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# SOIL INSTRUMENTATION FOR THE I-95 MIT-MDPW TEST EMBANKMENT

By

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## Abstract

This paper describes an instrumentation scheme adequate to document the stress-deformation history of a heavily loaded, soft foundation. The instrumentation includes hydraulic piezometers, vibrating wire piezometers, settlement plates and anchors, inclinometers, and vibrating wire total stress cells. The instrumentation is described, installation procedures outlined, and instrument performance evaluated.

## I. Introduction

### *A. History of Research Project*

In June 1965, the Soil Mechanics Division of the Department of Civil Engineering, M.I.T., began an in-depth research study for the Massachusetts Department of Public Works on stability and deformation of heavy embankments on thick deposits of soft soils. The primary objective for this research was to obtain critically needed fundamental information on the reliability of techniques of predicting stability and deformation of such embankments. The strategy for accomplishing this objective was to instrument an actual embankment, predict its behavior, document the measured behavior, compare reality with prediction, and then evaluate methods of prediction. This paper is limited to the description and evaluation of the instrumentation.

The first two and one-half years of the research work involved preconstruction activities. First, a field test site was chosen, being Station 246 of an embankment for Interstate 95 to be built across a soft, compressible tidal meadow in Saugus, Massachusetts. The construction plan for the embankment

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envisioned these operations: excavation of thin surface peat, placement of a granular backfill below water, compaction of granular embankment to about Elev. +18 ft, placement of a temporary overload to about Elev. +40 ft, removal of the overload after a suitable period of time, and then construction of the pavement.

Following selection of the test site, undisturbed samples of high quality were taken from a deep boring at the test site. Laboratory tests were performed to support preliminary predictions of foundation performance. Concurrently, the researchers developed specifications for field instrumentation to be placed at the test site. The instrumentation design provided for the following:

1. Observation wells and piezometers to measure the pore water pressures under and adjacent to the embankment;
2. Settlement rods and platforms to measure vertical deformations;
3. Inclinometers to measure lateral and vertical deformations;
4. Total stress cells to measure vertical embankment loading.

All of these instruments were placed after the peat had been removed and granular backfill placed to Elev. +9 ft. To establish initial conditions in the foundation soils, a few piezometers and settlement rods were installed at nearby Station 245 before any construction activities.

Early construction work began in December 1967 at the test section. Readings were taken at Station 245 instrumentation before and during the installation of the Station 246 instrumentation (February to July 1968). All Station 246 instrumentation was in operation by early July 1968. Readings of the Station 245 instruments were discontinued after a few weeks overlap with the main instruments. Readings were taken daily on most instruments during the embankment construction period. The embankment was completed to Elev. +36 ft in November 1968. Filling then stopped and was resumed in April 1969 to Elev. +40 ft by May 1969.

### B. *Objectives of this Paper*

This paper has two primary objectives:

1. Description of a field instrumentation scheme adequate to document the stress-deformation history of a heavily loaded, soft foundation;
2. Evaluation of the performance of the various instruments used.

There are several unusual features of the instrumentation that need evaluation, such as the instrumentation tunnel, the total stress cells buried in the embankment, the piezometers off center line and beyond the toe of the embankment, and the use of electric vibrating wire instruments. Actual field data are portrayed to illustrate the performance of the instruments. A separate publication will use the field data to evaluate methods of design and analysis.

## II. Field Instrumentation Scheme

### *A. Soil Conditions*

The site for the I-95 embankment crosses a tidal meadow consisting of a thick deposit of soft grey clay known locally as the Boston blue clay. It is a silty clay of medium plasticity with numerous sand and silt lenses. The plan location of the project is shown in Fig. 1; the soil profile in Fig. 2. At the M.I.T. test section the peat is about 6 ft thick, the sand 10 ft thick, and the clay 130 ft thick. The clay is overconsolidated at the top due to sea level lowering and becomes normally consolidated at about Elev. -80 ft. The undrained strength near the top is about 1.0 tsf, ranging to a low of about 0.65 tsf near the middle. The average compression index for the clay profile is about 0.2. Under the heavy embankment, stability of the foundation was expected to be a serious problem, and settlements on the order of several feet were predicted.

### *B. Instrumentation Plan*

Figure 3 shows the instrumentation as installed at the Preconstruction Test Section (Station 245), and Fig. 4 at the M.I.T. test section (Station 246). Detailed information on the installation of individual instruments were documented in a recent report (MIT 1969). Table 1 summarizes all the instrumentation.

Several features of the M.I.T. test section stand relatively unique:

1. Instrumentation Tunnel

Rather than carrying the instrument leads to the surface of the embankment concurrently with construction progress, the instrumentation located under the embankment terminates in a 5-ft diameter, 110-ft long flexible steel plate tunnel in order to minimize interference with construction activities and damage to instruments. The tunnel was placed initially at a slope (i.e., Elev. +8 ft at the center line and Elev. +6 ft at the toe) in order to offset expected differential settlement. The leads of piezometers and stress cells were brought into the tunnel through flexible protective hosing. The tunnel sits along a line where all the settlement platforms, settlement rods, inclinometers, observation wells and the bench mark were installed. All instruments can move independent of the tunnel. The tunnel is illustrated in Fig. 5. In Fig 5a the left end of the tunnel rests at about the center line of the embankment. Figures 5b and 5c show the details of connecting the piezometers to the tunnel.

2. Total Stress Cells

Three clusters of stress cells, each cluster consisting of three cells, were installed at Station 246 +33 at the center line, and at 30 ft and 60 ft offset from the center line. The decision for using stress cells was made after the filling had

Table I

FIELD INSTRUMENTATION FOR I-95
MIT TEST EMBANKMENT

| TYPE OF INSTRUMENT | NUMBER OF STRUMENTS | |
|---|---|---|
| | STATION 245 | STATION 246 |
| Double tube hydraulic piezometer | 6 | 33 |
| Vibrating Wire electric piezometer | 0 | 6 |
| Observation well | 0 | 5 |
| Settlement rod | 6 | 12 |
| Settlement platform | 0 | 5 |
| Inclinometer | 0 | 6 |
| Benchmark | | 1 |
| Total stress cell | 0 | 9 |
| Total drilling, ft | | |
| 6″ | 0 | 957 |
| 4″ | 0 | 1001 |
| 3″ | 677 | 3089 |
| Total cost | | |
| Instrumentation | \$4000 | \$85,000 |
| Tunnel | 0 | \$17,000 |

