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CONSTRUCTION MONITORING OF SOFT GROUND RAPID TRANSIT TUNNELS

VOLUME I: A DEFINITION OF NEEDS AND POTENTIAL DEVELOPMENTS

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16. Abstract The Urban Mass Transportation Administration (UMTA) Tunneling Program Concentrates its efforts on reducing tunneling costs, minimzing environmental impact and enhancing safety as it applies to the planning, organization, design, construction and maintenance cycles of rapid transit tunnels in the urban environment. This study investigates the area of construction monitoring of rapid transit tunnels in soft ground.

Soft ground tunnel construction monitoring has the potential to reduce construction costs, safety hazards and environmental impacts. Monitoring can diagnose face stability and ground movement problems, and allow appropriate preventive or remedial action. Monitoring provides data for prediction of ground movements and allows the compilation of useful legal documentation. Such data are also required for improving design and prediction methods.

Monitoring practices now in use do not usually allow full utilization of the data for the project from which they were gathered. Deficiencies in present practices are pointed out, and a systematic approach to monitoring is presented. Information presented will aid owners, designers, specification weiters and instrumentation engineers. A computer program for data storage, interpretation and retrieval is proposed. An interim quality control specification for instrumentation procurement is presented, and instrumentation hardware improvements are suggested.

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PREFACE

The investigation described herein was sponsored by the Rail Programs Branch, Office of Research and Development, of the Urban Mass Transportation Administration (UMTA), U.S. Department of Transportation. Program management was by the Urban Rail Technology Program Office, Ronald J. Madigan, Manager at the Transportation Systems Center (TSC), Cambridge, Massachusetts. George Kovatch and Andrew Sluz of TSC's Mechanical Engineering Division provided the technical direction on this contract. Birger Schmidt of Parsons, Brinckerhoff, Quade & Douglas, Inc. was project manager and principal writer of Chapters 1 through 4, 6 and 7. Thomas R. Kuesel, also of Parsons Brinckerhoff, was Officer in Charge of this project. The instrumentation hardware and field application aspects of the study were largely the work of C. J. Dunnicliff, Principal of Soil and Rock Instrumentation, Inc., Newton Upper Falls, Massachusetts. Mr. Dunnicliff, who was the principal writer of Chapters 5, 8 and 9, also provided significant and valuable input to other chapters--in particular, Chapters 4 and 6.

The writers express their gratitude to a large number of consultants, contractors and instrumentation experts who have contributed considerable assistance at various stages of the study.

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1. INTRODUCTION

1.1 Background

More than a hundred thousand dollars can easily be spent on instrumenting and monitoring a soft ground tunnel construction project. It is more difficult to measure the economical return on this investment. Often vast quantities of accumulated data are either unreliable, irrelevant, or simply not used because of a lack of incentive or knowledge to draw the appropriate conclusions in the available time, and because there is no clear course of action outlined to which the data apply.

To be of use for the project itself and for the profession, monitoring instrumentation must be deployed for a purpose and must be made an integral part of design and construction of the project. Procedures for reading, interpretation and implementation of conclusions, must be carefully defined in advance.

Deficiencies of current practices are apparent through all stages of monitoring: planning and design, project specifications and instrument procurement specifications, instrument selection, installation, reading, interpretation and implementation. Where just one or two of these steps are deficient, the whole monitoring program falters, and its potential benefits may be wiped out.

While most significant tunnel construction parameters may be monitored using available equipment, two classes of problems hinder the full utilization of the instrumentation potential. These are: the shortage of experienced engineers at several levels of design and construction, and the lack of incentive under present contracting practices in the United States. The lack of uniform standards and instrumentation philosophies is also detrimental.

A basic purpose of the research reported herein is to reduce the cost, enhance the safety and minimize the environmental effects of underground construction, thereby making underground rapid transit systems more desirable. The study has considered all types of soil conditions in urban environments including adverse groundwater conditions and soil-rock mixed face conditions, and typical bored rapid transit tunnels to a depth of about 160 feet. The study includes shafts and tunnels placed in cut.

This report systematically examines the entire practice of tunnel construction monitoring, beginning with an analysis of its basic purposes. The report goes on to explain the potential uses and benefits of monitoring and to define the most useful monitoring parameters. Through an analysis of current practices and available methodologies, a number of deficiencies are identified.

There is a need to improve the general practice of tunnel construction monitoring, through educational efforts and efforts to provide appropriate incentives. There is also a need to improve the speed, accuracy and relevance of data processing and interpretation. Certain deficiencies in the installation and maintenance of instrumentation hardware are also identified, but these are not as serious as the deficiencies of the general practice and data handling aspects. Proposals are made herein for the improvement of monitoring practice, for a unified computerized method of data handling and interpretation, and for improvements and modifications of instrumentation hardware. An Interim Quality Control Specification for Procurement of Geotechnical Instrumentation has been prepared and is included as an appendix to this report.

The results of this research effort are summarized in more detail in Section 1.2.

Portions of this report, notably Chapters 2, 3, 4 and 6, contain useful information for rapid transit tunnel owners and design managers in decision-making positions. Design engineers, specification writers and instrumentation engineers will find other chapters of particular interest.

1.2 Research Results and Recommendations

Early in the studies it became apparent that a single hardware/software system for monitoring critical tunnel construction parameters was not feasible. Instead, a monitoring system must consider all phases from concept to implementation. It was also found that the greatest deficiencies of existing practices are not in the hardware itself, but in the proper application of the principles of instrumentation, instrumentation installation, interpretation and implementation. This need is best attacked by educational efforts and attempts to promote or enforce the proper use of monitoring instrumentation.

Though the greatest needs are related to applications, some significant weak links in the hardware have been isolated.

A quality control specification for instrument procurement is recommended and outlined, and one specific hardware item as described below is proposed for development. A need for a unified computerized data processing system has also been identified and its development proposed. In the following paragraphs, the most significant findings and recommendations are summarized.

Functions of Construction Monitoring. Construction monitoring serves at least four functions:

1. Diagnostic Functions. These consist of determining the interaction of the soil and groundwater with excavation and construction processes, and the suitability of construction and dewatering details for a specific soil environment, to form the basis for modification of these details during construction.

2. Predictive Functions. These permit prediction of soil behavior, face stability and ground movements later on in the same tunnel, and permit predictions of future settlements of streets and buildings, and the effects of driving an adjacent tunnel.

3. Legal Functions. These are documents that show actual settlements, displacements and strains of streets, buildings and other structures which are valuable material during litigation, and may even eliminate court proceedings. Such records are also necessary to show a contractor's compliance with contract requirements.

4. Research Functions. These correlate soil and groundwater parameters and construction details with observed ground behavior. They build up greater confidence in pre-construction assessment of ground movements and their effects and provide better future design data.

Benefits of Construction Monitoring. Construction monitoring alone will not improve the economic, safety or environmental impact of soft ground tunnel construction, but if monitoring is properly planned and executed, with enforcement based on pre-established procedures and criteria, benefits can far exceed monitoring costs. Apart from legal benefits, at least the following possible benefits may be identified, though not all may be incurred on a single contract.

1. Reduction of construction costs through the elimination or reduction of hazards of various kinds, through the possible elimination of certain protective measures such as underpinning, through avoidance of delays due, for example, to unanticipated water inflows, and many other items. This benefit can be increased if monitoring is concentrated most heavily in the early phases of construction, thereby providing timely input to reevaluation of construction plans (see Section 3.2).

2. Reduction of environmental effects, for example, by diagnosing causes of excessive settlements so that remedial measures can be instituted. This benefit is possible only if construction documents contain the clauses required to enforce such remedial measures (see Section 3.3).

3. Increased safety of tunnel workers (and surface traffic) by allowing, for example, prediction of tunnel face instability due to water inflows through groundwater monitoring (see Section 3.4).

4. Verification of structural design assumptions through stress and strain measurements of temporary and permanent structures, for purposes of safety and durability (see Section 3.5).

5. Documentation to verify dimensions, etc., for acceptability, to provide appropriate data on ground movements and movements of buildings for legal purposes, and data gathering for other legal and contractual purposes (see Section 3.6).

6. Control of contractor's performance to ensure an acceptable final product, to provide data for enforcing appropriate modifications of contractor's procedures by diagnosing problem areas, such as safety (see Section 3.6).

7. Advancements in the state-of-the-art of tunnel design and tunneling procedures can only be made on the basis of observations of behavior (see Section 3.8).

Improving Monitoring Practice. By far the greatest deficiencies have been found in the current engineering practice of monitoring. To combat these deficiencies mentioned earlier, it is proposed to prepare and distribute a set of manuals and to establish an educational program. Outlines of the manuals are presented in Appendices D and E. Proposed programs for improving monitoring practices are presented in Chapter 6. The manuals and the improvement programs are outlined briefly below:

1. Manual 1 - Uses and Benefits of Soft Ground Tunneling Instrumentation. Addressed to decision makers in rapid transit planning, designers and specifications writers, this manual describes the needs and benefits of tunnel monitoring, a systematic approach to tunnel monitoring including selection of monitoring parameters, specifications required for full utilization of monitoring, and data processing and implementation procedures.

2. Manual 2 - Selection and Use of Instrumentation for Monitoring Performance of Soft Ground Tunnels. This manual is addressed to geotechnical engineers and specifications writers responsible for project design, preparation of construction documents, and supervision of construction, as well as engineers and technicians responsible for instrument installation, maintenance and data collection. It contains criteria for selecting proper instruments to monitor specific parameters in given environments, procurement and installation specifications, and recommendations regarding maintenance and data collection.

These manuals are to be working drafts, subject to modification after a trial period. It is proposed that these Manuals be used and their directives, standards and guide specifications implemented on one or several upcoming tunnel contracts in Washington, Baltimore or elsewhere, so that their contents are tested under prototype conditions for acceptability, utility and legal problems.

It is also proposed to establish an Institute of Tunneling Instrumentation and Exploration, or a similar type of organization with a permanent staff, whose functions would include, among other items:

1. A central data bank for instrumentation, exploration and tunnel monitoring data.

2. Short courses, seminars and/or workshops to be conducted by invitation or by directive, in general or in connection with specific projects.

3. Consultation services on a limited basis, on unusually difficult or extraordinary projects, to clients involved with government supported projects.

4. Consultation services to the government on questions associated with research and development of monitoring and exploration instrumentation for tunnels.

5. Establishment of this institute will require cooperation, coordination and combined funding from several organizations and agencies, public as well as private. It is not proposed that a wholly new organization be formed. Rather, one or several existing bodies can be consolidated and their purposes expanded to include these functions (see Chapter 6).

In assembling material for this research effort, it was necessary to inventory commercially valuable geotechnical instruments, and to initiate a specialized literature retrieval system. The inventory is discussed in Section 6.4, and a categorized listing of geotechnical instruments is included as Appendix B. An evaluated or annotated version of this listing is proposed to be included in Manual 2. The literature retrieval system is described in Section 6.5, and recommendations are used for development and dissemination of the system.

Computer Program Developments. A computer program of general character, that may be used both for interpretation of monitoring data and for design purposes is not needed at this time. Finite element or similar programs analyzing interaction between tunnel structure and soil, or ground movements due to tunneling are potentially useful for research purposes. When refined, they are also useful for design, but of limited use directly for monitoring. There is, however, a need for a unified computerized data processing system to handle large quantities of monitoring data. Such a computer program, called "Data Processing System for Soft Ground Tunnel Construction Monitoring" (DAPSOG) is described in Chapter 7 and a specification is presented as Appendix F. This program would be capable of:

1. Accepting field data from monitoring of ground movements and groundwater levels.

2. Sorting and storing such data.

3. Outputting (printed or plotted) selected data or ranges of data.

4. Performing certain simple interpretive analyses on the stored data.

In addition, the program will store, for analysis and output, basic tunnel geometry data. The program is simple, and modifications and additions may be performed by users with little effort. In view of the relative simplicity and low cost of this computer program, and its significant potential, an early development of this program is recommended.

Instrumentation Hardware Deficiencies and Possible Improvements. As indicated, most critical parameters can be monitored adequately with existing hardware systems. However, the state of hardware development is far from perfection, though many suggested improvements are minor details.

1. Quality control specifications for instrument procurement are urgently needed. For durability and reliability, each component of a hardware system must be properly selected, designed and assembled; all components must be compatible and resistant to environmental attacks. While the basic ideas behind existing instruments are often excellent, too often deficiencies are found in packaging and in quality control of simple components or complete systems. A well-written and enforced quality control specification would reduce such deficiencies (see Section 5.9, and Appendix A).

2. Installation methods for single and multiple point extensometers, piezometers, inclinometer casings, and others are, in general, cumbersome, time consuming and hence, costly. There is a need for development of more efficient installation methods, possibly associated with minor changes of instrument designs (see Sections 5.2, 5.5 and 5.6).

3. In the light of the above observation, a combined single point extensometer and piezometer for rapid installation above the tunnel crown is proposed herein (see Sections 5.2 and 5.5, and Appendix G).

4. Monitoring of loads in bracings and tiebacks is currently unreliable; instrument modifications are needed for higher reliability (see Section 5.7 and Chapter 9).

5. Inclinometer measurements are, at present, relatively cumbersome, costly, and frequently unreliable due to real or suspected malfunctioning of the sophisticated equipment. There is a need for improving inclinometer reliability and, when reliability has been achieved, for complete automation of the data acquisition and interpretation (see Section 5.6).

6. Street settlement monitoring by conventional methods tends to obstruct traffic. The development of methods that would minimize traffic obstruction is desirable. Several feasible schemes are suggested in the text (see Section 5.3).

7. The gap between the tunnel lining and the surrounding soil is frequently a significant diagnostic parameter that could be used to help define modifications of construction procedures. This gap may be monitored by simple means, but there may be a future need for improved techniques (see Section 5.8).

8. Tunnel lining geometry is difficult to measure where it is most needed, that is, at the tunnel face, due to tunneling equipment. There may be a future need to improve such measurements, to determine the lining shape as built, and to measure distortions that change with time (see Section 5.8).

9. Stresses in linings due to loads from grouting, from earth and water pressures, imposed distortions, and jack pressures from shield propulsion, are of use primarily for research purposes. Such measurements are costly and of limited accuracy. Also, they tend to interrupt construction progress (see Section 5.8.).

10. Automated groundwater monitoring is, at present, rarely needed but may be an attractive alternative

in the future. Such automatic monitoring can be performed with off-the-shelf components (see Section 5.2).

11. Provision of calibration check features are recommended as a means of verifying reading accuracy (see Section 5.9).

2. SIGNIFICANT TUNNELING PROBLEMS IN SOIL

2.1 Introduction

Later chapters will define the needs for, and benefits of, tunnel construction monitoring, as well as development priorities. This chapter describes the interaction between tunnel construction processes and the soil-groundwater environment, the cultural environment, and in particular the risks of tunneling. Once the typical tunnel construction processes and their effects have been defined, it is possible to determine which effects can be monitored, and which require monitoring to meet specific requirements or to counter specific identified problems.

A simultaneous study (Schmidt et al, 1974) analyzed in considerable detail a variety of geotechnical problems and hazards associated with tunnel construction in soil. The following paragraphs describe those processes and effects of tunnel construction that may be monitored, and whose monitoring may result in savings or reduction of hazards of various kinds. The description deals primarily with bored tunnels, but comments regarding cut-and-cover construction and shafts are also presented. The reasons for this emphasis are that bored tunnels in general pose the severest problems and are more difficult to monitor. In addition, the problems of deep excavations are, in general, already well known (for example, see Sverdrup & Parcel, 1973).

2.2 Typical Tunneling Procedures

Most modern rapid transit tunneling in soil employs a shield which is shoved forward by jacking against the erected lining. The shield is a steel cylinder, 10 to 15 ft. long, provided with a vertical or slanted cutting edge in front. The top (crown) front edge is often hooded; i.e., the leading edge overhangs the bottom (invert) front edge, to provide protection for workers close to the face and to assist in keeping the working face stable in granular soils (see Fig. 2.1).

Soil is excavated from the working face manually or mechanically, for example by clay spades, by a large articulated pick and hoe combination, or by rotating or articulating

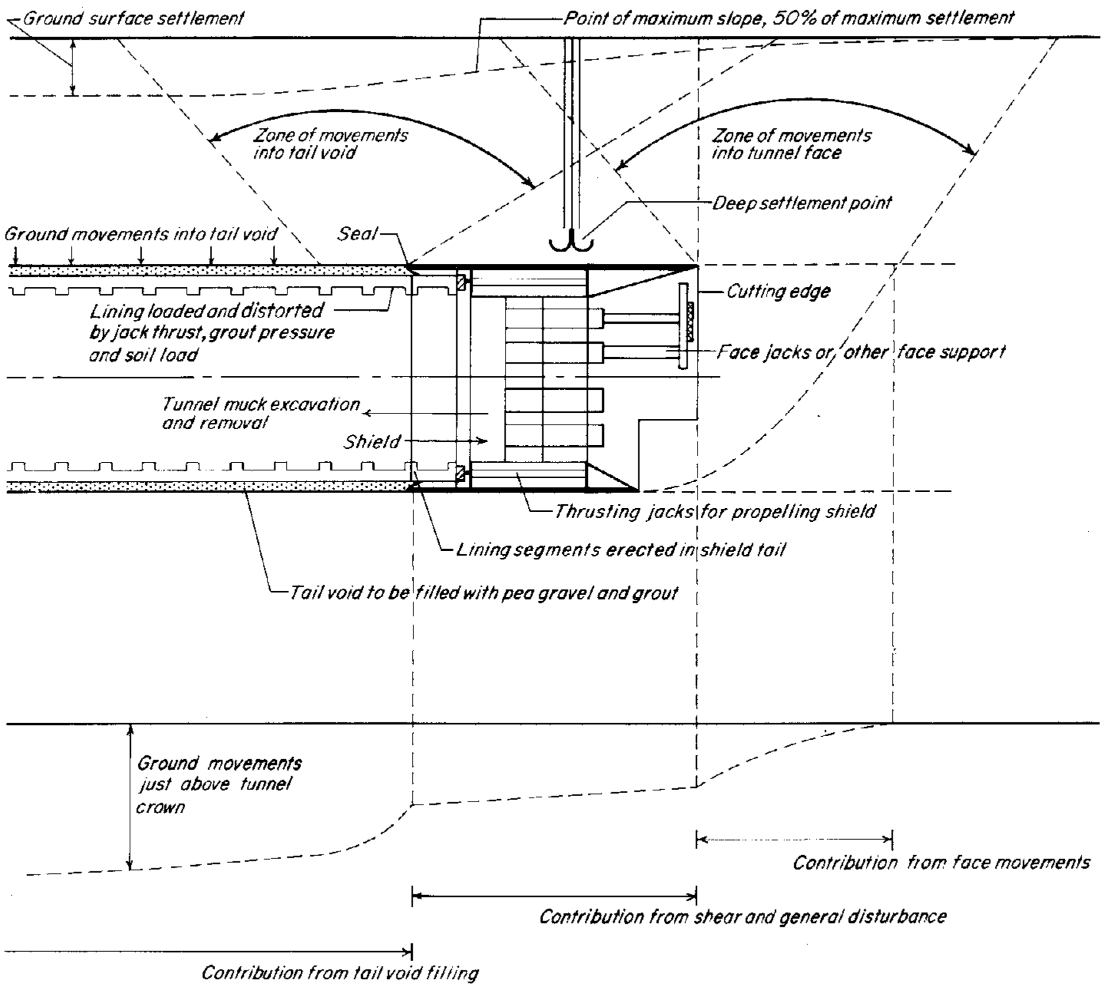


Figure 2.1. Processes and Ground Movements in Standard Shield Tunneling

arms provided with cutting teeth or blades. Muck is removed by conveyor belts and/or muck cars, or by other means.

The face may be open, or it may be partly or fully supported by breast boards pressed against the face by face jacks, or other similar means. Alternatively, the rotating or oscillating digger may be virtually closed to provide face support, and the individual arms or blades may be tilted to allow muck to enter the tunnel in carefully controlled quantities as the digger wheel rotates. Face support may also be provided by pressurizing the entire tunnel with compressed air, or, in some unique tunnel machines, by placing only the isolated face under air, water or slurry pressure.

Typically, the tunnel lining consists of a number of prefabricated segments, erected and bolted together to form a ring inside the tail of the shield. As the shield is then shoved forward by jacking against the last erected ring, a void is left between the lining and the soil of a thickness theoretically equal to the sum of the clearance between ring and tail, and the shield tail thickness. The lining may also consist of steel rings or ribs with an H-shaped cross section, with timber or steel lagging between rings. This is later followed by a permanent concrete lining. It is sometimes possible to expand a lining directly against the soil behind the tail, if the soil stand-up time is adequate.

The void usually left behind the lining must be filled, usually in two stages, with pea gravel or sand, followed by cement grout, to prevent soil from entering the void. Sometimes, an additional stage of grouting is performed under higher grouting pressure, to ensure complete filling of all voids. The first stage of grouting can only employ relatively low grouting pressures; otherwise grout will enter the shield through the clearance between the lining and the shield tail. This clearance is notoriously difficult to seal against high pressures. In very soft squeezing or flowing ground, it may not be possible to fill the void before the soil flows in.

2.3 Ground Movements Due To Tunneling

Ground movements depend on soil and groundwater conditions, tunnel geometry, the contractor's general procedures, details of tunneling equipment and methods, and the care with which the tunnel is built. When ground conditions are predictable and the contractor has the proper (pecuniary) incentives, ground movements can be reduced to the level of insignificance. When this incentive does not exist and when ground conditions are unpredictable, ground movements can be very large, to the point of tunnel face collapse and daylighting.

Soil moves toward the tunnel opening from above and laterally through the tunnel face, and into the tail void space (see Fig. 2.1). Soil movements are also generated by the shoving of the shield whenever the shield axis and the tunnel axis do not exactly coincide. Water flows may also carry soil particles into the tunnel and, finally, the change of the soil stress conditions around the tunnel brings about elastic or elasto-plastic strains and displacements.

Soil movements caused by singular incidents or effects tend to appear abrupt when observed at or near the tunnel. The disturbance spreads as it moves up toward the ground surface and results in fairly widespread subsidence effects. For this reason, observations at or just above the tunnel are much more useful for diagnostic purposes than surface settlement observations.

Settlements and horizontal displacement at the ground surface and below, and stresses and strains are all roughly proportional to the amount of soil lost during tunnel construction. This quantity of lost ground, defined as the difference between the amount of soil excavated and removed and the theoretical tunnel volume, either as a volume per foot of tunnel or as a percentage of the theoretical tunnel volume (generally based on the outside dimensions of the shield), is unfortunately quite unpredictable.

2.4 Soil Displacements at the Tunnel Face

In cohesive soils, the displacements from above and laterally toward the tunnel face are greatly influenced by the ratio of the full overburden pressure to the undrained shear strength of the soil, p_z/c. For low values of this ratio, less than about unity, ground movements are essentially elastic and small. For greater values, greater and greater plastic deformations occur until complete collapse occurs at a ratio of six to seven. With p_z/c values smaller than about four, soil displacement occurs during excavation but stops when excavation stops; however, for higher ratios, displacements continue also during work stoppage.

In granular soils where the tunnel face is rendered stable either because of an inherent small cohesion of the soil, because of an apparent cohesion due to a small water content of the soil, or by artificial face support, soil displacements at the face are usually insignificant. Ground movements of significance are usually due to face instability; for example, running of cohesionless, dry sand, flowing of granular soil below the water table, or slabbing or ravelling of slightly cohesive or fissured materials. In many instances, an initially stable face becomes unstable if exposed for a length of time.

Ground movements are much more irregular and unpredictable when associated with face instability than with squeezing cohesive soils. There is also a significant dynamic effect in some types of soil; i.e. the loss of ground may depend on the time of exposure at the face and, hence, the rate of construction.

2.5 Soil Displacements Caused by Shield Movements

When the shield is shoved forward exactly following a path parallel to the shield axis, only shear displacements caused by the friction of soil against the shield occur. Many rapid transit tunnels, however, are driven on a spiral or circular horizontal curve. Hence, the shield is forced to plow through the soil, and the resulting overexcavation is a function of the length of the shield and the radius of curvature, or the angle between the direction of the shove and the direction of the shield axis at any time. Vertical curves, of course, have the same effect.

Even on theoretically straight (tangent) sections, there are always irregularities in soil conditions, in the equipment or in the application of jacking forces that cause the shield to pitch and yaw. Therefore, there is always a certain minimal quantity of lost ground that depends on the accuracy and consistency of the steering of the shield. The roll commonly experienced by a shield has little or no effect on ground loss but is desirable for other reasons, since a good deal of equipment is usually fixed to the shield and workable only in a near-horizontal position.

2.6 Soil Displacements into the Tail Void

As the soil loses its support behind the tail end of the shield, it tends to move vertically and laterally toward the tunnel. In soft clays and silts with a high p_z/c ratio, and in dry cohesionless sands or flowing sands below the water table, whose stability is not otherwise controlled, the tail void may become completely filled in very short order. In contrast, stiff clays and many cohesive granular soils will remain stable with minimal movements long enough even for erection of liners behind the shield. Where complete tail void filling occurs, the associated ground loss is generally about three to four percent of the tunnel volume; a significant quantity.

The injection of pea gravel, or, more rarely, sand into the tail void through grout holes in the lining will partially arrest this soil movement. Later applications of grout will ideally fill the void completely.

2.7 Effect of Expanding Liner and Grouting

It is possible to reverse the movements of soil by application of large forces during the expansion of a lining behind the shield, or by high-pressure grouting of the tail void. Near the shield, only low pressure grout can usually be used because of the difficulty in sealing the space between the lining and the shield. Hence, high-pressure grouting is employed where needed at a distance of several tens of feet behind the shield. The soil movements usually caused by liner expansion or grouting are measured only in fractions of inches; hence, they are generally not significant. Once soil and liner are in contact directly or through grout, no further ground loss occurs. Even if the tunnel lining distorts severely, the tunnel volume remains virtually constant.

2.8 Volume Changes in the Soil

As the tunnel is excavated, the radial soil stresses near the tunnel are greatly reduced while the tangential stresses may be significantly increased. Therefore, large shear stresses are set up, and the stresses previously carried by the excavated soil are transferred to adjacent soil volumes. In saturated cohesive soils, these stress changes take place with very little immediate volume change, because water cannot move rapidly through the impervious soils. In granular soils, however, the shear strains frequently cause dilation or loosening when the soils are relatively dense, and sometimes cause a volume decrease or densification when the soils are quite loose. In a granular soil with some cohesion or cementation, the shear strains may cause the soil to lose some of its cohesion so that an adjacent tunnel driven afterward would be driven through soil of lesser quality than the first tunnel.

Other types of volume changes are associated with manipulations of the groundwater table. A lowering of the water table either deliberately for stabilization purposes, or because the finished tunnel sometimes acts as a drain, will increase the effective overburden pressures in the underlying soils. In compressive silts and clays the resulting settlements can be significant. In stiff clays, the permanent stress changes will cause some reconsolidation or expansion as the soil seeks a new moisture equilibrium. Though the associated volume changes are quite small, the effects on lining loads and distortions can be appreciable.

2.9 Groundwater, and Pore-water Pressures

More tunneling problems and hazards are associated with the occurrences of groundwater than with any other single factor. Without groundwater control, otherwise firm, ravelling or manageable running ground can turn into severe conditions of soil and water flows that in the extreme may bury the tunneling equipment in catastrophic fashion. The gradients of water flowing toward the tunnel opening may weaken granular soils to the point where quicksand conditions occur. When an aquifer, a permeable lens or stratum filled with water, is opened up by the tunnel excavation, a sudden inflow of water may carry large quantities of soil into the tunnel. A chimney may be created to the ground surface or, if stronger cohesive soils exist above, large horizontal soil flows may occur, resulting in widespread settlements or local collapse depending on the geological conditions.

These occurrences of instability, even if not as catastrophic as indicated above, cause substantial delays in tunneling and a considerable risk to the tunnel workers. The associated ground movements can have serious effects on overlying and adjacent utilities and structures, and be a significant hazard to surface traffic.

For these reasons, tunneling below the groundwater table in granular soils or soils with aquifers must usually employ measures of groundwater control: pre-construction dewatering, compressed air in the tunnel balancing the water pressure, or combinations, or on occasions grouting or freezing. Monitoring of groundwater pressures is employed to ascertain the need for and the adequacy of such control measures.

2.10 Distribution of Ground Movements Above and Around the Tunnel

When the loss of ground has been estimated, the ground surface settlements, displacements and strains can be predicted as long as the soil volume remains reasonably constant (Peck, 1969; Schmidt, 1969; Schmidt, 1974). Even if the volume of the settling soil changes slightly, these quantities are fairly predictable. However, when the loss of ground is highly irregular, as when it is associated with random instability of the face or the tail void, the distribution as well as the quantities become unpredictable.

A settlement trough is created above the tunnel, roughly symmetrical about the centerline surface trace, and having an inverted bell shape roughly equivalent to an error curve (see Fig. 2.2). Its width depends on the depth and diameter of the tunnel and is predictable when the subsiding soil volume is essentially unchanged. It is wider in a densifying soil and narrower in a loosening (bulking) soil. In the direction of the tunnel driving, a continuous action occurs wherein the soil at the surface first stretches as the tunnel face approaches and is then recompressed as the tunnel face passes. The greatest horizontal displacements occur where the settlement profile changes curvature. The maximum horizontal displacements are roughly one-third of the maximum settlements but depend somewhat on the tunnel geometry. Tensile strains are maximum about 50 percent farther away from the centerline than the maximum horizontal displacements. At depth, horizontal displacements occur roughly as indicated on Fig. 2.2.

Variations to this general displacement picture occur, for example, when a cohesive soil overlies the tunnel crown, and ground loss is experienced through widespread water-induced soil flows into the tunnel. In such cases settlements can have a much wider distribution than indicated above. Where local instabilities occur at the crown or at one side or another, the corresponding settlement may be significantly narrower than when the whole tunnel cross section contributes to ground loss. Not infrequently settlement profiles appear to be composed of two superimposed profiles; one caused by general ground movements, the other from local instabilities.

In bouldery ground, it is often difficult or impossible to fill a void left from the removal of a boulder at the periphery of the tunnel. This void creates significant and irregular settlements if it collapses before it can be grouted. Boulder removal must frequently be executed in front of the cutting edge of the shield. This action reduces the available soil support and potentially increases the magnitude

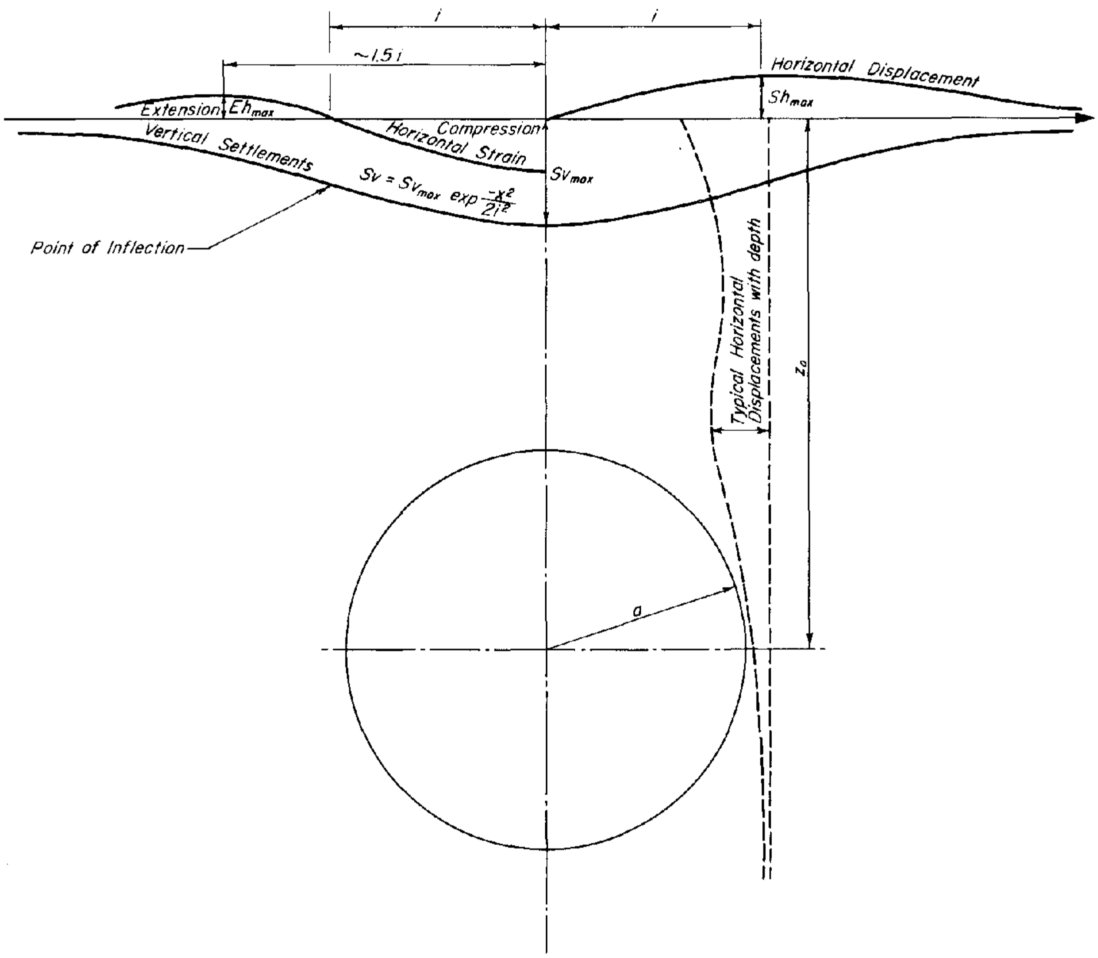


Figure 2.2. Idealized Distribution of Ground Movements due to Tunneling

of general settlements and, consequently, the risk of instability and associated catastrophic ground loss.

2.11 Stresses and Strains in Tunnel Liners

When a liner ring is erected in the shield tail, it is subjected only to loads from its own weight, yet it usually experiences a slight squat due in part to play in the joints. The first significant stresses come from the shield propulsion jacks. These may be more severe than any other stresses the lining will ever experience. As the shield moves forward, the lining is exposed to the tail void, and the loading of the liner depends on the timing of soil collapse, pea gravel and/or grout filling of the void, and the pressure of the grout. If a later secondary grouting is performed, the loading changes at that time. Since grouting is never performed uniformly, the loading of the lining is quite irregular. In very weak soils, the load will quickly approach a condition of relatively uniform radial pressures, approximately equal to the overburden pressure. In stiff, cohesive soils, long term reconsolidation will eventually (after months or years) lead also to relatively uniform radial loads often approximating the overburden pressure. At some depth in granular soils, the load may be somewhat smaller than the overburden pressure, controlled by construction procedures and arching of the soils above the tunnel. Load changes also take place when air pressure is returned to normal in a compressed air tunnel, or when the groundwater rises again after dewatering.

Each load change brings about changes in compressive stresses and sometimes also in the moment distribution. However, except in occasional instances where large voids or other irregularities have been left behind the lining to produce large unbalanced loads, overstressing due to excessive moments is highly unusual. An aging process takes place in cohesive soils which causes distortion of the lining (vertical diameter decreases, horizontal diameter increases), while, barring unusual external influences, linings in granular soils generally retain their shape.

