PB2001-101580

Research Report
KTC-90-23

GROUND MODIFICATION SYSTEMS (STONE COLUMNS AND WICK DRAINS IN JEFFERSON COUNTY)

by

Bobby W. Meade
Research Investigator

and

David L. Allen
Chief Research Engineer

Kentucky Transportation Center
College of Engineering
University of Kentucky
Lexington, Kentucky

in cooperation with
Transportation Cabinet
Commonwealth of Kentucky

and

Federal Highway Administration
U.S. Department of Transportation

September 1990

PB2001-101580

1. Report No. KTC-90-23	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Ground Modification Systems (Stone Columns And Wick Drains In Jefferson County)		5. Report Date October 1990
		6. Performing Organization Code
7. Author(s) B. W. Meade and David L. Allen		8. Performing Organization Report No.6 KTC-90-23
9. Performing Organization Name and Address Kentucky Transportation Center College of Engineering University of Kentucky Lexington, KY 40506-0043		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. Experimental Project 1-B
12. Sponsoring Agency Name and Address Kentucky Transportation Cabinet State Office Building		13. Type of Report and Period Covered Final
		14. Sponsoring Agency Code

15. Supplementary Notes

Publication of this report was sponsored by the Kentucky Transportation Cabinet with the U.S. Department of Transportation, Federal Highway Administration.

16. Abstract

The purpose of this study was to document construction procedures, monitor field performance, and make recommendations as to the effectiveness and future use of stone column and wick drains for ground modification purposes. This project involved the use of wick drains to accelerate consolidation in bridge approach foundations and stone columns to improve foundation support for reinforced soil walls supporting bridge abutments.

This effort was executed by use of field inspections, photologs, and instrumentation to monitor field response. Instrumentation included earth pressure meters, multipoint settlement gages, slope inclinometers, settlement platforms, and piezometers. Wick drain performance was satisfactory. One reinforced soil wall failed and the other was marginally stable. The failure appears to be the result of a combination of a weak foundation layer that was disturbed by stone column construction, high foundation pore pressure, and rapid wall construction.

17. Key Words Stone Columns Wick Drains Wet method Pore Pressure	18. Distribution Statement Unlimited with Kentucky Transportation Cabinet approval.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified		

INTRODUCTION

As a part of the upgrading and widening of Interstate 65 in Kentucky, the bridge over the Louisville and Nashville Railroad and the Northern Ditch in Jefferson County, Figure 1, required replacement. A contract was awarded in 1984 for construction of the part of Interstate 65 that included the bridge.

The replacement bridge is located in an area where foundation soils vary in thickness up to 50 ft. The bridge abutment foundations are approximately 30 ft thick. Foundation soils classify as A-6 or A-7 based on the AASHTO system or as CL based on the Unified Soil Classification System. Natural moisture contents range from 23 to 35 percent and liquidity indices range from 0.10 to 0.65. Results of consolidated-undrained tests of foundation soils indicate internal friction angles range from 20.5 degrees at Station 2016+75 to 35 degrees at Station 2013+05. Effective cohesion from these tests range from 0 psf at Station 2013+05 to 538 psf at Station 2016+75. Table 1 presents a summary of triaxial data.

Analyses indicated the foundation soil support was inadequate for the original design recommendation of a four-span bridge. Two primary alternatives considered were lengthening the bridge and ground improvement using stone columns supporting reinforced soil abutment walls for a two-span bridge. The ground improvement alternative was chosen because a savings of $1,265,000 was anticipated and construction time would be reduced due to an increased settlement rate as a result of the additional foundation drainage from stone columns. Foundation excavation to solid rock and replacement with rock instead of stone columns was discussed at the preconstruction conference, but the contractor chose not to propose that change.

Analyses performed by the Department of Highways, Division of Materials indicated the approach foundation settlement rate would be unacceptably slow. Prefabricated wick drains were used to dewater the approach foundations and accelerate settlement.

STUDY OBJECTIVES

Because the wick drains and stone columns were considered experimental, a study was initiated to monitor construction procedures and performance of the wick drains and stone columns. The objectives of the study were:

1. to document construction procedures and obtain experimental data on the wick drains;

2. to analyze field behavior by using settlement, pore pressure and stress cell data;

3. to document and monitor reconstruction of the reinforced earth wall; and

4. to make recommendations on the effectiveness and future use of these methods.

GROUND IMPROVEMENT LAYOUT AND DESIGN

The bridge is approximately 237 ft long with the south abutment (Abutment 1) located at approximately Station 2017+04 and the north abutment (Abutment 2) located at approximately Station 2019+41. The approaches extended for approximately 1,100 ft from each abutment.

Stone columns extended 39 ft back of the retaining wall at Abutment 1 and 29 ft back of the retaining wall at Abutment 2. Stone column design called for 6-ft center-to-center triangular spacing and a column diameter of 3.25 ft. Stone columns extended through the working platform to bedrock. The working platform consisted of 2 ft of stone separated from the foundation by a geotextile fabric. The stone column layout is shown in Figure 2.

A stress concentration ratio of 2.0 for short and long term conditions was assumed for the stone column design. The stability analysis of the stone column treated foundation performed by the Division of Materials yielded the following results:

South Wall

Short-Term Safety Factor = 0.9 (without stone columns)
Long-Term Safety Factor = 1.3 (without stone columns)

Short-Term Safety Factor = 1.4 (with stone columns)
Long-Term Safety Factor = 1.4 (with stone columns)

North Wall

Short-Term Safety Factor = 1.5 (without stone columns)
Long-Term Safety Factor = 2.1 (without stone columns)

Short-Term Safety Factor = 2.3 (with stone columns)

Long-Term Safety Factor = 2.3 (with stone columns)

The settlement analyses of the walls yielded the following:

South Wall

9.4 in. in 1.4 years - without stone columns
6.0 in. in 35 days - with stone columns

North Wall

13.0 in. in 5.2 years - without stone columns
8.0 in. in 46 days - with stone columns

Times and magnitudes of settlement are for 90 percent consolidation. The stone column treated foundation design is shown in Appendix A. Typical centerline sections at the bridge abutments are shown in Figures 3 (Abutment 1) and 4 (Abutment 2).

Wick drains were installed on 6-ft triangular spacing and extended through a 2-ft drainage blanket to bedrock. The wick drain treated areas extended from the stone column treatment at Abutment 1 (approximately centerline Station 2016+30) south to Station 2005+00 and from the stone column treatment at Abutment 2 (approximately centerline Station 2019+70) to Station 2035+00. The treated area typically extended from about centerline to 200 ft right of centerline. The wick drain treated area is shown in Figure 5 and a typical cross section is shown in Figure 6.

INSTRUMENTATION

Two groups of instrumentation were used in this study. One group was for monitoring the bridge approach foundations. The other instrumentation was for monitoring the reinforced soil walls and stone column treated foundation. The focus of the first group was to monitor the magnitude and time required for consolidation of the approach foundations. This group of instrumentation included pneumatic piezometers, settlement gages, and settlement platforms. The piezometers were installed near centerline at Station 2007+00 and were gradually shifted to the right of centerline to a maximum of 55 ft right at Station 2028+50. Piezometer locations are listed below.

Piezometer No.	Station	Elevation(ft)	Depth(ft)*
1	2007+00	444.2	10
2	2010+00	444.5	10
3	2013+00	449.9	5
4	2015+00	449.3	5
5	2021+50	445.5	9
6	2024+50	445.4	9
7	2026+50	444.7	10
8	2028+50	445.1	10
9	2007+00	429.2	25
10	2010+00	429.5	25
11	2013+00	429.9	25
12	2015+00	431.3	23
13	2021+50	435.5	19
14	2024+50	435.4	19
15	2026+50	434.7	20
16	2028+50	435.1	20

* Depth is the distance in feet from the foundation-embankment interface.

All piezometers were installed by April 1985. Piezometers were installed after clearing and grubbing but before other construction or wick drain installation began. The piezometer pressure lines were extended through the drainage blanket and embankment to exit at the toe of the embankment.

Mercury-filled settlement gages and settlement platforms were installed to monitor foundation settlement. Both were placed on the foundation before the embankment was constructed. Settlement gages were located at Station 2010+00, in the south approach, and Station 315+60 of Ramp C, in the north approach. The settlement gages are multipoint with the end point at centerline and other points extending right of centerline approximately 25 ft. Settlement platforms were located at Station 2008+00 (140 ft right), Station 2013+00 (140 ft right), Station 310+00 (10 ft right, Ramp C), and Station 315+00 (10 feet right, Ramp C).

Instrumentation for the retained earth walls and stone columns included slope inclinometers, settlement gages, earth pressure meters, and settlement monitoring points. Monitoring points were located on centerline and 75 ft east of centerline on each wall. The slope inclinometers were located directly in front of the walls at 50 ft right of Station 2017+15 (south wall) and on centerline at Station 2019+35 (north wall).

Settlement gages were located on the working platform at the south retained earth wall. Earth pressure meters were located at both walls. Four meters were placed at the north wall with two being between stone columns and two were centered on the stone columns. Two meters were placed at the south wall with one located between stone columns and one centered on a stone column.

CONSTRUCTION SCHEDULE

Construction began in December of 1984 but due to utility relocation and construction phasing little embankment construction was performed at the bridge or bridge approaches until mid 1985. Clearing and grubbing was mostly completed by March 1985. Piezometers were installed at the south approach in the first week of March and at the north approach in the last week of April. Wick drain installation was complete at the south approach in July 1985. Wick drain installation was complete at the north approach in September 1985.

Settlement platforms and the settlement gage at the south approach were installed in June. Settlement platforms and the settlement gage were installed at the north approach in May 1985. Embankment construction followed wick drain and drainage blanket construction. Embankment construction began at Station 2013+00 on July 30 and proceeded south. The southern embankment was essentially to grade by mid September. Construction of the north approach embankment was begun and completed in September 1985.

Stone column construction at the north abutment wall began in December 1985 and was completed in January 1986. Reinforced soil wall construction for the north abutment began on January 23, 1986 and was essentially complete in two weeks. The stone columns for the south abutment were completed by February 4, 1986. The reinforced soil wall for the south abutment was begun in the last week of February and essentially completed in three weeks.

CONSTRUCTION PROCEDURES

Typical wick drain installation was to place a geotextile fabric on the foundation and place two feet of stone on the fabric. The wick drain locations were marked on the stone drainage blanket. Wicks were forced into the foundation to bedrock and cut off above the stone drainage blanket. After the wicks were completed, a second layer of fabric was placed on the stone to complete the drainage blanket. The fabric limited the intrusion of fine particles into the drainage blanket. The north approach foundation included an old landfill. It was necessary to drill holes for the wick drains in the north foundation.