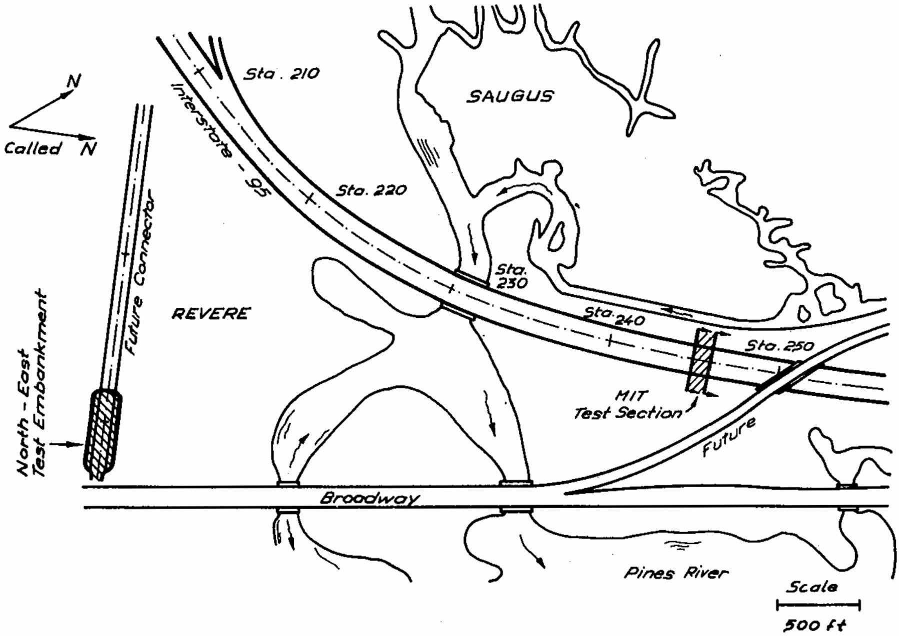



Figure 1. Location of the Test Embankments

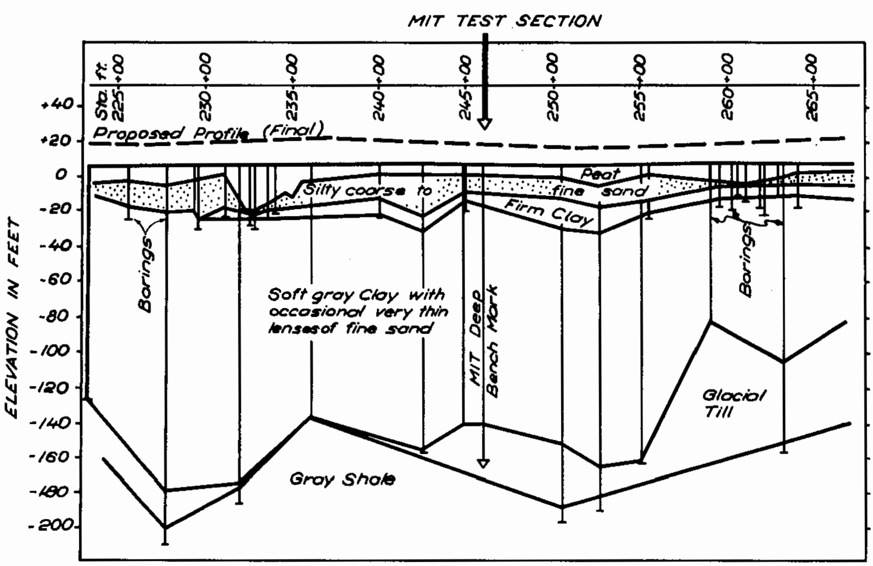



Figure 2. Soil Profile Along I-95

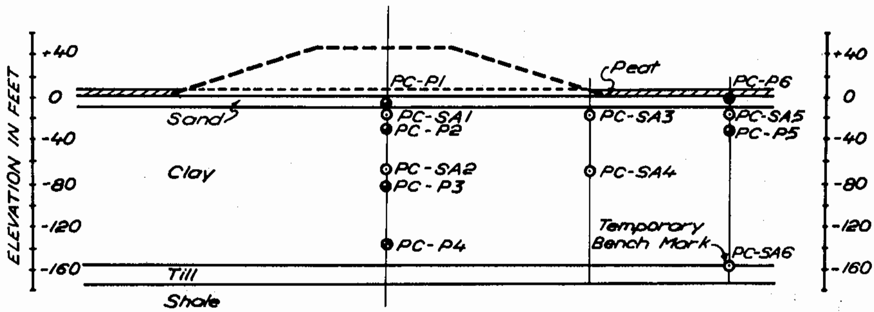



Figure 3. Preconstruction Instrumentation, Station 245

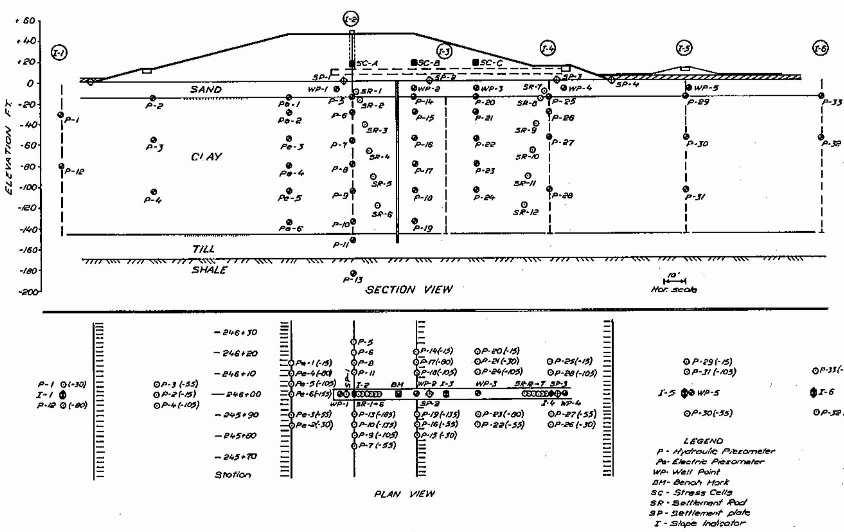
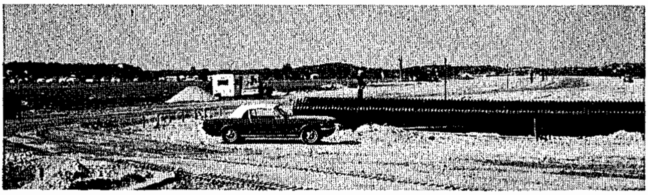
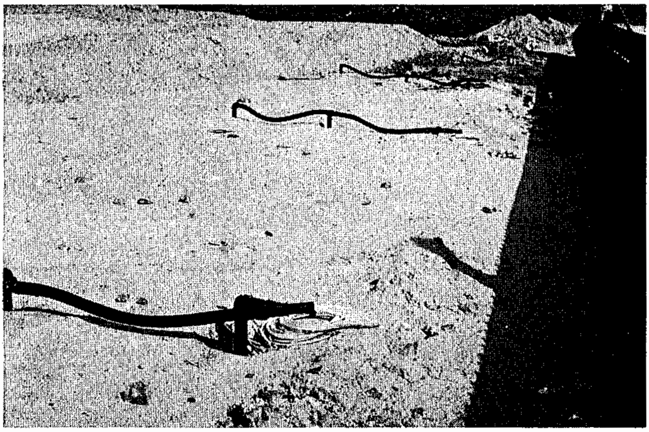



Figure 4. MIT-MDPW Test Section Instrumentation, Station 246

A


B


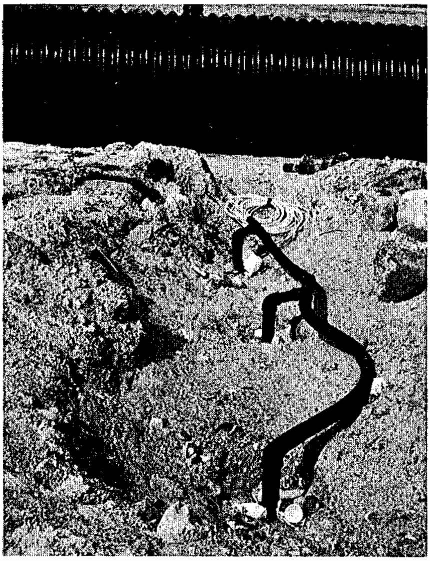
C


Figure 5. Instrumentation Tunnel

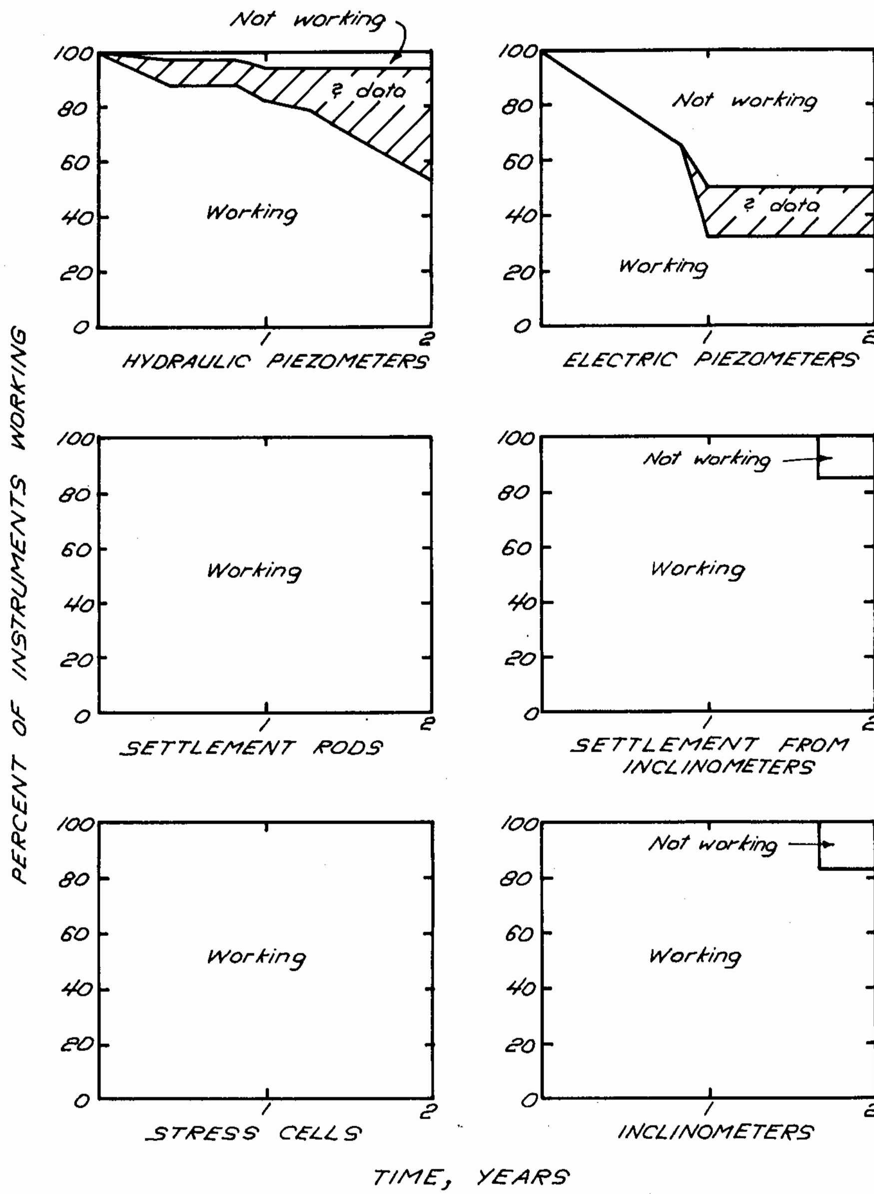



Figure 6. Two Year Performance Record of Instruments

already started, thus they were installed within the embankment, approximately at Elev. +17 ft. The stress cells measure total vertical stresses.

3. Off-center Line Piezometers

Besides a thoroughly instrumented center line profile, the test section consists of lines of piezometers at 30, 60, 95, 160, and 225 ft offset from the center line. The intention has been to observe the pore pressure response in the full lateral extent, since this information is essential both for stability and settlement analyses.

The data collected from off-center piezometers contributed significantly to the decision not to build the side berms which were included as options in the original design.

*C. Longevity of Instruments*

Figure 6 indicates the percentage of instruments working during the first two years of operation. Three categories of evaluation are shown:

1. Working
2. Working, but data questioned
3. Not working.

The stress cells and settlement rods performed without any instrument failure. Piezometers, both hydraulic and electric vibrating wire types, lost about 50 percent by the end of the two-year period. Although this loss is high, it is not unusual for similar installations where large settlements occurred.

## Details of Instruments

*A. Piezometers*

**Station 245.** Figure 3 shows the locations of the five hydraulic piezometers, designated PC-P1 through PC-P5, installed at the Preconstruction Section. The piezometers were Geonor A/S, M-206 field piezometers. The piezometer consists of a hollow metallic stem with a conical point at the bottom; three sintered bronze filters are mounted on the stem and are separated by chamfered brass rings. The top of the hollow stem was attached to a drilling rod (E-rod) connector and sections of E-rod were attached directly to the sensor. Two 3/16-inch plastic tubes were used as riser pipes. The installation procedure is described in Appendix B.

**Station 246.** Figure 4 shows the location of the thirty-three hydraulic and six electrical piezometers installed at the MIT-MDPW Test Section. The piezometers under the embankment terminate at the instrument tunnel, whereas the ones beyond the toe are grouped in four manholes.

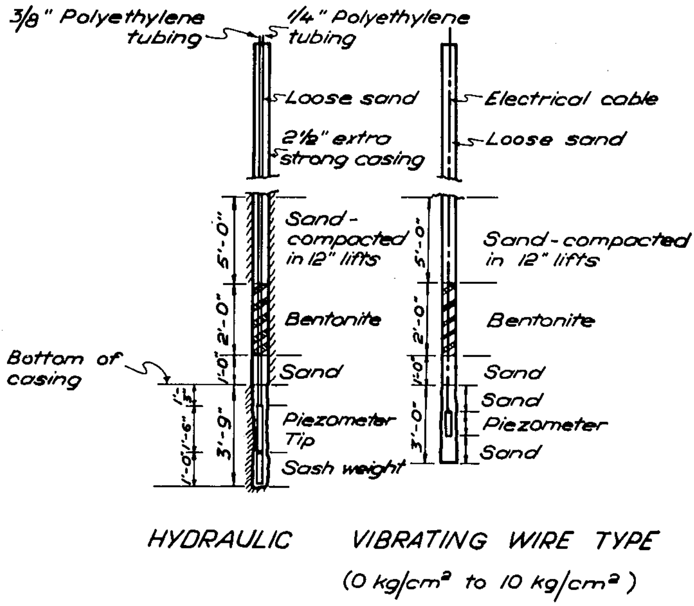



Figure 7. Details of Hydraulic and Vibrating Wire Piezometers

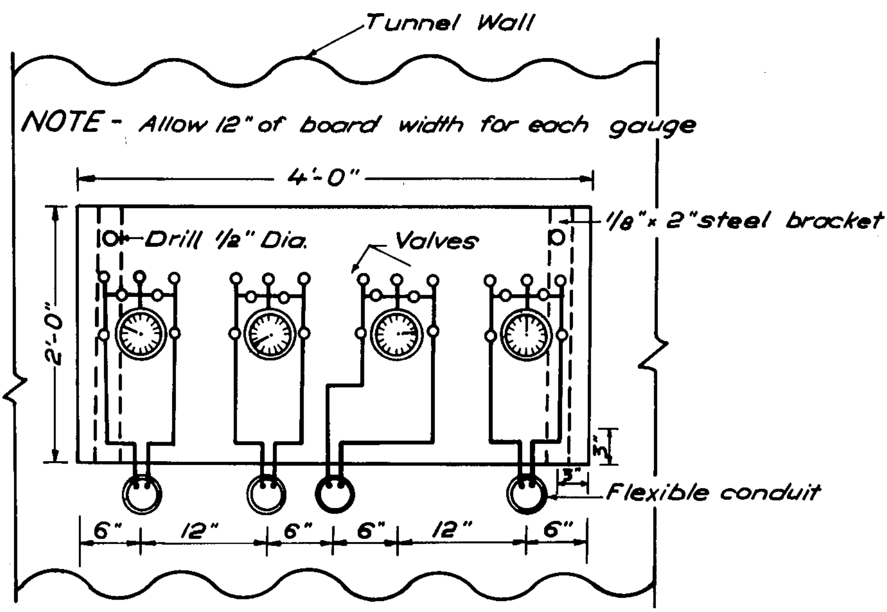



Figure 8. Gage Boards in Instrument Tunnel

The hydraulic piezometers were manufactured by Geomeasurements, Inc., and consist of an 18-in. long porous plastic ("Vycron") sensor with two plastic riser pipes, a 3/8-in. reading lead and a 1/4 in. flushing lead. A sash weight was tied to the tip to aid installation. Details of the installed peizometer are shown in Fig. 7. All leads were brought into the instrument tunnel or a manhole by attaching an elbow to the top of the steel casing and running flexible hose from the elbow to the tunnel or manhole. The leads are connected to a 4½-in. diameter weatherproof combination gauge with a range from -30 in. mercury to +60 psi. A typical arrangement of gauges is shown in Fig. 8. The installation procedure is described in Appendix B.

