells, on the other hand, require an operator to directly read the polarized light through the gage and compare this reading with recognized patterns for determination of load (Hanna, 1985).

Figure 3.36 – RST Load cell.

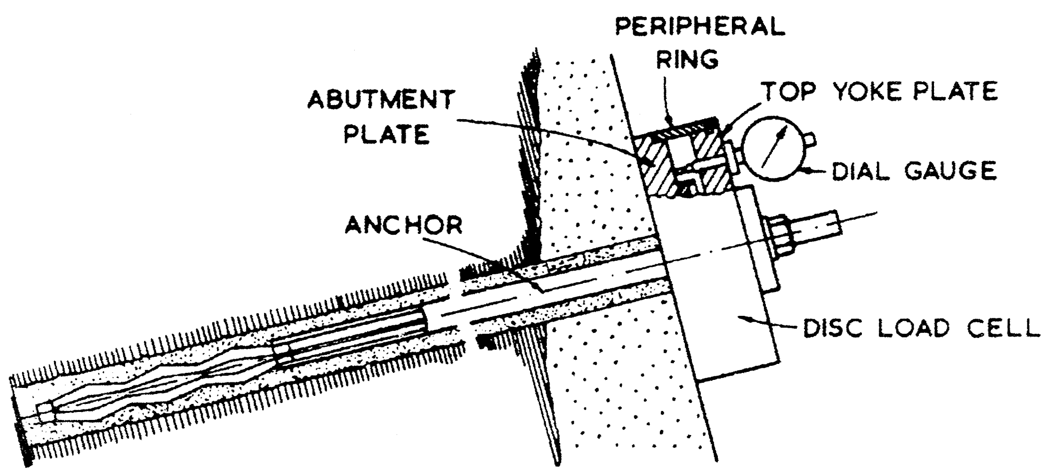


Figure 3.37 - Typical load cell installation for a tieback anchor (Hanna, 1985).

Advantages & Disadvantages

Hydraulic load cells may be less accurate than electrical resistance and vibrating wire type gages. They require rigid load bearing surfaces. On the other hand, hydraulic cells offer robust designs, low profiles and remote readout possibilities. Similarly, strain gage type units offer remote readouts and automated data collection - however they have the same limitations as mentioned for strain gages in Section 3.2.4.

Interesting Applications

Load cells are particularly suited to measuring the forces in tie-downs. A recent transportation project in the Middle East required 470 anchors to secure a concrete slab that is part of a 23 km six-lane highway. The slab passes under an overpass, and was built in an excavation 6 to 7 meters below the ground surface (Figure 3.38). Uplift pressures caused by the high water table are compensated by the tie-downs. Hydraulic load cells were attached to 20 of these anchors. As the water table was elevated subsequent to construction, the load cells provided critical monitoring of the forces in the tie-downs. These instruments performed well in this capacity and allowed engineers to adjust tie-down lock-off loads to accommodate additional pressure from raising the water table (Thogersen & Sorensen, 1991).

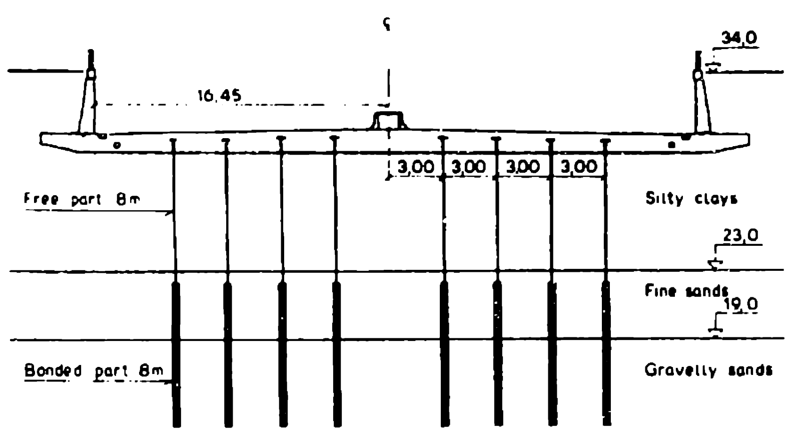


Figure 3.38 – Cross section of tied down highway section in the Middle East (Thogersen & Sorensen, 1991).

3.2.7 Time Domain Reflectometry

Time Domain Reflectometry (TDR) is a developing method that uses lengths of coaxial cable to measure deflections in the ground and on structures (Figure 3.39). Its application has been reported for both structural and geotechnical applications by Kersey (1994), Dowding and Pierce (1994a, 1994b), Dowding and Huang (1996), Dowding et al. (1996), Pierce, et al. (1994) and Kane et al. (1996). Adoption of this technology is underway in many areas worldwide.

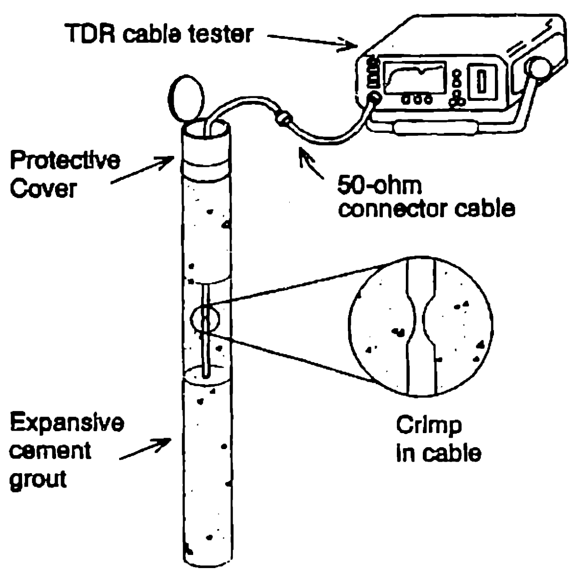


Figure 3.39 – Typical TDR setup (Dowding & Pierce, 1994b).

The concept of TDR is similar to that of radar. This technology uses pulsed signals along a length of coaxial cable to measure deformations in the cable. The signal is sent out over the cable, and the reflected signal is compared to the input. This comparison, called the *reflection coefficient*, can be used to determine the location and nature of cable faults such as kinks, or the height of water in a standpipe. The concept was originally developed to test the integrity of electrical transmission cables (Kane, et al., 1996).

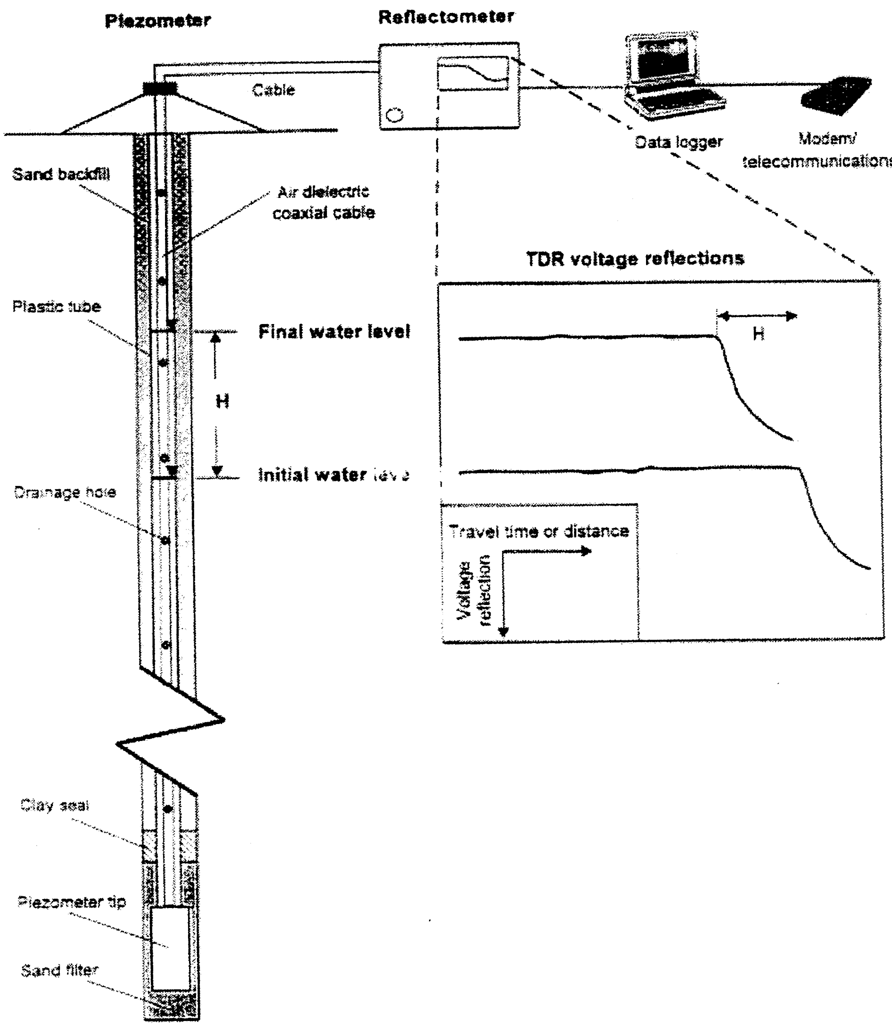


Figure 3.40 – TDR monitoring of piezometric water elevations (Dowding, et al., 1996).

