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Instrumentation Reliability at Harvard Square Station

ROBERT P. RAWNSLEY, HENRY A. RUSSELL, and WILLIAM H. HANSMIRE

ABSTRACT

As a result of recent emphasis on mass transit, the Massachusetts Bay Transportation Authority has undertaken a large expansion program for its rapid transit facilities. Part of the expansion program was to modify the existing Harvard Square Station in Cambridge, Massachusetts. Slurry walls (concrete cast in place within slurry-filled trenches) were employed for both excavation and final lateral support. In some instances the wall passed within 7 ft of existing buildings of Harvard University. Concern for those buildings resulted in the use of specialized instruments to measure ground, building, and slurry wall movements. The instrumentation program had sufficient redundancy that key parameters were measured by more than one instrument. In addition, detailed field observations were recorded to accurately relate the instrument measurements to construction activity and geologic conditions. In most cases, the instruments performed as well as or better than anticipated. Some instruments were found to be inappropriate for the actual construction conditions. In the cases of poor instrument performance, sufficient redundancy existed that adequate measurements by other instruments were able to serve the monitoring function. The key to the reliability of the instrumentation program was the people involved. Instrumentation installation was done by experienced professionals. Instrument monitoring was performed by trained people who were on the job for extensive periods of time, were interested in the results, and were responsible for interpreting the measurements.

As a result of the recent emphasis on mass transit, the Massachusetts Bay Transportation Authority (MBTA) of Boston, Massachusetts, has undertaken a large expansion program for its rapid transit facilities. A major portion of this expansion program is the extension of the heavy rail Red Line northwest to the Alewife section of Cambridge, Massachusetts. To accomplish the 3-mile extension, the existing Harvard Square Station had to be modified and enlarged to allow realignment of the tracks to enable them to pass through the existing station and make a turn of approximately 90 degrees to the north. The modification of the station required nearly complete demolition of the existing structure and construction of new entrances and passenger platforms.

Slurry walls were employed for both excavation and final support. In some areas the slurry wall passes within 7 ft of existing buildings of Harvard University. Because of concern about ground and building movement, extensive instrumentation was used.

In addition to monitoring movements, strain gauges and pressure cells were installed in the slurry wall at selected locations to better determine the actual slurry wall behavior. That instrumentation was added as an FHWA/UMTA research effort after construction had started. A site plan and typical section of an instrumented panel are shown in Figures 1 and 2, respectively. A summary of the field monitoring is presented by Hansmire et al. (1).

INSTRUMENTATION PROGRAM

The primary purposes of the instrumentation and monitoring were to provide reliable information, to foresee problems, and to allow corrective measures to be taken to prevent damage to adjacent struc-

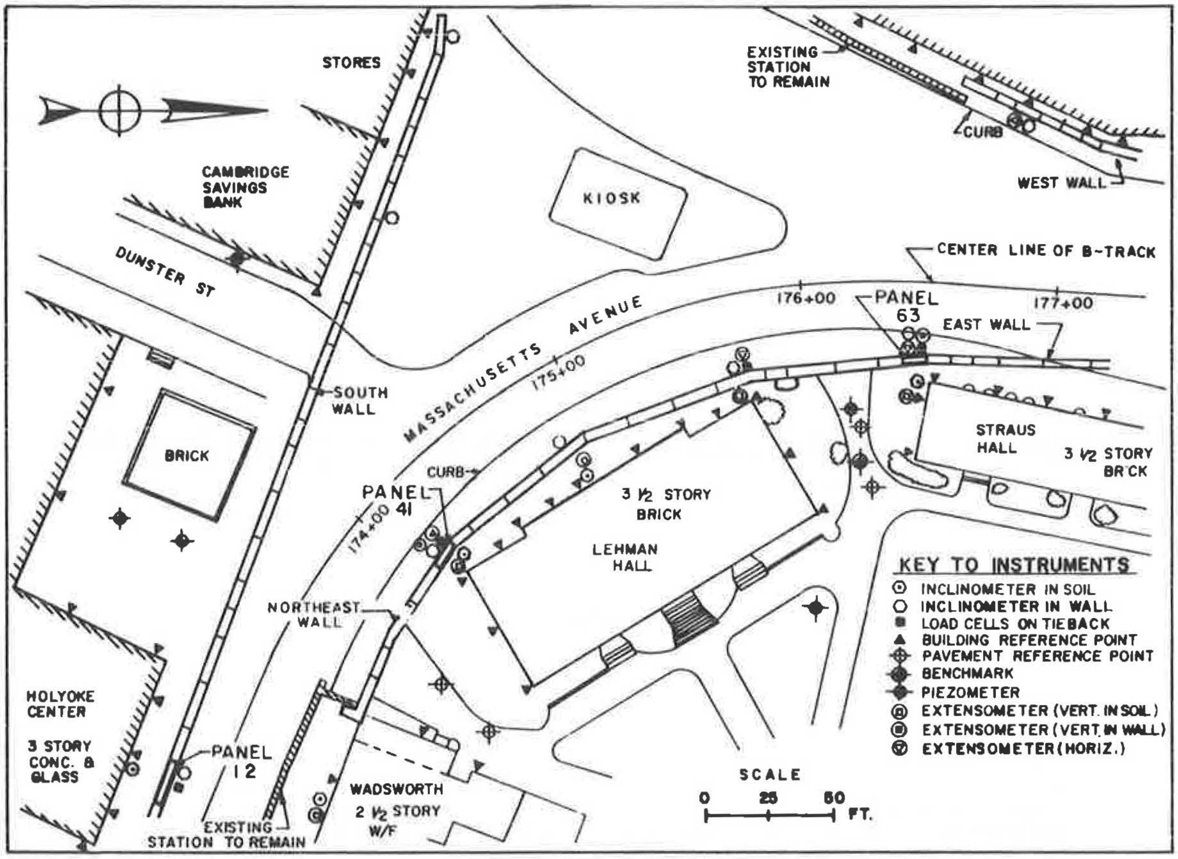

FIGURE 1 Site plan, Harvard Square Station.

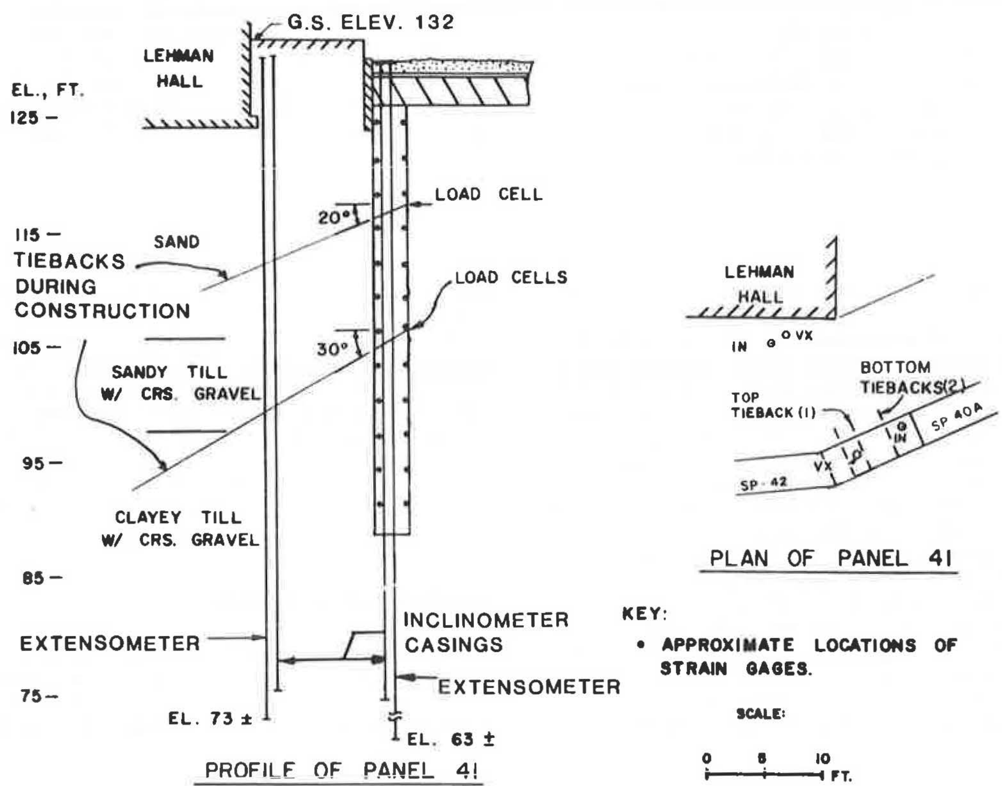

FIGURE 2 Typical instrumented section, plan and profile (Panel 41).

tures. The program also documented the case of only small or no movements and monitored the construction procedures for installation and performance of the tieback system.

Measurements were made of ground and building movements within the zone of influence of the excavation. Redundancy was provided by using more than one instrument to measure key parameters. The monitoring program included detailed field observations to accurately relate the instrument measurements to construction activity and geologic conditions.

A brief summary of the instruments and their specific purposes follows:

- Surface settlement reference points: a reliable method for monitoring settlement of buildings using optical surveying techniques.
- Benchmark: an essential reference for level surveys.
- Inclinometer: determination of lateral movement of slurry wall and soil adjacent to buildings.
- In-place inclinometer: variation of typical inclinometer that provided practically immediate determination of lateral movement at the time of reading.
- Horizontal extensometer: measured lateral movement away from excavation where inclinometers could not be used, such as under buildings; deepest anchor was beyond tiebacks and could verify their adequacy if movement developed.
- Inclinometer with subsurface settlement sensors: inclinometer with additional capability of settlement measurements; used adjacent to buildings.
- Deep settlement sensor: precise determination of settlement of slurry wall or soil at depth between slurry wall and building.
- Tiltmeter: redundant measurement on buildings intended for indirect detection of settlement.
- Load cells: direct measurement of force in tieback to monitor maintenance of required lateral support.
- Piezometers: groundwater level monitoring.
- Strain gauges: direct measurement of strain to compute actual bending moments in slurry wall.
- Earth pressure cells: measurements of soil pressures acting on the slurry wall.

Proper timing of instrument installation and reading was considered important to obtaining the desired results. A summary of the sequence that was followed is given in Table 1.

TABLE 1 Sequence of Instrument Installation and Construction Schedule

Event No.	Instrument Installation and Initial Measurement	Event No.	Construction Event
1	Preconstruction monitoring program including deep benchmarks, distant survey reference points, and building reference points		
		2	Begin construction contract
3a	Install inclinometer casing in soil near buildings		
3b	Install vertical extensometers in soil next to buildings		
3c	Install tiltmeter reference plates on buildings		
4a	Establish building and surface reference points		
4b	Establish base line for horizontal movements by triangulation and offsets from transit line		
4c	Establish survey reference points on instruments		
5	Install additional piezometers		
6	Complete initial readings on all instruments installed to date		
		7	Begin slurry wall construction
8	Install inclinometer casing in slurry walls		
		9	Install temporary columns and place decking or permanent roof
		10	Begin station excavation and install tiebacks
11	Install tieback load cells at critical sections		
12	Install horizontal extensometers		
13	Continue reading instruments as construction proceeds		

DESCRIPTION OF INSTRUMENTATION

Surface Settlement

Before any construction was done on the site, a program of preconstruction settlement monitoring was instituted. This program consisted of optical surveys to develop a baseline elevation for future measurements. This program established procedures and provided accurate initial elevations before construction. (See Appendix for requirements for the surveys.)

Six level circuits were performed before construction. A total of 47 reference points on adjacent buildings and 11 pavement markers was surveyed. During construction, additional reference points were added bringing the total number of reference points on buildings to 87. Generally, readings were taken biweekly during construction. Additional surveys were performed as required on selected points in areas of heavy activity or where readings indicated movements had occurred.

Surface settlement measurements were made on the preconstruction settlement monitoring points and the additional points. These additional points were shallow surface settlement points consisting of cross marks or pins set in pavement, curbs, building steps, and sidewalks. These points were generally located on a 25-ft grid outside the excavation for a distance at least twice the depth of excavation.