The electrical piezometers were Geonor A/S vibrating wire piezometers with a range of 0-5 kg/cm$^2$ or 0-10 kg/cm$^2$ depending on depth of installation. The piezometer output is the frequency of vibration of a wire stretched from a fixed support to a sensing diaphragm. The frequency is read directly by a digital frequency meter. The piezometers were calibrated in the laboratory for hydrostatic pressure and temperature. A final calibration was made during installation by recording the frequency every five feet as the instrument was lowered into the cased hole full of clear water. The installation procedure is described in Appendix B.

**Observed Data.** The piezometer data have been summarized by plotting water level elevation or total head. This is simply the elevation to which water would rise in an open standpipe whose bottom is at the same elevation and location as the corresponding piezometer. Data from shallow observation wells are also summarized and, therefore, excess pore pressures (in ft of water) may be computed as the difference of the piezometer and well readings.

Figure 9 summarizes the piezometer data at the Preconstruction Section, Station 245. Almost no data could be collected from mid-February to mid-April 1968 due to freezing weather conditions. Later experience showed an antifreeze mixture (4 parts by volume methanol, 2 parts glycerol, 3 parts water) to be effective under sustained cold spells with temperatures of -10°F to -20°F. During the second half of July 1969, the Preconstruction Section piezometers were monitored concurrently with the MIT-MDPW Test Section peizometers at Station 246. After a consistent behavior was observed between the two sections, further readings of the Preconstruction piezometers were terminated and they were buried under the rising embankment.

Figures 10 through 14 summarize the piezometer data at the MIT-MDPW Test section through June 1969. All the piezometers were in operation in mid-July 1968, with the embankment at Elev. +10 ft. Continuous construction brought the embankment fill to Elev. +36 ft on 22 November 1968. The piezometers were monitored daily (except weekends) during construction. Construction stopped until mid-April 1969; the piezometers were read twice a week during this period. Construction resumed in mid-April and the embankment was brought to its final elevation, Elev. +40 ft, by mid-May 1969. During this last stage the piezometers were monitored daily.

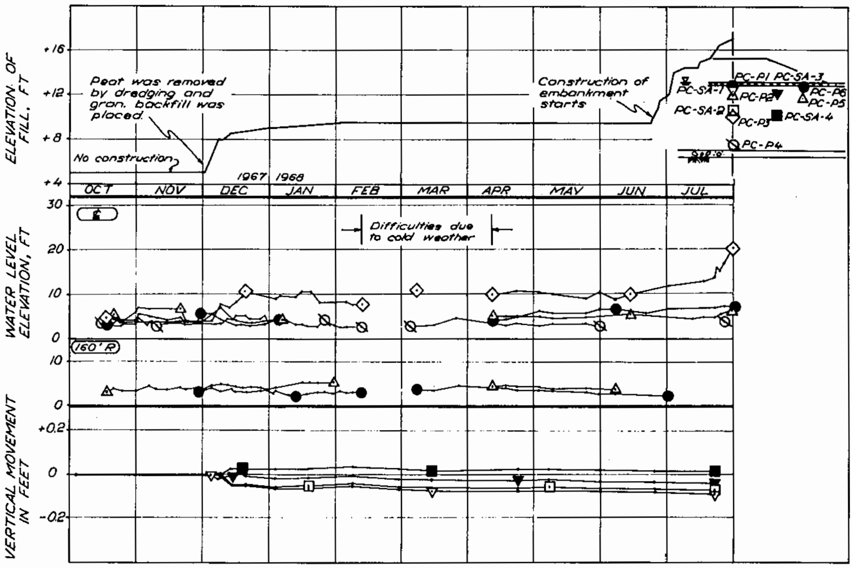



Figure 9. Preconstruction Instrumentation Data

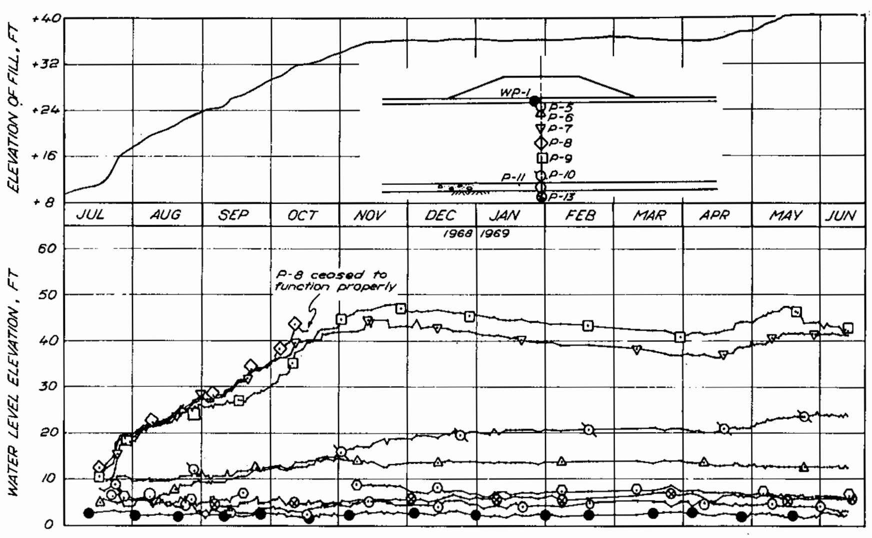



Figure 10. Pore Pressures Along the Center Line

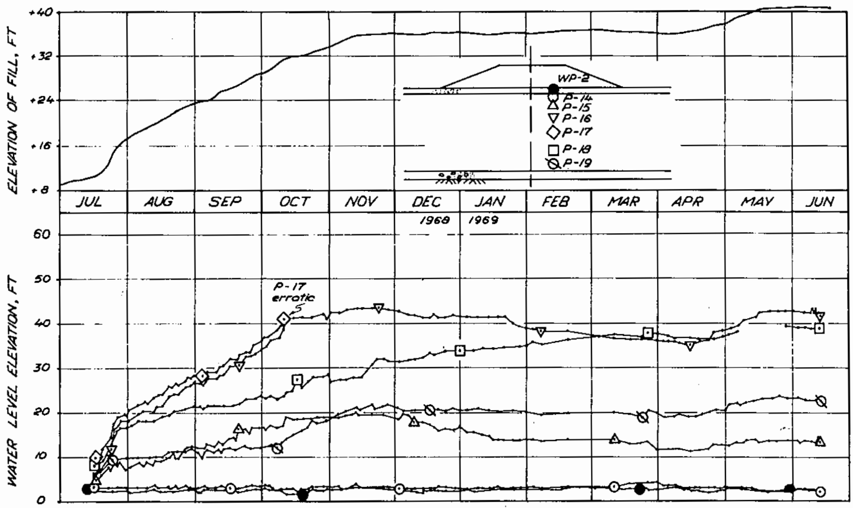



Figure 11. Pore Pressures 30 Feet Right of Center Line, Hydraulic Piezometers

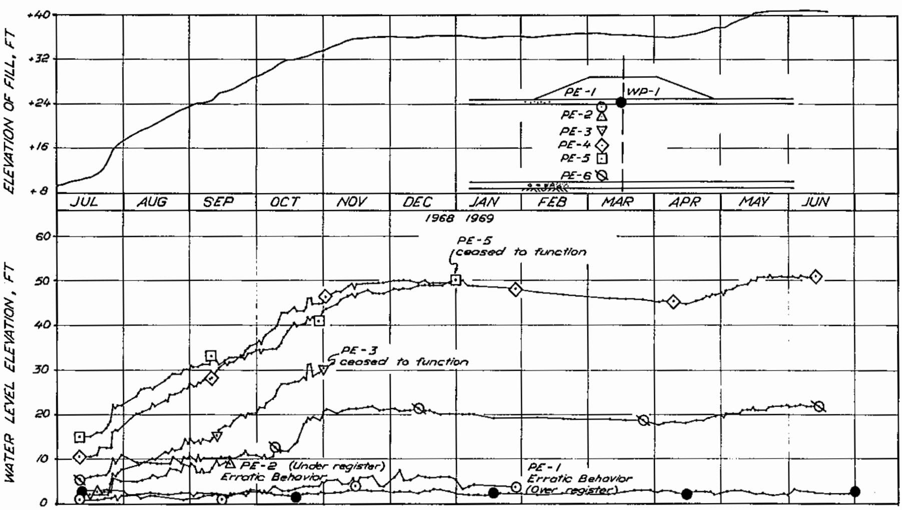



Figure 12. Pore Pressures 30 Feet Left of Center Line, Electric Vibrating Wire Piezometers