The design of the coaxial cable can be important. The theory behind TDR relies on a change in cable impedance to alter the nature of the reflected signal. Coaxial cables work well in this application because they consist of a central conductor surrounded by an insulator, which is surrounded by another conductor. This design gives each cable a characteristic impedance that is a function of the insulating material. As the insulator is deformed by a cable fault, the characteristic impedance of the cable changes in that area.

The design of the insulator is important, for reasons that will be discussed later. Most standard coaxial cables use solid polyethylene, however some designs use air, and others use polystyrene foam.

Research is ongoing on the use of TDR for measuring piezometric water elevations, and for monitoring slope and structural movements. In piezometric applications, air is used as the insulator, and the outer jacket of the cable is perforated with a series of holes to allow the free movement of water into the insulating layer. A pulsed signal is sent down the cable and when it enters the water, the voltage of the reflected signal drops (Figure 3.40). The length of this signal drop is measured and can be converted to height of water, knowing the speed of the pulse in the cable. Thus, the total head can be easily measured with the pulsed signal. TDR cables can be easily retrofitted into existing standpipes (Dowding, et al., 1996).

In structural applications, researchers have embedded the cables in concrete slabs (Huston, et al., 1994), and have studied the measurement of bridge scour. In the latter application, the cable was grouted in a borehole adjacent to the bridge footing. As scour caused deformation of the footing, the cable was displaced and movement was measured (Dowding & Pierce, 1994b).

TDR has also been used to monitor the deformation of rock slopes. In these applications, the stiffer polyethylene insulator adequately measures rock deflection because the forces required to mobilize the rock are greater than those required to deform the plastic. However, for soil measurements, softer polystyrene should be used. Polyethylene is stiffer than soil – thus there could be a soil slope failure that does not kink the cable.

Dowding and Pierce (1994a) report that deformation shear bands as small as 2 mm in width can be measured with the TDR system (Figure 3.41). In this experiment, the reflection coefficient for different lengths of cable was measured while the cable was sheared. As the size of the shear zone grew, the magnitude of the reflection coefficient grew. When shear zones reached 2 mm, the change in reflection coefficient was measurable. Figure 3.41 also shows that shorter cable lengths (data sets plotted higher on the graph) have a greater change in reflection coefficient versus shear deformation. Resolution is such that different shear bands separated by only 6 mm can be detected for short cable lengths, e.g. under 30 m.

Shear can be quantified in cable runs up to 268 m, and detected in lengths up to 530 m (Kane, et al., 1996).

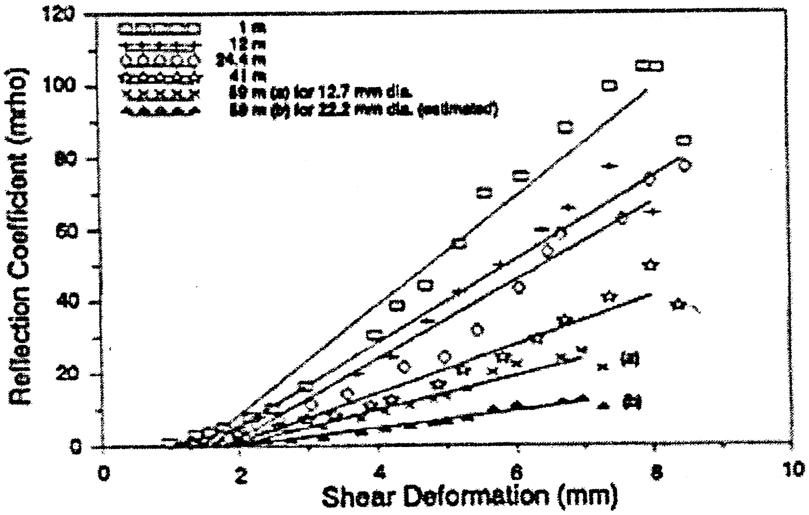


Figure 3.41 – Reflection coefficient versus shear deformation relationships for short cable lengths (Pierce, et al., 1994).

The TDR equipment conforms to the USGS SDI-12 standard mentioned in Section 3.3, which means it should be compatible with data collection equipment available from a variety of manufacturers. Automated data collection is suggested, as strip chart recorders that have been used in the past have yielded data that is difficult to evaluate.

Advantages & Disadvantages

Experience to date shows that TDR monitors are more precise than inclinometers (Dowding & Pierce, 1994a). In fact, shear bands that are 1/300 of the size detectable by traditional methods are measurable by TDR. As mentioned previously, TDR is also compatible with existing standpipe piezometers, making retrofit relatively simple. Furthermore, the cables are thin making multiple installations in a single small borehole possible. Finally, all electronics are located above ground, making maintenance more convenient (Dowding & Pierce, 1996).

However, TDR is not without its challenges. First, cable extension has been difficult to measure by this method, although careful analysis of the reflected signal can show the location of necking that is caused during extension. Secondly, short cable lengths have shown the greatest precision. When cables over 600 m in length are used, shear band detection is reduced to those that are ≥ 1 m in size. (Dowding & Pierce, 1996a) Finally, the composition of the grout used to support TDR cables in place is important. It has been shown that cables in free air, or embedded in sand do not accurately measure deflections in soft soils. Successful grout formulas have been reported in the literature by Dowding and Pierce (1994a).

Interesting Applications

Kane, et al. (1996) report several applications of TDR for geotechnical data acquisition. Caltrans has employed several systems to monitor slope stability in landslides throughout the state. In Del Norte County, a cable was installed in a 280 ft. deep borehole on a slope beneath U.S. Highway 101. A check several months later confirmed a kink in the cable at the same depth as a deflection measured by an adjacent inclinometer.

In perhaps an even more interesting application, optical TDR cables are being studied for structural deformation measurements. A single fiber optic cable, with filters built-in along its length has the capability of measuring strains distributed throughout the instrument. When connected to the proper multiplexing input source and by using filters on the output signal, the potential exists to address up to 1000 sensors within the cable. Both static and dynamic strains can be measured using optical TDR equipment (Kersey, 1994).

3.2.8 GPS Systems

Global Positioning Systems (GPS) were initially developed by the military to provide positional information during combat (Leick, 1992). Since the development of this technology, civilian GPS receivers have become available for such uses as automotive navigation. As such, the systems are highly effective, with relatively inexpensive equipment that provides adequate accuracy for this purpose. A recent development in GPS, however, is the use of the system for deformation monitoring of large civil infrastructure. While navigation by GPS requires accuracy to tens of meters, deformation monitoring requires millimeter precision or less. To achieve this precision, special methods must be employed.

GPS systems rely on a network of 24 geosynchronous satellites that are deployed in a network above the earth. These satellites transmit signals that are monitored by a receiver in the GPS unit. By triangulation on the signals of multiple satellites, the GPS unit can calculate its position in three coordinate planes (North-South, East-West, and vertical).

Accuracy of GPS units is a function of the type of signal measured from the satellites. Two frequencies, known as L1 and L2, are transmitted by the satellite network. On these signals, two sets of codes are transmitted – a P-code and a C/A code. The P-code is known as the "precise" code and can be used to measure position within ±15 m. This code is encrypted by the military and is not designated for civilian use. The C/A code, on the other hand, is the "coarse acquisition" transmittal, wherein noise and deliberate errors are introduced into the signal. This permits measurement accuracy within ±100 m (Collier, 1993).

When used to directly measure the C/A signal, GPS units are not suitable for deformation monitoring. However, there are other methods to compute position using the satellite network. In fact, using a network of receivers to measure phase differences in the carrier signals allows accuracy to the millimeter scale.

Two methods of millimeter scale position measurement with GPS are documented by Collier (1993). The first, known as static carrier phase positioning, requires that the receivers be left in one position for several hours to accurately measure the carrier signal. The second method, kinematic carrier phase positioning, is a more efficient, albeit slightly less accurate method for position measurement. In the latter case, one GPS unit is designated as the base, and remains in the same known place throughout the testing program. Another roving unit is then used to take measurements at different points. These measurements are computationally related to the base position, and are taken for only a minute or two at each point of interest.