Benchmark

Just before construction and as part of the construction contract, three benchmarks were installed to a depth of 6 ft into bedrock. All benchmarks were surveyed, using existing MBTA datum, before construction to establish initial elevations.

Inclinometer

Before all structural excavation, inclinometer casings were installed in the soil between the proposed slurry wall and the existing adjacent buildings. Additional inclinometer casings were installed within the slurry wall of the new station. Inclinometers were used for monitoring lateral deformations of the soil, rock, and slurry walls.

Inclinometer casings placed within the slurry wall were installed through a 6-in.-diameter steel pipe that had been installed before concreting as part of the steel reinforcing cage. The bottom of the steel sleeve was plugged with Styrofoam to prevent the filling of the pipe with concrete. The inclinometer casing within the slurry wall was installed to a minimum depth of 10 ft below the slurry panel to ensure fixity. This was done by drilling through the 6-in.-diameter steel pipe and into the underlying soil or rock by rotary methods.

A 6-in. slip coupling (telescopic) was installed at the base of the slurry wall to prevent buckling of the inclinometer casing, should the concrete panel settle. The annulus between the inclinometer casing and the steel sleeve was filled with low-strength grout.

An inclinometer with high precision was employed to reliably detect movements of less than 0.10 in. The Slope Indicator Co. Digitilt Model 50306 readout and Digitilt Sensor Model 50325 were used. The nominal precision for measurements at the top of the casing was on the order of 1/10,000 of the total length of casing (for a 50-ft casing, this is 0.06 in.). However, experience with the instruments available revealed that a precision on the order of ±0.02 in. at the top of a 50-ft casing could be obtained with good reading techniques.

Field data acquisition was performed manually. To facilitate data processing, a computer was used for calculation. The results of the computer calculations were carefully reviewed to detect and eliminate obvious errors, such as transcription errors. All data were plotted manually.

In-Place Inclinometer

To provide capability for making rapid, timely measurements of horizontal movements adjacent to buildings in an area of heavy construction activity, an in-place inclinometer was used. The in-place uniaxial inclinometer consisted of six separate sensors attached vertically to one another by jointed stainless steel tubing. The sensors were set in the inclinometer casing perpendicular to the direction to be measured. The standard Digitilt Model 50306 readout was used for readings at a junction box located at the top of the casing. Lateral movements were determined at six depths and were easily read and interpreted. The precision of the in-place inclinometer was considered to be significantly greater than that of the regular torpedo device and was ideal for frequent or nearly continuous monitoring. Precision was on the order of ±0.005 in. for a 50-ft casing.

Horizontal Extensometer

The instruments were installed at predetermined locations along the slurry wall as well as through existing busway walls in areas where temporary support was required after removal of the existing roof slab. The instruments were the groutable type (Model E-10G manufactured by Irad Gage, Lebanon, New Hampshire).

The extensometers, which were considered simple and reliable, were located at different elevations to monitor wall and soil movements behind the wall. In particular, horizontal extensometers were used at locations where inclinometers could not be readily installed, such as beneath existing building foundations. The intent was to determine the extent of significant horizontal movements adjacent to the excavation. The extensometers were installed either individually at special sections or in conjunction with other instruments for more comprehensive monitoring of movements.

Eight horizontal extensometers were installed, seven with six anchors and one with four anchors grouted at varying distances from the readout head located on the wall surface. The anchor farthest from the wall was set far enough away from the excavation to be considered to be outside the zone of influence and beyond the longest tieback anchor. This anchor was considered fixed and served as a reference for the determination of absolute horizontal movements. For most installations, an ATLAS-COPCO ODEX system airtrack was used for advancing the borehole; this was the same rig that was used for tieback installation. Extensometers in the slurry wall were presleeved with an 8-in.-diameter steel pipe. In all instances the borehole was drilled at an angle of between 5 and 10 degrees down from horizontal. This slight angle allowed circulation of the drilling fluids for cleanout of the hole and facilitated grouting. When drilled, the borehole was surveyed by a closed hydraulic differential measuring system to locate the elevation of the borehole end.

The extensometer was read with a Mitutoyo micrometer depth gauge with a sensitivity of 0.001 in. The precision of the horizontal extensometer is on the order of ±0.002 in.

Inclinometer with Subsurface Settlement Sensors

Inclinometer casings at selected locations in the soil were equipped with special inductance rings located on a flexible rubber sleeve outside the inclinometer casing to measure settlement. The measurement system consisted of a modified Slope Indicator Co. Sondex Settlement Probe Model 50819. To achieve a higher degree of accuracy the measurement system was modified to use a Mitutoyo Model 192-116 height gauge. All measurements were computed relative to the initial bottom depth reading.

Deep Settlement Sensor

In critical areas, particularly near buildings, settlements were measured at depth with a vertically oriented, multiple-position rod extensometer. The instrument is similar to the horizontal extensometer described previously. All deep settlement sensors were installed by conventional rotary drilling methods.

Each installation contained four or six anchor points at which settlements were determined. The deepest anchor was fixed in rock below the excavation bottom and was considered to be below the zone of possible movements. The accuracy of the surface settlement measured by the deep anchor (which was, in effect, acting as a deep benchmark) could be verified by independent optical settlement surveys on the top of the instrument. This instrument was read with a Mitutoyo depth micrometer to 0.001 in. The precision of the instrument was on the order of ±0.002 in.

Tiltmeter

Ceramic reference plates (tilt plates) were fixed with a polymer epoxy to building walls and columns before construction. Often, however, the surface of the true structural building frame was not, or could not be, exposed. Installation was then often on facades or furred-out walls. The inclination of this plate was determined by a Slope Indicator Co. Model 50344 tiltmeter. The tiltmeter sensor was portable and easy to read. The instrument was sensitive to 10 sec of arc, which corresponds to a tilt of 0.03 in. in 50 ft. Precision of the instrument was on the order of ±20 sec of arc. These instruments were newly available and were believed to have promise as a simple monitoring tool.

Load Cells

Temporary lateral support of the slurry walls was provided by tiebacks. To monitor the tieback loads during and after tensioning, hollow center load cells were used. The cells were generally placed on both tieback levels of specific slurry wall panels at the locations of extensometers and inclinometers. The load cells were manufactured by Soil and Rock Instrumentation, Inc., and were designed for a tieback working load (100 percent design) of 282 kips with an overload factor of 2. The body of the cell was constructed of a high-strength steel, 4.5 in. high and 8 in. in diameter. A 5.5-in. hole in the center of the cell accommodated the tieback strands. Six pairs of temperature-compensating, bonded, resistance gauges and 12 unstressed strain gauges wired in series to form a four-arm Wheatstone bridge were mounted around the steel cell. The readout unit for the load cells was a Vishay Instruments, Inc., Model P-350 portable strain indicator. The sensitivity of the load cell was 0.1 percent of the maximum load or ±5.6 kips.

Piezometers

In addition to piezometers installed during the design phase exploration program, additional piezometers were installed at selected locations where the soil conditions or adjacent structures required close monitoring of the groundwater. The piezometers were read before construction and routinely during the several years of construction. All piezometers were Casagrande single-tube or double-tube type. Precision of readings was on the order of ±0.02 ft. The piezometers were read by using an electronic water level device that consisted of a contact probe lowered to the water level in the tube.

Resistance Strain Gauges

Hitec strain gauges were mounted on vertical reinforcing steel of both the front and rear faces of the reinforcing cage for two slurry wall panels. The Hitec HBW-35-125-6 strain gauge is a resistance-type foil gauge preassembled onto a weldable shim for ease of field installation. The shim is approximately 0.25 in. wide by 0.50 in. long. The Hitec gauge is connected to the surface with a four-conductor grounded cable having a polypropolene sheath.

The gauges were mounted on the interior face of the reinforcing steel. Each location was ground flat with a rotary grinder to provide a "slot" for the foil pad and the mounting tabs of the transition zone. When they had been ground smooth, the gauge tabs and ground area of the rebar were cleaned with acetone to remove any oil or other foreign matter.

After cleaning, the gauge was held in position and welded in place using an Ailtic spot-welding machine. The welds were performed following Hitec's recommendations: welding progressed from the center of each side of the tab outward to the end of the tab. This method ensures a good flat bond of the strain gauge shim to the rebar. The transition zone tabs were welded to each side of the rebar to securely fasten the transition zone and cable connection to the steel.

Additional waterproofing of each gauge was accomplished by spray-painting the welded area with a clear lacquer and, when dry, covering the entire area of the gauge with Scotch 2210 elastomeric compound. The cable from the foil to the transition zone was covered with Johns-Manville Duc-Seal to protect the thin wire connecting the foil to the transition zone from any abuse while the steel cage was handled.

Vibrating Wire Strain Gauges

The IRAD embedment gauge, Model EM-5, is a vibrating wire strain gauge approximately 6 in. long with a 1-in. flange at both ends. The flange at each end of the tube contained two 1/16-in.-diameter holes for mounting. The rubber-coated, two-conductor, grounded cable connects at the center of the tube and is sealed into the tube with a plastic potting compound.

The embedment gauges were mounted adjacent to the Hitec gauge locations. The purpose of this mounting was to provide measurements redundant with those made on the reinforcing steel. The gauges were mounted near and parallel to the vertical axis of the steel on the inside face of both sides of the steel cage. The gauges were supported by a 16-gauge wire, and the transition zone was taped to the reinforcing steel.

Earth Pressure Cells

Six earth pressure cells were installed to measure the earth pressure acting directly against the wall. The earth pressure cell chosen was the IRAD vibrating wire earth pressure cell in which the pressure inside an 8-in.-diameter fluid-filled flat jack is measured by a vibrating wire pressure transducer attached to a short tube. The cells were installed on the front and rear of the wall approximately 2 in. outside the reinforcing steel cage. The earth pressure cells were covered with approximately 2 in. of concrete between the cell and the soil. This installation was made despite concern that embedment in concrete would make meaningful measurements impossible. The cells were not jacked into the soil because the glacial till soils contained gravels and cobbles.

INSTRUMENTATION PERFORMANCE

An account of the performance of all the instrumentation used in the Harvard Square Station project is given in this section. Comparison is made between actual performance and predicted performance, and explanations of difficulties encountered and possible solutions to these difficulties are presented.

The overall performance of the instrumentation program at Harvard Square was judged to be good. The intent of the monitoring program was achieved. In most cases, the instruments performed as well as expected, or better. However, some instruments failed to have meaningful use. Detailed accounts of the performance of each instrument type, with suggestions for possible improvements, follow. In a subsequent section reliability is addressed.

Standard Inclinometer

The precision of the standard inclinometer was anticipated to be 1 in 10,000 or 0.06 in. in 50 ft on the basis of manufacturer's information. However, with frequent monitoring and careful reading techniques, it was found that precision of ±0.02 in. could be achieved. The primary reason such excellent precision was achieved was the frequent monitoring of the instruments for each phase of construction, which yielded a large number of repeated measurements. However, this does not account for the occasional "bad" readings. A high percentage of readings fits the limits of good precision, especially when many readings were taken in a single day.

The only major problem experienced with the inclinometer casings occurred during the winter seasons. Because many of the inclinometers were located within the slurry wall, the groundwater in the casings would freeze as the soil was excavated adjacent to slurry panels. The only feasible solution to the problem was to check the water level in the inclinometer casings regularly and, when needed, bail the casing to a depth equal to or below the top of excavation adjacent to the inclinometer.

In-Place Inclinometer

The precision of the in-place inclinometer was expected to be 1 in 100,000 or 0.006 in. in 50 ft. This precision was attained at Harvard Square.

Problems related to the in-place inclinometer were keeping the connections clean and dry under construction conditions and difficulty in transferring the inclinometer from one location to another. Because of the sensitivity of the in-place inclinometer, great care must be taken to ensure that all connections remain clean and dry. Should the connections become damp, the instrument's precision would be greatly decreased and thus the usefulness of the instrument would be greatly reduced. Further, it is not recommended that the in-place inclinometer be used when frequent relocation of the instrument is anticipated. Because of the nature of the instrument, it is both cumbersome and susceptible to damage when being moved.