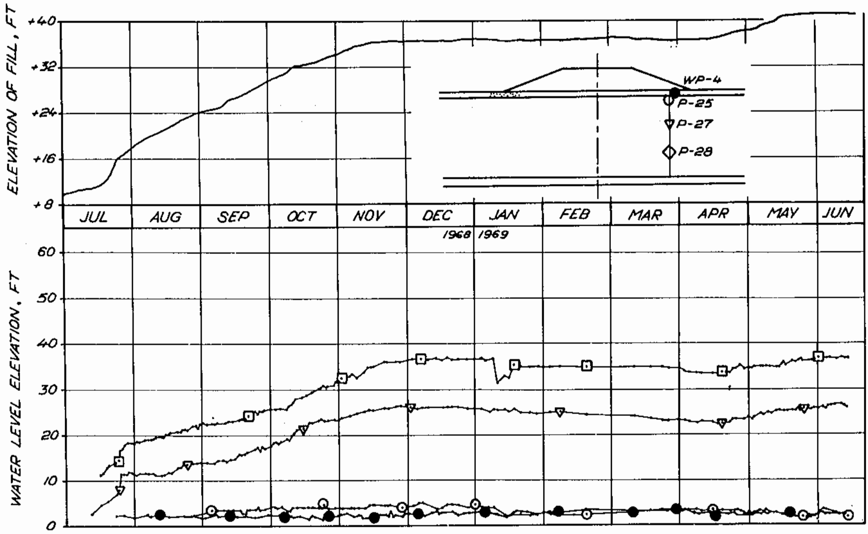



Figure 13. Pore Pressures 96 Feet Right of Center Line

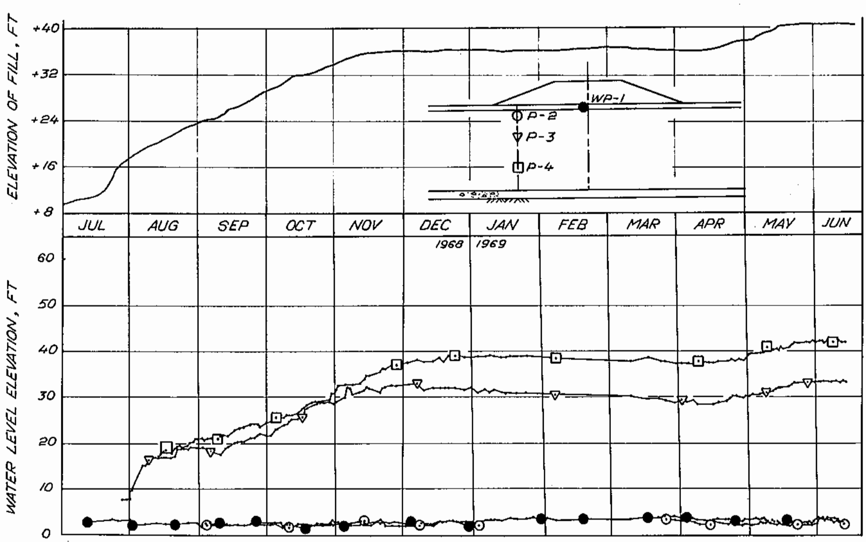



Figure 14. Pore Pressures 95 Feet Left of Center Line

Table 2
Detailed Status of Piezometers

| Piezometer Number | HYDRAULIC: Circulation OK, Data OK | HYDRAULIC: Circulation OK, Data (?) OK | HYDRAULIC: Circulation OK, Data ? | HYDRAULIC: Circulation N.G., Data (?) OK | HYDRAULIC: Circulation N.G., Data ? | HYDRAULIC: Circulation N.G., Data N.G. | ELECTRIC: Reading of 0000 cps | ELECTRIC: Reading of Oscillates | ELECTRIC: Data O.K. |
|---|---|---|---|---|---|---|---|---|---|
| P – 1 | | | ✓ | | | | | | |
| P – 2 | | | ✓ | | | | | | |
| P – 3 | | | | ✓ | | | | | |
| P – 4 | ✓ | | | | | | | | |
| P – 5 | | | | | ✓ | | | | |
| P – 6 | | | | | ✓ | | | | |
| P – 7 | | | | | | ✓ | | | |
| P – 8 | | | | | ✓ | | | | |
| P – 9 | | | | ✓ | ✓ | | | | |
| P – 10 | | | | ✓ | ✓ | | | | |
| P – 11 | | | ✓ | | | | | | |
| P – 12 | ✓ | | | | | | | | |
| P – 13 | | | ✓ | | | | | | |
| P – 14 | ✓ | | | | | | | | |
| P – 15 | | | | | ✓ | | | | |
| P – 16 | | ✓ | | | | | | | |
| P – 17 | | | | | ✓ | | | | |
| P – 18 | | ✓ | | | | | | | |
| P – 19 | | | | | ✓ | | | | |
| P – 20 | ✓ | | | | | | | | |
| P – 21 | | ✓ | | | | | | | |
| P – 22 | | | | ✓ | | | | | |
| P – 23 | | ✓ | | | | | | | |
| P – 24 | | | ✓ | | | ✓ | | | |
| P – 25 | ✓ | | | | | | | | |
| P – 26 | ✓ | | | | | | | | |
| P – 27 | ✓ | | | | | | | | |
| P – 28 | | | | | ✓ | | | | |
| P – 29 | ✓ | | | | | | | | |
| P – 30 | | ✓ | | | | | | | |
| P – 31 | ✓ | | | | | | | | |
| P – 32 | ✓ | | | | | | | | |
| P – 33 | ✓ | | | | | | | | |
| PE – 1 | | | | | | | | ✓ | |
| PE – 2 | | | | | | | | | ✓ |
| PE – 3 | | | | | | | ✓ | | |
| PE – 4 | | | | | | | | | ✓ |
| PE – 5 | | | | | | | ✓ | | |
| PE – 6 | | | | | | | | | ✓ |

**Evaluation of Peizometers.** Two-year performance records for double tube hydraulic piezometers and vibrating wire electrical piezometers are summarized in Fig. 6. The detailed status of piezometers at the end of this two-year period, July 1970, is summarized in Table 2. These data clearly show that a high percentage of all piezometers fail within two years of installation. Particularly alarming is the width of the crosshatched band of questionable data.

The hydraulic piezometers have performed for a longer period than the vibrating wire piezometers. However, during the embankment construction period, the electrical piezometer data have been compatible with data from symmetrical hydraulic piezometers.

One main cause of piezometer failure has been the differential settlement of the casing and the piezometer sensor. Due to large casing settlement the plastic tubing just above the sensor is cut or pinched off. This was observed visually when one of the piezometers monitored by the consulting engineers was pulled out of the ground for inspection. The sudden loss in pressure experienced by several piezometers supports this explanation. Several additional causes of failure may be cited for electrical piezometers: shift in zero frequency, leakage through O-rings at the sensor, and air at the sensor.

Piezometers P-6, P-8, and P-10, located at the center line of the embankment, were replaced in December 1970 to ensure that reliable pore pressure data are obtained during consolidation and subsequent removal of overload.

### *B. Settlement Rods, Anchors and Platforms*

**Station 245.** Figure 3 shows the location of five settlement anchors and one temporary bench mark, designated PC-SA1 through PC-SA6, which were installed at the Preconstruction Section. These instruments measured the early settlement and heave prior to the installation of the Station 246 instrumentation.

The settlement anchors were manufactured by Borros A/B. The anchor consists of two parts: the point and three anchor rods. The point is a 1¼-in. hollow steel cylinder with three circular milled grooves. A short section of 1 in. steel pipe is connected to the point and additional sections added as necessary. The anchor rods are approximately 3/8 in. in diameter and are welded together at their upper ends and connected to a round plate. A short section of ¼-in. steel pipe is then connected to the plate and additional sections added as necessary. The installation procedure is described in Appendix A.

**Station 246.** Twelve settlement rods, five settlement platforms, and one permanent bench mark were installed at Station 246 (see Fig. 4). All of the settlement rods and three of the settlement platforms terminate at the instrument tunnel. Level surveys are made from the permanent bench mark which is in the tunnel.

The settlement rod (see Fig. 15) is a 1-in. standard pipe with a hollow disk welded 3.0 ft from the bottom. The disk is 1½ in. in diameter and ½ in. thick. The settlement platform (see Fig. 16) is a 4.0-ft square, ½ in. thick steel plate. A 1½-in. steel pipe is welded to the steel plate and extended through the fill as necessary. The installation procedures are described in Appendix B.

**Observed Data.** Settlement-heave data at the Preconstruction Section are shown in Fig. 9. Figure 17 summarizes the settlement data at the MIT-MDPW Test Section through June 1969. Data for SP-4 and SP-5 are now shown due to their lack of precision. These two platforms are outside the tunnel and had to be surveyed using a distant bench mark.

**Evaluation of Settlement Rods, Anchors, and Platforms.** All of the settlement rods, anchors, and platforms performed satisfactorily during the period of embankment construction.

## *C. Inclinometers*

**Station 246.** Figure 4 shows the locations of the six inclonometers installed at the MIT-MDPW Test Section. These are dual purpose instruments since they give data on vertical movement as well as horizontal movement within the foundation.

The inclinometers were manufactured by Slope Indicator Co., Seattle, Washington. The inclinometer, shown in Fig. 18, consists of a 3-in. inside diameter aluminum casing having four continuous longitudinal grooves. The casing is assembled from five-foot sections connected by telescoping couplings which allow 6 in. of vertical movement at each coupling. The casing is installed in a 6-in. open borehole which extends through the clay and 5 ft into glacial till. The longitudinal grooves are aligned parallel and perpendicular to the axis of the embankment and the annular space backfilled with sand or peastone.

Lateral movements were determined by a special electronic torpedo which is lowered into the casing and rides in the longitudinal grooves. The sensing element of the torpedo is essentially a pendulum-actuated Wheatstone bridge circuit. A reading is made at three points within each five-foot section of casing. The slope of the casing at each point is calculated, and integrating from the fixed bottom yields a plot of lateral movement versus depth.

Vertical movements were determined by a special settlement torpedo which is lowered down the casing with a steel tape. Spring loaded arms catch the bottom of each five foot section and the depth to the bottom of each section is recorded. The elevation of the top of the inclinometer casing is then used to convert the depth readings to elevations.

**Observed Data.** Periodic observations of the inclinometers were made. The data for horizontal displacements in the plane of the embankment are summarized in Fig. 19 as a function of embankment height and time. The horizontal displacements along the axis of the embankment (north-south direction) were negligibly small, i.e., less than 1 inch.