GPS has been shown to be more accurate in lateral positioning (North-South and East-West) than in vertical positioning (Collier, 1993). Even so, reports by Collier and Duff & Hyzak (1997) show that accuracy to ±5mm has been achieved in any coordinate plane, with more refined techniques being accurate to ±2mm. Some important factors in precision include the orientation of the GPS antennae. Collier notes that out-of-parallel antenna orientation between the two GPS units resulted in vertical measurement errors that reduced accuracy to ±10mm.

Automated data collection is essential for GPS positioning. Duff & Hyzak (1997) report that a bridge project with 10 monitoring sites and three reference sites can generate over one megabyte of data per hour of monitoring. Automation is facilitated, however, by the availability of high-speed data radios which can transmit data to a central processing unit.

Advantages & Disadvantages

GPS deformation monitoring has been shown to be a quick method to take multiple measurements that are only slightly less accurate than precision leveling. Line of sight is not required between any measurement points, and the equipment is relatively inexpensive.

However, proper antenna orientation can be critical. Furthermore, an open view of the satellite network is required which prevents GPS use underground, and limits its use in urban settings. Accuracy is reported as a percentage of baseline distance (approximately 0.4 parts per million). Thus, long baselines will tend to decrease precision in these measurements. Finally, GPS readings can be affected by tropospheric disturbances. and thus environmental factors must be accounted for during data measurement.

Interesting Applications

GPS is a relative newcomer as a structural deformation monitoring tool. Even so, interesting applications are appearing in the literature for a diverse range of uses. For instance, GPS has been used to measure the deflection of a bridges in Florida and Texas (Duff & Hyzak, 1997). In these applications, permanent reference GPS receivers are located on the bridge, and a mobile unit attached to an automobile is used to take quick measurements of deflections along the bridge. In this manner, the performance of the entire structure can be tested quickly as part of a routine maintenance program. Multiple bridges, similarly outfitted with a reference GPS receiver, may be tested in the time it takes to drive from one to the other.

Another interesting application has been reported by the mining industry. In combination with traditional leveling stations and extensometers, GPS has been used to monitor an open cut excavation in Australia (Joass, 1993). This system was placed following an initial failure at the mine site, and was able to successfully predict a more major failure, giving the owners time to apply mitigating measures that prevented a major catastrophe.

3.3 Data Collection

As has been mentioned, automated data collection is possible for many geotechnical instrumentation schemes. This section will present an overview of some typical data collection schemes and equipment. Before beginning, however, it is useful to examine some of the benefits and disadvantages of automation.

Some of the benefits of automation, compared to traditional methods of data collection are as follows:

1. Less staff time is required for recording data from the instruments.

2. Instruments that generate large amounts of data become feasible elements in the geotechnical monitoring scheme.

3. Data can be directly downloaded into a computer database allowing for easy post-processing. For example, charts of pressure vs. time may be created very quickly for a range of piezometers on a construction site.

4. Early warning systems can be easily incorporated into an automated data acquisition system. Hence, an instrument that takes a measurement exceeding a specified threshold can then alert officials about a potential problem via phone, modem or pager.

5. Data may be made readily available across a distributed network of computers, allowing multiple parties access to the information.

6. Data may be collected and transmitted nearly instantaneously from these instruments, if required.

7. Automated data collection systems are increasingly affordable when combined with low-cost personal computing power.

This automation does not come without its disadvantages, however. When designing such a system, the following should be considered as some of the potential problems:

1. Many electronic instrumentation systems need to be corrected for environmental factors such as relative humidity and temperature. Therefore, additional data must be collected, adding to the amount of information that must go through post-processing.

2. Harsh monitoring environments, including extreme temperatures and submerged operation, require robust instrument designs, particularly in long-term monitoring applications.

3. Redundant gages should be incorporated into the monitoring scheme. Automated data collection could lead to a "hands-off" approach to the instrumentation network and failures in the gages might not be immediately noticed. A certain percentage of electronic instruments should be expected to fail during their lifetime.

4. In addition to providing redundancy, the operation and maintenance plan for the instrument network should include periodic switching out of some instruments in mission critical applications. This should decrease the chance of data loss to a gage failure.

5. Electronic instruments generally require calibration* and can experience measurement anomalies such as drift. Redundancy, in addition to lowering the chance of data loss, can provide additional data for checking the accuracy of electronic gages. This was a critical component of the monitoring work done at the Tower of Pisa (Burland, 1995).

3.3.1 Preliminary Considerations

Section 3.3 will not examine in detail the portable data acquisition units commonly available. Comparing the individual merits of each product is a subjective exercise best left to the end user. It is important, however, to understand that such equipment exists.

* Most modern data acquisition equipment can automatically calibrate electronic instrumentation.

Portable data collection equipment is available from every major supplier of geotechnical instrumentation. In addition, several companies specializing only in data acquisition products, sell units that are capable of reading output signals from geotechnical gages. Modern portable equipment is capable of stimulating and reading vibrating wire type gages, and often can include the necessary equipment for gage calibration and temperature and humidity measurement. Often, these portable battery-powered units are the best choice, particularly for small instrumentation schemes.

As mentioned previously, the use of electrical gages is subject to the following considerations:

1. Electrical resistance gages are subject to lead effects such as increased resistance with increased lead length. In a strain gage installation, the resistance of the lead is unaffected by changes in strain at the gage. Therefore, long leads will make the installation less sensitive, because a larger percentage of the total resistance measured in the system does not change with changes in strain. Lead resistance is also affected by temperature.

2. Vibrating wire gages provide a frequency for their output signal, which may be transmitted over much greater distances than a resistance signal. However, these gages require an input stimulation in order to develop an output signal.

3. In general, electrical gages should be shielded from severe weather – particularly storms that generate lightening which could destroy the gages (Hanna, 1985).

3.3.2 Data Collection & Distribution

An important consideration in the development of a data acquisition system is the compatibility of the instruments and the data loggers. It is unlikely on a large job that every gage will be made by the same manufacturer. One way to ease compatibility issues is to use equipment that conforms to recognized standards. For instance, SDI-12 has been developed as a set of standards for environmental data acquisition (SDI-12 Support Group, 1998). It specifies the design of the electrical interface, the communication protocol and the timing for microprocessor based sensors and data recorders. Equipment from different manufacturers that conforms to the SDI-12 standard may be easily incorporated into the same monitoring network.

A typical data collection system consists of the elements shown in Figure 3.42. The purpose of each element is as follows.

Multiplexer

In many cases, gages may be directly connected to the recording unit. However, most recording units have a limited number of inputs – on the order of 6 to 12 for the CR10 unit offered by the Slope Indicator Company. Thus, for large arrays of instruments, multiplexers add flexibility to the system by providing more inputs and distributing the output signals as necessary. The same Slope Indicator unit can control up to 6 multiplexers, and each multiplexer can service 16 to 32 gages – an order of magnitude increase in capacity (Slope Indicator, 1998).

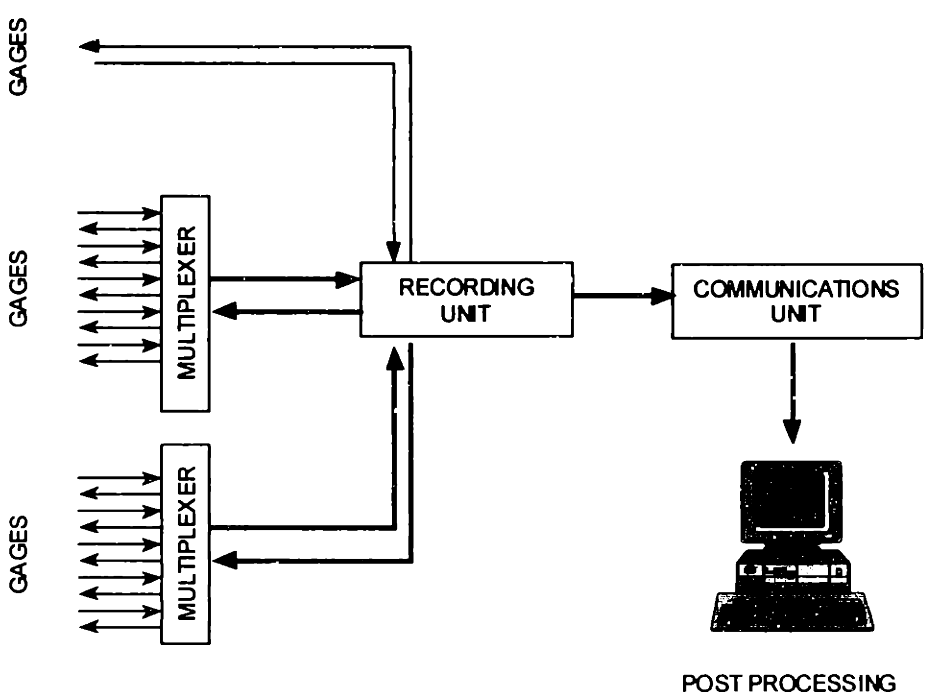


Figure 3.42 – Elements of an automated data collection system.