Typical stone column construction was to first place geotextile fabric and stone on the foundation for a working platform. Stone column locations were marked on the working platform and holes were drilled. A vibratory probe having water jets at the tip was then used to enlarge the hole and compact stone in the hole to construct the columns. When the stone columns were complete at an abutment, the working platform was brought back to design thickness and the reinforced soil wall was constructed.

SOUTH ABUTMENT WALL FAILURE

Construction of the south abutment reinforced soil wall began on or about February 20, 1986. The wall was approximately 30 ft high and was essentially complete in three weeks. A slope inclinometer indicated lateral movement of the foundation during construction; and by March 14, approximately three weeks after wall construction began, the reinforced wall and backfill had moved laterally as much as 5 ft and vertically as much as 1.5 ft. The wall tilted backward, toward the embankment, as much as 1 ft from vertical. The wall and backfill remained intact and moved away from the shale embankment. Large tension cracks developed at the backfill - embankment interface. Photographs of the failed wall are shown in Figures 7 through 10. Figure 7 shows the site, as viewed from the northeast, during construction of the stone columns. Figure 8 shows the toe of the wall and the upthrust foundation in the Northern Ditch. Figures 9 and 10 show the failed wall and embankment with the metal straps of the reinforced earth structure exposed in the fissure in Figure 9.

The south wall was dismantled and the backfill was removed. Undisturbed soil samples were obtained and insitu vane shear tests were conducted. Additional instrumentation, piezometers and slope inclinometers, were installed at the north wall to more closely monitor it. A trench was excavated in the failed south foundation to observe the stone columns.

POST FAILURE ANALYSIS

The Geotechnical Section of the Division of Materials conducted an extensive post failure investigation. Due to the similarities of foundation soils from the north and south abutment areas, north abutment data were used in conjunction with south abutment data in the analysis. A summary of their findings is discussed in the following paragraphs.

Settlement data for the north wall indicated 90 percent consolidation occurred approximately 120 days after the wall was begun rather than the 46 days predicted. This was attributed to the coefficient of consolidation of post failure foundation soil samples being much lower than the value used in stone column design. It is also possible that drilling the columns smeared the walls of the stone columns and reduced the horizontal drainage component C_h. It was concluded that a combination of less than expected soil permeability, reduced C_h, water added to the foundation by jetting during the wet method column construction, vibration during column construction, and rapid wall construction probably led to failure of the wall.

Vane shear tests were performed in the south abutment foundation after the wall and backfill had been removed. These tests indicated undrained shear strengths of 1,500 psf above elevation 445 ft, 920 psf between elevations 435 ft and 445 ft, and 1,500 psf below elevation 435 ft. This corresponds to the foundation layers observed in the predesign subsurface exploration. These tests indicate generally improved foundation shear strengths but a weaker middle layer relative to the rest of the foundation.

Laboratory tests conducted on foundation samples collected after the failure indicate high liquidity indices and silt contents between elevations 430 ft and 435 ft. Silt content averaged 52 percent above elevation 435 ft and 54 percent below elevation 430 ft. Between elevation 430 ft and 435 ft the silt content averaged 66 percent with a high of 77.7 percent. Between elevations 430 ft and 435 ft the liquidity index averaged 1.0 with a high value of 2.6.

A sample of the stone column aggregate contained approximately 14 percent finer than the No. 100 sieve.

FIELD DATA

Wick Drained Foundation

Settlement platform data indicate foundation settlement of 12.5 in. and 7.0 in. at Stations 2013+00 and 2008+00, respectively. Ninety percent settlement occurred in approximately 90 and 50 days after the embankment was completed at these respective locations. Settlement and embankment height are plotted versus time in Figures 11 and 12.

Piezometers 1 through 4 are located at depths of 5 to 10 ft in the wick drained foundation south of the bridge. Piezometers 9 through 12 are located at depths of 20 to 25 feet in the in the same approach foundation. All piezometers show foundation pore pressure changes that appear to reflect construction activity. All piezometers that operated past the time the embankment was completed indicate foundation pore pressure returned to preconstruction levels except at Piezometer 4. South approach foundation piezometer data are shown in Figures 13 and 14.

Piezometer 4 is located 20 ft right of Station 2015+00. This places Piezometer 4 approximately 125 ft from the nearest stone column. Foundation pore pressure at this location rose as the embankment was constructed and remained at a higher level than at other locations. Piezometer 4 ceased to operate approximately 2 weeks before stone column construction began for the south abutment, but it is probable that excessive foundation pore pressure existed when stone column construction began.

While pore pressure generally returned to near preconstruction levels, pore pressure at piezometers nearest the surface and near the boundaries of the wick drained areas tended to be marginally higher than in the center of the drained area.

Foundation settlement at the north approach was 11.2 in. at Station 310+00 and 4.2 in. at Station 315+00. Ninety percent settlement occurred approximately 60 days after the embankment was completed at Station 310+00 and 20 days at Station 315+00. Embankment height and foundation settlement for both locations are plotted versus time in Figures 15 and 16. A settlement gage at Station 315+60 indicated settlement of 4.5 in. at centerline.

Foundation pore pressure in the north approach followed the same pattern as in the south approach foundation. All piezometers, except Piezometer 5, indicated pore pressure increases with embankment construction and pore pressure returning to initial levels or lower after the embankment was complete. Piezometer 5 is the nearest piezometer to the north abutment stone columns. Piezometer 5 indicated an elevated pore pressure until it ceased operating in May 1986. The north reinforced soil wall was not removed until September 1986. North approach piezometer data are shown in Figures 17 and 18.

<u>Reinforced Soil Wall</u>

Settlements of the retained earth walls were monitored by surveying the settlement points placed on the walls. Insufficient data were obtained prior to failure from the settlement gages placed on the working platform at Abutment 1 to permit measurement of settlement. Settlement points indicated wall settlement of 5.25 in. for the north wall and 2 in. (prior to failure) for the south wall. Settlement of the south wall after failure was approximately 18 inches.

Horizontal displacements of the walls were monitored with slope inclinometers placed adjacent to the walls. Inclinometer 1 (north wall) indicated 3.3 in. of movement at a depth of 10 ft at 240 days after installation. This inclinometer was installed January 22, 1986 and approximately 2.3 in. of movement occurred in 40 days. Lateral movement at Slope Inclinometer 1 is shown in Figure 19.

Slope Inclinometer 2 (south wall) was installed on February 21, 1986. The last data collected (March 14, 1986) before failure indicated lateral movement of nearly 16 in. Lateral movement at Slope Inclinometer 2 is shown in Figure 20.

Earth pressure meters indicated very little stress concentration in the stone columns. Meters at the north wall indicate pressures of 16 and 17 psi on stone columns and 14 psi between columns. Meters at the south wall indicate the stone column temporarily carried approximately 3.0 psi more pressure than the surrounding soil, but near the time of failure, the stone column had less load, approximately 1.0 psi, than the soil. Earth pressure data for the north and south abutments are shown in Figures 21 and 22, respectively.

REMEDIAL ACTION

After analyzing field data and laboratory tests from the north wall and failed south wall, the Geotechnical Branch of the Kentucky Department of Highways recommended several alternatives for remedial action. Two of the recommendations involved the south wall only. One was the complete removal of soil under the south wall and replacement with a rock-like black shale. The second was the reinstallation of stone columns with closer spacing and an enlarged reinforced soil volume behind the wall.

The analysis of the north wall indicated marginal stability and the probability of long-term settlement. A recommendation was made for the removal of the existing north wall, the addition of stone columns, the use of piling to support the bridge loads, and reconstruction of the north wall. Due to time and construction clearance constraints, a recommendation for the removal of both walls, the addition of a 100-ft span to each end of the bridge, and flattening the spill through slopes was made and ultimately accepted.

Construction of the bridge and approaches was completed in December 1988. Final costs of the chosen alternative was approximately $2.7 million plus a claim paid to the contractor of $1.3 million for costs incurred in removal of the original structures and embankment.

CONCLUSIONS

Wick drain treatment of the approach foundations appears to have been successful. Most piezometers indicate a return to near preconstruction foundation pore pressure soon after embankment construction was completed. Piezometers 4 and 5, which are the nearest piezometers to the south and north walls, respectively, indicate elevated foundation pore pressures continuing after the embankments were complete.

It is possible that a small area, approximately 20 ft, between the wick drain treatment and stone column treatment developed high pore pressures due to placement of the embankment. The elevated pore pressure was reflected in the nearest piezometers and could have caused elevated pore pressure in the abutment foundation prior to stone column construction. Evidence of this was the observance of water and fine soil bubbling from the stone columns prior to construction of the reinforced soil retaining wall.

Post failure field vane shear tests conducted in the south abutment foundation indicated a soil layer with an undrained shear strength of 920 psf or about 60 percent of the shear strength of the surrounding soils. Unconsolidated undrained triaxial tests indicated effective cohesion and friction angle of 500 psf and 5 degrees, respectively, in this layer. This layer also had higher silt content and moisture content than expected. The wet method of constructing the stone columns introduced additional water and use of the vibrating probe further disturbed the sensitive foundation layer.

The combination of a foundation soil layer of relatively low shear strength, slower than anticipated consolidation of the foundation, and rapid construction of the reinforced soil wall resulted in failure of the wall structure. Rapid construction of the wall elevated foundation pore pressure, as evidenced by water flowing from the slope inclinometer and data from nearby piezometers, to the point where some lateral movement occurred. A little lateral movement, 2 to 3 in., allowed tension cracks to develop and separate the wall structure from the more stable shale embankment.

Earth pressure meter data indicated the stone columns did not carry significantly more load than the soil foundation. The fact that the columns did not carry additional load means that the maximum shear resistance of the columns was not utilized. The primary benefits of stone columns appear to be the densification of the native soils resulting from the dynamic actions involved with construction of the columns, the replacement of native soils with stone, and increased drainage of the foundation. In this case, the siltation of the columns probably reduced the column drainage capability and shear strength significantly.

RECOMMENDATIONS

It is recommended, in the case of wick drained foundations, that drainage paths from the drainage blanket to external drainage systems be provided.

Detailed and extensive subsurface explorations would be beneficial at sites involving stone columns. Continuous sampling would allow more precise determination of the soil parameters and possible different soil types involved. Field tests such as permeability, vane shear, and dutch cone penetration should be conducted. Laboratory testing should include determination of soil strength and permeability.

Instrumentation for monitoring lateral movement, settlement, and pore pressure should be installed prior to construction of any stone column treated foundation. Pore pressure data from triaxial tests should be used to relate pore pressure to stability. Data from piezometers could then be used to control wall construction rate to maintain an adequate factor of safety.

