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16. Abstract

A retaining wall is typically used to form the permanent wall of an excavation whenever space requirements make it impractical simply to slope the sides. During the last 20 years, reinforcement of the backfill has gained widespread popularity because of its flexibility, ease of installation, and economic advantages. A new design has been proposed by the TxDOT using facing panel units anchored into a cement stabilized backfill of sufficient strength to avoid the need for any soil reinforcement at all. As long as the intact strength of the stabilized soil has been sufficiently improved by cement addition, the structure becomes a conventional mass gravity structure.

In order to address the practical problems involved in any new design, two experimental walls were built at full scale, as part of on-going Texas Department of Transportation construction. These were extensively instrumented to check their long-term behavior, which, after some initial difficulties, has now been proven satisfactory.

This report documents the data obtained from the field instrumentation, both immediately after construction and for a period of two years afterwards. These results confirm that such a design, if properly engineered, can perform satisfactorily.

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MONITORING OF CEMENT STABILIZED

SOIL RETAINING WALLS

by

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Research Report 187-23
Research Study Number 0-187, Task 8
Research Study Title: Monitoring of Cement
Stabilized Retaining Walls

Sponsored by the
Texas Department of Transportation
In Cooperation with the
U.S. Department of Transportation
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November 1993

TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

IMPLEMENTATION STATEMENT

In order to provide an additional option for the design of soil retaining structures, a new design proposed by the Texas Department of Transportation was investigated and found to be viable. The verification of the design concept and field performance described in this report, now enables the potential use of this retaining wall design as an alternative to other proprietary and non-proprietary designs.

The Texas Transportation Institute is coordinating with department personnel in implementation of the research results to ensure that these will be relevant to highway department practice. Satisfactory utilization of this method of retaining wall construction may enable economies to be realized whenever earth retaining structures are required. The design is expected to be inexpensive in situations where stabilizing material (notably cement) is cheaply available.

DISCLAIMER

ACKNOWLEDGMENTS

In addition to the help and assistance of the Bridge Division of the Texas Department of Transportation (technical contact - Mark P. McClelland, Austin), the help of the following individuals is gratefully acknowledged: Mr. Mike Ho of the District 12 soils laboratory in Houston; Mr. Ed Suchicki of D-12, Steve Simmons, Marion Noski and the other D-12 field personnel at Cypress-Fairbanks; and Don Harley and Andy Munoz, Jr., of the FHWA.

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SUMMARY

A retaining wall is typically used to form the permanent wall of an excavation whenever space requirements make it impractical simply to slope the sides. During the last 20 years, reinforcement of the backfill has gained widespread popularity because of its flexibility, ease of installation, and economic advantages. The most common commercial example uses galvanized steel earth retaining walls and select backfill to form the retaining wall mass behind a precast concrete facing.

A new design has been proposed by TxDOT using facing panel units anchored into a cement stabilized backfill of sufficient strength to avoid the need for any soil reinforcement at all. As long as the intact strength of the stabilized soil has been sufficiently improved by cement addition, the structure becomes a conventional mass gravity structure.

In order to address the practical problems involved in any new design, two experimental walls were built at full scale, as part of on-going Texas Department of Transportation construction. These were extensively instrumented to check their long-term behavior, which, after some initial difficulties, has now been proven satisfactory.

This report documents the data obtained from the field instrumentation, both immediately after construction and for a period of two years afterwards. The results of instrumentation readings indicate some continuing settlement, but this has so far been primarily uniform. Differential settlements as measured by horizontal inclinometers have been no greater than 11.0 mm (0.433 in) and overall absolute settlements from survey data have so far been of the order of 45.0 mm (1.8 in). Soil stresses and anchor forces are largely as expected, at least in the absence of traffic loading. The fill in the body of the material is settling more than at the face of the wall, as was originally indicated by analysis, but this is not causing any problems so far. Differential settlement between adjacent points along the walls has in some places been high enough to cause localized angular distortions of up to 1% to 2% of the cement stabilized fill. However, the visual performance of both the experimental sections and of the conventional sections has been acceptable.

These results confirm that such a design, if properly engineered, can perform satisfactorily.

1. INTRODUCTION

In conjunction with a previous project (1178), a new design of retaining wall was investigated. This utilized facing panel units anchored into a cement stabilized compacted fill. Only short anchors were required to retain the facing panels, as mechanical stabilization of the cross-section is achieved by the addition of cement, rather than by full length earth reinforcements.

Two full-scale experimental retaining walls were constructed and instrumented, in order to demonstrate the feasibility and applicability of this design. Deformations of the walls and foundations took place over an extended period of time, in addition to redistribution of soil stresses and panel anchor forces.

To measure these quantities, extensive instrumentation of the two experimental wall sections was installed, as described in this report. In order to obtain the full benefit of this, a monitoring program of the instrumentation was conducted in this project. This was desirable in order to provide continuity of data and a full record of performance both before and after construction of the experimental design.

This report summarizes the results of the monitoring study of the installed instrumentation, which documents approximately two and a half years' worth of field data and measurements, until just before the time that the overpass was opened to traffic.

2. EARLIER FULL SCALE EXPERIMENTAL CONSTRUCTION

2.1 GENERAL

As part of the original project, it was decided to construct some full-scale sections of wall to the experimental design, in conjunction with the ongoing TxDOT work at that time.

In cooperation with District 12 and the Bridge Division, potential sites for the full-scale field test section were identified, and the optimum site recommended as the Cypress/Fairbanks bypass of Highway 290 northwest Houston. A total of 10 retaining walls were to be constructed at this site, of varying heights and sizes.

The walls on the northeast side of the 290 were suggested as more suitable than those on the southwest side, being more protected from traffic and diversions on the existing road during construction, access being somewhat easier after construction, and including also the highest section of wall in this location.

The optimum compromise was therefore felt to be walls no. 7 and 9 to be built in this design. These are walls of 5 m (16 ft.) and 7 m (23 ft.) height in their maximum cross-section, which were high enough to be significant, without having too extensive an experimental section. These locations are shown in Figures 2.1 and 2.2.

2.2 FIELD CONSTRUCTION AT TEST SITE

The CyFair bypass project was let April 27, 1989, to Williams Brothers Construction of Houston for $25,119,788. The retaining wall subcontract was to Baytex Construction using VSL earth retained walls for $2,296,028, made up of 236 $\$/m^2$ (22 $\$/ft^2$) for the non-stabilized wall and 258 $\$/m^2$ (24 $\$/ft^2$) for the stabilized wall apparently just factoring in the cost of the extra cement.

Their source of borrow material was at the southeast, and they commenced working towards it, from west to east, so that work started with walls 1 and 2, and finished with walls 9 and 10. Figure 2.6 indicates the cross-section of the initial design used for walls 7 and 9. Construction of the first cement stabilized soil retaining wall (wall 7) was commenced in October 1989 at the Cypress-Fairbanks 290 bypass by the contractor (Baytex Construction) as part of CSJ 0050-06-003. Site supervision was provided by District 12 (Houston) personnel. Surveying control was provided by the contractor. Initial foundation preparation for wall 7 is shown in Figure 2.7.

VSL Retained Earth also agreed to cast suitably sized 100 mm (4 in.) diameter holes into certain panels to allow horizontal inclinometer measurements to be made. The short anchors used to anchor the face plates of the wall are shown in Figure 2.8.