Vertical and Horizontal Extensometers

The vertical and horizontal extensometers were expected to perform with a precision of ±0.005 in. It was found that the actual precision of the extensometers was greatly dependent on the care taken in the installation of the instruments. Great difficulty was encountered while installing certain horizontal extensometers because of adverse, tight working conditions. As a result of difficult installation, grout and soil may have entered a disturbed connection in the plastic tubing protecting the stainless steel rods, thus interfering with rod movement and decreasing instrument precision. In the horizontal position, more friction exists between rods and protective casing. In general, it was found that the vertical extensometers had a precision greater than that expected, ±0.002 in., and that the horizontal extensometers varied--some had a precision close to that of the vertical extensometers, others showed a precision worse than that which was expected.

Tiltmeter

Sensitivity of the tiltmeter is approximately one part in 10,000 or 10 sec of arc at 0 degree inclination. The precision of the instrument was expected to be ±20 sec of arc at 0 degree inclination. However, such precision was not achieved in this instance. This failure is attributed in part to temperature influences on the structure and to the fact that most installations were on nonstructural parts of buildings. For the small movements that took place, the tiltmeter was not of value and, therefore, was not a meaningful tool for monitoring on this project. It is recommended that the tiltmeter be used only when general movements or building settlements are expected to exceed 1 in., in which case sufficient tilt would take place to be measurable. Because 1 in. of settlement is usually unacceptable, the tiltmeter as implemented at Harvard Square is not considered suitable for such measurements.

Load Cells

The performance of the load cells was inconsistent. During tensioning of the tiebacks, there was often, but not always, a discrepancy between the jack used for tensioning and the load cell. Occasionally, the jack and the load cell were in agreement to within 5 percent of the applied load. The cause of discrepancy between the jack and load cell is believed to be loading eccentricities. In one instance, the tieback jack and a load cell were loaded together in the laboratory with excellent results. However, when the same load cell and jack were used in the field, the discrepancy was approximately 15 percent. Similar experience is reported by Fellenius for load measurement on piles (2).

When the load cells had been installed, the performance was fair. About one in five of the load cells began to malfunction after a few months of service. The remaining load cells appeared to have measured a change in tieback load with a precision of ±4 kips.

Sondex

It was found that for the small settlements that occurred at Harvard Square, the Sondex system was not sufficiently precise. Precision of measurement was on the order of a = ±0.1 to 0.2 in., which was the order of magnitude of settlement. Reliability was, therefore, not fairly tested.

Settlement Monitoring

The performance of the settlement monitoring instruments was excellent. This is a well-established technique, and other keys to success were the use of good reference points and rigorous procedures. Readings to 0.005 ft, based on a series of routine readings where no construction was taking place, were achieved.

Resistance Strain Gauges

The performance of the resistance strain gauges was extremely poor. Many of the gauges failed shortly after installation, and almost all of the remaining gauges exhibited great variations in readings. It is not recommended that resistance strain gauges be used embedded in concrete.

Vibrating Wire Strain Gauges

The performance of the vibrating wire embedment gauges was excellent. Of the more than 40 gauges installed, only one failed to function. All other gauges recorded consistently for the life of the project. From an engineering standpoint, however, uncertainty about concrete modulus and creep effects made interpretation of stresses quite difficult.

Pressure Cells

The performance of the pressure cells was, not surprisingly, extremely poor. Although the cells recorded consistent data, the magnitudes of earth pressure were inconsistent with what would be expected. It is believed that all cells were mechanically performing well because one cell was eventually removed from the slurry wall and tested in the laboratory with excellent results. The discrepancy in magnitude was most certainly the result of the fatal error of installation in concrete.

CONCLUSIONS

A broad way of answering the question of how reliable was the instrumentation is to ask, "Did the instrumentation do the job intended?" Broadly speaking, the answer was "yes" but in many forms and to different degrees. The instrumentation program was thought to have had thorough planning during design. Despite this, some instruments were found to be unreliable for a variety of reasons ranging from inappropriate application to flawed installation concept (e.g., pressure cells, tilt plates, resistance strain gauges, Sondex for too small movements). Other instruments were extremely reliable.

A discussion of the major conclusions follows.

Settlement monitoring with optical survey leveling was the source of the basic information from which potential for building damage could be evaluated. Reliability was high. Non-instrumentation specialists could understand the monitoring results that were provided with a high degree of precision.

Measurements of lateral movements in inclinometer casings provided confirmation that overall movements were small as indicated by the settlement monitoring. Reliability was high. The inclinometers did an extraordinarily good job of detecting subtle wall movements and consequently had a high degree of reliability to detect larger, possibly damaging wall movements, which, however, did not occur. The in-place inclinometer was difficult to move and should not be considered a readily portable instrument.

Load cells generally indicated that tiebacks carried the design load and their reliability for this function was fair. Where load dropped off, most often there was an inclinometer in the vicinity that showed little or no lateral movement. The reliability of load cells for indirect monitoring for potential building damage was low.

Vibrating wire strain gauges embedded in the slurry walls provided a measure of how much bending was taking place. Reliability was high. The more complex problem of concrete behavior under long-term creep conditions was not evaluated because of limitations in the design of the measuring system, not the instrument per se. On the other hand, the resistance strain gauges embedded in the slurry walls provided no interpretable information. This was believed to be due to instrument malfunction.

Tilt plates failed to give any meaningful information for two reasons. Located mostly on facades of buildings, they were subject to disturbance and noncorrectable temperature fluctuations and did not measure true building movement. Further, buildings would have had to move much more than acceptable before movement would have been detected by the tiltmeters. Tiltmeters were unreliable for the intended purpose.

Finally, the most important aspect of instrument reliability, when an instrument worked, was the people involved during implementation. Installation of instruments was done by qualified, experienced professionals who had a reputation to maintain. People reading the instruments were well trained, were on the job for extended periods of time, were interested in the results, and were responsible for interpreting the measurements.

ACKNOWLEDGMENT

The Massachusetts Bay Transportation Authority (MBTA) is the owner of this project. Design of portions of the station and the instrumentation for construction and research was done by Parsons Brinckerhoff Quade & Douglas, Inc. Coparticipant in the research was the Department of Civil Engineering, Massachusetts Institute of Technology. Research was funded by FHWA and UMTA.

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APPENDIX: SPECIFIED REQUIREMENTS FOR SETTLEMENT SURVEYS

Accuracy of Surveys

The accuracy requirements of the preconstruction settlement surveys were considered to be greater than that required for routine elevation surveys. Elevation surveys commonly involve rod readings to 0.01 ft and have a probable error of from ±0.01 ft to ±0.02 ft. For monitoring building settlements, greater accuracy was thought to be required--in the range of ±0.003 ft to ±0.005 ft.

Accuracy involves a number of factors and realistically cannot be obtained without special equipment and surveying procedures. The technical requirements for the preconstruction monitoring are outlined hereafter. They were intended to yield elevations with a probable error of ±0.003 ft (0.04 in.). It was recognized that a true indicator of the real accuracy of the readings would only be known after initial and a series of subsequent readings were made.

Technical Requirements for Instrumentation

Benchmarks

Each benchmark is to be constructed with an outer casing like that shown in Cording et al. (E.J. Cording, A.J. Hendron, Jr., H.H. MacPherson, W.H. Hansmire, R.A. Jones, J.W. Mahar, and T.D. O'Rourke. Methods for Geotechnical Observations and Instrumentation in Tunneling. Report UILU-Eng. 75 2022. University of Illinois, Urbana-Champaign, Dec. 1975.) The exact depth of the benchmark was determined at the time of drilling. In general, benchmarks were seated below the upper layer of highly fractured and weathered bedrock and below structure depth.

Reference Points

Several different types of reference points were required:

• Masonry anchor with machine screw (for use on brick or stone buildings),

- Lag bolt (for use on wood-frame buildings), and
- Masonry "PK" nails (for use on bituminous pavement or joints in concrete sidewalks or streets).

Surveying Equipment

Instrument: Lietz automatic level, Model B1, with a polarizing lens. Rod: Use same rod for all readings. Invar rod should be used and preferably purchased for exclusive use on the project. A Wild Model GPCE 10 was used on this project.

Field Procedures

Field procedures included

- Maximum line-of-sight of 100 ft;
- Balanced backsights and foresights;
- Careful plumbing of rod;
- To the greatest extent possible, use of identical party members for all surveys;
- Identical level circuit to be used for all surveys;
- Use of good quality pavement turning points (i.e., such as PK nails or equivalent); and
- Rod readings to 0.001 ft.

Instrumentation for Load Transfer in Socketed Pier Foundations

R. G. HORVATH

ABSTRACT

An investigation of methods to improve the performance of drilled pier foundations socketed into soft rock was made on full-scale test piers. Instrumentation of the test piers enabled load transfer behavior of the piers to be studied. Flatjack (FREYSSI) load cells were used to measure applied loads and the end-bearing component of pier load support. Vibrating wire concrete embedment strain gauges (Geonor) were used to determine axial and radial stresses at the middepth of the test section. Thus, load distribution along the length of the pier socket could be determined. The description, calibration, and installation of the instruments are briefly summarized. The satisfactory performance and reliability of the instruments are supported by the test data that, in general, were in good agreement with predictions from elastic solutions. Comparison of the results of several piers having different support conditions, and displacement measurements using telltale systems, also support the reliability of these instruments. The versatility of a flatjack load cell to perform three different functions: (a) passive load cell, (b) active (applied) load cell, and (c) void, at the base of one test pier subjected to multiple loading cycles, is also briefly discussed.

Field load testing of six full-scale, instrumented concrete piers socketed into weak rock was carried out to investigate methods of improving the performance of this type of foundation system (1,2). A summary of the load testing program is given in Table 1.

To gain a better understanding of the load transfer mechanisms operating in socketed pier foundations, measurements were made of

- The portions of load supported by shift resistance and end bearing,
- Displacements at the top, middepth, and bottom of the socket,
- Displacements in the rock adjacent to the pier socket, and
- Strains within the concrete pier at middepth.

Presented in the following sections are a brief summary of the test conditions and a description and discussion of each instrument, which includes calibration, installation, performance, and reliability.

TEST CONDITIONS

To minimize construction costs and to simplify inspection, instrumentation installation, and construction, a shale quarry was selected as the test site because the work could be carried out directly

TABLE 1 Summary of Field Load Testing Program

	PIER DESCRIPTION		TEST DESCRIPTION
P1	Smooth Shaft Conventional construction Void at base		Shaft resistance only
P2	Smooth shaft Conventional construction Load Cell at Base		Shaft resistance and End-bearing resistance
P3	Roughened shaft Shaft grooved (AFE) Void at base		Shaft resistance only
P4	Roughened shaft Shaft grooved (AFE) Load cell at base		Shaft resistance and End-bearing resistance
P5	Smooth shaft Conventional construction Preload cell at base		Preload applied to base Shaft resistance and End-bearing resistance
		A	Preload = 0.89 MN
		B	Preload = 1.78 MN
		C	Preload = 4.00 MN
		D	Shaft resistance only
P6	Roughened shaft Shaft grooved (ART) Void at base		Shaft resistance only

Notes: All test piers were auger excavated and had the following dimensions:
Socket diameter, D_s = 710 mm
Socket length, L_s = 1370 mm
Aspect ratio, L_s/D_s = 1.9
All load tests were axial compression tests.