Earth pressure data from this and another project (1) indicate little or no stress concentration in the stone columns. This body of data indicates that no stress concentration should be assumed for design purposes.

Stone columns can be an effective means of improving weak foundations. The material replacement and densification associated with stone columns permit deep foundation improvement. When flushed properly, stone columns can increase foundation drainage. Stone columns should continue to be considered for foundation improvement for future projects.

REFERENCE

1. Meade B. W.; and Allen D. L.; "EVALUATION OF STONE COLUMN STABILIZED EMBANKMENT FOUNDATION," Research Report KTC-89-58, Transportation Center, University of Kentucky, 1989.

Table 1. Results from consolidated-undrained triaxial tests

Location	Depth	Phi'	C'
	(ft)	(degrees)	(psf)
2008+00(100 ft. rt)	04-11	26.5	402
2008+00(100 ft. rt)	24-31	21.5	464
2013+05(38 ft. rt)	14-21	35.0	0
2016+75(60 ft. rt)	04-11	20.5	538
2016+75(60 ft. rt)	24-31	28.0	163
2019+32(51 ft. rt)	10-17	26.5	160
2019+32(51 ft. rt)	20-27	29.0	165
2028+00(87 ft. rt)	10-17	30.0	166

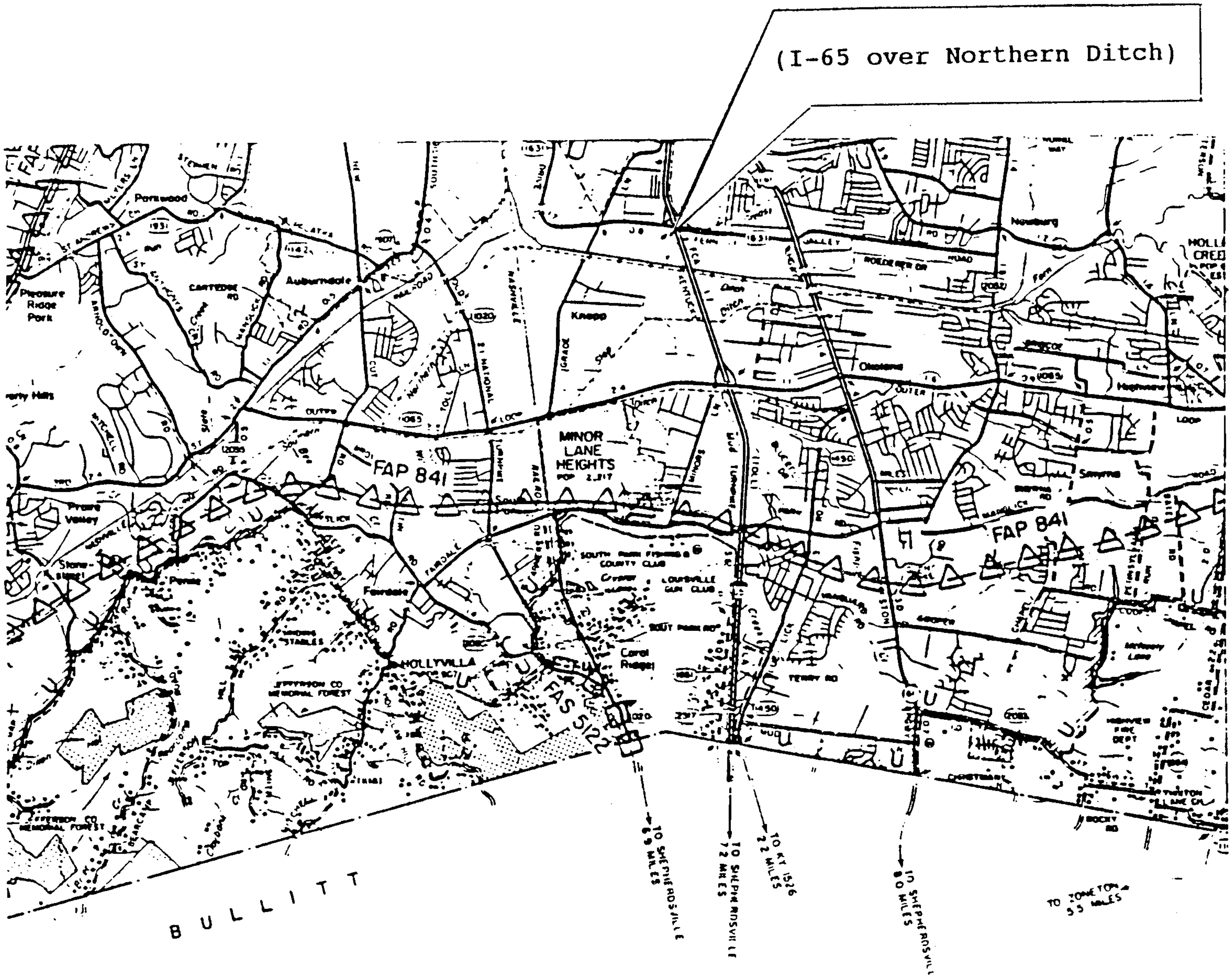

Figure 1. Ground Improvement Site (I-65 over Northern Ditch).

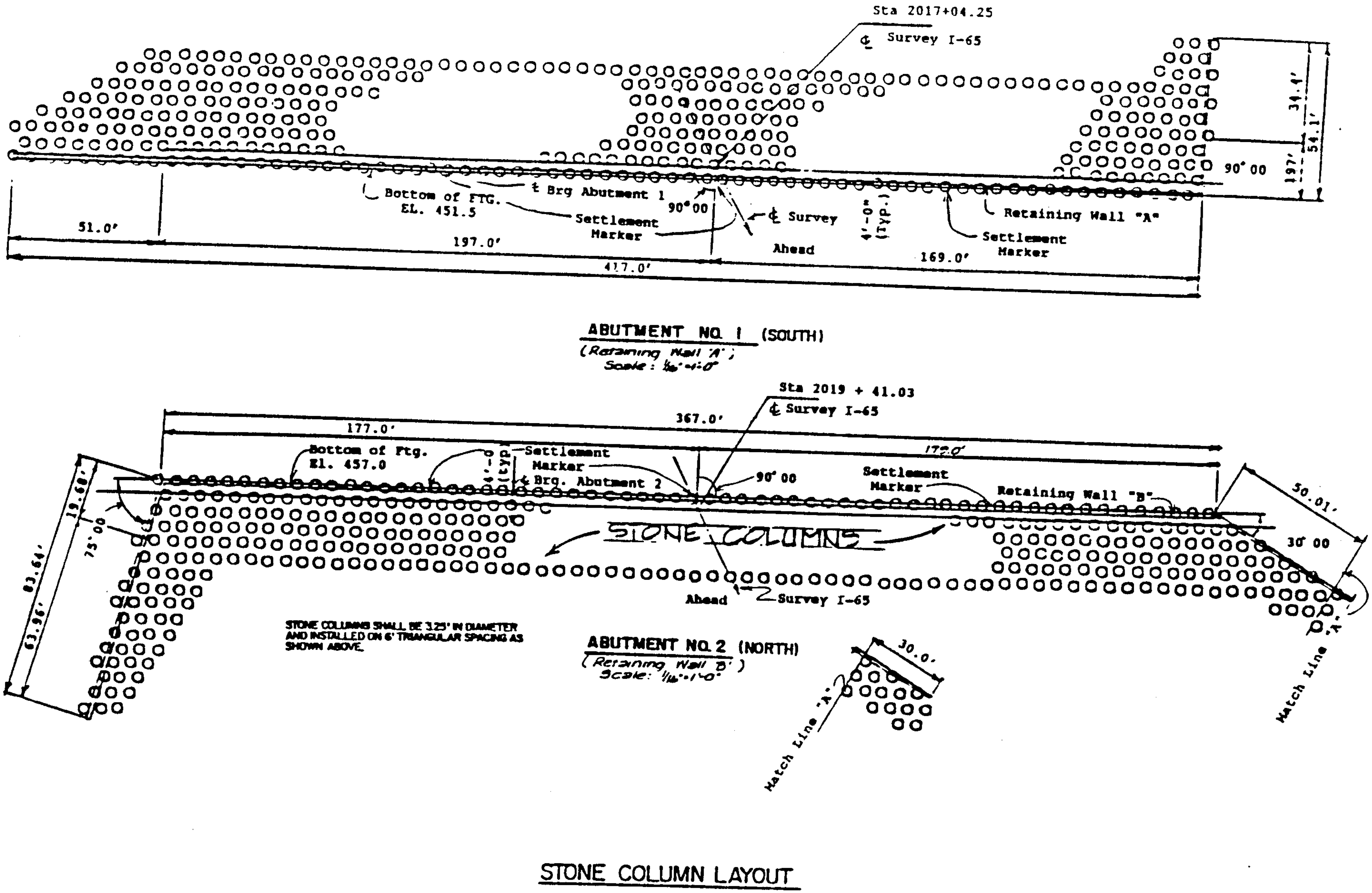

Figure 2. Stone Column Layout.