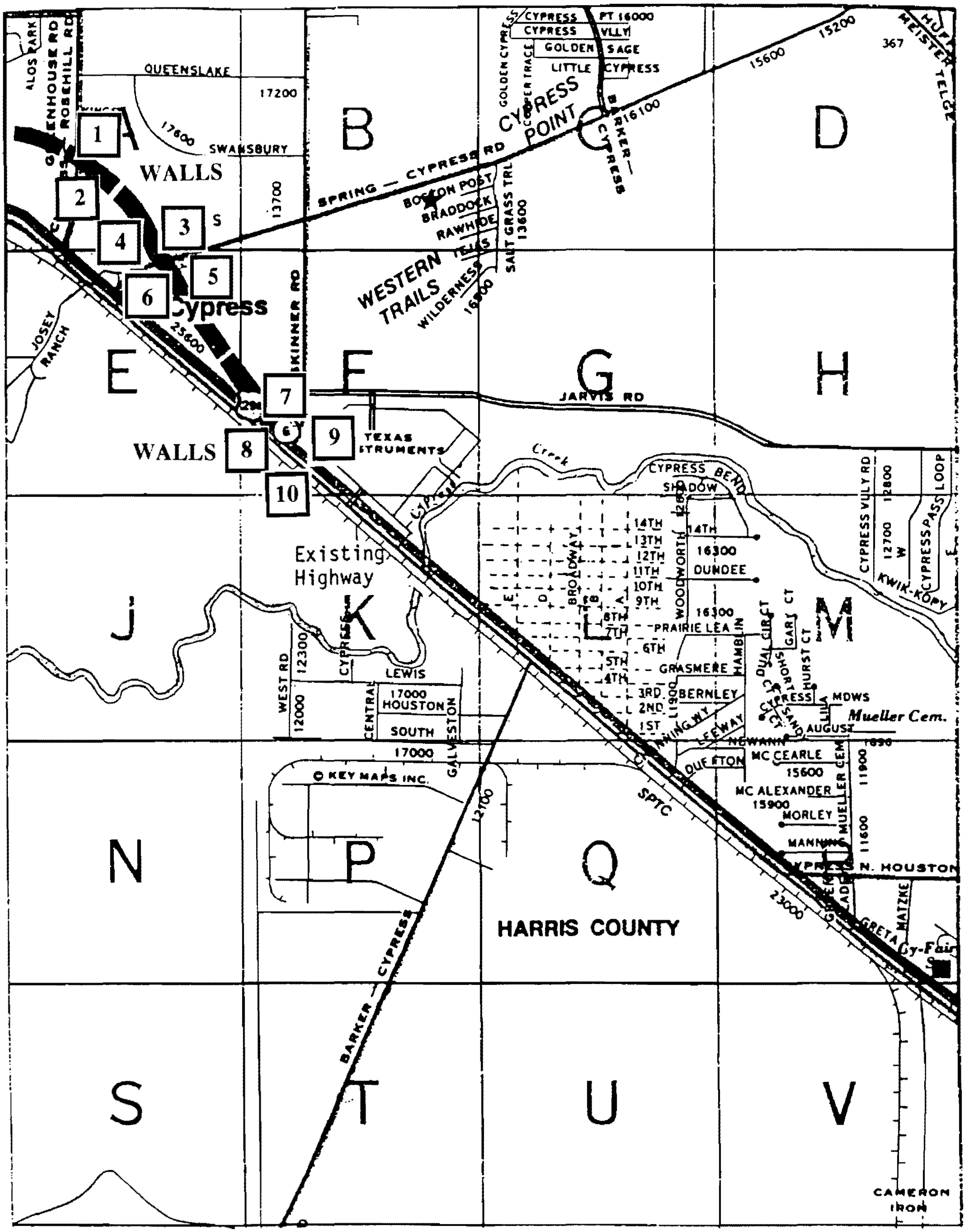


Figure 2.1 Location map of field construction

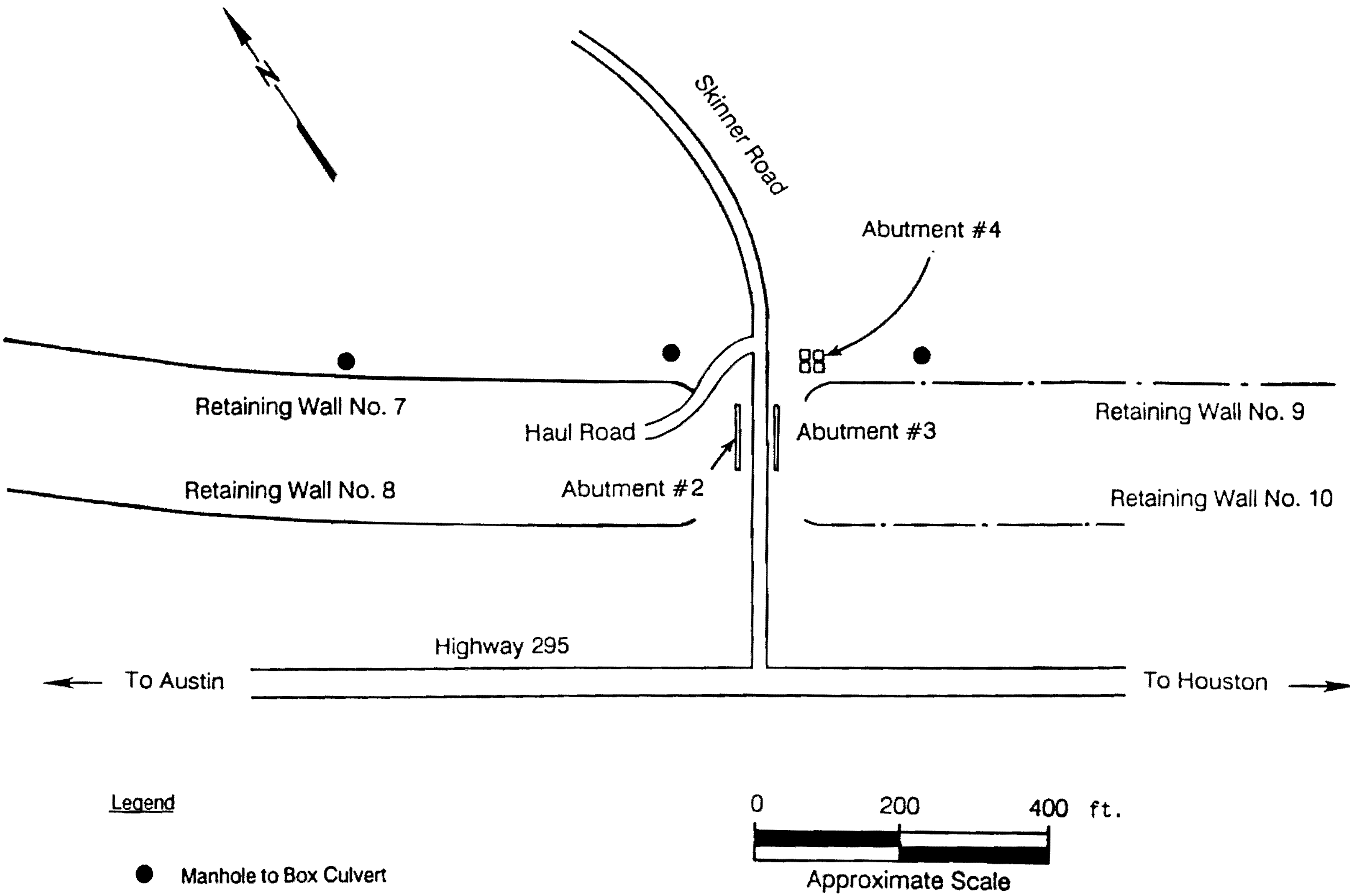


Figure 2.2 Site layout

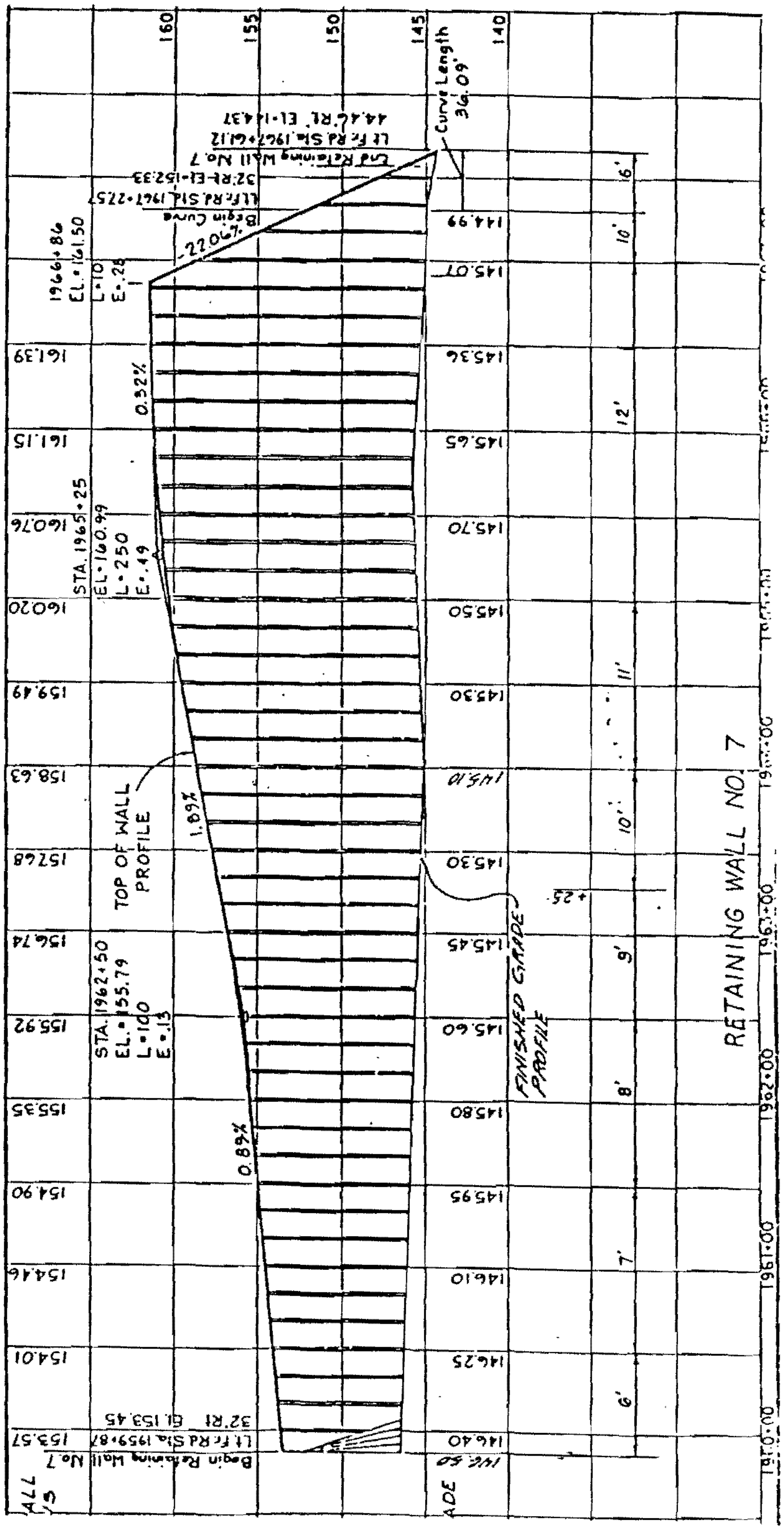


Figure 2.3 Elevation of wall 7

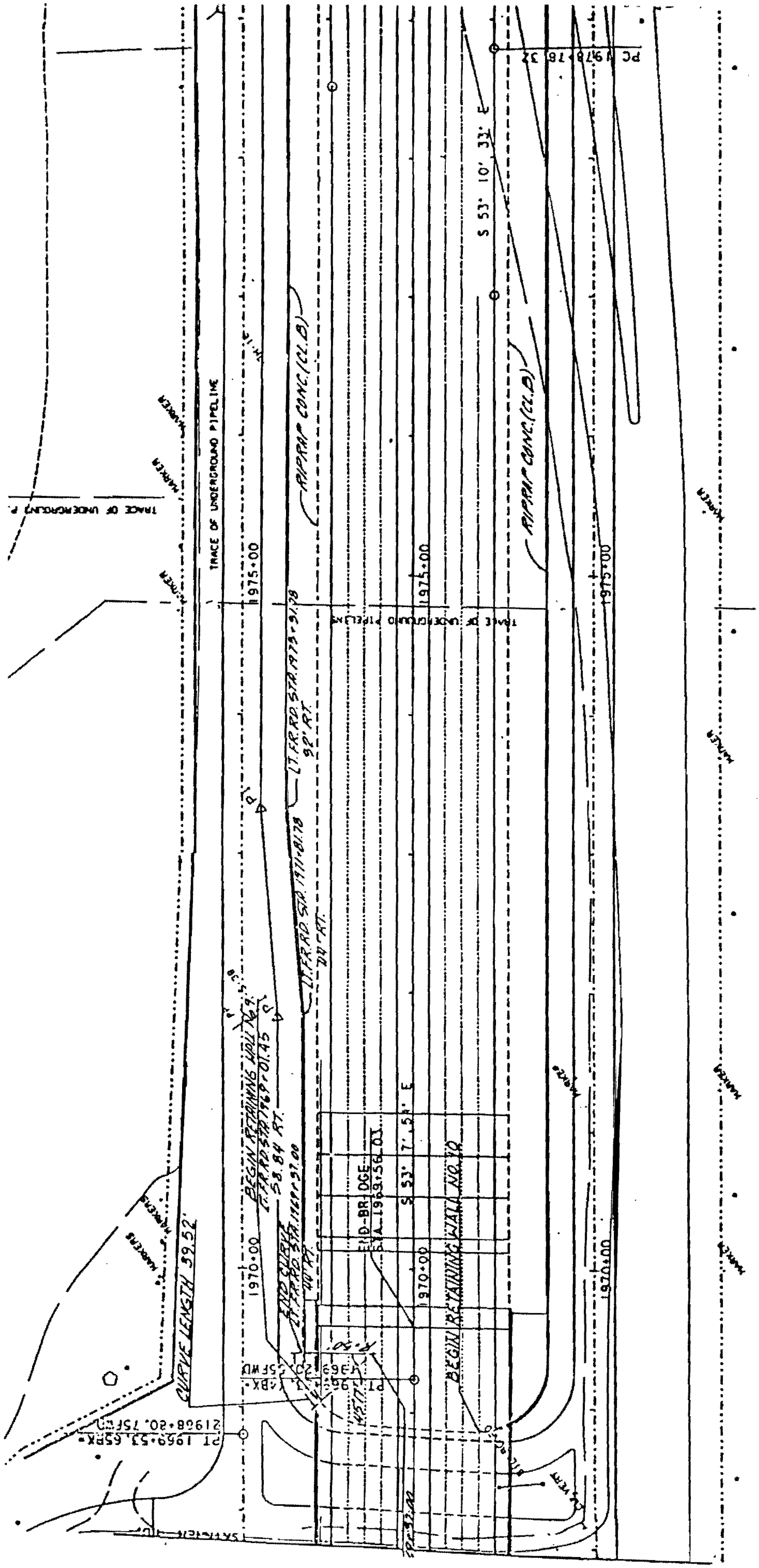


Figure 2.4 Plan of wall 9

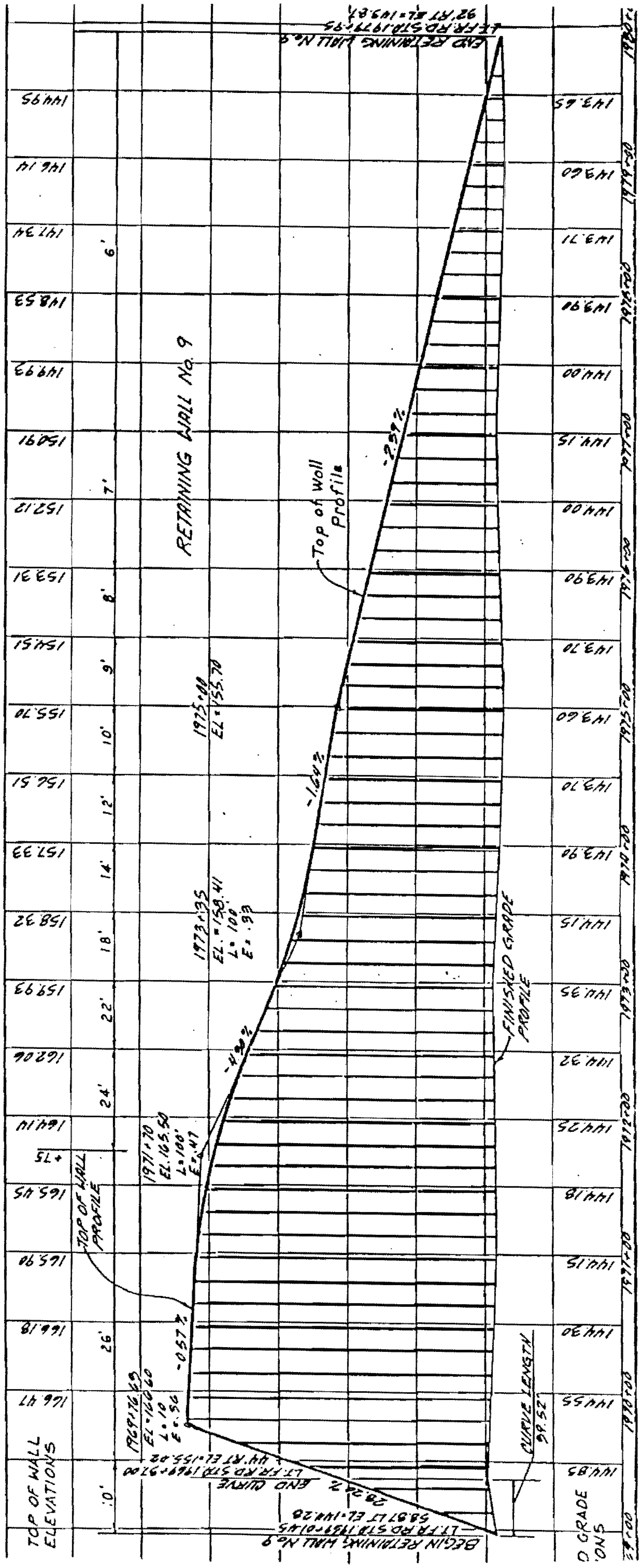
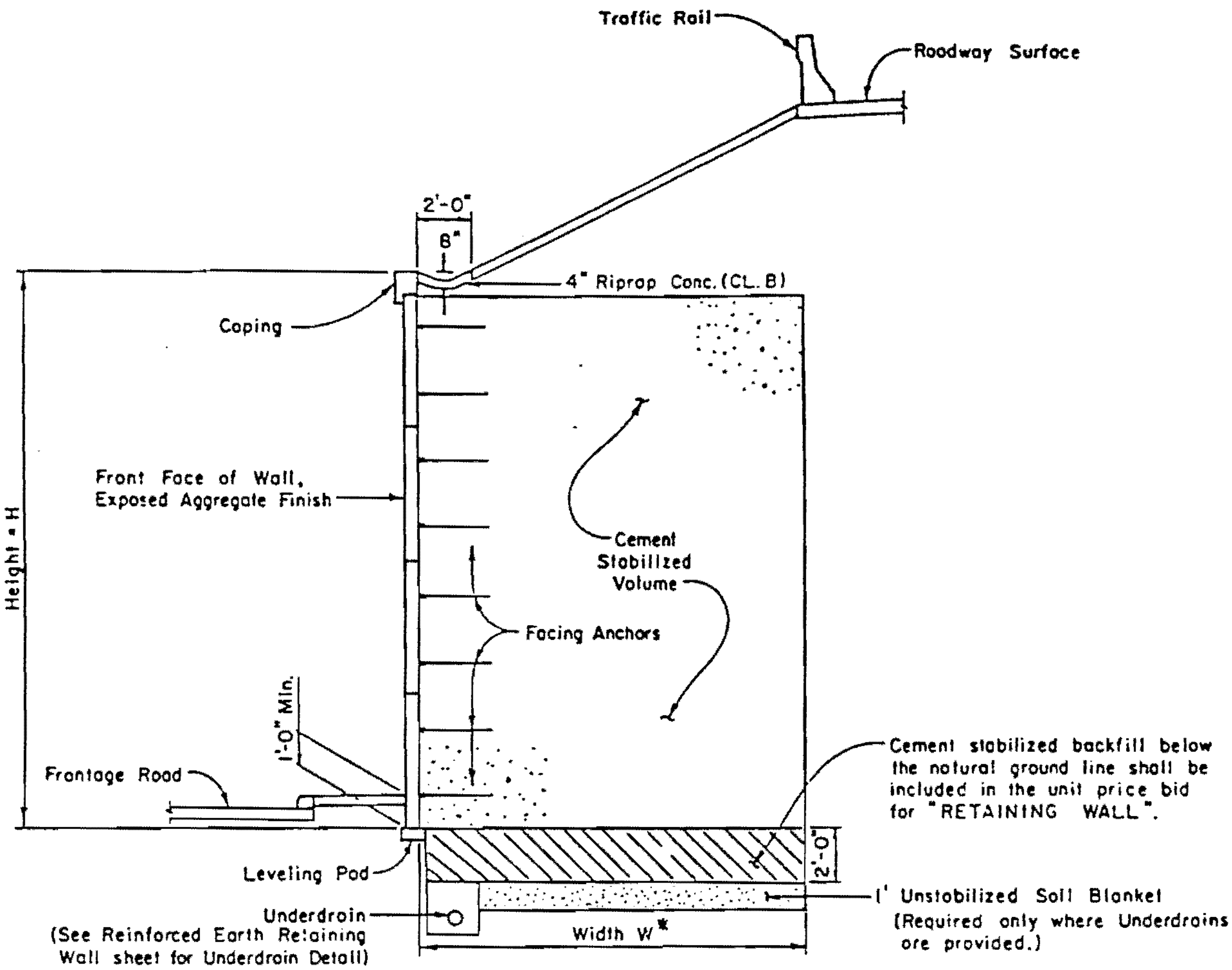
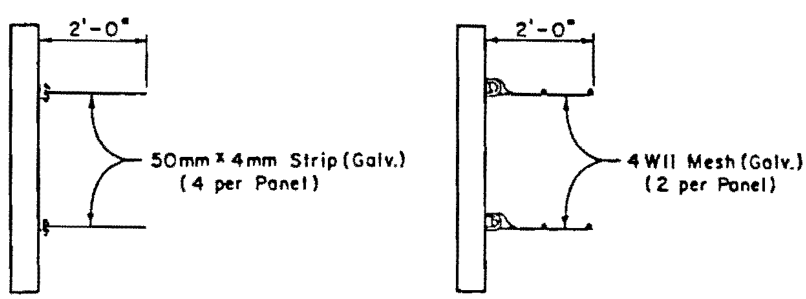


Figure 2.5 Elevation of wall 9



TYPICAL SECTION
(Wall at bottom of slope)

*See Retaining Wall Layout for width of Cement Stabilized Soil Block.



Reinforced Earth Panel

Retained Earth Panel

FACING ANCHOR DETAILS

Figure 2.6 Original wall details

Figure 2.7 Foundation preparation for wall 7

Figure 2.8 Facing panel anchors at wall 7

2.3 FIELD PROBLEMS AT TEST SITE

On the evening of October 31, 1989, several inches of rain fell on the site, and the next morning considerable distress was noted at the experimental wall section (wall 7), which at this stage had reached a height of 3 m (10 feet). The distress appeared to be associated primarily with massive settlement of a foot or more, at what appeared to be the location of an underground storm culvert. By all accounts, this had been loosely backfilled with clayey fill (possibly even sluiced into position) with no engineering control. Additionally, site drainage was also a problem at wall 7. This was true for both the base and top of the wall, causing poor drainage of the stabilized fill and increasing water pressures behind the face panels. There appeared to be considerable evidence of surface run-off over the top of the stabilized fill and down behind the face panels. Softening of the foundation at the toe therefore probably took place, with associated bulging outward of the wall face. Similar problems were also observed at the other conventional retained earth walls.

2.4 REMEDIAL WORK AT THE EXPERIMENTAL RETAINING WALLS

The foundation for one of the experimental walls (wall 7), as well as some of the conventional reinforced earth walls was subsequently improved through the use of remedial piling through the foundation soil. Remedial pilings were used underneath walls 2, 4, 5, 7 and 8. Figure 2.9 indicates the modified design for the walls, in which 800 kN (90 ton) precast concrete piles were installed 12 m (40 feet) below the existing grade level in two offset rows. The innermost row was installed at a 5 vertical to 1 horizontal batter. A photograph of the finished pile foundation is shown in Figure 2.10.

For a variety of reasons, foundation piling was not used under the final construction of wall 9 (or under conventional walls 1, 3, 6 and 10). This was because the foundation conditions were felt to be slightly better here and also because it was desirable to build at least one experimental wall on original soil. However, as a precaution, the design of wall 9 was altered to include an additional 2 m (6 feet) of stabilized fill underneath the base of the wall. This provided a substantial bedding layer to accommodate subsurface settlement. Figure 2.11 shows the modified design detail that was used to construct wall 9. Figure 2.12 shows a photograph of the foundation of wall 9 after preparation, with the stabilized fill extending well under the levelling pad.

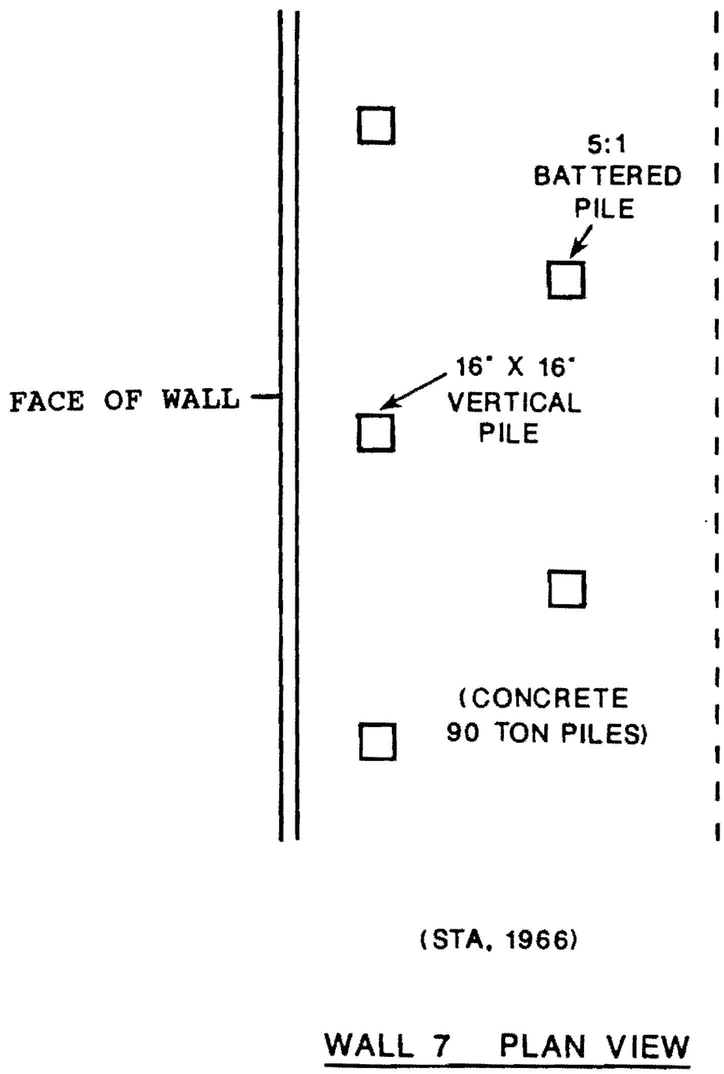
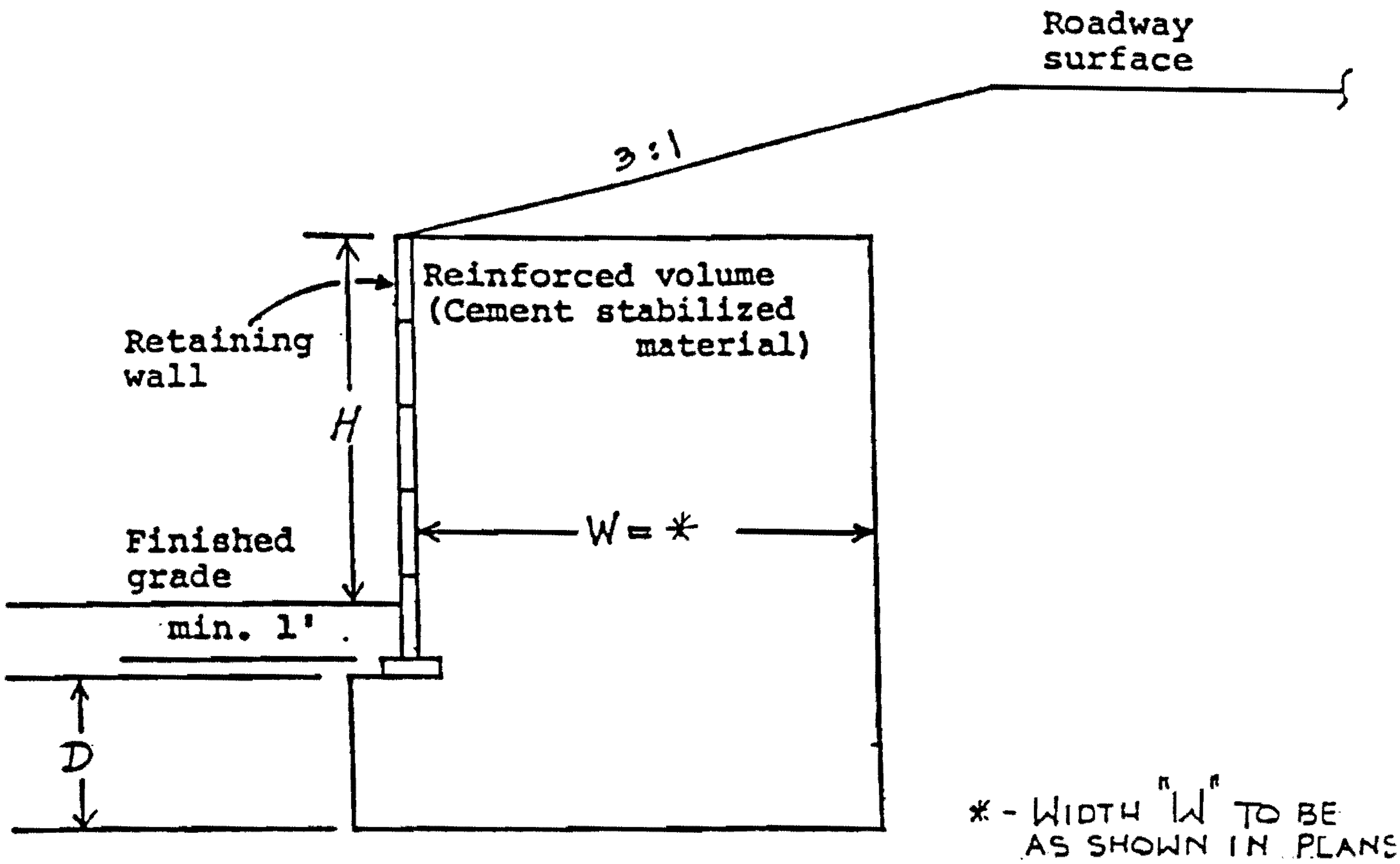


Figure 2.9 Plan of piled foundation for wall 7

Figure 2.10 Photograph of piled foundation for wall 7

Figure 2.12 Foundation preparation for wall 9



Not to scale

Section view

Sta.1969+50 to Sta.1971+80 : D = 6 feet
Sta.1971+80 to Sta.1972+70 : D = 5 feet
Sta.1972+70 to Sta.1975+80 : D = 4 feet
Sta.1975+80 to Sta.1976+20 : D = 3 feet
Sta.1976+20 to Sta.1979+95 : D = 1 feet

Figure 2.11 Modified design for wall 9

3. MONITORING PROGRAM

3.1 GENERAL

To determine the in situ performance of walls 9 and 7, a monitoring study of the site was undertaken. The goals of the study were to quantify and assess the behavior of the walls and to provide a history of performance of the experimental design. To accomplish this, various instrumentation was installed behind the two walls in both the stabilized fill and the soil. The instruments used were inclinometers, both vertical and horizontal, earth pressure cells, and load cells.

The construction of wall 9 and reconstruction of wall 7 occurred in stages and was performed during late 1990 and early 1991. After the remedial piling was placed beneath wall 7, the leveling pads for the two walls were installed. The construction of the two walls then proceeded with lifts of stabilized fill being placed behind the leveling pads. Simultaneously, the front panels of the walls were put into place on top of the leveling pads with anchors embedded in the stabilized fill. Props were used to keep the front panels in place and were left for a reasonable amount of time to let the stabilized fill cure and the bond strength of the anchors develop. As the lifts of stabilized fill and front panels were put into place, the field instruments were also installed.

3.2 INSTRUMENTATION AND INSTALLATION

During the construction of wall 9 and the reconstruction of wall 7, field instrumentation was installed to determine the movements and stresses in the soil and the stabilized fill, along with the forces applied on certain wall anchors. The movements were measured using vertical and horizontal inclinometers. The load cells measured the connection load, while the earth pressure cells measured the vertical and horizontal stresses in the soil. Figures 3.1 to 3.4 illustrate cross-sectional and elevation views of walls 9 and 7 indicating the instrumentation used and their locations.

3.3 INCLINOMETERS

(a) Vertical Inclinometers

A Sinco Digitilt Inclinometer, Model 50309-M with Model 50325-M sensor, and a Sinco Digital Indicator, Model 50309, were used to determine the lateral deflections of the vertical inclinometers. The inclinometer sensor was operated in a permanently installed vertical casing. The inclinometer casing consisted of 70 mm (2.75 in) OD plastic pipe that was capped at one end and contained internal longitudinal grooves. The sensor was placed in the casing ensuring that the wheels of the sensor lined up with the grooves and was lowered into the casing by an interconnecting cable. The principle of operation is shown in Figure 3.5. Readings were taken every 0.5 m (1.64 ft) to determine the deflection of the casing in the direction of the grooves. The readings were recorded on the indicator and transferred to data sheets manually.