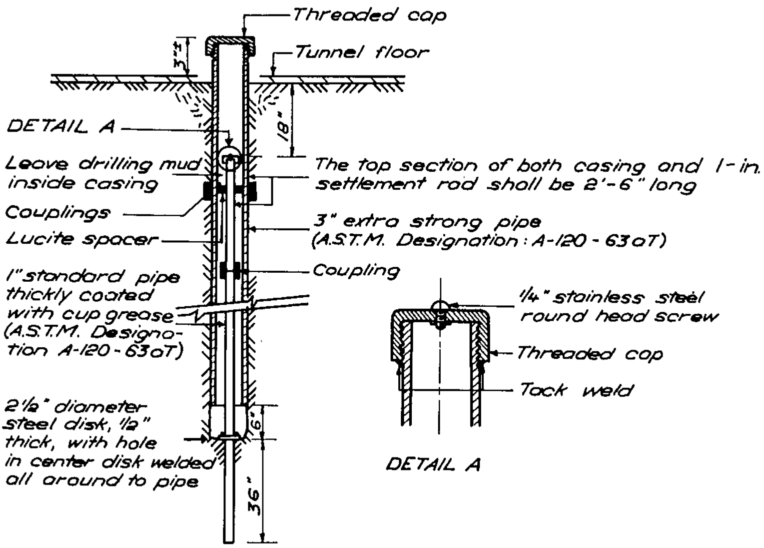



Figure 15. Details of the Settlement Rod

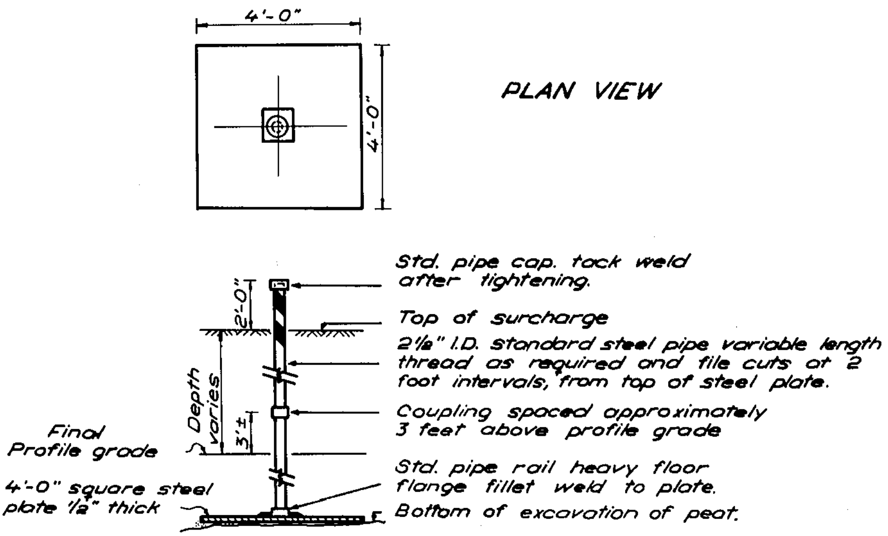



Figure 16. Details of the Settlement Platform

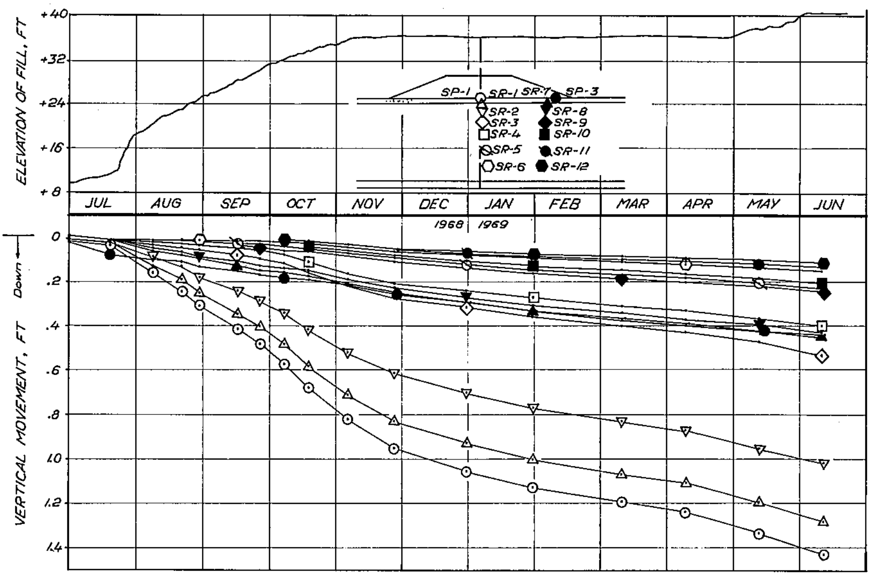



Figure 17. Settlement Data at the MIT Test Section

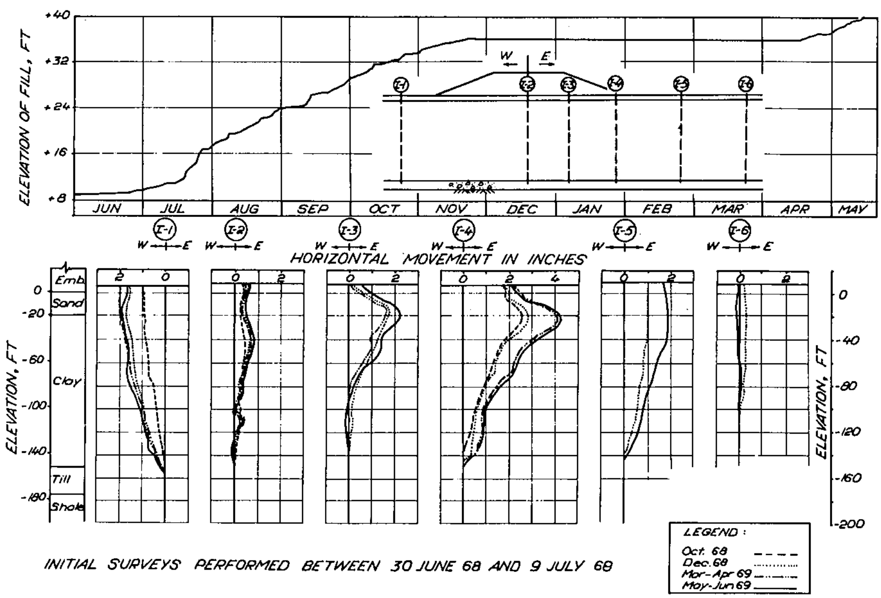



Figure 19. Horizontal Movement at the MIT-MDPW Test Section

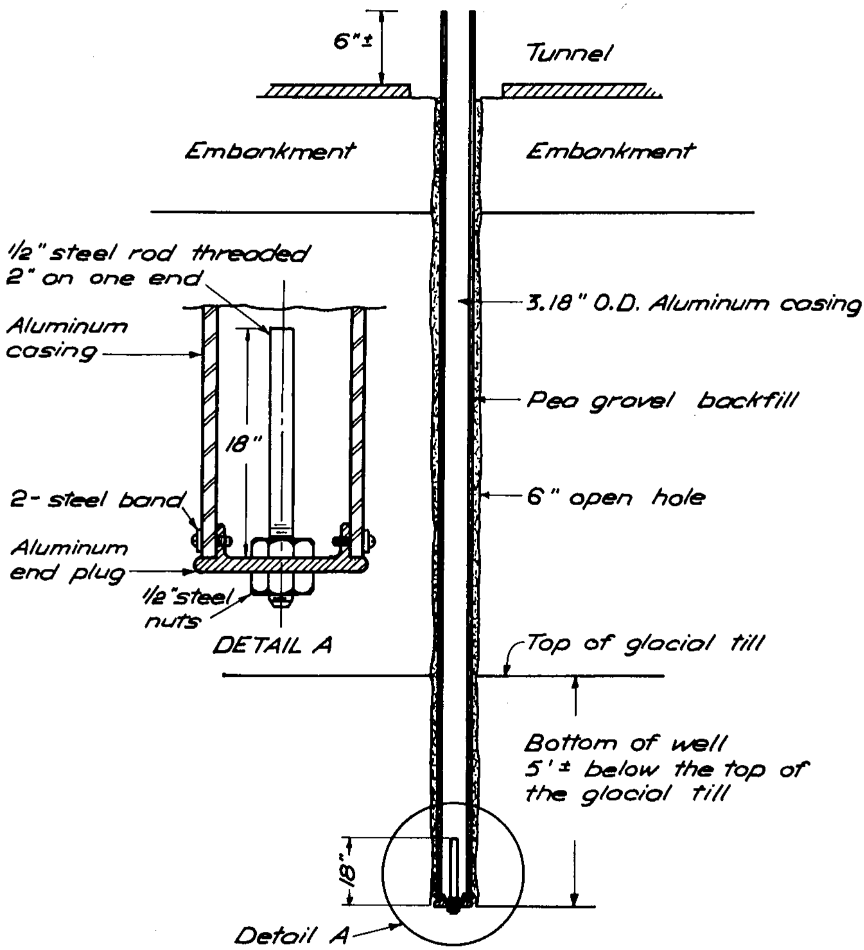



Figure 18. Details of the Inclinometer

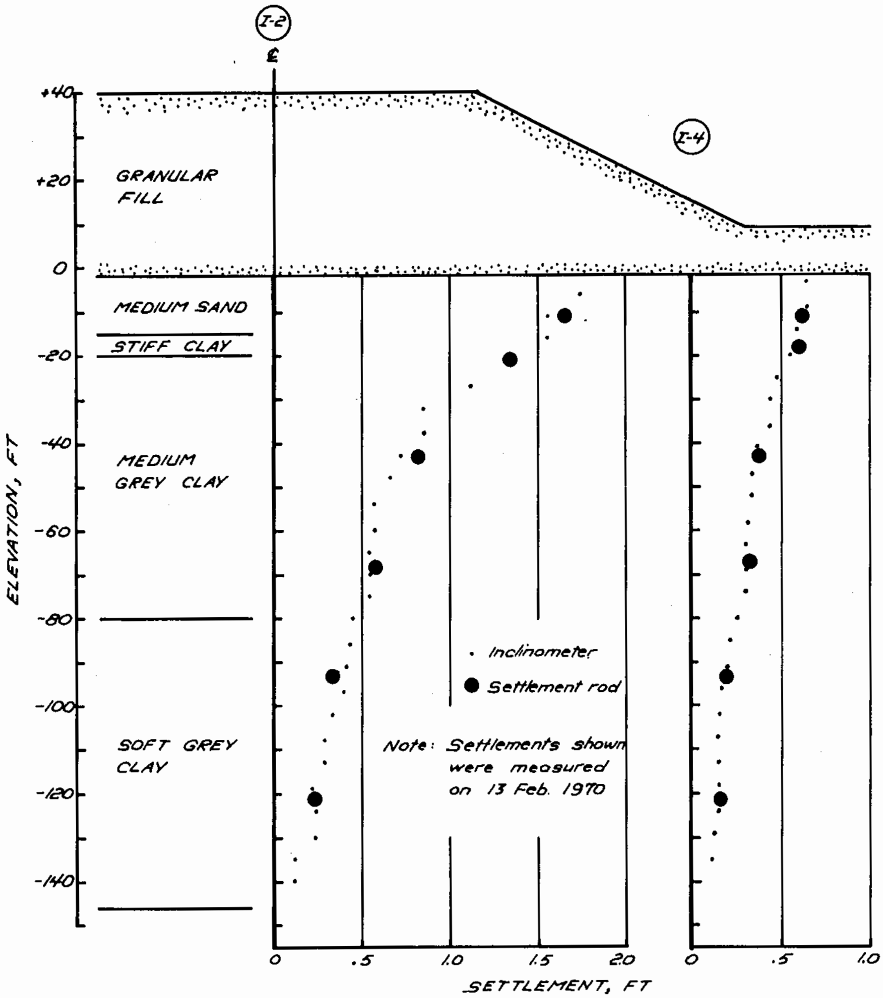



Figure 20. Comparison of Settlement Data From Inclinometers and Settlement Rods

Settlement data obtained in February 1970 from inclinometers I-2 and I-4 are compared with the settlement profile from the settlement rods in Fig. 20. The agreement is excellent. Figures 21 and 22 compare the settlement rod data to the corresponding settlement data obtained from vertical inclinometer surveys. Again the agreement is good.

**Evaluation of Inclinometers.** There have been no technical problems with the inclinometers. The data on lateral movement appear reasonable, and the vertical movement determined from coupling surveys is in excellent agreement with the settlement rod measurements. One inclinometer, I-6, was lost when vandals filled it with stones.

The lateral movement data appear reasonable but the following factors are thought to affect the precision of the measurements:

1. It is most likely that a 150-ft long inclinometer will not be installed exactly vertical. For the six inclinometers installed, the eccentricity from vertical varied from 4 to 38 in. Due to this initial imperfection in orientation, an inclinometer may not be able to reflect the imposed soil displacements with high precision.
2. During the installation a number of couplings may collapse and impose a condition where certain segments of the inclinometer are relatively less flexible for bending and cannot respond to vertical displacements.
3. In the survey of an inclinometer of 150 ft in length, approximately ninety readings are made. The position of the inclinometer is computed by integration of recorded slopes from the bottom (i.e., fixed point) to the top. Thus a very small bias in measuring system, which is hard to detect by the field technician, may accumulate to significant errors.

It is usual procedure to verify the horizontal location of the top of the casing by an independent optical survey. However, the location of the inclinometers inside the tunnel precluded this independent check. All inclinometer data reported here are based on the assumption that the bottom of the casing was fixed.

## *D. Total Stress Cells*

**Station 246.** A principal feature of the instrumentation at the MIT-MDPW Test Section is the total stress cells. The objective is to measure the distribution of vertical total stress imposed by the embankment on the foundation soil. The common assumption in practice is that the vertical stresses at the embankment foundation interface are distributed in direct proportion to the height of the embankment. The total stress measurements enable one to check the validity of this assumption and evaluate recent theoretical work done in this area.