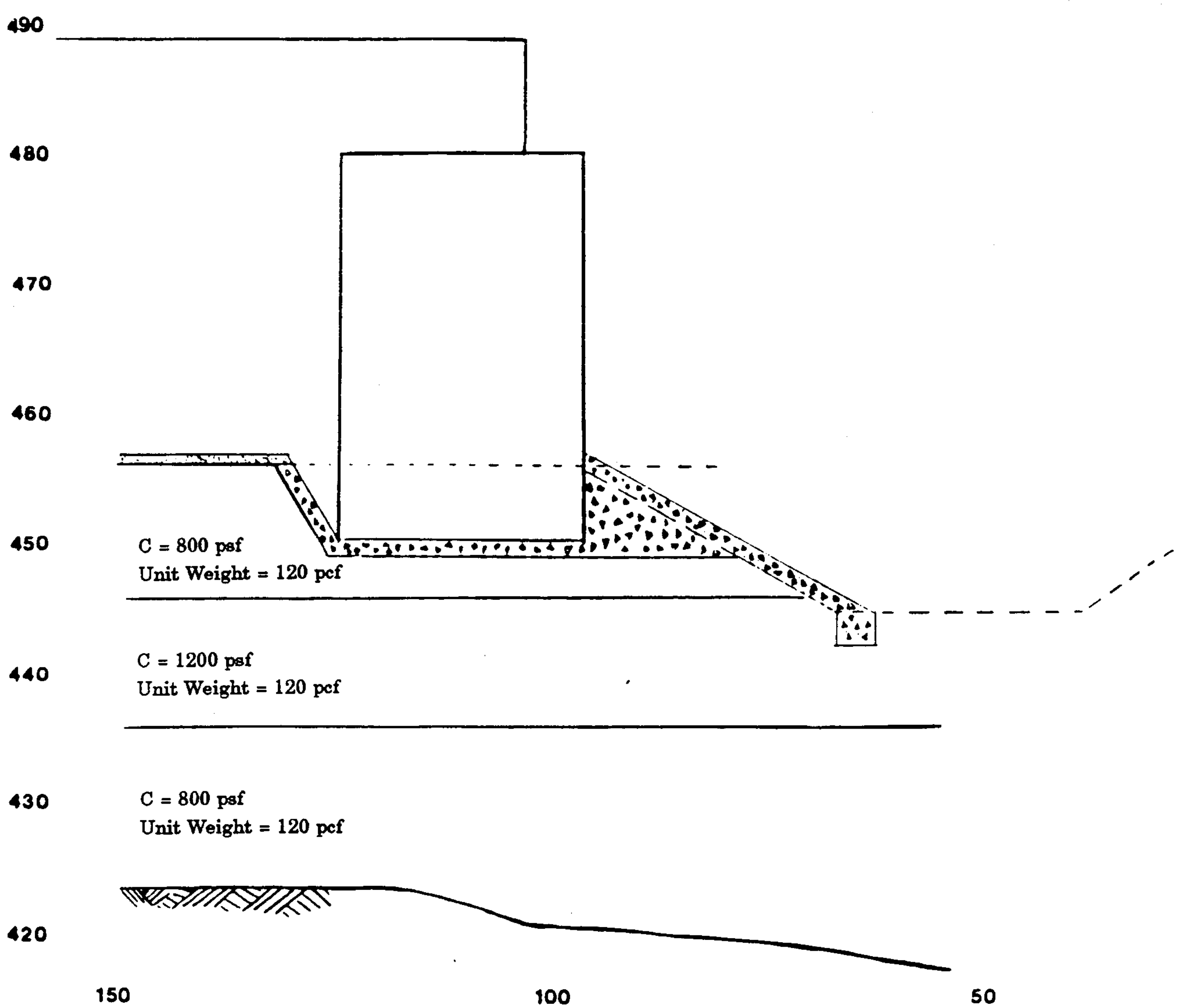

Figure 3. Centerline Section of Abutment 1 (South).

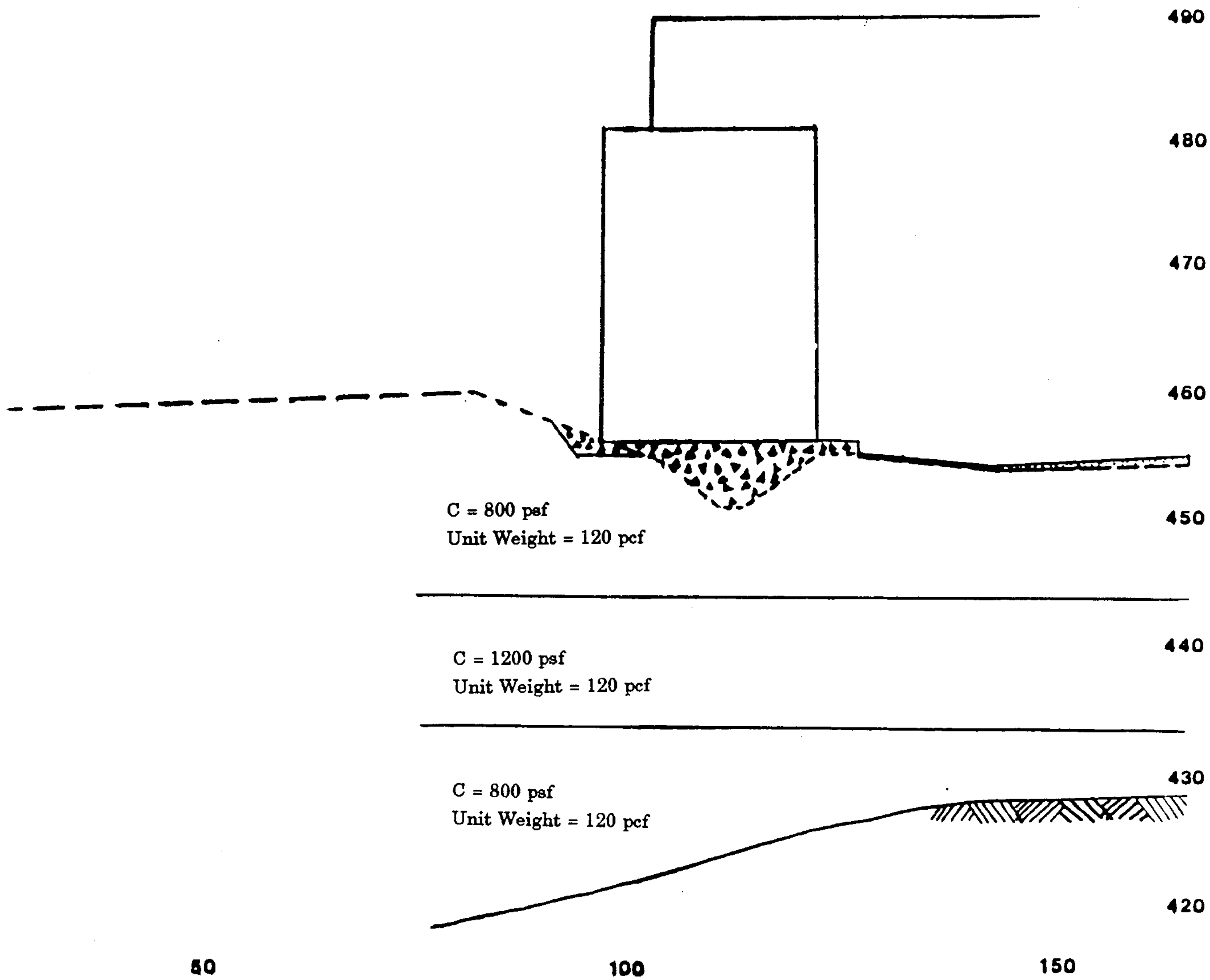

Figure 4. Centerline Section of Abutment 2 (North).

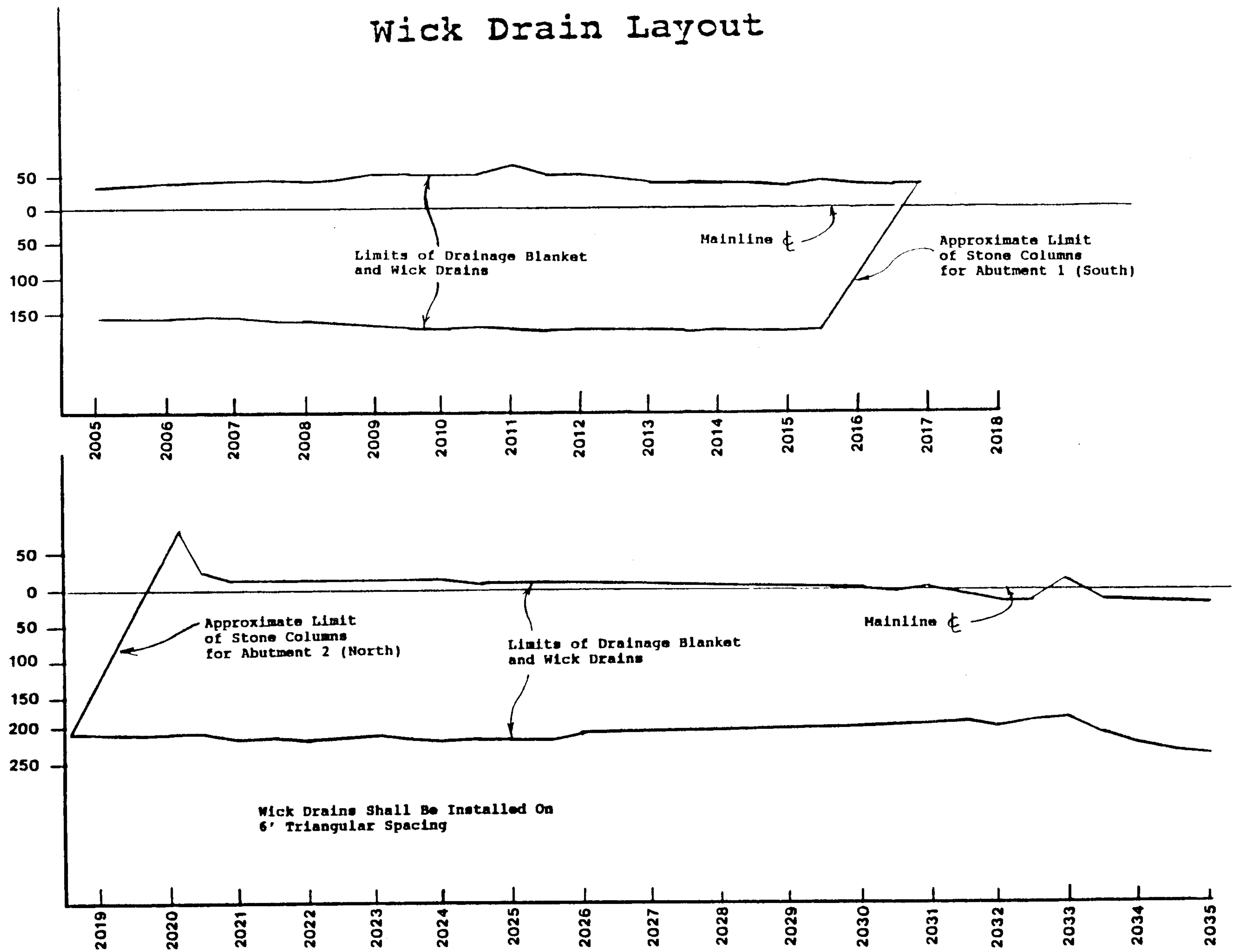

Figure 5. Wick Drained Area.