At wall 9 two vertical inclinometers were initially installed, one in the stabilized fill

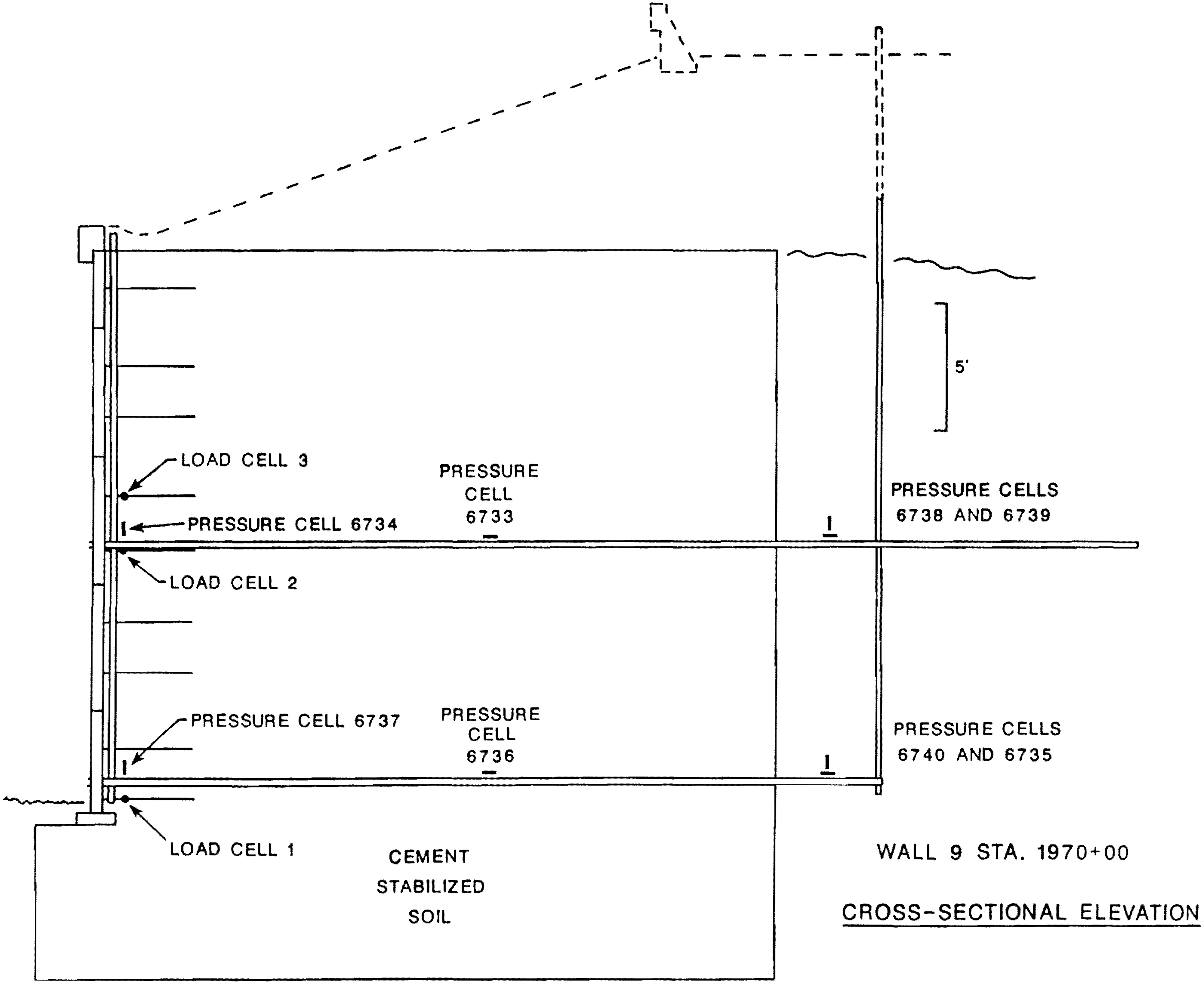


Figure 3.1 Cross-section of wall 9

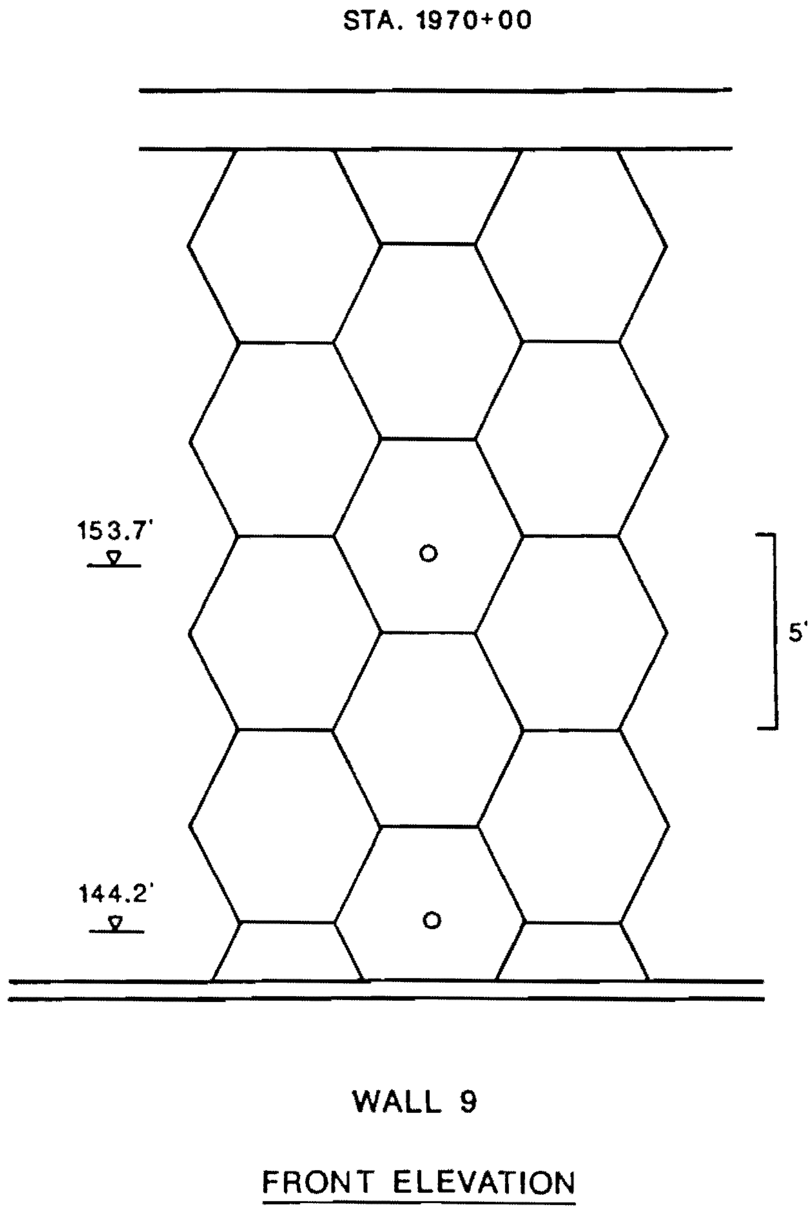


Figure 3.2 Front elevation of wall 9

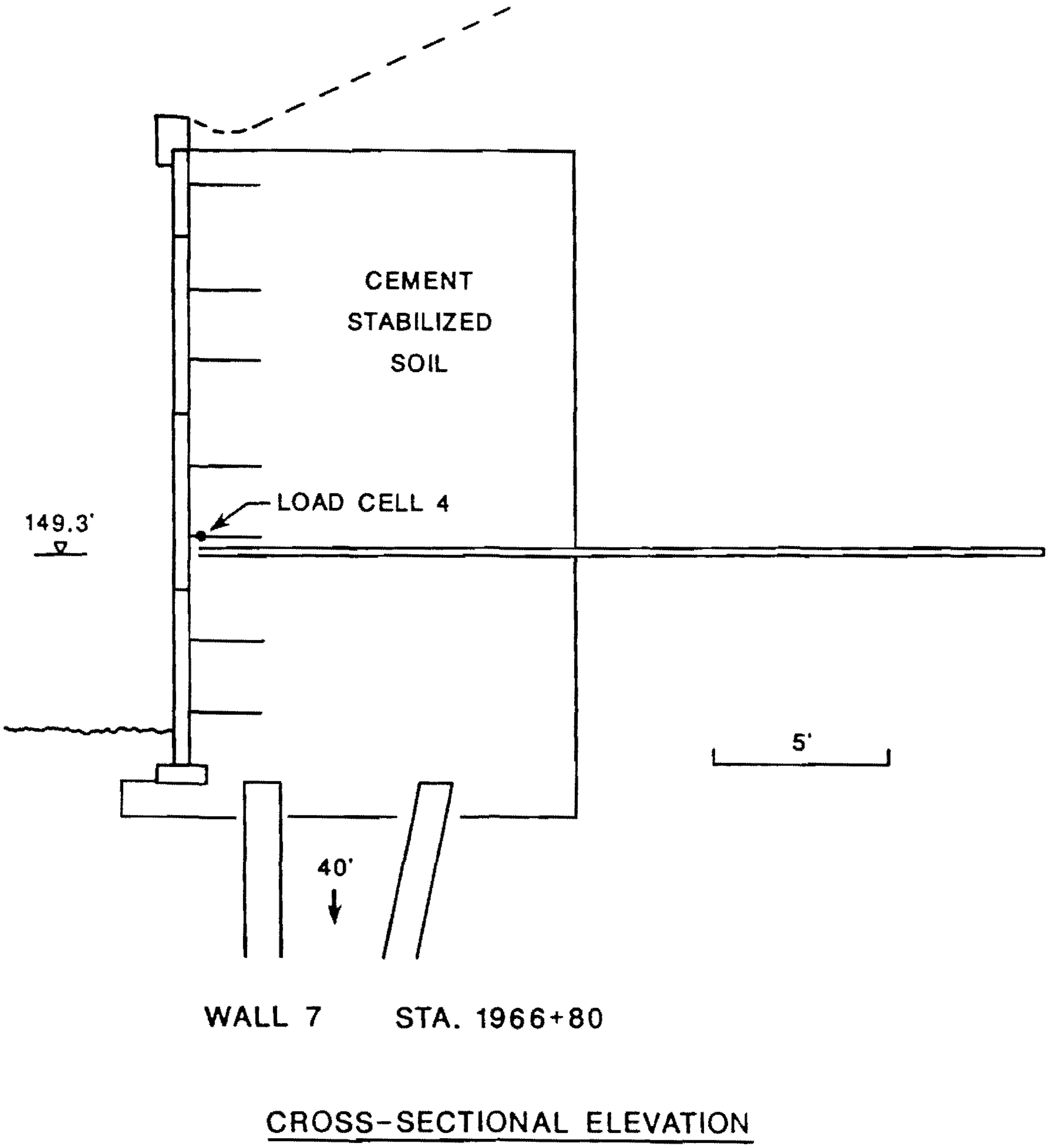


Figure 3.3 Cross-section of wall 7

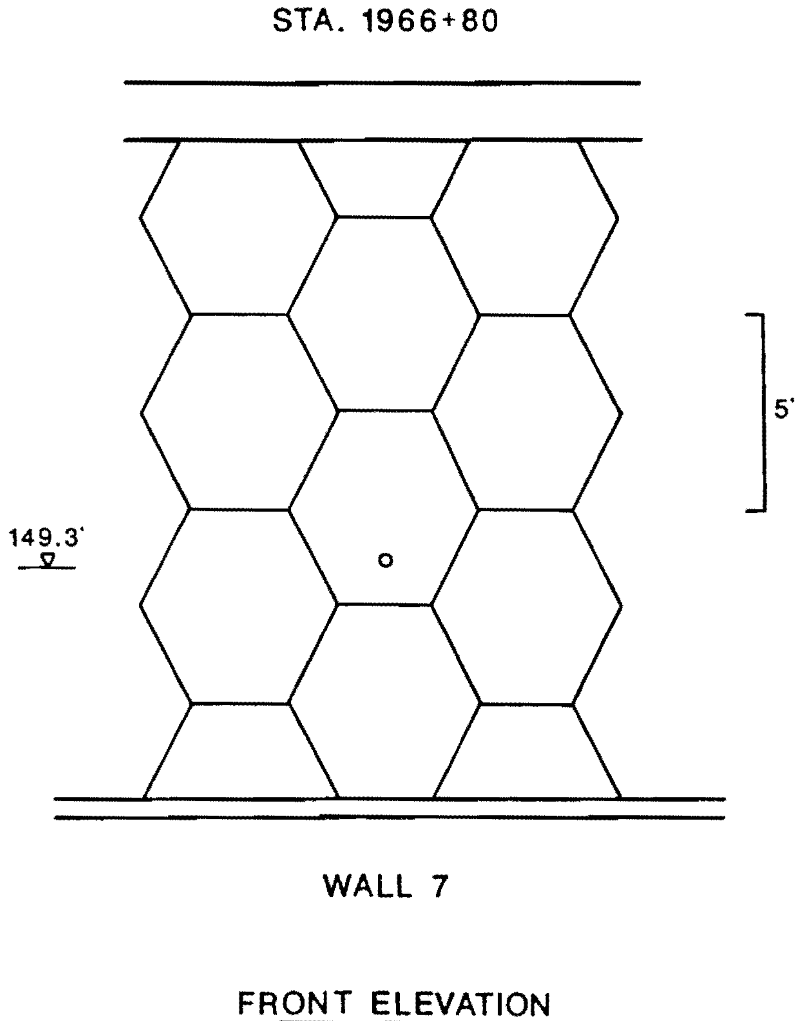


Figure 3.4 Front elevation of wall 7

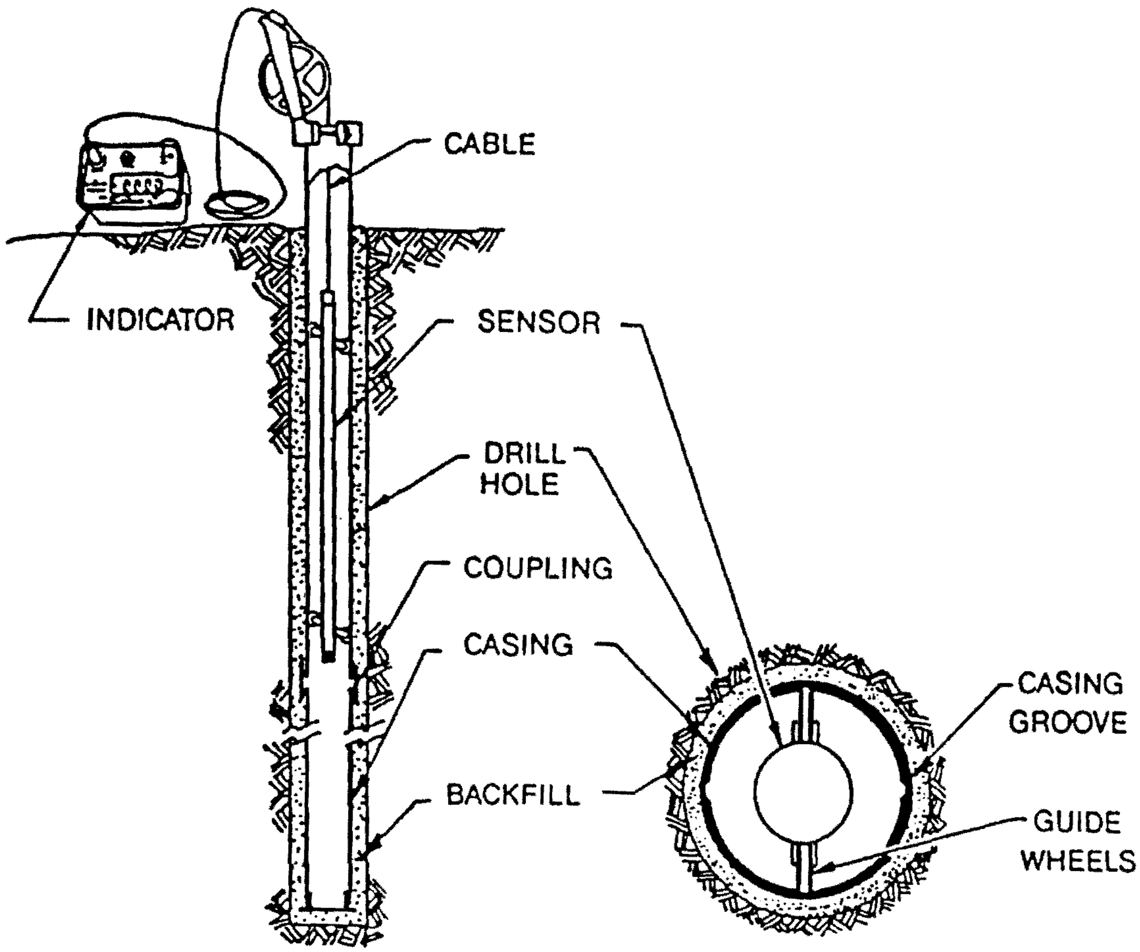


Figure 3.5 Principle of operation of vertical inclinometer

and the other in the soil (Figure 3.1), although the vertical inclinometer in the soil was later damaged and useful readings were never obtained from this unit. The capped end of the inclinometer casings were installed to a depth equivalent to the existing ground surface. The grooves of the casings were aligned parallel (i.e., N-S) and perpendicular (i.e., E-W) to the wall. Stabilized fill and soil were placed in lifts around the inclinometer casings, depending upon their location, to provide support. Figure 3.6 shows a photograph of the installation of the vertical inclinometers.

(b) Horizontal Inclinometers

A Sinco Horizontal Digitilt Sensor, Model 50329, and a Sinco Digital Indicator, Model 50309, were used to determine the vertical movement within the stabilized fill and soil. These were run inside permanently installed horizontal inclinometer casings with a pull cable arrangement. The inclinometer casings consisted of a grooved 85 mm (3.34 in) OD plastic pipe, while the cable wire was stainless steel. The sensor, which was linked to the digital indicator by an interconnecting cable, was attached to the pull cable and placed in the casing. The pull cable was then used to advance the sensor, with readings being taken every 0.5 m (1.64 ft) to determine the deflection of the casing. The principle of operation is shown in Figure 3.7.

Since the casing was only accessible from one end, a dead end pulley assembly was used to return the cable. For this a second plastic pipe of 20 mm (0.75 in) OD was attached parallel to the casing to house the returning cable. At the inaccessible end the two pipes were connected to a steel box, which housed the pulley. Consequently, the pull cable ran through the inclinometer casing, ran around the pulley, and returned in the attached plastic pipe. The separate pipe was used to avoid the risk of the probe or sensor becoming entangled with the return wire. Figure 3.8 shows a close-up of the end of a horizontal inclinometer installation, with the dead end pulley system.

Two horizontal inclinometer systems were installed in wall 9 and one in wall 7 (Figures 3.1 and 3.3). During installation of each of the inclinometers a small channel was dug at the appropriate elevation for the inclinometer casing and the dead end pulley assembly. The dead end pulley assembly was placed in the soil (Figure 3.8). The grooves of the inclinometer casing were placed vertically (ie. D-U) and parallel to the wall (N-S), so that the relative and absolute settlement/vertical heave could be determined. Sand and then stabilized fill were placed over the inclinometers, in the appropriate locations, to permanently embed them.

3.4 LOAD CELLS

Four electrical resistance load cells were used, three on wall 9 and one on wall 7, to determine the forces on the wall anchors. The load cells consisted of two electrical resistance strain gauges, bonded to the outer periphery of a steel cylinder. The strain gauges were oriented parallel and perpendicular to the axis of the cylinder to measure the axial and tangential strains. The strain gauges were connected to form a single full bridge network, which reduced errors by integrating the individual strain gauge outputs. The strain gauges

Figure 3.6 Inclinometer installation at half height

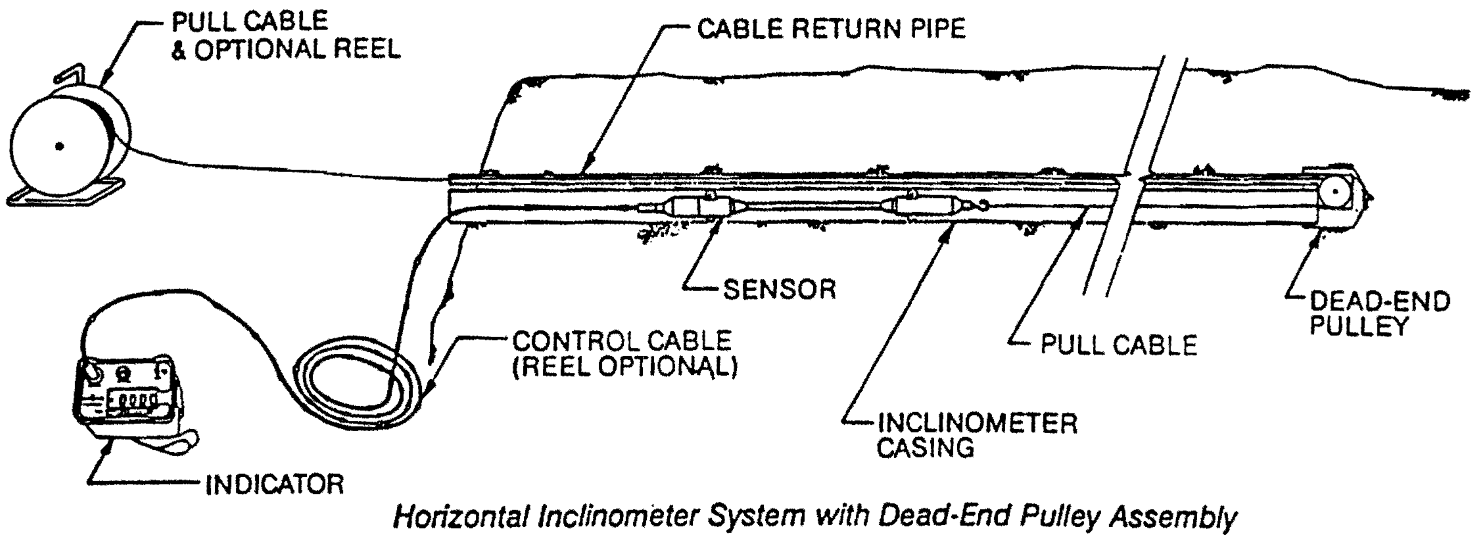


Figure 3.7 Principle of operation of horizontal inclinometer

Figure 3.8 Photograph of pressure cells and return pulley

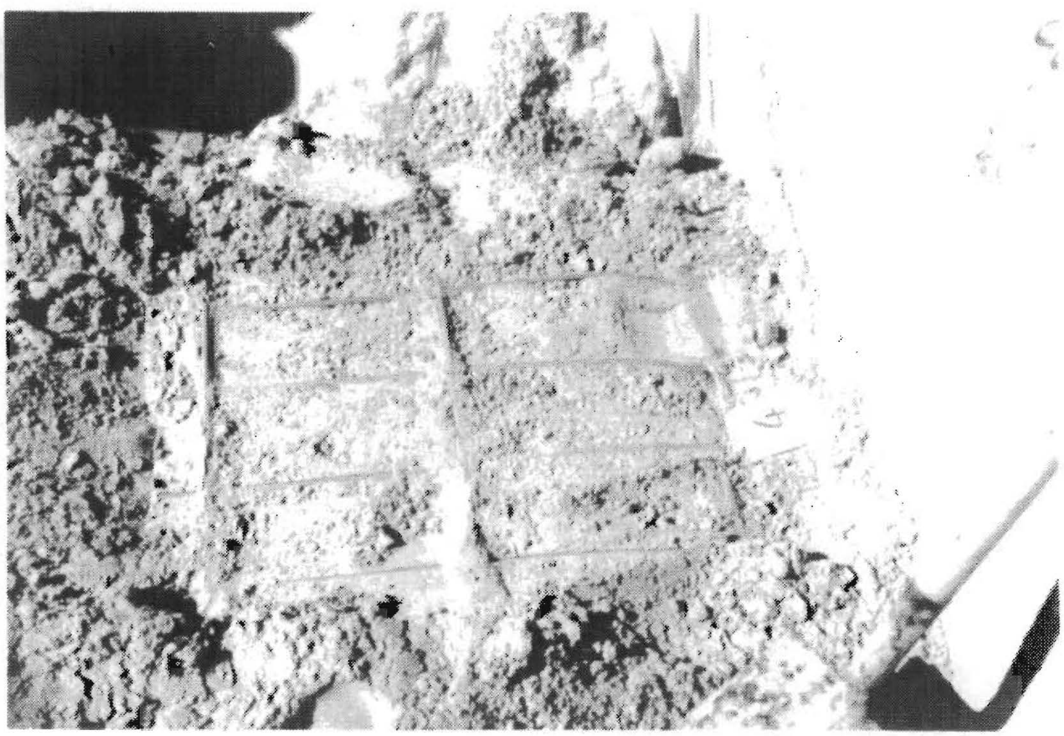


Figure 3.9 Photograph of anchor load cells

and full bridge were wrapped in electrical tape around the steel cylinder. The entire assembly was placed inside a larger diameter cylinder. This was done to protect the gauges from mechanical damage. A waterproofing compound was used to fill the void between the cylinders and to protect the gauges from water damage.