TABLE 2 Summary of Engineering Properties of Mudstone (Queenston shale)

Test Description			Test Results Range	Ave.
Unit Weight	γ	(kN/m^3)	25.8 to 26.1	25.9
Water Content	w	%	4.1 to 4.8	4.6
Liquid Limit	w_L	%	--	22
Plasticity Index	I_P	%	--	3
Rock Quality Designation	RQD	%	29 to 88	70
Shore Scleroscope Hardness	S_h		14 to 19	16
Point Load Strength	$I_s(50)$	(MPa)	0.56 to 0.91	0.69
Brazilian Tensile Strength	σ_t	(MPa)	0.21 to 1.03	0.64
Uniaxial Compression Test				
Compression strength	σ_c	(MPa)	4.70 to 11.10	6.75
Secant elastic modulus	E_s	(MPa)	400 to 1180	695
Poisson's ratio	v		0.19 to 0.35	0.30
Triaxial Compression Test (σ_3 = 0.7 to 3.5 MPa)				
Cohesion	c	(MPa)	--	1.2
Friction	ϕ	(deg)	--	43
Secant elastic modulus	E_s	(MPa)	500 to 1600	1000
Poisson's ratio	v		--	0.22
Direct Shear Test				
Peak: (σ_n = 0.3 to 0.6 MPa)	c	(MPa)	--	0.3
	ϕ	(deg)	--	54
Residual: (σ_n = 0.3 to 2.8 MPa)	c	(MPa)	--	0
	ϕ	(deg)	--	29
Goodman jack				
Elastic Modulus	E_g	(MPa)	740 to 1420	1085

on the exposed rock surface. The site was located in Burlington, Ontario.

Material Properties

The rock exposed at the site consisted of predominantly weak red mudstone (Queenston shale) that became massive with depth. A summary of classification data and engineering properties of the shale is given in Table 2.

The average properties of the concrete were

Uniaxial compressive strength	σ_c = 49 MPa
Elastic modulus	E_s = 35 GPa
Poisson's ratio	v = 0.27

More detailed information concerning the material properties may be found elsewhere (1,2).

Test Pier Details

The dimensions of the test section of each pier were socket diameter D_s = 710 mm and socket length L_s = 1.37 m (Figure 1). The top of each test section was located approximately 0.6 m below the ground surface. The sockets for each test pier were excavated with a truck-mounted auger (Hughes-Williams LDH 100) that produced shafts with relatively smooth sides. Three test piers were constructed in shafts of this type. Three piers were constructed in shafts with grooves approximately 25 mm deep and 40 mm high cut into the wall to increase the roughness. Three of the test piers were constructed with voids between the bottom of the pier and the bottom of the socket to eliminate end bearing. The remaining three piers were constructed with load cells at their bases.

Reactor Frame

The main components of the load-reaction system were the reaction beam, anchor piers, and the anchor connections (Figure 2). The system was designed for a maximum safe test load of 8.7 MN.

Instrumentation

Suitable monitoring equipment was selected to provide the basic data necessary to study the load transfer behavior of socketed piers. Profiles of the

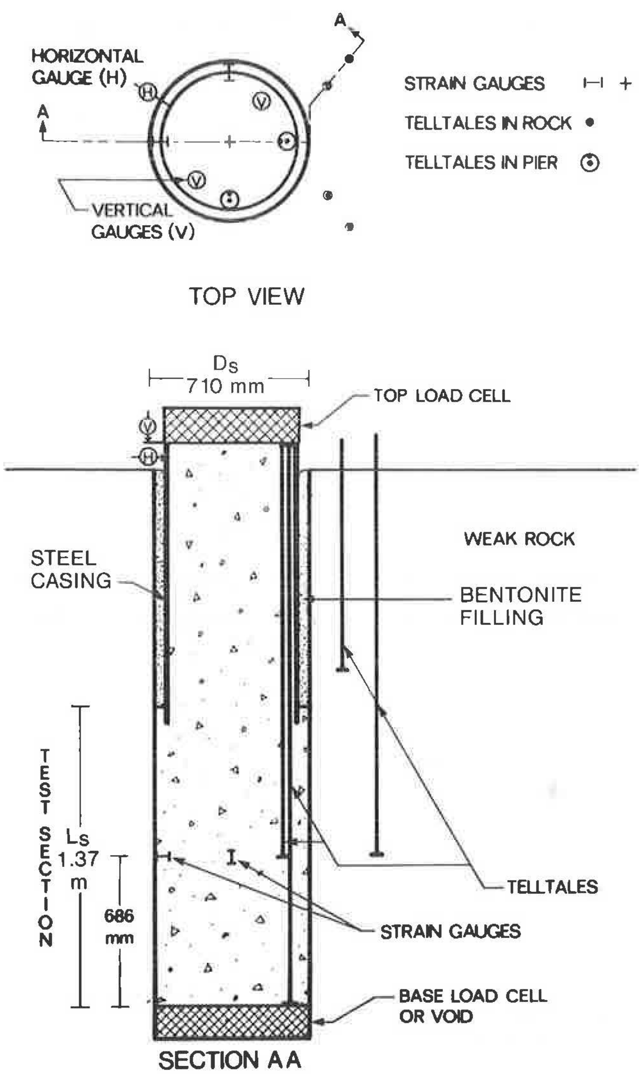

FIGURE 1 Typical test pier and instrumentation.

wall of each socket were made using a simple profilometer. The profilometer was basically a pantograph instrument, consisting of a feeler arm that followed the surface of the rock and a tracing arm that recorded the profile on paper.

Measurements of vertical displacements were made at two locations on top of each pier with dial indicator gauges fastened to wooden reference beams supported outside the influence of the test (approximately 2.5 m from the pier). Within each pier and in the surrounding rock, vertical displacements were measured using telltale rods and dial indicator gauges (Figure 1).

The test loads were applied and measured by means of oil-filled FREYSSI flatjacks. This type of flatjack was also used to measure end-bearing loads. Vibrating wire strain gauges were installed within the test piers for the purpose of estimating axial and radial stresses in the concrete during loading.

Construction and Instrumentation Installation

Each test pier was constructed individually to ensure that the concrete was placed on the same day that the socket shaft was drilled. The test piers were constructed between April 3 and April 18, 1980, and tested between May 17 and June 18, 1980. The sequence of construction and instrument installation was generally the same for each test pier. The steps were

1. The shaft was drilled with the auger to the required depth.
2. The shaft was visually logged, photographed, and tested with a Schmidt Hammer. Four profile traces, at 90-degree spacing, were made with the profilometer at the top and bottom of the test section.
3. Grooving of the socket wall (P3 and P4) was carried out at this point. Step 2 was repeated except for the Schmidt Hammer testing.
4. The bottom of the drilled socket was cleaned as required. Test piers P2, P4, and P5 were thoroughly cleaned by hand to provide a good clean surface for the bottom load cells.
5. A thin layer of grout was placed on the bottom of the socket (P2, P4, and P5 only) to provide a smooth contact surface for the load cells.
6. The load cell (P2, P4, and P5), or the void-forming device (P1, P3, and P6), was placed in position at the base of the socket.
7. Concrete was placed up to the middle of the test section and vibrated.
8. The concrete embedment strain gauges were installed (except P6).
9. The upper casing and telltale assemblies were placed in position.
10. Concrete was placed up to the top of the test section and vibrated.
11. After the concrete in the test section had set (next day) the casing above the test section was filled with concrete and vibrated.

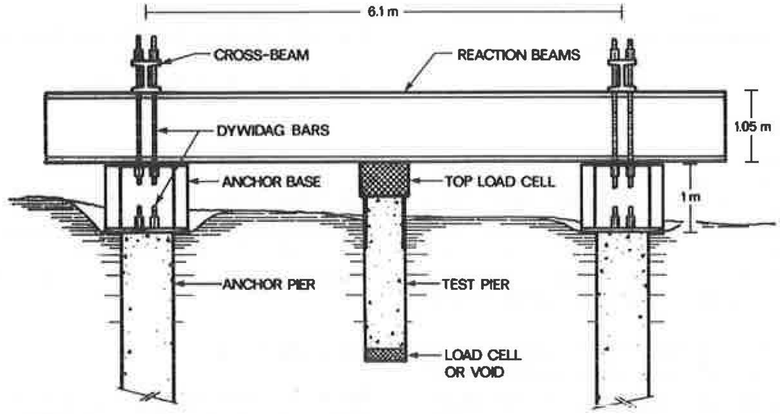

FIGURE 2 Load reaction frame system.

12. The annulus between the casing and the shaft, above the test section, was filled with bentonite to seal off any surface water.

13. Just before testing, the top of the pier was dry packed with grout to provide a smooth, level surface for installing the load transfer plates for the jacking system.

Load Application Procedure

A method of maintaining a constant rate of loading, similar to that used by Bozozuk et al. (3), was used for the tests. Load increments of 22 kN were applied at 15-min intervals. Each load was maintained for 13 min, and 2 min were allowed for adding the next load increment. Vertical displacement gauges at the top of the pier were read at 0-, 3-, 6-, 9-, and 13-min intervals for each load increment. Reading of all instruments, including horizontal displacement, rock displacement, strain gauges, and survey level, were taken at the 13-min mark.

Two test piers, P2 and P4, representing typical pier loading conditions (both shaft resistance and end-bearing resistance) were loaded in increments to 4.45 MN. This load was maintained for approximately 36 hr to observe possible changes of load-transfer behavior with the passage of time. On completion of the maintained-load portion of the test, incremental loading was resumed.

LOAD MEASUREMENTS

The loads applied to the top of the test piers and, in some tests, the loads transferred to the base of the socket in end bearing were both measured with load cells. The basic unit for each load cell was a FREYSSI flatjack pressure capsule that measures load hydraulically. All the load cells were calibrated before they were used in the field. The calibrations were obtained by duplicating in-service conditions as closely as possible.

Bourdon-type pressure gauges were used to measure the hydraulic pressure in the flatjacks. Two sets of gauges (0 to 6.9 MPa and 0 to 34.5 MPa) were used. The low-capacity pressure gauges were used to improve the accuracy at the lower stress levels. The gauges selected were accurate within ±1.5 percent.

Top Load Cell

The top load cell is unusual in that flatjacks were used for both applying and measuring the test loads. This load cell consisted of three to six 920-mm-diameter flatjacks positioned in series (Figure 3). The rated capacity of the flatjacks at a working hydraulic pressure of 14 MPa was 8.68 MN. Each flatjack was capable of expanding to a maximum opening of 25 mm.

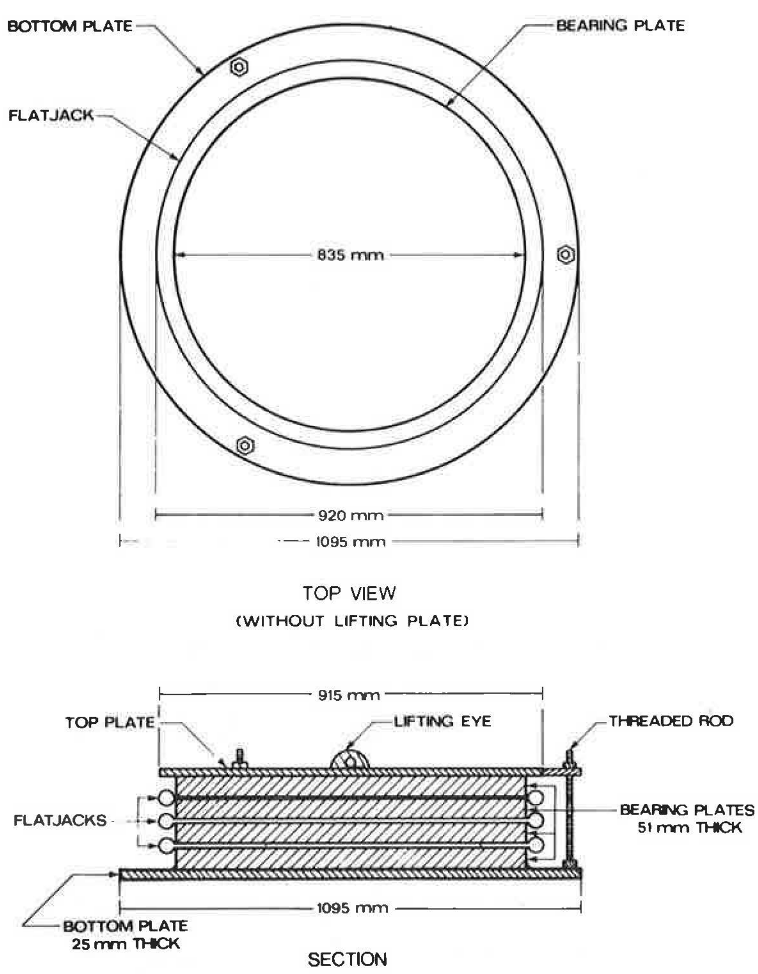

FIGURE 3 Top load cell.