Recording Unit

The recording unit is the brains of the data acquisition system. This equipment is responsible for reading electrical signals and passing them to the communications unit. Often, advanced capabilities are built into these systems. For instance, the recording unit may also be capable of providing electrical stimulation to vibrating wire strain gages. Some equipment can process electrical signals for output as engineering units. Additionally, the recording unit may contain alarms to warn of gage readings beyond predetermined thresholds.

Communications Unit

The communications unit transfers information from the recorder to storage or post-processing. Many options are available for making this connection. Typical communications facilities are shown in Table 3.1.

Table 3.1 – Communications Equipment (from Slope Indicator Catalog, 1998).

Technology	Type of Link
RS-232 Serial Communications (COM) Port	Direct connection
Network Interface Connection	Direct connection
Telephone Modem	Phone connection
Cellular Modem	Wireless connection
Radio Modem	Wireless connection

The RS-232 port is standard equipment on any personal computer. This direct connection is useful for transferring data over a simple cable and in an automated scheme could be connected to a portable laptop computer on site. Thus post-processing could occur with very little delay and decisions based on the measured data could be incorporated in daily operating routines.

A network interface connection is similar to the RS-232 but connects to a different port in the computer and uses coaxial cable. If the data acquisition system is used for long-term performance monitoring, this connection could facilitate data transfer to multiple computers nearby. The Slope Indicator system allows up to two hundred recording units to be connected on a network via this type of interface.

Modems are available that use many technologies. The short haul modem uses a dedicated four wire connection that may transmit data over 8 km to the remote computer. Standard phone modems use telephone lines for communication. This allows the recording unit to be controlled from computers in multiple locations over any distance, provided a phone connection can be made.

Wireless systems include the radio modem and the cellular modem. This equipment allows remote data acquisition over great distances. Radio modems generally require line-of-sight, or else need repeaters to help transmit data. Higher power radio equipment requires a special license to operate. The cellular modem, on the other hand, is not conducive to continuous connections due to the high cost of air time.

Post Processing

With all data acquisition systems, post processing capability is necessary. Once done using hand-held calculators to convert electrical signals to engineering units, post processing systems today rely on inexpensive computing power available in the personal computer. Major considerations focus on the choice of software, a decision which is best made based on individual needs. Ample data storage should be accessible considering the amount of information generated by the precision electronic instrumentation available today.

4 MONITORING SCHEME FOR A PROPOSED NEW BUILDING

4.1 Background

As a challenge to the Master of Engineering Program (MEng), the MIT Department of Civil and Environmental Engineering (CEE) requested a series of conceptual designs for a new academic facility to house an integrated CEE department. Currently, CEE occupies two structures located across campus from one another. This physical separation creates a rift in the department and makes the cohesive use of resources and talents difficult. In the fall of 1997, administrators in the CEE department proposed a new structure to replace both facilities and to be located on the property currently occupied by one of them, Building 48, also known as the Ralph Parsons Hydrodynamic Laboratory.

4.1.1 Building Concept

CEE administrators had specific requirements in mind for the conceptual design. First, the structure should be a hallmark of the current state-of-the-art in Civil Engineering. It should be structurally significant, while allowing flexibility in its design to limit prohibitions on future use. Secondly, the building should be designed as a "high performance" structure. This means that restrictions should be placed on allowable movements, and smart building systems should be integrated to permit self monitoring by the structure. Third, the building will need to accommodate all current CEE space needs. Finally, the proposed location is considered the future gateway into the MIT campus. Therefore, the structure should be aesthetically pleasing.

Some of these parameters directly affect the monitoring scheme that was chosen for this application. Specifically, the "high performance" requirement, and the notion that the site is perceived as the future locus of campus activity influence the selection of geotechnical instrumentation.

Two groups developed structural designs for this project (HPS Team, 1998). One of the two concepts (developed by Group B) is a building that in plan view looks like two rectangles

(Figure 4.1). The larger rectangle is 240' x 70', and the smaller 45' x 35' in plan, for a total floor area of 18,375 square feet. The structure has a sixteen-foot basement (floor to ceiling) and a five-foot foundation slab. Hence, the excavation on this site will be at least 21 feet deep. The foundation was designed to accommodate total building load of 14,000 tons, which is transferred to the slab around the perimeter. The design has no load-bearing interior columns.

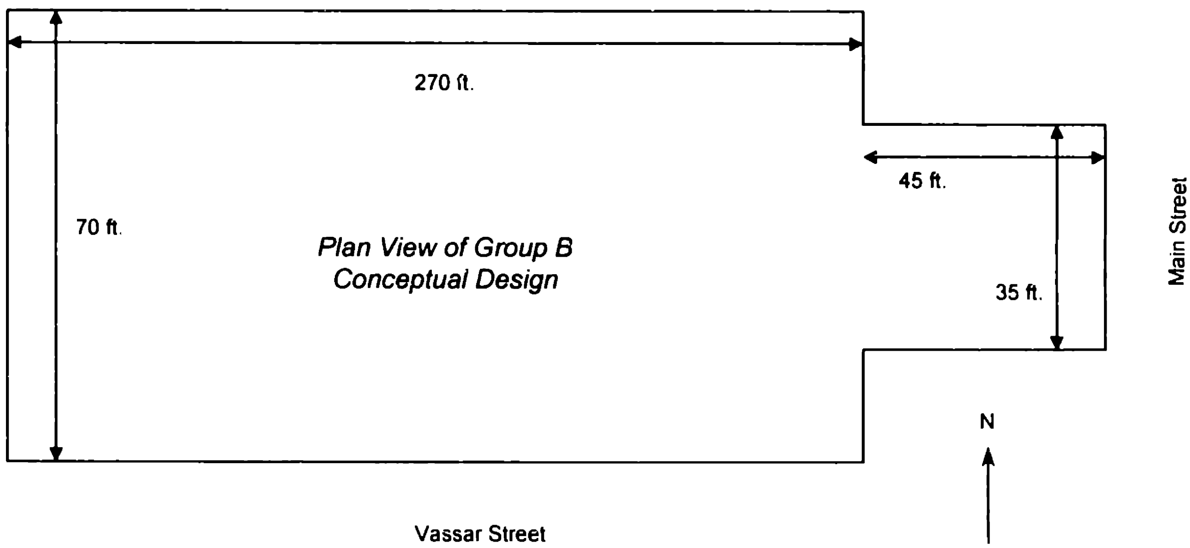


Figure 4.1 – Foundation plan for Group B conceptual design.

As part of the geotechnical work for this conceptual design, a combination pile and tie-down anchor system was developed to limit vertical movement in the lightweight structure. The piles are designed to carry the loads on the perimeter of the foundation. The tie-downs are designed to prevent uplift in the center of the raft that is caused by heaving soil under the deep basement. Heave is a problem because the weight of the center of the building is less than that of the excavated soil, exacerbated by buoyant forces from a water table that is relatively high in this area (Table 4.1).

Table 4.1 – Summary of loading for Group B structural concept.

	Total Load (tons)	Total Load (tons/ft^2)
Weight of Structure	14,000	0.762
Weight of Excavated Soil	22,770	1.239
Hydrostatic Uplift Pressure	7,450	0.406
Net Unloading of Soil	16,220	0.883

4.1.2 Site Conditions

Ground surface at the Building 48 site is +20 feet. The water table is located eight feet below the ground surface. The height of the water table may have slight seasonal variations, however borings from both the 1940s and the 1960s are in agreement with +12 feet above sea level as the average elevation (Haley & Aldrich, 1967).

Figure 4.2 is a soil stratigraphy for the site, showing the location of the proposed building. A representative description of each major soil layer follows.

Miscellaneous Fill

This layer consists of sand and gravel fill, occasionally mixed with miscellaneous material like asphalt, cinders and ash. Much of this material was dredged from the Charles River Basin to raise the elevation in this section of Cambridge. The miscellaneous materials are indicative of past industrial activity and construction on the site, and will be completely excavated to reach the depths required for a basement.