One external influence is of significance: the driving of an adjacent tunnel. This is particularly important since most rapid transit tunnels are driven in pairs. When the two tunnels are closer than one-half to one diameter clear, the driving of the second tunnel increases the load on top of the first tunnel and at the same time removes horizontal support. Sometimes, for this reason, the first tunnel is temporarily supported with braces or tierods.

2.12 Shafts and Cut-and-Cover Construction

Open excavations produce ground movements somewhat similar to those produced by tunneling. While a reduction of settlements due to tunneling is often difficult and uncertain, there are many options available for reducing ground movements produced by open excavations. Loads and stresses in the supporting structural members in an open cut are more significant than those in tunnel liners. A typical open excavation begins with the installation of soldier piles (H-piles driven or placed in pre-bored holes to a depth below the excavation floor or socketed into rock). Utilities crossing the line of the soldier piles are usually exposed and relocated as needed before the placement of the piles. Typical processes and effects of excavation are indicated in Fig. 2.3. As excavation proceeds, timber lagging is placed between the soldiers to retain the earth, and when excavation has reached a point beneath the first strut level, walers and struts are placed. These struts may be prestressed to minimize ground movements. Before the struts can be placed, however, the cantilevered soldier piles will move toward the opening. This movement is associated with a relaxation of the original soil pressures on the back, and ground settlements. The movement is generally unavoidable but can be minimized by: (1) using very stiff soldier piles, (2) placing the first strut very high, and (3) prestressing. The

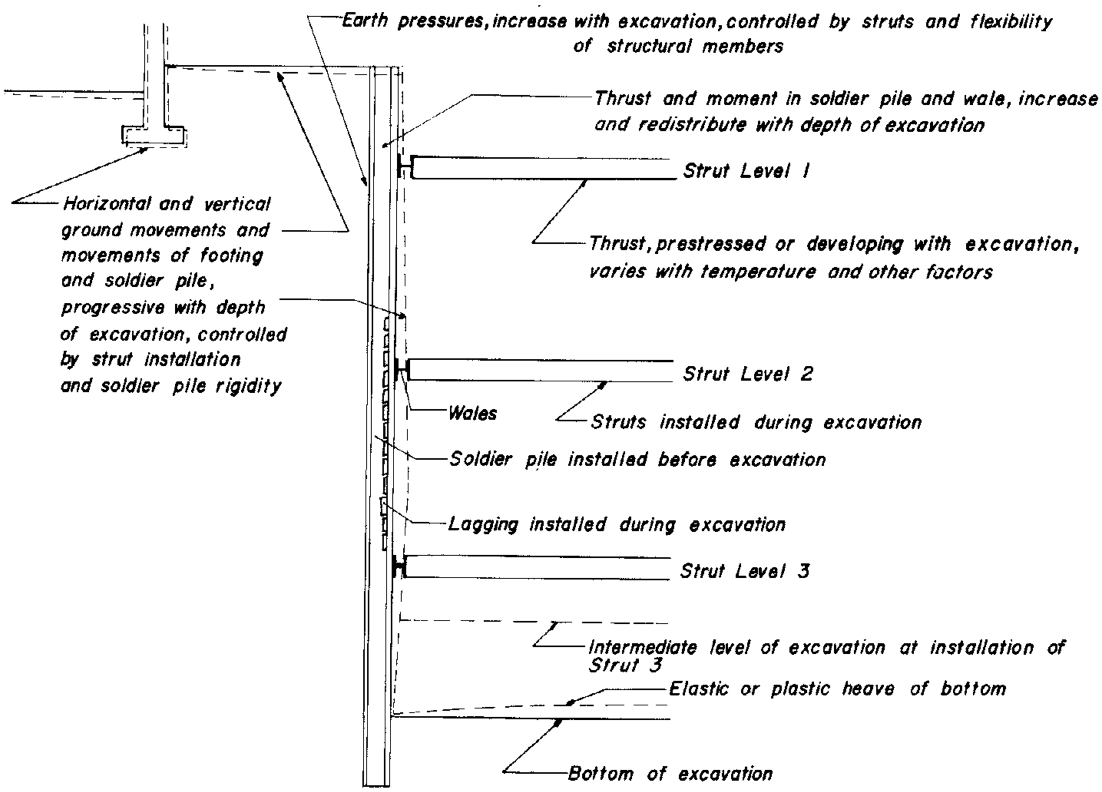


Figure 2.3. Processes in Cut Excavation with Soldier Piles

primary functions of prestressing are to eliminate the play in the connections and to eliminate the movements caused by elastic shortening of the strut.

As excavation proceeds below the next level of struts, but before these struts can be placed, additional stress redistributions on the back of the wall occur. The piles tend to bend inward, but are restrained by the upper strut and the earth below. Vertical and horizontal ground movements occur, moments develop in the soldier piles and walers, and the strut load changes. A prestressing of the lower strut will change loads and stresses throughout, but cannot generally be counted on to reverse movements that have already occurred. Excavation below the second strut will repeat these processes at a deeper level. The horizontal effect range of movements increases with the depth of excavation.

The bottom of the excavation tends to heave slightly due to the removal of overburden. In competent soils this rebound heave is elastic and nominal; in cohesive soils, the heave may be subject to a time delay. In softer cohesive soils, where the ratio of overburden pressure to shear strength is high, plastic displacements occur, and there is a risk of plastic bottom failure for very high overburden/strength ratios. These bottom heaves contribute to settlements behind the wall.

If the groundwater level is not lowered, waterflow through the timber lagging may erode the soil, leaving local voids and contributing to settlements. Without groundwater control, there is a risk of creating quicksand conditions, boils, or similar problems as the bottom of the excavation proceeds below the groundwater table. The factors controlling these effects are similar to those controlling face stability in tunnels. The groundwater level is usually drawn down by dewatering, unless soft strata exist in the environment which would compress intolerably due to dewatering.

A watertight wall consisting, for example, of steel sheets or a slurry wall is often employed where extensive dewatering cannot be performed. These walls must, of course, be designed for the water pressure. Where settlements cannot be tolerated close to the wall, a rigid slurry wall is often used.

It is often economically advantageous to eliminate cross-lot bracing, thereby allowing much easier access and more working room within the excavation. Instead of struts, earth or rock anchors are used. Their interaction with the wall and the soil is similar to that of the struts, except that the supporting force is inclined and the support is generated by stressing soil materials in the anchor zone, far behind the wall. Through prestressing of these anchors, much of the elastic deformation may be eliminated, but the anchors exert a vertical downward force on the wall, in some instances leading to excessive vertical wall displacements when the end bearing capacity of the wall is insufficient.

2.13 Safety Hazards

A study of accident statistics for heavy construction and tunneling (Schmidt, et al, 1974) shows that more than 80 percent of all accidents are due to errors in judgment or carelessness, rather than to adverse conditions. In tunnels, a large proportion of accidents is associated with muck handling. In deep excavations, a large proportion is due to materials handling. However, accidents caused by adverse geotechnical conditions are frequently severe and costly, primarily because they are often associated with severe construction delays.

In soil tunnels, the most frequent accidents generated by adverse or unsafe conditions are associated with face and roof instability. These include incidents caused by boulders or blocks of soil falling from a temporarily unsupported roof or the upper part of the face. The severity and occurrence rate of these incidents increase with the tunnel diameter. The risk is significantly increased when it is necessary to work manually in front of the excavator, in particular when excavation in front of the protective shield is required, for example to remove boulders or other obstructions (piles, rubble, etc.) that cannot be handled by the excavator and mucking equipment.

Face instability due to excessive hydrostatic head or groundwater flow is quite common, but fortunately in most instances is not dangerous to the workers. However, if a severe face blow does occur, the consequences can be severe. On infrequent occasions, a face blow can inundate and bury the entire tunnel excavator and shield and several hundred feet of tunnel or more in a matter of seconds or minutes; the hazard to tunnel workers is obvious.

Whenever face instability results in major ground loss, there is a possibility of burial of equipment and men, but other effects are also of severe consequences. Utilities above may, and frequently will, rupture, resulting in communication losses of various kinds. The rupture of water, sewer or gas lines severely aggravates the problems. Chimneying to the ground surface creates a severe hazard to surface traffic, and widespread settlements associated with major ground loss may severely impair existing structures within the zone of influence. Although major and catastrophic face or roof instability, ground loss, or face collapse are relatively rare occurrences, whenever they do occur they are extremely hazardous and costly for the tunnel workers and the tunneling operation, and for the surface and subsurface environment.

Another relatively rare type of accident is explosion due to accumulation of natural gas, or gas from leaking or ruptured gas mains. The most recent severe gas explosion accidents occurred in a sewer tunnel under Lake Erie at Port Huron (Mich.) and in a water tunnel at Sylmar (Calif.), with a total loss of 39 tunnel workers. Such accidents, though rare, are extremely costly. The occurrence of natural gas is geologically determined, and usually local experience will indicate the likelihood or risk of encountering gas. Gas is potentially more hazardous in tunnels that are unlined or lined with pervious liners than in tunnels lined with tight segments; hence, they are more hazardous in rock than in most soil tunnels, where the gas can only enter at the face. In a compressed air tunnel, the air overpressure tends to displace the gas away from the tunnel.

In braced, open cuts, severe accidents are occasionally caused by boulders or soil falling out of the excavation wall, or by bottom instability. The occasions are rare, but such incidents can be bothersome and cause significant delays. More often, severe accidents are caused by buckling of struts, pull-out of anchors or kick-in of the toe of the wall. There is almost always an inherent risk built into the support system of an open excavation. Struts are loaded in compression, and buckling failure caused by excessive strut loads most often occurs with little or no warning. Moreover, the system is without redundancy; if one strut buckles, or one earth anchor snaps, loads are distributed to adjacent members that are not usually capable of carrying these extra loads. The failure is therefore often progressive, and an entire wall can collapse if just one strut or anchor fails.

Excessive strut loads are sometimes caused by faulty design or overly optimistic earth pressure assumptions, but as often are caused by the buildup of water pressures that exceed design parameters. Kick-out of the toe of the wall may be caused by insufficient embedment of the wall below the excavation, in conjunction with a long vertical lower span of the wall. This is usually the result of careless design. On occasions, toe kick-out is caused by a loss of available passive pressure against the toe, due for example to upward water flow gradients (quicksand-like conditions). A relatively common kick-out failure occurs when a wall socketed into rock above the excavation floor is not properly tied back. The rock cannot usually be counted on to provide hold-back or end bearing in this situation without assistance from rock anchors or similar means.

2.14 Effects of Ground Movements on Existing Structures

Settlements and horizontal displacements caused by tunneling and deep excavations are generally unavoidable but can be minimized by proper construction procedures and care of construction. The ground movements can be classified in three categories in accordance with their general magnitude and their basic causes (see Schmidt, 1974).

The first category of ground movements consists of those movements that must be considered unavoidable under the given soil conditions and with the inherent capabilities of a selected construction method.

The second category of ground movements are those that result from locally improper but controllable selection of construction details. Examples are: too low air pressure, too late application of grout filling of the tail void, overexcavation beyond the leading edge of the shield, etc. Another ground movement in this category is caused by local soil weaknesses that result in minor runs or flows or squeezing of soil into the face or the tail void.

While the ground movements of the first category are relatively uniform with reasonably constant soil conditions, those of the second category are localized; they constitute the peaks of normally observed settlements. These peaks are, in general, considered normal but are to a certain extent avoidable.

Ground movements of the third category are those associated with major or catastrophic ground loss, primarily through the face. These are most often caused by the unanticipated encounter of a permeable soil of low cohesion, that is waterlogged with a substantial head and reservoir of water. Less commonly, major ground loss is caused by unanticipated weaknesses in the soil (local loss of cohesion in otherwise cohesive soils, abnormally low strength of clay) or by gross misjudgment of the soil character. These gross ground movements may be widespread, or may chimney directly to the surface. Such movements are sometimes extremely hazardous and costly as indicated in the previous section, and are always greater than anticipated.

During the design process for an urban tunnel, the designer must estimate the likely settlements along the tunnel centerline, and the likely range of discernible or significant settlements, considering first the settlements of the first category. To do this, the designer must know the pertinent soil conditions and anticipate the contractor's procedures. On the same basis, an estimate must be made of the magnitude of the second category, peak settlements and their likely frequency. Considerable experience is required for such estimates, and recorded case histories form the most valuable source of experience data. Settlements of the third category are generally to be avoided, but the designer must assess the risk of experiencing such extraordinary settlements and weigh the possible consequences against the cost of fully insuring against such risks.

2.15 Effects of Settlements on Utilities and Structures

Structures and utilities directly above the tunnel are subjected to several types of distress during the process of tunneling. Utilities that are roughly parallel to the tunnel are first subjected to extension, beginning several tens of feet in front of the shield's leading edge; then, as the shield moves by, to recompression, as the soil settles. With uniform longitudinal settlements, the utility eventually is unstressed but moved vertically downward. An additional lateral movement occurs when the utility is off the centerline of the tunnel.

In the same situation, a building structure is in addition affected by the longitudinal curvature of the settlement profile. A building of appreciable length along the centerline will first tend to split open at the top as it rides the hump of the settlement profile, but soon after, the tendency will be to close such cracks. The building will finally become essentially unstressed but settled. The extent to which a building or utility withstands such massaging depends on the relative size of the structure, its flexibility and ductility, and its strength in shear and horizontal tension.

Utilities running across the tunnel are subjected to permanent extension along the flanks of the settlement trough, and compression over the center of the tunnel. In addition, a permanent sag occurs following the settlement trough profile, as well as possibly some temporary or permanent lateral movements.

Building structures are typically located a short distance away from the tunnel right-of-way, as when a rapid transit tunnel follows the center of a street. Such buildings will settle in front, stretch, and frequently be subject to extension at the top, when they are located on the flank of the settlement trough. The rear of such buildings tends to remain immobile. These buildings are also usually subjected to shear stresses and displacements, particularly when they are tied together so that extension at the top does not occur.

Most buildings can withstand a good deal of strain without structural damage or even without noticeable cracks. The sensitivity of different types of buildings to differential settlements and curvature of settlement profile has been the subject of several analyses, primarily based on empirical data (see MacDonald, 1956; Feld, 1965; D'Appolonia, 1971; Grant et al, 1974).

Behind a retaining wall for an open excavation, the soil movements have quite similar effects. The settlement profile behind a retaining wall is generally less curved than that over a tunnel. Consequently, any buildings in the zone of influence are subjected to differential settlements, stretch and shear rather than curvature.

Even if a structure rests on deep foundations (piles or piers), it may suffer deleterious effects of ground movements. Settlements of the soil surrounding the deep foundations generate downdrag forces that may cause settlements of the structure. Most deep foundations are vertical and offer little resistance to horizontal displacements. Buildings on piles can therefore suffer damage due to horizontal extension even if they do not settle appreciably.

Based on settlement analyses and estimates, assessments of the settlement's effect on structures, utilities and other environmental effects, and risk analyses, the designer can optimize a tunnel design to include the most cost-effective combination of positive environmental protection and measures to minimize settlements and hazards. Positive protection against deleterious effects may include underpinning, protective walls or soil stabilization, utilities relocation, or even acquisition of endangered structures, all of which are relatively expensive. Protection may also be accomplished through preventive measures. Such measures might include: (1) increased efforts to determine a priori the relevant soil and groundwater conditions and their possible effects, (2) exclusion of certain types of construction details prone to generate excess movements, and (3) increased control over the contractor's procedures and workmanship. Monitoring of construction will allow evaluations of the contractor's performance to be made during construction and will provide the necessary data to predict tunneling effects further along in the construction. Monitoring will also provide data for enforcement of changes in construction details.

The approach to provide positive protection rather than preventive measures to preclude deleterious effects is conservative and usually safe but costly. It has been favored by authorities in many cities for political reasons and for reasons of distrust of preventive measures. In this research and development work, the second approach (preventive measures) is emphasized, and ways to make this approach safe and attractive are analyzed. Monitoring of tunnel construction is a key effort in this endeavor.

Chapter 3 will analyze in greater detail the specific needs for, and potential benefits of, tunnel construction monitoring. A number of examples and conditions will be examined and described but it is not possible, except in a general way, to cover all conceivable benefits of monitoring under a variety of conditions.

3. BENEFITS OF TUNNEL CONSTRUCTION MONITORING

3.1 Introduction

Tunnel construction monitoring has a great number of benefits and a few disadvantages. Depending on point of view and character of association with a tunnel project, different benefits carry different weights, and the priorities are not fully clear unless a specific point of view and philosophy is assumed. While owners and designers may find appreciable benefits, contractors may regard monitoring with considerable reservation. This is primarily because it would appear to interfere with and thus retard construction, but probably to a greater extent because it appears as a threat to his relative freedom of operations. The contractor's aversion to monitoring is to a great extent flavored by his inherent conservatism and the unknowns associated with unique approaches. The contractor knows the currently accepted "rules of the game," and while they do not necessarily lead to economical and equitably executed contracts, they are at least understood. Introducing new rules always introduces uncertainties that can only be resolved gradually through experience. To make up for these uncertainties, contract documents and formats must be very carefully thought out and prepared so that there is no doubt where all duties and responsibilities lie.

These problems are dealt with later on, but first the potential benefits of monitoring are described. These various potential benefits are not necessarily presented in the order of importance. Depending on the specific conditions, it may not be possible to take full advantage of several of these benefits.

3.2 Reduction of Tunnel Construction Costs

It is not always possible to measure the direct economic benefits of monitoring. The benefits may lie in unincurred cost overruns due to accidents, face instability, excessive ground movements, or decreased production rates. These benefits, though intuitively significant, cannot easily be demonstrated. When the benefits accrue in an obvious fashion during the design stage, they are measured more easily.

The most significant measurable cost benefit is the elimination of positive protection of existing structures and relocation of utilities. Monitoring cannot by itself provide protection but it provides a data base for making intelligent and cost saving decisions.

Traditionally, underpinning of a structure is more often executed because of the uncertainties in estimating settlements and their effects than because of the calculated effects of estimated settlements. Such underpinning may in many instances be eliminated. Assume for example that a building is located on the flank of the anticipated settlement trough, and category one settlement (see Section 2.14) of the building extremity is estimated to be one inch. In theory, no underpinning would be needed but there is a slight possibility that minor crack repairs may be needed. This building would usually be underpinned or otherwise protected at a cost of perhaps $150,000, because: (1) the settlement estimate is regarded as very uncertain, and (2) it is not politically attractive to expose private property to risk.

Through proper analysis one may determine the probability of creating category two settlements of about 2 inches and the probable associated repair cost may be estimated. Further, the slight probability of incurring major category three settlements may be estimated, and the effect of such settlement assessed in terms of hazards and rehabilitation costs.

If the risk of category two settlements occurring is 30 percent, and the repair cost is $50,000, the potential statistical cost is $15,000. If the risk of category three settlements is four percent and the rehabilitation cost is $500,000, then the potential statistical cost is $20,000. It is evidently advantageous to eliminate underpinning.

The trouble is that these risk analyses are extremely uncertain, and are based on assumptions regarding the contractor and the capabilities of his procedures, made at a time when the contractor is not even selected. However, if the effects of tunneling are monitored at an early stage of construction, predictions can be made through careful analysis and extrapolation of real and applicable data. Thus, as soon as monitoring data are available, the risks can be reassessed intelligently, and several options be made available. If excessive ground movements can be diagnosed and shown to be caused by specific construction details and procedures, the contractor can modify these procedures, and the effect of the modifications can be ascertained through continued monitoring. If excessive ground movements must be ascribed to ground conditions more adverse than anticipated, and construction modifications cannot be counted on to reduce the predicted settlements, then a decision may be made to execute underpinning.

In this last instance, it is not usually possible to finish the underpinning at the originally anticipated cost; also a delay in tunneling must be expected. The underpinning may cost \$200,000 instead of \$150,000, and two week's tunneling delay may cost another \$250,000, for a total of \$450,000. This is three times the originally estimated underpinning cost. Clearly, underpinning should be performed before construction begins unless there is a greater than 70 percent probability that underpinning will not be required. The Underpinning Design Chart shown in Fig. 3.1 formalizes this approach.

To determine if a deferred decision is appropriate, it is necessary to estimate costs of underpinning (CUP_1 on the chart), construction monitoring (CM), and repairs and rehabilitation if needed (CR_1, CR_2, CR_3). Recognizing that a deferred decision will, in some instances, result in a need for underpinning, the cost of that underpinning, including any delay costs, must also be estimated (CUP_2). To perform the risk analysis, probabilities of occurrence must be estimated (P_1, P_2, P_3), including the probability (P_4) that monitoring will in fact indicate the need for underpinning. A monitoring program is then required at the beginning of the project to verify magnitudes of settlements and probabilities of occurrence, so that the deferred decision may be made. At the second decision point, the term CR_nP_n refers to the largest of CR_1P_1, CR_2P_2 and CR_3P_3. The final verification of P_4 is made after executing the monitoring program at which time field values replace estimated probabilities.

While this entire risk analysis and decision sequence may appear complicated, it is not much different from the sequence of thoughts the owner and designer must intuitively go through in any case where a decision on underpinning or protection must be made. When used in this way, monitoring is an integral part of both design and construction. Construction documents must be carefully prepared to allow deferred decisions of this nature in an equitable fashion.

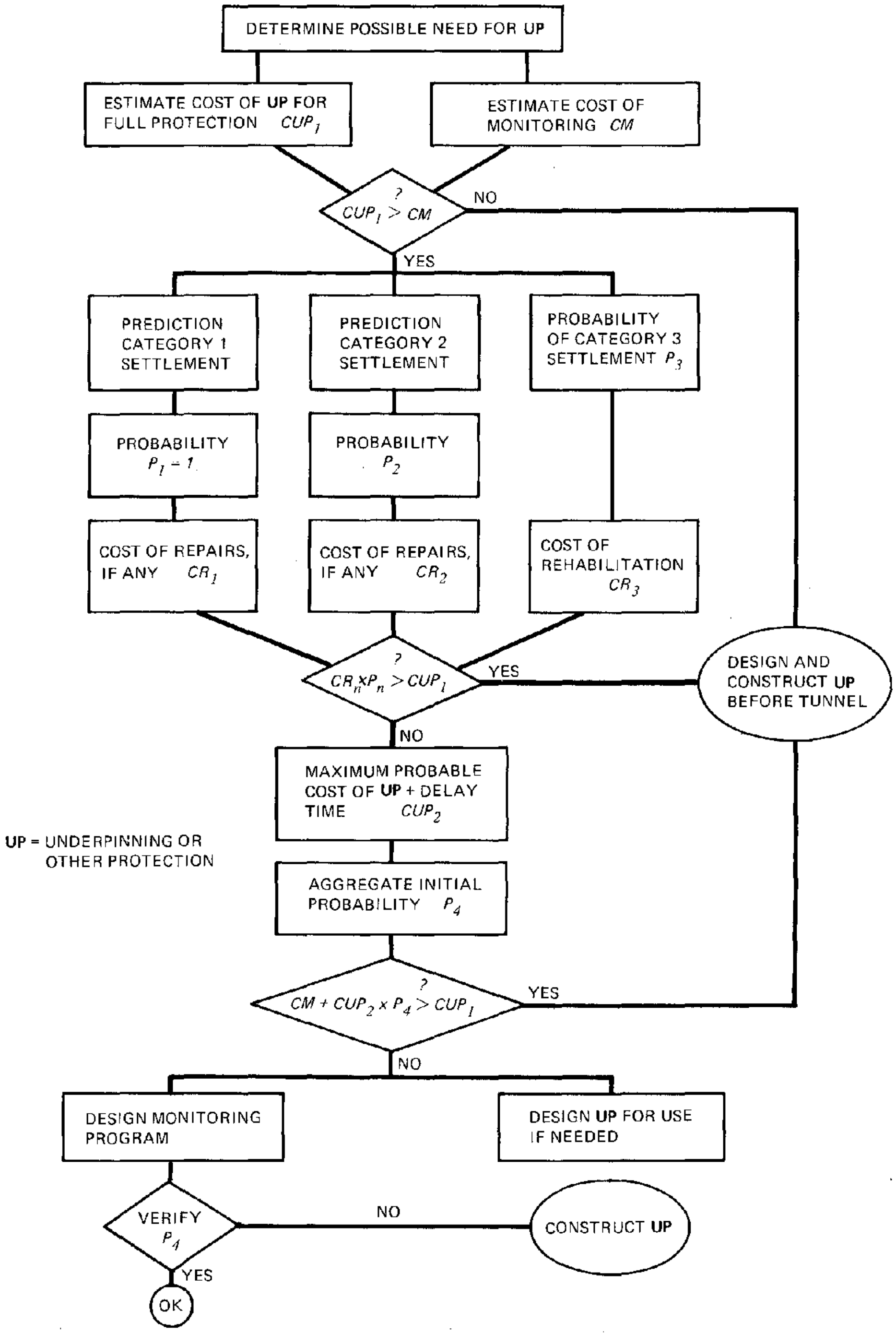


Figure 3.1. Underpinning Decision Chart

A recent example from the Washington Metro illustrates the use of this principle. Near the HUD Building, four single track tunnels of the Branch Route and the L'Enfant-Pentagon Route will pass directly beneath the bridge carrying 7th Street over I-95. The footings of the pier and the two abutments of this bridge will be only about 8 feet above the tunnel crown. Underpinning of this structure would be extremely difficult and would also cause appreciable hazard and deleterious movement. The cost of protection by grouting is estimated to be about $1,500,000. Yet the risk is small that the pier and abutments would become unserviceable due to tunneling settlements. At a comparatively small cost, the bridge deck can be carried on shims, which are continuously adjusted by jacking to preserve its integrity. In accordance with the procedures outlined above, the solution to this problem would be to include the grouting as a separate item in the construction document, with the provision that it be deleted if settlements predicted on the basis of monitoring data indicate that the risks to the structure are indeed nominal.

Elimination of underpinning is a tangible potential benefit of monitoring. Significant cost benefits, though less tangible, accrue from reduction of environmental effects and safety hazards, improved documentation and project control, and future design and construction improvements as described in the next sections.

3.3 Reduction of Environmental Effects

The most significant environmental effects that may be reduced by implementation of monitoring results are those associated with ground movements. Secondary effects are associated with lowering of the groundwater level. To reduce ground movements it is necessary to know their extent and their cause. Monitoring is, therefore, required to establish their general magnitudes, to check their acceptability, and to diagnose the causes of ground movements.

By careful interpretation of data it is possible to determine quite precisely the origin of ground movements at the leading edge of the shield, in front of the shield, or over the tail void. It is also possible to learn the horizontal extent of the origin of movement — whether localized over the crown, spread over the width of the tunnel, or wider. These data coupled with visual observations of tunnel construction procedures and events can accurately pinpoint the specific mechanisms of ground loss. Armed with this information, the contractor can change his construction details (though not his general procedure). At least the following options for modification are usually available, depending on the general procedure employed:

1. Adjust the air pressure in a compressed air tunnel to provide better support of soft clay or to counterbalance hydrostatic pressures.

2. Increase the efficiency of dewatering or temporarily halt construction until the groundwater level is at an acceptable elevation.

3. Institute or increase the use and effectiveness of face breasting.

4. Apply ground stabilization locally (grouting, freezing).

5. Prevent overexcavation in front of shield.

6. Increase jacking pressure on fully covered wheel excavator.

7. Apply tail void filling sooner and more frequently to reduce time of exposure of soil in the tail void.

8. Apply second stage grouting at higher pressure, to fill voids possibly left open.

Depending on the contractor's procedures many other options may be available. All of these modifications may be instituted at moderate cost and would serve to minimize settlements. They are, however, frequently not required to maintain tunnel safety and construction progress. The contractor, therefore, would have incentive to institute them only under cost penalty or specification requirement.

3.4 Increased Safety

Many of the effective controls for cost reduction and environmental protection also have a beneficial effect on safety. The risk of material falling out of the face, for example, is reduced by proper face breasting. Potentially the most severe hazard is the innundation of the tunnel by a massive soil and water flow. The risk of such occurrences can be virtually eliminated by proper surveillance of the groundwater pressures in the tunnel region, and by proper dewatering when called for.

Monitoring of groundwater pressures, or groundwater levels, serves to reduce or eliminate the hazards of major and minor soil and water flows into the tunnel and to check the efficiency and adequacy of dewatering procedures. Monitoring of water pressures immediately at the crown of the tunnel before the leading edge reaches the point of monitoring is particularly important in this respect, but to preclude the possibility of encountering water-bearing seams within the height of the tunnel, observation wells extending throughout the height of the tunnel are also useful. Depending on the general continuity of strata, these observation wells do not necessarily have to be located within the horizontal extent of the tunnel.

3.5 Verification of Design Assumptions and Adequacy of Structures

As a rule, the design of the finished structure is executed in the design phase and fixed in the contract documents. There are no opportunities for changing the design of segmented tunnel liners during construction, primarily because the lead time for delivery of such items prohibits significant changes. Even when the final lining is cast-in-place concrete, the thickness of the concrete is determined a priori, and the shield diameter is selected to accommodate the necessary dimensions. A change to a thinner lining during construction would result in minimal savings. Where there are initial uncertainties with regard to the adequacy of, for example, an innovative lining system, monitoring may be performed to verify the predicted behavior of such systems. There must, of course, be recourses established for the case where the systems are found to be inadequate. In general, instrumentation of the final lining structure will benefit only future construction projects, but it is highly useful for this purpose.

On occasion, erected tunnel linings are subjected to excessive distortion due to improper application of tail void grouting, very soft soils, irregularities within the soils or voids left behind the lining. Where such distortions persist, temporary tie-rods may be required to hold the shape of the lining. Measurement of immediate distortion and monitoring of distortion with time will help in determining these needs.

Where twin tunnels are driven close together, the driving of the second tunnel often creates distortions in the first tunnel, and it may itself become distorted. Monitoring is usually required to determine these effects and the needs for tie-rods.

3.6 Documentation and Project Control

As previously explained, there is a need for monitoring of ground movements, groundwater pressures and tunnel distortions, to diagnose causes of deleterious effects and to implement changes in construction procedures. They allow the contractor to modify procedures on his own initiative, and allow the owner's representative to enforce such modifications, provided the construction documents give him the power to do so.

For most types of structures, a verification of dimensions, an evaluation of the general quality and appearance of the final structure constitute the most important criteria for the owner's acceptance of the final structure. In the case of tunneling, it is apparent that the criteria for acceptance include many items directly associated with the manner in which the contractor performs his work. The contractor's performance has a direct effect on the final product, but the concern for safety and environment, plus specific criteria established for decisions concerning underpinning or utilities relocation, make the contractor's performance throughout construction subject to quality control. Construction monitoring of the types discussed constitutes a significant part of the project quality control. Construction documents must, in general, include criteria for acceptability based on monitoring results. Conventional construction documents usually include such criteria only to a very minor extent.

Monitoring serves additional purposes beyond project control and acceptance. Monitored settlement data of adjacent buildings and utilities are important legal documents and are extremely useful during later litigation and resolution of insurance claims. The amount of legal fees that may be saved by expediting these matters with the availability of adequate data can be significant.

3.7 Monitoring of Deep Excavations

The preceding paragraphs dealt with problems of bored tunnels. Most principles and concerns, however, are equally applicable to monitoring of deep excavations. The following discussion centers around problems and monitoring benefits related only to deep excavations.

Temporary retaining walls for deep excavations are designed for reasonably conservative earth pressures and groundwater level assumptions. The distribution of earth pressures, and the movements associated with the excavation, are quite uncertain, however, and monitoring is frequently called for to ensure the compliance with assumptions regarding water levels, and to minimize ground movements.

Cost savings may on occasion accrue by the monitoring of wall movements, settlements and strut loads. If wall movements are considerably smaller than expected, or the effect of strut prestressing better than anticipated, it is sometimes possible to eliminate some or all of the lower struts. Alternatively, the vertical spacing of struts may be increased, thereby not only reducing the cost of strutting, but allowing more efficient operations due to the improved working space. At the same time, monitoring of strut loads provides a measure of safety. Since buckling of a single strut or snapping of a single anchor can have catastrophic effects, monitoring of loads in some of these members allows the detection of dangerous overloads.

3.8 Data Collection for Improvement of Design and Construction Procedures

In the past, most monitoring of tunnel construction, and a great deal of deep excavation monitoring, have been performed to gather data for advancing the state of the art. In many instances, when monitoring was expected to benefit a project directly, this benefit was not fully achieved for reasons previously discussed. However, these data, when subjected to later analyses, have increased the understanding of soil behavior, soil structure interaction and the effects of tunneling and deep excavations.

The concepts discussed in the previous sections have been directed toward the optimization of monitoring benefits to the project from which the data are collected. Most of these data will be extremely useful also for improving future designs and construction contracts, particularly those executed in the same geological setting; for example, future contracts of the same transportation system.

Many concepts of soil behavior applicable to tunnel construction are reasonably well understood at this time. However, the owner's, designer's and contractor's confidence in predictions of soil behavior based on these concepts is limited for the following reasons:

1. The quality and quantity of geotechnical data (stratigraphy, soil properties) usually available for analysis are insufficient for proper correlation with empirical data and for thorough theoretical analysis.

2. The concepts and methods of analysis have never been fully tested by comparing predictions based on full analyses with measured performance.

3. The great variety of construction procedures has never been subjected to a full and critical review with respect to their effects on soil behavior in tunneling.

4. Recorded experience of soil behavior is limited to several reasonably well documented histories, many of which are not easily accessible, and most of which have not been subjected to analyses using modern concepts.

Due to the vagaries of tunneling and the peculiar effects of tunneling on soil behavior, theoretical work can only provide a background or framework, providing approximate functional relationships between parameters, causes and effects. No theoretical result can be trusted until verified and modified by empirical data. Because of the importance of being able to predict tunnel behavior in advance, it is of great importance to gather monitoring data intelligently and to make these data available for research.

Some of the most important concepts that require accumulation of data from experience are listed below. These concepts are not necessarily listed in order of direct importance. Consideration has also been given to the beneficial value that tunnel construction monitoring data may have on the development of these concepts, and also to the plausible short-range development possibilities.

1. The effect of construction details and procedures on soil behavior, especially ground movements. Very careful recording of construction details, is required, to correlate associated ground movements at depth and at the surface, as well as horizontal displacements.

2. The distribution of horizontal displacements due to tunneling, and their effects, as a function of type and magnitude of ground loss. The results of 1 above, are required, in addition to direct observations. The distribution of vertical displacements (settlements) as a function of ground loss is already fairly well known.

3. The effect of loosening or densification of soil on settlement profiles. Settlement data at multiple elevations are required.

4. The effects of construction details and soil properties on lining distortions. Monitoring data on distortions are required as a function of construction procedures and time.

5. Development and verification of new procedures for designing dewatering systems for tunnels. Pumping data, groundwater observations and observations of groundwater problems in tunnels are required.

6. Development of finite element computing methods for prediction of ground movements and lining stresses. Major problem is determining boundary condition as function of construction procedures, and appropriate soil properties. All data in 1 through 4, above are required.

7. Development of more appropriate and economical methods of tunnel lining design. Data on stresses and lining distortion are required.

The following are also required for deep excavations:

1. Improved analyses of beneficial effects of wall rigidity and prestressing of struts or anchors. Wall movement measurements, settlements, strut or

anchor loads are required.