Before installation, each load cell was tested in the laboratory to determine the calibration curve for the resistance measured and the corresponding force applied. In the field a 12 volt battery supplied the current to the strain gauges. The resistance change of the strain gauges due to deformation was read in millivolts and measured using a volt meter. The resistance change was then converted to the corresponding force using the calibration curve for the individual load cell.

The installation of the load cells in the field consisted of rigidly attaching the individual load cells to the wall anchors. For convenience the load cells were mounted to the wall anchors adjacent to the locations of the horizontal inclinometer casings, so the wires for the load cells could be run parallel to the casings. Stabilized fill was then placed around the anchors and the load cells, so that when the fill cured, it would bond to both of them. Figures 3.1 and 3.3 illustrate the locations of the load cells for the two walls. A photograph of an installed load cell is shown in Figure 3.9.

3.5 EARTH PRESSURE CELLS

A total of eight earth pressure cells were installed in both the stabilized fill and the soil behind wall 9. The earth pressure cells were made by Kulite Sensors Limited and were of the diaphragm type. This consisted of a circular disk in which the upper surface has a flexible circular membrane on it that deflects under the external soil pressure. The deflection of the membrane is measured by the strain gauges bonded to the lower surface of the membrane. These particular devices incorporated a full bridge of semi-conductor strain gauges diffused into a silicon diaphragm, for high sensitivity, and required only an input voltage to produce an appropriate output signal. The resistance changes measured by the strain gauges were recorded on a Sinco Digital Indicator, Model 50309, and were converted to stress using the factory provided calibration curve.

The earth pressure cells were installed at the same time as the horizontal inclinometer casings. The wires from the earth pressure cells were run along the inclinometer casings. The locations and orientations of the earth pressure cells were selected to try and determine the stress state in the soil, the vertical stress in the stabilized fill and the lateral stress on the wall. Consequently, the earth pressure cells were oriented both horizontally and vertically, with the vertical earth pressure cells having their axes parallel to the wall. Figure 3.1 illustrates the locations and orientations of the earth pressure cells. A photograph of an installed earth pressure cell is shown in Figure 3.8.

3.6 SETTLEMENT POINTS

Optical surveying points were located at various intervals on the levelling pad along both the experimental and conventional retaining walls. These points were used to monitor the magnitude and rate of settlement for the walls.

4. FIELD BEHAVIOR AND DATA REDUCTION

4.1 GENERAL

The instrumentation used for monitoring the performance of walls 7 and 9 consisted of inclinometers, load cells, earth pressure cells and optical settlement points. The installation, data acquisition and data reduction are discussed in the following subsections.

4.2 INCLINOMETERS

A total of four inclinometers were installed behind wall 9, and one inclinometer behind wall 7. However, the inner vertical inclinometer in the soil behind wall 9 was permanently damaged when a construction road was built adjacent to it. Table 4.1 indicates the dates of the initial readings of the inclinometers, as well as the directions of the fixed reference orientations for the positive deflections. Each time the inclinometer was surveyed the spring loaded wheels were first set in the grooves of the casing parallel to the reference direction and readings were taken every 0.5 m (1.6410 ft). For completeness, the sensor was rotated 180 degrees and readings were then taken again. This is so that the algebraic difference of the two readings is equivalent to the average of the two sets. This eliminates the zero drift between the readings. For each of the positive inclinometer directions, the algebraic difference of the initial reading is subtracted from each of the differences of the subsequent readings to yield the change in slope of the inclinometer casing.

To obtain the cumulative slope change, the slope changes at each interval for an individual reading were summed starting from the reference end of the casing (i.e., the bottom of the casing for the vertical casings and the face of the wall for the horizontal casings). The cumulative slope change values at each interval were then multiplied by the instrument scale factor to yield the deflection. In the case of the vertical inclinometers, the deflection is the absolute deflection; however, for the horizontal inclinometers this yields the relative deflection. To obtain the absolute deflection, surface surveying measurements of the reference end of the inclinometer casing (i.e., at the wall) must be added to the relative deflection. Using the above method for reducing the data ensures that the positive deflection is parallel to the reference direction.

Figure 4.1 shows a photograph of horizontal inclinometer measurements being taken on wall 9.

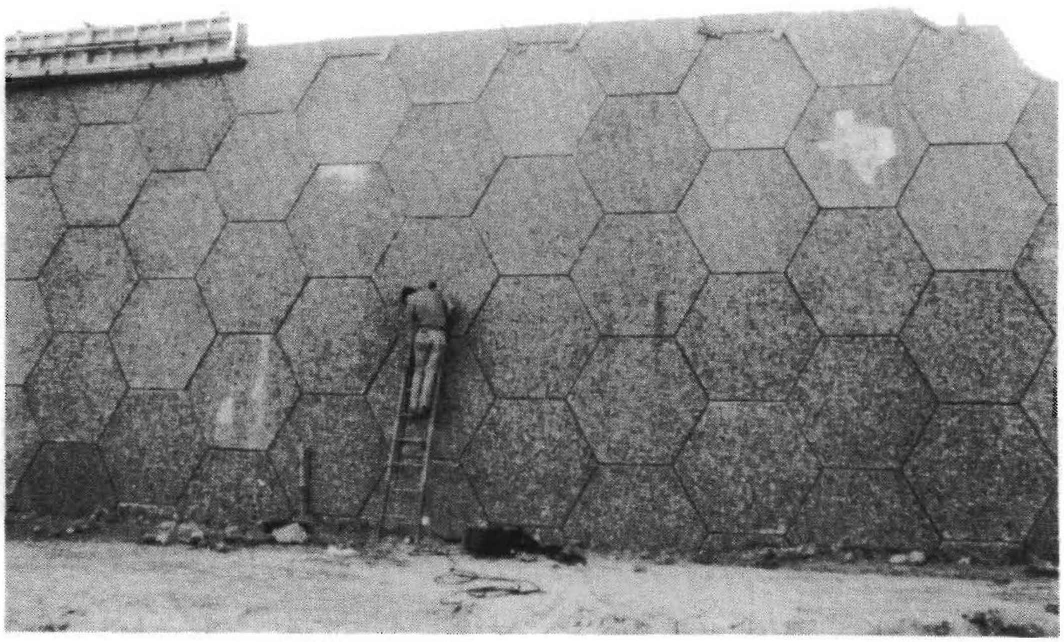


Figure 4.1 Inclinometer measurements in progress

TABLE 4.1 - INCLINOMETER ORIENTATIONS

Instrument	Date of Initial Reading	Positive Reference Direction	Positive Reference Direction
Horiz. 1 - lower	Dec. 20, 1990	Down	North (along the wall)
Horiz. 2 - upper	Jan. 11, 1991	Down	North (along the wall)
Vert. 1 - outer	Dec. 20, 1990	East (outward)	North (along the wall)
Vert. 2 - inner	DAMAGED	NEVER USED	
Horiz. 3 - wall 7	Apr. 4, 1991	Down	North (along the wall)

4.3 LOAD CELLS

The load cells installed on the anchors behind the walls worked initially, but then progressively shorted out over a period of about two years. This was presumed to be due to moisture or water effects. To determine the force on the anchors the resistance change in the strain gauges within the load cells was measured. These readings were converted to a force using the calibration curves.

4.4 EARTH PRESSURE CELLS

The earth pressure cells installed behind wall 9 continued working throughout the project and recorded the stresses in the soil and stabilized fill. The initial resistance changes in the strain gauges within each of the pressure cells were recorded on December 11, 1990. Additional readings were taken at select times thereafter. To determine the stress level on each pressure cell, the initial reading was subtracted from each reading taken and then converted to stress using the factory supplied calibration factor.

4.5 SETTLEMENT POINTS

The elevation of the optical surveying points were obtained at various time intervals. These were then subtracted from the original elevations of the points to determine the magnitude of the settlement. The error involved in calculating the settlements is $\pm$ 8mm x $(km)^{1/2}$ (Dunnicliff, 1988), where the $(km)^{1/2}$ is the square root of the horizontal distance in km's from the sighting point to the optical surveying point. The maximum error for the settlement measurements would be $\pm$ 2mm for a sighting distance of ~63 km.

It should be noted that the surveying measurements were taken for as long a period of time as possible. When the levelling pad was covered by a concrete strip the measurements ceased because an accurate thickness of the concrete strip was not known. The exception to this was the horizontal inclinometer holes for walls 7 and 9, which were surveyed each time an inclinometer reading was taken throughout the study.

5. OBSERVED PERFORMANCE OF THE WALLS

5.1 GENERAL

To determine the in situ performance of walls 7 and 9, a monitoring study of the site was undertaken. The goals of the study were to judge the behavior of the walls and to provide a history of performance of the stabilized soil. To accomplish this, various instruments were monitored over time. The instruments used were surveying points, inclinometers, earth pressure cells and load cells.

5.2 SETTLEMENT MEASUREMENTS

To determine the influence of settlement on the observed performance of the experimental walls, settlement measurements were taken over a period of time for walls 4, 5, 7, 8, 9 and 10. These measurements allow for a comparison to be made of the experimental walls (walls 7 and 9) against the conventional walls (walls 4, 5, 8 and 10). In addition walls 4, 5, 7 and 8 all have piling beneath their foundations, whereas walls 9 and 10 do not. Thus, a comparison of the effectiveness of the piling beneath the foundations of the walls can also be made.

(a) Results for Wall 4

Figure 5.1 illustrates some of the various stations where the optical surveying points were located along the wall. Optical surveying measurements were taken periodically over a 189 day period to determine the settlement of the various stations with time. Table 5.1 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.2 shows representative settlement versus time plots for a few stations for wall 4.

From the data the following can be observed. The magnitude of the settlement ranges from 6 mm to 27 mm and shows a periodic pattern along the length of the wall. The maximum settlement occurs at stations 1921+00 and 1925+00. The maximum differential settlement along the wall is 21 mm and occurs between stations 1925+50 and 1926+00. Since this occurs over an approximately 15 m length then the angular distortion is 0.0014. While this is not high enough to cause fracture in concrete it is still significant (Das, 1990; Bowles, 1982).

From Table 5.1 and Figure 5.2, the rate of settlement ranges from 0.05 - 0.26 mm/day over the last 58 days of measurements. While these rates are not significantly high, they do indicate that settlement was not necessarily complete at the time of the last reading.

(b) Results for Wall 5

Optical surveying measurements were taken periodically over a 252 day period to determine the settlement of the various stations with time. Figure 5.3 illustrates some of the various stations where the optical surveying points were located along the wall. Table

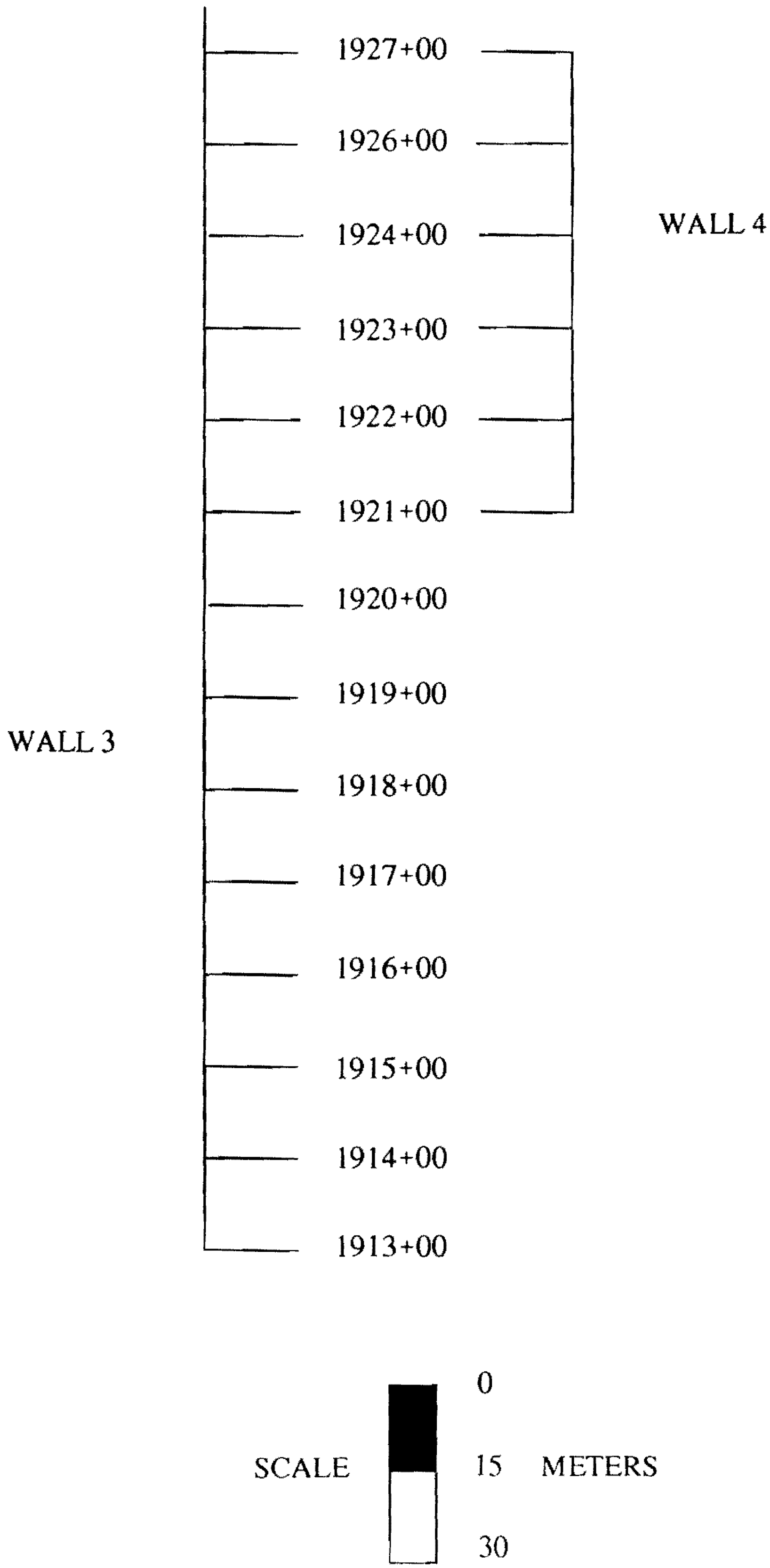


Figure 5.1 Surveying stations for wall 4

TABLE 5.1 - SETTLEMENT OF WALL 4 AT VARIOUS STATIONS

		TIME, DAYS	
STATION #	0	131	189
1927+00	0.000 mm	6.096 mm	15.240 mm
1926+50	0.000 mm	6.096 mm	12.192 mm
1926+00	0.000 mm	3.048 mm	6.096 mm
1925+50	0.000 mm	21.336 mm	27.432 mm
1925+00	0.000 mm	9.144 mm	18.288 mm
1924+50	0.000 mm	9.144 mm	15.240 mm
1924+00	0.000 mm	0.000 mm	6.096 mm
1923+50	0.000 mm	6.096 mm	9.144 mm
1923+00	0.000 mm	9.144 mm	15.240 mm
1922+50	0.000 mm	6.096 mm	21.336 mm
1922+00	0.000 mm	9.144 mm	21.336 mm
1921+50	0.000 mm	15.240 mm	24.384 mm
1921+00	0.000 mm	12.192 mm	27.432 mm

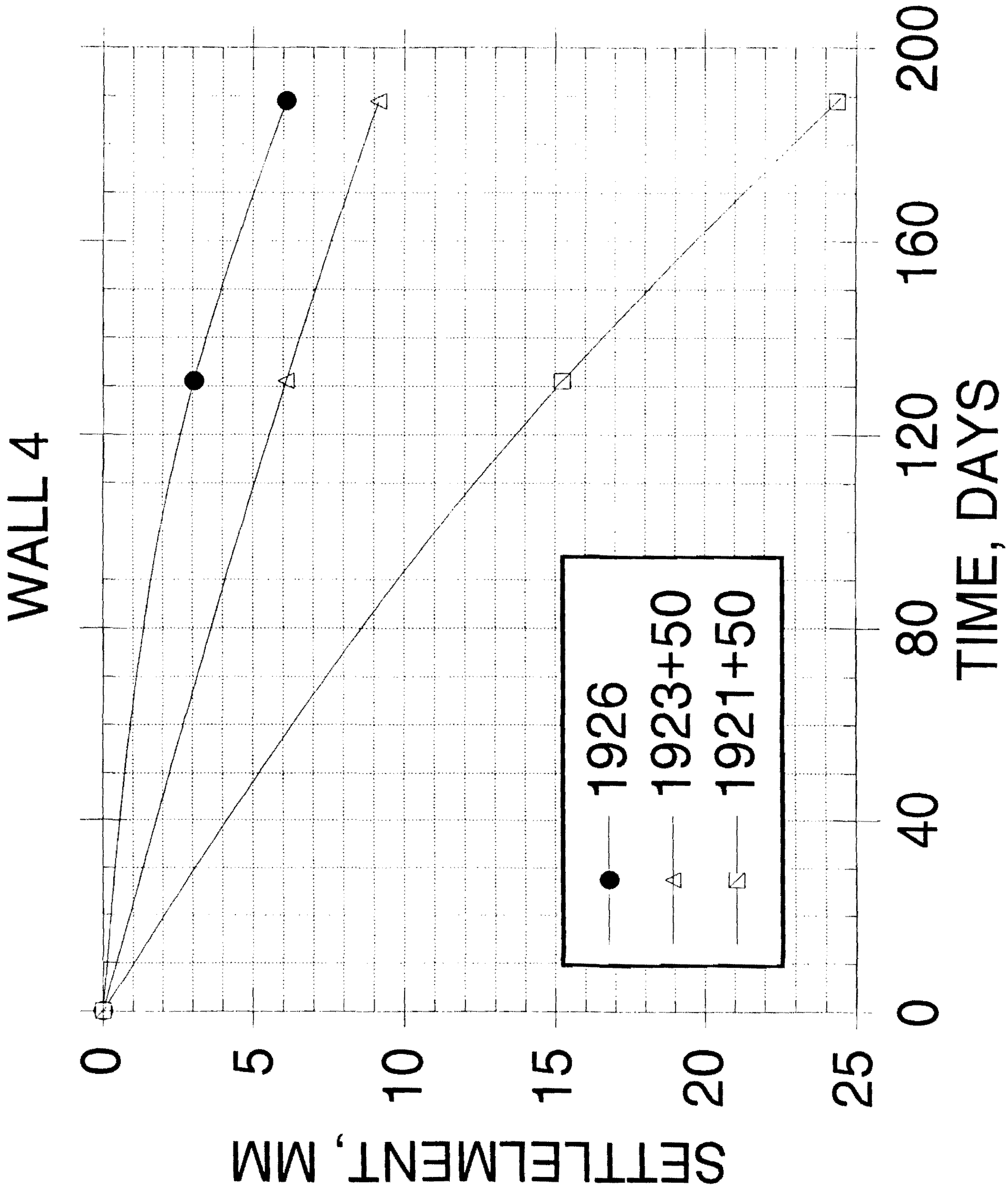


Figure 5.2 Settlement versus time for wall 4

5.2 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.4 shows representative settlement versus time plots for a few stations for wall 5.

From the data the following can be observed. The magnitude of the settlement ranges from 6 mm to 24 mm and shows a fairly uniform pattern along the length of the wall. The maximum settlement occurs at stations 1938+00, 1942+50 and 1943+00. The maximum differential settlement along the wall is 15 mm and occurs between stations 1939+00 and 1939+50. Since this occurs over an approximately 15 m length then the angular distortion is 0.01. This is much higher than for wall 4, and would for instance be high enough to cause fracture in concrete.

From Table 5.2 and Figure 5.4, the rate of settlement ranges from 0.000 - 0.105 mm/day over the last 58 days of measurements. In comparison to wall 4, these settlement rates are lower.

(c) Results for Wall 7

Figure 5.5 illustrates some of the various stations where the optical surveying points were located along the wall. Optical surveying measurements were taken periodically over a 252 day period to determine the settlement of the various stations with time. Table 5.3 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.6 shows representative settlement versus time plots for a few stations for wall 7.

From the data the following can be observed. The magnitude of the settlement ranges from 3 mm to 21 mm. The maximum settlement occurs at stations 1963+25. The maximum differential settlement along the wall is 15 mm and occurs between stations 1963+25 and 1963+75, and occurs over an approximately 15 m length. The angular distortion is 0.01, which is again high.

From Table 5.3 and Figure 5.6, the rate of settlement ranges from 0.0014 - 0.008 mm/day over the last 58 days of measurements. These rates are much slower than either wall 4 or 5.

(d) Results for Wall 8

Optical surveying measurements were taken periodically over a 161 day period to determine the settlement of the various stations with time. Figure 5.5 illustrates some of the various stations where the optical surveying points were located along the wall. Table 5.4 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.7 shows representative settlement versus time plots for a few stations for wall 8.