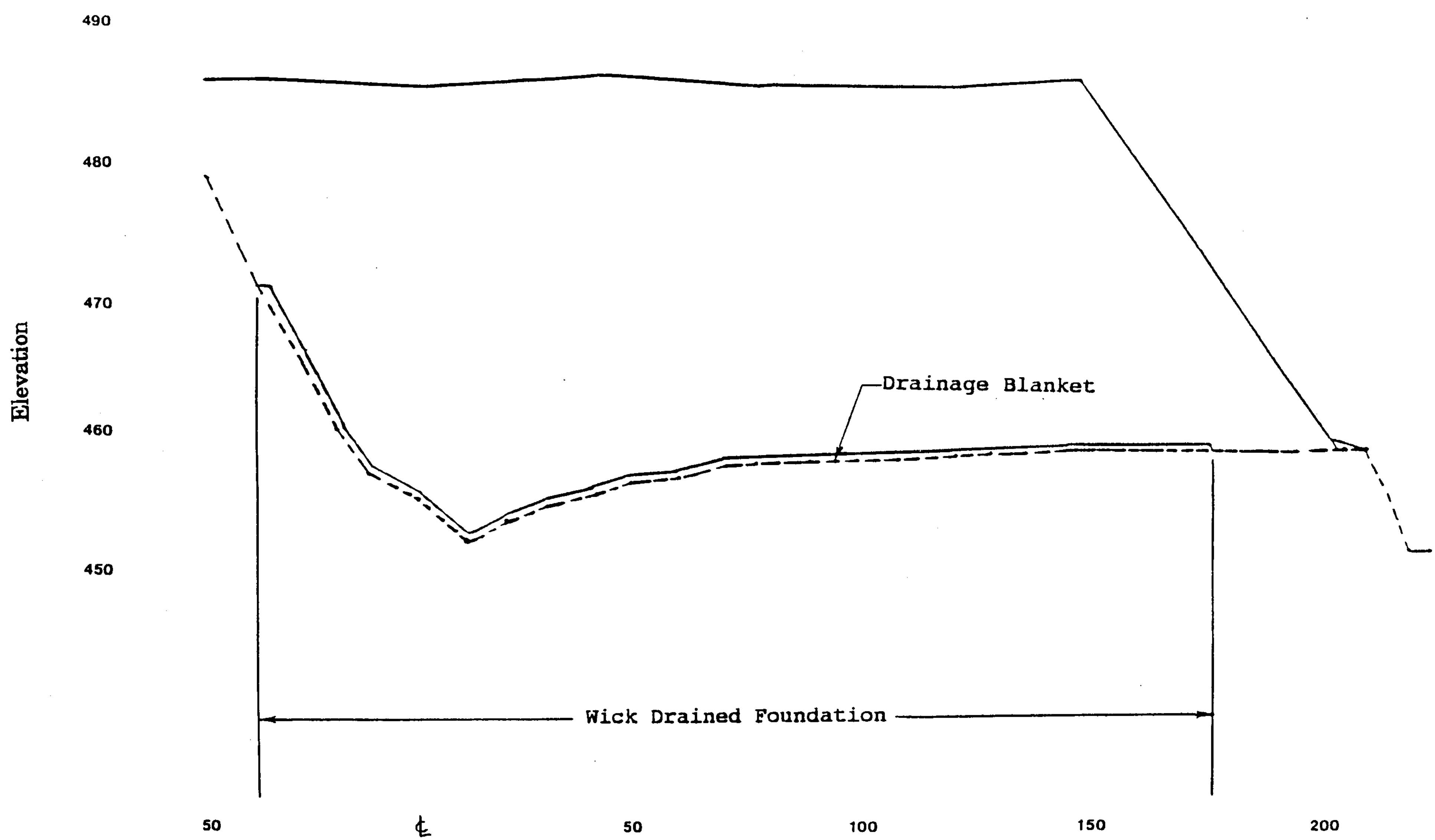

Figure 6. Typical Cross Section of Wick Drained Area.

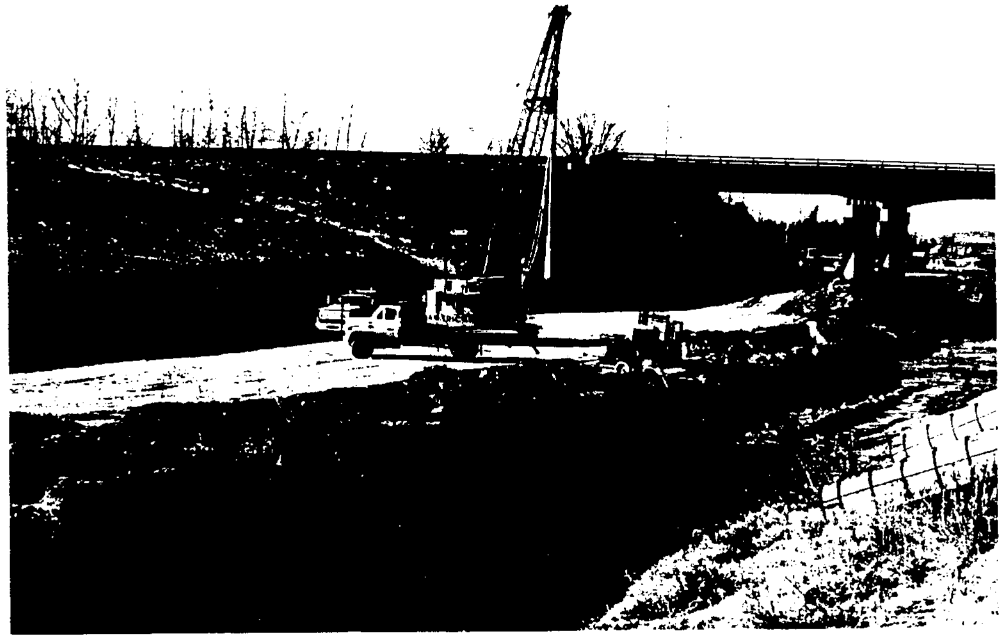

Figure 7. Abutment 1 Foundation.

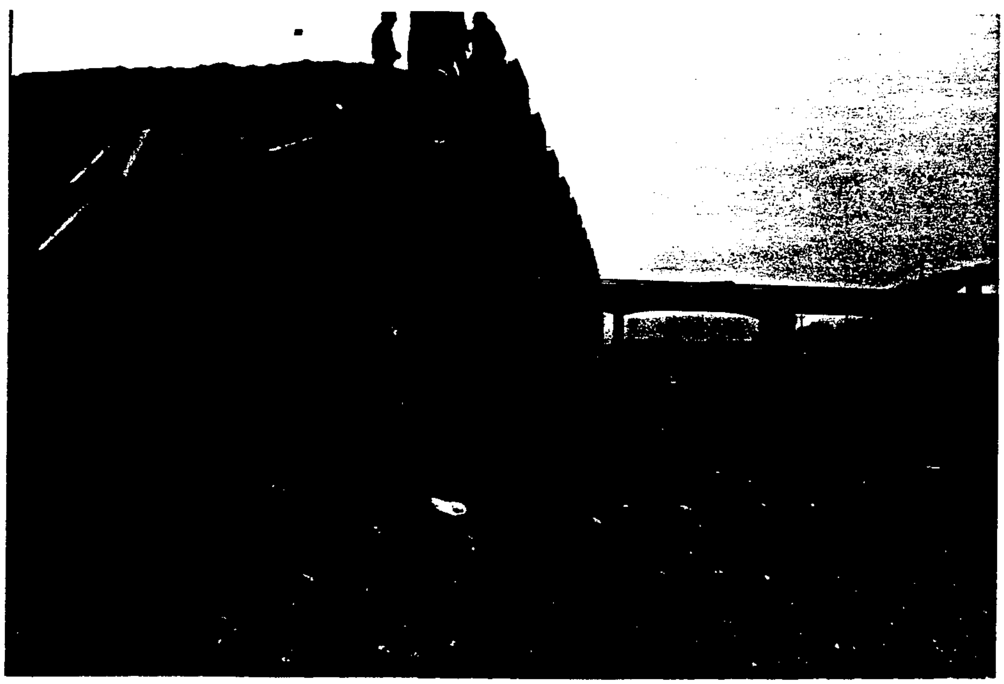

Figure 8. Toe of Failed Wall and Northern Ditch.

Figure 9. Failed Wall with Tension Cracks.

Figure 10. Failed Wall with Tension Cracks.

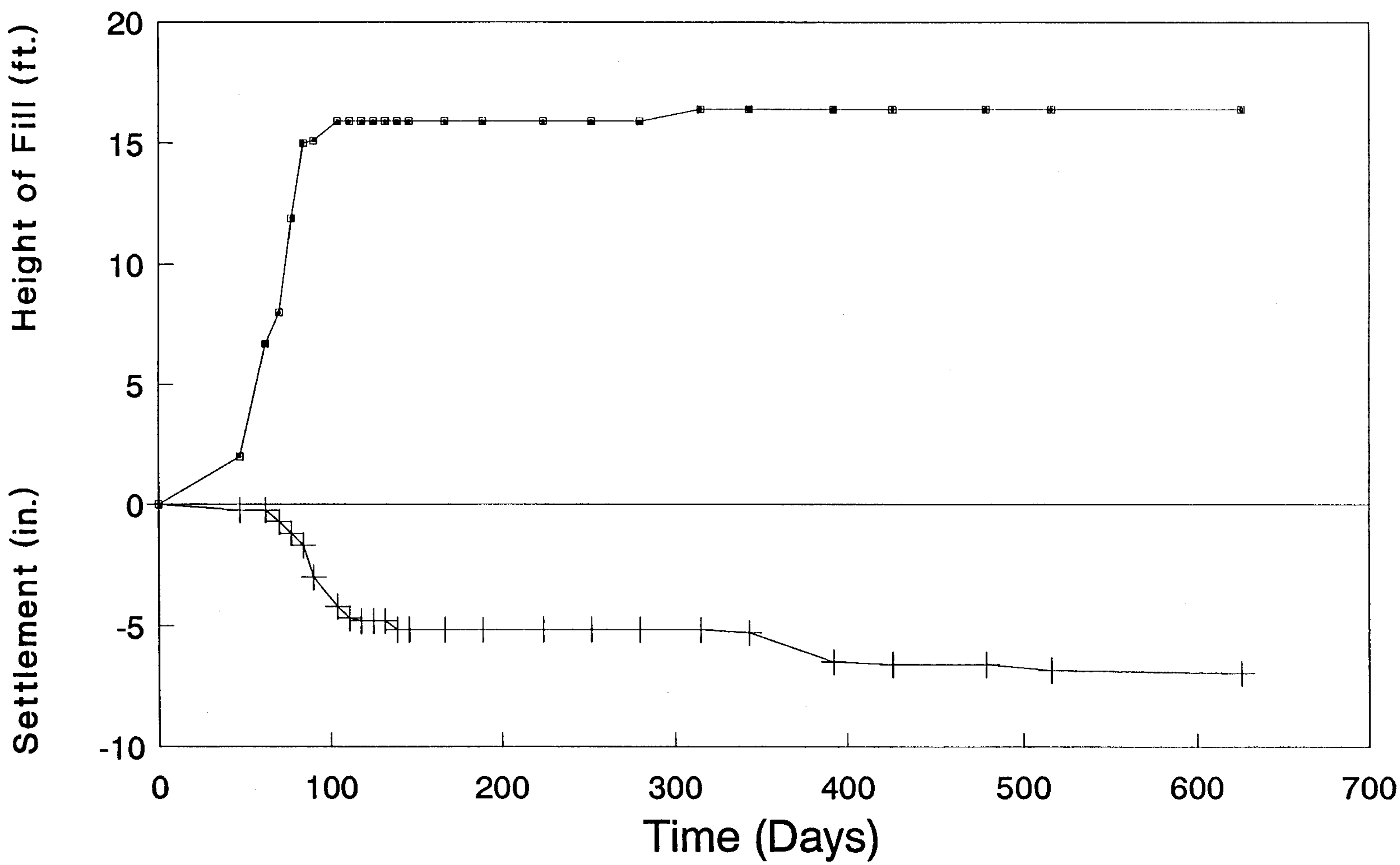

Figure 11. Foundation Settlement and Embankment Height at Station 2008+00.