2. Development of finite element methods for design of temporary and permanent walls (see 6, above).

Use of empirical data for research and development purposes requires that all appropriate input is available. For example, data from monitoring instrumentation are of limited value unless pertinent construction data and geotechnical data are also available for use in the analyses. Monitoring instrumentation should be planned and used only in the overall context of the research objective.

3.9 Alignment Control During Construction

Ultimately, the line and grade of the tunnel, and the tunnel clearances, are subject to control and acceptance by the owner. In the San Francisco BART System, the following method was used (Peterson & Frobenius, 1973). After completion of a tunnel, permanent centerline monuments were placed in the tunnel at intervals of approximately 1000 ft., and at TS (tangent-spiral) and SC (spiral-circular) curve points. From these monuments, measurements were taken radially to critical clearance points to ensure that the clearance envelope was in accordance with design requirements. In one stretch of the BART System, measurements were made on all rings to ensure that all clearance requirements were met, and to provide data for recalculation of track alignment if needed.

During construction, monitoring of lines and grade, other geometrical properties of erected lining rings, and shield attitudes, aid the contractor in steering the shield and in guiding erection procedures. Since time delays caused by survey work cannot be tolerated, it is in the contractor's interest to develop efficient methods to transfer tunnel centerline, stationing and grade from primary control monuments to the tunnel, to carry these forward through the tunnel, and to control the shield and lining erection operations within tolerances. Inefficient methods may cause delays. Inaccurate methods may cause ultimate rejection of parts of the finished tunnel.

Each tunneling setup imposes different requirements to and restrictions on, methods of line and grade control. In modern tunneling, laser beams are most often employed both for control of shield attitude and for verification of tunnel ring geometry. It is unnecessary to describe these methods in detail in this report. The methods are described in other works (see, for example, Peterson & Frobenius, 1973).

4. A SYSTEMATIC APPROACH TO TUNNEL CONSTRUCTION MONITORING

4.1 Introduction

No single monitoring instrument, monitoring parameter, or step in the monitoring process can be viewed alone. To be effective, a tunnel construction monitoring installation must have purpose and coherence, and also must be durable and accurate. To gain maximum advantage from the installation, the monitoring data must be read and put to use in accordance with a plan. In other words, the complete system of monitoring must be considered, beginning with the initial decision to consider monitoring as a construction or research tool, and then including the implementation of results from interpretations of monitoring data.

Instrumentation for monitoring the performance of tunnels involves engineers in more potential pitfalls than many other endeavors requiring instrumentation. Other types of instrumentation often entail the use of off-the-shelf hardware to make measurements in a controlled environment. While this may pose difficulties, the procedure is in general straightforward. Consequently, many engineers and scientists think of instrumentation as being merely the hardware aspect. In contrast to measurement in a controlled environment, tunnel monitoring is performed in extremely adverse and unpredictable surroundings. Major adverse factors are variable temperature and humidity, dirt, risk of damage by construction equipment or by vandals, and lack of care, understanding or interest on the part of construction personnel. Good engineering practice requires attention to many factors in addition to the hardware itself.

A systematic approach to tunnel construction monitoring, from conception to full implementation, includes at least the following tasks or steps: (1) definition of benefits and purposes; (2) design and project specification to draw full benefits; (3) selection of monitoring parameters; (4) selection of instruments and writing of specifications for procurement and installation; (5) procurement and installation; (6) reading and maintenance; (7) data storage and retrieval; (8) interpretation; and (9) implementation.

Some of these steps are fairly easy and can be resolved through the use of established theories and technologies. The difficult ones are those that require the bending of minds, those that involve changes in the ways engineers design soft ground tunnels, the way owners pay for tunnels, and the way construction is managed, inspected and controlled.

The chart shown on Fig. 4.1 describes the interaction and chronology of the various steps in the systematic approach to tunnel construction monitoring. All of the subsystems must operate properly for the complete system to succeed; just one weak subsystem may break the linkage and make the monitoring effort founder. A check list summarizing the most important concerns involved in the planning and execution of a monitoring program is presented in Appendix C. Section 4.6 very briefly identifies some of the most significant weak links in the monitoring system.

4.2 The Planning Process

The planning of a monitoring program is a systematic process, and should proceed along the lines indicated in Appendix C. Failure to address any one of the topics listed in the Appendix C check list will often result in a monitoring program which does not achieve its purpose. An early step is the identification of the potential benefit of monitoring.

In the ideal situation where a tunnel is to be driven through cohesive granular soil of very stiff clay, deep in an area with few or no adjacent structures or utilities, and where groundwater is not a problem, it would be hard to justify construction monitoring except for research purposes and alignment control.

Where soil conditions are less than ideal, such as in soft clay, cohesionless sands, or below the water table, monitoring may serve two purposes: safety in tunneling, and assistance to the contractor in expediting his work and avoiding delays. The first important monitoring parameter is the groundwater pressure.

Where buildings, utilities or other structures exist within the tunnel's zone of influence, but tunneling conditions are excellent, underpinning or other protection may not be employed, and monitoring of ground movements often is required to verify the original assumptions regarding the effect of tunneling and to serve as legal documents.

Where existing structures are within the zone of influence of the tunnel, and tunneling conditions are poor, but some underpinning decisions have been deferred (see Chapter 3), a full range of instrumentation should be installed.

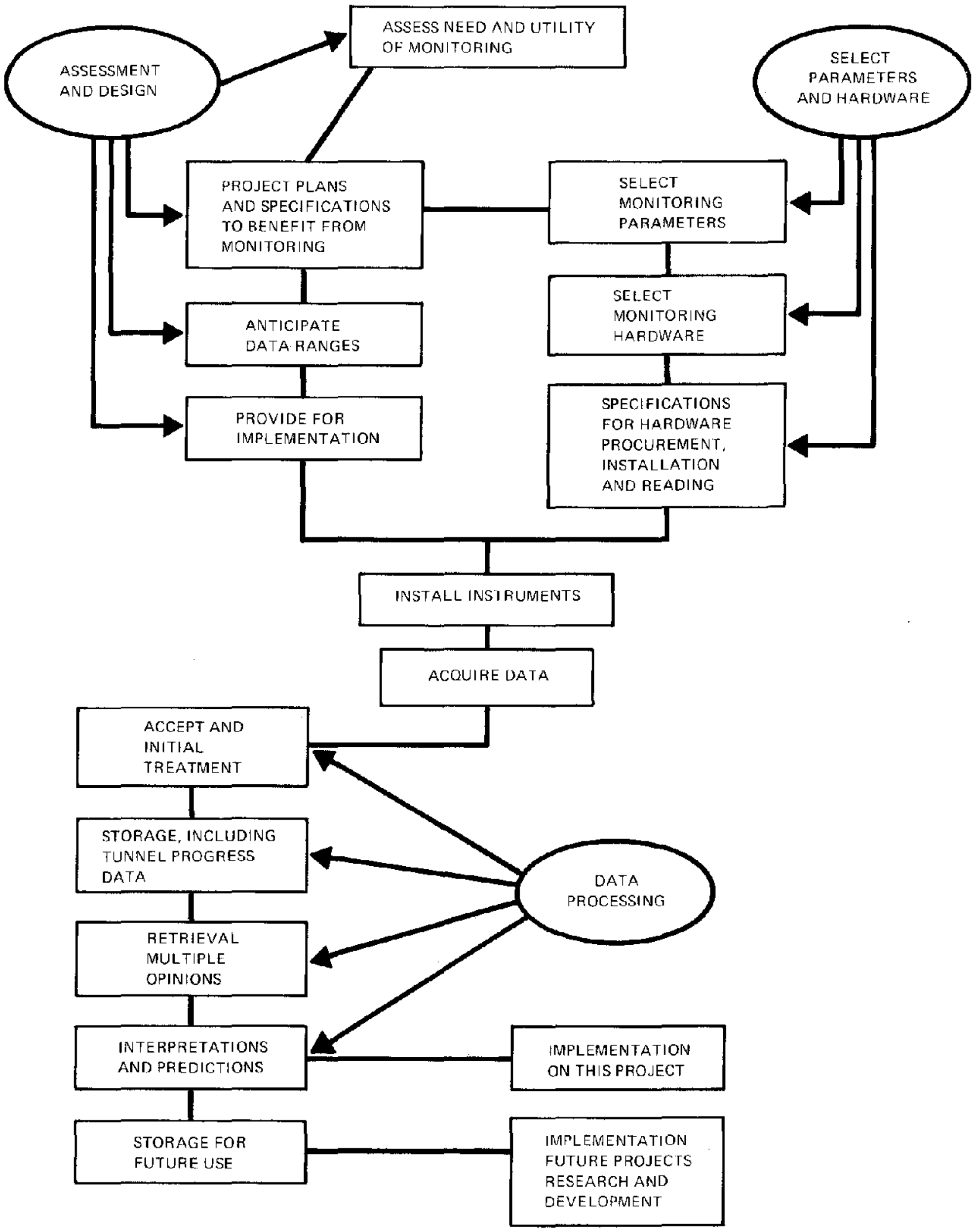


Figure 4.1. Systematic Approach to Tunnel Construction Monitoring

If, in the same situation, decisions have been made to underpin and protect conservatively at the outset, the utility of full range monitoring is reduced. It then serves primarily to enhance safety and economy of the tunnel driving itself. Similar arguments may be presented for deep excavations in urban areas.

It is, in theory, possible to evaluate the cost effect of tunnel construction monitoring. In practice, however, this is exceedingly complicated in the general case, and it may be quite difficult also in the specific case, where the benefits are primarily risk reduction and improvements of contractor's rate of progress. Such evaluations become a little more manageable where specific items may be avoided or salvaged, such as items of underpinning (see Chapter 3). No instances have been found where detailed analyses of this nature have been employed to evaluate the need for, or the benefit of monitoring. Instead, monitoring decisions have been made on a purely subjective basis -- or monitoring has been employed for research purposes with incidental benefits to the project on which monitoring was employed.

Once it has been determined on an objective or subjective basis that monitoring will be of benefit, the next steps are to select parameters to monitor, and to employ concepts of the observational approach to the designs of the tunnels, and the specifications. Parameter selection is covered in Section 4.3, but in this section the effects that the monitoring decision should have on the design and the specifications are described.

Since most tunnel construction monitoring in the past has been for research purposes, explicitly or implicitly, little attention has been paid to the maximization of monitoring benefits through proper designs and specifications. Other types of earthwork, in particular earth dam and embankment work, have benefitted from these concepts. In many instances, appropriate monitoring has been the only way to resolve uncertainties. Commonly, embankments are built on soft ground in staged construction where the construction rate and sequence is fully dependent on monitoring results. Unfortunately, the concepts are not so clear for tunnel construction monitoring.

The design of the tunnel structure itself (i.e. the lining), is not significantly influenced by monitoring results. However, monitoring results strongly influence decisions of what to underpin or relocate. During design, it must be determined which underpinning decisions can be deferred until construction, pending the outcome of monitoring data. Once decisions have been deferred, the contents and wording of specifications become critical, and the instrumentation program becomes an integral part of the design and specifications.

The primary concerns of the specification writer relating to monitoring are concerns that must be shared with the design engineer and the owner. These concerns are the following:

1. Distribution of responsibilities between owner, engineer and contractor.

2. Incentives encouraging quality workmanship, based on monitoring data.

3. Provide inspecting engineers with adequate powers to enforce construction modifications based on monitoring data.

4. Provide performance criteria by which the contractor's performance will be judged, and which will be used as a basis for deferred decisions and for forced construction modifications.

5. Provide for appropriate payment items for all anticipated construction items, including those related to deferred decisions, and including, for example, ordered downtime per diem.

6. Examine the expertise requirements for instrument installation, reading and interpretation.

7. Procurement and installation specifications for instrumentation.

A more detailed discussion of these items would be included in Manual 1.

4.3 Selection of Monitoring Parameters

The selection of tunnel construction parameters that should be monitored for a given project depends on the soil and groundwater conditions, the configurations and relative locations of existing structures, the distance between adjacent tunnels, and the nature of deferred decisions. The parameters may be categorized as groundwater conditions, ground movements, observations on existing structures, soil-structure interaction data, and progress data.

<u>Groundwater Conditions</u>. Wherever tunneling or deep excavations extend below the groundwater table, the water table should be monitored to warn against possible instability of tunnel face or excavation bottom, to check the efficiency of dewatering procedures, to determine the compressed air requirements, and for several other purposes. The following parameters may be considered:

1. Monitoring of groundwater level at tunnel centerline by:

    (a) well open throughout entire depth from ground surface to below tunnel invert

    (b) well open to one or several identified aquifers

    (c) well located at critical point near and above crown

    (d) well open throughout height of tunnel.

2. Monitoring away from centerline, (a) and (b) above.

3. Monitoring of dewatering progress, i.e. time history of pumping volumes and drawdown in pump wells.

4. Monitoring of water infiltrating tunnel or excavation; associated soil flows, if any.

<u>Ground Movements</u>. These may be monitored for legal or diagnostic purposes or to provide data for predictions of ground movements at other locations.

1. Surface settlements. May be measured using simple surveying tools. They frequently interfere with surface traffic, but generally are very useful. These measurements are:

    (a) settlements along centerline of tunnel, which are taken as periodical measurements near location of shield

    (b) settlements at points away from centerline, preferably along lines at right angle with centerline.

2. Subsurface settlements require installation of instrument. These measurements are:

   (a) settlement of ground directly above tunnel crown; most useful diagnostic measurement

   (b) settlement variation vertically above tunnel crown (multiple settlement points in vertical line)

   (c) settlements of deep points away from centerline (for research purposes primarily).

3. Surface horizontal displacements and strains may be monitored by simple surveying tools. They interfere with surface traffic, but are useful particularly for correlation with building damage and for research. These measurements are:

   (a) displacement along line at right angle with centerline (see 1a)

   (b) displacements along centerline (interesting but usually less useful)

   Strains may be calculated from displacements or may be measured directly (requires instrument installation).

4. Subsurface horizontal displacements require instrument installation, but are a useful diagnostic tool in tunneling and in particular in deep excavations. These measurements are:

   (a) horizontal displacement of excavation wall, or soil directly behind

   (b) horizontal or radial displacement of soil adjacent to tunnel, at right angle to centerline.

   (c) horizontal displacement of soil parallel to centerline toward tunnel face.

5. Tail void encroachment is the rate of soil filling the tail void; useful for diagnostic purposes.

6. Ground heave such as bottom heave of excavations and

heave of ground at tunnel invert; occasionally useful in certain soil conditions.

Observations on Existing Structures. Existing structures are monitored for legal and insurance reasons, for the record, and for research purposes. Correlations can be made in this way between soil displacements and building damage. Observations which can be made are:

1. Before-and-after observations, including elevations and displacements, crack surveys, photographs, etc.

2. Settlements monitored during construction such as settlements of peripheral walls and settlements of interior columns.

3. Horizontal displacements and strains can be directly or indirectly measured.

4. Tilt measurements of building walls and floors. These have been rarely used in the past but are potentially effective because quick-acting tilt meters are now available.

5. Strain in utilities is of limited use except when utilities are truly continuous.

Soil-Structure Interaction Monitoring. This type of monitoring is directly useful for projects involving deep excavations, and is primarily useful for research in the case of bored tunnels.

1. Structural stresses in braced walls, such as loads in struts, loads in tiebacks, and moments in walers, soldier piles, sheet piles or walls. Loads in struts and tiebacks are particularly important for safety and economy.

2. Distortions of tunnel linings are measured to verify that distortions will remain within tolerances and to determine need for temporary tie-rods. Measurements of interest include: increase of horizontal diameter, and lowering of crown (invert usually not accessible until long after installation).

3. Earth and water pressures on walls and linings. In tunnels, this is of use primarily for research and development. Frequently useful in deep excavations,

particularly behind slurry-walls.

4. Stresses due to circumferential thrust and moments in tunnel linings are primarily of use for research.

5. Earth pressures within the soil mass are measured for research only.

Progress Monitoring. To analyze cause and effect, it is necessary to maintain a complete record of relevant construction data together with other monitoring data. This construction record will also contain information needed for geometrical control during construction, and for steering. The data required for correlation include at least the following:

1. Stationing and time at beginning of each shove (ring) for reference.

2. Attitude of shield relative to theoretical attitude.

3. Position of shield relative to theoretical position, vertical and horizontal.

4. Push data, start and stop of each shove, pressure and jacks used.

5. Curvature of tunnel (from design drawings) to estimate theoretical ground loss due to plowing.

6. Incidences of extraordinary ground losses, observed distress, and other unusual events.

7. Environmental factors which may affect monitored data, for example, temperature or nearby construction activities.

For deep excavations, progress monitoring should include at least the following:

1. A record of depth of excavation versus time, at close stationing.

2. Time of installation of all walers and struts, with preload records, if any, and depth of excavation below strut at time of installation.

3. Incidences of extraordinary ground losses, observed distress, or any other unusual events.

4. Complete as-built construction plans and records, including, for example, records of soldier pile driving.

5. Environmental factors that may affect monitored data, for example, temperature or nearby construction activities.

Systematic Guide to Parameter Selection. A systematic guide for the selection of basic monitoring parameters for various construction conditions is presented in Tables 4.1 and 4.2. This guide, however, is grossly simplified. It is difficult to include all possible tunnel construction conditions in a simple table. Local practices, regulations and labor conditions often influence the selection of monitoring parameters.

Horizontal Extent of Monitoring. In previous paragraphs, the term "zone of influence" appeared repeatedly. It is clearly of importance to determine how far away from construction activities it is necessary to monitor their effects. The cost of monitoring increases with the area that must be covered, yet monitoring must be carried out to the distance where significant effects are likely to be found.

On the basis of case histories, the probable limits of significant effects may be found. Some of the basic findings were described in Chapter 2, and probable limits have been extracted from a number of case histories (see Schmidt, 1974). Recommended zones of influence in various conditions would be presented in Manual 1; therefore, only a brief summary is presented in this report.

Under ideal conditions, where ground loss is evenly distributed, and no loosening or densification of the soil occurs, the shape of the subsidence profile is well defined. The magnitude of settlements is directly proportional to the ground loss, and with a definition of significant ground movement in terms of absolute magnitude (e.g., 1/4 inch), the range is well defined. Densification of overlying soils, and the possibility of horizontally widespread ground loss (for example, from horizontal soil-water flows) both tend to widen the range, while loosening of overlying soils and tendencies toward predominant roof or topface instability tend to concentrate settlements. Thus, an appraisal of soil and water conditions will allow an estimate of the significant range.

Table 4.1. Selection of Monitoring Parameters for Bored Tunnels

Soil Conditions / Water Conditions		Ideal soil, Cohesive Granular, Stiff clay, No boulders	Soft Clays or Silts	Cohesion-less sands	Many Boulders and obstructions
Above groundwater	A	1	--	1	1
	B	1	--	1	1
	C	1 or 3	--	5	4
Below groundwater; no water control conplated	A	1	2	--	2
	B	1	3	--	2
	C	2 or 3	3 or 4	--	4
Below groundwater; compressed air	A	--	2	2	2
	B	--	3 or 4	3 or 4	4
	C	--	5	5	4
Below groundwater; dewatering or other control	A	2	2	2	2
	B	2	3 or 4	2 or 3	2
	C	3	5	5	4

<u>Legend For Status Of Nearby Structures</u>

A: Structures outside zone of influence.
B: Structures inside zone of influence, underpinned.
C: Structures inside zone of influence, not underpinned or underpinning decision deferred.

<u>Legend For Parameter Selection</u>

1: No monitoring required; or spot settlements only.
2: Groundwater monitoring; spot settlements.
3: Groundwater if appropriate; surface settlements; structures monitoring.
4: Groundwater if appropriate; surface and subsurface settlements; possibly surface horizontal displacements, structures monitoring.
5: As 4 plus subsurface horizontal displacements; tail void encroachment; lining distortion; possibly also temporary and permanent stresses in lining and earth and water loads in lining-employed for limited lengths only or for research.

--: Not applicable.

<u>NOTE</u>: The above guide is a gross simplification, and selection must always recognize specific needs and constraints of each project. Progress monitoring is required in all cases, and should include observations of ground movement from within tunnel and other factors listed in text "Progress Monitoring".

Table 4.2. Selection of Monitoring Parameters for Cut-and-Cover Tunnels

Water Conditions \ Soil Conditions		Ideal soil, cohesive granular, stiff clay	Soft clays or silts	Cohesion-less sands
Above Groundwater	A	1	--	1
	B	1	--	1
	C	1 or 3	--	4
Below ground-water; no water control contem-plated	A	1	2	--
	B	1	3	--
	C	2 or 3	5	--
Below ground-water; dewatering or other control	A	2	2	2
	B	2	3	2
	C	2 or 3	5	4

Legend For Status of Nearby Structures

A: Structures outside zone of influence.
B: Structures inside zone of influence, underpinned.
C: Structures inside zone of influence, not underpinned or underpinning decision deferred.

Legend For Parameter Selection

1: No monitoring required; or spot settlements only.
2: Groundwater monitoring; spot settlements.
3: Groundwater if appropriate; strut or anchor loads; spot settlements.
4: Groundwater if appropriate; strut or anchor loads; surface and subsurface settlements and horizontal displacements of structures, soil and supporting walls; possibly stresses in walls and earth and water pressures on walls.
5: As 4 plus bottom heave.
--: Not applicable.

NOTE: The above guide is a gross simplification, and selection must always recognize specific needs and constraints of each project, and the conservatism of temporary support design. Progress monitoring is required in all cases, and should include all factors listed in text "Progress Monitoring".

4.4 Selection and Installation of Instrumentation Systems.

Having selected the significant monitoring parameters, the ranges and time frames of monitoring and other basic monitoring requirements, the next step is to select the monitoring system. A monitoring system includes the instrumentation components, layouts and installation plans. Methods and criteria for selecting appropriate instrumentation systems are presented in Chapter 5.

Before procurement and installation specifications are written, delegation of responsibilities must be determined for procurement, installation, reading, maintenance and implementation. In addition, the necessary types of personnel must be made available to perform the various tasks. Many design and consulting firms and most contractors have limited experience with geotechnical instrumentation for tunneling purposes. Complex instrumentation is, therefore, often best left in the hands of specialist firms. Depending on the complexity of the instrumentation and the character of the problems, numerous contractual setups are feasible. For this purpose, the instrumentation tasks may be separated as shown in Table 4.3, which also shows several types of contractual setups.

In referring to Table 4.3, note that the inspecting engineers very often would be the same as the design engineers. It is highly productive for the execution of the design intents regarding monitoring, that the design engineers carry out at least the interpretation and implementation tasks.

The instrumentation specialist crew may be part of the design engineer's staff if adequately qualified. The specialist may instead be hired by the design engineer or, rarely, by the owner. Very commonly, he may be hired by the contractor.

The parameters whose monitoring is of prime interest to the contractor should in general be left to the contractor to monitor, either as a lump sum item included in other items, or separately with unit costs. The last method is usually preferable, because with unit prices for hardware and service hours, additional work can easily be requested by the engineer or the owner. These parameters would normally include all parameters associated with location and direction control, but might include groundwater parameters (because groundwater may be a significant problem for the contractor) and even ground movements and settlements if the contractor has been given responsibilities to minimize these at penalty.

Table 4.3. Monitoring System Responsibilities

Task	Design Engineers	Contractor	* Specialist	Inspecting Engineers
Design Instrumentation System, Specifications	X		X	
Furnish Instruments		X	X	
Install Instruments		X	X	X
Maintain Instruments		X	X	X
Read and Report		X	X	X
Amend, Interpret, Implement			X	X

* Note: * Many tasks possibly paid for as consulting services.

Ordinarily, however, groundwater and ground movement monitoring are executed in the owner's interest and should be carried out at no cost penalty to the contractor. To avoid conflicts, the specialist may well be hired and administered by the contractor. However, since the contractor has little control over the instrumentation specialist's actions (additional work may be ordered by the owner's representative), the cost of the specialist cannot be entered as a bid item. Instead, a fixed cost may be entered in the bid by the owner (designer), and the contractor only adds a markup. This method, or variations thereof, is fairly common in Europe, but not in the United States. For the method to be fair and equitable, the contractor must be given full information in the bid documents about the nature of all monitoring, so he can judge the possible interference with his work.

The actual payments for instrumentation work would, with this arrangement, be in accordance with invoices from instrument suppliers and instrumentation specialists, with the contractor's markup added. In this way, the contractor, while still retaining suppliers and specialists with his team, has no interest in procuring the least expensive equipment and services. Thus, quality performance is more likely to ensue. It is even possible for the owner (designer) to select or prequalify instrumentation specialists and to pre-select all instruments.

The inspecting engineers, as a rule, must verify grades and dimensions, and perform other surveying duties. For this reason, all surface settlement and horizontal displacement data are usually most appropriately obtained by the inspecting engineers. Deep settlement points, inclinometer casings, piezometers and the like should usually be installed by the instrumentation specialist and may be read either by him or by the inspecting engineers.

As far as possible, measurement taken within the tunnel itself, near the working face, should be taken under the contractor's auspices to avoid construction delays. Location and displacement or distortion measurements are logically made using the same basic surveying setup arranged for alignment control. The inspecting engineers, however, must supervise this data collection and must be empowered to request additional measurements at a fair price.

If monitoring measurements are taken in addition to the regular types of measurements (stresses in linings, etc.) for research purposes, these measurements would normally be taken by the researchers' staff. Special arrangements must be made for taking these measurements in accordance with the specific scope of the research, funding details, etc.

Maintenance of instruments and replacement of defective instruments will, in most instances, be the responsibility of the contractor, who will ordinarily execute this responsibility in collaboration with the instrumentation specialist.

4.5 Data Processing and Implementation.

Tunneling is a dynamic process. The tunnel face moves ahead at a rate between 20 and 80 feet per day, and the effects that are monitored may be experienced in a matter of hours. Hence, frequent readings must be taken in concentrated areas, and once the face has passed through an area, effects not properly measured are irretrievably lost. Therefore, emphasis must be placed on the reliability and accuracy of the data collection methods.

Because of the dynamics of the process, monitoring data do not particularly benefit the specific area where the data are gathered. Once the effects of tunneling have occurred, they are generally irrevocable. The data must be employed to benefit the tunneling process along the line, basically in two ways. First, the data must be used as diagnostic tools, to analyze the efficiency and adequacy of the contractor's plant and methods. These methods include: dewatering, face support, grouting behind lines, and other items, by their effects on the surrounding ground. Second, they must be used to predict effects that may influence utilities, structures and adjacent tunnels at later stations.

There is only a limited time available to perform these diagnoses and predictions. It is therefore essential to employ an efficient, premeditated process of data storage, retrieval and interpretation, that will allow expedient implementation of monitoring results. In the past, manual data processing has been relied upon, and implementation procedures have been inefficient both because of time lag and because of the lack of clear courses of action shown by the data.

4.6 Weaknesses in Current Practices

The following paragraphs describe certain weaknesses that prevail in current state-of-the-art practices in tunnel monitoring. In addition, suggestions for improvement are included.

Planning and Utilization of Monitoring. By far the greatest deficiencies are associated with the proper planning and use of construction monitoring, rather than with the instrumentation or the methods of installation, reading and maintenance. These deficiencies are described in the following paragraphs.

1. The owner and his engineers frequently have little appreciation of the potential benefits of monitoring. Their personal past experiences often do not include a significant number of tunneling projects, and the profession's aggregate experiences include only few tunneling projects where monitoring was successfully employed.

2. The designers have little incentive, when not prodded by the owner, to draw all the possible benefits of monitoring, when "safer" conventional procedures exist and have been accepted in the past, though such procedures may lead to more costly and, perhaps, less satisfactory results.

3. The specification writers have little experience in the preparation of specifications that will allow full utilization of monitoring data, guarantee the installation and maintenance of reliable instruments, and produce meaningful data interpretation and implementation with minimum time lags.

4. Tunnel designers have little experience in selecting schemes of monitoring to benefit tunneling, in selecting proper parameters to monitor, and in selecting proper instrumentation to monitor these parameters.

5. Present contractual setups and distribution of responsibilities do not always encourage the production of meaningful, reliable data, the utilization of these data, and the implementation on the project.

6. Interpretation of tunnel construction monitoring data is difficult and requires types of expertise that are not abundant at this time.

To improve the current practices of tunnel construction monitoring, a set of manuals should be prepared outlining the problems and their solutions, and initiating a nationwide program for dissemination of monitoring knowledge to appropriate officials and engineers. The details of this recommendation are presented in Chapter 6.

Instrumentation Hardware, Installation, Data Collection and Maintenance. Instruments are available to monitor nearly all parameters of interest to the tunnel engineer. Improvements are possible in the following areas.

1. Installation practices to produce faster and more efficient installations

2. Instrument accuracy, reliability and durability through minor design changes

3. Reading efficiency by using, for example, modern laser based surveying techniques

4. Proper instrument selection to monitor desired parameters.

These possible improvements are discussed at some length in Chapter 5. Recommendations for a combined extensometer/piezometer are presented in Chapter 8, and specifications for this instrument are presented in Appendix G.

Data Processing. Processing of monitoring data, to be useful for the project, must be systematic, pre-planned, efficient and swift. While the final analysis and decisions on courses of action must involve a good deal of engineering judgment, the handling, storage and display of data are suited for computerization. Such computerized data processing has been employed with limited scope in the past, but never in a systematic way to include a complete series of monitoring data for tunnel construction monitoring. A unified computer data processing system will allow the immediate display of meaningful data combinations in selected time and areal ranges, and certain types of interpretive analyses.

Such a system should be developed to include ground movement, groundwater and tunnel progress data. Later versions may include additional items as the need arises. A description of the proposed computer program and its use is presented in Chapter 7, and a technical specification is presented in Appendix F.

5. INSTRUMENTATION HARDWARE, INSTALLATION, DATA COLLECTION AND MAINTENANCE

5.1 Introduction

This chapter describes the shortcomings of present-day instrumentation. A critical view has been taken of the state-of-the-art of available instruments and techniques used for monitoring the construction of soft ground urban tunnels, but no attempt is made herein to summarize the state-of-the-art in detail.

Methodologies for monitoring various parameters are discussed in turn, including a brief restatement of the purpose of monitoring that parameter, currently available methods, shortcomings, and possible advancements. Possible developments which have not been pursued further during this contract are discussed in some detail in this chapter, and recommendations are made for appropriate future research efforts. Possible developments which have been pursued further during this contract are outlined in this chapter and discussed in detail in later chapters. The selection of emphasis has been made during interim reviews and discussions with the Transportation Systems Center of the Department of Transportation. A summary of possible innovations, together with a priority rating for future effort or research, is included in Section 5.10.

Automatic Data Acquisition Systems. During the early phases of this research effort, some emphasis was placed on automatic data acquisition systems. It was felt that increased safety might be attained at a reduced cost by greater application of automatic systems. However, it was recognized early that other approaches would be more useful, and this effort was discontinued. The following paragraphs briefly describe this effort.

Several geotechnical instrument manufacturers include automatic data acquisition systems in their product lines. The need for such systems arises when the volume of data cannot be handled reliably or economically by manual means. Automatic data acquisition systems are also employed to link engineers with monitored data, either when the site is geographically remote or when the engineers' office is at a location distant from the site.

Engineers responsible for monitoring performance of urban soft ground tunnels will normally be located in the city of construction. Driving rates are such that monitoring at any one station is normally required only for a brief period of time. Vandalism is a major factor in selection of hardware for an urban construction project. All these factors tend to negate the need for automatic data acquisition systems, and consequently their use in urban tunnel projects is very rare. Peck (1970) makes a strong case for minimum automation and sophistication of field instrumentation and states, "In my view, as a broad generalization, if one can with sufficient accuracy make a direct visual observation with a graduated scale, he should not use a micrometer. If he can use a micrometer, he should not use a mechanical strain gauge. If he can use a mechanical strain gauge, he should not use an electrical one. Mechanical instruments are to be preferred to electrical devices and simple electrical devices depending upon simple circuits are to be preferred to more complex electronic equipment. That is, where a choice exists, the simpler equipment is likely to have the best chance for success."

A summary has been made of automatic data acquisition systems available from geotechnical instrumentation manufacturers, and is presented in Table 5.1. These systems generally result from modification and re-packaging of commercially available components. If the need for such a system should arise on an urban tunnel project, it is strongly recommended that selection not be limited to those systems available from geotechnical instrument manufacturers. Instead, they should include specialist manufacturers of electronic systems. Examples of such manufacturers are: Columbia, Consolidated Controls, Data Craft, Datel, Digitec, EMC, Esterline Angus, Flure, Gould-Brush, Hawkeye, Hewlett-Packard, Honeywell, Hybrid Systems, Kay Instruments, Leeds & Northrup, Monsanto, and others.

Two accounts of automatic recording inclinometers are included in the literature. Bromwell et al (1971) describe an instrument developed at M.I.T. which allows measurements to be made as the probe is pulled continuously out of the casing. Data is recorded on IBM compatible tapes; computer programs with plotting routines have been prepared to process the data. Tesch (1972) describes a Bureau of Mines instrument consisting of inclinometer probe, winches and speed controller, over-the-hole sheave device, automatic data acquisition system and calculator. The equipment is operated

Table 5.1. Automatic Data Acquisition Systems Available From Commercial Geotechnical Instrument Manufacturers

Category and Type of Instrument	Description of System	Manufacturer
Horizontal movement gage-deflectometer	Monitor, alarm console and telemetry system.	Slope Indicator Co. Terrametrics.
Torpedo inclinometer-accelerometer type	Various portable systems. Either teletype and data coupler (punched tape later read in tape reader and transferred to magnetic tape, then to X-Y plotter) or magnetic tape unit.	Geotechniques International. Geo-Testing. Slope Indicator Co.
Pneumatic piezometers, wire embedded extensometers with electrical sensors, microseismic detectors (system used at Libby Dam, Montana)	Combination of semi-automatic and fully automatic recording. Data logging system collects and processes data from subsystems and transfers data to teletype, auxiliary paper tape punch or data printout. Piezometers and extensometers selected manually, displayed digitally and then, using a record button, are automatically transferred to the teletype. Microseismic (rock-noise) monitoring system automatically records data from up to 9 of 16 rock-noise sensors. Counting, storage and printout on the teletype of the rock-noise pulses are automatic. Record intervals may be selected.	Slope Indicator Co.

Table 5.1. Automatic Data Acquisition Systems Available From Commercial Geotechnical Instrument Manufacturers (Continued)

Category and Type of Instrument	Description of System	Manufacturer
All categories of instrument using vibrating wire strain gage as basic sensor	Remote reading systems, single or multi-channel generally with receiver, timer, telemetry system with output on strip chart, punched tape, punched card or magnetic tape.	Gage Technique Geonor Maihak Telemac
Pneumatic piezometers	Control unit sequentially selects and reads each sensor and records reading on chart recorder or digital printer.	Geo-Testing Terra-Technology
Hydraulic piezometers	Control unit with low volume change solenoid valves sequentially switches piezometer to a pressure transducer. Reading displayed on digital meter and recorded on digital printer.	Soil Instruments

from within a van truck.

McVey and Meyer (1973) describe successful use of a 10-channel automatic data acquisition system, assembled from commercially available components and environmentally packaged for use with underground resistance strain gage instruments. The system can also be used for any direct current voltage parameter up to 1,000v. Environmental control is provided by a compressed-air type air-conditioning system. Hof (1974) reports on the development by the U.S. Bureau of Mines of a self-contained data acquisition probe. The probe is installed completely within a borehole and is used to measure and record rock deformations around an underground opening. Components are commercially available solid-state devices, and are compatible with linear variable differential transformers (LVDT) or with conventional resistance strain gages.