**Mechanical Features of Total Stress Cells.** The total stress cells were manufactured by Geonor A/S of Norway and commercially available under the brand name of P-100 Earth Pressure Cells. Figure 23 shows a plan and cross

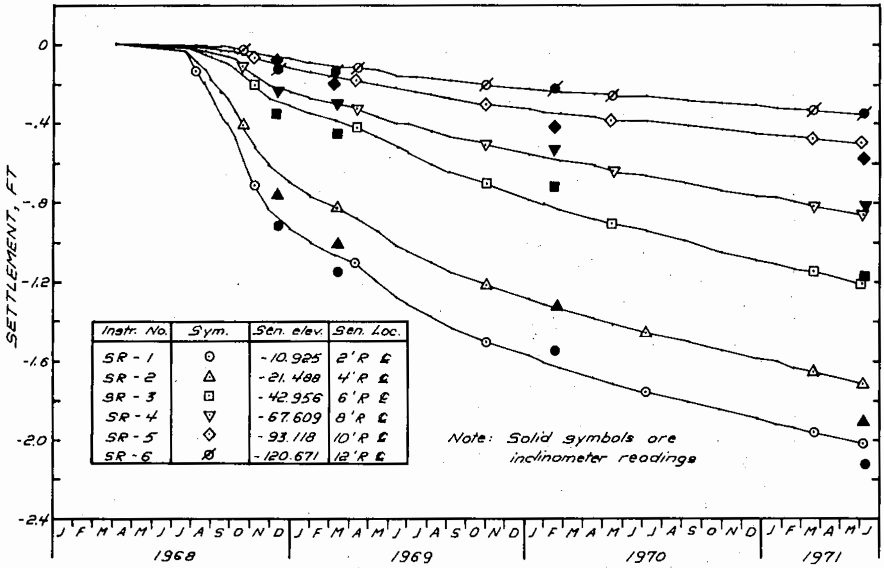



Figure 21. Comparison of Center Line Settlement SR-1 to SR-6 vs. I-2

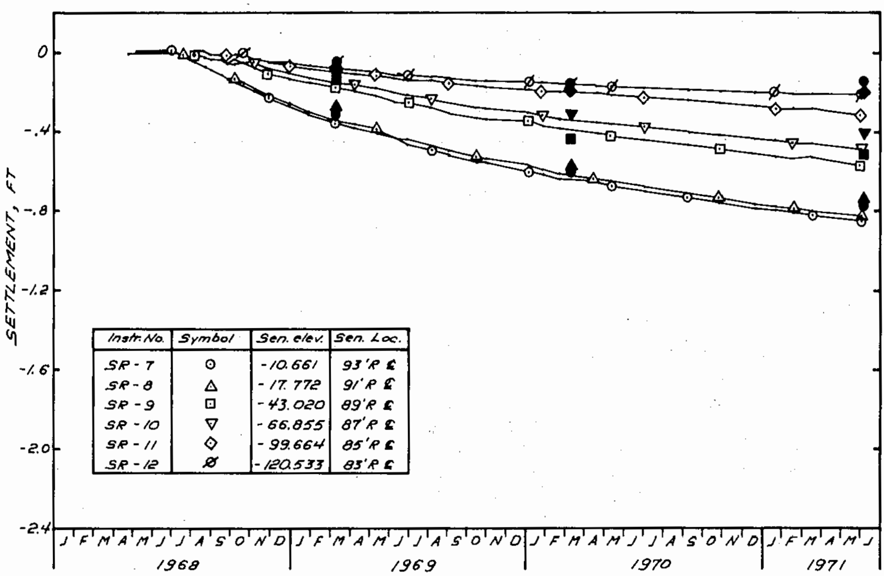



Figure 22. Comparison of Settlement 95 Fleet Right SR-7 to SR-12 vs. I-4

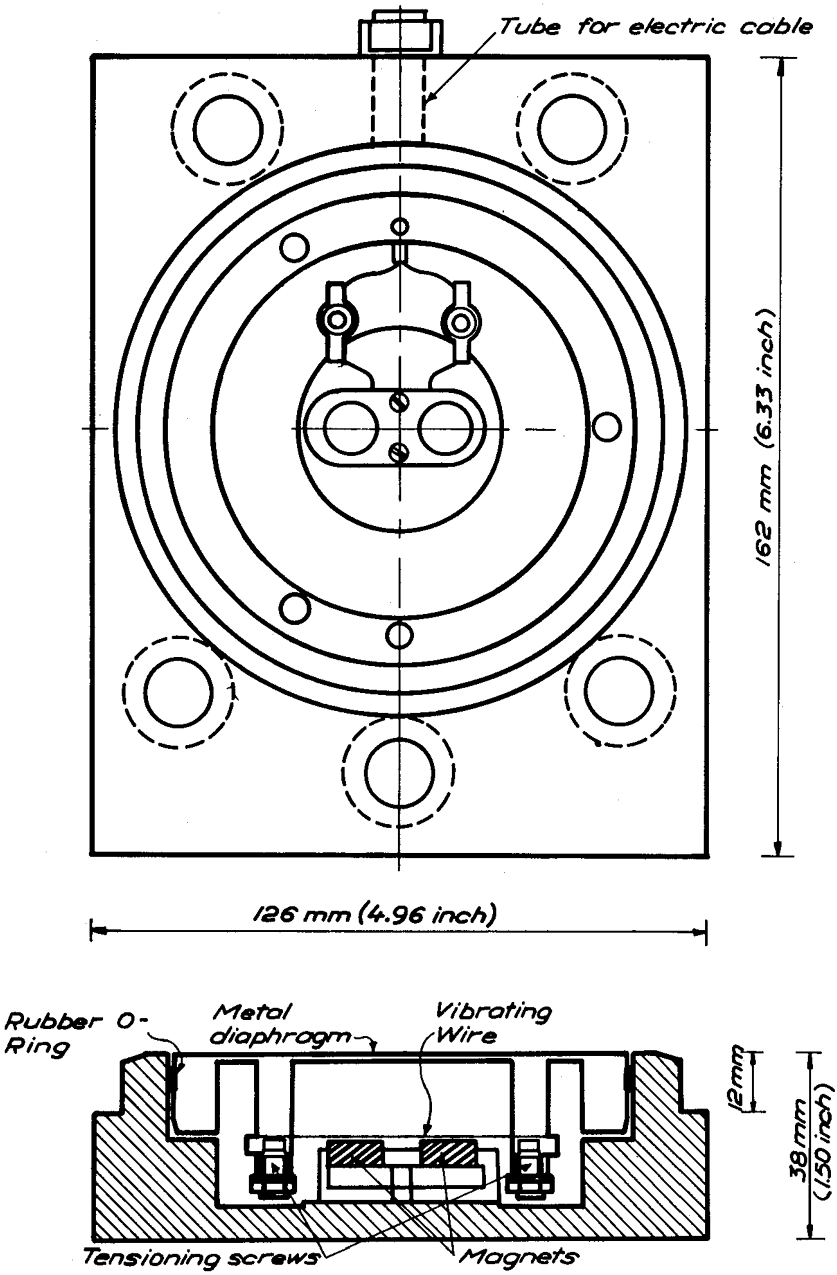



Figure 23. Details of the Total Stress Cell

section of the cell. The particular geometry of the cell is due to the fact that they were designed to be mounted on sheet piles at another ICEP project. Difficulties encountered in driving the sheet piles on which the cells were mounted led to abandonment of that attempt to measure total stress. The remaining stress cells were used for the I-95 project.

The stress acting on the metal diaphragm deforms it and causes tension in the wire stretched between the two diaphragm posts. The natural vibrating frequency of the wire depends on the tension and thus the stress action on the diaphragm. One electromagnet activates the wire and the other picks up the vibrating frequency which is amplified and measured directly with a digital frequency counter.

**Laboratory Calibration of Total Stress Cells.** Since the cells were originally designed to measure lateral earth pressures in soft and medium-soft clays, Geonor recommended using the hydrostatic calibration obtained by means of a water pressure tank. The I-95 embankment is course to medium sand with scattered gravels and boulders. Therefore, it was felt necessary to observe the behavior of the cells in a granular medium. The ultimate goal of this study was to develop the best approach in determining the calibration curves for granular soil.

The laboratory tests were carried out in a 35.12 in. diameter steel bin, 27 in. high. The dry uniform medium-fine sand (commercially called No. 1/2 sand) was placed by the "raining technique" at a rate of 5 pounds per minute with a height of fall of 10 in. In this way a uniform density of 102 ± 5 lb/cu ft was achieved. The bin was filled 9 in. and then the cell placed horizontally with the sensing diaphragm up. Another 9 in. of sand was placed above the cell. The pressure was applied through a cylindrical pressure bag and a 5 in. plywood shim was placed between the pressure bag and the steel top cap which is bolted to the bin during loading.

Figure 24 shows that the response of the cell to applied pressure through the medium sand followed a pattern close to the water pressure calibration. A tendency of underregistration for the dense state and overregistration for the loose state was observed. This is consistent with the tests of Plantema (1953). A hysteresis effect was observed with higher registering in unloading than in loading. This is in agreement with the tests of Leussink and Prange, and Plantema (1953). The hysteresis effect should be considered in a case of loading and then unloading. Such is the case at I-95.

Figure 24 also shows that within the range of loading at the I-95 embankment (i.e., up to a level of 25 to 30 psi) the calibration for the loose sand backfill is almost linear with a slope somewhat different than that of the water pressure calibration.

**Cluster Scheme for Field Observation.** The measurement of total stresses in a soil mass is a most difficult task. Depending on the characteristics of the

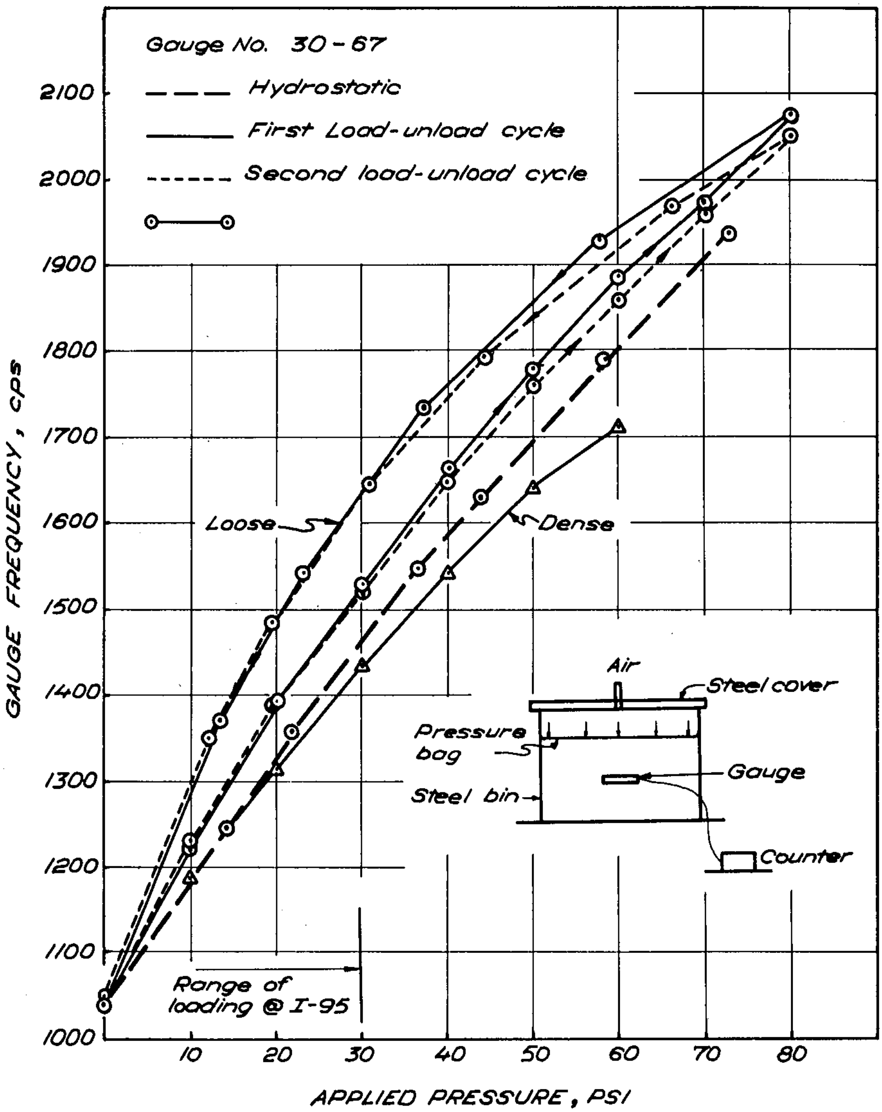



Figure 24. Laboratory Calibration of the Total Stress Cells

measuring device and the way it is placed in the soil, the device will sense more or less stress than would have existed had the device not been present. Taylor (1945) studied the effect of geometry and compressibility of a stress cell in reference to over- and underregistering. Guidelines for diameter to thickness and diameter to compressibility ratios were proposed to minimize these effects. Hadala (1967) examined the influence of placement methods on the response of the Waterways Experiment Station type total stress cells placed in sand and clay. Also, based on his observations on data scatter, he concluded that using a single stress cell to measure the magnitude of stress in a soil mass with any reasonable degree of confidence is a fruitless effort. Hadala recommends that an average of at least three cell measurements should be used if 20 percent accuracy is required nine times out of ten. Taylor's (1945) observations on three Carlson cell clusters placed in Arkabutla Dam (U.S. Corps of Engineers) are in agreement with Hadala's suggestions.