Peat and Organic Silt

This layer was deposited at the bottom of the tidal mud flats that once covered the area. Organic silt and peat are soft, compressible materials that are poor bearing surfaces. Complete removal of these materials will be required for the construction of the foundation.

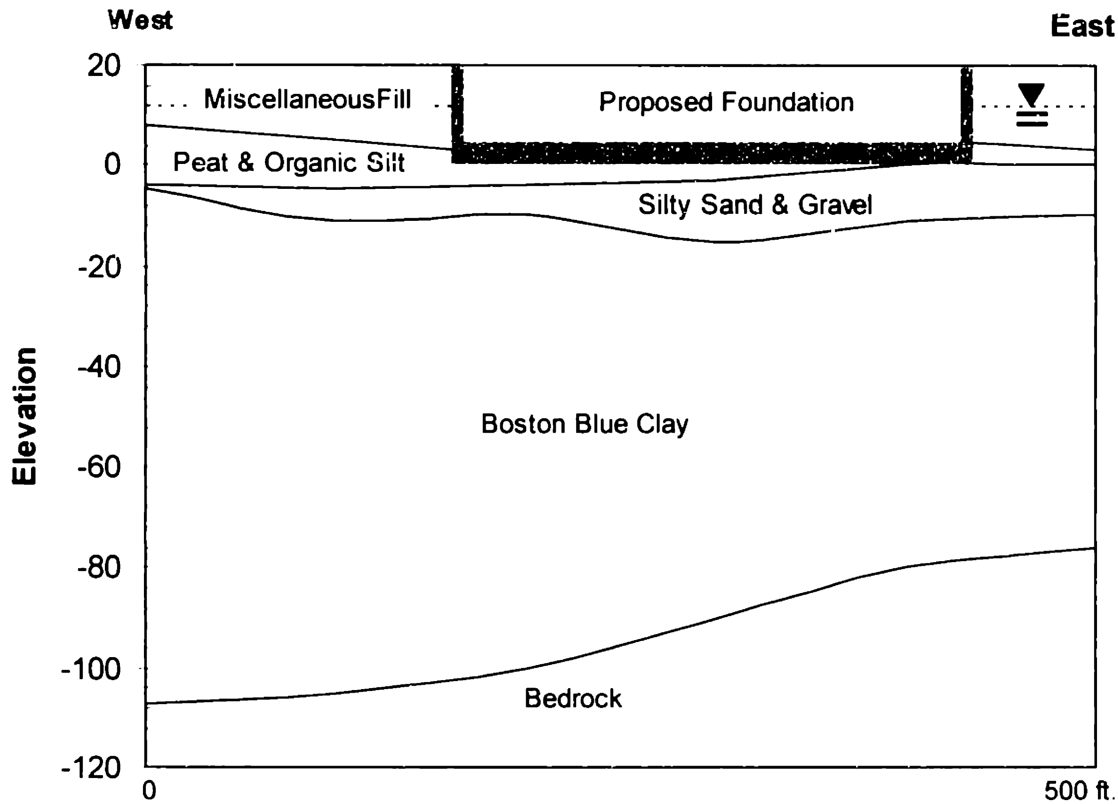


Figure 4.2 – Stratigraphy of Building 48 site – profile along Vassar Street (Haley & Aldrich, 1967).

Silty Sand and Gravel

These materials vary in depth throughout the site. They are capable of providing sound bearing, and are the primary materials underlying the raft foundation for Building 48. In areas under Building 48 where this layer was thin, compacted granular fill was added to augment the thickness of the layer. Sand provides good drainage and low compressibility. Settlement in sand layers usually occurs shortly after the application of loading.

Boston Blue Clay

This material is typical of the clay that can be found throughout the region. Deposited during the Pleistocene, Boston Blue Clay (BBC) is a medium plasticity blue-gray to green colored material that, on this site, contains numerous fine sand lenses. This clay was deposited in a marine environment. When sea levels fell subsequent to deposition, the upper layers of the clay were exposed to weathering to form a stiff upper crust that may be visually identified by its characteristic yellow color (Aldrich, 1970). The BBC layer varies in thickness throughout the MIT campus. The stiff upper crust is frequently used as a bearing layer for shallow foundations.

Till and Bedrock

The till is an unsorted layer of rocks and cobbles and varies in thickness across the region. Deposited during glaciation, material sizes can range from sand to boulders. The bedrock throughout the area is Cambridge argillite, derived from siltstone, claystone and shale (Aldrich, 1970). This layer can be heavily weathered at the surface.

4.1.3 Engineering Properties and Stress History of BBC

The following is a summary of the engineering properties of the BBC. Data were compiled from Haley & Aldrich (1967), and were compared to those of Berman (1993). A representative consolidation test from soil on the site is presented in Figure 4.3.

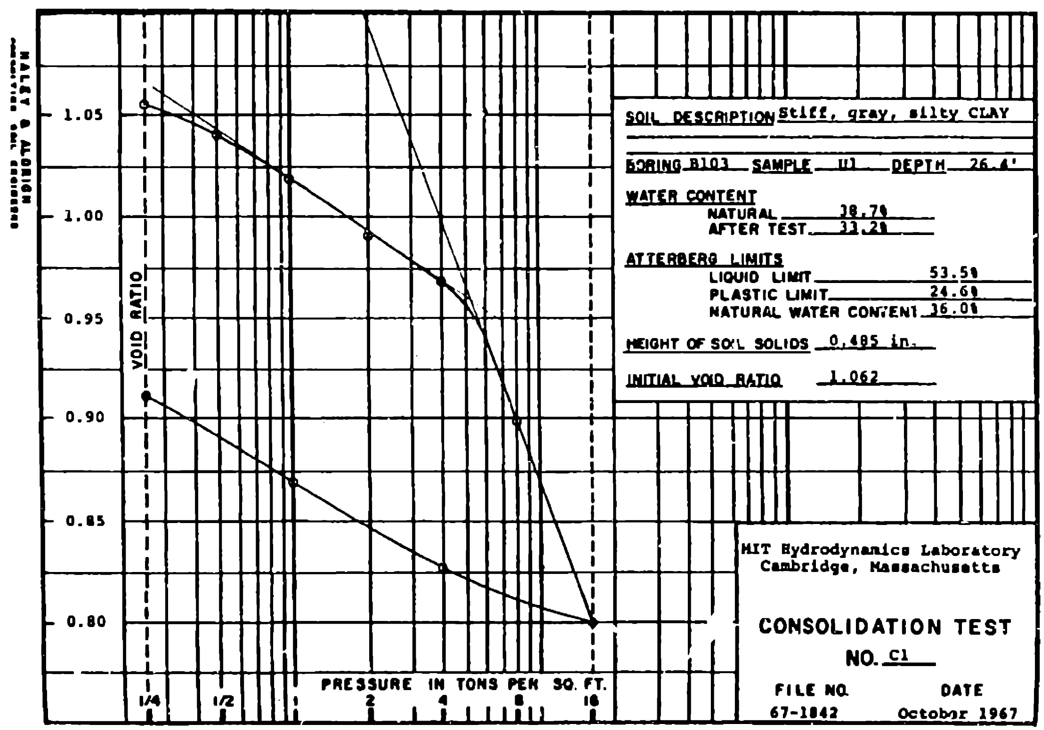


Figure 4.3 – Results of consolidation testing on BBC from Building 48 site, el. –6.4 ft. (Haley & Aldrich, 1967).

Berman (1993) presented data on multiple tests done for the soil underlying the Biology Building on the MIT campus (Building 68). Building 68 is located approximately 500 feet southeast of Building 48. Due to the unique history of the area, clay properties such as stress history are variable even across this short distance. As such Building 68 data cannot be used directly in design for Building 48. However, where Haley & Aldrich data are lacking, Berman provides another source to strengthen some assumptions made about the soils underlying Building 48. Furthermore, the Berman data benefit from over 20 years of advances in the quality of geotechnical testing procedures.

Figure 4.4 is a plot of the Atterberg Limits and the stress distribution vs. elevation. Atterberg Limits and preconsolidation pressure (σ'_p) were provided in laboratory test results from Haley & Aldrich (1967). Vertical total (σ_{vo}) and effective (σ'_v) stress distributions were calculated assuming a unit weight of the soil equal to 118 lb/ft^3, and assuming a hydrostatic water table at elevation +12 ft.