Initially two of the flatjacks (partially expanded) were used as passive load cells. The remaining jacks were used as active jacks; that is, they were expanded by pumping hydraulic fluid into the jack thus applying the desired load to the top of the test pier. However, due to problems related to the lateral stability of the flatjack system, it was necessary to modify the procedure. Subsequently, all jacks in the system were initially deflated. A single flatjack was then inflated to near its maximum aperture (25 mm), closed off using the appropriate valves, and maintained as a passive load cell. If additional vertical movement was necessary, another flatjack was activated and the procedure was repeated.

The advantages of using flatjacks for applying the test load include

- At high loads, there is no problem with ram friction that can occur with piston-type hydraulic jacks;
- As long as the top surface of the pier is perpendicular to the shaft, the applied load will be parallel to the pier axis; and
- It is relatively easy to ensure that the applied load is concentric.

The top load cell performed well throughout the testing program. However, two incidents of equipment failure occurred that must be mentioned because they concern safety. It is important to note that both mishaps could have been avoided.

In the first case, a section of steel pressure tubing connected to one of the flatjacks pulled out of its fitting at a load of about 8 MN. Consequently a high-velocity stream of hydraulic fluid was expelled from the flatjack and covered a car about 10 m away. After this accident, all hydraulic fittings were changed to threaded-type connections. A partial shield (sheetmetal) was also installed around the top load cell to protect personnel.

In the second case, one flatjack ruptured at a load of about 7.5 MN. The high-velocity stream of oil expelled from the flatjack bent the protective shield and traveled about 10 m. The rupture occurred because the test pier had been accidentally constructed approximately 150 mm off the centerline of the load reaction frame. Consequently, the reaction frame and load cell were subjected to severe lateral and twisting distortion during loading.

Base Load Cells

The base load cells were made up of two or three 600-mm-diameter flatjacks positioned in series (Figures 4 and 5). The rated load capacity of these flatjacks was 3.46 MN at a working pressure of 14 MPa. The maximum displacement capacity was 25 mm for each flatjack. Two flatjacks were expanded approxi-

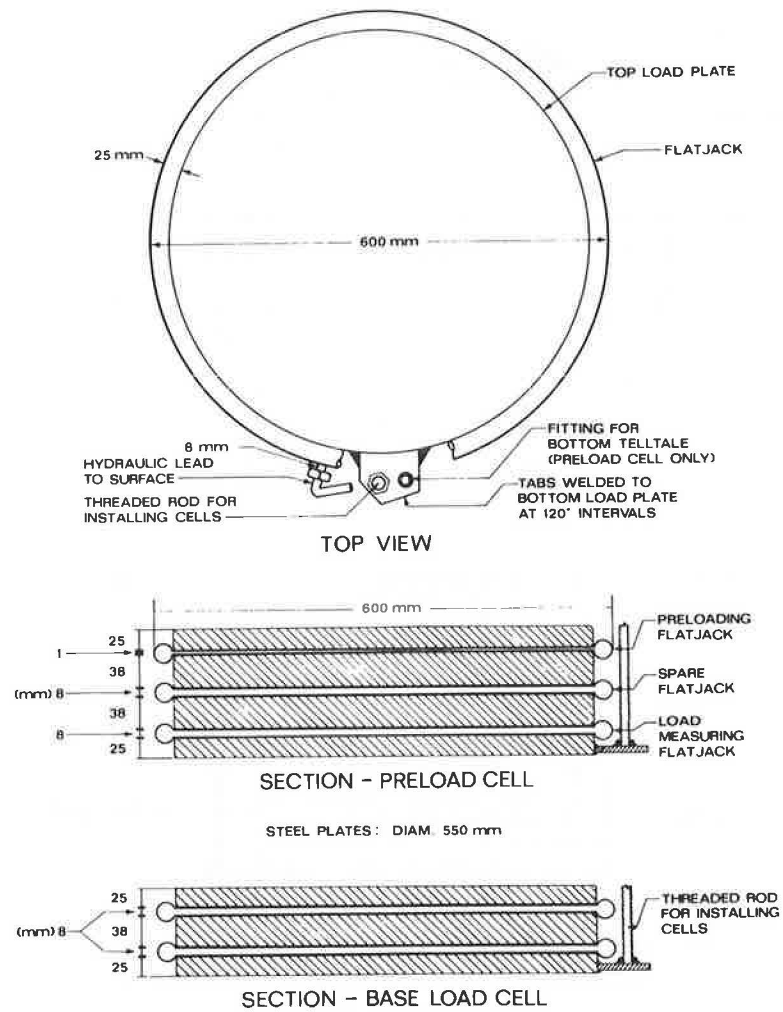

FIGURE 4 Base load cells.

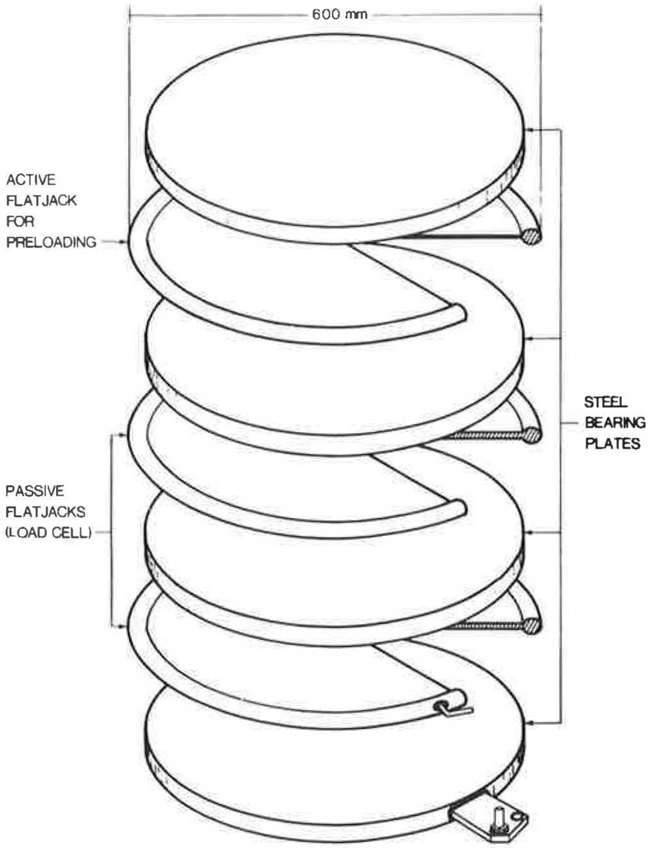

FIGURE 5 Base load cell—exploded view.

mately 6 mm before being placed at the base of the excavation. These cells were connected to pressure gauges in a closed hydraulic system and were used to monitor the load transferred to the bottom of the test piers. Only one flatjack was necessary for this purpose. However, a second one was included as insurance in case there was a malfunction of the first one.

A multiple-loading test procedure was developed to observe the effects of preloading the base of a drill pier in weak rock to improve the load-displacement performance (1). To implement the multiple-loading method, a suitable preload cell was required at the bottom of the test pier. This preload cell was capable of performing three different functions: (a) measuring the load transferred to the end bearing at the pier base (passive load cell), (b) applying a load at the pier base (active load cell), and (c) eliminating end-bearing load (void). The preload cell used for the cycled loading test on this project (1) consisted of three oil-filled FREYSSI flatjacks positioned in series (Figures 4 and 5). All three flatjacks were partly expanded to about 6 mm and calibrated before being placed in the test pier shaft.

A brief explanation of the versatility of the preload cell may be provided by a description of how the cell could function during different loading cycles. During loading cycle A (combined shaft resistance and end-bearing resistance), all three flatjacks could be used as passive load cells. Each flatjack would be connected to a pressure gauge (or transducer) in a closed hydraulic system to monitor the load transferred to end bearing at the bottom of the pier. For loading cycle B (end-bearing resistance only), one flatjack, an active load cell, would be connected to a hydraulic pump. This flatjack could then be expanded by pumping oil into it so that a load would be applied to the pier base. The other two flatjacks would continue to function as passive load cells. Loading cycle C (shaft resistance only) would be carried out with the valves for all three flatjacks opened. Thus, there would be no resistance to compression of the flatjacks back to their original shape. End-bearing resistance would therefore be essentially eliminated. Details of a recommended method for field testing drilled piers using a multiple-loading method are presented elsewhere (4).

Including a spare flatjack in the load cells proved to be a worthwhile precaution because one of the flatjacks in the preload cell began to leak during the initial portion of a test. The faulty flatjack was isolated from the system by opening the valve to the atmosphere, while pumping oil into the other two flatjacks until the faulty flatjack was completely compressed. The load test was then restarted using the two remaining flatjacks in the base load cell.

Calibration

The flatjacks for the top load, base load, and preload cells were calibrated using the 5.3-MN Baldwin Testing Machine at the University of Toronto. The calibration of the top load cell (8.25-MN capacity) was also verified on the 17.8-MN capacity testing machine at the Department of Mines and Resources, Elliot Lake Laboratory for Mining Research, Canadian Centre for Mineral and Energy Technology.

Representative calibration curves for the various load cells are shown in Figures 6-8. The gauge pressure-versus-load calibration curves for flatjack 6 of the top load cell and the top flatjack of the base load cell (cell 2) are shown on Figures 6 and 7, respectively. These curves were determined using linear regression analyses. The curve for flatjack 6 of the top load cell (Figure 6) is based on 190 data points (10 loading and unloading cycles) and had a coefficient of correlation, $r^2 = 0.9997$. The curve for the top flatjack of base load cell 2 (Figure 7) is based on 144 data points (8 loading and unloading cycles) having $r^2 = 1.0000$.

The load-versus-displacement calibration curve for base load cell 2 is shown in Figure 8. This curve was determined using the continuous plotting device on the testing machine. Eight cycles of loading and unloading were used and the curve of best fit (straight line) for each cycle was almost identical. The maximum deviation of any data "point" from the average curve was about 6 percent.

Reliability

The flatjack load cells performed well and, on the basis of the calibration testing and load testing results, the loads measured during the field testing were presumed to be accurate to about ±2 percent.

An indirect evaluation of the reliability of the base load cell measurements may be made by comparing the load-displacement curves for two test piers, P1 and P2, that were both constructed using conventional auger techniques. P1 had a void at its base (shaft resistance only) and P2 had a load cell at its base (combined shaft and end-bearing resistance). A comparison of the shaft resistance-versus-displacement curves for the two piers indicates almost identical behavior in terms of shaft resistance (Figure 9). Shaft resistance for P1 was measured

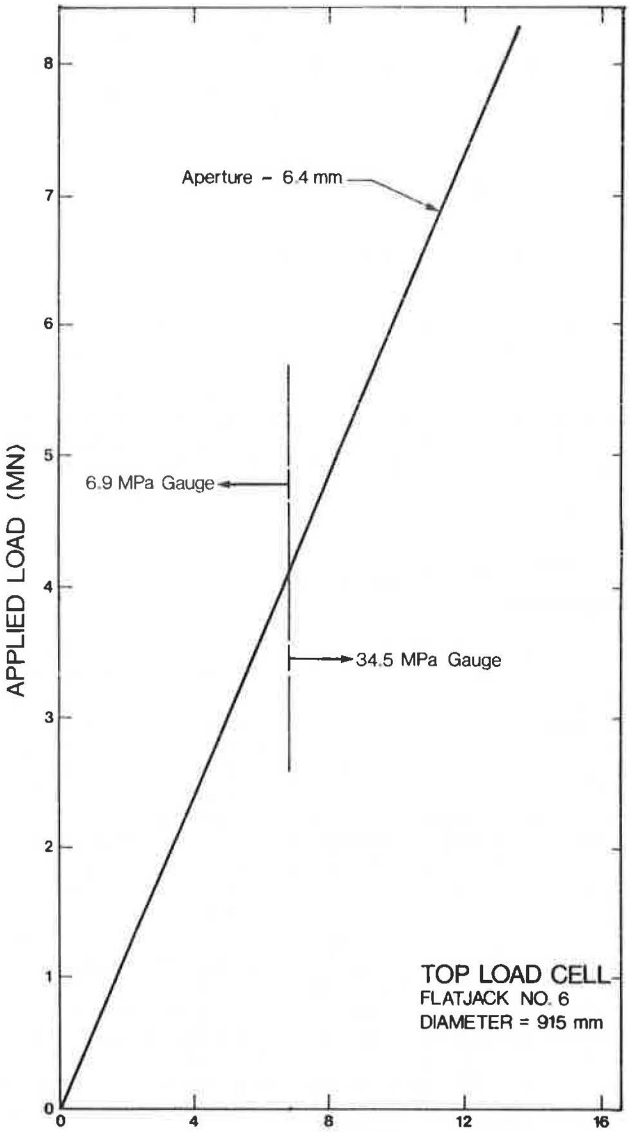

FIGURE 6 Calibration curve, applied load cell, flatjack 6.