In light of this previous experience with total stress cells installed in a soil mass the scheme shown in Fig. 25 was adopted. This scheme calls for three stress cells bolted on a 2.0-ft diameter 1 in. thick circular steel plate. The cells are placed at 3¼-in., 5¼-in. and 7¼-in. distances away from the center of the plate to observe the overall stress distribution imposed on the plate.

**Field Installation and Field Calibration.** The stress cells were installed in three clusters: at the center line, 30 feet right of the center line, and 60 feet right of the center line. Figure 26 shows the location of the three clusters. At the time the cells were installed, the average fill elevation was +20.0 ft and three circular funnel type holes approximately 3 feet in depth were excavated (see Fig. 26). To prevent potential arching the average diameter of the hole was about eight times the diameter of the cell cluster. The cell cluster was placed on firm soil, leveled and monitored for zero load reading. The backfilling was made by "raining" the material through a No. 4 sieve. Slight compaction by tamping was used. After 2 ft of backfill were placed, filling continued with regular embankment fill. After each foot of backfill the cells were monitored and field density tests were performed. Above Elev. +20 ft the filling was carried on in the regular way with rubber-tired compaction, and field densities were taken at an average of one per foot of fill. Assuming that up to Elev. +25 ft the vertical total stress acting on the cells would be practically equal to the average fill density times the height of fill, the observations in this range were treated as field calibrations. Figure 27 shows a typical field calibration established in this manner. The hydrostatic calibration was used as a reference line in extrapolating the field calibration for higher stress levels. The nearly linear behavior of the field calibration for the particular range of stress level was deduced from the laboratory calibration studies (see Fig. 24).

**Typical Data of One Cluster.** Figure 28 shows elevation of fill versus total vertical stress measured by the three cells of Cluster A. The maximum difference

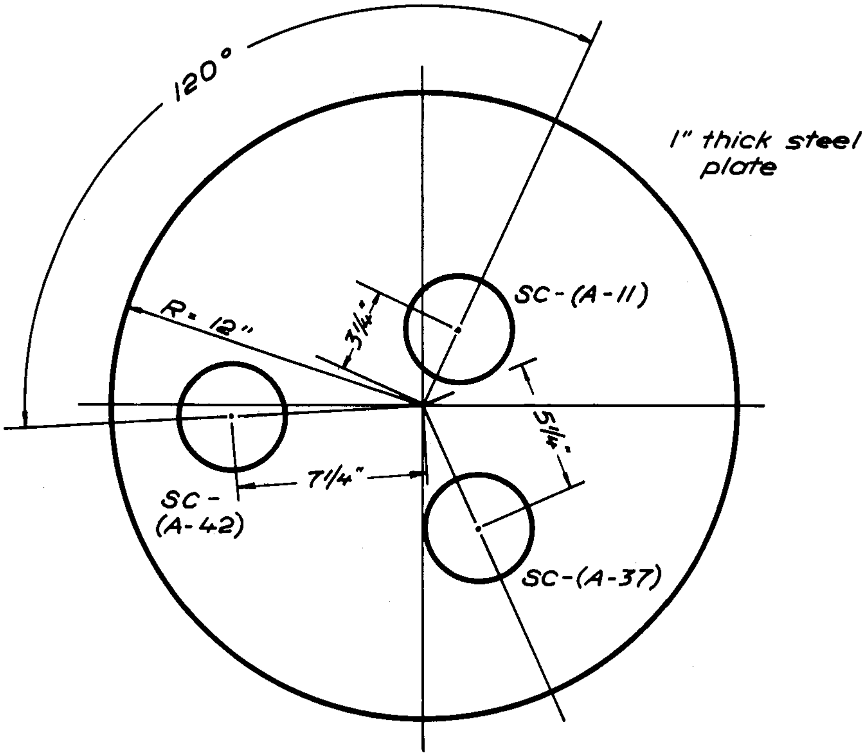
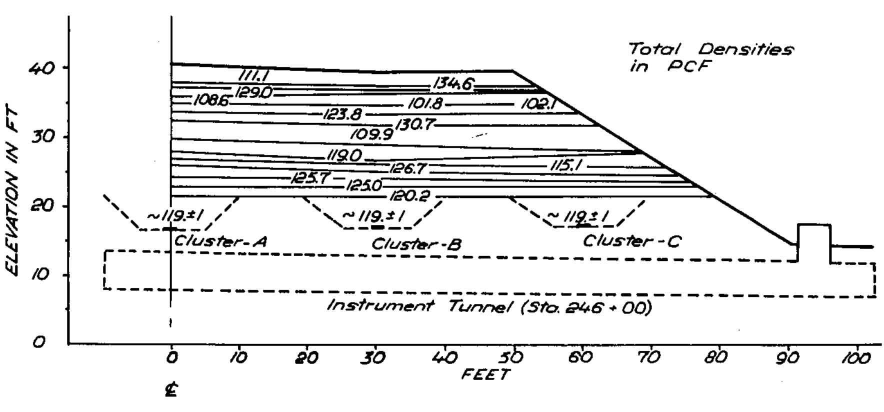



Figure 25. Details of Total Stress Cell Cluster




Digure 26. Location of Total Stress Cell Clusters

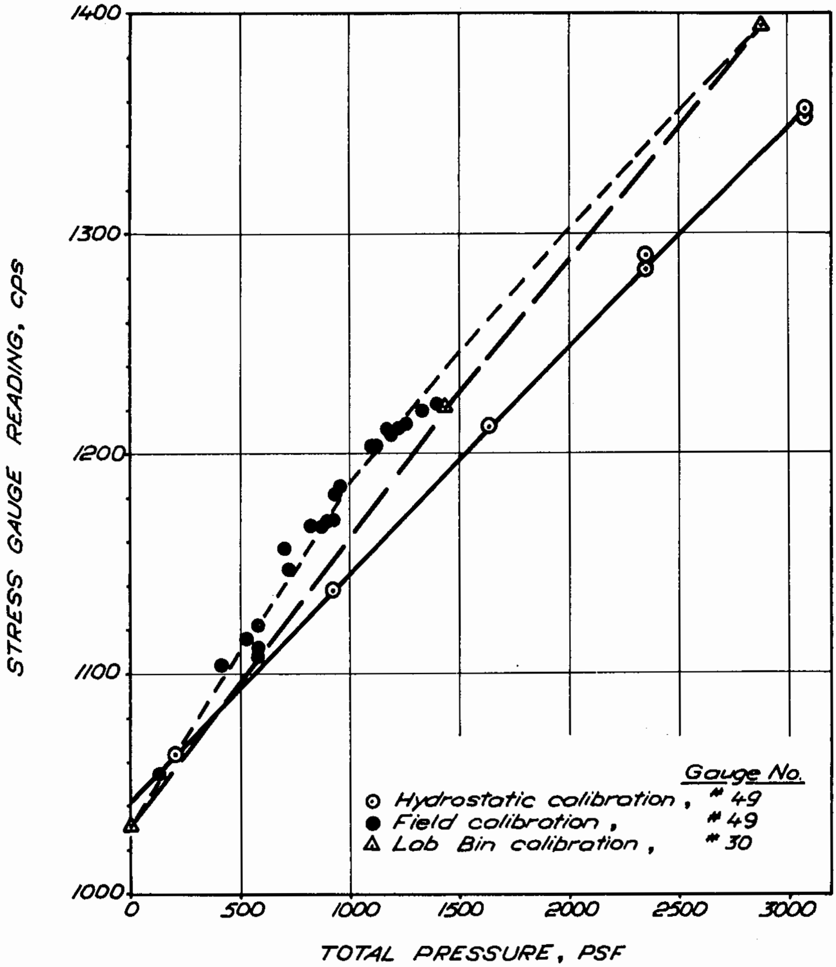



Figure 27. Field Calibration of Total Stress Cells

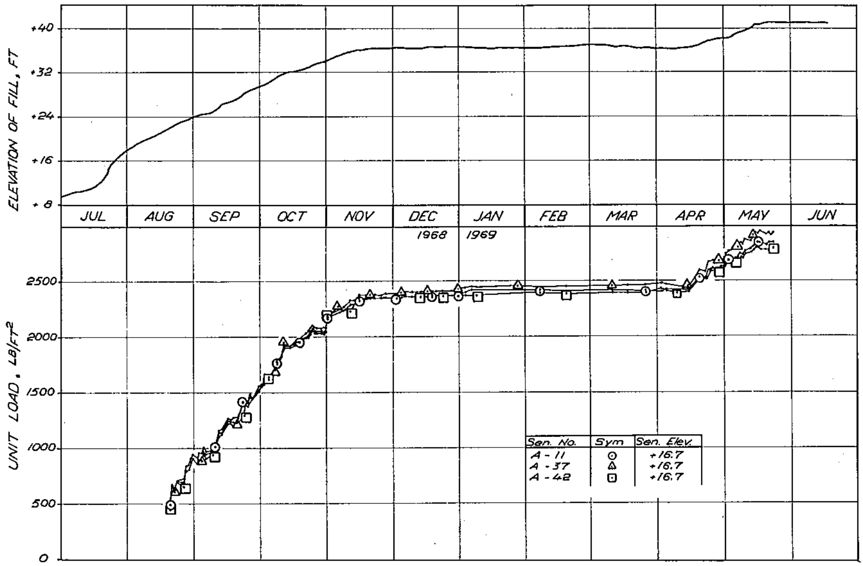



Figure 28. Total Stress Cell Measurements for Cluster "A"

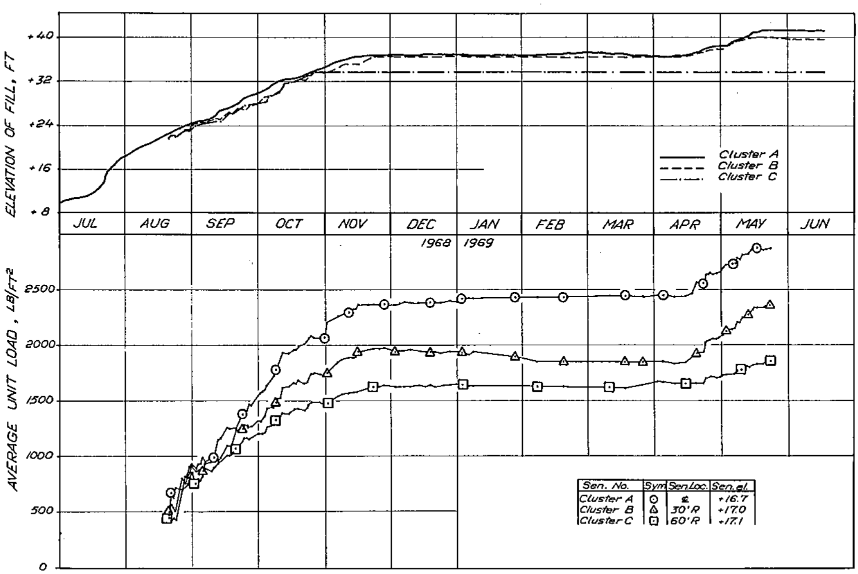



Figure 29. Total Stress Measurements for Clusters "A", "B", and "C"

in individual cell readings is about 150 $lb/ft^2$ over a total magnitude of nearly 3000 $lb/ft^2$. Similar observations were made from Clusters B and C.

**Data of Average of Clusters A, B, and C.** Figure 29 shows elevation of fill versus average measured total vertical stress for Clusters A, B, and C. Note that between November 20, 1968 and April 20, 1969, during which time no filling was made, Cluster A and Cluster C have picked up an increment of approximately 100 $lb/ft^2$ and 50 $lb/ft^2$, respectively, and Cluster B experienced a release of approximately 100 $lb/ft^2$.