Soil and Rock Instrumentation, Inc. is currently making measurements of liner stresses at the Port Richmond Intercepting Sewer Tunnel in Staten Island, New York (see Section 5.8). The volume of data cannot be handled by manual means, and a survey has been made of appropriate data acquisition systems adequate for use with 300 resistance strain gages. The selected system consists of four basic units manufactured by Digitec (United Systems Corporation) of Dayton, Ohio; a scanner, clock, digital voltmeter and printer. These units have been assembled into an automatic data logging system capable of monitoring and providing hard copy data of up to 1000 points at selected time intervals.

The four units are:

1. Model 636 Digitec Scanner. The unit is capable of sequentially connecting up to 1000 points (with variable dwell time) to a readout device. The scanner also provides for continuous cycle operation or manual select mode operation.

2. Model 662B Digitec 24-hour clock with a program interval time which can be set for 1, 5, 10, 30 or 60 minutes.

3. Model 268 Digitec digital millivoltmeter capable of resolving 1 microvolt.

4. Model 691 Digitec digital printer, with 21 column

parallel entry drum printer capable of printing 1.5 lines per second.

The scanner, clock and digital voltmeter have been mounted on two 19-inch x 4-1/2 inch high relay rack panels, enclosed in a portable carrying case weighing 75 lbs. The printer is an 11-lb, self-contained unit. This system is currently functioning satisfactorily.

5.2 Groundwater Monitoring

Measurements of groundwater level or piezometric head are used for control of dewatering efficiency and for assessment of tunnel and excavation stability and safety.

Currently Available Methods. Instruments available for measuring groundwater level and pore water pressure are tabulated in Appendix B. The state-of-the-art of available hardware is largely satisfactory, and proven methods are available. Selection of particular instruments depends on many factors related to the specific needs and constraints of each project. However, a general order of preference is given in Table 5.2.

Frequency and Cost of Monitoring. The cost of piezometer installation in a 100-foot deep borehole, including instrument cost, boring and specialist supervision, ranges from $1,500 to $2,500 per installation. Depending on conditions, the number of installations for a 3000-foot tunnel contract could range between 10 and 40. Since most piezometers deployed for tunnel construction monitoring purposes would be placed in relatively permeable soils, and since only relatively long-term water movements are of interest, there is usually no need to use quick-response piezometers.

Improved Installation Procedures for Piezometers. Current piezometer installation procedures are cumbersome, time-consuming and costly. Installation generally includes five basic interdependent aspects: (1) advancing a borehole, (2) supporting the borehole wall, (3) placing the piezometer in the borehole, (4) removal of temporary measures used to support the borehole wall, and (5) filling the space between piezometer and borehole wall with a permanent material.

Table 5.2. Types of Piezometer in Order of Preference

Type	Advantages	Limitations
Standpipe including wellpoints	Simple. Reliable. Long experience record. No elaborate terminal point needed. Heavy liquid version available for reducing response time and overcoming freezing problems.	Slow response time. Danger of tubing buckling. Tubing must be raised nearly vertical. May create traffic interference and danger to reading personnel. Freezing problems.
Pneumatic	Level of terminal independent of tip level. Rapid response.	Must prevent humid air from entering tubing.
Vibrating wire strain gage or semiconductor pressure transducer	Level of terminal independent of tip level. Rapid response. High sensitivity. Suitable for automatic readout.	Expensive. Temperature correction may be required. Errors due to zero drift could arise (although most manufacturers have overcome major problems).

Significant economy can be achieved through simple innovative measures. There is a need for a systematic evaluation and updating of installation procedures in an effort to reduce installation costs. Specific aspects which merit attention are:

1. Extent to which boring time can be reduced by use of hollow-stem augers. Rapid boring with a hollow stem auger is possible in many soils, and enables instruments to be installed through the stem while the auger itself supports the borehole wall.

2. Extent to which boring time can be reduced by use of wire-line drilling techniques. This technique was developed for deep rock drilling and has recently been introduced into soft ground subsurface investigations. The drill rod also serves as casing to support the borehole while wash water circulates down the inside of the rod and up between rod and borehole wall. Sampling tools are lowered within the drill rod on the end of a steel cable, and latch within the rod at the base section just above the bit. When samples are not required, a roller bit replaces the sampling tool. Major problems for soft ground use arise in uncemented granular materials, which tend to wear out the bit attached to the drill rod, and to blow in while the sampling tools are withdrawn.

3. Support of a borehole by using mud rather than casing nearly always results in faster and less expensive instrument installation. However, mud cannot be used to support holes in all soil types and conventional bentonite drilling muds cannot be used for supporting piezometer boreholes. Use of these muds would entail filling the soil voids and adversely affect piezometer response. Most piezometers are therefore installed in cased boreholes, with consequent large cost. Degradable drilling muds are used during the installation of wellpoints, the mud being degraded at the appropriate time by addition of a catalyst so that soil permeability is unimpaired. Use of degradable muds could, if proven satisfactory during a field research program, greatly reduce piezometer installation time.

4. Improved methods for sealing piezometers in place so that they respond correctly to pore water pressure in the immediate vicinity of the sensor. The three basic methods of sealing are by: (1) pushing directly into the soil, (2) grouting within a borehole, and (3) use of a tamped bentonite backfill. The first method is possible only in soft soils, and relies for its seal on complete contact between soil and pipe above the piezometer. The second method requires careful choice of grout mix, and has on occasion been proven unsatisfactory. The grout must be suitably impermeable and must not be less compressible than the surrounding soil. If the piezometric level may be drawn down, the grout must not shrink if access to water is withdrawn. The third is the most common and proven method, but entails a tedious and costly installation procedure, using "bentonite balls" or "pi-pellets". Typically, a full day is required, using a boring rig, drilling crew and one technician to place and compact an effective, tamped bentonite seal in a 60- to 100-foot deep borehole. The cost would be $500 with boring costs additional to this figure. Basic objectives of research in this area would be to:

   (a) Determine in which soil types a pushed-in piezometer forms an adequate seal, which piezometer shapes seal most readily, and how long the contact between pipe and soil must be.

   (b) Prepare standard specifications based on field testing, for grout mixes for use in various soils (both for use in piezometer seals and for backfilling around other borehole instruments). Various additives should be considered, including polyurethane closed cell foams, gelatin, and bentonite.

   (c) Determine necessary length of seal in various soil types under various piezometric heads.

   (d) Determine which seals create pore pressure increase for a significant period of time due to swell of the sealing material.

(e) Develop more efficient tamped seal techniques than the standard "bentonite ball" procedure, by inhibiting instantaneous swell, by use of a tremie placing tool, or by use of pneumatically or hydraulically actuated tamping methods.

(f) Develop techniques for packaging and installing several piezometers at different elevations in a single push-in probe or in a single borehole.

Such a research program would commence with a literature search and a survey of government agencies (Department of the Army, Corps of Engineers; Department of the Interior, Bureau of Reclamation; Department of Agriculture, Soil Conservation Service; geotechnical instrumentation manufacturers; private geotechnical engineering consulting firms and geotechnical departments of universities) to define the state-of-the-art in this area. The survey should include European experience. The program would, depending on needs as revealed by the state-of-the-art study, proceed to laboratory and field testing phases, and finally to preparation of a brief field manual. A brief survey of the literature has revealed only one paper describing a controlled series of field tests to compare various piezometer seals. Peters and Ellis (1972) describe tests during which they developed a preformed bentonite ring seal; however, the results were inconclusive.

<u>Automatic Monitoring of Groundwater</u>. In order to minimize data acquisition costs it may be desirable to provide for automatic monitoring of groundwater level at selected locations. Automatic monitoring can be performed with off-the-shelf components, but no manufacturer makes a convenient and complete off-the-shelf package specifically for this purpose. A promising measuring system appears to be the purge-bubble principle in combination with an open standpipe piezometer. In employing this principle, a tube is run within the piezometer riser pipe to a known elevation within the piezometer itself. The upper end of the tube is connected to a compressed nitrogen bottle so that nitrogen passes into the tube at a controlled rate, as restricted by a throttling valve, and bubbles up through the riser pipe. The pressure in the gas line is equal to the hydrostatic pressure of water above the lower end of the tube, and is monitored by an appropriate pressure recorder. Substantial use of the purge-bubble principle for measuring water levels has been made by Statham Instruments, Inc. of Oxnard, California.

Development of Groundwater Monitoring Instruments. Piezometers provide a means of measuring the piezometric head above a tunnel crown, and as such are a means of monitoring the groundwater regime above and ahead of the tunnel. Current practice entails making one boring per piezometer, and that boring is not used for a second purpose. Major installation cost results from the boring operation itself. An equally important need exists for monitoring settlement of the soil just above the crown. This is normally achieved by use of a single point rod extensometer installed in a boring. It is believed practicable to pre-package a piezometer and single point extensometer together and install them in one borehole. This technique promises to allow increased use of meaningful instrumentation at reduced cost. A research effort toward the above goal has been pursued, and is described in Chapter 8 and Appendix G.

5.3 Monitoring Ground Surface Settlements

The most common type of tunnel construction monitoring is the measurement of ground surface settlements due to tunneling. Such measurements can be made with little initial installation costs and with abundantly available ordinary surveying instruments and manpower. However, the low instrumentation investment is deceiving. Considering the cost of data acquisition, recording and interpretation, monitoring of surface settlement can be as expensive as other types of monitoring that require costly installations.

Surface settlements are measured along the tunnel centerline primarily near the shield location, and at five to eight points along lines at right angles to the centerline. These measurements provide a basis for predicting settlements further along in the project, and to diagnose sources of lost ground. In cut-and-cover construction, settlements are measured along lines at right angles to the excavation walls. The ultimate purposes of these measurements include reductions of safety hazards and environmental impact, resolution of underpinning problems, project control and many other items (see Section 4.3).

Currently Available Methods. Surface settlements are normally measured by optical levelling techniques from a benchmark. Measured settlements generally range from 0.01 ft to 1 ft. Observation points are marked on the ground surface (often the street pavement or sidewalk) with paint. Gould and Dunnicliff (1971) discuss accuracy of optical levelling techniques. Settlement surveys usually are carried out at second or third order accuracy with maximum error of closure about 0.03 to 0.1 ft in a circuit one mile in length. Sensitivity of the level bubble on ordinary optical instruments restricts the precision justified in rod readings to the nearest 0.005 ft on sights 200 ft or longer. Second order requires limiting sight distances, balancing foresight and backsight, carefully plumbing the rod, reading on well-defined marks and stable turning points. The circuit should be closed on a benchmark and the apparent closing error distributed in proportion to the square root of distance from the starting bench. With careful work it is possible to limit errors in elevation of individual points on the circuit to 0.005 ft (1/16 in.), but accuracy of ± 0.01 or 0.02 ft is more common. Wherever possible measuring points and benchmarks should be observed from one instrument setup. If a benchmark can be read from that station, accuracy can be within ± 0.004 ft.

The benchmark itself has an important function in measurement accuracy. Benchmarks established on substantial permanent structures ordinarily do not contribute error to settlement observations. However, benchmarks placed at shallow depths in soil probably move to some extent, and the movement may be sufficient to interfere with the desired accuracy of a survey. Apart from effects of frost heave and seasonal moisture changes, construction activities may settle a surface bench by subsoil densification from blasting or pile driving, by consolidation from nearby loading or drawdown, or as a result of extension strains directed toward an excavation.

To minimize movement of a bench for precise surveys, a pipe or rod with a drive point should be installed in a borehole carried down to unyielding strata. The central rod should be protected from drag with an exterior casing or the pipe should be coated with a bond-breaking material such as asphalt or oil-soaked waste.

Frequency and Cost of Monitoring. In the beginning of a tunnel contract, settlement points may be spaced as close as 20 ft along the centerline, and cross-sections may be monitored every 40 ft, with up to eight points on a cross-section. On occasion, settlements may be measured after each shield shove (every two to three feet of tunnel progress, or possibly ten times during a three-shift day), but more commonly, settlements may be measured once or twice each shift at all points within 150 ft from the shield location. This amount of data gathering would occupy a survey crew fully during all shifts. Assuming a 30-foot tunnel progress per day, the cost would be of the order of $25 per foot of tunnel, including some plotting and data review. At later stages of construction, the number of observations would be reduced, and possibly only one survey crew would be employed. The cost would then drop to about $10 per foot of tunnel.

Shortcomings in Available Methods. The conventional procedure has several disadvantages:

1. Sighting to a point on a street pavement causes traffic interruption and risk of injury to the survey rodman.

2. The cost of monitoring, including reduction and plotting of data, may amount to $100,000 per mile of urban tunnel.

3. In evaluating ground movement, settlement measurements are required beneath the pavement rather than at the pavement surface. Structural strength of the pavement itself may cause bridging across a settling base course, and surface settlement measurements will give a false record.

In view of these imperfect aspects, an effort has been made to outline a methodology which will enable settlement readings to be taken without traffic interruption, at a lesser cost than conventional survey, and which will overcome the pavement bridging problem. An acceptable method should be capable of measuring settlements up to 12 in. with an accuracy of 0.01 ft.

Possible Innovations. Beneficial innovations in measurements of surface settlement are a reduction in cost and effort (or increased coverage for the same effort), and the limitation or removal of personnel exposure to traffic hazards.

Several laser beam instruments are now available which allow the measurement of elevations quickly and with a one-man crew. A laser beam is set to rotate at a fixed elevation. A rod equipped with a laser-light-sensitive strip is placed vertically at locations where the elevation is desired.

When the beam hits the sensitive strip, the vertical position of the rod relative to the laser beam level is determined electronically and then displayed on a digital display unit held by the rodman. This principle is used in the Gradosite system, (Coherent Radiation Co.) and similar principles are used for the Beacon and Rod-Eye from Laser Alignment, Inc., and the Rotolite Laser and Laser Eye Detector from Spectra-Physics. The accuracy is reported to be within $\pm$0.01 foot, and the range about 1,500 ft.

This or a similar system would reduce the manpower and the time required for monitoring, but would not eliminate the traffic risks though the time of exposure would be reduced. The system is possibly amenable to semi-automation of data gathering since a recording device could be attached to the display unit.

A complete cross-section profile could be obtained from a full profile settlement gage. The various commercially available gages are enumerated in Appendix B. The gage would be pulled across the pavement surface and a record of data automatically made at a terminal unit set up temporarily on the sidewalk, or on the wall of an adjacent building. The overflow type, balloon type and inclinometer are all unsuitable. The overflow type requires the probe to be at the same elevation as the terminal unit, and the reading cycle of the balloon type is too long. Data reduction of inclinometer data entails successive addition of inclinations, and the assumption that each measured inclination is representative of a finite length of the measured surface. Local surface irregularities would render this assumption invalid. Two other types may be suitable, one with a positive pressure transducer at the bottom of a liquid filled hose and the other with a negative pressure transducer at the top of a liquid filled hose. Major aspects requiring study are temperature sensitivity of the transducer itself, change in liquid specific gravity due to ambient temperature change, and design of the liquid tube and liquid so that thermal expansion of the liquid matches that of the tube.

A solution to the pavement bridging problem could be approached in several ways. First, use could be made of shallow subsurface anchor posts as a means of transferring movement of material beneath the pavement to accessible surface points. The post would be sleeved with an outer pipe and fitted with a rapid disconnect cap. Second, targets could be embedded at the bottom of shallow holes drilled to a depth below the pavement, and depth to the targets determined using a mechanical probing rod, or by using electrical resistivity or electromagnetic induction techniques. (e.g., see Weber et al, 1973). Third, a pipe could be buried in the base of a slot sawn across the pavement, and a full profile settlement gage pulled through the pipe.

There are now available several small boring machines capable of drilling small-diameter horizontal holes for installation of cables, pipes or casings, provided no obstructions are encountered. One such boring machine is the Mighty Mole, manufactured by McLaughlin Mfg. Co., which uses percussion drilling techniques. Such equipment may be used to place casings under streets or other inaccessible areas, through which full profile settlement gages could be pulled, without the need for closing for traffic even temporarily. None of the proposed methods overcomes the need to refer every measurement to a benchmark.

5.4 Monitoring Ground Surface Horizontal Displacements

Surface horizontal displacements and strains are measured particularly for correlation with building damage and distress in utilities. These are usually measured at right angles to the tunnel axis or the excavation wall, and less frequently along the axis of the tunnel.

Currently Available Methods. Gould and Dunnicliff (1971) discuss currently available methods for monitoring horizontal movements by conventional surveying techniques. The most common method is to hold a steel tape or scale at right angles across a line of sight between a fixed transit position and a permanent foresight. Accuracy is generally ± 0.005 ft. Alternatively, horizontal movement is monitored by chaining distances with an engineer's steel tape. On level ground the probable error is roughly 1/5000 of the distance. A third method is triangulation. Triangulating over distances less than 500 ft with a base line chained to 1/10,000 and triangle closure within 10 sec yields an accuracy of about ± 0.02 to 0.04 ft in determination of position.

Frequency and Cost of Monitoring. The frequency with which horizontal displacements are measured is highly variable with the conditions. In the past they have been measured almost exclusively for research purposes on bored tunnels. Greater use has been made of such measurements for deep excavations. The measurements would often be made intermittently by the survey crews, as time permits.

Shortcomings in Available Methods and Possible Innovations. Available methods are laborious and costly, and frequently result in interruption to traffic. However, few major innovations appear to be on the horizon. Gould and Dunnicliff (1971) suggest that electronic distance measurements, photogrammetric methods and lasers should find increasing use in geotechnical engineering.

5.5 Monitoring Subsurface Settlement

The purpose of monitoring subsurface settlement is identical to that for surface settlement (see Section 5.3). However, by monitoring settlement just above the tunnel crown information is gained much earlier than possible from surface measurements, thereby enabling more effective remedial action. Furthermore, measurements of surface settlement alone may mask the real origin of lost ground and result in inappropriate remedial action.

Currently Available Methods. Instruments available for measuring subsurface settlement are tabulated in Appendix B under "vertical pipe settlement gages" and "embedded extensometers". Further details are given by Dunnicliff (1971), and accuracy of available methods is discussed by Gould and Dunnicliff (1971). The state-of-the-art of available hardware is largely satisfactory, and proven methods are available. Selection of particular instruments depends on many factors related to the specific needs and constraints of each project. However, a general order of preference is given in Table 5.3. If a subsurface settlement gage is installed over a tunnel crown, measurements must always be referenced to a benchmark and therefore a survey crew is required. If a subsurface settlement gage is installed alongside a tunnel it is often possible to set an anchor at a depth below the tunnel invert to serve as a benchmark. A survey crew is then not required and substantial economy will result.

Frequency and Cost of Monitoring. Single point gages may be required along the centerline every 20 or 40 feet in the beginning of a project, and every 100 to 500 feet later. On a given project, only a few multiple point gages may be used. The cost of installing a single point gage in a 100-foot deep borehole, including instrument, boring and special-

Table 5.3. Types of Subsurface Settlement Gage, in Order of Preference

Type	Advantages	Limitations
Measurements Required at Single Point		
Anchor post or single point embedded rod extensometer with mechanical readout.	Simple. Reliable.	Requires manual access to read (may create traffic interference and danger to reading personnel).
Single point embedded rod extensometer with electrical readout.	Rod preferable to taut wire. Can be read remotely without traffic interference.	More prone to malfunction, damage and vandalism than mechanical readout.
Measurements Required at Several Depths		
Magnet/reed switch vertical pipe gage.	Anchors follow pattern of settlement without falsification. Simple and reliable.	Requires manual access to read (may create traffic interference and danger to reading personnel).
Multi-point embedded rod extensometer with mechanical readout.	Simple. Rods preferable to taut wires.	Rods can hang up within surrounding sleeves if many anchors in one hole, thereby falsifying readings. Requires manual access to read (may create traffic interference and danger to reading personnel).
Multi-point embedded rod extensometer with electrical readout.	Can be read remotely, without traffic interference.	Rods can hang up. More prone to malfunction, damage and vandalism than mechanical readout.

NOTE: All these instruments may require temperature correction, either due to thermal straining of the measuring linkage or thermal sensitivity of electrical sensors.

ist supervision ranges from $1,000 to $1,500. Figures for a 5-point gage range from $2,000 to $3,000.

Shortcomings in Available Methods and Possible Innovations. Major shortcomings in available methods are: (1) tedious and expensive installation procedures, and (2) inadequate procedures for check calibration in place. These shortcomings and possible innovations are discussed below.

1. Improved Installation Procedures. As for piezometer installation (see Section 5.2), many current procedures for installing subsurface settlement gages are cumbersome, time consuming and costly. Installation generally includes five basic interdependent aspects: advancing a borehole, supporting the borehole wall, placing the gage in the borehole, removal of temporary measures used to support the borehole wall, and filling the space between gage and borehole wall with a permanent material. Specific innovations which merit attention are:

    (a) Extent to which boring time can be reduced by use of hollow stem augers or wire line drilling (see Section 5.2).

    (b) Updating use of drilling muds for support of boreholes in soft ground. Many large exploratory drilling firms employ mud engineers, whereas the site investigation contractor who installs geotechnical instruments usually has limited expertise in use of drilling muds. If proper techniques are employed, use of a mud-supported borehole rather than a cased borehole nearly always results in a faster and less expensive instrument installation. Settlement gage installation should benefit from technology transfer in this area.

    (c) Preassembly of various parts of the hardware to minimize installation time. Most instrument installation requires use of a boring rig and crew at an average cost of $400 per day. Instrument preassembly minimizes rig standby time and can create significant cost savings. A good example of installation cost reduction by prepackaging of a magnet/reed switch vertical pipe settlement gage is outlined by Taylor (1974). This instrument consists of a series of ring magnets placed at different levels within a borehole, an access pipe passing up the borehole through all the magnets, and a probe unit connected to an electrical cable and buzzer. The probe unit contains a reed switch that, when lowered down the access pipe, closes at a repeatable position within each magnetic field and activates the buzzer. Distance to each magnet is noted from cable graduations, and hence a record of subsurface settlement is obtained. The device was originally developed for use in stiff clays and when installed in softer soils there has been a danger of the magnet holders slipping in the borehole. This potential problem has largely been overcome through use of various laborious procedures which themselves increase installation time and therefore cost. The author describes a new clamping arrangement whereby very strong springs, attached to the magnet holders, are held in by a cord. The cord is cut by an explosive cutter when the correct depth is reached, substantially reducing installation time.

2. Check Calibration In-Place. The general need for in-place calibration checks is discussed in Section 5.9, and those remarks apply to electrical sensors associated with subsurface settlement gages. However, a simple and ingenious technique for ensuring correct readings, that could advantageously be adopted by other manufacturers, has been developed by Interfels for their rod extensometers. Each rod is provided with a bayonet connection to its anchor permitting it to be disconnected and checked for free sliding within its protective sleeve.

<u>Proposed Instrument Development</u>. As discussed in Section 5.2, it is believed practicable to pre-package a piezometer and single point extensometer together and to install the package in a single borehole, thereby leading to increased utility. A research effort toward this goal has been pursued, and is reported in Chapter 8 and Appendix G.

5.6 Monitoring Subsurface Horizontal Displacements

Measurements of horizontal movement below the ground surface are used for assessment of potential building damage, for evaluation of liner grouting efficiency, and to examine the sources of lost ground. These measurements are particularly useful behind excavation walls.

Currently Available Methods. The instrument used for monitoring horizontal movements of ground below the surface is the inclinometer. The basic measuring principle is illustrated in Fig. 5.1. In all cases, inclination readings are recorded at specified depth intervals and calculations are made to define the profile of the inclinometer casing. Horizontal movement is determined by comparison with the initial profile. Commercially available inclinometers are listed in Appendix B under "Torpedo Inclinometers". Further details are given by Dunnicliff (1971), and accuracies of the various inclinometer types are discussed by Gould and Dunnicliff (1971). There are at present at least 18 different inclinometers available on the market. The primary difference between the systems is the selection of the sensor to sense deviation from the vertical. A reliable and widely used linear resistor sensor has been used in the Slope Indicator Co. Series 200-B inclinometer for many years. From the standpoint of overall acceptability based on accuracy, cost, durability and availability from multiple sources, the accelerometer is superior to all other sensors. The technology gained from the space program has provided dual axis accelerometers as small as 0.5 inches in diameter and 1.5 inches long with resolution better than 0.001 percent of full scale, temperature sensitivity of less than 0.1 percent per degree F, operating temperature ranges from $-40^{o}F$ to $+200^{o}F$, shock survival of from 25 to 100G, and non-linearity of less than 0.05 percent of full scale. There are at least 48 manufacturers of accelerometers, thereby assuring a wide selection and competitive pricing.

Frequency and Cost of Monitoring. Measurements of ground subsurface horizontal movement are normally not made as part of a routine program to monitor the performance of bored soft ground tunnels. However, extensive and frequent measurements are often made adjacent to buildings if there is a concern for potential damage, alongside cut-and-cover excavations to ensure adequacy of support, and at test stations examined for research purposes. Urban cut and cover excavations for rapid transit tunnels will normally include monitoring by use of inclinometers.

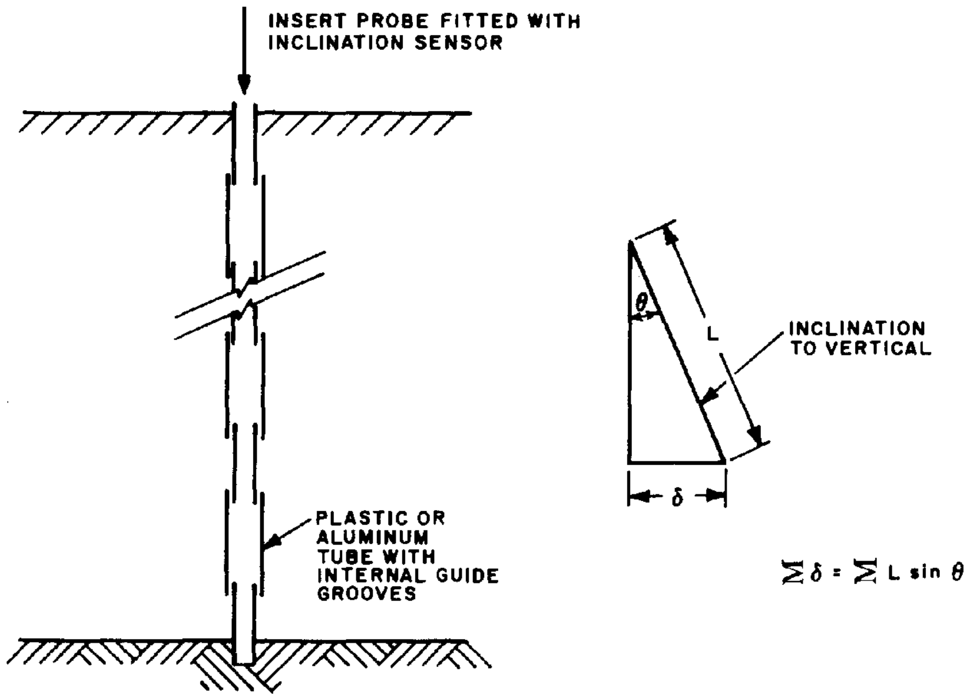


Figure 5.1. Inclinometer for Measurement of Subsurface Horizontal Displacement

The cost of installing an inclinometer casing in a 100-foot deep borehole, including casing, boring and specialist supervision ranges from $2,000 to $3,000. The cost of the inclinometer itself (which may be used in many different casings) without automatic readout facilities, ranges from $3,000 to $7,000. Most inclinometers are available for rental of about 10 percent of purchase cost per month. The cost of taking one set of measurements in a 100-foot deep borehole, computing, plotting and evaluating the data, assuming an experienced operator and no instrument malfunction, is between $75 and $150. Based on these figures, the cost of installing four casings and then monitoring movements daily over a one month period, including inclinometer rental, ranges from about $17,000 to $30,000.

Shortcomings in Available Methods. At present, inclinometer measurements are tedious and costly. The most sophisticated inclinometers are prone to malfunction during field use, with the result that the user tends to mistrust field data. Several manufacturers have either partially or completely developed inclinometers with automatic data acquisition capability, but none has yet proven to be adequately reliable.

In preparation for monitoring subsurface movements at a particular station during construction of a tunnel, substantial time, effort and cost must be expended in installing instruments at that station ahead of the tunnel face. Measurements must then be made of the test station during the brief time that active ground movements take place as the tunnel is driven. Under such a situation one of the following is required:

1. An instrument which provides real time data with a means of verifying correctness. In the event data is shown to be incorrect, a backup instrument can immediately be used. Instrument reliability is clearly of great importance, but some lack of reliability can be tolerated.

2. An instrument which provides raw data of such assured correctness as to preclude the need for duplicate readings. Such an instrument can be handled by a technician.

3. An instrument which provides data, either manually or automatically recorded, with some reliability. A highly skilled and trained observer, so familiar with the characteristics of the particular instrument that he can recognize incorrect data as it is recorded, and a backup instrument.

Accepting limitations of presently available inclinometers, only requirement (3) above is attainable. No inclinometer provides real time data, and no inclinometer provides raw data of assured correctness. During measurement at test stations of the Washington Metro System, requirement (3) above was attained by assigning instrument reading responsibilities to University of Illinois geotechnical engineering Ph. D. candidates, but such skilled personnel are not normally available.

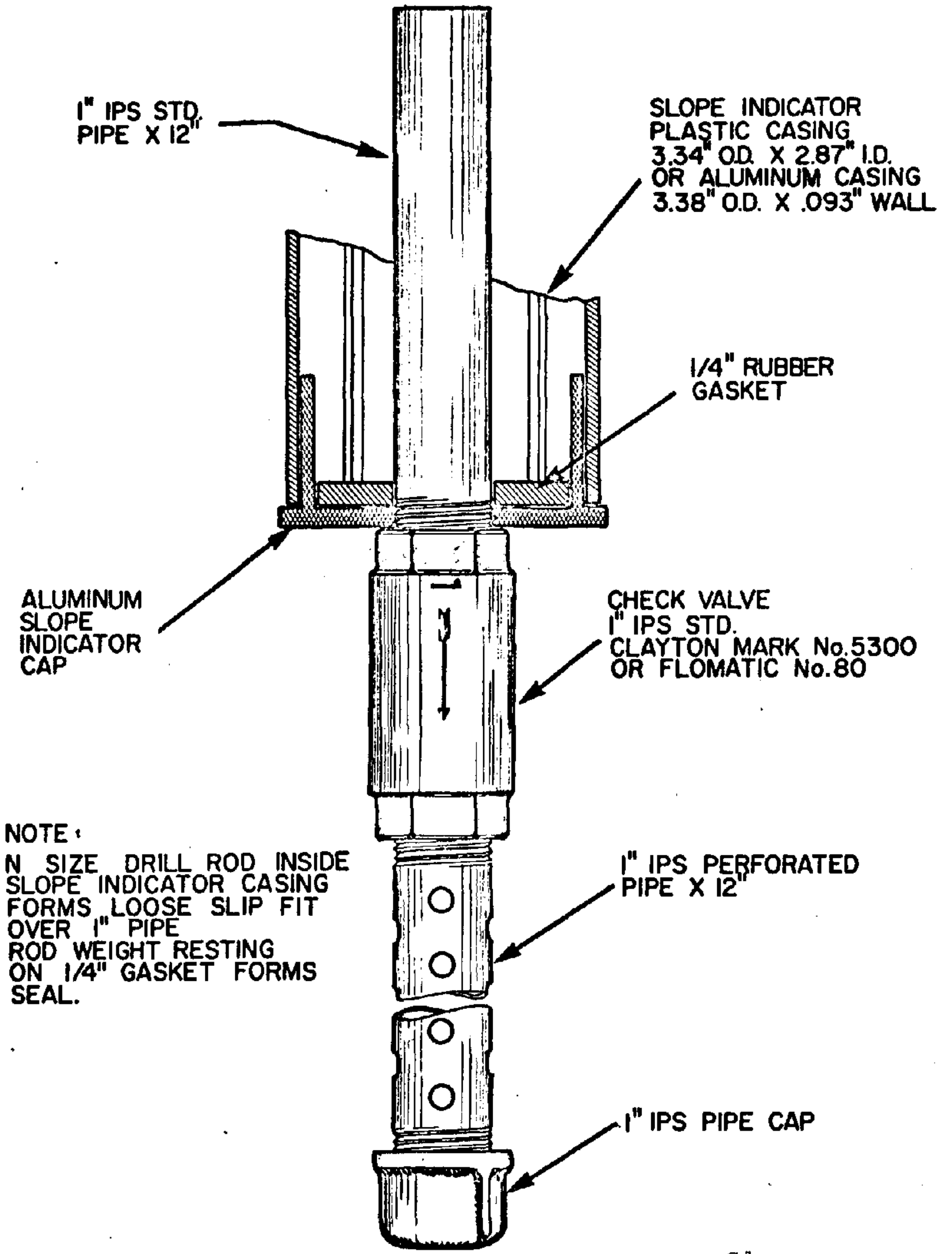
There is a need for improving inclinometer reliability and, when reliability has been achieved, for complete automation of data acquisition and interpretation facilities.

As discussed in Section 5.9, only minor investment is made by instrument manufacturers in research and development of new instruments, with the result that new instruments are generally de-bugged by and at the expense of the user. Adequate and timely improvements in inclinometer reliability will result only from a funded research and development effort. Work under this contract has not included preparation of a specification for such an effort, but some preliminary work has been done to provide a basis for determining what development is required. Results of this work are described below.

Possible Innovations. Improvements are needed in casing installation procedures, sensor packaging, readout unit and accessories. Several innovations in these areas are described below.

1. Casing installation procedures. Much of the need is common to that discussed in Section 5.5 for subsurface settlement monitoring instruments. Possible innovations include use of hollow stem augers, wire line drilling, updated drilling muds. The traditional method (Slope Indicator Company, undated) of installing the inclinometer casing in boreholes has been to advance a 6-inch diameter hole supported with drill casing or drilling mud and to lower the inclinometer casing to the bottom of the borehole. Depending on soil type, the annular space between inclinometer casing and borehole wall is then filled with a granular material or a grout, while the drill casing is withdrawn or the drilling mud displaced. The customary combination is either a cased hole with granular backfill or a mud supported hole with a grout backfill. The former entails a lengthy and painstaking sequential operation of bumping up the drill casing a short distance, filling that unsupported depth with backfill and compacting the backfill. The latter entails use of a grout pipe in the annular space, and hence requires the borehole to be as large as 6 in. in diameter.

   A recent innovation by Slope Indicator Company has enabled grouting the inclinometer casing in a 4-in. diameter hole, thereby reducing drilling costs and installation time. As shown in Fig. 5.2, grout is injected through N size drill rod until the annular space is filled with grout. A volume



Slope
Indicator Co.

Figure 5.2. Base Assembly for Grouting Inclinometer Casing in Place

of water equal to the internal volume of drill rods is then injected; the rods are lifted off the bottom gasket, and water is pumped in until it runs clear from the top of the casing.

2. Sensor Packaging. Present sensor packaging often limits angular measurement to ±30 degrees with respect to vertical. Orientation control is accomplished by using a special casing. With today's technology the above limitations can be overcome.