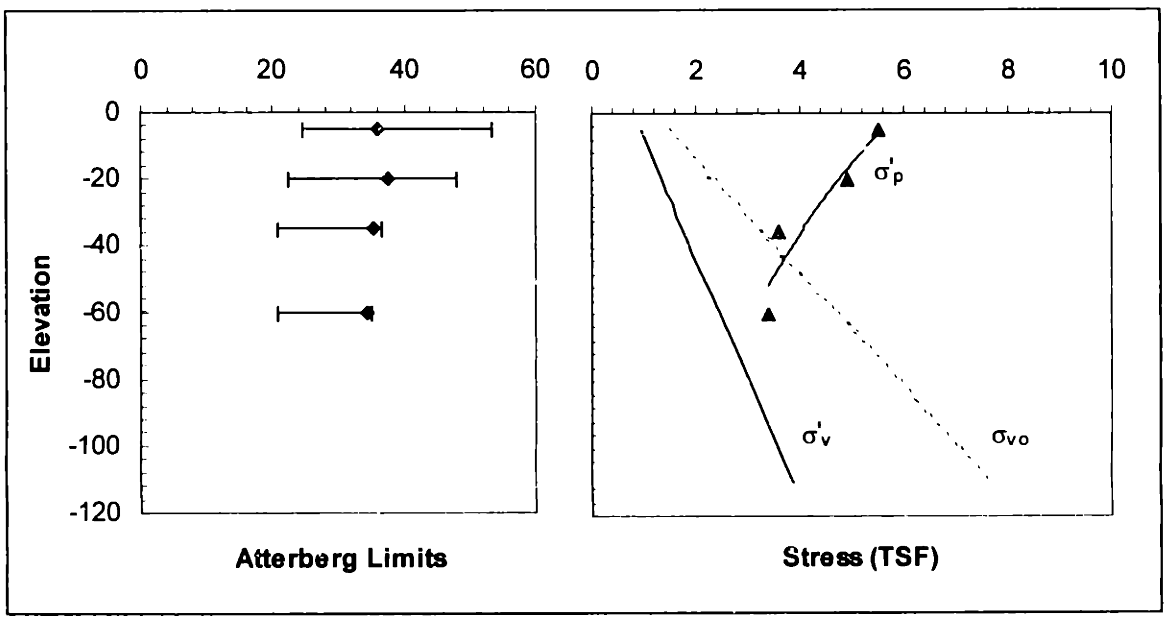


Figure 4.4 – Atterberg limits and stress profile for Building 48 site (Haley & Aldrich, 1967).

Preconsolidation data from Haley & Aldrich are very limited. To generate a more complete OCR profile (Figure 4.5), data from Berman (1993) were also included.

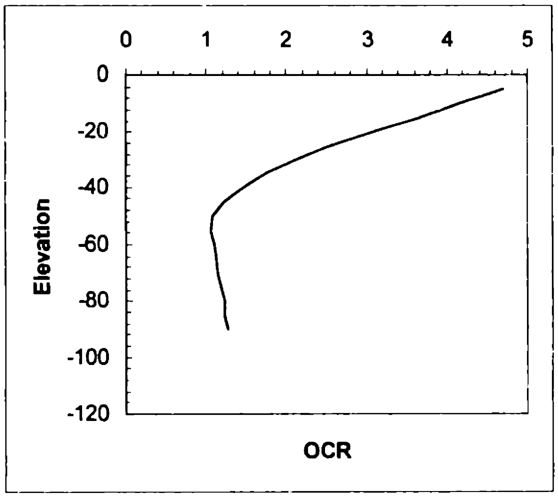


Figure 4.5 – OCR profile used for the conceptual design (Haley & Aldrich, 1967; Berman, 1993).

Standard 1-D compressibility parameters, RR and CR, were calculated from the slopes of the compression curves from Haley & Aldrich (1967). Only four compression tests were completed during this study, therefore RR and CR data are applied over thick layers of clay. There are no direct measurements of the coefficient of consolidation, c_v, in the report by Haley & Aldrich. However Berman (1993) reports typical values for BBC based on OCR and net loading (swelling or compression). Table 4.2 is a summary of these properties. Note that the c_v data *for swelling* are listed since the load of the conceptual structure is less than the weight of the excavated soil.

Table 4.2 – Recompression ratios, virgin compression ratios and coefficients of consolidation for BBC under Building 48 site (Haley & Aldrich, 1967; Berman, 1993).

Elevation (ft.)	RR	CR	c_v (ft^2/day)
-10	0.032	0.159	0.2
-20	0.031	0.170	0.2
-30 to -40	0.027	0.155	0.3
-50 to -120	0.023	0.194	0.6

4.2 Need for Geotechnical Monitoring

Three main factors contribute to a need for geotechnical monitoring on this site. First is the fact that the building has been designated as a high performance structure. Sensitive measurement equipment will be installed in the engineering laboratories, and any building movement due to adjacent construction is considered unacceptable.

The second factor is the probability that construction is likely to occur adjacent to the building in the future. This is an academic environment that is tightly connected to the research community in the private sector. Building 48 is located next to Technology Square in Cambridge, an area that has seen continued development over recent years. Hence, it can be expected that undeveloped parcels adjacent to MIT property will likely see future construction, especially in light of the research collaboration between MIT and the private sector in this area.

Furthermore, it was previously mentioned that the administration perceives that the locus of MIT campus activity will shift toward the Technology Square area. In fact, long range planning documents propose the future main entrance for MIT to be located at the intersection of Main and Vassar streets. If indeed this is the plan, it is only reasonable to expect that other campus building activities will occur in this area surrounding the Building 48 site. It is prudent, then, to incorporate a geotechnical monitoring scheme in the early planning stages of the new academic building. Not only will this allow performance monitoring of new foundation technologies, but it will also complement future monitoring activities for adjacent construction.

The third factor is the subsurface condition of the site. As previously mentioned, there is a net unloading of the soil, $\Delta q = 0.883$ TSF due to construction. Without tie-down anchors, the soil at the center of the building would tend to heave up to 4 inches (HPS Team, 1998). The final design is intended to limit total movement at any point to ¼ inch.

The goal of the monitoring scheme is to provide a comprehensive data set that depicts total and differential settlement in the excavation and for the foundation raft. Furthermore, the system should provide redundancy in data collection to ensure quality. An additional benefit can be realized from the system if instruments are added to measure dynamic inputs, and to verify the performance of state-of-the-art foundation construction techniques. To address these separate issues, accelerometers and load cells are incorporated in a system that includes traditional inclinometer/extensometers, piezometers, leveling stations and tiltmeters.

4.3 Design of Monitoring Scheme

4.3.1 Structural Details

As previously mentioned, a major challenge to the design of the foundation is the net unloading of the soil beneath the center of the building. If not properly mitigated, this problem could lead to long term differential heave of the soil under the foundation. To address this problem, the foundation design uses tie-down (or tie-back) anchors that physically connect the basement slab to the bedrock. A tie-down (Figure 4.6) consists of pretensioned high-tensile steel cables that are grouted into a borehole in either soil or bedrock. The grouted length of the tie-down carries the stresses in the anchor. The free end is physically connected to the structure being tied down, in this case the basement slab. The tie-down plan for Group B's structure is shown in Figure 4.7.

4.3.2 Instrument Selection and Installation

A variety of instrumentation is recommended in this application. The proposed system incorporates electronic sensors that will be connected to an automated data acquisition system. Chases will be cast into the concrete basement slab during construction allowing for easy wiring of the instruments. Where the chase leaves the building at the piezometers, PVC pipe will be installed underground to protect the wiring. Redundant instruments will be incorporated into the scheme that allow for calibration of the electronic systems and provide additional data for QA/QC on the measurements. Figure 4.8 is a plan view of the proposed instrumentation scheme.

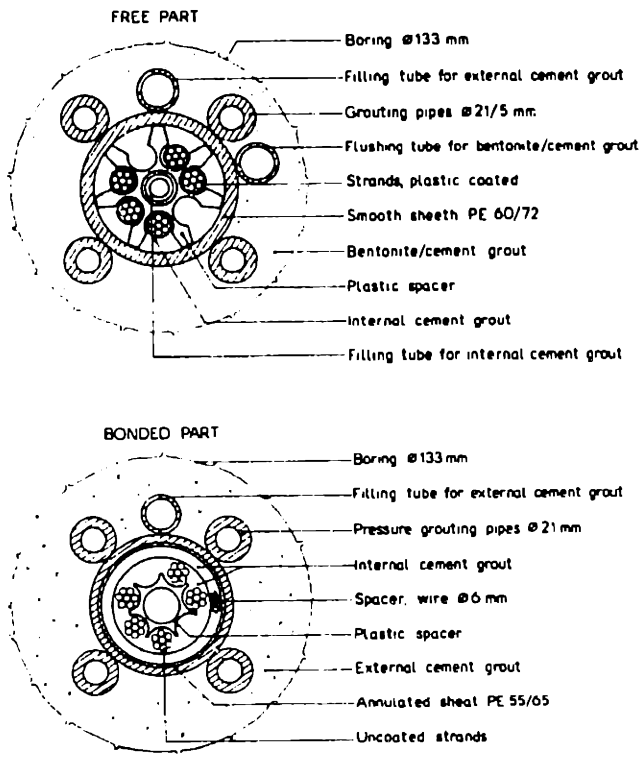


Figure 4.6 – Details of a tie-back anchor (Thogersen & Sorensen, 1991).