Figure 30 shows for several fill heights the geometry of the embankment, the total vertical stress distribution equivalent to average unit weight times the embankment height, and the measured total vertical stresses. For the maximum fill configuration, Elev. +40 ft, the theoretical prediction given by Perloff et al (1967) for homogeneous, isotropic, linear elastic embankment and foundation has been shown for comparison. The agreement is very good.

## IV. Conclusions

1. The instrumentation system has sufficiently documented the performance of a heavily loaded, compressible foundation during the loading period. Replacement of some piezometers was made to adequately document the post-construction period.

2. The deformation and total stress instruments have performed very satisfactorily and only minor changes to the designs and installation procedures would be made for subsequent jobs.

3. Major changes in the design and installation procedures for piezometers, both electrical and hydraulic, will be made for future installations. There are many cost and time incentives for developing reliable, multiple sensor piezometers to fit in one borehole, as well as to solve longevity problems due to large deformations, shifting calibrations (electric) and trapped air (hydraulic). Piezometer sensors will not be placed below rigid casings where large settlements are expected. Double tubing for hydraulic piezometers will be used at all times.

4. Major improvements are also planned for data acquisition and data handling problems. The procedures used for this project were slow, used a lot of skilled manpower and produced job records that were somewhat unwieldy for rapid and diverse use.

## V. Acknowledgments

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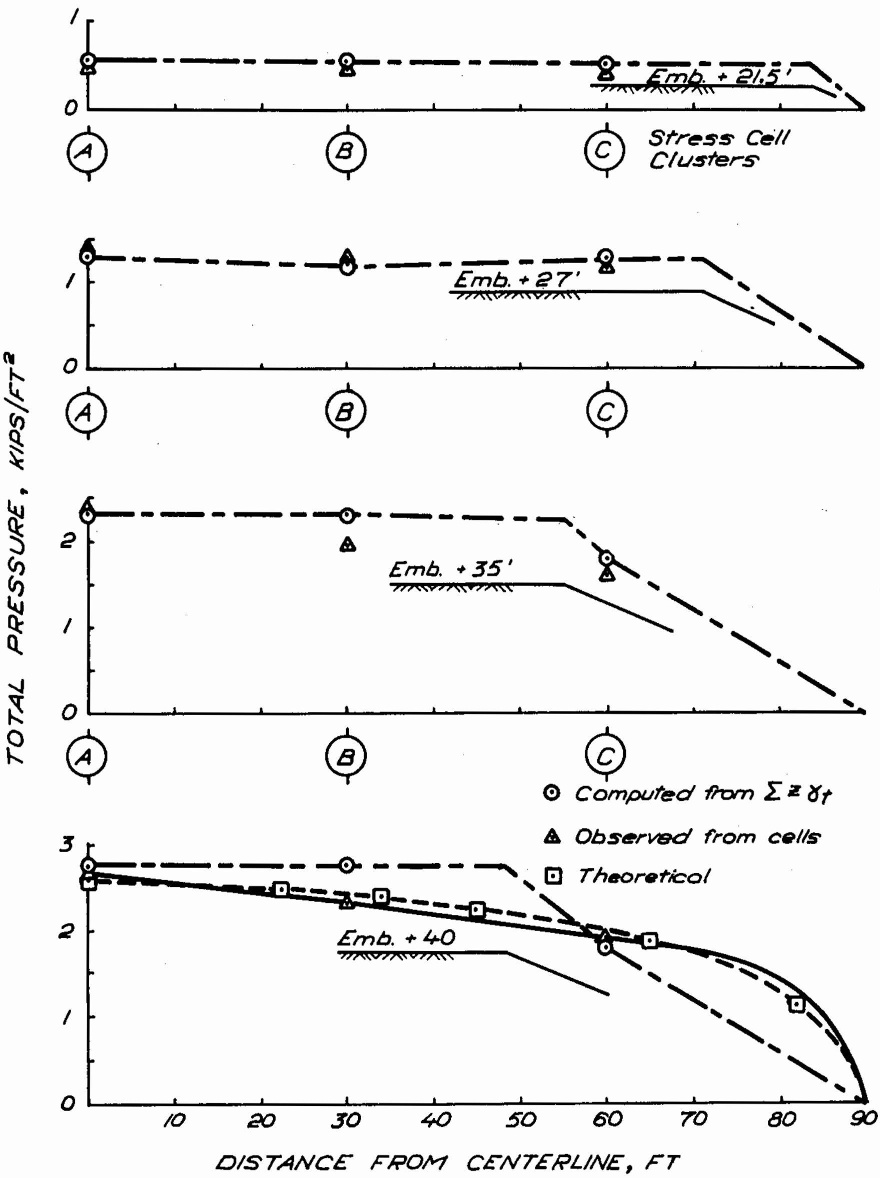


Figure 30. Comparison of Theoretical and Measured Total Stress Values

have received from the sponsors, especially Mr. Charles Whitcomb, Deputy Chief Engineer for Highway Design, Mr. Robert T. Tierney, Deputy Chief Engineer for Highway Maintenance, Mr. John J. Lyons, Research and Materials Engineer, and Mr. Paul McHugh, Assistant Research and Materials Engineer, all of the Massachusetts Department of Public Works.

The Perini Corporation constructed the embankment and installed the main test section instrumentation through a subcontract with Geomeasurements, Inc.

The M.I.T. principal researcher was T. William Lambe, Edmund K. Turner, Professor of Civil Engineering. Walter Beckett, Joseph Guertin, Kjell Karlsrud, Robert Kirby, and Robert McPhail of the field measurements staff and several Northeastern University Coop students have done most of the work. Robert Kirby made important additions and editing to the text. The contributions of all are appreciated.

The opinions, findings and conclusions expressed in this publication are those of the authors and not necessarily those of the Massachusetts Department of Public Works or the Bureau of Public Roads.

## APPENDIX A – REFERENCES

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# APPENDIX B – INSTALLATION PROCEDURES USED AT STATIONS 245 and 246

## I. Hydraulic Piezometers – Preconstruction Section

1. Advance 2½-in. diameter casing to within 50 ft of piezometer sensor elevation by any method.
2. Wash to bottom of casing.
3. Attach the piezometer to a sufficient length of E-rod, being careful not to crimp the plastic tubing which goes inside the E-rod.
4. Press or drive the piezometer to the desired elevation, adding E-rod extensions as required. Leave the E-rod in place with approximately 5 ft extending up from the ground surface. The natural clay provides the piezometer seal.

## II. Hydraulic Piezometers – MIT-MDPW Test Section

1. Advance 2½-in. diameter casing through fill by any method; seal casing by pushing into the top of the clay layer.
2. Mix thick mud; sealing properties stressed not weight.
3. Wash ahead of casing with mud to 10 ft above center line of sensor.
4. Push casing to 2.0 ft above center line of instrument; wash out to bottom of casing with clean water.
5. Take split spoon sample.
6. Wash 1.75 ft below the center line of the sensor until the returning water is reasonably clear (do not recirculate water). Stop pumping, raise the wash rods 5 ft and wait for 10 minutes. lower the rods to the bottom of the hole and pump for 5 minutes. If the wash water remains clear the rods can be removed.
7. Lower the piezometer and sash weight into the hole. Place the sash weight on the bottom and check to see if the bottom is firm. If the bottom feels soft remove the piezometer and rewash the hole.
8. Keeping the piezometer leads tight, slowly add sand until it fills the collection zone and is up in the casing. Tamp thoroughly using a tamping hammer.

9. Siphon 4 in. of water out of the casing. Add about one cup of bentonite balls, one at a time, until the water is 1½ in. below the top of casing. Wait 3 to 5 minutes and then add sand (or ¼ in. pea gravel) until the water is ½ in. below the top of casing. Wait 3 to 5 minutes and then tamp the seal, gradually at first (i.e., 6-in. drop), then increasing the drop to 12 and then 18 in. Continue tamping until the seal does not compress more than ½ in. for 10 blows. Repeat the above process until the seal is 24 in. thick.
10. Place 5 ft of sand in 1-ft lifts, tamping each lift thoroughly; then, keeping the leads tight, backfill the rest of the hole with loose sand.

## III. Electrical Piezometers – MIT-MDPW Test Section

1. After hole has been advanced to final depth and cleaned by the same methods as specified for hydraulic piezometers, lower the electrical piezometer to the bottom of the hole with the porous brass tip removed. Measure the exact depth of the sensor.
2. Monitor the piezometer for a minimum of 30 minutes and record readings every 5 minutes. When it is established that the instrument is in equilibrium, pull the instrument up taking readings at 5-ft intervals for a field calibration.
3. Without touching the instrument, remove from the water and record the frequency reading. This is called a zero reading.
4. Turn the instrument upside down in a bucket of warm water and put on the previously deaired porous brass tip under water. Cover the tip and the filter with two prophylactics and secure with an elastic. Securely fasten a thin string to the tip of the prophylactics.
5. Lower the instrument several feet into the water-filled hole and tear the membranes with the string.
6. Proceed as for hydraulic piezometers.

## IV. Settlement Anchors – Preconstruction Section

1. Advance a 2½-in. diameter open hole either through overlying granular soils or to within 50 ft of the proposed anchor elevation, whichever is closer.
2. Attach a 1.0-ft section of 1-in. pipe to the anchor point. This section of pipe should have one left hand and one right hand thread. The left hand threaded end is attached to the point using a special coupling. Count the number of turns necessary to tighten the coupling.
3. Attach lengths of ¼-in. rod and then the 1-in. pipe by sliding it over the ¼-in. rod.

4. Lower the anchor to the bottom of the open hole adding lengths of ¼-in. rod and 1-in. pipe as necessary. Drive the anchor to the desired depth below the bottom of the hole.
5. Hold the 1-in. pipe securely, tap the ¼-in. rod down 5 in. This will force the three prongs at the tip of the anchor out into the surrounding soil.
6. Unscrew the 1-in. pipe by rotating clockwise at least the number of turns noted when it was attached to the anchor.
7. Bump the 1-in. pipe back 3.0 ft.

## V. Settlement Rods –MIT-MDPW Test Section

1. Advance casing through fill by any method; seal at the top of the clay layer.
2. Mix thick drilling mud; sealing properties stressed.
3. Using mud wash ahead of casing to 3.0 ft above proposed sensor elevation.
4. Push casing to 6 in. above sensor elevation.*
5. Wash to sensor elevation with drilling mud. Do not clean out hole with clear water.
6. Lower the settlement rod into the hole until it rests on the bottom and then push it 3 ft into the clay.

## VI. Settlement Platforms – MIT-MDPW Test Section

1. Excavate to Elev. +5 ft through embankment fill using a grade-all.
2. Place platform at bottom of excavation and carefully level it.
3. Backfill around settlement platform with sand.

## VII. Inclinometers

1. Advance 6-in. diameter casing through the fill by any method; seal the casing by pushing into top of clay layer.
2. Mix thick mud with sealing properties stressed, not weight.
3. Advance 6-in. open hole to bottom of clay and then drill 5 ft into the underlying till. Keep hole full of drilling mud at all times to prevent cave-in.
4. Lower aluminum casing into hole. Successive prefabricated 11-ft sections (made up of two 5-ft sections joined together by a 2-ft coupling with 6 in. left between each section) are lowered using special clamps. Each 11-ft

*For future installations, push casing to no closer than twice the estimated differential settlement between sensor location and top of ground.

section is joined together by a 2-ft coupling with 6 in. left between each section. The couplings are coated with beeswax and the casing kept full of clear water to overcome bouyance.

5. Align grooves parallel and perpendicular to embankment.
6. Backfill annular space between aluminum casing and borehole with pea gravel. Continuously vibrate the aluminum casing during backfilling. Rodding the backfill with a ½-in. steel rod proved most effective for controlling arching of the backfill.
7. Wash casing out until you can circulate clear water.