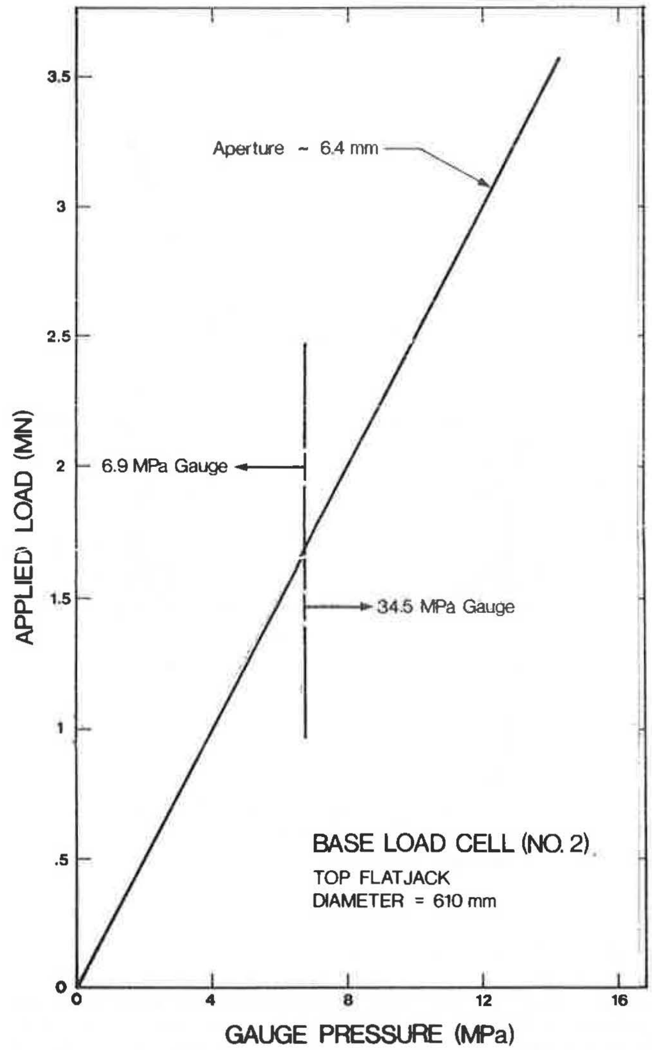

FIGURE 7 Calibration curves, base load cell 2, top flatjack.

directly, whereas shaft resistance for P2 was calculated using

$$Q_s = Q_t - Q_b \tag{1}$$

where

Q_s = shaft resistance load,
Q_t = applied load, and
Q_b = end-bearing load (measured using load cell).

This comparison suggests that the end-bearing loads measured by the base load cell are reliable. The reliability of the base load cells was also supported by a comparison of the results with behavior predicted on the basis of elastic analyses. A summary of a comparison of measured values of base load and predicted values using elastic solutions (5) is given in Table 3. The agreement between measured and predicted values is extremely good.

Only one flatjack, out of the seven used for the base load cells, failed to operate. This failure was presumed to have been caused by a leak in a pressure-tubing connection, which may have been damaged during installation.

DISPLACEMENT MEASUREMENTS

Vertical and horizontal displacement measurements for the piers were made using dial indicator gauges (0.025 mm per division). The measurements were referenced to timber beams supported on steel rods driven into the soft rock. A survey level and steel scales (1-mm divisions) were used to cross check vertical displacement at the top of the test piers, to measure vertical uplift of the anchor piers, and to verify that the reference beam supports did not move.

Pier Displacements

Vertical displacements were measured at the top of the test pier at two locations 180 degrees apart (Figure 1), at middepth, and at the bottom of the test section at two locations each 90 degrees apart,

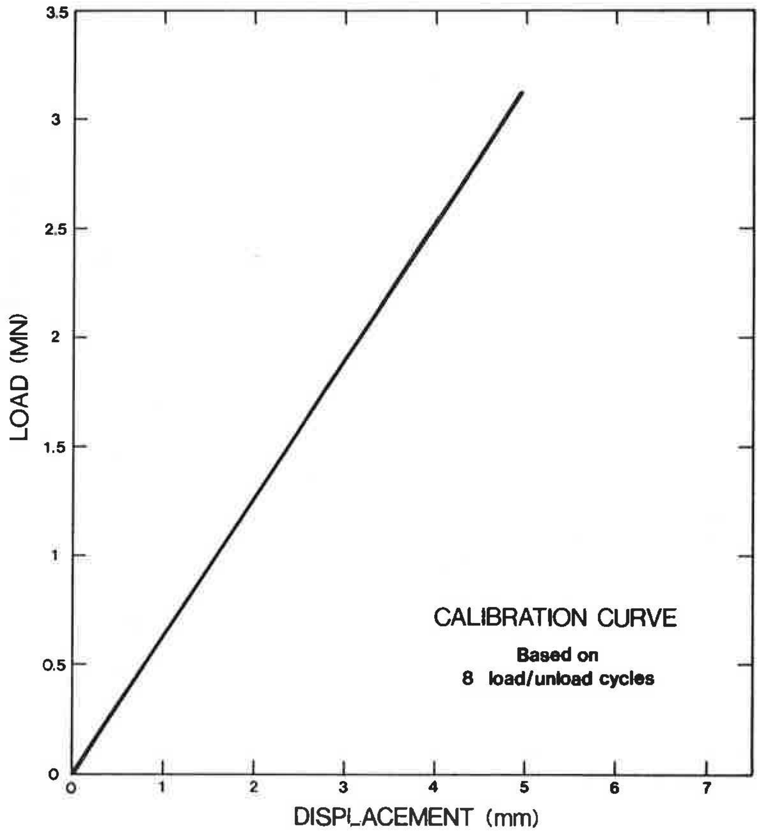

FIGURE 8 Calibration curves, base load cell 2—load versus displacement.

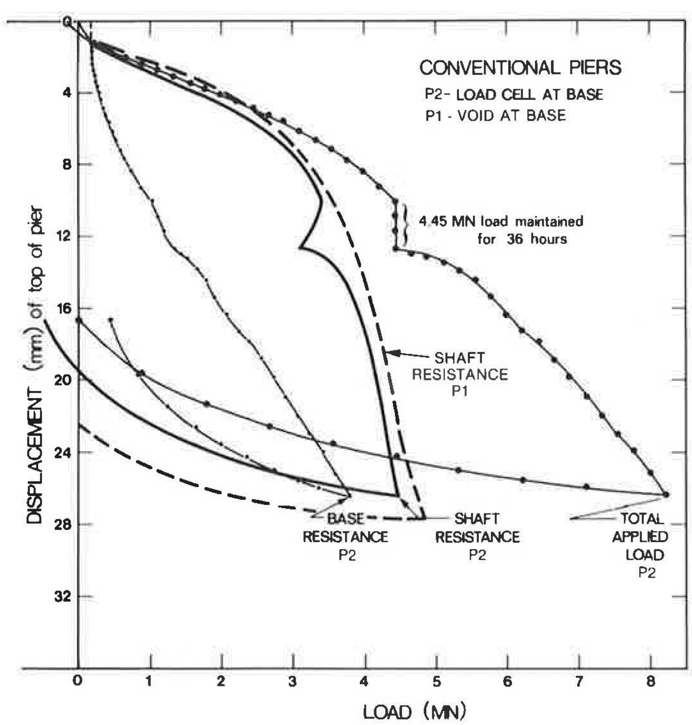

FIGURE 9 Comparison of load-displacement behavior for test piers P1 (shaft resistance only) and P2 (shaft resistance and end bearing).

TABLE 3 Comparison of Measured and Predicted Values of Base Load (Q_b/Q_t)

	TEST PIER		
	P2	P4	P5
Applied Load, Q_E (MN)	2.2	2.4	*2.1
Q_b/Q_t (measured)	.15	.20	*.20
Q_b/Q_t (predicted by elastic analyses)	.20	.20	*.20

* average value based on several test cycles

using a system of telltale rods (Figure 1). The telltale system consisted of a threaded rod inside a copper tube sleeve. A large washer was fastened to the base of the rod for embedment in the concrete. Tape was used as a spacer between the washer and the tubing to allow vertical movement of the rod and to seal the bottom of the tube to prevent concrete from seeping in. Dial gauges referenced to the top of the test pier were used to measure these telltale displacements. The telltale systems were only intended to measure displacements and did not provide the precision or accuracy necessary to determine strains within the pier. Typical load-displacement curves for a test pier are shown in Figure 10.

Horizontal displacement was measured at the top of the test pier at one location.

Rock Displacements

Vertical displacements within the rock mass adjacent to the test piers were measured at approximate depths of 0.9 m and 1.8 m at two locations using telltale rods (Figure 1). The telltale system consisted of threaded rods grouted at the bottom of a 50-mm-diameter percussion-drilled hole. The holes were located at distances of about 300 mm and 600 mm outside the pier-rock interface. Displacements were measured using dial gauges attached to the wooden reference beams. Typical load-displacement curves for the rock adjacent to the test pier are shown in Figure 11.

STRAIN MEASUREMENTS

Description

Geonor P-250 embedment vibrating wire strain gauges were installed in all of the test piers (except P6) at the midpoint of the test section.

Three gauges were used in each pier: a single gauge located on the axis and oriented to measure axial strain and two gauges (90 degrees apart) located near the perimeter and oriented to measure radial strain (Figure 1). The three gauges were fastened to a frame, made from copper tubing, and were placed in position after the concrete had been placed up to the midpoint of the test section.

The strain gauge measurements were used to estimate axial and radial stresses acting at the midpoint of the pier test sections during load testing so that load or stress distribution along the socket length could be determined. The axial and radial stresses may be calculated from strain measurements using the following linear elastic equations for an axisymmetric pier (6):

$$\sigma_z = \{[E_c(1 - v_c)]/[(1 - 2v_c)(1 + v_c)]\}\{\varepsilon_z + [v_c/(1 - v_c)]\varepsilon_r + [v_c/(1 - v_c)]\varepsilon_\theta\} \quad (2)$$

$$\sigma_r = \{[E_c(1 - v_c)]/[(1 - 2v_c)(1 + v_c)]\} \times \{[v_c/(1 - v_c)]\varepsilon_z + [v_c/(1 - v_c)]\varepsilon_\theta\} \quad (3)$$

where

E_c = Young's modulus of the concrete,
v_c = Poisson's ratio of the concrete,
ε_z = the axial strain (measured),
ε_r = the radial strain (measured), and
ε_θ = the circumferential strain ($\varepsilon_\theta = \varepsilon_r$ assumed).

The pier is assumed to be isotropic and elastic and to have a uniform distribution of radial and axial strain across the pier section.