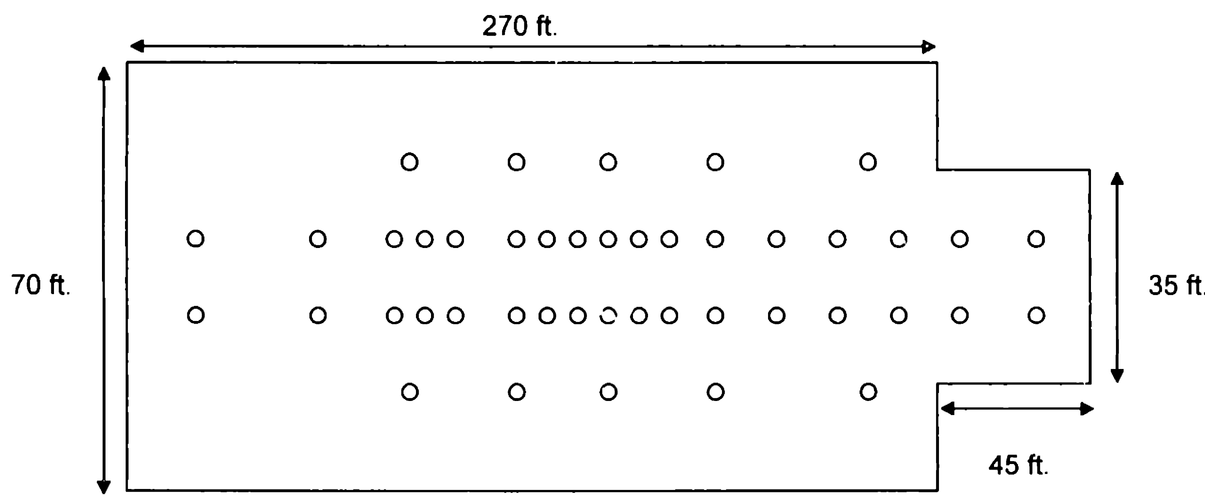


Figure 4.7 – Tie-down locations in foundation raft for conceptual design – plan view.

Load Cells

A total of 12 hydraulic load cells will be installed on selected tie-down anchors to measure changes in tension occurring after construction of the building. Readings from these gages will allow engineers and researchers to compare measured tie-down response to long-term stresses and pore pressures in the ground. This data can then be correlated with measured raft deflections to obtain a more accurate picture of the stiffness of the foundation system. A large number of tie-downs will be instrumented because changing failed gages after installation, although possible, is an extremely difficult task. Hydraulic load cells are considered excellent instruments for long-term monitoring, with service lives of at least 50 years (Hanna, 1995). They are insensitive to temperature, humidity and dirt. The hydraulic cell will be connected to a pressure transducer that is external to the load measuring device. If there is an electrical failure in this monitoring system, it is a relatively easy task to switch in a new transducer. An electrical failure internal to a vibrating wire or electrical resistance load cell would render the cell useless.

Tiltmeters

An array of 20 biaxial tiltmeters will be placed in the foundation mat in order to provide information on raft tilt in two perpendicular axes at each station. In the center of the foundation, tiltmeters will be used to verify the performance of the tie-downs in restricting differential movement within the raft. As some of this equipment has been shown to exhibit measurement anomalies such as drift, data from these gages will be correlated with periodic inclinometer readings and GPS leveling to verify accuracy.

Inclinometers/Extensometers

Seven combination probe inclinometer/extensometers will be placed around the perimeter of the building to allow periodic monitoring of subsurface ground movements during and after construction. The deep seated movements that this equipment can show can then be compared to the surface movements measured by the tiltmeter array. Inclinometer data will be used to generate a continuous profile of subsurface ground movements next to the excavation during construction. Following construction, probe extensometers will be used in the same casing to measure settlement in the clay surrounding the building.

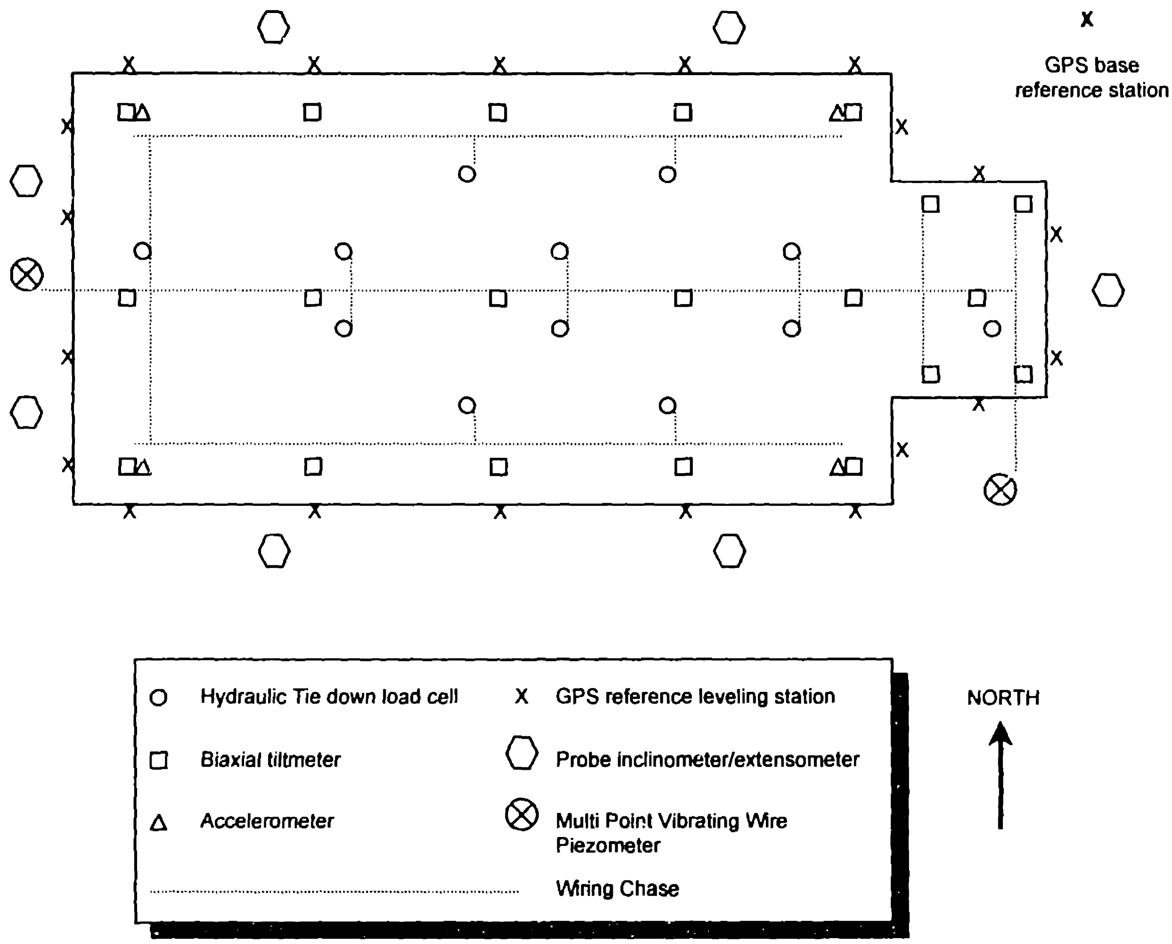


Figure 4.8 – Proposed instrumentation layout for conceptual academic building.

Piezometers

Two multi-point vibrating-wire piezometers will be installed before construction. Multi-point units have been specified to allow for pore water pressure monitoring at different elevations in the clay layer, during and after construction. For this installation, piezometers at -10 ft., -20 ft., -30 ft., -50 ft. and in the bedrock are specified, separated by borehole packers (Figure 4.9). Piezometer data will be used to create a profile of pore water pressures that can be used to calculate an effective stress profile beneath the structure. The bedrock piezometer will be used to monitor groundwater pumping in the region. The piezometers will also be used to monitor seasonal fluctuations in the height of the water table. Geonor (1998) documents 25-year accuracy greater than 0.5% of full scale for their piezometers.