   Probes should be designed for conversion from housing a vertical accelerometer to a horizontal accelerometer. Sensors are currently available from Schaevitz, Donner, Columbia, Bendix, Daytronics, Honeywell, and others that enable an angular measurement to be made ±90 degrees from either horizontal or vertical. Use of these larger range accelerometers need not increase probe size or reduce system precision.

   Orientation techniques that would eliminate the need for a special casing should be explored. Critical remote positioning of antennae, inertial platforms, guidance headings, and industrial remote positioning offer wide areas of investigation that should result in the capability to remotely position and control an inclinometer.

   Minor innovations include use of a double O-ring seal at the connector between probe and cable to ensure seal integrity under a pressure of up to 1,000 ft of water. Some manufacturers have overcome the need for such a seal by permanently attaching the probe to the cable. While overcoming potential leakage problems, this expedient removes the opportunity to replace damaged cable readily and also to use variable cable length depending on casing depths at each particular project.

3. Readout Unit. The readout unit is used to determine inclination data at each specified depth interval. Data are either recorded manually or on magnetic tape. The data are then processed either by hand calculations or by computer; no unit provides real time data. Several attempts have been made to provide automatic data acquisition facilities, notably

by Geotechniques International, Geo-Testing, Inc., M.I.T. (Bromwell et al, 1971), Slope Indicator Co., and U.S. Bureau of Mines (Tesch, 1972), but none provides data of adequate assured correctness. Several borehole logging companies make directional surveys of drillholes for the petroleum and mining industries; for example, Schlumberger Well Services, Welex, Dresser Atlas Operations, and Sperry-Sun, all of Houston, Texas. As a prelude to developing improved and automatic readout units, contact should be made with these companies to possibly transfer their technology to the tunneling industry.

Various avenues should be pursued to provide a visual automatic readout unit. By use of currently available Large Scale Integrated (LSI) circuitry in the Complementary Metal Oxide Substrate (CMOS) family, including Random Access Memory (RAM), Read Only Memory (ROM), Field Effect Transistor (FET), Programmable Read Only Memory (PROM), DC to DC converters, hybrid chips, etc., it is possible to provide a readout unit which permits the user to follow the probe visually and which automatically calculates all required data on a real time basis. Automatic recording of all data can also be provided. In addition, use of the technology will greatly increase reliability, reduce cost, reduce equipment size by a half, and reduce power consumption by a factor of 1,000.

4. Accessories. Accessories provided with inclinometers vary considerably. Some systems provide no method to handle the cable adequately and other systems provide a de-reeling device which is too big and heavy for easy handling under field conditions. Research is required to develop a lightweight, portable de-reeling device with automatic means of metering the probe depth within the borehole. Some steps in this direction have been taken by Telemac, which markets a de-reeling device with an automatic cable length counter. The Bureau of Mines (Tesch, 1972) and M.I.T. (Bromwell et al, 1971) have also worked on this problem.

Portable test stands are available from some manufacturers, with which spot checks can be made on system accuracy. However, their use is rare.

Contributing factors are the effort required to make check measurements and general lack of good engineering practice. Recommendations for improved engineering practice are included in Chapter 6. Reasearch is needed to develop a portable automatic test stand. The stand could consist of an automatic level system, perhaps similar to systems used in automatic optical survey levels, in conjunction with an electronic sequence servo system. The system would automatically cycle the probe through its entire range and provide a go/no go indication of adequate functioning. The same sequencer could provide a series of pre-selected voltages that are compared to actual accelerometer output for ensuring system operation within manufacturer's specifications.

5.7 Monitoring of Anchor and Bracing Loads

The state-of-the-art of predicting loads in tie-back anchors and struts is essentially empirical and derived from measurements on relatively flexible bracing systems, few of which were prestressed. However, deformation control now normally includes prestressing of tie-backs and bracing at the time of and subsequent to installation. Since prestressing allows some control of earth pressures as well as ground movements, available empirical load prediction procedures are not necessarily applicable. Field monitoring of both member loads and deformation within the soil mass provides the basis for readjustment of the support system geometry and changes in the prestressing loads in order to better control ground movements.

Existing methods of monitoring loads in tie-back anchors and in bracing struts and walers have often proved inadequate, and there is a need to establish better standardized techniques. In particular, the reliability of measurements under the varying temperature and moisture regimes of field conditions must be improved. In addition, problems of compatibility between gages and the structural members on which gages are used must be reconciled.

An evaluation of existing measuring systems has been made, using basic theories, case histories and manufacturers' literature. The results of this study are presented in Chapter 9. The proposed Manual 2 (see Chapter 6) would contain specific recommendations of usage based on this study.

5.8 Monitoring Within the Tunnel

Measurements made from within a soft ground tunnel itself include surveying for alignment control, monitoring of tail void encroachment, lining location and distortation, lining stresses, and shield jacking loads. Surveying for alignment control is discussed briefly in Section 3.9. The remaining four items are discussed below.

Tail Void Encroachment. Monitoring the magnitude and distribution of the tail void space between the tunnel lining and the surrounding soil, and the manner in which it is closed, may help define modifications to construction procedures. Control of tail void closure is closely related to ground surface settlement. Unnecessarily frequent filling with gravel and grout increases construction costs, whereas insufficiently frequent filling may result in distress to overlying structures due to excessive settlements. When tunnel liners are instrumented to measure liner stress and earth loading for research purposes, it is also necessary to monitor tail void closure to evaluate the data.

Efforts to monitor tail void closure are rarely made, and there may be a future need for simple and reliable methodologies. Possible methodologies are:

1. Periodic visual inspection by temporary removal of grout ports, and a qualitative record.

2. Installation of a disk-shaped measuring plate, anchored in some manner to the soil face, through selected grout ports. Subsequent periodic inspection by temporary removal of grout ports and a quantitative record of closure by measuring with a ruler between plate and liner.

3. Periodic removal of grout ports and use of a periscopic viewing device coupled to a remote measuring tool. A rigid periscope, such as a commercial borescope, has limited viewing capacity due to inadequate light intensity. Standard units are rigid and have a viewing range of about one foot, at which point detail is lost in shadow. Adaptation to create a flexible probe would not solve the basic viewing range problem. Range and visibility problems can be solved by using a fiber optic periscope. Fiber optic systems consist of bundles of fine glass fibers that collimate light within the fiber walls. The ends of the fibers are polished as a lens. Light can be transmitted along the bundle and emerges to produce a coherent image to the viewer. The image is a mosaic of the light from each fiber and, similar to the eye of an insect, appears faceted and provides wide peripheral vision. Light losses range from 5 to 50 percent, depending on material length. By coupling the fibers to a flexible mechanical direction controlling device,

the periscope could be positioned within the tail void at will. Calibration of the mechanical device could provide a means of measuring tail void width at a distance from the grout port. Fiber optic technology has been advanced in recent years for use in internal medical examinations, and additional development effort for use in monitoring tunnels should be minor.

4. Observation and measurement of the gap may not necessarily provide sufficient data on which to judge the extent of gap closure. For example, material may apparently fill the gap but may either be very loose or discontinuous. Additional information on tail void encroachment may therefore be obtained by use of a penetrometer through grout ports. The penetrometer would consist of a shaft with conical point, connected to a calibrated load indicator, and could be a simple adaptation of a standard field penetrometer.

Lining Location and Distortion. Tunnel liner rings are subject to significant distortion and displacement at various stages during construction of the tunnel. These distortions and displacements are regularly measured using conventional surveying techniques.

The times at which lining location and distortion measurements are of potential use are:

1. When the ring is erected within the shield. Tie rods may be required at this stage until first stage grouting is complete.

2. After first stage grouting, a few feet behind the shield. In good soil, where no second stage grouting is required, and where no neighbor tunnel will pass by, this will nearly give the final shape and location of the ring.

3. After second and/or third stage grouting, if employed.

4. In difficult soils, where continued distortions are suspected, several times until stable. Tie rods may be required.

5. During and after passing of neighbor tunnel. Depending on distortions, tie rods may be required.

6. If line and grade are not met, resurvey may be required.

While All rings are usually measured at least once, measurements for final location and distortion or for other purposes are frequently done only on selected rings, as soon as possible after erection.

Since measurements are difficult to make in the tunnel when the full crew is working on advancing the tunnel, detailed surveys are often done in slack hours. One of the three working shifts per day is often designated for maintenance and make-up work, and detailed surveys are done during this shift.

Near the tunnel face, excavating and erection equipment take up all the central space of the tunnel, and it is impossible to gain access to the centerline and the invert. Measurements in this area are usually done by running one or two target lines offset from the centerline, and measuring the location of the springline points, thereby defining the horizontal diameter and the horizontal location of the as-built centerline, and measuring the elevation of the crown. On the assumption that the sum of the horizontal and vertical diameters is equal to the theoretical constant, the centerline location and the horizontal and vertical distortions are then defined. Ordinary high-precision theodolites, levels and tapes are used for these purposes. In recent years, lasers used for steering purposes are also used as baselines for distortion measurements. Further behind in the uncluttered part of the tunnel, the invert is still inaccessible, and measurements here, though easier, are made on the same principles.

To determine when tie rods can be removed, a judgment is usually made regarding the causes of potential distortion. When the causes cease to exert their influences (after final grouting, after casting secondary liner, after passing of neighbor tunnel, etc.), tie rods are removed. The use of stress or strain measurements might be considered in tie rods to verify this judgment. Measurement of the effect and efficiency of grouting and other procedures might also be considered. Unfortunately, these tie rods are usually put to use for equipment support or working platforms, and are therefore subject to extraneous influences that make interpretation difficult. The interpretation of such tie rod data would pose numerous difficulties and uncertainties. At the present time, the best available method to determine if tie rods can be removed is to actually loosen a bank of tie rods and measure the deflections. If deflections are too great, the tie rods can be tightened again and grouting or other measures performed before removal.

Ring measurements are usually performed by the surveying crew responsible for steering and line and grade, in hours when the crew is not occupied by these functions. As in all tunnel surveying work, such measurements are time consuming and interfere considerably with tunneling operations. Complete measurements of lining distortion, when required, are made either by using conventional surveying procedures or by use of tape, wire or rod extensometers. Available types of portable extensometers are listed in Appendix B under "Simple Portable Deformation Gages."

Shortcomings in Available Methods and Possible Innovations. Improved methods of measuring tunnel ring location and distortion, leading to a reduction in time, effort and interference required, and an increase in accuracy, would benefit the tunneling operation. These methods would give incentive to a greater number of measurements closer to the tunnel face and thus give the owner and the contractor better control sooner than is currently possible. Such improved procedures could eliminate a number of occasions where rebuilding of rings is required, and give better background data to resolve questions associated with installation and removal of tie rods.

Possible improved methods for measuring lining distortion are:

1. Use of a variable length tape extensometer and a trilateration procedure to define positions of selected measuring points on the liner with respect to one of the points. This procedure is being used at instrumented stations of the Port Richmond Intercepting Sewer Tunnel, Staten Island, N.Y., where eight points are monitored around a 10-foot diameter ring. The extensometer consists of a length of conventional steel survey tape in series with a spring loaded dial micrometer. The ends of the system are

provided with universal joints which mate with receptacles welded to the liner. Variable tape length is created by drilling holes in the tape and attaching a micrometer to the tape at the appropriate hole for each measured span. A repeatability of ± 0.01 inches appears possible with this system. Measurements will continue during 1974 and 1975. Earlier measurements of lining distortion have been made using aluminum or steel telescoping tubes fitted with a conventional screw micrometer. However, sag tends to reduce precision, and variable length requirements are less easy to satisfy if tubes are used. Use of either a wire or tape extensometer would create substantial access problems in a larger diameter tunnel.

2. Use of a camera, flood lamp and reference rods to photograph the tunnel profile. The flood light creates a single line between the illuminated and shadow part of the tunnel. An enlarger is later used to assist in plotting the profile on a standard sheet. Robertson (1973) describes use of this equipment to survey overbreak during construction of a 13-foot diameter horseshoe tunnel in rock for the High Island Water Scheme in Hong Kong. The same procedure was used in an effort to determine shotcrete thickness by photographing before and after shotcreting. Details of the equipment are:

   (a) Camera with wide angle lens and reasonably fast film, ASA 400 or higher. The camera is set up about 15 ft from the lamp and at a constant height (termed the "guide line") above finished invert.

   (b) 300W flood lamp on tripod. The lamp socket can be moved backward and forward in the reflector, and this has the effect of moving the shadow line forward and backward in relation to the lamp unit. The reflector is also mounted on a ball joint, thus allowing the shadow to be fixed vertically and at the correct station without having to put the tripod on the exact station. A check is carried out with an optical square to see that the section is normal to tunnel centerline.

(c) An 8- or 10-foot long reference rod, depending on the size of the tunnel. This is placed on a small adjustable tripod on the tunnel centerline either with a theodolite or with reference to a laser beam. A small cross member on the rod is adjusted to agree with the guideline. The rod is held in front of the lamp so that it is illuminated and can be used as a reference when plotting the profile.

(d) Enlarger. The negative is placed in the enlarger and projected onto a printed sheet of paper on which the clearance lines for the tunnel and guideline level are drawn. The enlarger is adjusted to the clearance lines and to the guideline level, so that the reference rod scales the necessary 8 or 10 feet, and the profile is traced by hand on the sheet of paper. Additional copies can then easily be obtained using a conventional copying machine.

The author reports that surveys of six to eight stations can be made in one hour, no interference is provided by existing tunnel lighting, and that surveys can be made with only minor delays while tunnel operations proceed. The equipment was furnished by Molander and Sons of Stockholm, Sweden, but can be made up from commercially available parts. No specific information is available concerning accuracy, but with care and practice the system could be developed to give reasonable results when checking the thickness of nominally 2-inch thick shotcrete.

3. Use of photogrammetric techniques. Proctor and Atkinson (1972) describe use of photogrammetric techniques to determine the positions of circumferential points which are critical in terms of traffic clearance for the Second Mersey Tunnel in England. Two individual cameras, separated by a rigid bar, formed a stereometric camera used for photographing targets on the tunnel liner. Floodlights were used, and camera orientation and location were coordinated by conventional optical survey. Data were processed by computer. The authors

report substantial cost and interference to construction, but an accuracy of better than 0.5 inches is implied. Several recommendations are made by the authors in the event this technique is used for routine work and any future research in this area should recognize the recommendations.

Another technique using stereo photography is reported by McVey et al (1974). Several 35-mm photographs were made of targets on the wall of a mine and the data analyzed by techniques and equipment similar to those used in cloud chamber film analysis. The authors report test results in a 20-foot wide drift with deformation measurement accuracies better than ± 0.020 inches. Displacement data are provided in all three orthogonal directions. The system bases its determinations on target displacement measurements that occur with respect to reference targets anchored at depth behind the wall of the underground opening. This aspect of the system may make it useless for soil tunnels, but the procedure could be used with repeated setups in a similar manner to that described by Robertson (1973) with a reference rod and conventional survey to define camera location.

4. Use of a laser and spatial chopper (Rasmussen, 1972). The system consists of a combination of a two-dimensional scan with a parallel beam which is spatially modulated in two dimensions. The beam is scanned by reflecting it from a mirror which is tilted and turned. Motion of the mirror is provided by two stepping motors, one driving "mirror tilt" (vertical scan) and the other driving "mirror turn" (horizontal scan). Each step of the driving motor steps the beam 1 in. at 300 ft. The parallel beam, which is about 1 in. in diameter, is spatially modulated both horizontally and vertically. Positions of target which reflect part of the beam, may be determined within the beam by evaluating the phase of the reflected beam. Beam modulation has a wave length of about 1 in., but has different horizontal and vertical frequencies so that the frequencies may be separately evaluated. The location of the target can be determined by knowing the location of the beam (determined from the steps taken by the motors from some point) and the loca-

tion of the target within the beam (determined by the phase of the reflected light).

Further development of the system is required before field use in soft ground tunnels, but the concept has advantages.

Lining Stresses and Shield Jacking Loads. The need for monitoring lining stresses and shield jacking loads occasionally arises during construction of soft ground tunnels. Stresses result from loads due to grouting, from earth and water pressures, imposed distortions, and jack pressures due to shield propulsion. The need to monitor stresses will generally result either from a designer-initiated research effort or from evaluation of an innovative construction technique by the contractor. For example, a contractor may elect to experiment with a thinner liner or an unproven shield, and may then require monitoring of longitudinal liner stresses due to shield jacking. While such a monitoring effort is rare, the opportunity currently exists to report on the early phases of a research program at the Port Richmond Intercepting Sewer Tunnel, Staten Island, New York, discussed earlier in this section. Strain gages are being installed in the lining to:

1. Monitor circumferential strains and hence determine circumferential stresses and bending stresses in the lining.

2. Monitor circumferential strains and hence determine the total pressure distribution around the outside of the lining.

3. Monitor circumferential strains so that curvature changes can be derived.

4. Monitor longitudinal strains and hence determine longitudinal stresses in the lining due to shield jacking.

Such monitoring requires nearly 300 strain gages at each instrumented station. The only practicable gage, both from the standpoint of bulk and economy, is the bonded resistance strain gage. Installation of such gages requires special skill, the lack of which has, in the past, often led to malfunction. Automatic data acquisition arrangements for monitoring strain gage readings are described in Section 5.1.

Measurements are also being made of shield jacking loads by use of electrical pressure transducers in each jack hydraulic line. Transducer readings are recorded on an analog recorder, thereby providing input for interpretation of measured longitudinal strains. During the course of this work, various innovations are being made to acquire necessary data with minimum interference to construction progress.

5.9 Quality Control and In-Place Calibration of Hardware

Two shortcomings apply to many types of geotechnical instruments, and have not been discussed in detail in the above sections. There is an urgent need for a quality control specification for use in instrument procurement. There is a need for developing and implementing methods by which correct functioning of embedded instruments can be verified in place. These two needs are discussed below.

Quality Control Specification for Instrument Procurement. A significant number of geotechnical instruments malfunction during or after field installation. While major blame can be attributed to improper engineering practice (see Chapter 6) there are many cases on record in which an installed instrument malfunctioned due to inadequacies in the hardware itself.

The ideal instrument has the following characteristics (Dunnicliff, 1971):

1. Proper engineering design of each and every component.

2. Ability to answer the specific questions which have led the engineer to make field observations.

3. A satisfactory past performance record.

4. Simple design. The more complex the design, the more probability there will be of malfunction.

5. Sufficient durability to minimize the possibility

of damage due to construction activities, long term deformations of its environment, and vandalism.

6. Adequate accuracy for the length of time over which it is required to function.

7. Adequate reading range.

8. A calibration that can be checked both immediately before and after installation.

9. A calibration which is maintained for the length of time over which data is required.

10. An installation procedure which causes least possible interference with normal construction progress and which creates boundary conditions representative of the construction as a whole.

11. An installation procedure consistent with the capabilities of available installation personnel.

12. Minimum reaction to changing environmental conditions such as temperature and humidity.

13. A minimum degree of automation consistent with overall convenience and economy.

14. A data acquisition and reduction procedure consistent with the capabilities of available personnel and equipment, and which minimizes interference with construction progress.

While no instrument is likely to have all these characteristics, many manufacturers pay insufficient attention to them. The two major areas in which existing instruments prove inadequate for the user's need are:

1. Proper engineering design of each and every component. Each component of any instrumentation system must be designed with equal care. This is frequently not done. For example, the sensing unit itself may be entirely suitable for field use but the connecting tubing or wiring quite unsuitable. Many terminal units or measuring consoles are constructed to standards inapplicable for field use. The most common and significant aspects of improper engineer-

ing design encountered in commercially available geotechnical instruments are:

(a) Inadequate waterproofing of electrical circuits (insulation, connections, entry points into sensors).

(b) Inadequate precautions against galvanic corrosion when two metals are in contact in a non-dry atmosphere. In this situation, a battery circuit is set up and corrosion occurs.

2. Proper quality control of components and completed product before shipment to the customer.

These inadequacies result from the way that geotechnical instrument firms developed. Many of the firms were created by geotechnical engineers, who recognized a need for commercially available instruments. The number of products was generally small and, through repeated field use and experience, many early instruments achieved reliability. However, in recent years, the market, the number of instruments and the variety of instruments have significantly increased (see the numerous types of available instruments listed in Appendix B). Many manufacturers lacked personnel with a thorough basic training in mechanical and electronic design, with the result that insufficient attention has often been given to instrumentation design per se. There has been a tendency for geotechnical instrumentation manufacturers to adopt and persist with a particular sensing system rather than keep abreast of technology which would improve the system. The competitive nature of the market appears to have discouraged manufacturers from investing substantial research and development funds in new instruments, with the result that a new device is generally de-bugged by and at the expense of the user. In a number of cases, a manufacturer has preferred to build his own sensor rather than buy a proven unit from a recognized manufacturer of sensors. In short, the geotechnical engineer often buys instruments from other geotechnical engineers. This is not necessarily valid for all manufactured instruments, but is applicable to the market as a whole.

Wide use of a good quality control specification for instrument procurement would assist in countering the trend discussed above. Usual field instrument specifications, although intended to ensure the ultimate consumer of quality, reliability, and value, merely contain catch phrases such as "good engineering practice", "good manufacturing practice", or "current commercial standards". Restrictions are often imposed which prevent use of present day techniques. Many requirements are insufficiently definitive or are not enforceable. There is no existing standard which would enable a consumer to evaluate his proposed purchases with regards to safety, operational environment or performance. Although U.L. (Underwriters Laboratories) approval requirement has brought some semblance of safety, reliability, quality and uniformity to the commercial market, there has been no attempt made to provide some form of quality standardization to the field instrumentation industry.

A review of some products currently on the market has shown that instruments intended for construction site use are generally not even weatherproofed. Units that operate from 115V source in a construction environment are generally not U.L. approved. In order to provide the consumer with an evaluation guide, and to help standardize his quality and reliability requirements, a standard specification is needed.

The specification must not restrict or prevent advancement in the state-of-the-art of engineering design, but rather should provide the manufacturer with guidelines to ensure that the customer's requirements are met in the areas of:

1. Packaging parameters to meet actual environmental and safety requirements (both mechanical and electrical).

2. Environmental requirements for cables and accessories used with the basic instrumentation.

3. Calibration and quality assurance requirements.

4. Documentation requirements.

A quality control specification suitable for interim use is presented in Appendix A. The specification utilizes Military Specifications (MIL. SPECS.) and Federal Standards as guidelines. MIL. SPECS. are intended as a guide of "what to look for" since they tend to restrict versatility of design and increase cost. In addition, they may be up to ten years old. Items such as passivation methods, colors, plating types and thickness, packaging densities and weights are more reasonably covered by unwritten commercial standards, good manufacturing practices, and good engineering practices. However, no other standard written documentation is available, and use of a quality control specification in this form will serve the immediate need. The interim specification presented in Appendix A is not definitive, but, like all such specifications it is subject to change resulting from further development and study.

It is recommended that a more complete quality control specification for field instrumentation be researched, written and published. This document should specify the exact quality-reliability requirements, tests and parameters. This will result in a document that will be self-contained, complete, and reasonably enforceable as a method of ensuring that the ultimate user receives worthy equipment. The work would begin with a review of existing MIL. SPECS., Federal Standards, Commercial Specifications and specifications and standards prepared by the Institute of Electrical and Electronics Engineers (I.E.E.E.), National Electrical Manufacturers Association (N.E.M.A.) and National Association of Manufacturers. Where appropriate, material from these specifications and standards would be abstracted. In preparing additional material, the adequacy of proposed new specifications would be verified by a test program in the actual use environment. Particular attention during the test program would be given to durability, weatherproofing, prevention of galvanic corrosion and field splicing of cables.

As an example of the extent to which the interim quality control specification (as presented in Appendix A) requires elaboration, the following is a draft of section 3.1 (a), MECHANICAL REQUIREMENTS, Packaging:

> Packaging of all portable electronic assemblies intended for outdoor use must ensure that the unit will operate in an outdoor environment of dust, rain and snow without additional protection. Adequacy of packaging will be confirmed by the following test:
>
> > The housing will be closed by the normal cover retaining hardware. The unit will then be submerged in water until the top of the unit is 6 in. below the water surface.
> >
> > The unit will remain submerged for one hour. After one hour the unit will be opened and correct functioning verified. The inside of the housing will be inspected for water, and will be deemed unacceptable

if more than 0.1 cc of water has passed through to the inside.

The equipment will then be set up as for field use, six feet from a coarse water spray. The spray will be directed at equipment and control panels at a rate of 15 gals. per min. The equipment will be deemed unacceptable if it fails to function correctly during the first five minutes of spraying.

Preparation of a complete quality control specification would require about 24 man-months of effort.

In-Place Calibration Checks. Few geotechnical instruments have a feature whereby reading correctness can be verified in place. Most tunnel performance monitoring instruments are required to function in an adverse environment. If instruments are installed from the ground surface to monitor the behavior of buildings or subsoil as the tunnel is driven, installation will normally be made well in advance of the heading reaching the test station. There is, therefore, an opportunity for drift in instrument calibration during that time interval. An instrument, whether installed from the ground surface or from within the tunnel, may be required to serve a long-term purpose. There is a strong tendency among civil engineers to presume data is incorrect if it indicates an adverse event. However, it is incumbent on the designer of the monitoring program that if the measurement is important, he provides for verification of correctness. An in-place calibration check feature, or a means of removal and testing, is therefore advantageous and creates confidence in data which might otherwise be suspect. Muller and Muller (1970) comment that despite all precautions, it is not always possible to prevent instruments from giving false readings or from failing completely. They state that all instruments which are installed in boreholes must be provided with facilities whereby they can be removed and checked at any time. The only alternative is to acquire data by at least two independent measuring systems. This is clearly a costly alternative.

Certain instruments are either portable or can be removed from the ground for calibration checking. Among these are portable deformation gages, most vertical pipe settlement gages, full profile settlement gages, inclinometers, some deflectometers and embedded extensometers. However, the opportunity to make these checks is often ignored, which reinforces the need for improved engineering practice (see Chapter 6).

Check calibration of rod extensometers has been discussed briefly in Section 5.5, and check calibration of torpedo inclinometers by means of an automatic test stand has been discussed in Section 5.6. However, there is a general need for provision of instrument features which enable calibration to be checked in place.

The following embedded or permanently attached instruments are examples of features whereby calibration can be checked in place:

1. The closed hydraulic and pneumatic combination piezometer. The pneumatic piezometer consists of a diaphragm on which is mounted a check valve. Air pressure is applied to one side of the diaphragm and increased until equality with pore water pressure on the other side of the diaphragm triggers the check valve in the air line. Measured air pressure is assumed equal to pore water pressure. In the combined version two hydraulic lines connect to the pore water side of the diaphragm, thereby enabling the diaphragm to be backpressured by controlled increase of hydraulic pressure while reading the pneumatic piezometer. The presence of the hydraulic lines nullifies some of the attractive features of a pneumatic piezometer, but provides a long term correctness check function.

2. A new type of vibrating wire piezometer developed by the Norwegian Geotechnical Institute and marketed by Geonor of Oslo. A diaphragm instrument, with a vibrating wire strain gage sensing unit measuring diaphragm deflections, measures the difference in pressure between the two sides of the diaphragm. It has been known for some time that the inside face of this type of piezometer diaphragm should be subjected to a known pressure if the pore pressure was to be measured accurately. This is easily done by ventilating the instrument through tubes in the connecting cable and slowly circulating an inert gas at a known pressure. Changing the gas pressure gives a method of checking the slope of the calibration curve, but not a check on the zero reading. This stumbling block has been overcome by simply perforating the metal diaphragm and covering its outer face with a flexible neoprene membrane. Pore pressures deform the diaphragm as before, but during calibration the gas pressure can be increased pro-

gressively until it just exceeds the pore pressure. When the neoprene membrane lifts off the diaphragm, the vibrating wire shows the true zero conditions.

3. An unbonded resistance strain gage, described by Monfore (1958) and Berwanger (1969 and 1970). Strains are determined by measuring the changes in electrical resistance of the elastic wire with conventional strain bridges. Long time stability is achieved by referring all strain measurements to a built-in invar length standard. The operation of the gage is best described using Fig. 5.3 which shows the gage mounted on the surface of a solid. The essential parts of the gage are an invar tube, a piston fitted into one end of the tube, and a small diameter elastic wire stretched inside the tube from the piston to the other end of the tube.

   The wire is adjusted so that it is under slight tension when the piston is in the normal position (that is, when the piston shoulders are in contact with the end of the tube). A spring (not shown in Fig. 5.3) ensures a positive contact between the piston shoulders and the end of the tube. Insert 2 serves as a stop for the outward movement of the piston. Outward movement is accomplished by applying air pressure within the tube. The elastic wire is used as a convenient and accurate method of measuring 'd'. The change in the electrical resistance of the wire as the piston is moved outward from the standardizing position until it contacts insert 2 provides the data necessary for determining 'd'. For small temperature changes, the length 'S' may be assumed constant; for larger changes a correction is required. The gage was designed for use in monitoring concrete strains, but the basic principle of the standardizing unit is applicable to many other types of strain gage and has many other applications.

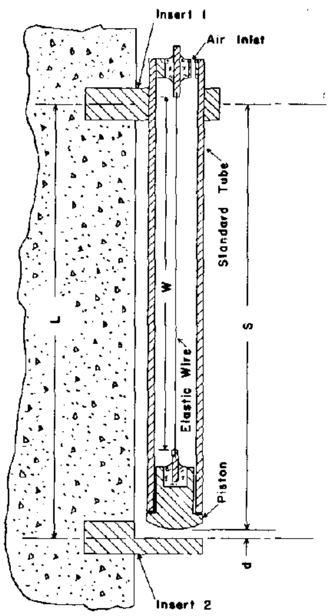


Figure 5.3. Standardizing Strain Gage

4. The double tube closed hydraulic piezometer incorporates a means of checking reading validity, merely by reading hydraulic pressure separately at each tube. However, there is no means of performing a complete calibration check.

If instrument manufacturers were to provide a calibration check feature wherever practicable, the utility of tunnel monitoring instrumentation would be greatly increased. The first step toward this goal would be an assessment of which instruments lend themselves to any of the following:

1. Packaging so that any single reading can be made by two independent methods.

2. Packaging to make possible complete in-place calibration.

3. Packaging so that any single reading can be referenced to a standard.

4. Check calibration by removal and testing.

A requirement that manufacturers shall, wherever feasible and economical, provide in-place calibration check features has been included in the quality control specification in Appendix A.

5.10 Summary of Possible Innovations

Table 5.4 summarizes the innovations discussed and recommended in the various sections of this chapter. Table 5.4 includes a rating of priority for future effort or research. The rating takes into account both the need for and feasibility of achieving improvements, the potential cost of development work and the possible increase or decrease in hardware cost on a commercial basis. However, no cost-benefit analysis has been performed, and the rating is largely subjective though for the most part quite accurate.

Table 5.4. Summary of Possible Innovations

Parameter or Description	Section Reference	Possible Innovations	Summary of Additional Work under This Contract	Outline of Recommended Future Effort or Research	Rating of Priority for Future Effort or Research (1 High; 4 Low)
Groundwater level	5.2	Improved installation procedures for piezometers	None	Investigate boring by hollow stem augers and wire line drilling, and support by degradable drilling mud. Determine sealing limitations of push-in piezometers. Specify grout mixes for sealing. Determine necessary length of seal. Determine effect of swelling seals. Develop improved tamped seal techniques. Develop multi-piezometer packages.	2
		Instrument for automatic monitoring	None	Develop simple inexpensive instrument.	4
		Develop combined extensometer/piezometer	Specification for development	Develop new instrument. See Chapter 8 and Appendix G.	1
Ground surface settlement	5.3	Various improved procedures for monitoring settlement trough	None	Develop improved procedure, considering laser with semi-conductor, full profile settlement gage, subsurface marker to overcome reading falsification by pavement bridging.	3
Ground surface horizontal movement	5.4	Increased use of electronic distance measurement, photogrammetric methods and laser	None	None	4
Subsurface settlement	5.5	Improved installation procedures	None	Investigate boring by hollow stem augers and wire line drilling equipment, and support by drilling muds. Pre-assembly of parts prior to installation.	2
		Develop combined extensometer/piezometer	Specification for development	Develop new instrument. See Chapter 8 and Appendix G.	1
Subsurface horizontal movement	5.6	Improved installation procedures	None	Investigate boring by hollow steam augers and wire line drilling equipment, and support by drilling muds.	2
		Improved inclinometer sensor packaging, readout unit and accessories	None	Develop inclinometer system with improved reliability and complete automation of data acquisition and interpretation facilities.	2
Load in tie-back anchors and bracing	5.7	Improved techniques for measuring, using strain gages	State-of-the-art evaluation of existing techniques	Improve engineering practice. See Chapter 9.	1
Tail void encroachment	5.8	Improved hardware	None	Develop simple instrument.	4
Lining location and distortion	5.8	Various improved monitoring procedures	None	Develop improved procedure, considering triangulation using tape extensometer, photographic, photogrammetric and laser techniques.	4
Lining stresses and shield jacking loads	5.8	Improved hardware systems	None	Develop improved strain gaging and pressure transducer systems.	4
Quality control specification for instrument procurement	5.9	Preparation of standard specification	Interim standard specification	Prepare final comprehensive specification (see Appendix A).	1
In-place calibration checks	5.9	Increased provision of calibration check feature	None	Enforce provision of checks through quality control specification.	2

6. RECOMMENDATIONS FOR IMPROVED ENGINEERING PRACTICE

6.1 Introduction

While instrumentation hardware and software are important for a tunnel monitoring program, they are only the tools. As Peck (1970) points out, the emphasis should be on observation rather than instrumentation. Monitoring has been successfully employed on many projects, but all too often a dubious or negative outcome has resulted from instrumentation.

Failure of a monitoring program may result from inadequate attention being paid to any one of the many steps and tasks involved in the planning and execution of such a program. Failure may arise out of faults in the instrumentation or observational procedures, or from improper selection of appropriate procedures. However, even if instruments are properly selected, installed and read, the program may yet fail because the instrumentation fails to give answers to the proper questions, or because no plans have been made to react properly to the results of the monitoring program.

To improve on the engineering practice of monitoring, there is clearly a need to improve on the planning,design and execution of monitoring programs. Too few geotechnical engineers are fully aware of the advantages and the shortcomings of instrumentation, and of the specific tunneling problems to which instrumentation may be employed. Too many tunnel designers and specifications writers know little about the benefits of monitoring, and how designs and specifications may be prepared to obtain this benefit. Many owners have shown little interest in resolving problems through instrumentation and observation because the professionals they employ have little incentive to present such solutions to them.

To overcome some of the obstacles to the successful implementation of monitoring programs for soft ground tunneling, a greater awareness of the benefits, possibilities and pitfalls of instrumentation must be bred. In addition, the practitioners must be equipped with the proper standards and resources to properly design and implement their programs. This requires a broad educational effort.

It is proposed that manuals be prepared for the use of owners and practitioners to provide them with suitable standards and guidelines for designing and implementing monitoring programs. It is also proposed that educational programs be employed in a deliberate effort to further the art of instrumentation, and that a National Institute of Tunnel Instrumentation and Exploration be formed to implement these tasks, and to serve as a repository of needed information, including documented case histories. Such a National Institute could be organized within the framework of existing organizations and could be funded by any combination of private or public funds.