From the data the following can be observed. The magnitude of the settlement ranges from 21 mm to 48 mm and shows a fairly uniform pattern along the length of the

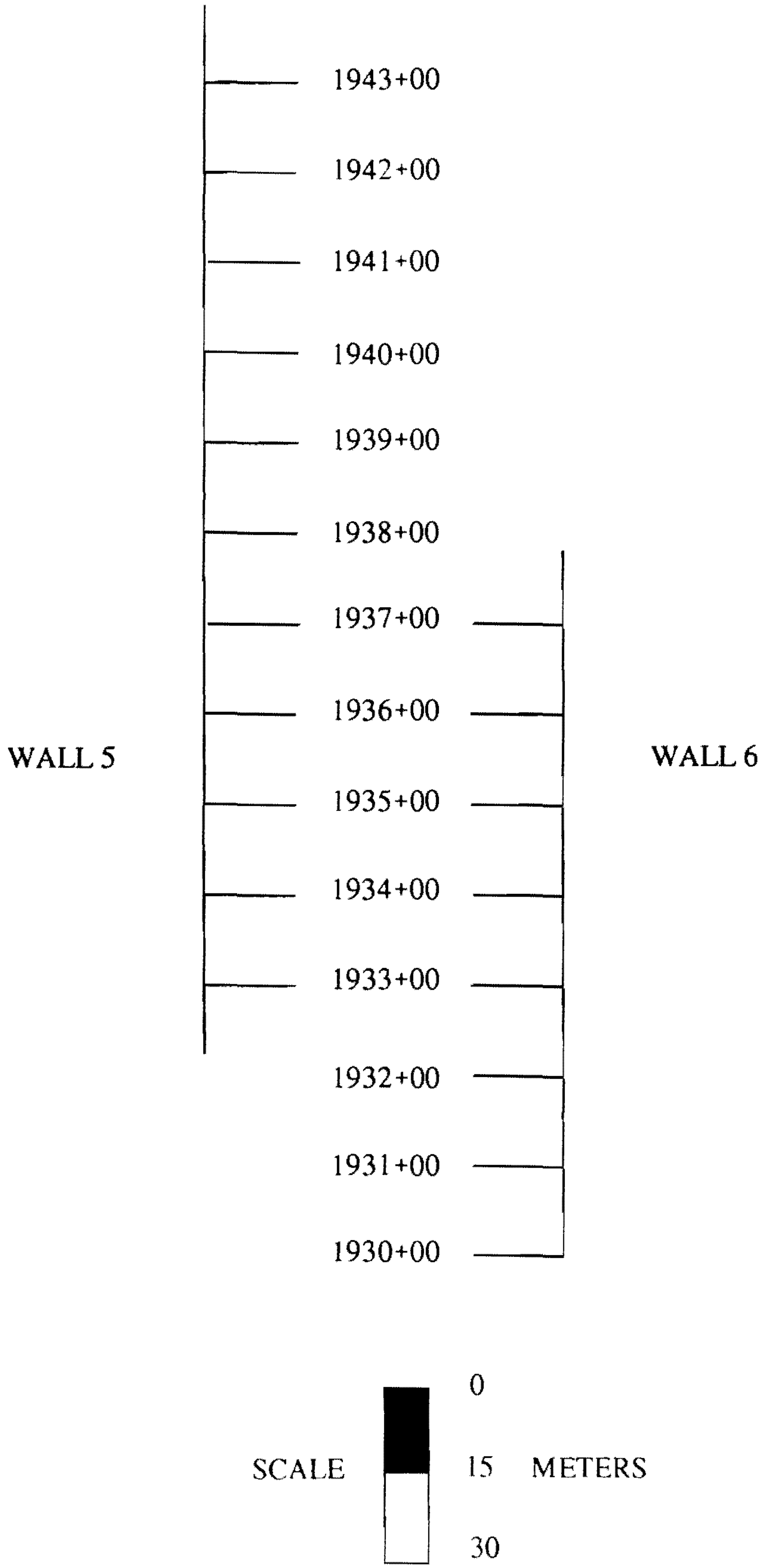


Figure 5.3 Surveying stations for wall 5

TABLE 5.2 - SETTLEMENT OF WALL 5 AT VARIOUS STATIONS

		TIME, DAYS		
STATION #	0	129	194	252
1943+60	0.000 mm	15.240 mm	15.240 mm	15.240 mm
1943+50	0.000 mm	15.240 mm	15.240 mm	15.240 mm
1943+00	0.000 mm	15.240 mm	24.384 mm	24.384 mm
1942+50	0.000 mm	9.144 mm	24.384 mm	24.384 mm
1942+00	0.000 mm	18.288 mm	24.384 mm	22.384 mm
1941+50	0.000 mm	9.144 mm	15.240 mm	15.240 mm
1941+00	0.000 mm	6.096 mm	12.192 mm	15.240 mm
1940+50	0.000 mm	6.096 mm	12.192 mm	15.240 mm
1940+00	0.000 mm	6.096 mm	12.192 mm	18.288 mm
1939+50	0.000 mm	12.192 mm	15.240 mm	21.336 mm
1939+00	0.000 mm	3.048 mm	6.096 mm	6.096 mm
1938+50	0.000 mm	6.096 mm	9.144 mm	12.192 mm
1938+00	0.000 mm	15.240 mm	21.336 mm	24.384 mm
1937+50	0.000 mm	6.096 mm	15.240 mm	15.240 mm
1937+00	0.000 mm	6.096 mm	7.144 mm	9.096 mm
1936+50	0.000 mm	3.048 mm	6.096 mm	6.096 mm
1936+00	0.000 mm	6.096 mm	12.192 mm	15.240 mm
1935+50	0.000 mm	6.096 mm	6.096 mm	6.096 mm

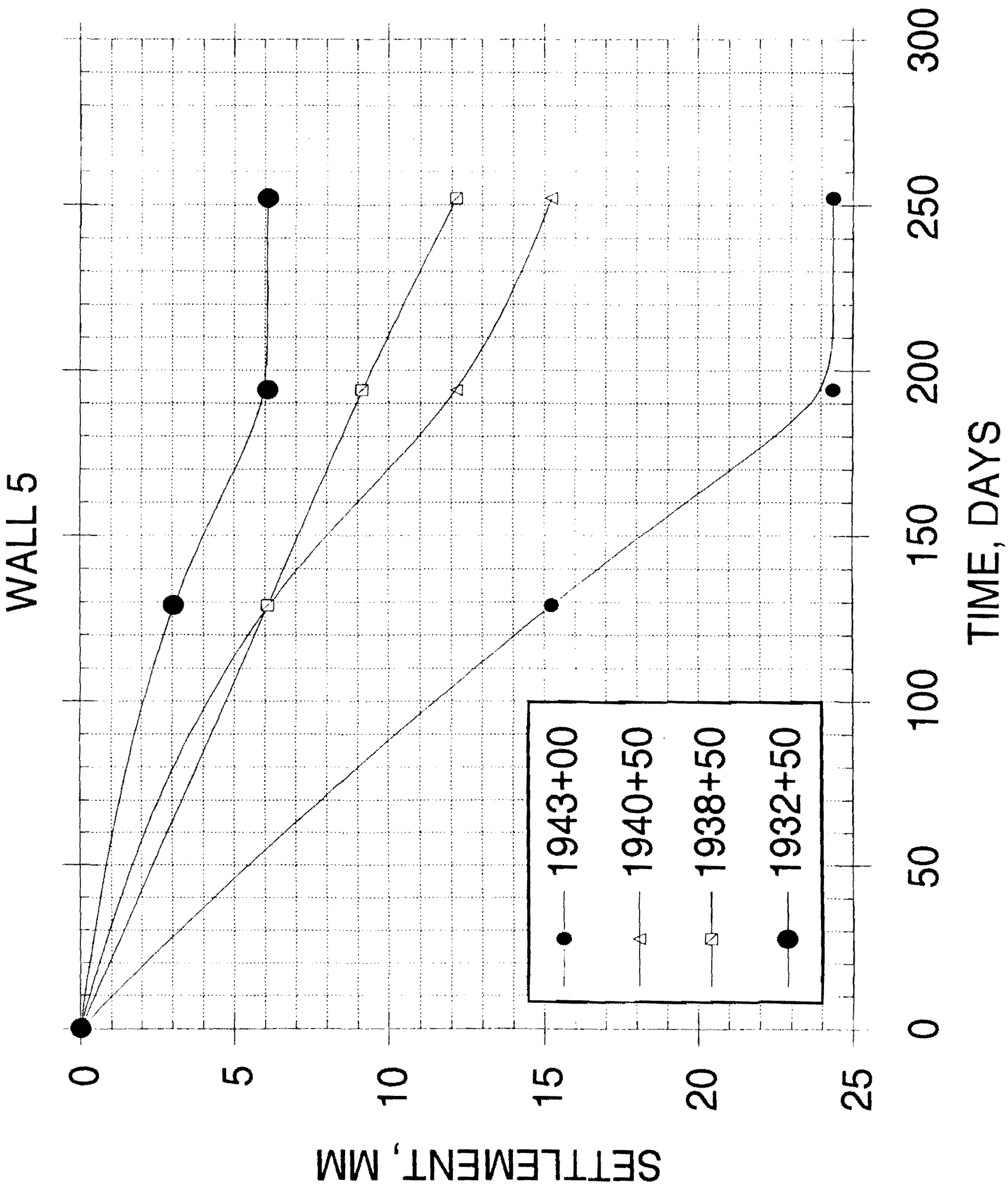


Figure 5.4 Settlement versus time for wall 5

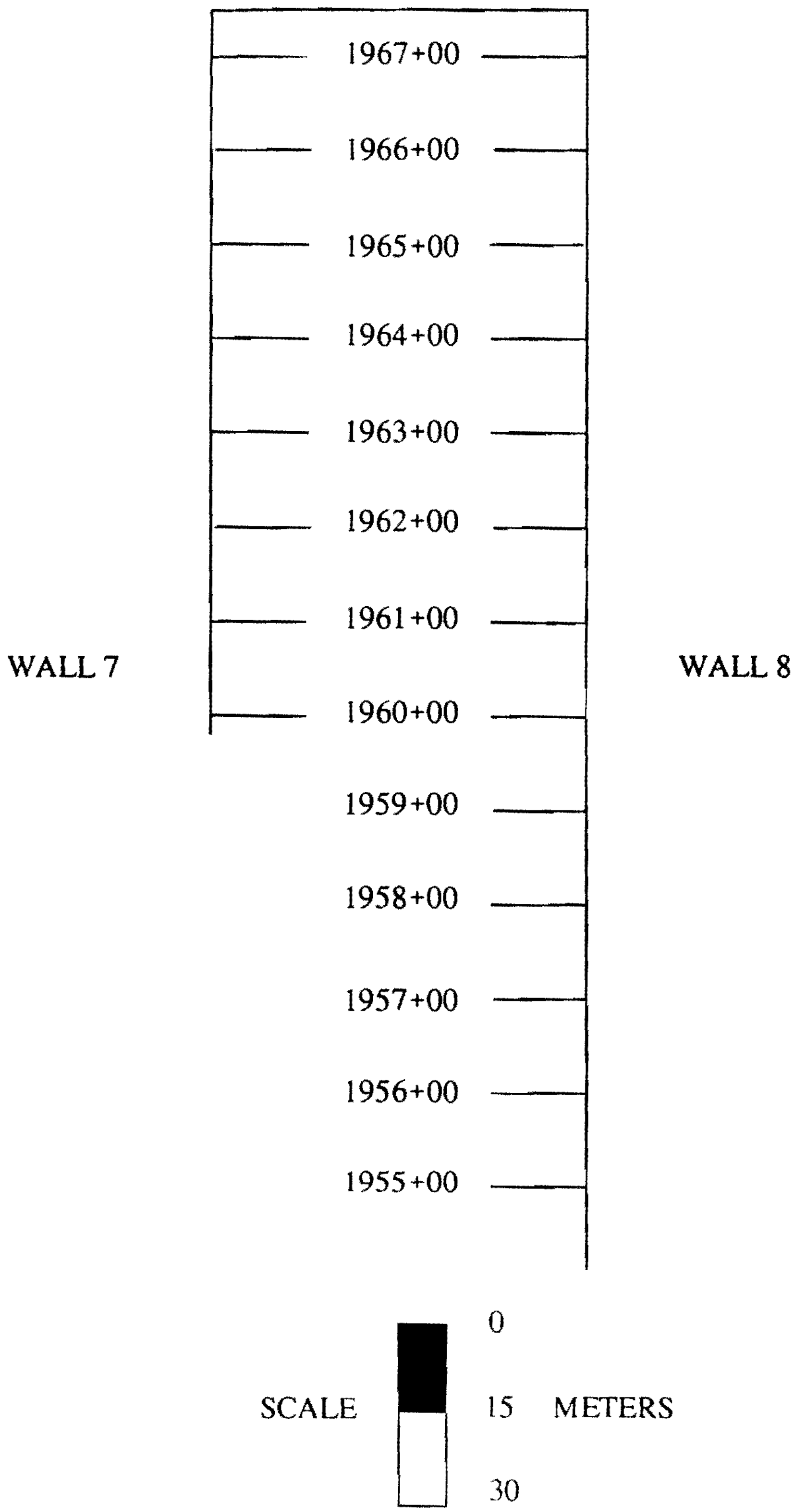


Figure 5.5 Surveying stations for walls 7 and 8

TABLE 5.3 - SETTLEMENT OF WALL 7 AT VARIOUS STATIONS

		TIME, DAYS		
STATION #	0	129	194	252
1967+25	0.000 mm	0.000 mm	3.048 mm	4.096 mm
1966+75	0.000 mm	0.000 mm	0.000 mm	2.048 mm
1966+25	0.000 mm	3.048 mm	6.096 mm	9.144 mm
1965+75	0.000 mm	3.048 mm	3.048 mm	3.048 mm
1965+25	0.000 mm	0.000 mm	9.144 mm	18.288 mm
1964+75	0.000 mm	3.048 mm	9.144 mm	12.192 mm
1964+25	0.000 mm	3.048 mm	3.048 mm	12.192 mm
1963+75	0.000 mm	3.048 mm	3.048 mm	9.144 mm
1963+25	0.000 mm	3.048 mm	9.144 mm	21.336 mm
1962+75	0.000 mm	6.096 mm	9.144 mm	18.288 mm
1962+38	0.000 mm	6.096 mm	12.192 mm	15.240 mm
1962+00	0.000 mm	0.000 mm	0.000 mm	9.144 mm
1961+50	0.000 mm	0.000 mm	3.048 mm	6.096 mm
1961+00	0.000 mm	0.000 mm	3.048 mm	6.096 mm
1960+50	0.000 mm	0.000 mm	0.000 mm	3.048 mm
1960+00	0.000 mm	0.000 mm	3.048 mm	6.096 mm

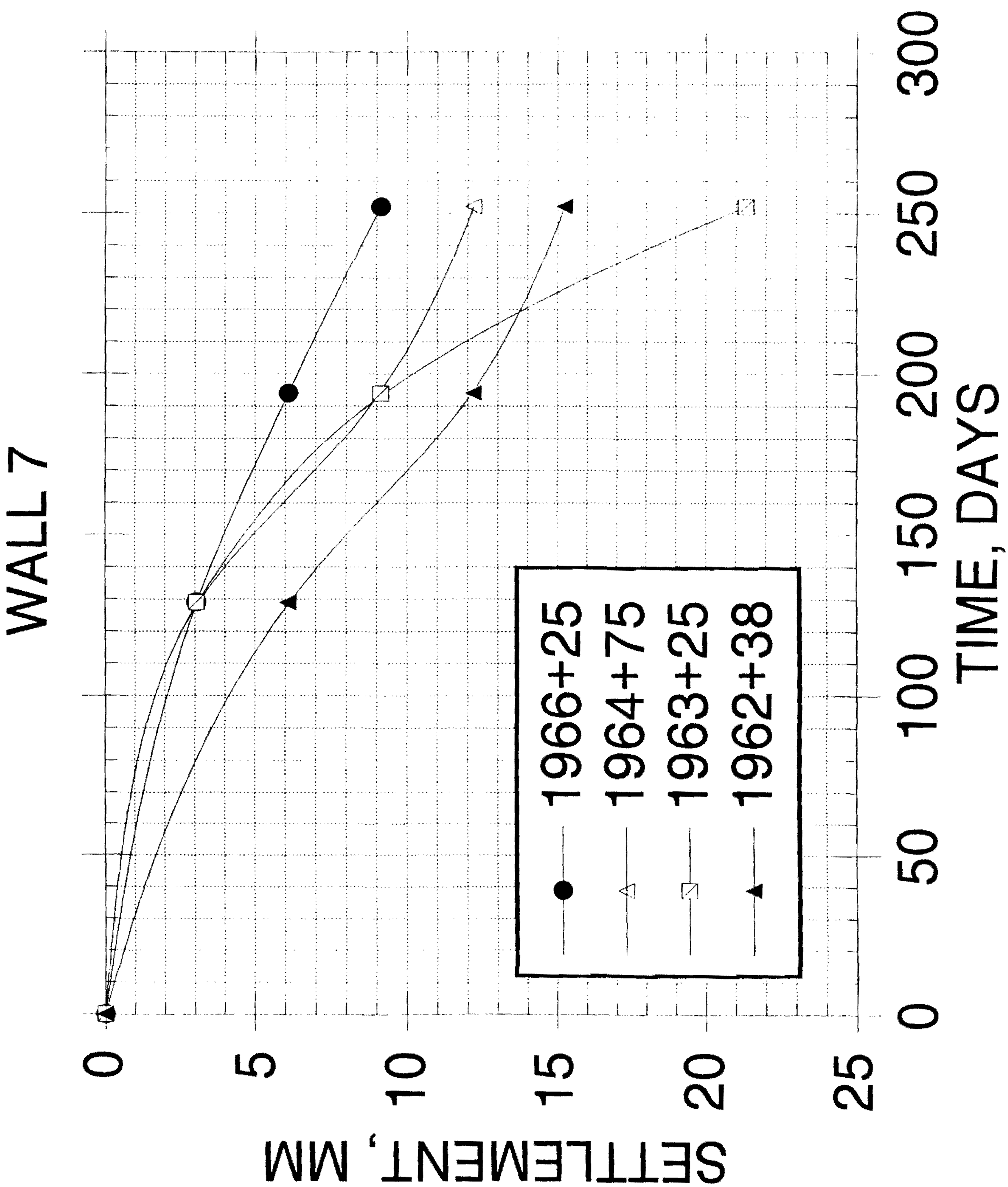


Figure 5.6 Settlement versus time for wall 7

TABLE 5.4 - SETTLEMENT OF WALL 8 AT VARIOUS STATIONS

	TIME, DAYS		
STATION #	0	105	161
1965+50	0.000 mm	3.048 mm	21.336 mm
1965+00	0.000 mm	9.144 mm	45.720 mm
1964+50	0.000 mm	3.048 mm	45.720 mm
1964+00	0.000 mm	0.000 mm	33.528 mm
1963+50	0.000 mm	3.048 mm	24.432 mm
1963+00	0.000 mm	0.000 mm	21.336 mm
1962+50	0.000 mm	0.000 mm	21.336 mm
1962+00	0.000 mm	0.000 mm	21.336 mm
1961+50	0.000 mm	0.000 mm	21.336 mm
1961+00	0.000 mm	0.000 mm	24.384 mm
1960+50	0.000 mm	0.000 mm	24.384 mm
1960+00	0.000 mm	0.000 mm	24.384 mm
1959+50	0.000 mm	3.048 mm	30.480 mm
1959+00	0.000 mm	9.144 mm	39.624 mm
1958+00	0.000 mm	12.192 mm	39.624 mm
1957+50	0.000 mm	9.144 mm	36.576 mm
1957+00	0.000 mm	6.096 mm	39.624 mm
1956+50	0.000 mm	6.096 mm	36.576 mm
1956+00	0.000 mm	12.192 mm	45.720 mm
1955+50	0.000 mm	15.240 mm	45.720 mm
1955+00	0.000 mm	15.240 mm	42.672 mm
1954+50	0.000 mm	12.192 mm	48.766 mm

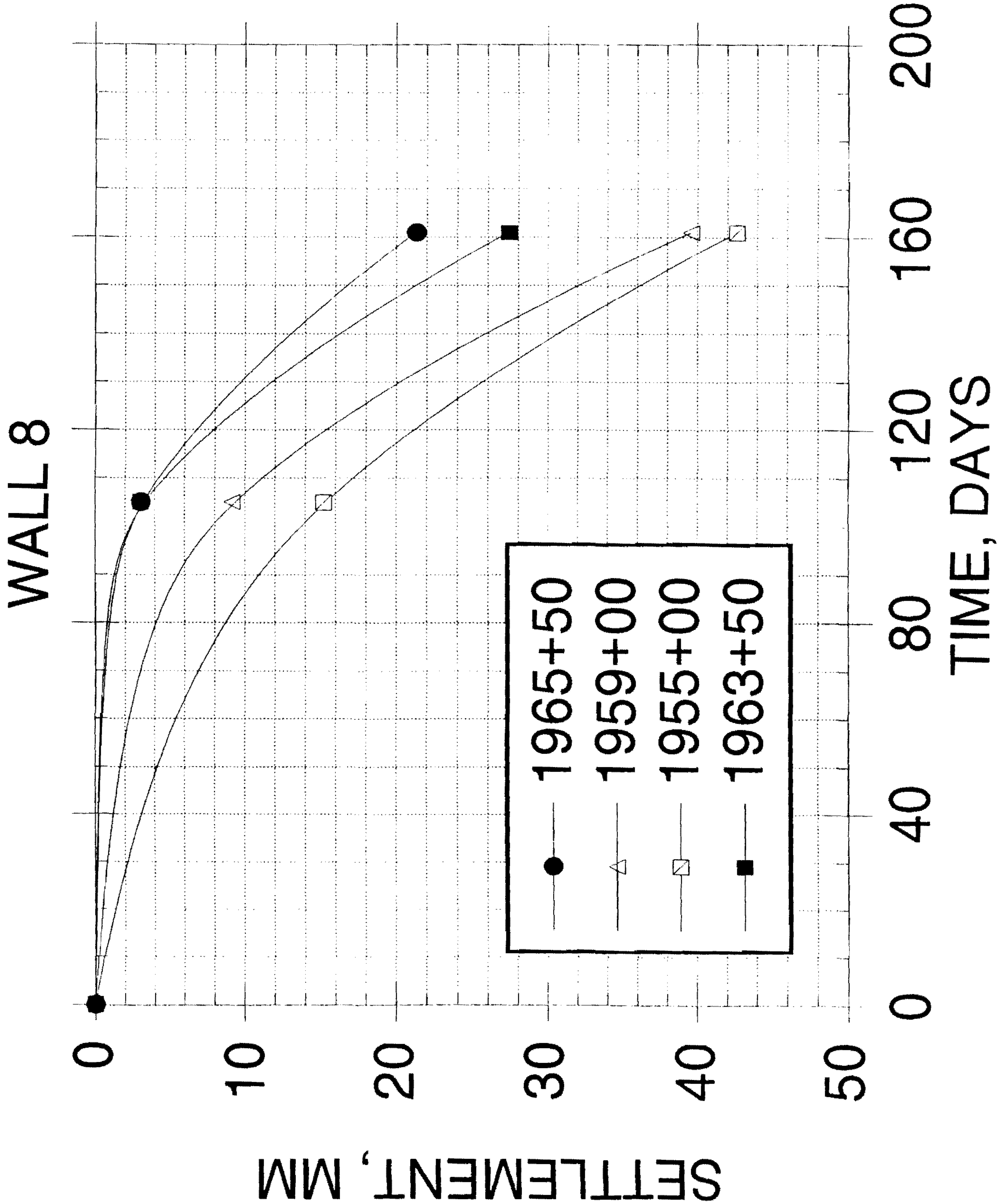


Figure 5.7 Settlement versus time for wall 8

wall. The maximum settlement occurs at stations 1954+50. The maximum differential settlement along the wall is 24 mm and occurs between stations 1954+50 and 1955+00. Since this occurs over an approximately 15 m length then the angular distortion is 0.0163. From table 5.4 and Figure 5.7, the rate of settlement ranges from 0.38 - 0.54 mm/day over the last 55 days of measurements. These values were the highest of the walls measured.