It should be noted that the use of strain measurements to estimate stress in concrete is not an

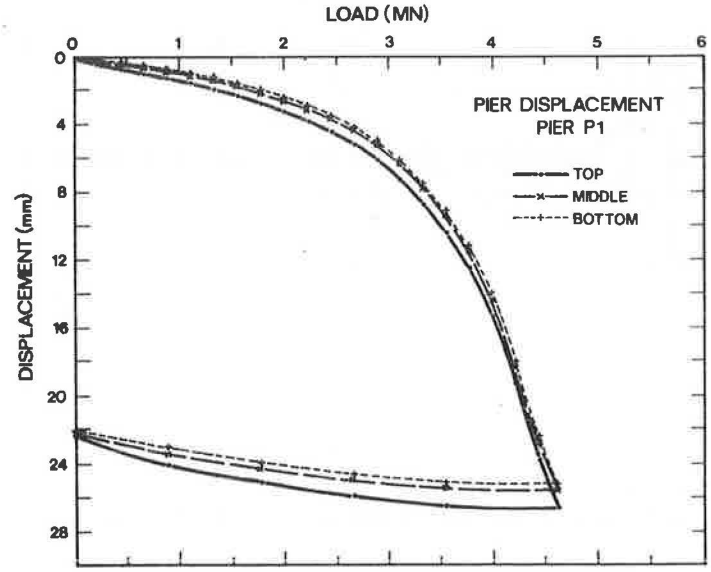

FIGURE 10 Typical load-displacement curves for test pier.

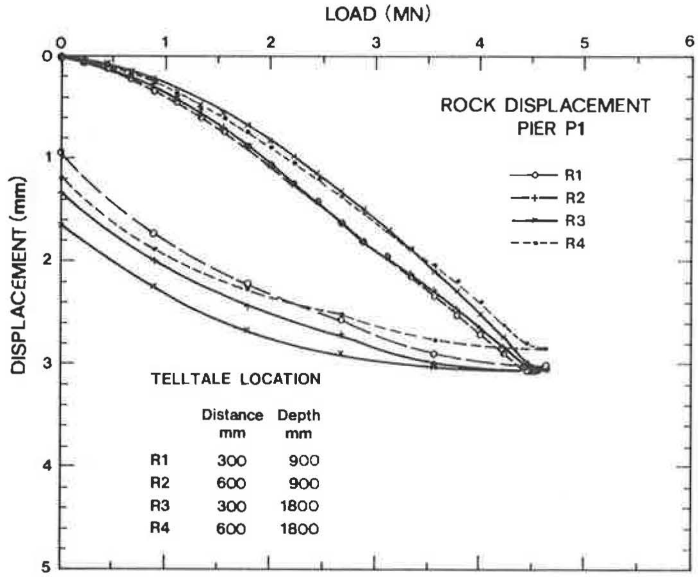

FIGURE 11 Typical load-displacement curves for rock adjacent to test piers.

ideal approach to the determination of load distribution. Knowledge of the elastic modulus of the concrete is necessary, and nonhomogeneity, stresses induced by curing, and creep behavior of the concrete can cause difficulties in the interpretation of the data.

The testing was of short duration; therefore, the influence of concrete creep on strain measurements would be negligible. Strain measurements were also obtained when the concrete was placed and at various intervals before testing (concrete curing period). All of the strain gauges indicated that concrete expansion occurred during curing.

Calibration

The strain-frequency calibration factor supplied by Geonor was verified in the laboratory. The reported accuracy of these gauges is ±2 microstrain over a range of 1250 microstrain.

Reliability

Some difficulties experienced with the electronics during load testing on piers P3 and P4 were traced to inadequate grounding of the readout instrument. After this problem was corrected, the strain measuring system functioned well. The reliability of the strain gauge measurements may be indirectly assessed by comparing the measured strains to expected values of strain for the various load support conditions.

Load tests were performed on conventional piers that had essentially zero base resistance. Thus, the applied load was supported only through shaft resistance. In these tests, the measured radial strains were negative (compression) indicating that the pier was being compressed radially inward during loading (Figure 12a). This behavior indicates the tendency for shear dilation (volume expansion) to occur at the pier-rock interface or within the rock, or both. Also, the strain data were consistent in that both axial and radial strains decreased (compression) with increasing applied load. Load tests were also carried out on piers with combined shaft resistance and base resistance components. In all of these tests, the measured radial strains were positive, indicating that the pier was expanding in a radial direction (Figure 12b). This behavior is comparable to that which occurs during a compression test on a concrete cylinder and indicates Poisson's effect. These strain data were consistent in that axial strain decreased (compression) and radial strain increased (expansion) with increasing applied load. Thus, all of the measured strain data correctly reflected the anticipated strain behavior for the load support conditions tested.

A summary of a comparison of values of axial strain measured using the strain gauges with values measured using the telltale systems and with values estimated using a simplified elastic analysis is given in Table 4.

The reliability of the data obtained from the vibrating wire strain gauges may also be assessed by examining the load distribution in the test piers. This is discussed in the next section. In the simplest case, test pier P1 (shaft resistance only), the determined load distribution was exactly as anticipated. Thus the strain gauge performance in this case can be judged to be extremely good. In the other test piers, the conditions were more complicated: combined end-bearing and shaft resistance, grooved shafts, preloading of base, and cycled loading were involved. Thus the reliability of the strain gauge measurements in these cases could not be evaluated.

LOAD DISTRIBUTION IN TEST PIERS

Load distribution in test piers P1 through P5 was determined from measurements of applied load at the top of each pier, axial and radial strain at the middepth of the test sections, and end-bearing load (or known boundary condition) at the base of each pier. Thus, the load distribution curves determined for all the piers were based on data from the same three locations. The top and bottom loads were measured with load cells except in the case of piers that had voids at their bases for which zero end-bearing load was assumed. The load (or stress) at pier middepth was determined indirectly using vibrating wire strain gauges embedded in the concrete pier.

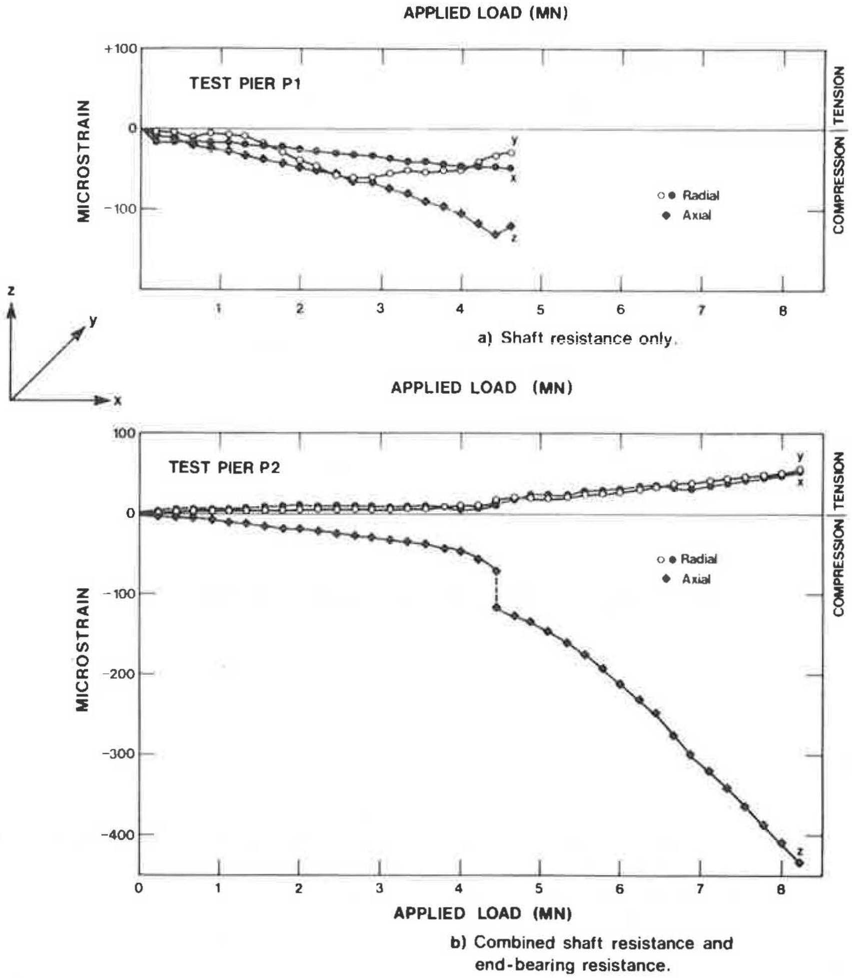

FIGURE 12 Measured values of axial and radial strain.

TABLE 4 Summary of Estimated and Measured Values of Axial Strain

	PIER DESCRIPTION	STRAIN IN PIER @ Q_a = 2 MN (x 10^{-6}) CALCULATED Upper Limit	CALCULATED Best Estimate	MEASURED Strain Gauge	MEASURED Telltales*
P1	Conventional - void at base	41	21	46	110
P2	Conventional - end bearing	30	17	18	0
P3	Grooved - void at base	41	20	89**	37
P4	Grooved - end bearing	31	18	10**	110
P5A	Conventional - end bearing preload	30	23	29	130
P5B	Conventional - end bearing preload	30	28	42	70
P5C	Conventional - end bearing preload	30	30	48	110
P5D	Conventional - void at base	30	15	28	37

* Strain based on telltale measurements were determined using difference between middle and bottom telltale and only provide a crude approximation of strain in the test pier. (Accuracy of dial gauge = .001 in. is equivalent to a strain of 37 x 10^{-6})

** Readings may be erroneous due to difficulties with readout box.

The load distribution curves for conventional socketed piers, P1 and P2, at various magnitudes of applied load are shown in Figure 13. The distribution curves for test pier P1 (void at base) indicate that shaft resistance was distributed uniformly over the length of the socket for all values of applied load (Figure 13a). This distribution behavior is consistent with the load distribution predicted from analytical studies based on elastic theory (7). The distribution curves for pier P2 (load cell at base) are distinctly different from those of P1. In the elastic loading range $Q_T < 2.2$ MN, little shaft resistance was mobilized in the lower half of pier P2 (Figure 13b).

This behavior is not consistent with analytical solutions that predict uniform distribution of the load (constant slope) over the socket length (7). The reasons for this discrepancy are unclear. However, the similarity in the shape of the distribution curves for other similar piers suggests that the initial small preload applied to the base load cells (to ensure seating) may be the cause of this inconsistent behavior.

During the maintained load increment for pier P2 (applied load = 4.45 MN), the slopes of the upper and lower portions of the distribution curves began to equalize and load distribution or shaft resistance along the socket became essentially uniform at an applied load of 6 MN (Figure 13b).

CONCLUSIONS

The instrumentation used in a field testing program to investigate methods of improving the performance of rock socketed piers has been described and discussed. The results obtained from the test program demonstrate the reliability of the instruments used.

The load cells consisting of a series of FREYSSI flatjacks performed well. The load cells were reliable and the results obtained using these cells were estimated to be accurate to within ±2 percent.

In the case of test pier P1, the Geonor P-250 embedment vibrating wire strain gauges were judged to be reliable and satisfactory. All of the strain measurements for the test piers were consistent with anticipated behavior. When questionable data were obtained, the cause was improper grounding of the readout equipment and not a fault of the instruments themselves.

Loads (stresses) calculated using the strain gauge data enabled determination of the load distribution within the pier-socket system.

Displacement at the top, middepth, and bottom of the test piers and in the rock immediately adjacent to the piers was reliably measured using a combination of dial indicator gauges and telltale systems.

The versatility of a flatjack load cell to perform three different functions has been described.