6.2 Current Efforts with Educational Goals

Several endeavors to improve engineering practice are currently underway. These endeavors include:

1. Preparation of a manual by the University of Illinois at Urbana for the National Science Foundation under Grant No. GI - 33644X, entitled "Methods for Geotechnical Observation and Instrumentation in Tunneling". An early draft of this publication has been examined, and is referred to herein as Cording et al (1974). It discusses planning and implementation of instrumentation programs for soft ground and rock tunneling, including major features of design and construction of tunnels in soil and rock, with special emphasis on prediction by use of an instrumentation program. Appendices include information on commercially available hardware. Completion of the manual is expected by the end of 1974.

2. An American Society for Testing and Materials (ASTM) Symposium held in Washington, D.C. on June 25, 1974 entitled "Performance Criteria and Monitoring for Geotechnical Construction". The symposium included a session on "Planning of Instrumentation Programs to Monitor Geotechnical Construction". The objectives of the session were to gain an improved understanding of good engineering design practice for monitoring the performance of geotechnical construction, and to agree on educational methods by which the practice of performance monitoring can be improved. The topic was examined by a panel of four engineers who are concerned with improving the practice of performance monitoring.

3. Graduate course work at universities, in particular at Purdue University and the State University of

New York at Buffalo. At most universities, even those with excellent geotechnical course programs, geotechnical instrumentation is only the subject of one or a few lecture hours in the curriculum. Commencing in the Spring of 1975, the graduate program at Purdue University School of Civil Engineering will include five hours of lectures on geotechnical instrumentation. Lectures will include basic concepts, types of instruments for measuring load, pressure and deformation, a discussion of necessary steps in planning and executing instrumentation programs, and illustration of these steps by use of case histories. Similar efforts are being planned by the State University of New York at Buffalo, in the hope of including geotechnical instrumentation in course work commencing in the Fall of 1975.

4. Plans by the State University of New York at Buffalo to establish a workshop during which engineers and technicians would learn geotechnical instrumentation techniques. Present intentions are for a first workshop in 1975 or 1976, to be repeated on a continuing basis as necessary. A parallel effort is underway to develop a laboratory for demonstration of instrumentation, for use both during workshops and in graduate courses.

5. Examination by the Transportation Research Board (TRB) Soil Mechanics Committee of the need for establishing a geotechnical instrumentation committee. This proposed committee would be separate from the other technical committees and would deal specifically with the role of instrumentation in transportation problems. Current plans call for development during 1974 of proposed scope of the committee's activities, and a meeting in January 1975 to discuss the proposals in detail and decide the best implementation plan for an instrumentation program within TRB.

These five separate endeavors are all concerned with the need and recommendations for improving engineering practice of performance monitoring. While they are not all specifically directed toward tunneling, the need is common to all types of construction involving geotechnical engineering. Improvements directed toward any type of construction will enhance the utility of instrumentation for monitoring the performance of soft ground tunnels.

Any plans made to improve engineering practice should be coordinated with Standing Subcommittee No. 6 "Education and Training" of the U.S. National Committee on Tunneling Technology. Initial planning of this Subcommittee's activities includes consideration of the preparation of a syllabus and references for courses in excavation and tunneling for the guidance of universities in the United States.

Occasional papers in professional publications or papers presented at conferences such as the Rapid Excavation and Tunneling Conferences touch on monitoring of tunnels or describe case histories. Newsletters from the U.S. National Committee on Tunneling Technology publicize the availability of pertinent publications. Occasional private and semi-public seminars are held on related subjects. The latest was by the Department of Transportation in collaboration with WMATA, the Washington Metro authorities, in December, 1973. On the whole, however, efforts have been sporadic and have lacked coherence. The only English-language publication devoted to tunneling, the British "Tunnels and Tunneling", has contained only few articles on subjects related to monitoring.

While at least one textbook deals with geotechnical instrumentation (Hanna, 1973), no textbook treats the subjects of monitoring of soft ground tunnels -- in fact there is no acceptable and up-to-date textbook on soft ground tunneling.

When practitioners on occasion show poor judgment in the use (or non-use) or selection of instrumentation, one of the reasons is no doubt the scarcity of written reference material; and the lack of education in the subjects.

6.3 Manuals of Instrumentation Practice

It is proposed that two manuals be prepared to provide guidance and information concerning the utilization of instrumentation for construction monitoring of soft ground tunnels. These manuals would serve the informational needs of the owner, his agents and designers, and instrumentation practitioners.

The manuals would be addressed to two separate but interdependent groups: (1) the decision makers, owners, general designers and specification writers, who would benefit from guidance in the planning and design phases of tunneling, and

(2) the geotechnical engineers and specification writers responsible for detailed project design, preparation of construction documents and supervision of construction. The latter group includes engineers and technicians responsible for instrument installation, maintenance and data collection, who would benefit from guidance in the detailed planning and implementation phases.

The proposed title of the first manual is "Uses and Benefits of Soft Ground Tunneling Instrumentation for Planners and Designers". This manual would contain guidance and information to help make decisions regarding the employment and implementation of data collection systems, select monitoring parameters, write construction specifications and help in the design of data processing and interpretation systems, with emphasis on problems of soft ground tunnel and cut-and-cover construction.

The proposed title of the second manual is "Selection and Use of Instrumentation for Monitoring Performance of Soft Ground Tunnels". It would present guidance for selection of instrumentation hardware for monitoring of specific parameters, and guide specifications for instrumentation procurement and installation. Maintenance of instruments, data acquisition and reporting would also be included. This manual would contain check lists, instrument inventories and other practical tools for the use of the instrumentation program designers and specification writers.

These manuals would first be presented to users as working drafts. After some use, it is anticipated that review, updating and rewriting of sections may be desirable. When general acceptance by the professions and authorities has been achieved, it may be possible to assign standard or directive status to portions of these manuals.

Annotated draft outlines of these manuals are presented in Appendices D and E.

6.4 Other Written Material

To serve informational and educational needs on a broad basis, various types of written material are needed including some or all of the following items:

1. Textbooks
2. Manuals/Handbooks

3. Standards/Codes of Practice
4. Specifications
5. Check Lists
6. Inventories of Instrumentation Hardware/Lists of Suppliers
7. Reference Lists/Literature Retrieval Systems.

These items are not currently available for tunneling instrumentation. Hanna (1973) presents a summary of instrumentation equipment and procedures, with little emphasis on their use in tunneling, and without a categorized instrument inventory. A manual discussing methods of geotechnical observation and instrumentation for tunneling is under development at the University of Illinois (Cording et al, 1974).

The manuals proposed in section 6.3 would be a first step toward covering the first five items of the above list. The last two items are described below.

Inventory of Instruments. Having selected parameters that should be measured during a performance monitoring program, it is necessary to select instruments which will best fulfill the purpose of the program. Currently there is no categorized listing of commercially available geotechnical instruments. An engineer designing a program must rely on instrument manufacturers' catalogs, the general technical literature, and his and his colleagues' experience. As a result, an engineer will often select instruments that are less than adequate for the task in hand, a contributory factor in the failure of many programs to fulfill their purpose.

The lack of a suitable summary has been overcome by preparation of a categorized inventory of geotechnical instruments. This is presented in Appendix B of this report. Instruments have been grouped into five major sections, according to measured parameter:

1. Load measurement
2. Pore water pressure measurement
3. Earth pressure measurement
4. Deformation measurement
5. Temperature measurement

Under each major section the various commercially available instrument types have been listed, together with the name of the manufacturer or service agent for each. An alphabetical listing of manufacturers' names and addresses is included in

Appendix B. This version of the inventory merely contains a summary listing of instruments, and makes no attempt to evaluate the various alternative instrument types. It is proposed that an evaluated version of the inventory be included as an appendix to the second manual. The evaluated inventory would contain a table for each instrument category (see the category index in Appendix B) and each table would include data under the following headings:

1. Instrument type
2. Applications
3. Parameter measured
4. Principle of operation
5. Advantages
6. Limitations
7. Accuracy
8. Fig. No.
9. Manufacturers or Service Agents

An evaluated inventory could provide a rapidly accessible source of information which will enable a design engineer to select the most suitable instrument for the task at hand.

Literature Retrieval. During the preparation of this report, an extensive search was made of the available literature. Coupled with similar efforts before this search, a total of nearly a thousand publications referring to geotechnical instrumentation have been accumulated or examined.

A literature retrieval system, consisting of a monthly publication,"Geotechnical Abstracts", is available for use by the geotechnical profession. Abstracts of papers in the areas of soil mechanics, rock mechanics, foundation engineering and engineering geology are contained in the literature. A matching adjunct, the "Geodex Retrieval System", is based on coordinate indexing and the use of keywords. The 347 keywords have been selected for use by the non-specialist geotechnical engineer, and include only the following that have a bearing on tunnel monitoring instrumentation:

1. Deformation
2. Detailed field performance
3. Instrumentation
4. Measurement
5. Piezometer
6. Settlement
7. Strain gage
8. Tunnel

Since 1969 this literature retrieval system has accumulated 6,500 references, 480 of which are assigned the keyword "tunnel" and 220 the keyword "instrumentation". Only seven of these references, however, are assigned both of these keywords. Hence, the Geodex System has been of limited value for locating specific instrumentation literature.

To organize the nearly 1,000 references examined, it was found convenient to assign keywords to each, in a fashion similar to that used for "Geotechnical Abstracts". In this method, keywords specifically related to instrumentation are used. The list of keywords is presented in Table 6.1. In addition to the keywords, a tabular cross-reference listing has been prepared to indicate which topics are covered by each keyword.

This effort has been pursued only as far as necessary for this research effort, and has not resulted in a literature retrieval system which could, in its present form, be reproduced and made available to the geotechnical profession. However, it is believed that the ground work has been done for a specialized system which would be of value to geotechnical engineers responsible for tunnel design and supervision of construction. The system would also be of value to future researchers in this area and to the proposed Institute for Tunneling Instrumentation and Exploration.

The additional effort required to produce the system in a form for use by others would include: completing card abstracts and an author index (both now in draft form), punching sets of keyword index cards, and reproduction.

6.5 A National Institute for Tunnel Instrumentation and Exploration

The preparation of manuals, literature retrieval systems, reference and inventory lists is of little value unless a deliberate effort is made to publicize these items and put them to use. The concerted effort could also meet educational needs.

It is proposed that a National Institute for Tunnel Instrumentation and Exploration be established, whose functions would include at least the following:

Table 6.1. Tunnel Monitoring Keywords

LOAD MEASUREMENT

- Strain gages
- Load cells

PRESSURE MEASUREMENT

- Piezometers
- Earth pressure cells
- Stressmeters
- Pressure transducers

DEFORMATION MEASUREMENT

- Surface movement points
- Portable deformation gages
- Pipe settlement gages
- Remote settlement gages
- Full profile settlement gages
- Heave points
- Horizontal movement gages
- Inclinometers
- Embedded extensometers
- Conventional survey
- Benchmarks

TEMPERATURE MEASUREMENT

- Temperature sensors

OTHER INSTRUMENT TYPES

- Moisture sensors
- Vibration sensors
- Flow transducers
- Liquid level sensors
- Microseismic detectors

SENSING SYSTEM

- Vibrating wire
- Resistance
- Accelerometer
- Potentiometer
- LVDT
- Photoelastic
- Pneumatic
- Hydraulic
- Reed switch
- Photography
- Miscellaneous electrical
- Mechanical
- Optical

OTHER KEYWORDS

- Automatic data handling
- Planning
- Installation
- Analysis
- State-of-art
- Costs

1. Information Depository. Library, literature retrieval system, inventories, lists of suppliers, lists of consultants, data bank for recorded case histories.

2. Consultation Services. Consultation and guidance for governmental agencies and other interested parties, insofar as practicable without interfering with private practitioners. Primary functions would be review and coordination.

3. Depository, maintenance and renewal of computer programs such as the DAPSOG program; instruction in proper use of these programs (see Chapter 7).

4. Educational Services. Provide guidance and reference material for graduate courses, guest lectures, and short courses. Provide teachers and guest lecturers as needed. Arrange workshops and other dissemination efforts.

5. Follow-up on written material. Update all manuals, reference lists, etc.

6. Research. Act as consultants and clearinghouse on research related to instrumentation for tunneling.

7. International Liaison. Through international and national societies and committees, such as the British Tunneling Society, the German STUVA, the Japanese Tunneling Society, and others.

If such an Institute can be established, it would be possible to include other functions demanding similar types of expertise and directed toward similar types of users. Such functions would be related to the prediction of tunneling performance and, in particular, to the acquisition and use of exploratory data before and during design. Both tunneling and deep excavations in soil and rock should be included. As indicated in Schmidt et al (1974), there are similar needs of information dissemination for exploration purposes as for monitoring.

Clearly, the cost savings potential afforded by available instrumentation and exploration methodologies have not been fully transferred to field practices. Most efforts have been expended on research and development by governmental agencies, universities and private organizations. Only a small effort has been made to disseminate the information to the users. Despite occasional conferences and various distribution channels of technical literature, instrumentation and exploration technology -- and an understanding of their benefits -- remain in the domain of a few specialists. This knowledge fails to reach the people who make cost-sensitive decisions in soft ground tunneling.

From 1971 through 1973, the U.S. Government Technology R & D Fund budgeted outlays of $79 million. Of this amount only 0.5 percent, or $130,000 per year, was allocated for Education and Information Dissemination. It is reasonable to assume that only an infinitesimal sum went for instrumentation and exploration technology. Discounting some fractional funding by private research institutions, this explains the lack of penetration of the current information dissemination programs.

An intensive education and dissemination effort, reaching down to the lowest decision-making base, thus seems to be the action required to realize the potential nationwide cost benefits from the existing technology. The Institute would provide this effort.

The educational and dissemination efforts should at least reach people of all of the following categories:

1. Authorities - Government, state, municipal, transit
2. Consulting - Planners, designers, cost estimators, specification writers, specialist consultants
3. Construction - Contractors, unions, insurance companies, equipment manufacturers
4. Research and Education - Educational bodies, research institutions, individual researchers

To reach this variety of people, different types of programs would be required, including short courses and workshops attacking both the philosophy and the technology of monitoring and exploration. Some of these workshops may be carried out before, and in conjunction with, specific projects. They may be conducted for the people who will actually participate in these projects, or they may be carried out independently. Engineers may even be required to participate in such workshops as a contractual requirement.

Depending on the final scope and program of the Institute, its services might be handled by about four specialists plus ancillary staff and equipment, at an annual cost estimated to be approximately $400,000. Compared with past budgetary allocations for education and information dissemination in the excavation technology R & D funds, this is clearly a large sum. Measured against annual national tunneling expenditures of about $2 billion, the cost is one-fiftieth of one percent.

There are already several national and international organizations dedicated to the improvement of tunneling and deep excavation practices. Some of these have been established by private initiative through existing organizations, others by public initiative with considerable private participation. It would not be logical to establish yet another independent Institute; rather, the proposed Institute should be established as an extension or consolidation of existing efforts. No doubt a governmental initiative would be helpful and public funds would be welcome, but the appropriate extent of public funding cannot be determined in the framework of this report.

7. A UNIFIED COMPUTERIZED DATA PROCESSING SYSTEM

7.1 Background

Processing and interpreting monitoring data are key subfunctions of a monitoring program for tunnel construction. For monitoring data to benefit the project from which the data are taken, it is necessary to provide a means of data storage and retrieval of meaningful data sets to facilitate analysis and interpretation. Logical and complete data storage is also required for later research utilization of the data.

Moderate quantities of data can be handled manually and stored on work sheets or in books, but the increased use of monitoring will result in the accumulation of vastly greater quantities of data than has previously been the general practice. For example, a 3,000-foot length of twin tunnels, moderately instrumented, might include more than a thousand surface settlement points, and as many horizontal displacements will be monitored; some fifty deep settlement points, and a similar number of observation wells. From each of these data collection points, 20 to 50 individual data items may be collected; all associated with a date and time of collection. In addition, a complete record of tunnel program data must be kept. In total, perhaps 25,000 to 50,000 individual readings and associated time indications may be collected. A computer program is suggested to organize and store these data and to provide the overview required for rapid data interpretation. The program will: (1) accept the data, (2) check the data for serious errors, (3) store the data, and (4) allow retrieval of any portion of the data set in a variety of output formats. The following sections will describe previous computer uses for similar purposes, outline the program itself and its contemplated usage, and present development plans for the program, with associated cost estimates for establishment and use. Appendix F is a specification for this program.

7.2 Previous Use of Computerized Data Processing Systems

There is basically nothing new about computer programs for processing of large quantities of data. Accounting programs store huge amounts of data in various account files, and incorporate sorting mechanisms and selected output functions. Programs used by researchers in the nuclear sciences sort and analyze vast quantities of experimental data. In principle, most of these programs are simple, including input, checking, sorting, analyzing and output functions, with the book-keeping functions being dominant. Since the basic principles are simple, and the specific requirements are quite variable, such programs are usually developed originally for new purposes.

Programs developed in the past specifically for geotechnical data processing are few, and usually for limited purposes. Only one fairly general geotechnical data processing program is known to exist. This is the Field Data Acquisition and Storage (FIDAS) program, developed by Applied Geodata Systems, Inc. This program accepts and stores data from measurements of surface settlement, horizontal displacement, porewater pressures (hydrostatic head), fill height, and earth pressure. FIDAS was written for a large computer (IBM 360-67) using interactive modes through remote terminals, and is quite complex in its executive functions. It is useful primarily for the monitoring of instrumented large embankments on soft soil and similar structures. It does allow a variety of output options, but does not perform any analyses or cross-correlations of data.

The FIDAS program is available through user-owned or leased remote terminals throughout the country, but at this time has seen very little use, primarily because it has not been actively marketed over the last several years.

Data handling programs have been used for specific limited purposes in geotechnical engineering. For example, a computer program was used to process strut load measurements for deep excavations for BART (San Francisco). This program accepted raw field data (including temperature data), calculated strains and loads in struts, and apparent earth pressures on the retaining wall. The data were available for tabular output.

Nicu et al (1973) report that three computer programs were used for data processing on a research project involving instrumented piles, programs for processing inclinometer data, for Carlson strain meter data and stress meter data. Using simple conversion constants, these programs accepted raw data and output deflections, strains, pressures and apparent earth pressure coefficients.

In Monmouth, England, earth movement detectors were installed in a slide-prone highway cut. The field instruments were hooked up directly with a central computer which performed long term monitoring functions. This type of monitoring, with or without computer hook-up, is also often used in deep open-cut mines. Similar monitoring, using scanning techniques, has also been used in underground mining (see for example McVey and Meyer, 1973), but these techniques have emphasized automatic data acquisition rather than data storage, handling and interpretation.

Monitoring, processing and display of laboratory data with the help of computers are well known techniques, though are not much used by geotechnical engineers. The installation of automatic laboratory equipment, however, is on the increase. Poulos and Gerrard (1972) list a number of relatively simple test data reduction and display programs, also including programs for field data processing (borehole alignment, soil sampling parameters and pressure cells). There is no indication, however, that any of the field data programs have seen significant use.

Computer programs have also been used to display exploratory boring data in profiles, and to store regional boring data. Computer programs are common for the direct interpretation of slope indicator and other types of borehole displacement measurements.

7.3 Alternatives and Selections of Program Details

The development of a computer program for processing large quantities of monitoring data is a logical step. The success of such a program in practice will depend on the proper selection of parameters and subsystems to be included in the program, the proper degree of interpretation, the ease of using the program and the possible turn-around time. If the program becomes too large, it gets unwieldy and not amenable to modifications, discouraging users from gaining full confidence in all the program's parts. Yet it must encompass enough parameters to permit meaningful data correlations to be performed.

It seems prudent to begin with a computer program of limited size, incorporating only some of the most important parameters, and parameters that are related to each other. The program can be expanded and modified to include other parameters, and other types of raw data as the need arises.

The type and level of interpretation and display may be changed as experience is gained.

The following paragraphs describe the case for inclusion, exclusion or future inclusion of various parameters and other features in the computer program:

1. Progress monitoring must be included. No monitoring data have any value without a record of progress (by station, depth, stage of work, etc.).

2. Basic data must be included. Basic data are geometric design data such as elevations and dimensions. They are essential for interpretation.

3. Ground movement data are proposed for the program. As far as computer treatment is concerned, the most important ground movement data are surface settlement data, because they are frequently taken in large quantities. From a diagnostic viewpoint, the most important data are deep-seated settlement data. Horizontal displacements are occasionally measured at the same locations as settlements. Building settlements and displacements belong to the same pattern of tunneling induced movements. Deep-seated horizontal displacement data are not proposed for immediate inclusion because such data (inclinometer data) are already ordinarily computer processed, and because these data are fairly infrequently taken for tunnel construction monitoring. They may be included easily at any time.

4. Groundwater data are proposed for the program. Groundwater monitoring is particularly important for prediction of safe tunneling conditions. Groundwater data should be included because of their importance, though the quantity of data points may be limited.

5. Lining distortion may be included in future program versions, logically joined with alignment control data.

6. Alignment control data may be included in future program versions. These data are needed for the contractor's construction control and may be processed in conjunction with other programs designed for tunnel machine guidance. The required programming would vary greatly with the specific guidance procedures selected by the contractor. Hence, a separate

program would be suggested for this purpose. Lining distortion data often are developed using the same survey or a laser system and logically would be included with alignment control data.

7. Stresses and strains in linings are not included in the program. Lining stresses and strains are usually measured at a limited number of locations, primarily for research purposes. When needed, special programs will be written for this type of data processing.

8. New or unusual methods of instrumentation will often require accommodation of different types of raw data, and different types of calibration. The program should be adaptable to new instrument types.

Output formats should be flexible and meaningful, with a variety of combinational options, and should have a choice of time and location selections (date to date - station to station). Depending on availability of hardware, plotted or printed output should be made available by including a plotting on line printer. Selected output options are explained in Section 7.4.

The types of interpretive functions which should be included are necessarily subjectively selected. Every engineer has methods of exercising judgment, using theory and empirical data to suit the engineer's temperament and knowledge. The degree and type of interpretation also will depend on the type of problem and on the type of contractual arrangements between owner and contractor.

Basic requirements for interpretive functions may be set up. These interpretive functions should use the original, or properly corrected and adjusted, data in a standardized and simple fashion. They should relate measured data to a formalized framework of an empirical or theoretical nature to permit local verification or modification of these relationships for making predictions. The treatment should not only result in empirical constants for further use, but also give an idea of the reliability of such constants when used for prediction. This may be done by relating the data statistically to the framework. Finally, the data treatment and display should be amenable to diagnostic functions; i.e., critical behavior of critical parameters should be clearly visible on plotted output.

To provide rapid analyses, direct user-computer interaction may be desirable. Time-sharing on a large computer would then be the desired mode of operation. However, it is at least twice as expensive to produce and check an interactive program, and it is felt that at least the first version of the program should be operated in the batch mode. If properly managed, turnaround time on a small computer can be kept to within a few hours. Computers of the IBM 1130 size or similar are available in many engineering offices. It is proposed that the program be written in FORTRAN IV for use on computers of this size.

For output, the simplest method would be to plot by characters on the line printer. This option should certainly be available. Many types of plotters are available today, in a wide price range and with highly variable speed and accuracy. It would be proper to select a plotter that could perform a single plot in no more than five minutes, and is at least as accurate as a line printer. The most economical plotters of reasonable accuracy are modern versions of incremental drum plotters. The first version of the program should be written to include output programming for this type plotter. When the program is put to actual use, modifications may be needed to suit the specific available output hardware.

Cathode-ray tube (CRT) displays are fast and reasonably accurate but have the disadvantage that hard copy from most models is available only by photographic means. If in the future this type of output is desired, appropriate programming may be added at a nominal cost.

7.4 Description of Program and Proposed Use

A detailed description of the proposed computer program, Data Processing System for Soft Ground tunnel construction monitoring (DAPSOG) is found in the program's technical specification, Appendix F. The program accepts input of basic geometric data, tunnel progress data and instrumentation (monitoring) data, stores the data after checking for serious errors, and retrieves selected data in selected output formats. Certain interpretive functions are included and may be called on as options; no predictive functions are included at this time.

The program would be set up on a computer of suitable size and with suitable disk cartridges and other ancillary facilities. The computer should be available to the user several hours per day with minimal waiting times.

Data will be coming in from the field in field books specially prepared with input forms. In this manner, re-writing of field data onto input forms for the keypunch operators is avoided. As the data input cards are being prepared, the engineer decides which outputs are desired at this time, and proper cards are prepared for these outputs. All cards are then fed to the computer, and in one continuous operation, the computer checks, edits and stores the data and presents the desired outputs in the desired formats.

Table 7.1 shows the plots that would be available at the user's option from the proposed computer program. While the input, storage and output functions of the program as described in Appendix F are fairly straightforward, it is necessary to describe the use of the proposed interpretive functions.

The output, whether printed or plotted, is used for at least three purposes. These are: to warn against risks associated with groundwater, to diagnose sources of lost ground and thus causes of settlement and other ground movement and to provide data for prediction of future ground movements.

Plots of single point ground movements versus time (Plot 1, Table 7.1) are correlated with tunnel progress data (Plot 2). In addition, the plots will allow analysis of tunneling effects at specific points. This type of plot is basic and necessary, but does not afford great insight into causes and effects of ground movements.

Groundwater elevations may be examined by plots of groundwater elevation versus time (Plot 3) for single points, compared with tunnel progress data (Plot 2) to evaluate possible effects on tunnel progress from groundwater. More conveniently, a plot may be used of groundwater elevations as a function of stationing (Plot 4). This plot would also show a longitudinal tunnel geometry, tunnel and ground surface elevations, and location of tunnel heading at a given date. Thus, the groundwater elevations will be displayed directly with other pertinent geometrical data.

A plot of centerline settlements at a given date (Plot 5)-- either at the surface or at depth -- will show variations of settlements indicative of variabilities of workmanship or effects of local soil variations. This plot will also show the location of the working face at the given date. This plot is not particularly useful in diagnosing sources of ground movements, but it very effectively displays the uniformity of

Table 7.1. Summary of Plot Options, DAPSOG Program

Plot No.	Fig. No.	Plot Description	Purposes or Uses
1	F-6	Single point settlement or displacement vs. time	Checking and verification; time rate monitoring, e.g. of consolidation
2	F-7	Tunnel progress data vs. time	Progress review; correlate with other plots
3	F-8	Groundwater elevation vs. time, single point	Groundwater monitoring; correlate with other plots
4	F-9	Longitudinal geometry and groundwater at given time	Groundwater analysis
5	F-10	Centerline settlements, at given time	Settlement analysis, variations along centerline
6	F-11	Centerline settlements vs. distance from shield	Settlement analysis, diagnose sources of settlements
7	F-12	Cross-section settlements and displacements, at given station and time	Settlement analysis, width of trough, % ground loss, regularity, etc.

movements. Therefore, it may indicate predictability, or reliability of future predictions of ground movements. A comparison of the shape of the longitudinal subsidence profile with a theoretical profile will provide further indications of predictability.

Plots of single point (deep or surface) settlements versus distance from working tunnel face (Plot 6) are more useful for diagnostic purposes. These plots will quite precisely show at what time during the tunnel excavation shield shoving or lining erection increments of ground movement occur and at what location they occur relative to the position of the shield. A comparison with a theoretical relationship between settlement and distance to the tunnel face will provide empirical constants for use in predicting settlements further along. A statistical analysis will show the regularity of the centerline settlements.

Finally, plots of settlements and horizontal displacements of the ground surface along lines at right angles to the tunnel centerline (Plot 7) will provide a large quantity of information. Fitting these cross-section settlements to the theoretical bell-shaped settlement profile (the error function, see Chapter 2 and Appendix F) will describe the regularity (hence, predictability) of these settlements. The width of the settlement trough may be compared to the theoretical width, which assumes essentially elastic, uniform and constant-volume ground movements. Important clues to ground behavior, causes and effects may be derived from this comparison (see Schmidt, 1974 for elaboration on these subjects). The volume of the settlement trough will be calculated and related to the volume of excavated soil in the tunnel. The apparent relative ground loss, defined as the settlement trough volume over the excavated soil volume, gives other important clues to ground behavior and thus serves diagnostic purposes. The apparent relative ground loss and the trough width parameter are the most important items used for predicting ground movements further along.

Depending on the specific circumstances, many other correlations may be useful. For example, the ratio between horizontal and vertical maximum surface movements may present clues to soil behavior, as may the distance from the centerline of the maximum horizontal displacement, when compared with theoretical values. Comparisons between movements of buildings and of free field surface markers are often of interest when examining effects of settlements on buildings and assessing present or possible future building damage.

The full benefit of the DAPSOG program can only be effected by the employment of knowledgeable and experienced professionals for interpretation and implementation of the results. The state-of-the-art of interpreting tunneling ground movement data is approaching the level of "fair", but only few professionals have been exposed to interpretation and prediction problems. For this reason, professionals entrusted with these problems should be given adequate time to study these problems and should not be burdened with too many other duties during interpretation work. The possible role of the Institute in maintaining, promoting and assisting in the use of programs such as DAPSOG is described in Chapter 6.

7.5 Development Program and Costs

Only a moderate effort is required to write the DAPSOG program as described in the technical specification, Appendix E. The tasks will include:

1. Systems analysis to arrive at the most feasible and effective system of main program, sub-routines and storage facilities.

2. Coding, which is the writing of all program routines, including output routines.

3. Testing of the program in the course of development of sub-routines and the complete program, using furnished real or fictitious data.

4. Documentation which consists of a clear description of the complete program, including the interaction of all sub-routines, to such detail that modifications and additions may be performed later with a minimal effort, and a User's Manual in explicit engineering language, containing all necessary input forms.

The manpower required may include at least one systems analyst and one or two programmers. Keypunching services and similar semi-technical services will also be required. Undoubtedly, questions will arise, during analysis and coding, concerning the goals of the program, and suggestions to improve the program will come about. For these and other reasons, it will be necessary to employ the consulting services of a professional who is thoroughly familiar with the basic goals of the program. This professional will also provide assistance in the preparation of the User's Manual and proper input forms, and provide data for testing of the program.

An estimate of the required time and costs of the program development is given in Table 7.2. It is estimated that approximately 13 weeks, or three months, will be required to develop the program, at a total cost of about $23,500. However, no contingencies are included in these numbers. During the writing of computer programs, unanticipated problems often occur which increase the cost and time required. Therefore, firms or individuals rarely write programs at a fixed price; rather, they prefer arrangements that will allow at least payment for costs associated with delays and increased efforts due to such problems. A more realistic cost figure would probably be approximately $35,000, with a total time of four months.

The cost of operating the computer program should be examined for an actual project. In this estimate, it will be assumed that the computer and ancillary services are not operated solely for the DAPSOG program so that only hours of actual operation of the DAPSOG program will be charged. Assume setup expenses and one year of operation at a rate of approximately one hour per day, six days a week. The cost estimate would be:

Setup and test program	$ 500.00
Consultation and educational services	3,000.00
Keypunching and verification (15,000 cards)	2,000.00
Disk cartridges (purchase 2)	150.00
Computer (300 hours @ $50 ea)	15,000.00
Total for one year	$ 20,650.00

This number does not include any time expended in data acquisition and interpretation. It may be presumed that data acquisition can be simplified and data storage and processing efforts eliminated, with a resultant reduction in manpower. The expenditure indicated roughly corresponds to the cost of one expert consultant for a period of four months.

Table 7.2. DAPSOG Program, Time and Cost Summary

TIME SUMMARY	Analysis	Coding	Testing	Documentation	Totals
Systems Analyst, hours	80	50	40	50	220
Programmer, hours	40	200	160	50	450
Keypunch, Drafting, Secretarial, etc., hours		50	80	80	210
Computer, hours		25	40		65
Consultant, hours	40	20	30	50	140
Elapsed Time, weeks	2	5	4	2	13

COST SUMMARY	Item	No.	Rate	Total
Systems Analyst	hours	220	$23	$5,060.00
Programmer	hours	450	16	7,200.00
Keypunch, etc.	hours	210	12	2,520.00
Computer	hours	65	50	3,250.00
Consultant	hours	140	27	3,780.00
Expenses: Rental of peripherals, if necessary, travel, expendables, xerox, etc.				1,690.00
Total, Without Contingencies				$23,500.00

Note: These figures reflect mid-1974 rates and do not include contingencies or profits.

8. RECOMMENDATIONS FOR A COMBINED EXTENSOMETER/PIEZOMETER

8.1 Introduction

Measurements of groundwater level or piezometric head are used for control of dewatering efficiency and for assessment of tunnel and excavation stability and safety. Piezometers installed just above the tunnel crown provide a means of monitoring the groundwater regime above and ahead of the tunnel (see Chapters 2 and 5).

Measurements of settlement just above the tunnel crown permit an evaluation of the source of lost ground and hence provide data for the planning of remedial measures. Measurements also permit a prediction of settlement at other similar locations. Such measurements are made by using a single-point embedded extensometer. By monitoring settlement just above the crown rather than at the ground surface, information is gained much sooner, and the specific cause of lost ground is not masked by strains within the soil above the tunnel (see Chapters 2 and 5).

A separate boring is, at present, required for each instrument. The total cost of furnishing and installing one piezometer and one extensometer above the tunnel crown at a depth of 100 feet is approximately:

1. Accessible instruments (access available to tops of borings) $2,500 to $4,000

2. Inaccessible instruments (access not available to tops of borings) $3,500 to $5,000

These costs do not include readout equipment. The merits of making these measurements are insufficiently recognized, and the cost of doing so is a discouragement. By packaging a piezometer and extensometer together, increased utility of a single installation and significant cost savings are possible. This chapter discusses alternative hardware and installation procedures, makes a selection between the alternatives for a development program, and concludes with a cost estimate of development and actual usage. A technical specification for manufacture, installation and testing of instrument is presented in Appendix G.

8.2 Instrument Performance Criteria

Criteria for the instrument, which are considered both desirable and feasible, have been set as follows:

1. The instrument should permit measurement of settlement to a precision of ± 0.01 foot with a range of 18 inches and piezometric head to a precision of ± 0.3 foot of water with a range of 200 feet of water.

2. Installation should be possible through all soft ground materials, including boulders. In particular, provision must be made to set the extensometer anchor in a boulder, and for the piezometer to give adequately rapid response (99 percent within one hour) in materials with permeability down to 10^{-5} cm/sec.

3. Installation should be possible to depths of 160 feet.

4. Sampling, stratigraphy definition, logging and in-situ testing should be as feasible in the instrument boring as in any conventional site investigation boring.

5. Boring should be possible using equipment currently owned by the average boring contractor.

6. Provision must be made at street level for rapid reading with minimum interruption to traffic and minimum danger to reading personnel.

7. The instrument should, if possible in a single model, be applicable to all surface and subsurface conditions, and to all anticipated settlements and changes in pore pressure.

8. If safe access is available to the top of the instrument boring, it should be possible to furnish and install one instrument at 100-foot depth at a cost not greater than $2,000.

Recent experience in installing and monitoring instruments from the ground surface for the Port Richmond Intercepting Sewer Tunnel, in Staten Island, New York, has demonstrated that it can be extremely hazardous to read instruments on the pavement of a busy street. Use of warning signals and barricades is ineffective. Safety can be maintained only by use of flagmen or a police detail, resulting in substantial cost and severe traffic interruption. For such situations, a method must be found whereby instruments can be read remotely. A remote reading capability, however, increases hardware costs, and is unnecessary at locations where a tunnel is not beneath a busy street. It is therefore recommended that two designs be examined, one with and one without remote readout capability. These have been termed "inaccessible" and "accessible" instruments, respectively.