(e) Results for Wall 9

Figure 5.8 illustrates some of the various stations where the optical surveying points were located along the wall. Optical surveying measurements were taken periodically over a 182 day period to determine the settlement of the various stations with time. Table 5.5 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.9 shows representative settlement versus time plots for a few stations for wall 9.

From the data the following can be observed. The magnitude of the settlement ranges from 6 mm to 45 mm. The maximum settlement occurs at stations 1973+50 and 1974+00. The maximum differential settlement along the wall is 12 mm and occurs between stations 1973+00 and 1973+50. Since this occurs over an approximately 15 m length then the angular distortion is 0.008.

From Table 5.5 and Figure 5.9, the rate of settlement ranges from 0.000 - 0.124 mm/day over the last 98 days of measurements. These rates are fairly low and are similar to those observed for 5 and 7, but are less than that of walls 4 and 8.

(f) Results for Wall 10

Optical surveying measurements were taken periodically over a 359 day period to determine the settlement of the various stations with time. Figure 5.8 illustrates some of the various stations where the optical surveying points were located along the wall. Table 5.6 indicates the magnitude of the settlement recorded at the various stations at select times, whereas Figure 5.10 shows representative settlement versus time plots for a few stations for wall 10. (It should be noted that the settlement measurement of wall 10 was taken at one time, 359 days, so that the dashed lines are simply linear interpolations in Figure 5.11).

From the data the following can be observed. The magnitude of the settlement ranges from 9 mm to 37 mm. The maximum settlement occurs at station 1971+00. The maximum differential settlement along the wall is 24 mm and occurs between stations 1970+50 and 1971+00. Since this occurs over an approximately 15 m length then the angular distortion is 0.0163.

(g) Summary

1) The walls with piling beneath their foundations, walls 4, 5, 7, and 8, showed less settlement than those without the piling, walls 9 and 10.

2) Of the walls with pilings, wall 7 shows less settlement than walls 4, 5, and 8,

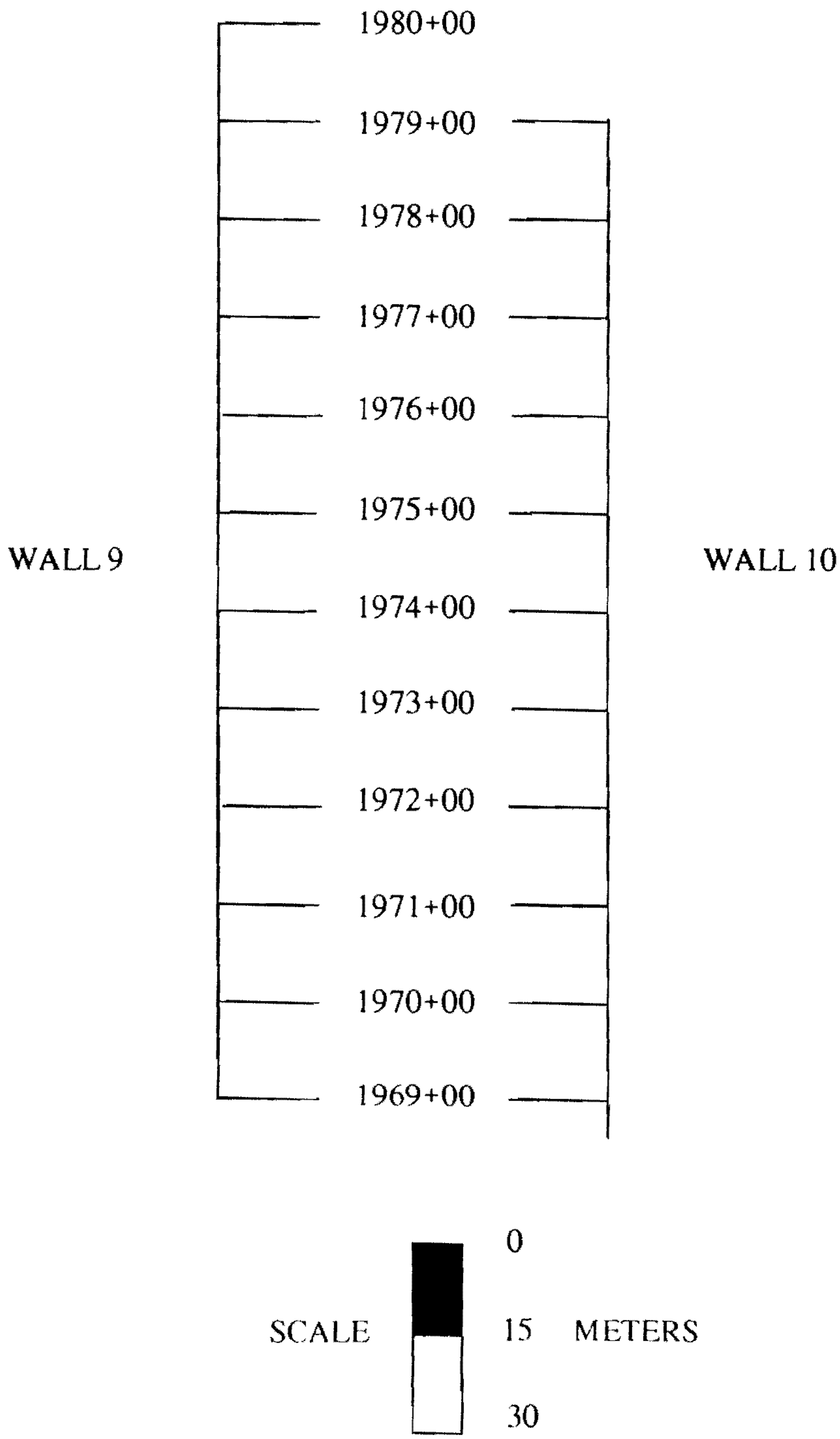


Figure 5.8 **Surveying stations for walls 9 and 10**

TABLE 5.5 - SETTLEMENT OF WALL 9 AT VARIOUS STATIONS

	TIME, DAYS			
STATION #	0	43	84	182
1970+50	0.000 mm	3.048 mm	6.096 mm	6.096 mm
1971+00	0.000 mm	9.144 mm	15.240 mm	15.240 mm
1971+50	0.000 mm	3.048 mm	9.144 mm	18.288 mm
1972+00	0.000 mm	9.144 mm	15.240 mm	24.384 mm
1972+50	0.000 mm	12.192 mm	21.336 mm	24.432 mm
1973+00	0.000 mm	21.336 mm	24.384 mm	33.528 mm
1973+50	0.000 mm	24.384 mm	39.624 mm	45.720 mm
1974+00	0.000 mm	24.384 mm	39.624 mm	45.720 mm
1974+50	0.000 mm	21.336 mm	30.480 mm	36.576 mm
1975+00	0.000 mm	15.240 mm	30.480 mm	36.576 mm
1975+50	0.000 mm	21.336 mm	27.432 mm	33.528 mm
1976+00	0.000 mm	12.192 mm	21.336 mm	27.432 mm
1976+50	0.000 mm	9.144 mm	15.240 mm	24.384 mm
1977+00	0.000 mm	6.096 mm	6.096 mm	18.288 mm
1977+50	0.000 mm	6.096 mm	6.096 mm	18.288 mm
1978+00	0.000 mm	3.048 mm	6.096 mm	9.144 mm
1978+50	0.000 mm	3.048 mm	3.048 mm	15.240 mm
1979+00	0.000 mm	3.048 mm	9.144 mm	15.240 mm
1979+50	0.000 mm	3.048 mm	3.048 mm	6.096 mm

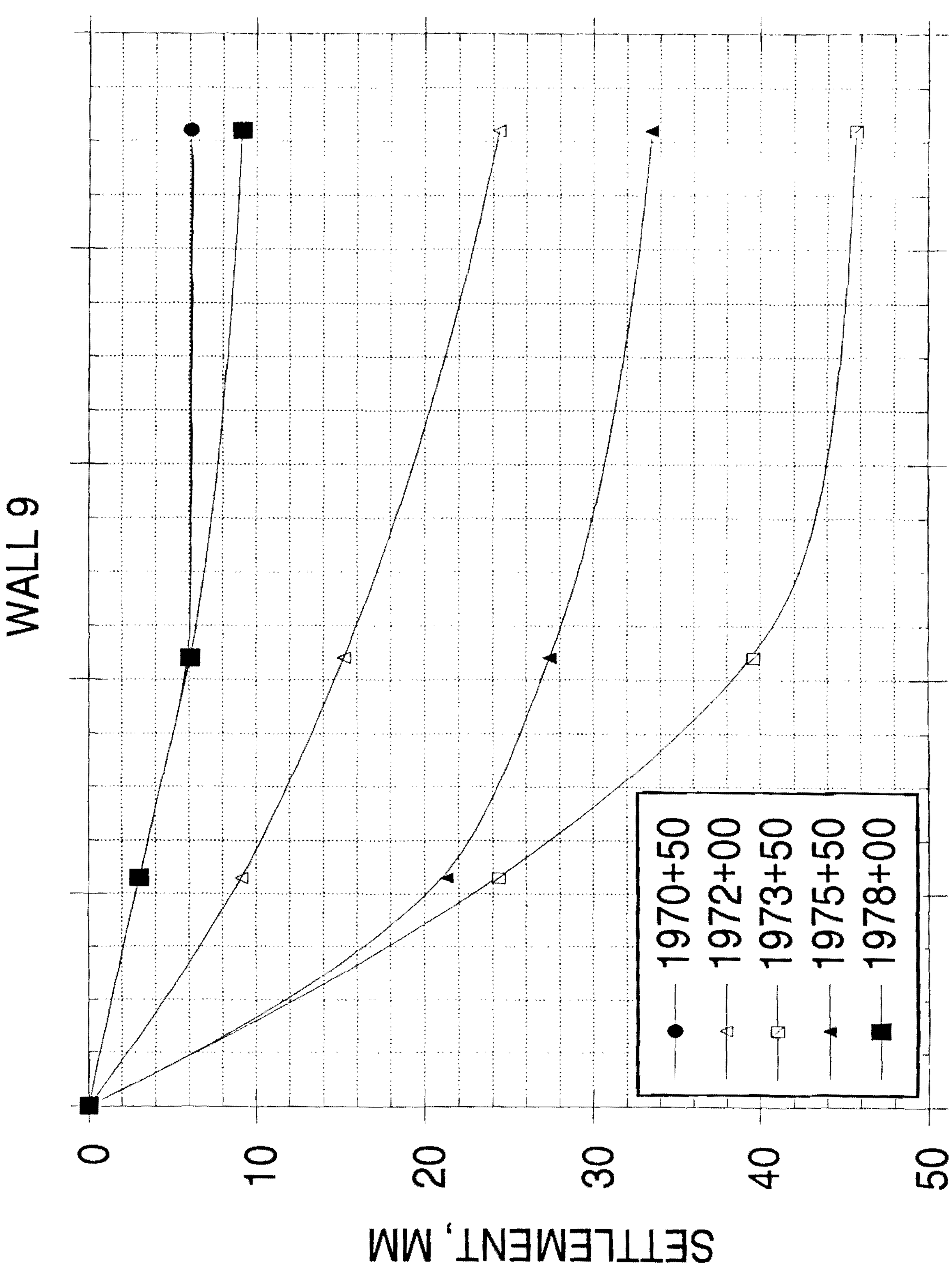


Figure 5.9 Settlement versus time for wall 9

TABLE 5.6 - SETTLEMENT OF WALL 10 AT VARIOUS STATIONS

		TIME, DAYS
STATION #	0	359
1969+50	0.000 mm	9.144 mm
1970+00	0.000 mm	9.144 mm
1970+50	0.000 mm	12.192 mm
1971+00	0.000 mm	36.576 mm
1971+50	0.000 mm	15.240 mm
1972+00	0.000 mm	12.192 mm
1972+50	0.000 mm	18.288 mm
1973+00	0.000 mm	12.192 mm
1973+50	0.000 mm	12.192 mm
1974+00	0.000 mm	15.240 mm

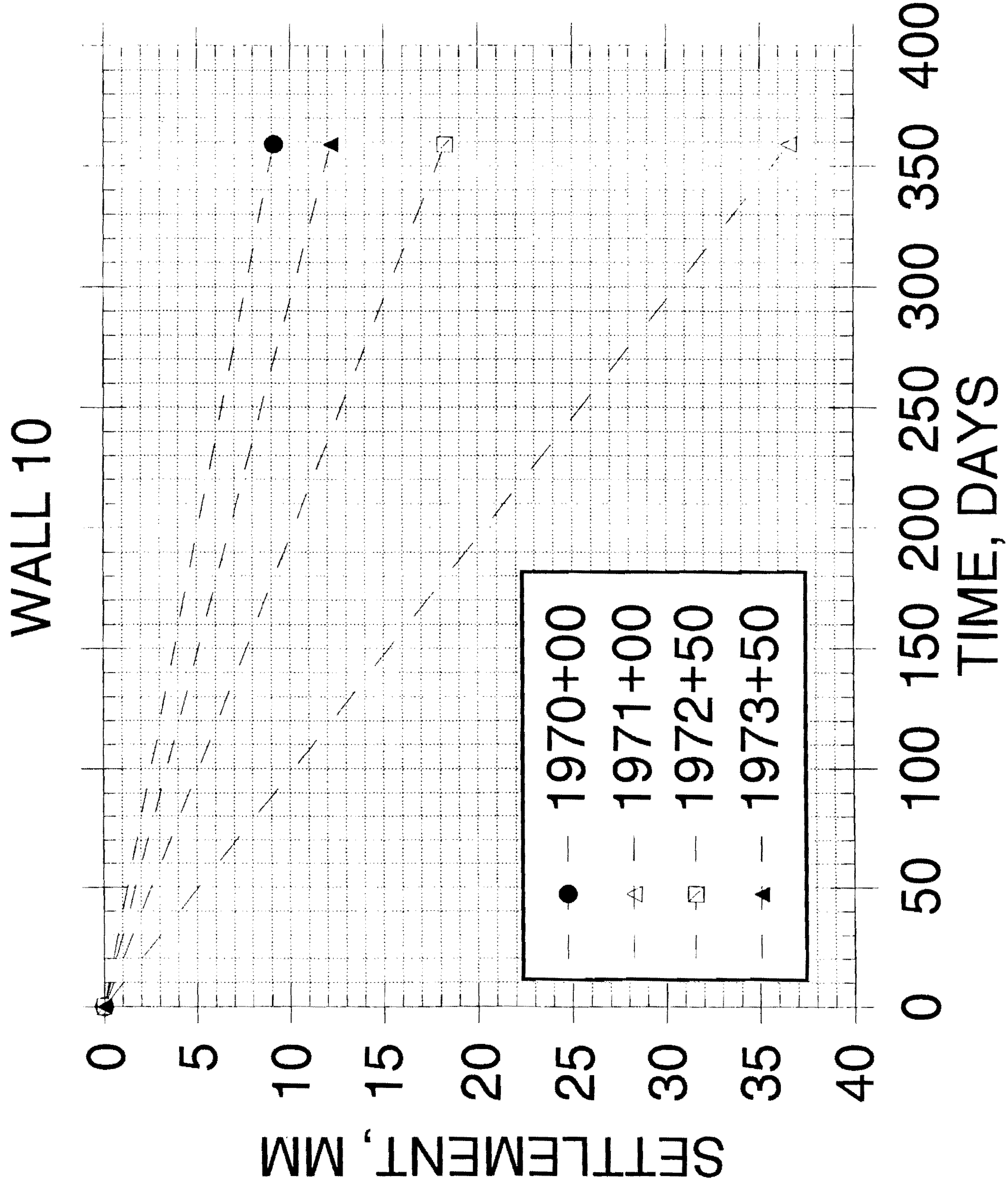


Figure 5.10 Settlement versus time for wall 10

possibly because of both the piling and the cement stabilized soil.

3) Walls without the stabilized soil show higher rates of settlements over the period of time measured, especially walls 4 and 8.

4) Differential settlement and angular distortion were quite high for some of the walls, especially for walls 5, 7, 8 and 10, to an extent that cracking of brittle materials might be expected.

5.3 MONITORING RESULTS FOR STATION 1970+00 - WALL 9

(a) Settlement Results

To determine the magnitude of settlement at station 1970+00 for wall 9, the settlement of the intersection of the upper (Horizontal #2) and lower (Horizontal #1) inclinometers with the wall face were measured over time. Figure 5.11 illustrates the settlement versus time plot for the inclinometers. This plot indicates that the magnitude of settlement is 2.3 mm (0.09 in) for the lower inclinometer and 3.6 mm (0.14 in) for the upper inclinometer. Additionally, the rate of settlement ranges from 0.000 to 0.001 mm/day for the last 150 days, which indicates that settlement has stabilized.

A key question in determining the performance of the wall is whether the face of the wall is behaving essentially as a rigid body or is deforming. To answer this question the difference between the settlement values for the upper and lower inclinometer casings was determined. The difference was then subtracted from the initial distance between the two inclinometers, which was 2.896 m (9.5 ft). As an additional check, the distance between the two inclinometer casings was measured directly. The results of the two methods yielded essentially the same values and provided an independent check on one another. The results of the analysis are given in Table 5.7, which indicates that the face of the wall is behaving as a rigid body.

(b) Horizontal Inclinometers

The settlement and/or vertical heave for the stabilized fill and soil is illustrated by the relative and absolute deflections of the two horizontal inclinometer casings shown in Figures 5.12 to 5.15. In general, both the stabilized fill and the soil behind the wall settled uniformly for the first year. However, more recently the deflections show the settlement to increase approximately linearly with distance away from the face of the wall. The magnitude of the settlement is 11.0 mm (0.43 in), which is quite small. While the exact reason for the change is not known, it is possible that it is due to the increase in construction work in the vicinity of Station 1970+00 on wall 9.