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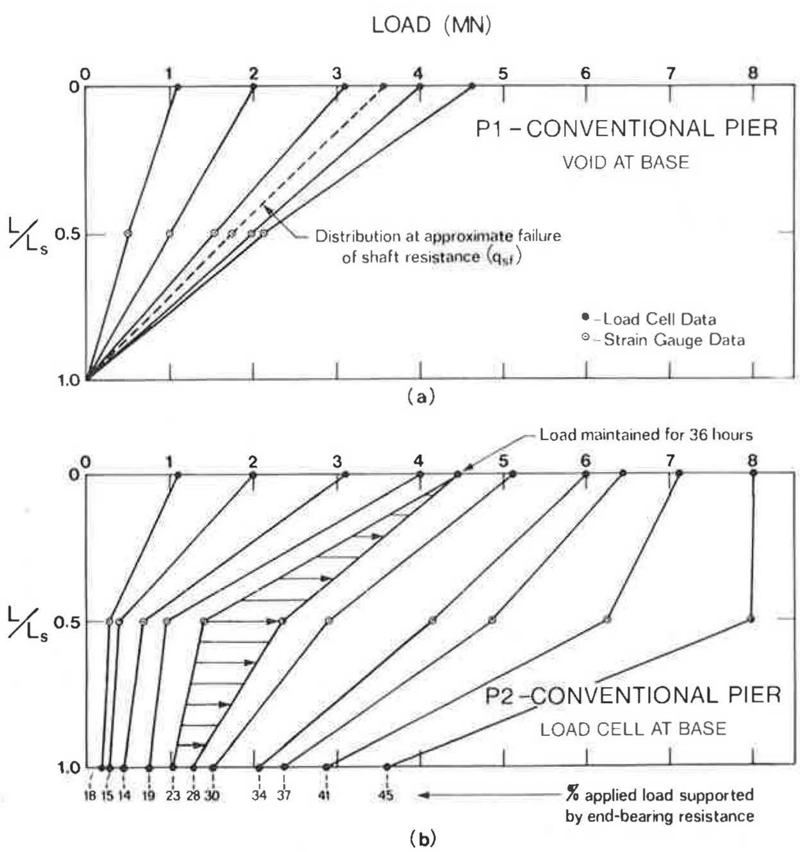

FIGURE 13 Typical load distribution curves for conventional piers.

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Closing Remarks on Reliability of Geotechnical Instrumentation

JOHN DUNNICLIFF

ABSTRACT

In the closing remarks delivered at the Symposium on Reliability of Geotechnical Instrumentation, three subjects are discussed: a "recipe" for reliability, the parameters that can be measured most readily, and a plea to users of instrumentation.

These closing remarks will address three topics: First, a recipe for reliability. Second, which parameters can be measured most reliably? Third, a plea to users of instrumentation.

A RECIPE FOR RELIABILITY

When this symposium was being planned, I wrote a recipe for reliability. Having now read the six papers that have been presented, I have made a few changes and will define what I believe are the major ingredients. There are two types: instrument ingredients (three of these) and people ingredients (five of these).

Instrument Ingredients

Simplicity

Follow the KISS (keep it simple, stupid) principle. For example, mechanical and hydraulic devices are generally more reliable than electrical devices.

Self-Verification

This term means that instrument readings can be verified in place. For example:

- Telltales on a rod extensometer with a method of disconnecting the rod from the anchor, so that a check can be made for free sliding;
- Duplicate transducers (e.g., a vibrating wire and a pneumatic transducer packaged within the same housing to create a piezometer with two independent methods of reading); and
- Checking remote-reading borehole extensometers with a dial gauge at the head.

Durability in the Installed Environment

The transducer must have proven longevity to suit the application. Cables, tubes, or pipes that connect the transducer to its readout must be able to survive imposed pressure changes, deformation, water, sunlight, and chemical effects such as corrosion and electrolytic breakdown.

People Ingredients

Thorough Planning

Details of planning requirements are given by Dunnicliff (1). The ingredients include

- McGuffey's "System Design," (see paper by McGuffey in this Record) including development of the best predictive model.
- Use of the best contract practices. Abramson and Green (see their paper in this Record) say, "Many owners effectively encourage low survivability rates by using low-bid procurement procedures." That is a succinct statement about a very large and serious problem.
- Rawnsley, Russell, and Hansmire (see their paper in this Record) address backup and redundancy, in their discussion of Harvard Square Station: "Redundancy existed, with key parameters being measured by more than one instrument." Hannon and Jackura (see their paper in this Record) say in their summary, "When feasible, alternative procedures should be used for backup to estimate soil stress conditions."
- Comprehensive factory calibration and quality assurance are important. Note that this is a people ingredient not an instrument ingredient. Hannon and Jackura say, "All instruments should also be subject to bench or calibration testing, or both, to ensure performance and specification compliance."

Installation Care

Planning for installation usually includes gaining the cooperation of the construction contractor. Without this, reliability is hard to achieve.

Regular Maintenance and Calibration

The need for regular maintenance and calibration is well demonstrated in the paper in this Record by Bordes and Debreuille. For example, portable readout units should be calibrated frequently.

Care During Data Collection

For example, in the field the person reading an instrument should always study changes with respect to the previous reading. Substantial changes may indicate a reading error or the need for rapid remedial action.

Care During Data Processing and Interpretation

This ingredient includes McGuffey's "Engineering Interpretation Methodology."

Summary

In summary, experience and knowledge are vital to the people ingredients. Rawnsley, Russell, and Hansmire say:

> The key to the reliability of the instrumentation program was the people involved. Instrumentation installation was done by experienced professionals. Instrument monitoring was performed by trained people who were on the job for extensive periods of time, were interested in the results, and were responsible for interpreting the measurements.

In their summary Hannon and Jackura say,

> Instrumentation personnel should be experienced and knowledgeable about potential problems associated with the placement and monitoring of the particular instruments selected for use.

McGuffey had operator knowledge as one of his five major items contributing to reliability. It is therefore agreed that experience and knowledge are vital. But perhaps even overriding these is motivation. Discussing responsibility for instrumentation, Baker (2) said:

> Who has the motivation? Who cares about the data? The person with the greatest vested interest in the data should have direct line responsibility for producing it accurately.

Conclusion to a Recipe for Reliability

In my view, unreliability can more often be attributed to the people ingredients than to the instrument ingredients. The message is clear: We, the users, need to make a strong effort to improve the state of the practice.

WHICH PARAMETERS CAN BE MEASURED MOST RELIABLY?

The four parameters, pore pressure, total stress, deformation, and load and stress in structural elements, need to be rated. They will be rated here in order of increasing reliability.

Total Stress in Soil

The difficulties are well illustrated by Hannon and Jackura. They divide them into two groups, first the ability of the cell to measure the stress around it (cell design) and, second, underregistration because the cell is in a soft cocoon of backfill (cell placement). I believe the larger problem is the second one. S.D. Wilson (personal communication, 1984), on the basis of his extensive experience measuring total stress within embankments dams, states:

> When earth pressure cells are installed in a horizontal plane in compacted fills for embankment dams, the cells typically register only 50 to 70 percent of the added vertical stress as embankment construction continues.

There is a need to develop a method of hand compaction around the cells that prestresses the soil to match the prestress in the remainder of the fill without damaging the cells. This is, of course, extremely difficult to do. The Comision Federal de Electricidad at experimental laboratories in Mexico City has constructed a large laboratory facility to test the response of embedment earth pressure cells to applied loads. It is hoped that improved installation techniques will result from tests now in progress.

Finally, I rate total stress as the least reliable parameter because of one other fundamental factor: Measurements are point measurements in a heterogeneous environment, and therefore a small number of measurements may not be representative of overall conditions.

Pore Pressure

The instruments are satisfactory. Installation problems are difficult, but they can be solved. The main problem, well illustrated by McGuffey, is the same as the one mentioned last for total stress: Measurements are point measurements in a heterogeneous environment. The problem is not as severe with pore pressure measurements, but is still significant.

I want to say a few words about the paper by Bordes and Debreuille, in this Record, because their conclusions apply to pore pressure measurements. A most believable and impressive case for vibrating wire instruments is presented in their paper. I have been looking for such a paper for more than 10 years and welcome this clear and convincing information. However, I am going to disagree with their Parisian graciousness. They say:

> Although the instruments discussed in this paper come from the same manufacturer, the conclusions drawn therefrom have a much wider scope. They apply to all vibrating wire instruments, provided of course that construction is of a high standard.

I will mention briefly two experiences with vibrating wire instruments from another "leading manufacturer," from whom "construction of a high standard" might be expected.

1. During first filling of the reservoir behind an embankment dam, a vibrating wire piezometer indicated a piezometric level that caused concern. Filling was stopped. The piezometer reading continued to rise. When the indicated piezometric level rose above pool level, everybody discounted the measurements and filling continued.
2. Vibrating wire pressure transducers have recently been used to measure oil level in oil tankers. Many have been unreliable, and several hundred have been returned to the manufacturer.

I truly believe that the conclusions drawn by Bordes and Debreuille do not necessarily apply to all vibrating wire instruments. How do users know whether all the details discussed by the writers are handled with similar care by all manufacturers? As one example, is the aging issue raised by Bordes and Debreuille handled adequately by other manufacturers?

Load and Stress in Structural Elements

I rate this parameter more reliable than pore pressure because it is measured on or in a material made and controlled by people. Discussion of this parameter is subdivided into use of three types of instrument: load cells, strain gauges on elastic elements, and strain gauges in or on concrete.

1. Load cells serve extremely well. Abramson and Green indicate the need for good bearing plates and taking care of eccentricity. Rawnsley, Russell, and Hansmire talk about problems with using hydraulic jacks for load measurement and confirm what many others have found:
   - Up to 20 percent overregistration during loading and
   - Up to 5 percent underregistration during unloading.

   The problem is caused by friction between the piston and the cylinder, and hydraulic jacks should not be relied on for load measurement.
2. Strain gauges on elastic elements also serve well. Measurements on structural steel, for which a reliable conversion from strain to stress can be made, have a long and successful history.
3. Strain gauges on or in concrete cause problems in converting strain to stress, and the problem is aggravated if measurements are other than extremely short term. Horvath and Abramson and Green discuss this problem, and Abramson and Green recommend three methods of dealing with it:
   - Controlled laboratory tests;
   - Dummy no-load gauges; and
   - Measuring the load directly, where possible (e.g., across a concrete pile).

My experience has been that

- Controlled laboratory tests rarely model field conditions adequately and
- Dummy no-load gauges are of little use because they do not account for strain caused by creep under load. This leaves three options:
- Measure load directly, as suggested by Abramson and Green;
- Where possible, use concrete stress meters instead of strain gauges, taking great care to ensure intimate contact between the instrument and the concrete, either by following the installation methods recommended for the Carlson stress meter or by using a poststressing tube as provided in the Gloetzl stress meter; and
- Create, as part of the structure in the field, an "unconfined compression test specimen," under known load, and measure strain with strain gauges or multiple telltales in this part of the structure, to determine modulus. This can be done at the top of piles and drilled piers during test loading, sleeving if necessary below the ground surface to create the "specimen."

Deformation

Deformation can be measured with greatest confidence. Instruments can often be simple. A single instrument can provide data for a large and representative zone. If you can answer your geotechnical question with deformation measurements, please do so.

The extensive topic of deformation measurements has not been covered in this symposium, and at first this seems to be a shortcoming. However, I do not think its inclusion would alter my view that the main impediment to reliability is the people ingredient of inadequate experience, knowledge, and motivation.

MY PLEA TO THE USERS OF INSTRUMENTATION

Hansmire says, in his introduction to this Symposium, "The negative experiences, the failures, are not often reported." Abramson and Green say, when talking of the failure of instrumentation schemes: "The distilled experience of many engineers who have suffered the consequences of instrument or program failures should help reduce the incidence of future occurrences."

It is clear that we learn from our mistakes and the mistakes of others. I have described about 20 mistakes, of which I have been guilty or with which I have been associated, in a series of three articles to be published in Geotechnical News (published quarterly by Bitech Publishers, Ltd., 801-1030 West

Georgia Street, Vancouver, British Columbia, Canada, V6E 2Y3, and distributed to registered members of the Canadian Geotechnical Society and the United States National Society of ISSMFE; others may subscribe by contacting the publisher). My purpose in writing these articles, entitled "Lessons Learned from Imperfect Field Monitoring Programs," was to help others to avoid making the same mistakes. In each case the mistake and the lesson learned are stated. This is planned as an ongoing section in Geotechnical News. The ball will soon be rolling; please keep it rolling by contributing lessons learned from your mistakes so that I may avoid adding them to my already long list.

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