Accelerometers

Accelerometers can serve many purposes in this application. Four of these gages have been specified for the corners of the raft. One purpose of these instruments is to measure dynamic inputs into the foundation during adjacent construction. Large structures being built nearby might require driven piles as part of their design. Thus the accelerometer array may be used to measure the input from pile driving operations, and potential effects on the foundation may be studied.

Also, the proposed building is located adjacent to an active railroad and subway. Due to the high-performance nature of the facility, accelerometers can provide critical information about the transmission of forces into the foundation from these transportation systems. If laboratory instrumentation will not function properly because of these dynamic inputs (which has already been shown in some of the labs in Building 48), studies can be done to determine acceptable dynamic input levels, potentially reducing the cost of mitigation measures.

Leveling Stations (GPS)

Exterior settlements will be measured using GPS positioning and leveling stations. This is an excellent opportunity to employ modern surveying techniques for a unique purpose, and ultimately could lead to methods for widespread use of GPS equipment in the geotechnical profession. Reference sockets will be permanently grouted into the foundation on the external wall of the building. The sockets will have a cover to protect them when they are not in use. A base GPS reference station will be installed in an open area Northeast of the building. This station, a concrete pillar, will be founded on the sand layer at elevation –5 ft. to ensure long term stability of the reference.

4.4 Integration of Instrument Network into Building Data Systems

The array of instruments outlined above will be integrated into the building monitoring network. Leads from the gages will be attached to a data recorder through multiplexers, if necessary. The recorder, in turn, will be directly connected to the computer system using a network interface connection. The benefits of this type of monitoring system are many. First, automated data collection schedules could be directly programmed via terminals throughout the network. Data storage can be administered on a campus wide network that will provide for integrity and security through automated back up schemes and password protection. Building services, like the Physical Plant could use this data to monitor the structure and to schedule maintenance if necessary. One of the greatest potential benefits is for research. Students and faculty will be able to access data directly, and in some cases, develop customized monitoring programs that suit particular research needs. And finally, alarm systems can be built in that warn of potential problems that might occur, especially during adjacent excavation and construction.

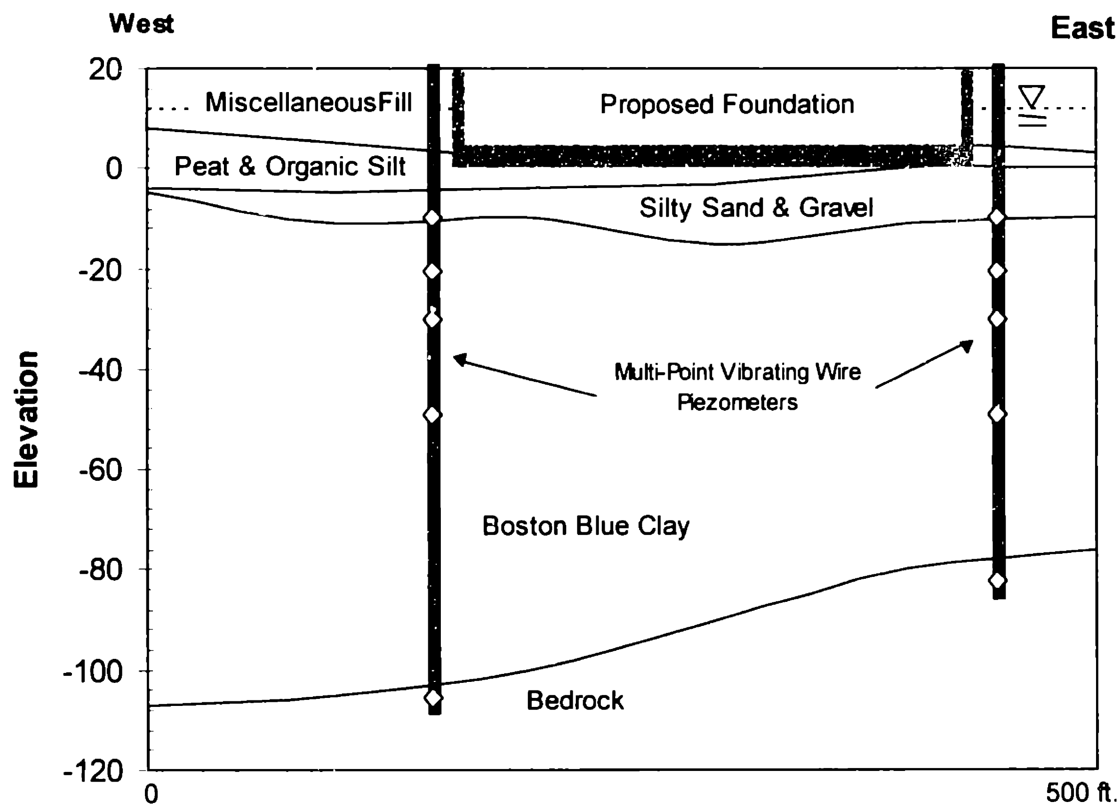


Figure 4.9 – Stratigraphy showing location of piezometers.

4.5 Operations and Maintenance

As part of the overall Operations and Maintenance (O&M) plan, the proper functioning of electronic devices should be verified on a regular basis. Furthermore, for mission critical data, regular change-out of a small percentage of instruments should be initiated, where feasible. It is important that the data collection system be treated as part of the larger computer network, and not be ignored. Too often, unsupported computer systems become a greater burden than benefit. A designated department liaison for the CEE should be appointed to oversee the operations of the geotechnical monitoring program.

This monitoring system should actually require little human intervention after installation. Most of the gages will be automated. GPS and inclinometer readings need only be taken occasionally, and can provide critical back-up data for the hard-wired systems. Thus, the long-term operations costs of this type of network should be negligible when compared to other costs associated with running an academic facility.

5 CONCLUSIONS

Geotechnical engineers have for many years struggled with predicting damage to structures caused by ground movements. This problem is of particular importance in urban environments, where construction activities such as excavations often impact large structures nearby. Since the early work by Skempton and MacDonald (1956), many engineers have attempted to study structural damage and to develop predictive methods for assessing potential building movements due to ground subsidence. Findings to date have not resulted in a comprehensive paradigm for making such calculations.

Some of the limitations of previous work include the obvious simplifications that need to be made to mathematically model something as complicated as a building. Variables in these studies have included the type of structural system (i.e., load bearing walls versus structural frame), and predicted ground deformations and settlement profiles near excavations. Even these classifications require extensive simplifications of actual structural and soil behavior.

What is clear from all of the work done is the benefit of high quality geotechnical monitoring data. Technologies exist for the insitu measurement of a range of soil and structural properties. Some of the instruments have limitations in the quality and repeatability of measurements. In particular, instruments requiring extensive soil disturbance for installation (e.g. earth pressure cells) that attempt to measure local soil properties in their immediate vicinity will often generate erroneous readings. Data in this case is more a function of installation procedure than a function of the insitu properties that are being monitored.

Even so, there are technologies that have been field-proven to give highly accurate readings. Most interestingly, new methods are still in development for making these types of measurements. Adaptations of grossly inaccurate surveying methods by geotechnical standards (e.g. GPS) have yielded refinements that make the technology suitable for precision measurements. The future of geotechnical instrumentation is in development, and new methods hold the hope of quickly producing high quality, repeatable measurements. The profession will benefit by staying abreast of these developments.

In setting out, the initial aim of this work was to develop a quasi standardized method of monitoring the buildings surrounding an urban construction site during excavation. However, the tremendous site-to-site variability and the large range of construction techniques and structural building systems prevents such standardization. The revised scope was to develop a monitoring system for a specific project.

The project chosen was a foundation monitoring scheme for a conceptual academic building at MIT. This structure is proposed to house an integrated Civil and Environmental Engineering department on campus. As a hallmark of building technology, the concept is a high-performance structure, with computerized self-monitoring and environmental systems, and tight restrictions on allowable building movements.

To address these considerations, a network of tiltmeters, load cells, and accelerometers was developed for integration with the foundation slab and tie-down anchors. These sensors will be connected to a campus wide computer network to allow data access to faculty, staff and students. To complement the slab monitoring system, leveling stations, piezometers and inclinometer/extensometers will be sited outside of the building. The data produced by the external instruments will be used to monitor construction activities, and as a quality check on the networked data collection scheme. It will also provide additional information on the performance of the building after occupation and during any subsequent adjacent construction activities. Ultimately, the data from these sensors may prove useful in future civil engineering research endeavors.

Continued research efforts should be focused on TDR and GPS. These two developing methods have the potential to significantly advance the current state-of-the-art in measuring ground and structural movements, and have many interesting potential applications in geotechnical engineering.

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