(c) Vertical Inclinometer

The deflections of the vertical inclinometer, both parallel and perpendicular to the wall are shown in Figures 5.16 and 5.17. The deflection of the inclinometer perpendicular to the wall indicates that the upper portion of the wall is rotating away from the wall face, whereas the lower portion is being deflected towards the wall face (Figure 5.17). The

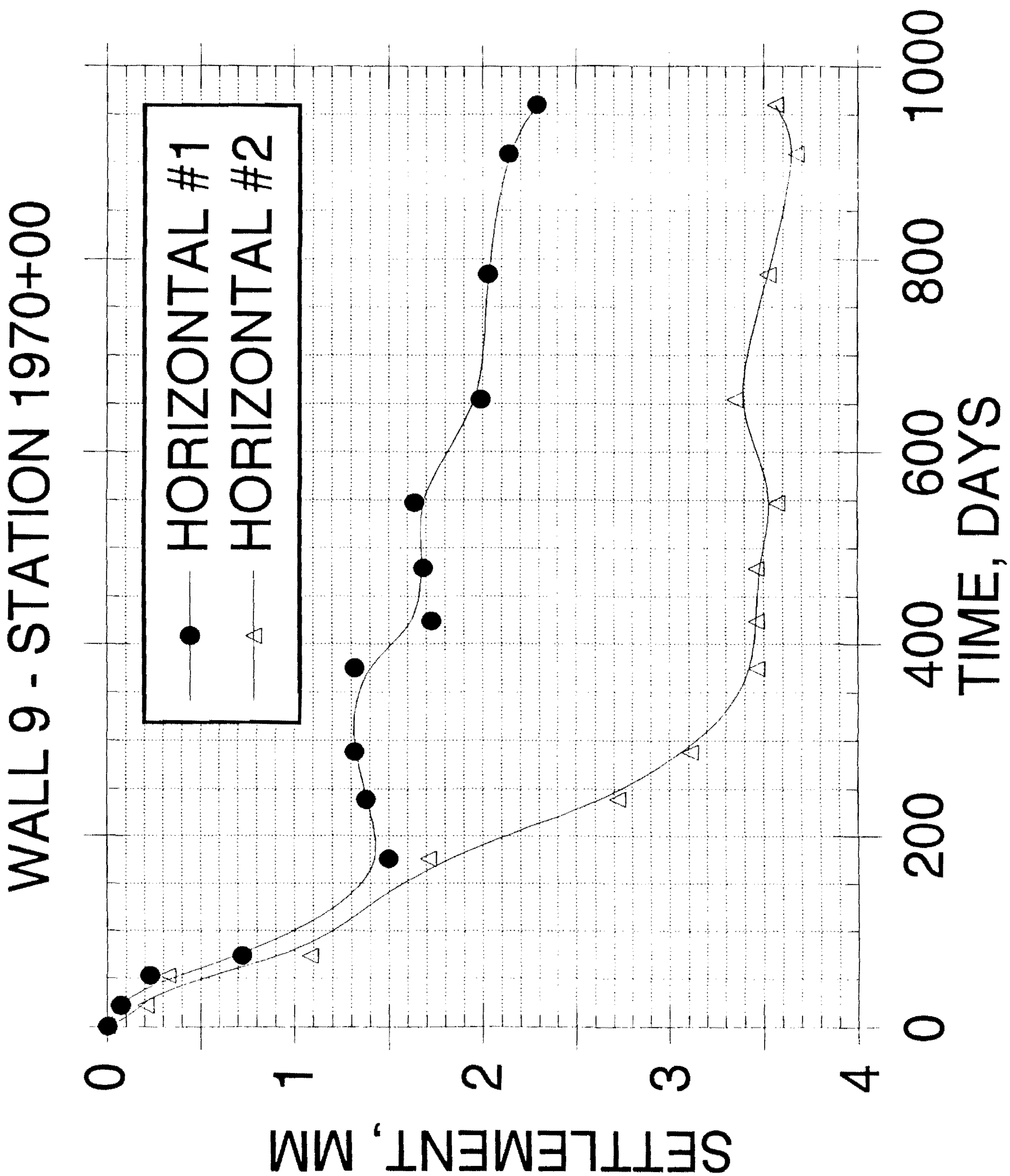


Figure 5.11 Settlement versus time for inclinometer casings 1 and 2 - wall 9

TABLE 5.7 - SEPARATION BETWEEN INCLINOMETERS FOR WALL 9

DATE	HEIGHT BETWEEN INCLN. (FT)
1/11/91	9.499
2/11/91	9.499
4/4/91	9.498
7/16/91	9.499
8/16/91	9.496
10/4/91	9.494
1/10/92	9.493
2/28/92	9.496
4/24/92	9.496
7/1/92	9.493
11/25/92	9.495
2/24/93	9.495
5/28/93	9.495
7/18/93	9.496

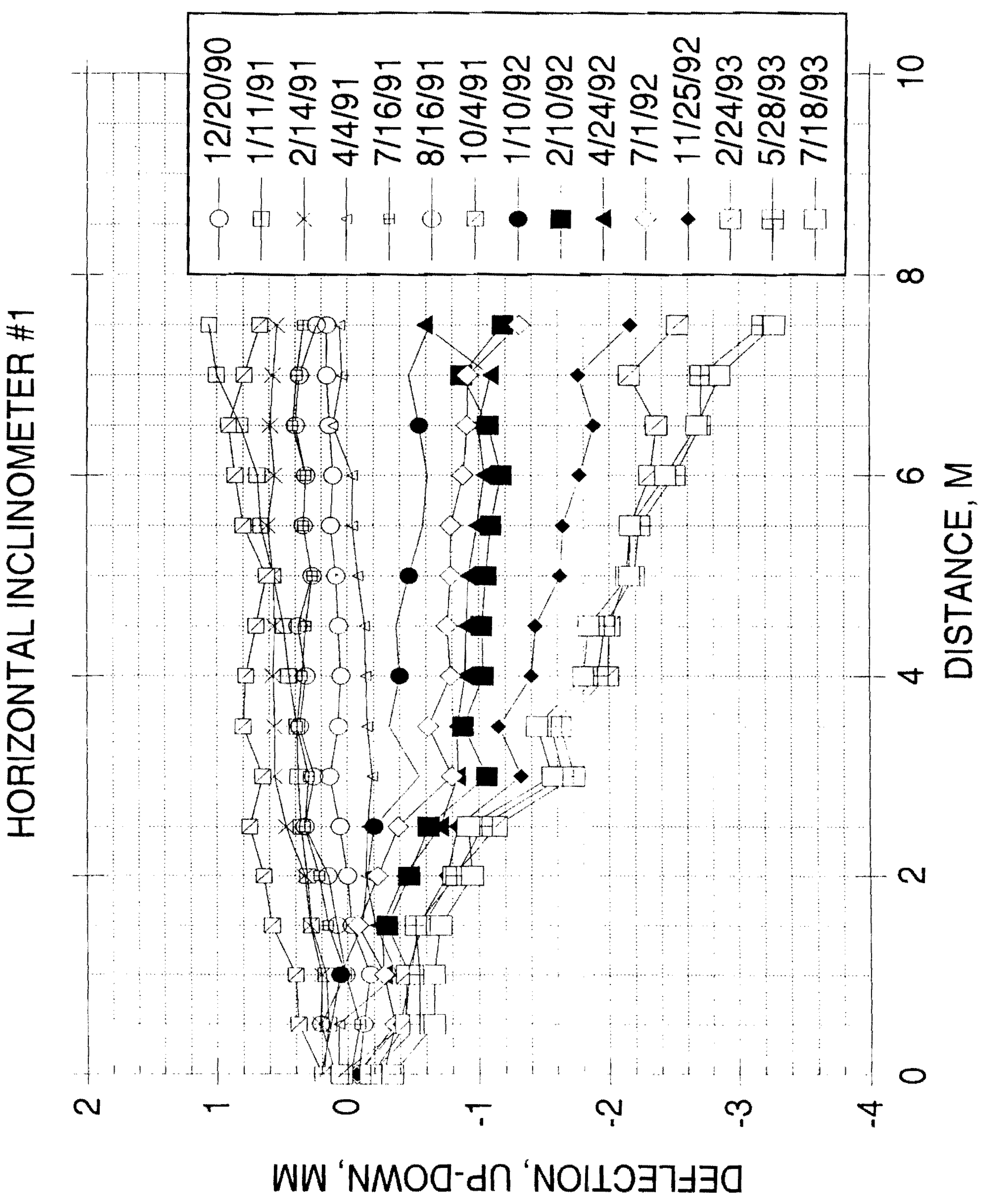


Figure 5.12 Relative deflection, horizontal inclinometer 1, wall 9

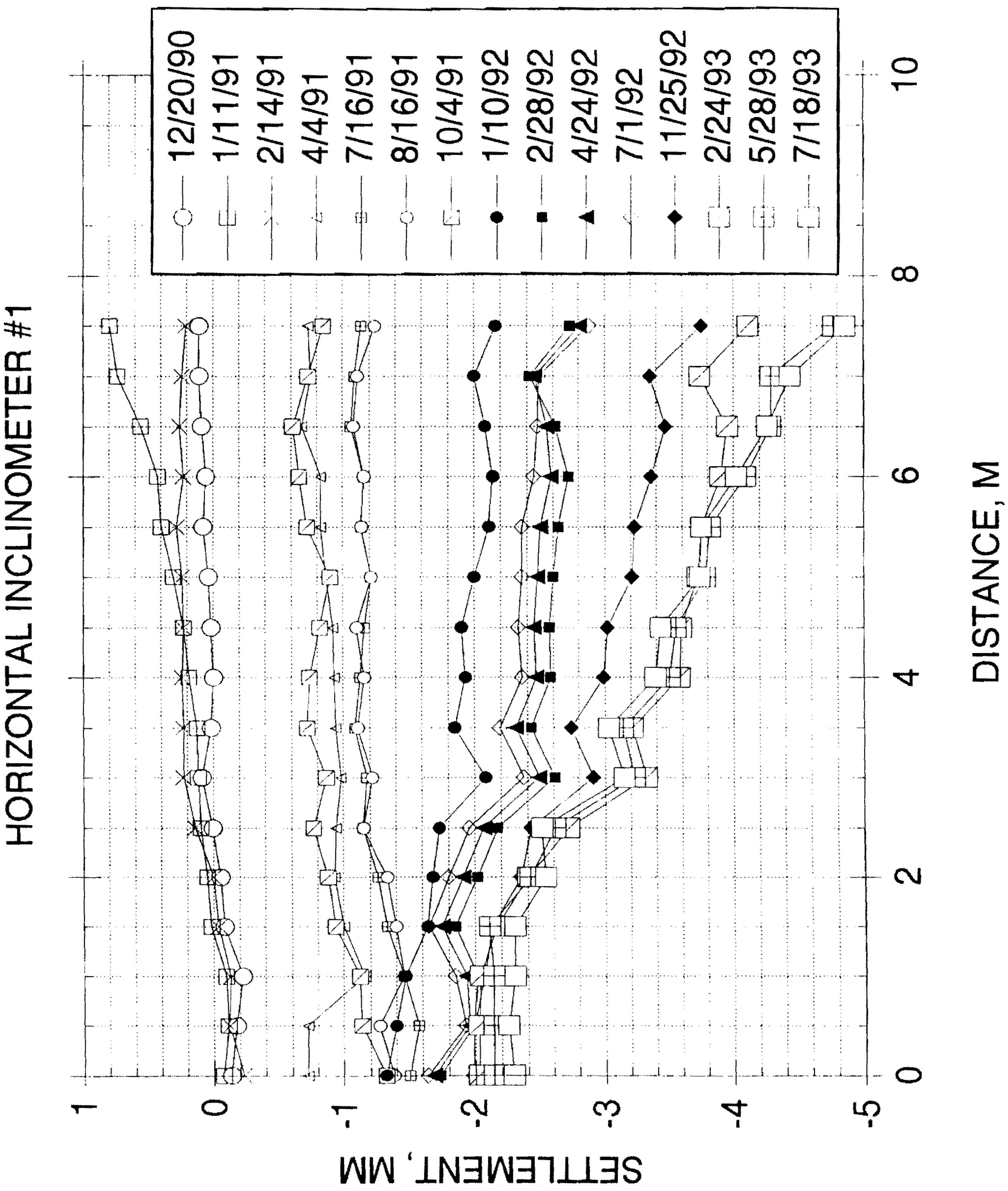


Figure 5.13 Absolute deflection, horizontal inclinometer 1, wall 9

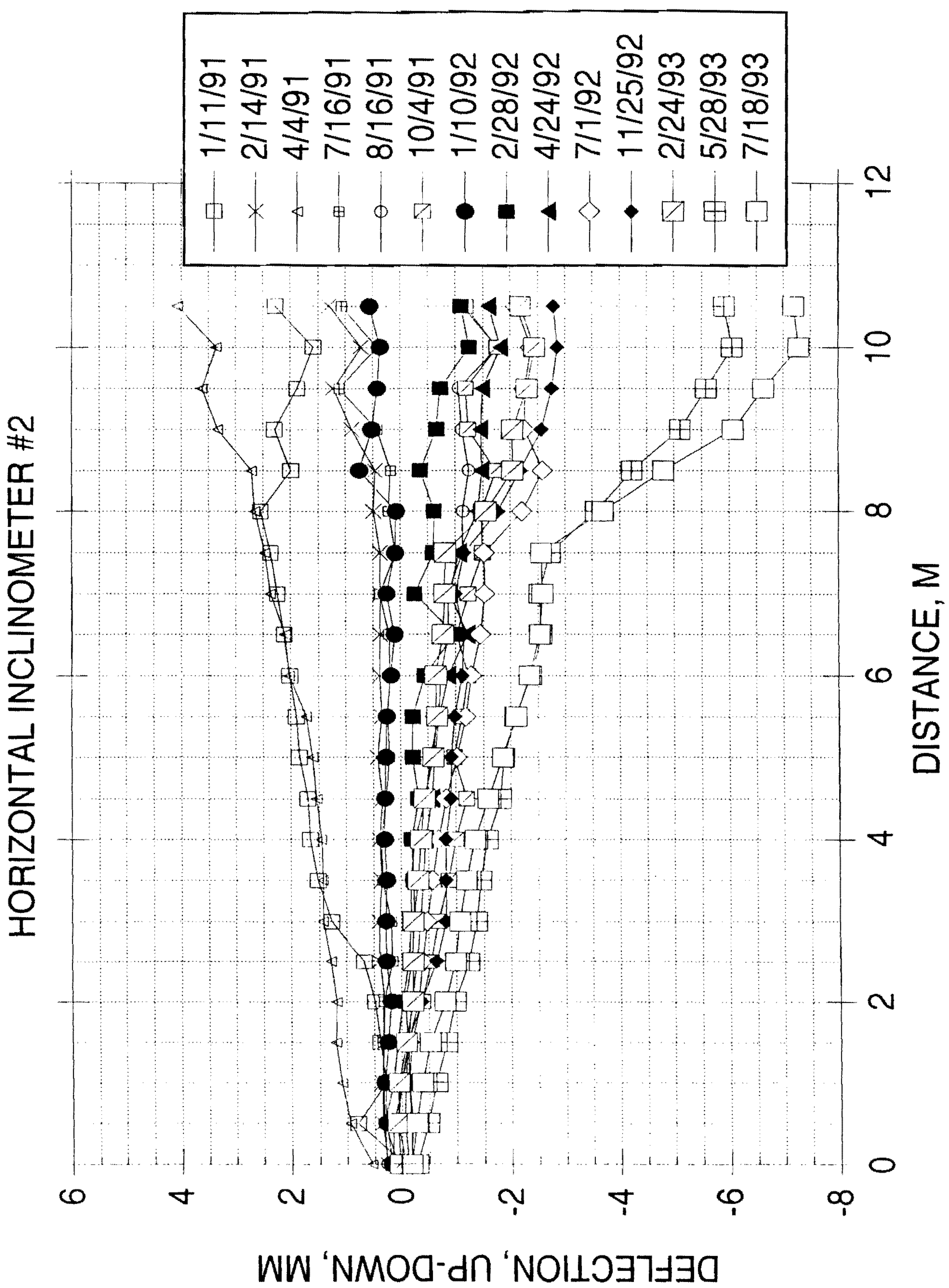


Figure 5.14 Relative deflection, horizontal inclinometer 2, wall 9

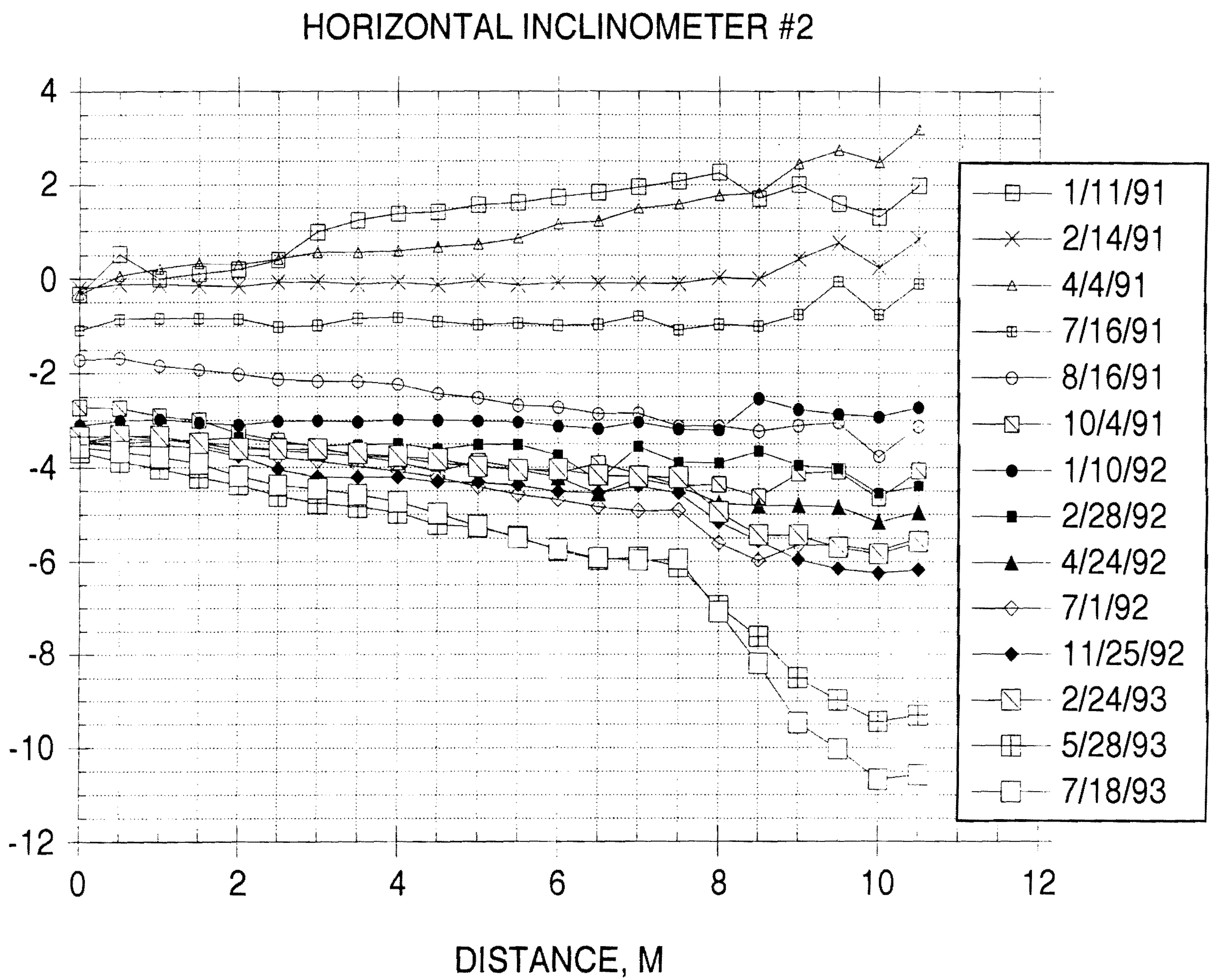


Figure 5.15 Absolute deflection, horizontal inclinometer 2, wall 9

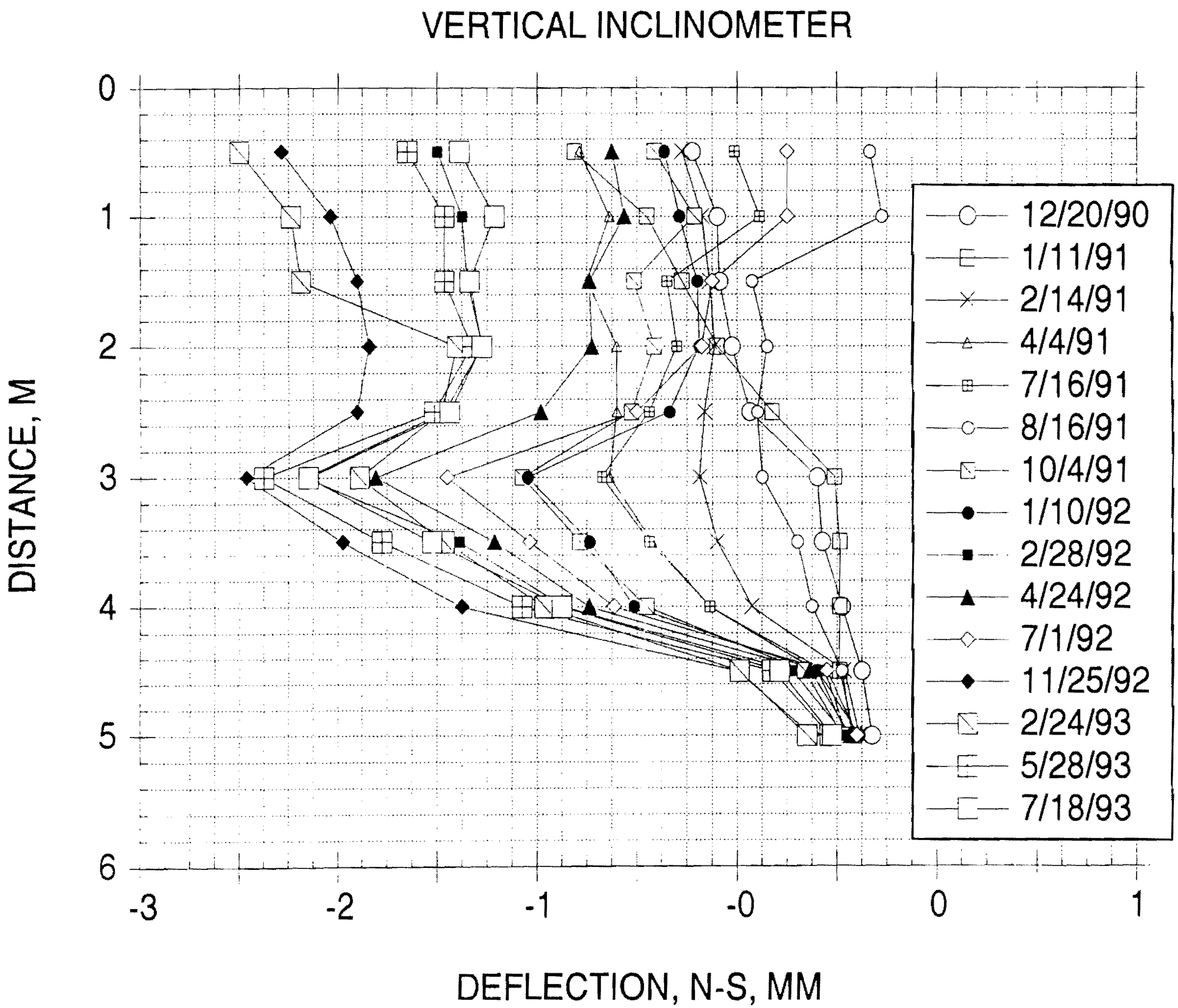


Figure 5.16 Vertical inclinometer deflection, parallel to wall 9

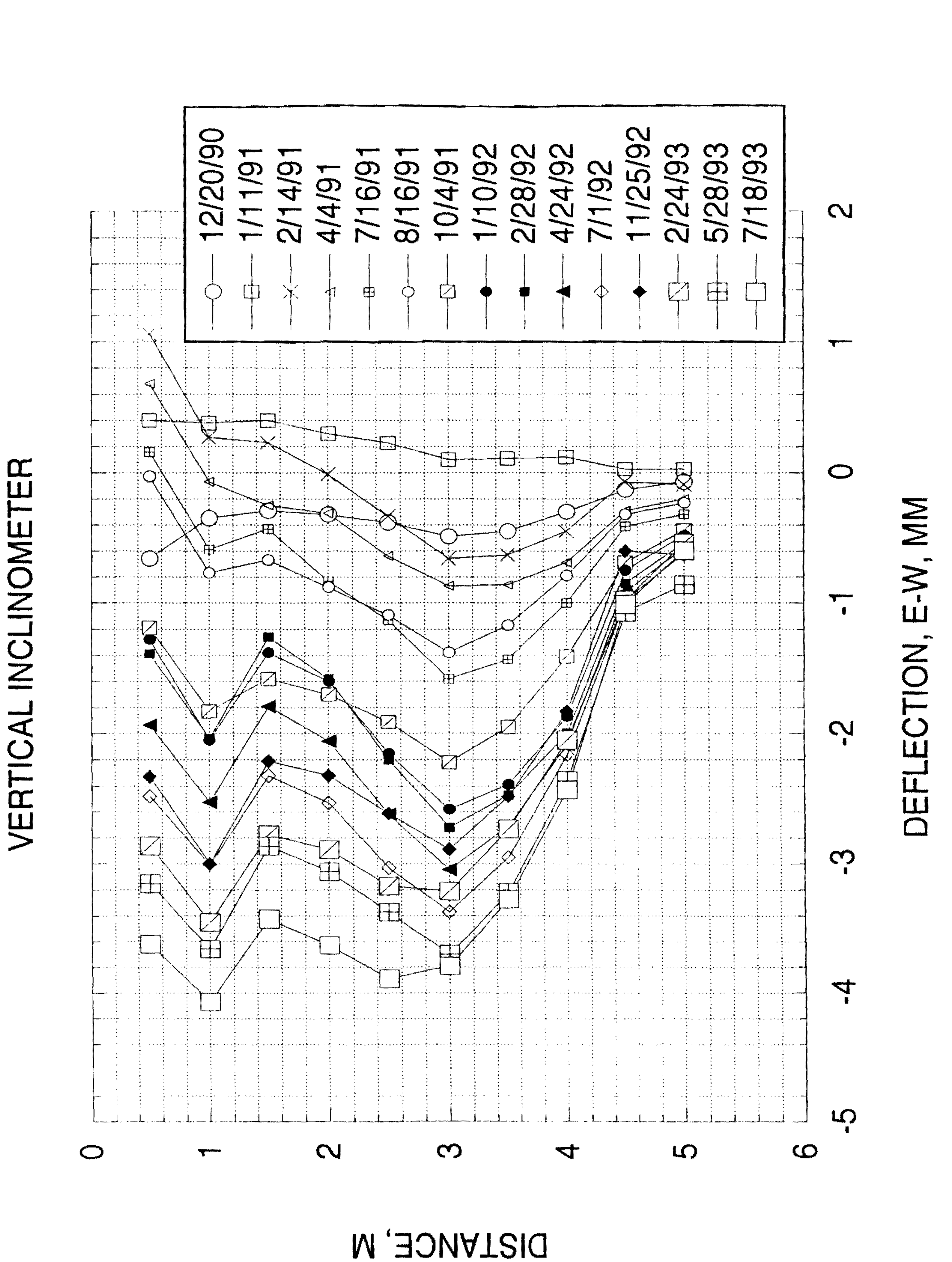


Figure 5.17 Vertical inclinometer deflection, perpendicular to wall 9

magnitude of the differential deflection, between the top and center of the wall, is 3.9 mm (0.15 in), which is the same as the maximum central deflection. It should be noted that the maximum central deflection occurs at approximately the same height as the top horizontal inclinometer.