8.3 Alternative Piezometer Hardware and Data Handling Procedures

The various types of piezometer currently available on the commercial market are listed in Appendix B. Most types are described by U.S. Corps of Engineers (1971), Dunnicliff (1972) and Hanna (1973). A general order of preference is presented in Table 5.2. These types provide an ample basis for selection of a suitable measuring principle, and basic sensor development is not necessary.

For the accessible instrument all criteria are satisfied by the simplest principle, the open standpipe piezometer, and there is no justification for considering a more complex device.

For the inaccessible instrument any of the following principles would permit measurement of piezometric head:

1. Standpipe with purge bubble

2. Pneumatic

3. Diaphragm-type pressure transducer with:

   (a) linear variable displacement transformer (LVDT)

   (b) linear resistor sensor

   (c) semiconductor sensor

   (d) piezoelectric sensor

   (e) piezoresistive sensor

   (f) vibrating wire strain gages

   (g) bonded resistance strain gages

   (h) unbounded resistance strain gage

Each of these ten principles requires different data acquisition hardware. Section 8.6 contains criteria for selection of instruments.

The closed hydraulic piezometer, without diaphragm, cannot be used if the readout is above the piezometric level, due to cavitation of water in the tubing.

The hydraulic diaphragm piezometer cannot be used if the readout is above the piezometric level, as in this situation no pressure balance can be achieved.

8.4 Alternative Extensometer Hardware and Data Handling Procedures

The various types of extensometer currently available commercially are listed under "vertical pipe settlement gages" and "embedded extensometers" in Appendix B. Most types are described by Dunnicliff (1971) and Hanna (1973). A general order of preference is given in Table 5.3.

All devices permit a measurement of the change in distance between the anchor (just above the tunnel crown) and the reading head (at the ground surface). If the anchor is settling, the ground surface may also be settling, and therefore this measurement alone cannot be used to determine settlement. Absolute settlement is determined by reference to a benchmark using optical survey techniques.

For the accessible instrument any of the following devices would permit measurement of change in distance between anchor and ground surface:

1. Anchor post

2. Rod extensometer with mechanical readout

3. Wire extensometer with mechanical readout

Mechanical readout consists of a graduated scale, dial gage or micrometer. Data are usually handled manually using a desk calculator to assist with computations. For the inaccessible instrument, measurement of change in distance between anchor and ground surface could be made using a wire or rod extensometer and an electrical displacement transducer. Possible displacement transducers are:

1. Linear or rotary potentiometer

2. Variable reluctance linear transducer

3. Linear variable displacement transformer (LVDT)

Each of these transducers requires different data acquisition hardware. Strain gages (bonded resistance, unbonded resistance, vibrating wire) in series with a coil spring may be eliminated as being unnecessarily complex for this application. Alternatively, an accessible instrument could be converted to an inaccessible instrument by linking the measuring point to a single-point remote settlement gage, in which case readout procedures would be those required for remote settlement gages together with an optical survey. However, this again appears to be an unnecessary complication.

8.5 Alternative Installation Procedures

As discussed in Chapter 5, installation of an instrument in a borehole generally includes five basic interdependent aspects:

1. Advancing a borehole

2. Supporting a borehole wall

3. Placing the instrument in the borehole

4. Removal of temporary measures used to support the borehole wall

5. Filling the space between instrument and borehole wall

A case has been made in Chapter 5 for improving various aspects of piezometer and subsurface settlement gage installation procedures. These are briefly summarized below:

1. Reduction of boring time by using hollow-stem augers or wire line drilling techniques.

2. Support of the borehole wall using degradable drilling muds.

3. Improved methods for piezometer sealing procedures, considering pushing into the soil, grouting and tamped bentonite backfill.

4. Preassembly of hardware, thereby minimizing installation time.

Installation procedures for combined extensometer/piezometer clearly depend on the selected hardware package. They should take into account the above proposed improvements.

8.6 Selection of Hardware, Data Handling and Installation Procedures for Development and Testing Program

A basic criterion for selecting hardware, data handling and installation procedures has been the requirement for compatibility between the two separate components. For example, the piezometer and extensometer should not require two different sets of data acquisition hardware unless those items are both simple and inexpensive.

Hardware should be consistent with the quality control specification (Appendix A) and should, if possible, be provided with features whereby in-place calibration checks can be made.

Hardware and Data Handling Procedures for Accessible Instrument. The open standpipe piezometer fulfills appropriate criteria set out in Section 8.2. By use of a 5/16-inch o.d. x 0.188-inch i.d. nylon standpipe, attached to a 1-1/4 inch diameter x 8-inch long porous tip in a soil of permeability 1×10^{-5} cm/sec, 99.99 percent response can be achieved in one hour (Penman, 1961). A schematic of the measuring principle is shown in Figure G-3, Appendix G. Depth to the water surface may be measured using any of the devices listed in Appendix G.

For monitoring subsurface settlement, either an anchor post, or a rod extensometer with mechanical readout would be suitable. The two instruments are illustrated schematically in Figures G-1 and G-2, Appendix G. A wire extensometer requires a means of maintaining wire tension, and is considered unsuitable.

Packaging of an open standpipe piezometer together with an anchor post or mechanical rod extensometer is discussed in Appendix G. Data handling procedures are straightforward, and would be essentially a combination of the two separate data handling procedures already discussed. Field data would, as appropriate, be used as input into the DAPSOG program described in Appendix F.

Hardware and Data Handling Procedures for Inaccessible Instrument. In 8.4 and 8.5 ten possible pressure sensors and three possible displacement transducers have been suggested. Recognizing that the two component parts should be consistent and that rod extensometers are preferable to wire extensometers, the following can be eliminated:

Rotary potentiometer displacement transducers

Standpipe piezometer with purge bubble

Pneumatic piezometer

It is therefore recommended that the inaccessible instrument be composed of a diaphragm-type electrical pressure transducer together with a rod extensometer using a linear potentiometer, variable reluctance linear transducer or linear variable displacement transformer (LVDT). A pressure transducer piezometer is described by Wissa et al (1974) and illustrated in Figure G-6, Appendix G. Packaging of these components is discussed in Appendix G.

Electrical cables must be laid to a reading point away from traffic interference, and suitably protected from damage by accident or vandalism. Data acquisition procedure will be by use of appropriate portable field readout units, conforming with the quality control requirements of Appendix A, and data handling would be either manual or accomplished using the DAPSOG program described in Appendix F.

Installation Procedures. Installation procedures in general have been discussed in Section 8.5. Specific installation requirements for the combined accessible and inaccessible instruments are discussed in Appendix G.

8.7 Outline of Development and Testing Program

It is recommended that a development, fabrication and testing program be initiated, to conduct the following work:

1. Design and fabricate the necessary hardware.

2. Conduct laboratory tests on electrical components.

3. Conduct field installation trails in an environment where settlement and pore pressure are changing.

4. Amend designs in accordance with results of field trails.

5. Develop production drawings and specifications for the hardware.

6. Prepare a manual describing theory of operation, installation procedures, data calculation and presentation procedures, and maintenance procedures.

7. Prepare guidelines for appropriate usage of the instrument on tunnel projects, including estimated user costs.

A specification for the program is presented in Appendix G.

8.8 Cost Estimate of Development, Fabrication and Testing

It is estimated that the cost of development, fabrication and testing will be approximately $148,000, and require about 24 months to complete. It is emphasized that this estimate of cost and time is based on June, 1974 prices and material delivery schedules, both of which are very unstable. The breakdown of this estimate is as follows:

Task A. Design of Instruments, Laboratory Testing, Design of Field Testing Program

Personnel	$32,000
Materials	$ 4,000
Laboratory and machining	$ 1,500
Total estimated cost	$37,500
Estimated time to complete	6 months

Task B. Hardware Fabrication and Field Testing

Personnel	$40,000
Materials	$19,000
Laboratory and machining	$ 1,500
General contractor	$ 3,000
Test boring rig and crew	$15,000
Total estimated cost	$78,500
Estimated time to complete	12 months

Task C. Amendments and Refinements to Hardware and Procedures

Personnel	$ 8,000
Materials	$ 4,000
Laboratory and machining	$ 500
General contractor	$ 500
Test boring rig and crew	$ 3,000
Total estimated cost	$16,000
Estimated time to complete	3 months

Task D. Preparation of Production and User Data

Personnel	$16,000
Total estimated cost	$16,000
Estimated time to complete	3 months

8.9 Cost Estimate of Usage

It is estimated that the cost of furnishing and installing instruments in a 100-foot-deep boring for actual usage will be:

Furnish one accessible instrument	$ 600 - $ 700	
Install one accessible instrument	$1,100 - $1,700	
Total	$1,700 - $2,400	
Furnish one inaccessible instrument	$1,000 - $1,200	
Install one inaccessible instrument	$1,300 - $2,000	
Total	$2,500 - $3,200	

Costs of readout equipment (one set for each job) are estimated as:

Accessible instrument	$ 150
Inaccessible instrument	$1,750

Costs of optical surveying, data acquisition, handling, interpretation and implementation are strongly dependent on specific project factors and on whether or not necessary personnel must be assigned solely to this work. These costs cannot readily be estimated at this time.

9. MONITORING LOADS IN BRACED EXCAVATIONS

9.1 Introduction

While the preferred choice of monitoring method is fairly clear for most geotechnical monitoring parameters, the choice is less clear when it comes to monitoring loads in tie-backs and struts. Consequently, load measurements in struts and tie-backs used to support excavations have frequently been unsatisfactory. Inadequate and erratic measurements have resulted both from lack of knowledge on the part of responsible engineers and from shortcomings in selected instruments. An educational effort is needed to partially remedy this situation. Accordingly, this chapter contains a review of published case histories and provides basic information for preparation of part of Manual 2 (see Appendix E). The major case histories reviewed are summarized in Table 9.1. Prior to dissemination of Manual 2, this chapter should be of value to any practicing engineer for planning and execution of a braced excavation monitoring program. In general, adequate instruments and techniques are available, and the problem is one of training engineers to employ these methods.

This chapter is divided into eight major sections. A brief discussion of the reasons for monitoring loads in braced excavations follows this section. The next section is devoted to a discussion of the problems encountered in trying to measure loads. The relationship of the total monitoring program (movements, pore pressures, construction sequences, etc.) to the measurement of loads is then presented, followed by a general description of the equipment used to monitor loads. This section is followed by two sections which present the preferred methods for monitoring loads in struts and tie-backs. The final section presents a summary.

9.2 Reasons for Monitoring Loads in Braced Excavations

There are many reasons for instituting a performance monitoring program during the construction of tunnels. All functions discussed in Chapter 3, diagnostic, predictive, legal and research, are applicable to load monitoring. Specifically, ensuring that the excavation is safe against collapse is a self-evident reason for monitoring loads. By

Table 9.1. Summary of Major Case Histories Reviewed

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Garrison Dam -H.H. Burke (1957)	Construction of a tunnel in rock.	Settlement over tunnel by survey. Heave at tunnel invert by survey.	Movement toward tunnel by survey.	Mechanical (Whittemore) strain gages in steel ribs. Electrical resistance strain gages.		Tunnel face movement with tape extensometer (horizontal and vertical diameter changes). Pressure cells (electrical resistance gages). Temperature of concrete.	Test Section for future tunnels. Greatest emphasis on mechanical measurements. ±600 psi unit stress estimated error in load. Low measured stresses.
Oslo Subway -E. DiBiaggio B. Kjaernsli (1961)	An 11.5 m cut in soft clay in Oslo supported by struts.	Optical leveling for sheeting and ground settlements.	Inclinometers.	Vibrating wire load cells	Pore pressures measured	Earth pressure. Temperature of clay	Particularly concerned with bottom heave failure. Load cells use for underwater measurements.
Oslo Technical School -Norwegian Geotechnical Institute (1962)	A cut in soft clay over bedrock for new extension of school.	Settlement rods with anchors with measurements taken by survey. Strain dial gage measured relative movement between anchored rod & settlement rod.		Vibrating wire strain gages.	Well point with plastic tube piezometer.		Instrumentation installed primarily to determine effect on surrounding building and ground movements. Temperature affected strut loads greatly due to overall shortening and lengthening of existing structures.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Measuring Instruments for Strutted Excavations -L. Bjerrum -T.C. Kenney -B. Kjaernsli (1965)	Description of load measuring equipment used on various projects			Portable hydraulic jacks. Mechanical strain gages. Vibrating wire strain gages. Load cells.			Estimated accuracies of load measuring systems. Vibrating wire strain gages ±10% Vibrating wire load cells ±20% Whittemore strain gages ±40%
MBTA North Station-Haymarket Square Extension -T.W. Lambe -L.A. Wolfskill -I.H. Wong (1970)	A cut up to 58 feet deep for a rapid transit tunnel through fill, silts, clay, till.	Selected settlements by survey.	Inclinometers located both on the sheeting and away from the excavation.	Vibrating wire strain gages.	Casagrande-type hydraulic piezometers. Vibrating wire piezometers. Observation wells.	Stress cells on bracing (damaged during driving)	Concern about movements in adjacent areas and structures. Performance of the excavation dependent upon construction details and sequences.
Foundation for Coldwater Intake -T.H. Hanna -J.E. Seeton (1967)	An excavation in till and rock supported by rock anchors, with soldier piles and wood lagging.			Mechanical strain gages for loads in tie-backs. (Normal force required for given deflection is proportional to axial load in wire)			Loads jacked into tie-backs. Concluded that construction procedures did not appreciably alter loads. Limited instrumentation program.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Pierre La-Clede Building, Clayton, Missouri -C.I. Mansur -M. Alizadeh (1970)	A 45' excavation supported by tie-backs.	Settlement points by survey.	Alignment stakes by survey.	Mechanical and electrical resistance strain gages in anchors. Mechanical strain gages in soldier piles.	Water table below bottom of excavation, therefore no pore pressure measurements were taken.		Stress readings rated to be accurate to ±1000 psi. In soldier piles, error typically 20% of load.
Mexico City Excavation for Sewer Siphon -J.M. Rodriguez -C.L. Flamand (1969)	A cut up to 11.3 m deep was made in the soft, sensitive Mexico City clays.	Shallow and deep bench marks, both inside and outside of the excavation.	Inclinometers.	Freyssinet load cells (hydraulic)	Pneumatic piezometers and Casagrande open type piezometers. (depended upon permeability of the soil in which located).		Concern about possible bottom heave failure. Precision and quantity of piezometers not sufficient for use in effective stress analysis.
Instrumentation for Earth Anchors -C.J. Dunnicliff (1971)	Description of equipment and techniques used to monitor loads in tied-back excavations.			Mechanical electrical resistance, and vibrating wire strain gages on rods. Electrical resistance, vibrating wire, mechanical, and photoelastic load cells.		Soldier pile stresses -electrical strain gages -mechanical strain gages	Overview of equipment available for measuring loads and stresses.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Structure in Ottawa, Ontario -G.C. McRostie -K.N. Burn -R.J. Mitchell (1972)	A 40' excavation with steel sheeting and struts.	Settlement points with precise level.	Optical measurements using theodolite. Tell-tale rods measured relative movement between sheeting and anchor points.	Load cells	Open standpipe piezometers. No-Volume change piezometers		Movements often related to construction events.
Pollution Control Center, London, Ontario -J.D. Scott -N.E. Wilson -G.E. Bauer (1972)	A 50' excavation with steel sheeting and struts.	Settlement of sheeting (survey)	Lateral deflection of sheeting (survey)	Mechanical strain gages	Borehole observation wells. Standpipe piezometers.	Vibrating wire earth pressure cells.	Erratic strut load readings
Rock Tunnels for Washington, D.C. Metro. -J.W. Mahar -F.L. Gau -E.J. Cording (1972)	Description of observations made during construction of a rock tunnel.	Extensometers to monitor rock mass movements		Vibrating wire strain gages. Electrical resistance strain gages.			
Soft Ground Tunnel in Washington, D.C. -W.H. Hansmire -E.J. Cording (1972)	Rapid transit tunnel through sands, gravels, silts, and clays	Extensometers. Interior movements with survey equipment.	Inclinometers	Vibrating wire strain gages	Double tube Casagrande type piezometer.		Research project for use as a future construction aid.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
BART Civic Center Subway Station -J.G. Thon -R.C. Harlan (1971)	An excavation up to 78' deep using a slurry trench wall and struts.	Settlement points (survey)	Inclino-meters. Transverse Extenso-meters	Vibrating wire strain gages	Piezometers		Water level had to be main-tained because of underlying compressible soils.
Discussion of Instrumenta-tion and Results for BART System. -T.R. Kuesel (1969)	Two braced excavations were dis-cussed with respect to instrumenta-tion and per-formance	Survey marks on buildings, sidewalks, and pave-ments. Deep settlement points.	Inclino-meters. Some ex-tensometers	Vibrating wire strain gages used to measure strut loads.			Summary of some instrumentation data on selected instrumented sec-tions. Struts were shield-ed from direct sunlight to re-duce temperature effects.
Chase Manhattan Bank -J.P. Gould (1970)	Tied-back excavation for bank building.	Wall and adjacent structure movements by survey.	Wall and adjacent structure movements by survey.	Mechanical strain gages.		Mechanical strain gages to monitor loads in beams.	
Manufacturer's Life Centre, Toronto. -G.D. Prasad -C.F. Freeman -D. Klajnerman (1972)	Deep exca-vation supported by both tie-backs and rakers.		Soldier piles and cables sur-veyed. Reference points at raker base.	Hydraulic jacks (periodic readings only)		Anchor move-ment	

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Excavation for a Structure in Washington, D.C. -K.R. Chapman -E.J. Cording -H.Schnabel,Jr. (1972)	A 45' excavation through sands, gravels, silts, and clays, supported by struts and rakers.	Settlement measurements in street and adjoining buildings by survey.	Lateral movements on north and south walls with extensometers.	Vibrating wire strain gages.			Strain gages were mounted on unloaded steel member to determine effect that differing thermal coefficients of expansion had on strain readings.
Bracing Performance for Chicago Excavation -E.P. Swatek, Jr. -S.P. Asrow -A.M. Seitz (1972)	An excavation up to 70' deep through medium to stiff clays. Sheet piling with struts.	Vertical settlements in streets & adjoining subway (survey)	Inclinometer	Mechanical strain gages.			Calculated strut loads did not rise and fall consistently with temperature.
CNA Center, Chicago -J.A. Cunningham -J.I. Fernandez (1972)	Excavation in clay overlain by sand and fill. Slurry wall with struts	Reference points by survey	Reference points by survey. Inclinometers				No load measurements.
Standard Oil Building, Chicago -J.A. Cunningham -J.I. Fernandez (1972)	Excavation in clay overlain by fill and sand. Tie-backs supported slurry wall.	Reference points by survey	Reference points by survey. Inclinometers				No load measurements reported although tie-backs were re-stressed.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Building Excavation in Boston -T.K. Liu -J.P. Dugan, Jr. (1972)	Excavation for structure included observation of tied back wall section.	Reference markers on buildings and ground. (survey)	Reference markers on buildings and ground. (survey) Inclinometers	Photoelastic load cells. Hydraulic jacks (initial)		Tell tale wire to check anchor movements	Surrounding buildings were founded above bottom of excavation. Concern about movements.
Soil Anchor Tie Back System in Minneapolis -M.L. Larson -W.R. Willette -H.C. Hall -J.P. Gnaedinger (1972)	50' deep excavation through sand and gravel in Minneapolis	Monuments (survey)	Monitored but method not specified	Load cells (unspecified type). Hydraulic jacks (initial)			
Bank of California Center, Seattle -G.W. Clough -P.R. Weber -J. Lamont, Jr. (1972)	Excavation in overconsolidated clay retained by soldier piles and tie-backs	Bench marks by survey. Heave points (electric)	Inclinometer	Load cells (hollow steel cylinders with unspecified type of strain gage). Hydraulic jacks (initial)			No static water table encountered.
A Study of the Bauer Earth Anchor. -M.D. Oosterbaan -D.G. Gifford (1972)	Excavation in dense sands and till in Boston.			Electrical resistance strain gage. Hydraulic jacks (initial)			Many resistance strain gages were lost during the course of the project.

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
BARTD Embarcadero Subway Station -W.J. Armento (1973)	A strutted excavation through sand and clay with slurry trench walls.	Painted markers and settlement points (survey)	Inclinometers	Strain gages	Observation wells and well points.		Instrumentation program not described in detail. Consolidation due to lowering of water table is a problem.
Operations Control Center Building -K.R. Ware -M. Mirsky -W.E. Leuniz (1973)	Tie-backs used to support 35'-50' excavation.	Soldier piles (survey). Settlement markers in streets (survey)	Soldier piles (survey)	Hydraulic jacks		Tie back movement with strain gages.	Baselines for horizontal measurements moved. Recommended survey methods when measuring creep in tie-backs.
MBTA South Cove Subway -W.E. Jaworski (1973)	Construction of a rapid transit tunnel in Boston through fill and clay	Settlement screws & level pins in building and ground (survey). Anchor points(lost) Heave points.	Inclinometers	Vibrating wire strain gages	Casagrande type hydraulic piezometers. Vibrating wire piezometers. Shallow piezometers.	Vibrating wire total stress cells.	Instrumentation program undertaken to determine effect of construction on adjacent structures.
MBTA Medford Cofferdam- -C.J. Dunnicliff -W.E. Jaworski (1974)	Construction of a rapid transit tunnel supported by steel pipe struts and steel sheeting	Survey settlements of sheeting	Survey measurements of movements in sheeting	Electrical resistance strain gages. Mechanical strain gages.			Erratic load readings Bonding difficulties Temperature problems

Table 9.1. Summary of Major Case Histories Reviewed (Cont.)

Project Name and Author(s)	Project Description	Parameter Measured and Instrumentation Type					Project Particulars
		Vertical Movement	Horizontal Movement	Load	Pore Pressure	Other	
Federal Reserve Bank, Boston -C.J. Dunnicliff -W.E. Jaworski (1974)	Tied back excavation in Boston	Survey settlements of soldier piles. Settlement pins in street.	Survey measurements of movements of soldier piles. Inclinometers	Tell-tale rod load cells. Mechanical load cells. Hydraulic jacks (initial)		Tie deformation monitored during proof testing.	Tell-tale rod load cell inexpensive, calibrated in place, self-compensating for temperature. Estimated accuracy of tell-tale rod load cell greater than ±5%.

monitoring strut or tie-back loads the engineer can also determine to some extent whether the contractor is using proper construction techniques and, if not, require revision of those techniques. By monitoring loads in a test section, it may be possible to reduce material and labor costs if load readings indicate that less support is needed for the excavation. It should be remembered, however, that load measurements are not sufficient for these purposes; other parameters should also be measured. Support loads must be correlated with soil and water conditions, horizontal and vertical movements, construction procedures, as well as any other factors that may affect loads or overall performance of the excavation.

Advancing the state-of-the-art is important in terms of its long range effect on the design and construction of future projects. While economic savings may not occur in a specific project, it is possible that significant economic savings may occur through advancements in design and construction knowledge. A properly planned and implemented braced excavation instrumentation program will provide significant information on earth pressures, the importance of specific construction details, and the relationship between support loads and movements.

9.3 Problems in Monitoring Loads

Many of the problems encountered in monitoring loads in braced excavations are no different from those encountered in other types of geotechnical instrumentation. Chapters 3 and 5 discuss some of these problems. Problems encountered when monitoring loads are often severe, largely because reading changes are small, and the instrumentation is sensitive. Some of the major problems are discussed below.

Environment. The most critical problem, and at the same time the most insurmountable, is the environment. The environment of the excavation refers not only to temperature, moisture, and other climatic features, but also to the construction environment, with its inevitable access restrictions and damage potential.

Although in some cases it may be possible to control the climatic environment of the excavation to a certain extent, in general very little can be done to reduce the effect of the climate on the instrumentation. The large temperature ranges often encountered in excavations can completely distort the output of load measuring devices (Dunnicliff and Jaworski, 1974). Significant steps have been made in recent years to compensate for temperature, but temperature problems still remain. The following discussion will illustrate the significance of temperature change if strain gages are used as load measuring instruments.

If strain gages are used to measure load, all causes of strain other than stress must be accounted for and subtracted from measured strain. Only then can modulus of deformation be used to convert strain to stress change, and hence to load change. The relationship between measured strain, stress change and temperature change, and the importance of accounting for temperature strain, is illustrated below:

Assume: L = Gage length of strain gage

ΔL = Total change in gage length of strain gage

ΔL_σ = Change in gage length of strain gage due to stress change

ΔL_T = Change in gage length of strain gage due to temperature change

$\Delta \sigma$ = Change in stress

ΔT = Change in temperature

E = Modulus of deformation of material on which strain gage is mounted

ξ = Thermal coefficient of material on which strain gage is mounted

Then:

Total strain = Strain due to stress change + Strain due to temperature change

$$= \frac{\Delta L_\sigma}{L} + \frac{\Delta L_T}{L}$$

The strain due to stress change is:

$$\frac{\Delta L_\sigma}{L} = \frac{\Delta\sigma}{E}$$

The strain due to temperature change is:

$$\frac{\Delta L_T}{L} = \xi\Delta T$$

Therefore: Total strain = $\frac{\Delta\sigma}{E} + \xi\Delta T$

At a recent project in Boston (Dunnicliff and Jaworski, 1974) for an extension to the rapid transit system a 50-foot deep cut-and-cover excavation was braced using 24-inch diameter pipe struts with 1/2-inch wall thickness. Design load was 520 kips. Strain was measured using a mechanical strain gage, and measured strain was therefore the sum of strain due to stress change and strain due to temperature change. Temperature was also measured and temperature strain subtracted before conversion to stress. By using the above equation, an error of 5°F in temperature measurement (corresponding to an error of 32 microstrain) caused an error of 35 kips in the calculated strut load. The need to take account of temperature strain is evident.

In addition to temperature, moisture has been found to affect load readings and overall performance of instruments. The various sealants and covers used to protect instrumentation generally have been successful in preventing moisture from reaching the sensor. However, these covers often act as heat sinks and cause temperature differentials to exist between the sensing instrument and the measured member, aggravating the temperature problem discussed above.

The rough treatment of equipment typically encountered in construction work presents a major obstacle to monitoring of loads in struts and tie-backs. Load measuring equipment is generally delicate, so that normal construction handling is likely to result in erroneous readings. Access to the instrumentation often is very difficult in braced excavations. Load measuring equipment is frequently located within the contractor's operation, therefore requiring the cooperation of the contractor during installation as well as during the monitoring phase. The degree of cooperation gained from the contractor is a major factor in success or failure of an instrumentation program.

Installation and Monitoring Procedures. The delicateness and sensitivity of load measuring instruments requires that installation be made with extreme care by qualified personnel. Alignment of gages, attachment to structural members, and protection against the environment all must be performed correctly. Improper installation often results in a drastic reduction in the effectiveness and reliability of the instrumentation program. Cording et al (1974) and Jaworski (1973) discuss some of the installation problems in detail. Correct installation not only requires that installation personnel be technically competent, but also that they have the patience and desire to ensure correctness.

When instruments are installed and operating, data must be obtained and processed. It is important that the observer be competent to take the readings and that he understands the importance of precise reading. Small reading errors may result in substantial erroneous load measurements. Garrison (1957) mentions the importance of using the same observer to take readings for ensuring consistency in reading procedures. This is particularly significant for instruments with non-digital readouts.

Expense. A well planned and executed instrumentation program is costly. As discussed in Chapter 4, the program must be approached in a systematic manner. Load measurement programs require considerable expenditure for both material and employee costs. If an instrumentation program is deemed desirable, the owner must be prepared to pay for correct planning and execution. If insufficient funds are allocated for instrumentation, the quality of the program will suffer, and little useful data will be obtained.

Required Accuracy. An important requirement when planning an instrumentation program is to establish reading accuracy. When applying this requirement to the instrumentation of loads, a variety of possible criteria can be given. Bjerrum, Kenney, and Kjaernsli (1965) list a range of accuracies for three typical instrumentation systems that vary from ±10 percent to ±40 percent of the true load. Depending on the particular project, either of these accuracies may be sufficient to satisfy the purposes of the instrumentation program. In general, an accuracy of ±10 percent of design load is desirable and would probably be required for any significant advances in the state-of-the-art of braced excavation design. Any effort to seek a greater accuracy is likely to be both unnecessary and fruitless (Andresen, 1972).

9.4 Total Monitoring Program

Normally, load measurements are only part of the total instrumentation and monitoring program necessary for a braced excavation. A typical monitoring program will include instrumentation to measure vertical and horizontal ground and structure movements, groundwater, occasionally stresses in walls, earth and water pressures on walls, and bottom heave. Chapter 5 lists in greater detail the parameters that may be measured in a braced excavation. However, a monitoring program does not simply consist of recording instrument data.

To permit an analysis of cause and effect it is necessary to maintain a complete record of relevant construction data together with other monitoring data. These data should include:

1. A record of depth of excavation versus time, at close stations.

2. Time of installation of all walers, struts and ties, with preload records, if any, and depth of excavation below strut or tie at time of installation.

3. Incidences of extraordinary ground losses, groundwater behavior, observed distress, or any other unusual event.

4. Complete as-built construction plans and records, including records of any pile driving.

5. Environmental factors which may, in themselves, affect monitored data, e.g. temperature, nearby construction activities.

In order to achieve any advancement in the state-of-the-art of design or construction of braced excavations the entire performance of the excavation must be evaluated, and measurement of a single parameter is generally of little value unless it can be related to at least one other parameter.

9.5 Load Monitoring Instruments

Appendix B presents an inventory of instruments available for use in monitoring loads. Loads are measured either directly using load cells or indirectly using strain gages. In general, loads in struts are measured using strain gages, and loads in tie-back anchors are measured using load cells. In Appendix B, there are at least 11 different types of strain gage and 11 different types of load cell, often with many manufacturers of each type. This section describes the instruments most commonly used to monitor loads in struts and tie-backs, and includes a discussion of advantages and limitations of each instrument. These data are summarized in Table 9.2. Instruments are illustrated on Figs. 9.1 through 9.8. Sections 9.6 and 9.7 describe the preferred monitoring methods in greater detail.

Mechanical Strain Gages. Mechanical strain gages provide a simple means of measuring the distance between two fixed points on the strut or tie-back. Typically, the distances are measured using a portable dial gage device graduated in 0.0001-inch increments. The most frequently used instrument is the Whittemore gage, which is used in conjunction with an independent temperature measurement of the structural members. The Demec strain gage, as shown on Fig. 9.1, is similar to the Whittemore gage, except for the method by which the dial gage is actuated. In the Whittemore gage the body telescopes, with one end bearing on the dial gage stem so that the stem moves directly with that end. In the Demec gage the movable end is part of a lever arm with the other end of the arm actuating the dial gage stem. In practice this arrangement is preferable because it substantially increases precision.

The major advantages of this system are basic simplicity, relatively low cost, and absence of delicate parts attached to the structural member. The gages are relatively easy to use and install. However, patience and care are required to obtain valid readings. The equipment and installation costs of a mechanical strain gage system are significantly less than an electrical system.

The major disadvantage of a mechanical system is one of access to the gage points. This is particularly severe as excavation progresses. Access may require use of the contractor's excavating bucket, or other construction stoppage, which in itself increases costs. Inaccuracies in mechanical systems have occurred primarily due to improper placement of

Table 9.2. Summary of Load Measuring Instruments

Instrument	Fig. No.	Advantages	Limitations
Mechanical strain gage	9.1	Inexpensive Simple Easy to install Minimum damage potential	Access problems Many corrections required Limited accuracy Readings are subjective
Electrical resistance strain gage	9.2	Inexpensive Remote readout Readout can be automated Potential for accuracy and reliability Most limitations listed opposite can be overcome if proper techniques are used.	Sensitive to temperature, moisture, cable length, change in connections, construction damage. Requires substantial skill to install.
Vibrating wire strain gage	9.3	Remote readout Readout can be automated Potential for accuracy and reliability Frequency signal permits data transmission over long distances Gages can be re-used	Expensive Sensitive to temperature, construction damage Requires substantial skill to install Risk of zero drift Risk of corrosion if not hermetically sealed.
Telltale load cell	9.4	Inexpensive Simple Calibrated in-place	Access problems Limited accuracy Cannot be used with all proprietary anchor systems.
Mechanical load cell	9.5	Direct reading Accurate and reliable Rugged and durable	Expensive Access problems
Electrical resistance strain gage load cell	9.6	Remote readout Readout can be automated	Expensive Sensitive to temperature Risk of zero drift Limited cable lengths

Table 9.2. Summary of Load Measuring Instruments (Continued)

Instrument	Fig. No.	Advantages	Limitations
Vibrating wire strain gage load cell	9.7	Remote readout Readout can be automated Frequency signal permits data transmission over long distances	Expensive Sensitive to temperature Risk of zero drift
Photoelastic load cell	9.8	Inexpensive	Limited capacity Access problems Requires skill to read

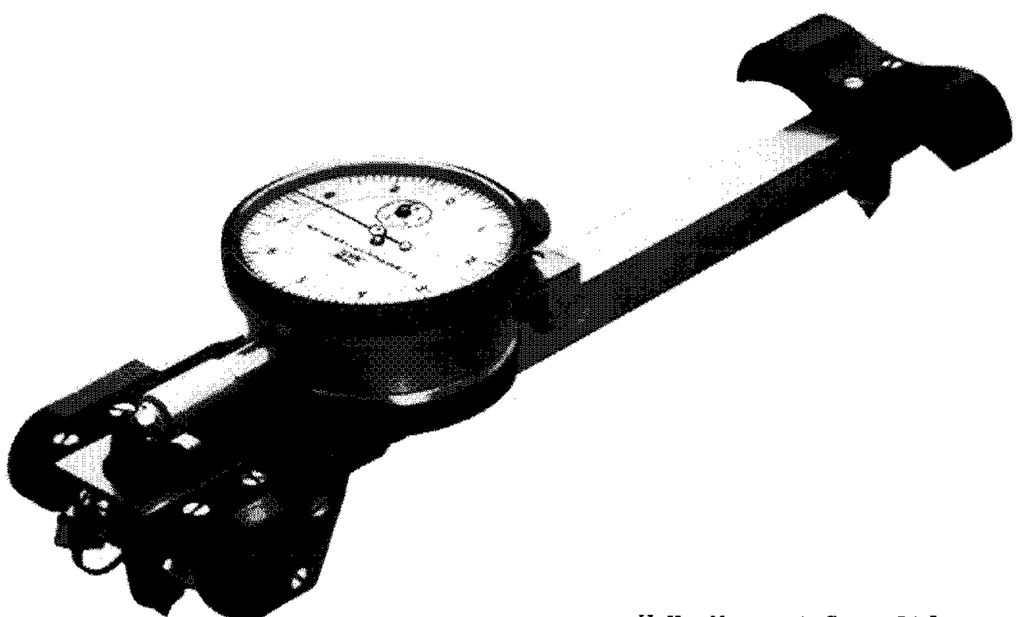


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Figure 9.1. Demec Mechanical Strain Gage

the measuring points. If the points are not rigidly attached to the structural member, strain readings will be worthless. Readings are somewhat subjective, and if maximum precision is required the same man should always take readings (Garrison, 1957). Use of the gage requires application of several corrections:

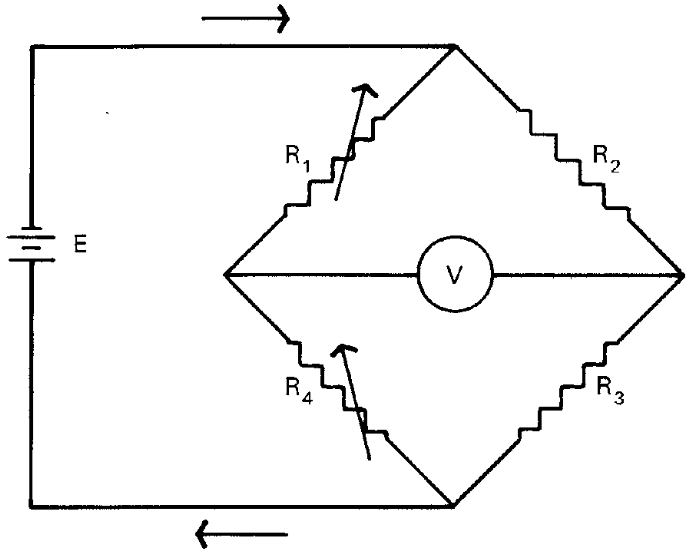
1. Temperature of structural member
2. Temperature of strain gage
3. Temperature of reference bar
4. Mechanical changes in strain gage
5. Bending of structural member

It is necessary to read the gage on a portable standard reference bar, thereby correcting for any change in mechanical characteristics of the gage. The gage itself is sensitive to temperature, as are most standard reference bars (even though of invar steel). Cording et al (1974) describe an ingenious method to overcome problems associated with temperature changes in the reference bar. This method is to use a bi-metal bar which maintains constant length independent of temperature change.