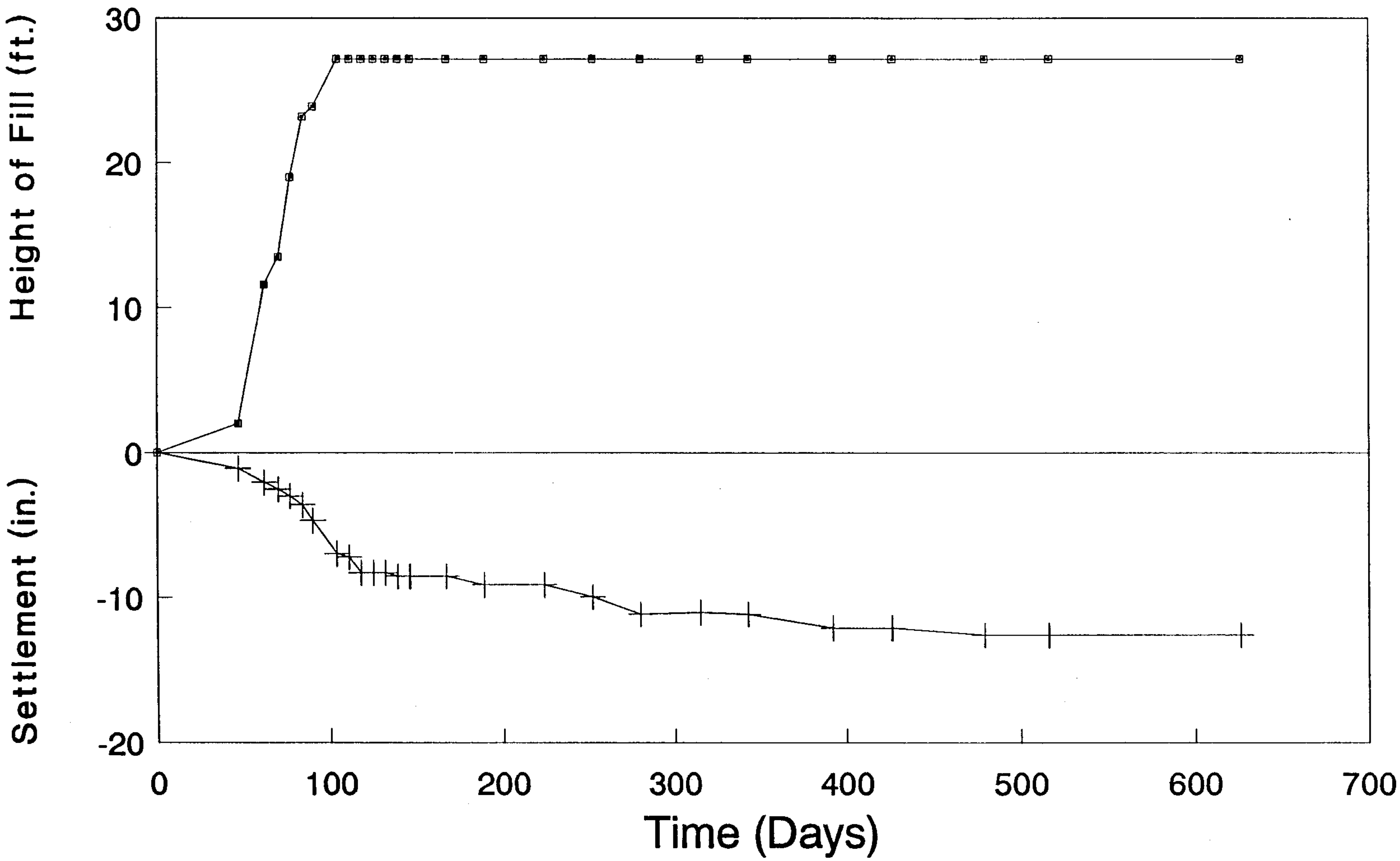

Figure 12. Foundation Settlement and Embankment Height at Station 2013+00.

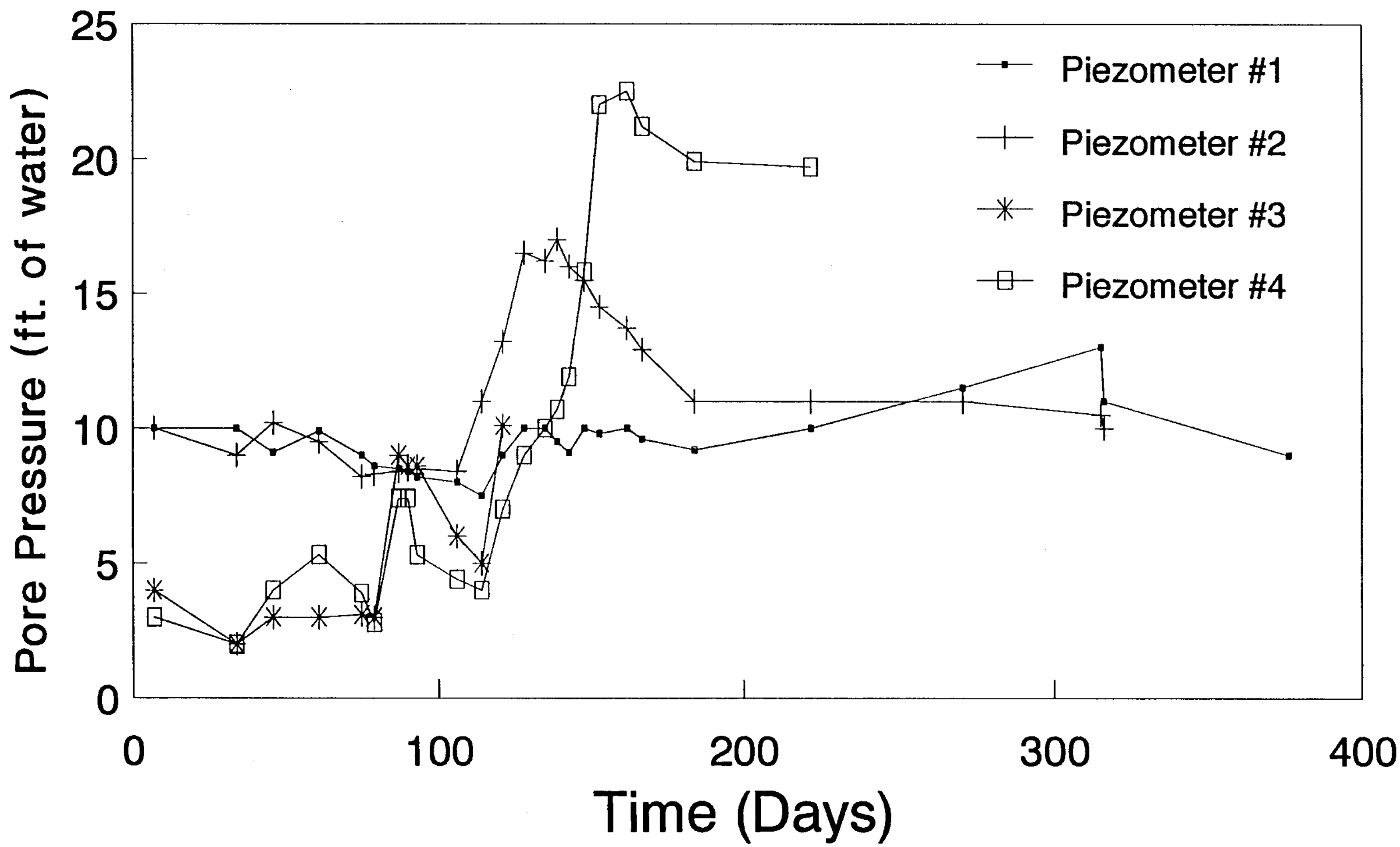

Figure 13. Foundation Pore Pressure at Station 2008+00.