The deflection of the inclinometer parallel to the wall indicates that the stabilized fill is being displaced southward, with the central portion of the fill having the greatest displacement (Figure 5.16). This is consistent with the observed settlements for wall 9, where the maximum settlement occurs south of station 1970+00.

(d) Earth Pressure Cells

Table 5.8 indicates the variation in stress with time for the various earth pressure cells. Initially the stresses recorded are largely as expected for gravity loading. However with the passage of time the stresses decrease and actually become tensile for some of the pressure cells. This variation appears to be predominantly seasonal, reaching a minimum in the summer months and a maximum over the winter. While the exact cause of this behavior is not known, a possible explanation for this could be that it is related to the presence or absence of water in the stabilized fill or soil. Since the pressure cells measure the total stress acting on their surfaces, the presence or absence of water could be critical to the measurement of the stresses. For example, arching of the soil and stabilized fill probably occurs around the pressure cells leading to stress concentration of the normal stress at the edges of the cells, instead of being uniformly distributed across it. Consequently, this could leave a void between the fill or soil and the face of the pressure cell. During the dry periods of the year (~April to September) this could cause the pressure cells to underestimate the stresses, whereas during the wet periods of the year (~October to March) the presence of water could fill the void left by arching, leading to a more uniformly distributed normal stress across the cell face causing an increase in the measured stresses.

(e) Load Cells

The variation in the forces on the anchors with time for the monitored anchors is given in Table 5.9. These initially increased with time, and then apparently stabilized, although load cells 1 and 3 failed within a year from internal shorting out. There did not appear to be any major seasonal variation. Interestingly, the forces on load cells 2 and 3, which roughly correspond to the location of the maximum deflection of the wall, were higher than that in the lower load cell 1 at the base of the wall. Typical magnitudes of the forces are from 1.5 kN (330 lbs.) to 3.5 kN (770 lbs.).

5.4 MONITORING RESULTS FOR STATION 1966+80 - WALL 7

(a) Settlement Results

To determine the magnitude of settlement at station 1966+80 for wall 7, the settlement of the intersection of the inclinometer (Horizontal #3) with the wall face was measured. Figure 5.18 illustrates the settlement versus time plot for the inclinometer, which

TABLE 5.8 - PRESSURE CELL TABLE FOR WALL 9

Date	Pressure Cell # 6733 (psi)	Pressure Cell # 6734 (psi)	Pressure Cell # 6735 (psi)	Pressure Cell # 6736 (psi)
1/11/91	17.55	17.10	11.70	10.35
2/14/91	17.55	7.20	4.95	9.90
4/4/91	17.75	-6.80	4.05	11.70
7/16/91	2.25	-30.60	1.35	4.50
8/16/91	1.35	-29.70	4.05	1.35
10/4/91	***	***	***	***
1/10/92	4.50	10.80	13.05	5.42
2/28/92	14.19	12.15	22.05	19.84
4/24/92	6.94	-7.43	16.87	16.55
7/1/92	-22.05	-27.00	-16.65	7.20
11/25/92	11.25	7.20	###	28.80
2/24/93	11.29	7.18	###	28.77
5/28/93	11.22	7.00	14.35	28.23
7/18/93	11.25	7.12	16.23	26.54

Date	Pressure Cell # 6737 (psi)	Pressure Cell # 6738 (psi)	Pressure Cell # 6739 (psi)	Pressure Cell # 6740 (psi)
1/11/91	22.50	16.20	22.05	9.00
2/14/91	12.60	16.65	21.15	0.45
4/4/91	4.95	14.85	16.20	4.05
7/16/91	-15.30	-9.45	-7.65	-5.85
8/16/91	-16.65	-14.40	-12.60	-5.85
10/4/91	***	***	***	***
1/10/92	13.89	0.45	14.85	16.21
2/28/92	18.91	4.95	17.55	20.70
4/24/92	-8.10	5.19	14.95	4.32
7/1/92	-26.10	6.30	4.50	-2.70
11/25/92	12.15	25.97	22.35	30.60
2/24/93	12.14	26.02	22.40	29.85
5/28/93	11.63	25.93	22.35	30.25
7/18/93	11.78	25.88	22.29	31.65

*** - no readings were taken on this date.

- the connection was corroded, so no readings were taken.

TABLE 5.9 - LOAD CELL TABLE FOR WALL 9

Date	Load Cell #1 (kg)	Load Cell #2 (kg)	Load Cell #3 (kg)
12/11/90	20.53	159.49	47.22
1/11/91	60.49	172.91	103.63
2/14/91	90.48	233.11	146.00
4/4/91	109.72	299.61	shorted
7/16/91	161.89	527.07	shorted
8/16/91	shorted	347.89	shorted
10/4/91	***	***	***
1/10/92	shorted	322.47	shorted
2/28/92	shorted	357.02	shorted
4/24/92	shorted	352.91	shorted
7/1/92	shorted	363.17	shorted
11/25/92	shorted	shorted	shorted

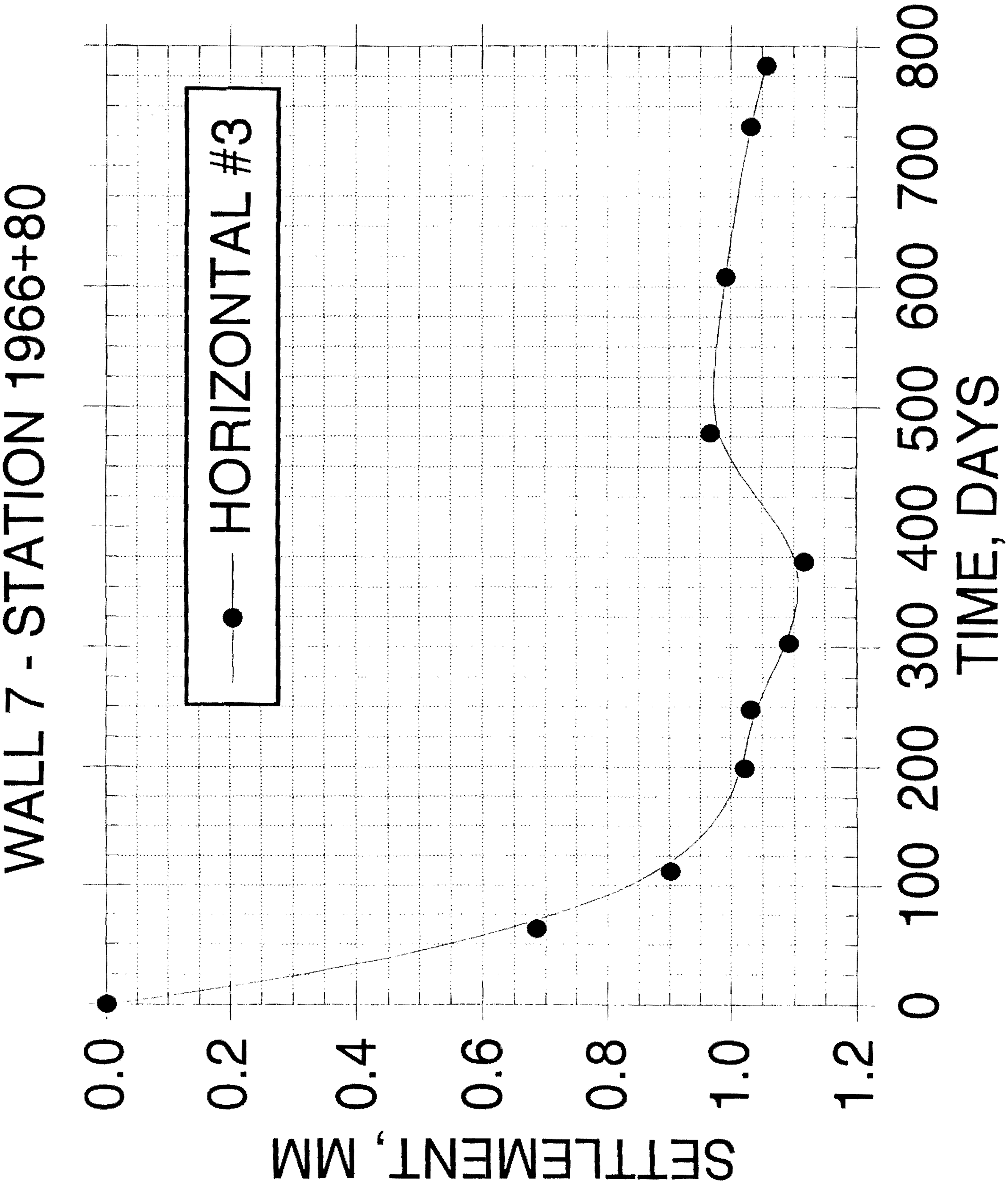


Figure 5.18 Settlement versus time for inclinometer casing 3 - wall 7

indicates that the final magnitude of settlement was 1.1 mm. Additionally, the rate of settlement ranged from 0.008 mm/day initially to zero after about 1 year, which indicates that settlement has stabilized.

(b) Horizontal Inclinometer

The relative and absolute deflections of the horizontal inclinometer are shown in Figures 5.19 and 5.20. Figure 5.20 indicates that the settlement of the stabilized fill (i.e., from 0 to 3 m) is nearly uniform and is quite small, being no more than 2 mm (0.08 in.). In contrast, the settlement in the soil increases with time and distance away from the wall face, reaching a maximum magnitude of 11.0 mm at a distance of 5 m in from the wall face. It should be noted that the magnitude of settlement in the soil has not obviously stabilized in the period of monitoring and doubled in the last two years, increasing from 5.4 mm in October of 1991 to 11.0 mm in August of 1993. This may be due to prolonged settlement of the compacted soil fill within the main embankment, or it may also be due to the increase construction in the vicinity of station 1966+80 at wall 7.

(c) Load Cell

Table 5.10 indicates the variation in anchor force with time. As noted above for the anchors in wall 9, there was some initial increase in force with time, although this particular load cell did not last long before shorting out. Typical final values were of the order of 1 kN (100 lbs.).

5.5 SUMMARY

The results of the instrumentation readings indicate that the displacements in the stabilized fill and soil are small, on the order of 1.0 to 11.0 mm. The largest contribution to the displacements is from settlement, which is uniform for wall 9 and varies approximately linearly with distance away from the wall face for wall 7. The remedial piling beneath wall 7 reduced the settlement in the stabilized fill. The face of wall 9 is settling as a rigid body. The maximum horizontal deflection of wall 9 occurs at the center of the wall and is 3.9 mm. The stresses in the stabilized fill and soil and the anchor forces are approximately as expected.

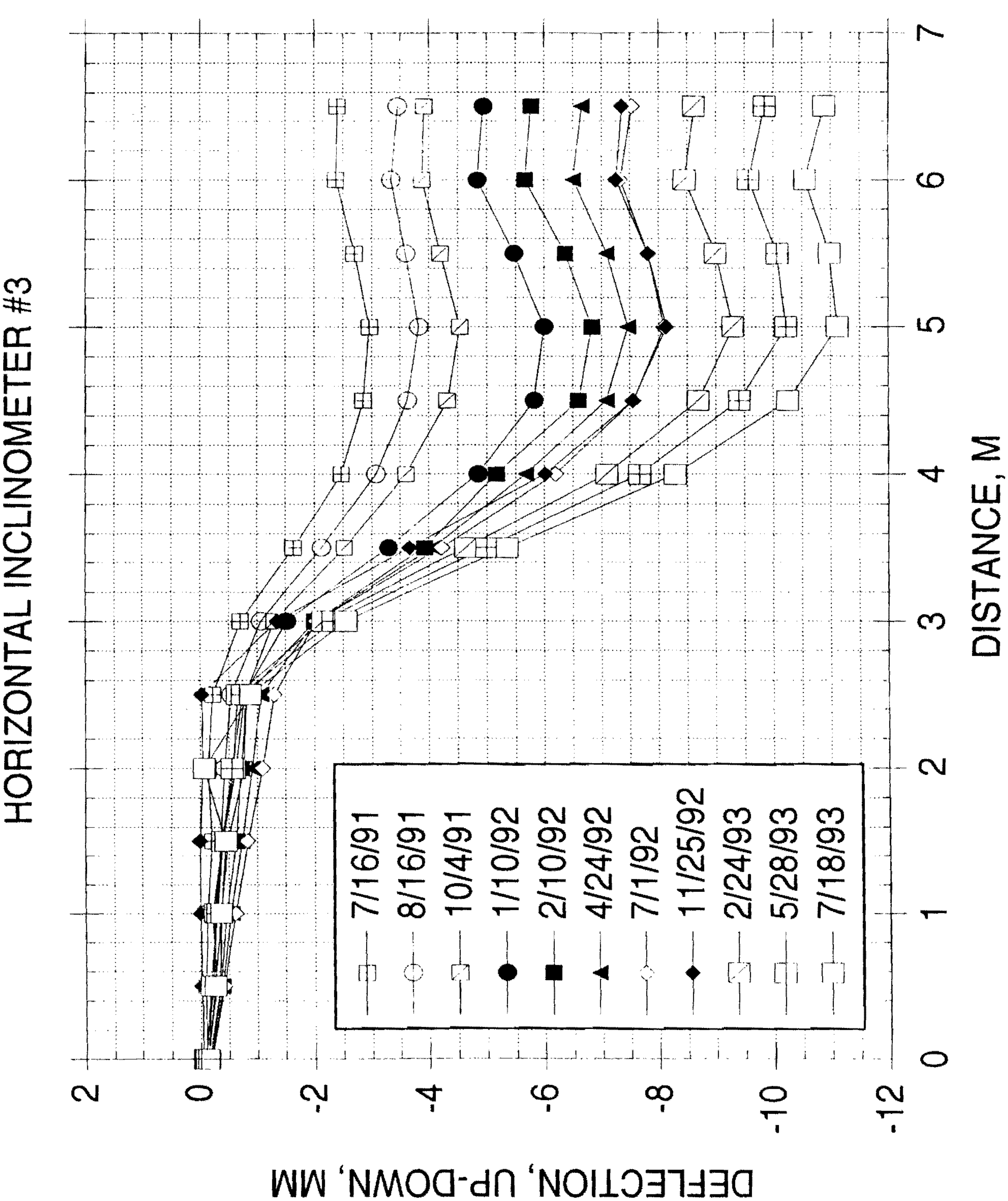


Figure 5.19 Relative deflection, horizontal inclinometer, wall 7

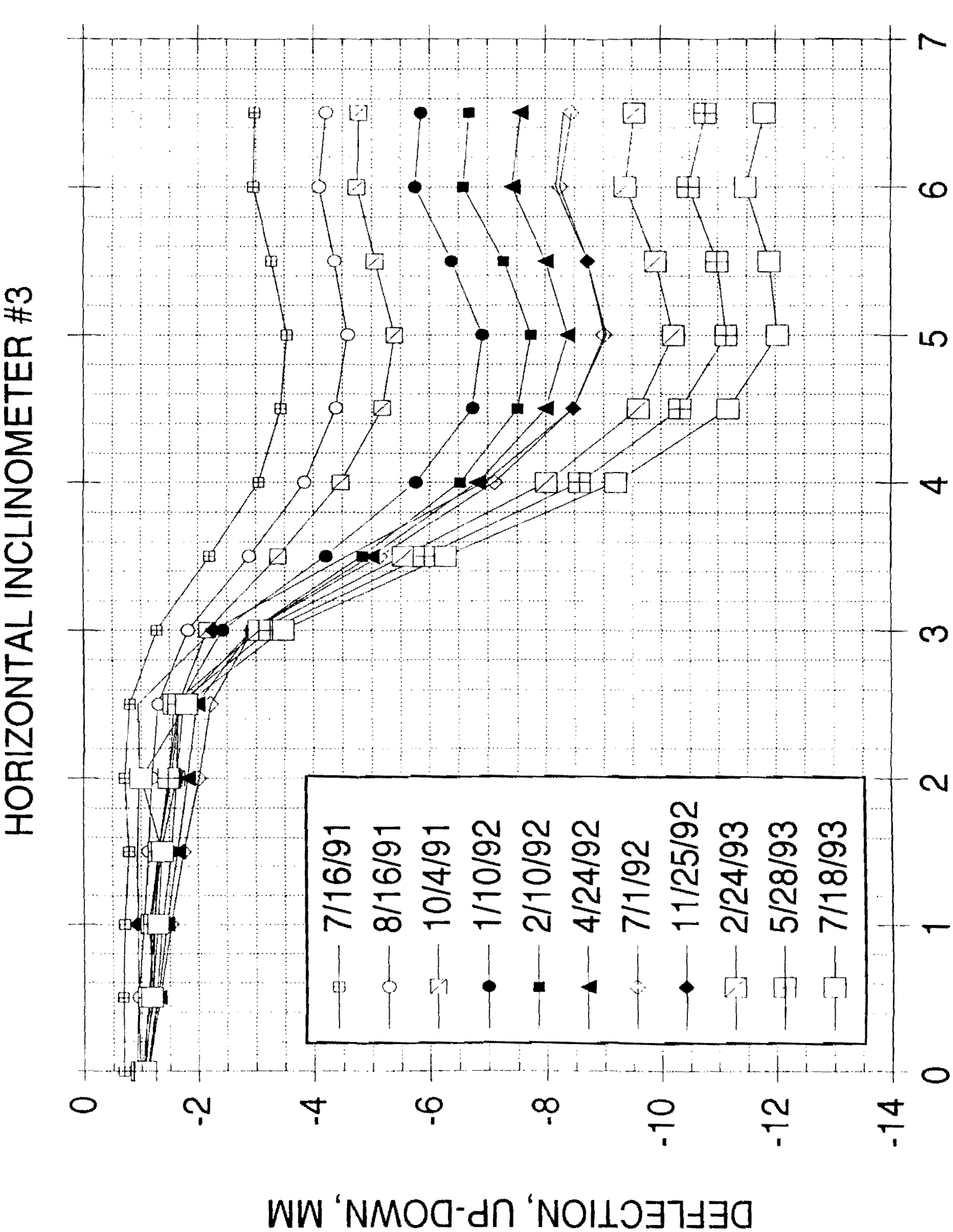


Figure 5.20 Absolute deflection, horizontal inclinometer, wall 7

TABLE 5.10 - LOAD CELL TABLE FOR WALL 7

Date	Load Cell #4 (kg)
12/11/90	
1/11/91	
2/14/91	
4/4/91	13.71
7/16/91	25.69
8/16/91	40.14
10/4/91	***
1/10/92	105.36
2/28/92	shorted
4/24/92	shorted
7/1/92	shorted
11/25/92	shorted

6. CONCLUSIONS

This monitoring program has largely confirmed the results of the previous TTI/TxDOT study, namely that it is possible to design and construct full-scale retaining walls to at least 8.5 m (28 ft.) in height using mainly cement stabilized soil to form the bulk of the cross-section. Only nominal anchors are required to retain any facing panels, which essentially just provide a finished appearance and may be either segmental or one-piece. Unlike conventional mechanically stabilized earth retaining walls, stabilization of the backfill material is done almost entirely by the addition of cement, rather than by the provision of reinforcing elements.

It is important to note that there are significant practical differences between this design and conventional mechanically stabilized earth walls (even though they may seem similar). These were outlined in the previous report 1178-1F dated November 1991. Chief among these is that the design is much more sensitive to differential settlement, being not as ductile as mechanically stabilized designs.

However, if properly engineered and constructed utilizing the data from subsurface investigations, such a design can perform well, as confirmed by the data from over two years of post-construction monitoring contained in this report. With proper foundation preparation, settlements and deformations were well within conventional limits, and soil stresses and anchor forces corresponded approximately to normal expectations.

7. REFERENCES

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