Electrical Resistance Strain Gages. Electrical strain gages have been extensively used to measure strains in a laboratory environment (Andresen, 1972) and are often the key sensing elements in common laboratory electronic sensing instrumentation (pressure transducers, load cells, etc.). The principle of the resistance strain gage (Fig. 9.2) depends on a linear relationship between the length of a wire and its electrical resistance. Gages are either bonded or welded to the structural member.

Resistance strain gages have been used on various braced excavation projects to monitor strains in support members (Garrison, 1957; Dunnicliff and Jaworski, 1974; Mahar, et al, 1972;Oosterbaan and Gifford, 1972), and significant problems have been reported.

The two major advantages of resistance strain gages are: remote readout facility for automatic reading, and relatively low cost. In addition, their performance has been proven in a controlled environment. The readout unit for the gages can



$$\frac{R_1}{R_4} = \frac{R_2}{R_3}$$

Basic Wheatstone Bridge Circuit

E = voltage source

V = voltmeter

R_1 = the resistance strain gage attached to the structural member (R_1 unknown)

R_2, R_3 = ratio arms that maintain R_2/R_3 constant, independent of temperature change

R_4 = known precision variable resistor

R_4 is adjusted to null voltmeter V

Then $\frac{R_1}{R_4} = \frac{R_2}{R_3}$ and $R_1 = R_4 \frac{R_2}{R_3}$

Hence R_1 = constant x R_4

Figure 9.2. Electrical Resistance Strain Gage

be placed at a location remote from the contractor's work area, and be connected to an automatic data acquisition unit, of which many types are available. However, automatic electronic systems which are designed for laboratory use are rarely suitable for field application. This problem is described in Chapter 5 and Appendix A. Although a resistance strain gage system is generally more expensive than a mechanical system, it is nevertheless less expensive than other remote sensing systems. Once the original outlay for readout equipment is made, additional gages increase cost only slightly. Under controlled laboratory conditions strain gages have proven to be very accurate, yielding extremely reliable data. They have been used with success in the aeronautics and aerospace industries, and clearly have the potential to fill needs of the tunneling industry. However, successful use in braced excavation has rarely been accomplished.

The major problem arising from use of resistance strain gages is their adverse reaction to the environment of an excavation. Temperature, moisture, long wire lead lengths, changes in connections, and construction damage all have been found to affect readings. The problems associated with moisture, wire lead lengths, connections and protection against construction damage largely can be overcome by using proper installation and calibration procedures. The temperature problem can be minimized by use of a full 4-arm bridge circuit at the measuring point rather than by use of the more normal half bridge. Attachment of gages by bonding is a time consuming and meticulous procedure, requiring considerable skill. Frequently insufficient skill is applied to this aspect, and reading drift occurs. Weldable gages (Cording et al, 1974), which are enclosed in insulated, hermetically-sealed covers, are easily affixed to a steel structural member. They largely overcome the reading drift problem, although they are less sensitive than bonded gages.

Vibrating Wire Strain Gages. The most commonly reported method of monitoring loads in struts across braced excavations is through use of vibrating wire strain gages mounted directly on structural members. The Norwegian Geotechnical Institute has used vibrating wire strain gages to monitor loads for many years and has reported relatively good success. The basic gage (Fig. 9.3) consists of two posts attached perpendicular to the supporting member, a distance of from 3 to 9 in. apart, with a taut wire between the posts. The wire is caused to vibrate by magnetic action at its natural frequency, and the vibration rate is then measured. Change in distance between the posts, and hence strain, is proportional to the change in the square of the vibration frequency.

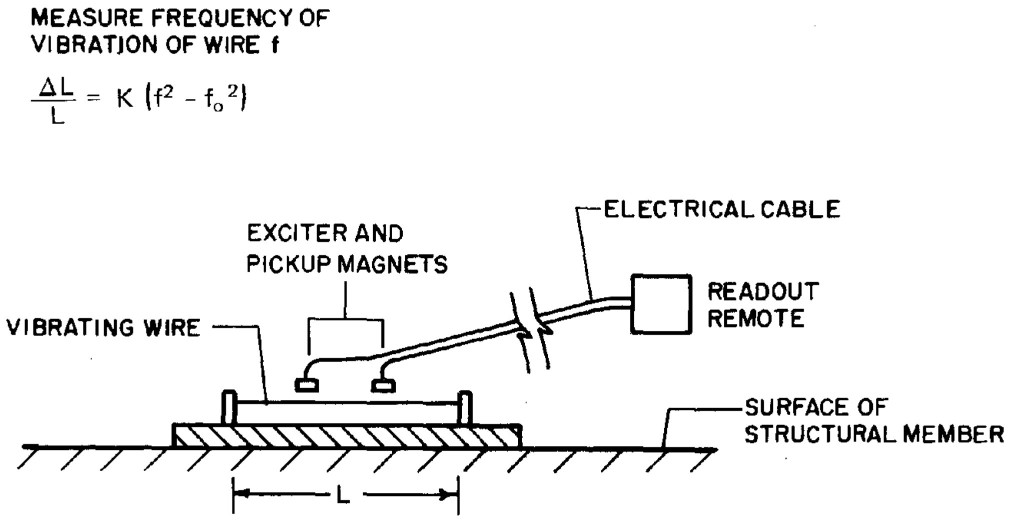


Figure 9.3. Vibrating Wire Strain Gage

Vibrating wire gages have been used to monitor loads at many different projects, and installation procedures and potential problems are well known. Thorough discussions of accuracy and sources of error are presented by Jaworski (1973), Bjerrum et al (1965), Norwegian Geotechnical Institute (1962) and Dunnicliff and Jaworski (1974). A recent definitive paper by O'Rourke and Cording (1974) describes gage performance, as well as accuracy and sources of error, and provides recommendations concerning choice of gage, installation, positioning, remote observation and supplemental measurement techniques.

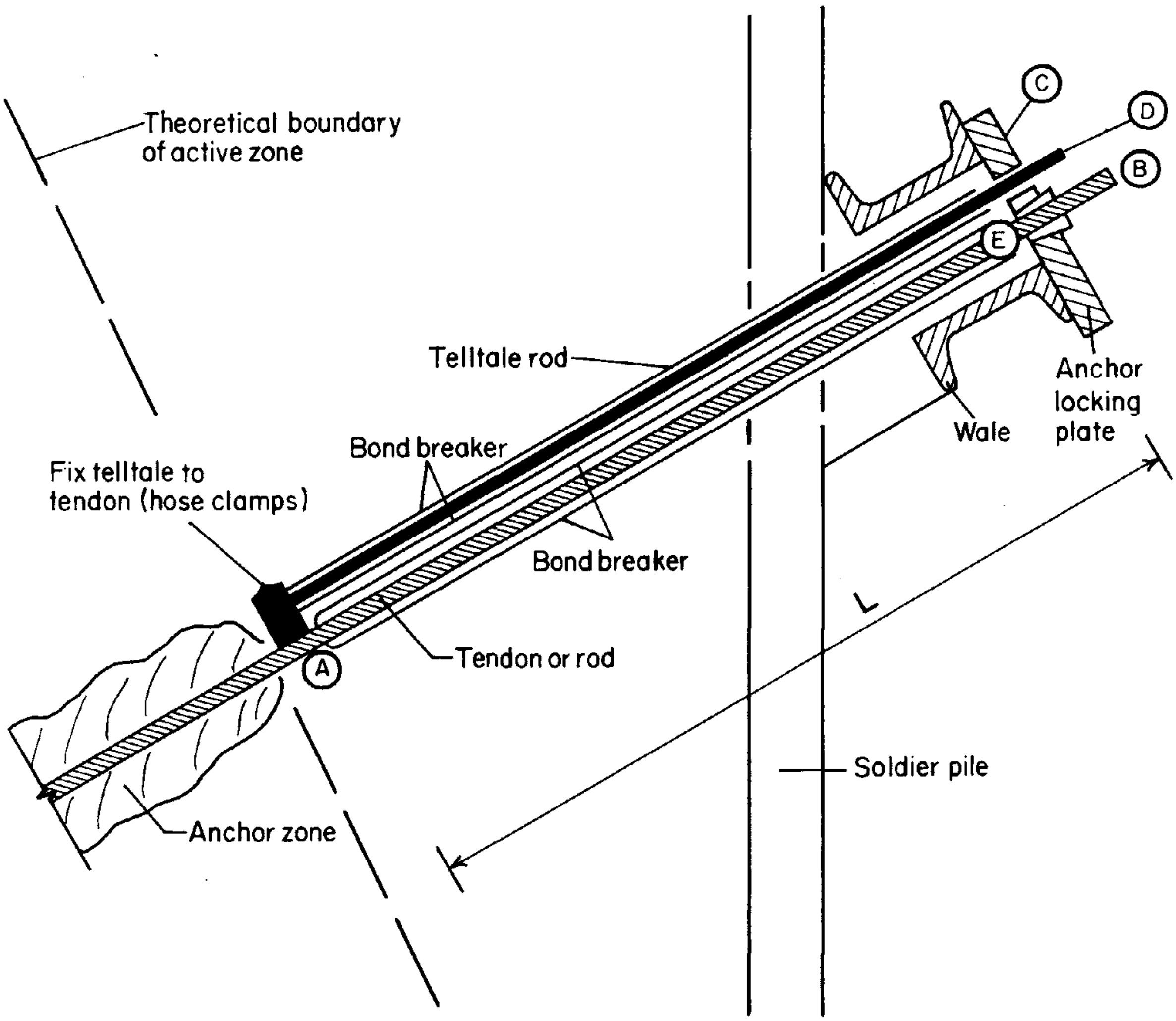
Several advantages are realized with the electrical resistance strain gage; notably the remote and automatic readout facility. However, a vibrating wire strain gage measures a mechanical property of the wire rather than an electrical property. Therefore electrical noise does not interfere with the output readings to the same extent as with a resistance strain gage. Signals have been transmitted over many miles without distortion. Accuracy and reliability can be high (O'Rourke and Cording, 1974), and although more expensive than electrical resistance gages, vibrating wire gages can be re-used.

Primary disadvantages of vibrating wire gages are their initial high cost ($55 to $150 each, depending on source), their sensitivity to temperature, potential for zero drift and corrosion. Temperature sensitivity is particularly severe if temperature differentials exist between the gage and structural member (Norwegian Geotechnical Institute, 1962). Moisture can cause corrosion of the vibrating wire and reduce the life of the gage. However, proper protective techniques are available. Jaworski (1973) attributes most of the zero drift problem to bending of the support posts and loosening of the locking nuts attaching the wire to the posts. The vibrating wire should be located as close as possible to the surface of the member, to reduce the effects of zero drift caused by any loosening of locking nuts or bending of support posts. Additional methods of minimizing the zero drift problem are discussed by O'Rourke and Cording (1974). Gages should be fully temperature compensated; i.e. they should have the same thermal coefficient of expansion as the structural member. Some advances have been made in this direction, but this should become a standard for any vibrating wire system.

Telltale Load Cells. Dunnicliff and Jaworski (1974) describe a simple load cell, consisting of a telltale rod, for use in monitoring tie-back anchor loads. The cell is illustrated on Fig. 9.4, and consists essentially of a length of sleeved 1/4-inch diameter steel rod. One end of the rod is attached to the tie just below the sleeved "unbonded" length, and the other end protrudes through the locking plate. The telltale rod remains unstressed while the portion of the tie alongside the rod acts as an elastic member in tension. The change in length of that portion of the tie is equal to the change in "stickout" of the rod, which is therefore a measure of load change in the tie. The change in "stickout" is measured using a portable dial micrometer. The cell is calibrated in place while stressing each instrumented tie, using a removable mechanical or electrical load cell in series with the stressing jack.

The primary advantages of this system are its simplicity, its minimal cost, and the fact that it is calibrated in place. The primary limitation is the requirement for sufficient annular space between anchor sleeve and inside diameter of the drill hole casing, which limits use to only certain of the proprietary anchor systems. Access to the tie end is required for reading. Accuracy is limited. Dunnicliff and Jaworski (1974) report an accuracy of about ± 5 kips when used to monitor loads in anchors with tensions up to 150 kips.

Mechanical Load Cells. Mechanical load cells, with a central hole through which the anchor passes, are used for measuring loads in tie-back anchors. Compression of part of



Load in tendon at (A) and (E) = P

Change in load in tendon at (A) and (E) = ΔP

Change in length L = ΔL

(D) moves with (A). (E), (B) and (C) move together.

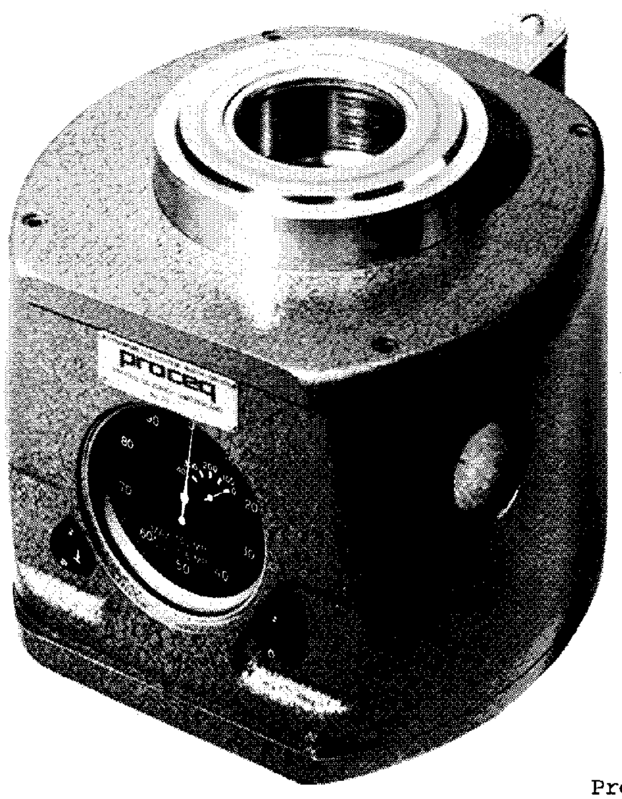
Therefore, change in "stick-out" distance (C) to (D) = ΔL

Hence if $\frac{L}{AE}$ is known, ΔP can be monitored by monitoring change in "stick-out" distance.

This is done with a portable dial micrometer. $P = \Delta L \frac{AE}{L} = \Delta LK$, where K is a constant obtained from field calibration.

Figure 9.4. Telltale Load Cell

the cell body is measured with a dial gage, normally via a lever arm or torsion system, and dial gage reading calibrated to load. A typical mechanical load cell is shown on Fig. 9.5.

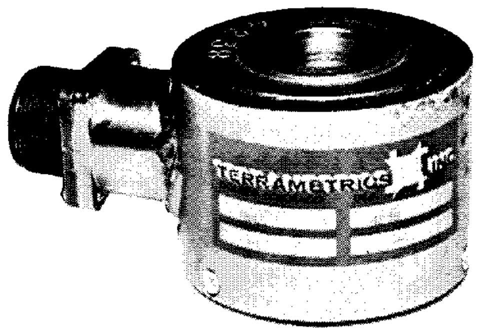


Proceq SA

Figure 9.5. A Typical Mechanical Load Cell

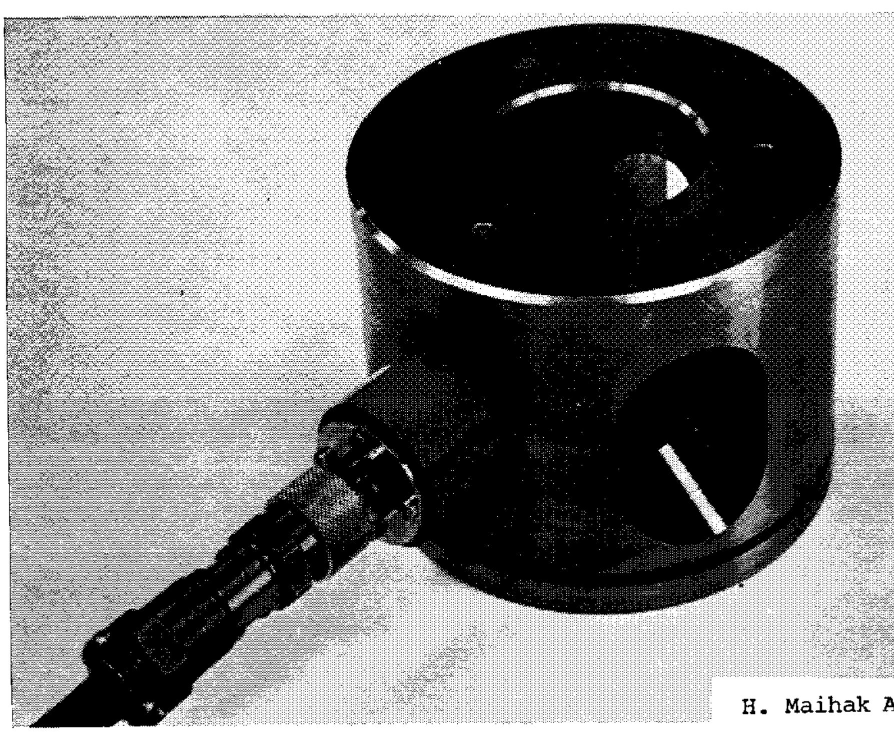
Mechanical load cells have proven rugged, durable, accurate and reliable. The only significant limitation is the requirement that access must be made available to read the cell.

Strain Gage Load Cells. Strain gage load cells suitable for use in monitoring tie-back anchor loads consist of a hollow cylindrical body. Strain gages, of either electrical resistance or vibrating wire type, are mounted on a machined part of the body so that strain can be calibrated to load. Typical cells are shown on Figs. 9.6 and 9.7. The major advantages and limitations discussed above for strain gages apply to these load cells.



Terrametrics, Inc.

Figure 9.6. Typical Resistance Strain Gage Load Cell



H. Maihak AG

Figure 9.7. Typical Vibrating Wire Strain Gage Load Cell

Photoelastic Load Cells. Photoelastic load cells, as shown on Fig. 9.8, consist of a disk of optical glass enclosed within a hollow steel cylinder. When the steel cylinder is loaded diametrically, its deformation applies a corresponding stress on the glass disk. The strain in the disk is visible in the form of photoelastic interference fringes when the disk is illuminated with polarized light and observed through a handviewer. The change in the number of photoelastic fringes observed in the pattern is directly proportional to the change in load on the steel cylinder, and hence, in the tie. An example of their use is described by Liu and Dugan (1972).

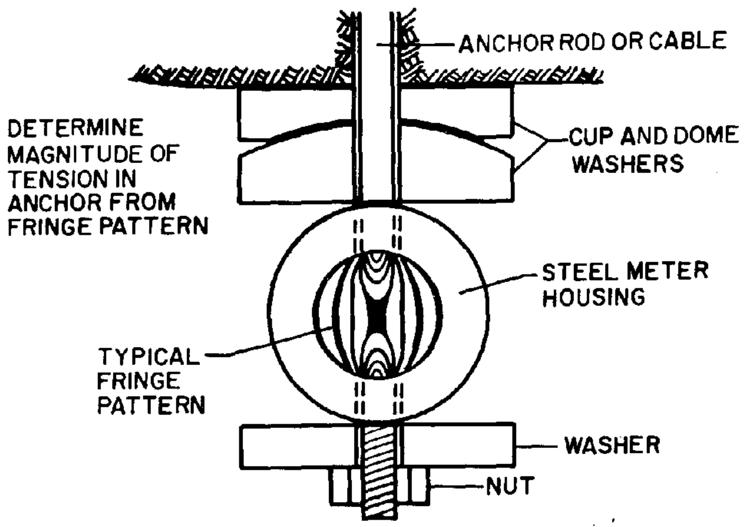
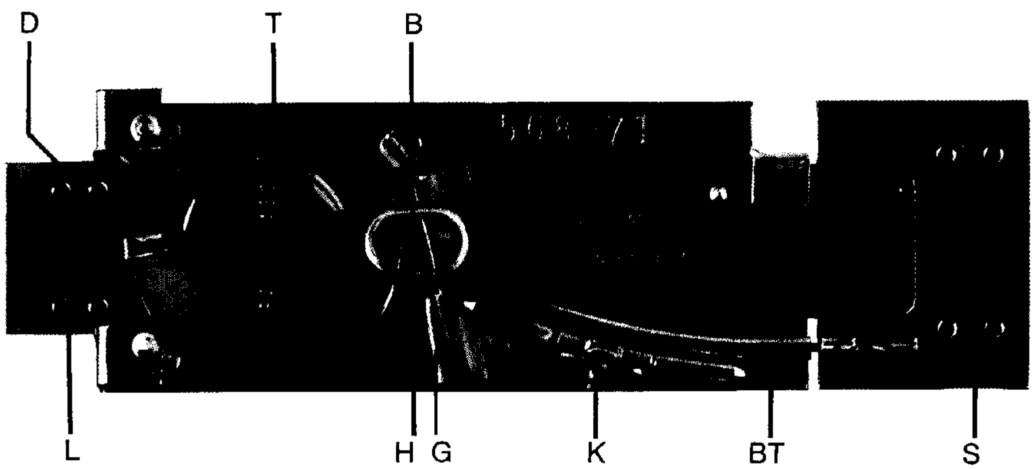


Figure 9.8. Photoelastic Load Cell

Photoelastic load cells are inexpensive, but currently available models with a central hole have a maximum capacity of 32 kips. Access is required to the cell for reading purposes, and reading requires both care and skill.

Other Load Measuring Systems. The above instruments are the most widely used for monitoring loads in braced excavations. However, other lesser known methods are sometimes used. Hanna and Seeton (1967) describe a strain gage based on the principle that the normal force required to deflect a tensioned wire a given amount is proportional to the tension in the wire. Portable hydraulic jacks have been used to measure loads on various jobs (Prasad et al, 1972), but with this system no continuous load monitoring is possible. Two other instruments show promise for measuring strut loads, but apparently neither has been used to date for this purpose. First, the standardizing resistance strain gage, as described in Chapter 5 and illustrated on Fig. 5.3; second, the Prewitt scratch strain gage, as shown on Fig. 9.9. The latter instrument provides an automatic record of strain by mechanical means, and appears to have good potential.



Method of Operation:

Pointer D scratches a record on target T of movements between attachment points of base plates L and S. Wires B are anchored to S and are guided to peripheral groove in target T by BT. When plates S and L are brought closer together, wire B causes target T to rotate in a counter-clockwise direction, proportional to the strain cycle being recorded. This combined action creates a slanted line on the target separating individual events. When S and L are separated, hold wires H prevent reverse rotation of the target while driver wires retract. Knob K is used to manually advance the target. Guide bar G simplifies target insertion. Targets are examined in a calibrated microscope, and radial scratch displacements are converted to strains.

Figure 9.9. Prewitt Scratch Strain Gage

9.6 Monitoring Strut Loads

Strain Gages vs. Load Cells. In recent years strain gages have been used more frequently to monitor strut loads than have load cells, primarily because inclusion of a load cell will tend to create non-typical loading conditions.

Lambe et al (1970) describe how construction details may affect loads measured in struts. Strain gages permit measurement of bending stresses, whereas a load cell does not. Load cells generally are more expensive than strain gages. In specific cases (DiBiaggio and Kjaernsli, 1961), load cells have been used for strut load monitoring where environmental factors were severe, but recent improvements in protection techniques for strain gage systems have eliminated the need for load cells in this application.

Preferred Method for Monitoring Loads. Since the most expensive feature of instrumentation programs is often the disruption to construction activities, remote readout is a desirable feature. Although occasional access to the instrumentation may be needed to check on any damage that may have occurred, instruments that require access for recording purposes should be avoided. Because of their remote readout capability and their proven records in the field and laboratory, vibrating wire and electrical resistance strain gages are emerging as the preferred methods for monitoring strut loads in braced excavations.

Vibrating wire strain gages have been used on many projects to monitor strut loads. Strain gages that are perfectly temperature compensated (equal thermal coefficients for gage and structural member) will provide optimum accuracy. The problem of thermal response, in particular due to temperature differential between gage and strut, and the problem of zero drift remain the most severe. Estimates of long term zero drift are necessary to judge the reliability of measurement. O'Rourke and Cording (1974) recommend that individual gages be mounted on unloaded sections of strut steel and monitored throughout the life of the project. However, this procedure cannot account for any drift effect resulting from stressing the member and gage. Establishment of a calibration set-up whereby gages could be stressed under calibrated load conditions is preferable. No-load gage readings should be taken after strut removal and then compared with the initial no-load readings taken before strut installation. Temperatures should be measured at the time of each recording, and readings should be corrected on the basis of temperature changes between the initial and subsequent values.

Electrical resistance strain gages have been used very successfully to monitor strains in laboratories. Their use in field measurements has yielded mixed results largely because of their sensitivity to environmental effects. Part of the problem stems from inexperience on the part of personnel undertaking the monitoring program. Since an electrical property of the wire rather than a mechanical property is being measured, it is important that these personnel have experience in field electronics. Engineers and technicians assigned to this task usually have training in geotechnical engineering but not in electronic instrumentation. If consulting assistance is obtained from electronic instrumentation engineers or technicians, their experience normally will be with laboratory instrumentation, and they will not be fully equipped to solve the severe problems imposed by the field environment. Consequently, the potential of electrical strain gages is good, even though the current state of practice is poor. For example, it may be necessary to use shielded cable to reduce electric noise pickup. A full four-arm Wheatstone bridge, as opposed to a quarter or half bridge, should be used at each strain gage location to reduce temperature effects on the measurements. Weldable gages (Cording et al, 1974) have not yet been used on a widespread basis, but the ease of installation and insulation of the gages appear to be great advantages. Their sensitivity, although less than that of bonded gages, normally will be adequate. Whether bonded or weldable gages are used, great care must be taken in making connections.

It may be feasible on some projects to use mechanical strain gages. The rather large inaccuracies that have been encountered (e.g. Bjerrum et al, 1965) are due most likely to improper placement of measuring points, and to improper temperature correction procedures. Cording et al (1964) recommend a procedure for drilling the fixed points into the strut so that no movement of the points is possible. By proper measurement of point placement, by proper application of temperature corrections, and by use of the Demec rather than the Whittemore type of gage, mechanical strain gages can be used to make strut load measurements of adequate accuracy. The Prewitt gage (Fig. 9.9) also shows promise.

There is no clear-cut preferable method for monitoring strut loads. Based on the present state-of-the-art, either electrical resistance or vibrating wire strain gages are recommended to be used as the primary measuring system, with the choice depending on experience and skills of available personnel. It is further recommended that a backup system be established, using mechanical strain gages. This system need not be as accurate nor be read as often as the primary system, but has very substantial value in verifying accuracy (or otherwise) of the primary system, at minimal cost. A cost effective research effort would consist of a field program to evaluate, on the same project, the relative merits of the various types of strain gages. The various types would be installed on the same strut. The same competent personnel would install the equipment using proper installation procedures as described above. In this way, many of the variables between projects (environment differences, installation procedures, construction differences) would be reduced or even eliminated, and an accurate comparison between the various gages should be possible. Thus a standard and reliable procedure could be established.

Location of Strain Gages. Location depends on the measuring objective. If axial load is the sole parameter of interest, (e.g., as a means of backfiguring earth pressures acting on the sheeting), gages should be mounted on the web at the neutral axis, thereby minimizing the effects of pure bending. If a pipe strut is used, gages should be mounted 90 degrees apart around the pipe. Jaworski (1973) and Norwegian Geotechnical Institute (1962) evaluate the validity of using only two strain gages rather than four or six at a particular location. However, if bending stresses are required to evaluate strut overstress, gages must be placed on opposite sides of and at a distance from the neutral axis.

To avoid end effects due to the connection between strut and wale, strain gages are generally mounted at least 5 ft from the ends of the struts. If stress concentrations near strut ends are of interest, then gages should be placed at those areas.

Finally, strain gages should be placed at locations causing minimal interference to the contractor during construction. This reduces risk both of gage damage and of a negative attitude on the part of the contractor.

Protection Against the Environment. Virtually all strain gage systems require that the instrumentation be protected against both the construction and climatic environment. Protecting the instrumentation against construction damage is often a major problem. The probability of damage must be recognized at the outset and, wherever possible, appropriate protective covers and conduits provided. The necessary precision of strut load measurements requires that special measures be taken to minimize adverse environmental effects. Established procedures are available for protecting vibrating wire and electrical strain gages from the influence of moisture. Compensation procedures for temperature are imperfect, but use of proper techniques reduces the problem to tolerable limits. Although partly temperature-compensated strain gages are available, most gages provide more accurate readings under constant or nearly constant temperature conditions. Any attempt to reduce temperature variations will probably result in more accurate strain readings.

9.7 Monitoring Tie-Back Loads

Load Cells vs. Strain Gages. Load cells have been more commonly used to monitor tie-back loads than have strain gages. The tensioned member of a tie-back anchor generally consists either of a single steel rod or several parallel stranded wire tendons. Strain gages are inapplicable for use on stranded wire tendons since no convenient method is available for attaching the gages. Furthermore a single load cell with a central hole can surround an entire group of tendons. It is possible to attach strain gages to steel rods, although the rate of gage attrition is generally high (Oosterbaan and Gifford, 1972). Installation of strain gages requires that they be attached to the rod prior to anchor installation. Subsequent access to the gages cannot be obtained, and no examination or maintenance is possible. Gage attachment is difficult since there is little available surface area on the structural member.

Preferred Methods for Monitoring Loads. There are few published case histories describing use of instrumentation to monitor tie-back anchor loads. Dunnicliff (1971) reviews some of the available equipment. It is clear that there is good reason to prefer load cells rather than strain gages.

All types of load cells discussed in Section 9.5 are suitable. Dunnicliff and Jaworski (1974) report successful use of telltale and mechanical load cells, while Liu and Dugan (1972) report successful use of photoelastic load cells. Resistance strain gage and vibrating wire strain gage load cells have also been used successfully. The choice depends on factors listed in Table 9.2, past personal experience of the engineers executing the monitoring program, and load cell availability. In view of the simplicity and economy of the telltale load cell, it seems logical to use this method wherever feasible. As discussed in Section 9.5, its use requires at least one other type of load cell for in-place calibration.

Portable hydraulic jacks have been used to monitor tie-back loads. This procedure, which is cumbersome, time consuming and inaccurate, is generally used only when approximate spot checks of loads are required.

The need for an independent load monitoring system for tie-back loads is probably less than for monitoring loads in struts, but the argument for checking all instrumentation readings is still valid and a backup system is warranted. The need for a backup system is clearly dependent, among other factors, on the purpose and required accuracy of the monitoring system. Rather than installing two different types of load cell, direct checks should be made to determine correct functioning of the selected type. This can be done in one of two ways, although neither way is always practicable. First, one or more load cells, on a special test frame or rod, can be retained on the site in a loaded condition. Readings can be examined for drift, and the cells can be checked periodically with a calibration check. Second, it may be possible to check and calibrate selected cells in place by pulling a tie to unload the cell and then releasing the tie. For this test, it is necessary to connect a calibrated load cell in series with the cell under test.

Protection Against the Environment. Since load cells are located on the walls of excavations above the working bottom, damage during construction is much less likely to occur than would be the case with strain gages in strutted excavations. The less cluttered nature of tie-back supported excavations also aids in decreasing the chance of damage. Nevertheless, load cells are equally susceptible to the climatic environment (temperature, sunlight, rain, etc.) Most load cells are sensitive to temperature variation, and some form of protection against the climate is often desirable. Protection may consist simply of a shield against the sunlight or it may consist of a strong mechanical protective box or channel.

9.8 Summary and Conclusions

1. Major problems are encountered in monitoring loads due to the effect on measurements of temperature, moisture and the adverse construction environment.

2. In general an accuracy of ± 10 percent of design load is desirable. Efforts to seek a greater accuracy are likely to be both unnecessary and nonproductive.

3. A monitoring program to measure bracing loads must normally be accompanied by measurement of other parameters. To permit an analysis of cause and effect, a complete record must be maintained of all relevant construction and monitoring data.

4. Load measurements can be made directly by using load cells or indirectly using strain gages. Sensing devices are either mechanical, electrical resistance, vibrating wire or photoelastic.

5. For monitoring strut loads, strain gages are used in preference to load cells. A cost effective research effort would consist of a field program to evaluate, on the same project, the relative merits of the various types of strain gages for monitoring strut loads. If such a program were performed, a standard and reliable procedure could be established for use on future projects.

6. Based on the present state-of-the-art, either electrical resistance or vibrating wire strain gages are recommended for use as the primary measuring system in strut load monitoring. The choice depends on experience and skills of available personnel. A backup system should be established, perferably using mechanical strain gages.

7. If electrical resistance strain gages are used for monitoring strut loads, engineers and technicians assigned to the task must have both geotechnical and field electronic instrumentation skills. Neither the average geotechnical engineer nor the average electrical engineer have appropriate skills.

8. If vibrating wire strain gages are used to monitor strut loads, test gages should be mounted on special sections of strut steel and monitored throughout the life of the project. This will provide a means of evaluating long term zero drift. Gages should have the same thermal coefficient of expansion as the structural member. The vibrating wire should be as close to the member as possible.

9. If mechanical strain gages are used to monitor strut loads, great attention must be paid to proper placement of measuring points, preferably by drilling into the strut.

10. For all types of strain gages, temperature corrections are of the utmost importance. Temperature correction procedures have been discussed.

11. For monitoring tie-back loads, load cells are used in preference to strain gages. Telltale, mechanical, electrical resitance strain gage, vibrating wire strain gage and photoelastic load cells are suitable. The choice depends on factors listed in Table 9.2, past personal experience of the engineers executing the monitoring program, and load cell availability. In view of the simplicity and economy of the telltale load cell, this method should be used whenever feasible. A backup system is desirable, although less necessary than for strut load monitoring.

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