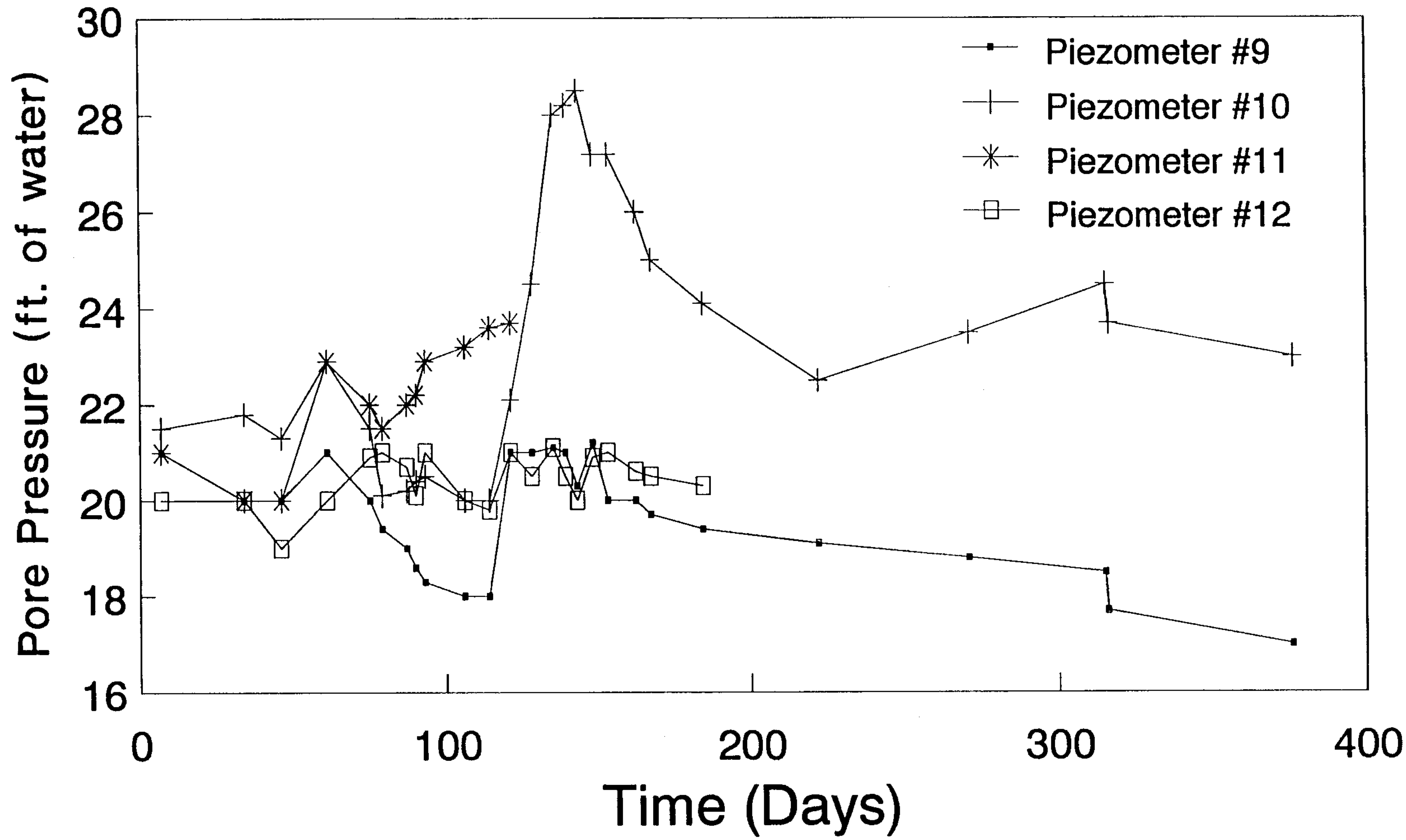

Figure 14. Foundation Pore Pressure at Station 2013+00.

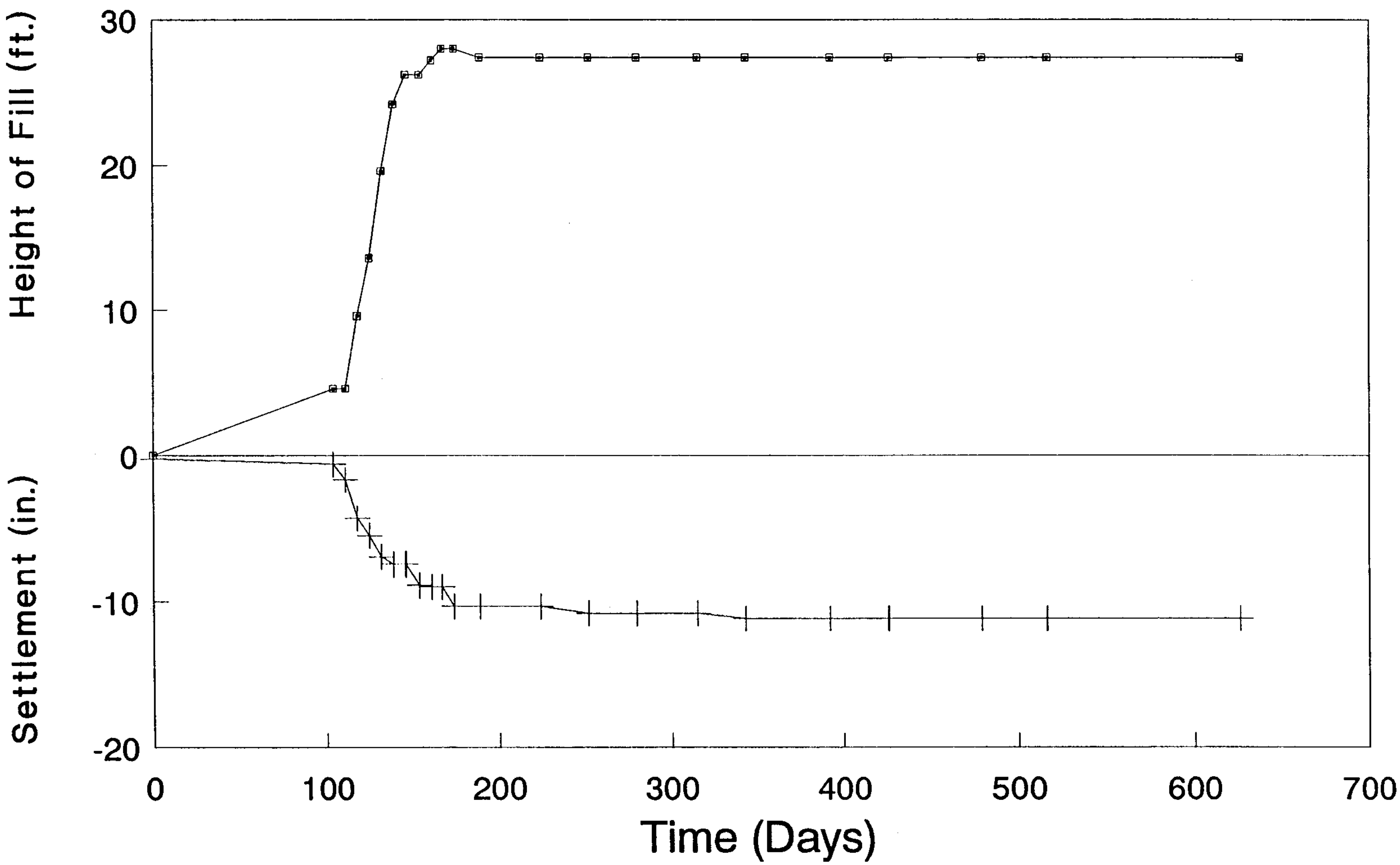

Figure 15. Foundation Settlement and Embankment Height at Station 310+00 (Ramp C).

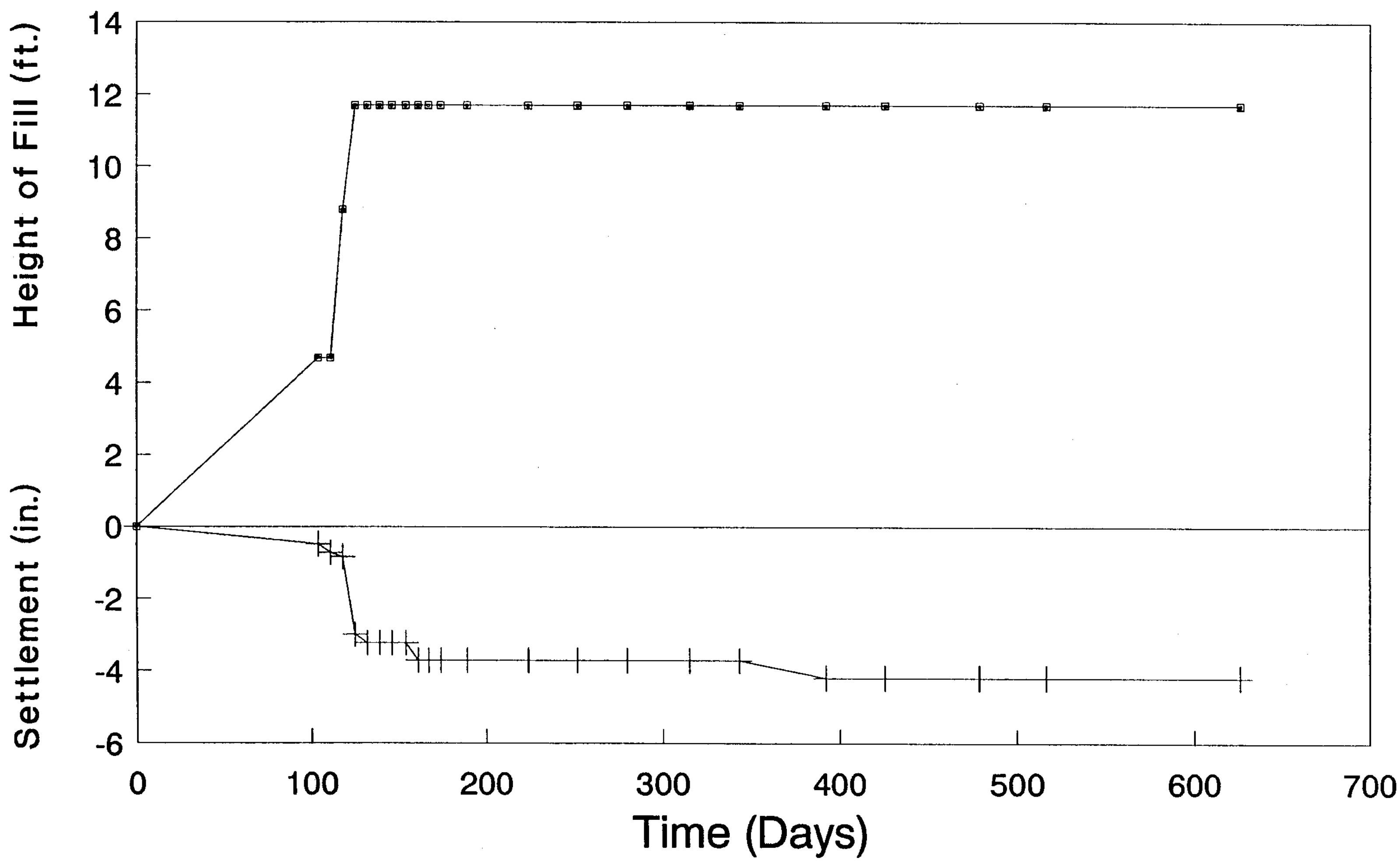

Figure 16. Foundation Settlement and Embankment Height at Station 315+00 (Ramp C).

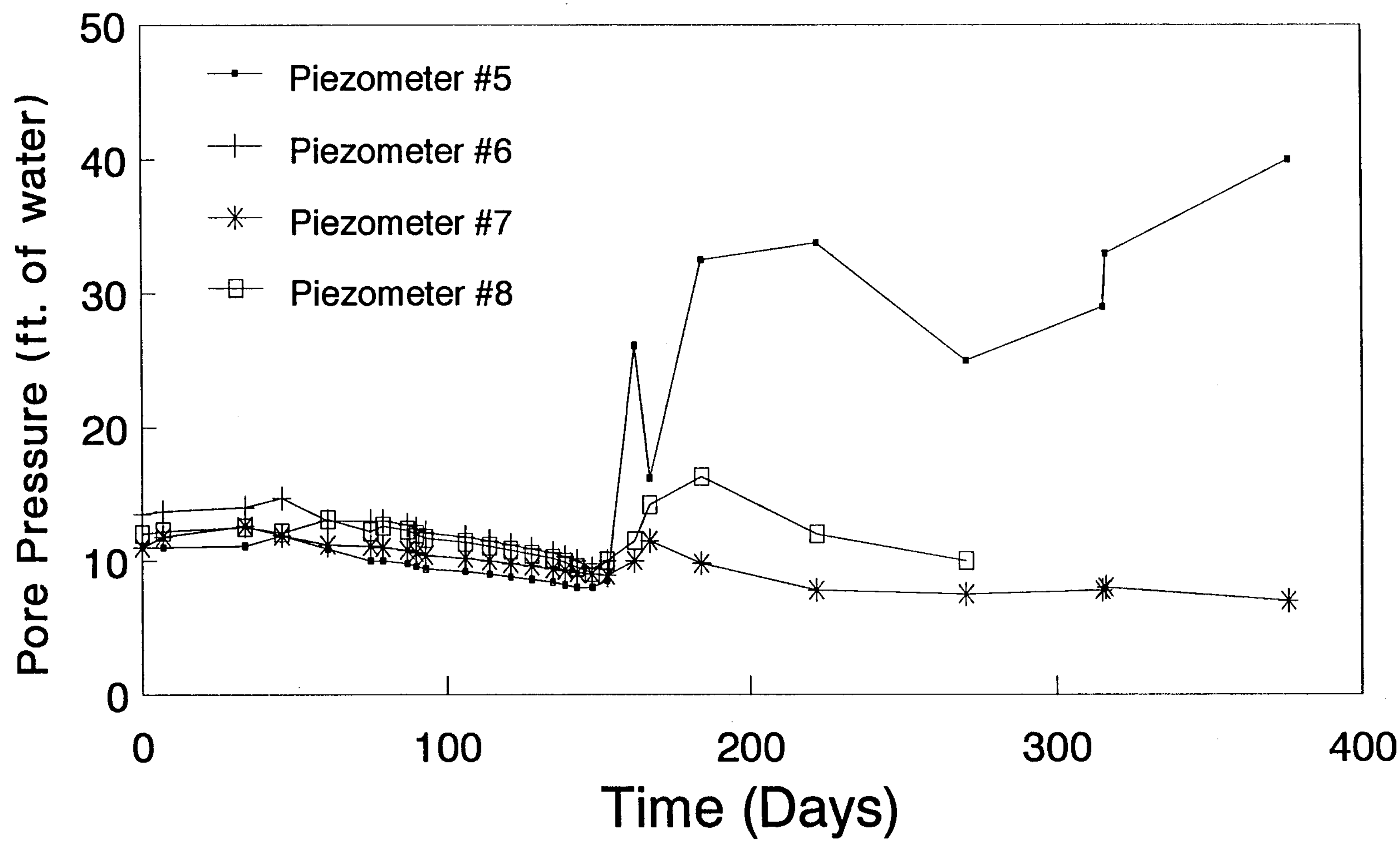

Figure 17. Foundation Pore Pressure at Station 310+00 (Ramp C).

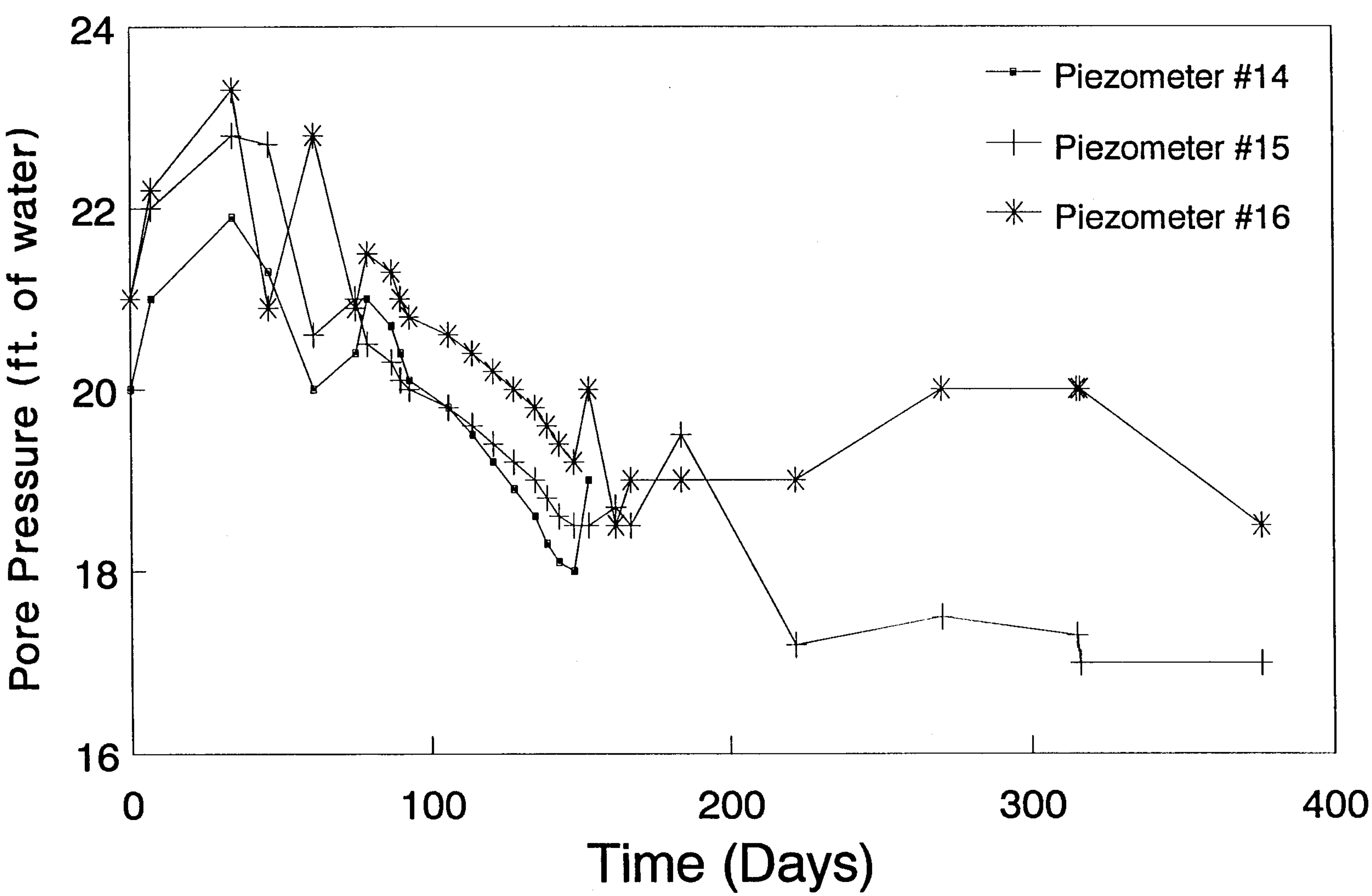

Figure 18. Foundation Pore Pressure at Station 315+00 (Ramp C).

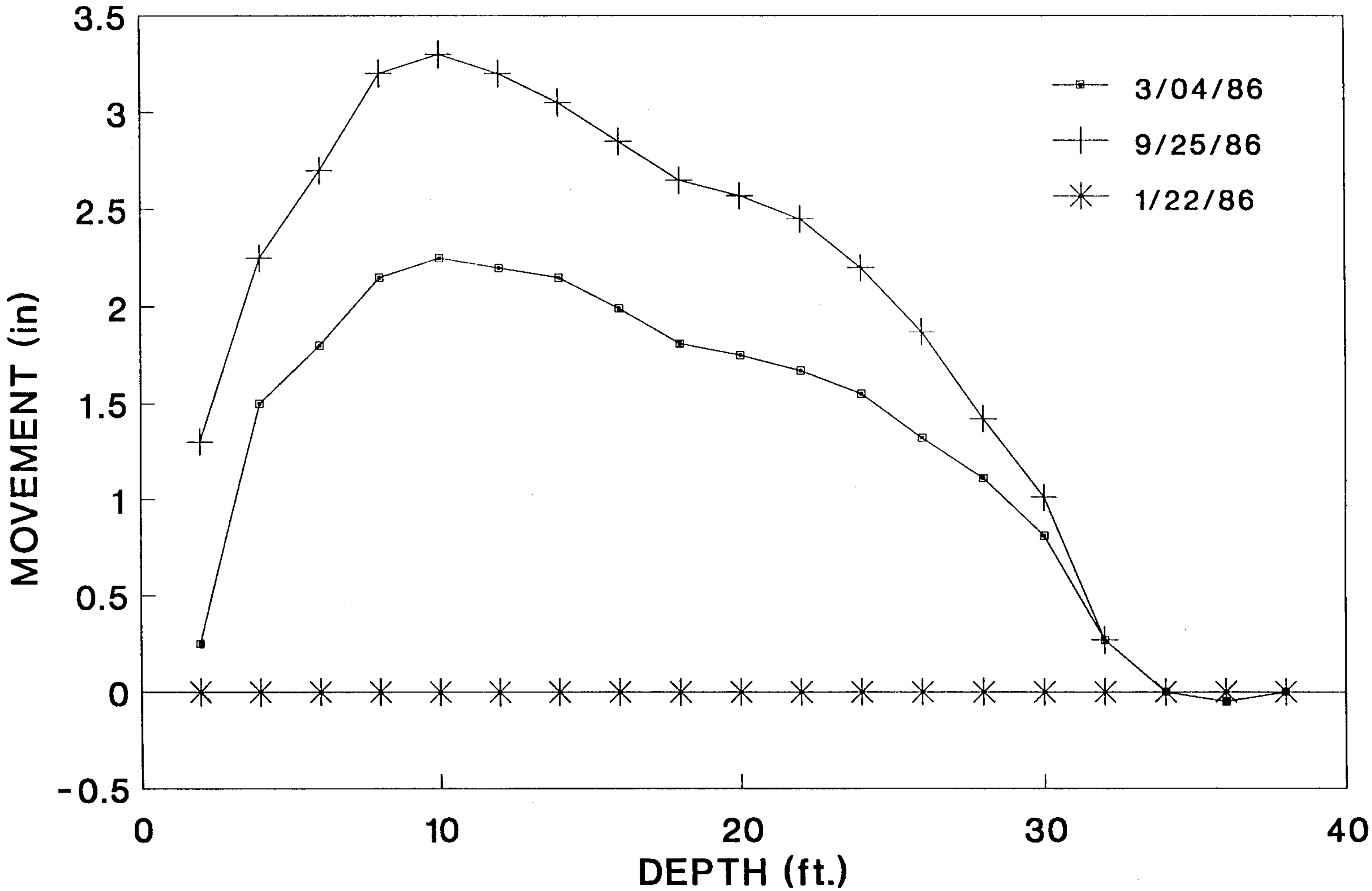

Figure 19. Lateral Movement (S.I.1) of Foundation at Abutment 2.

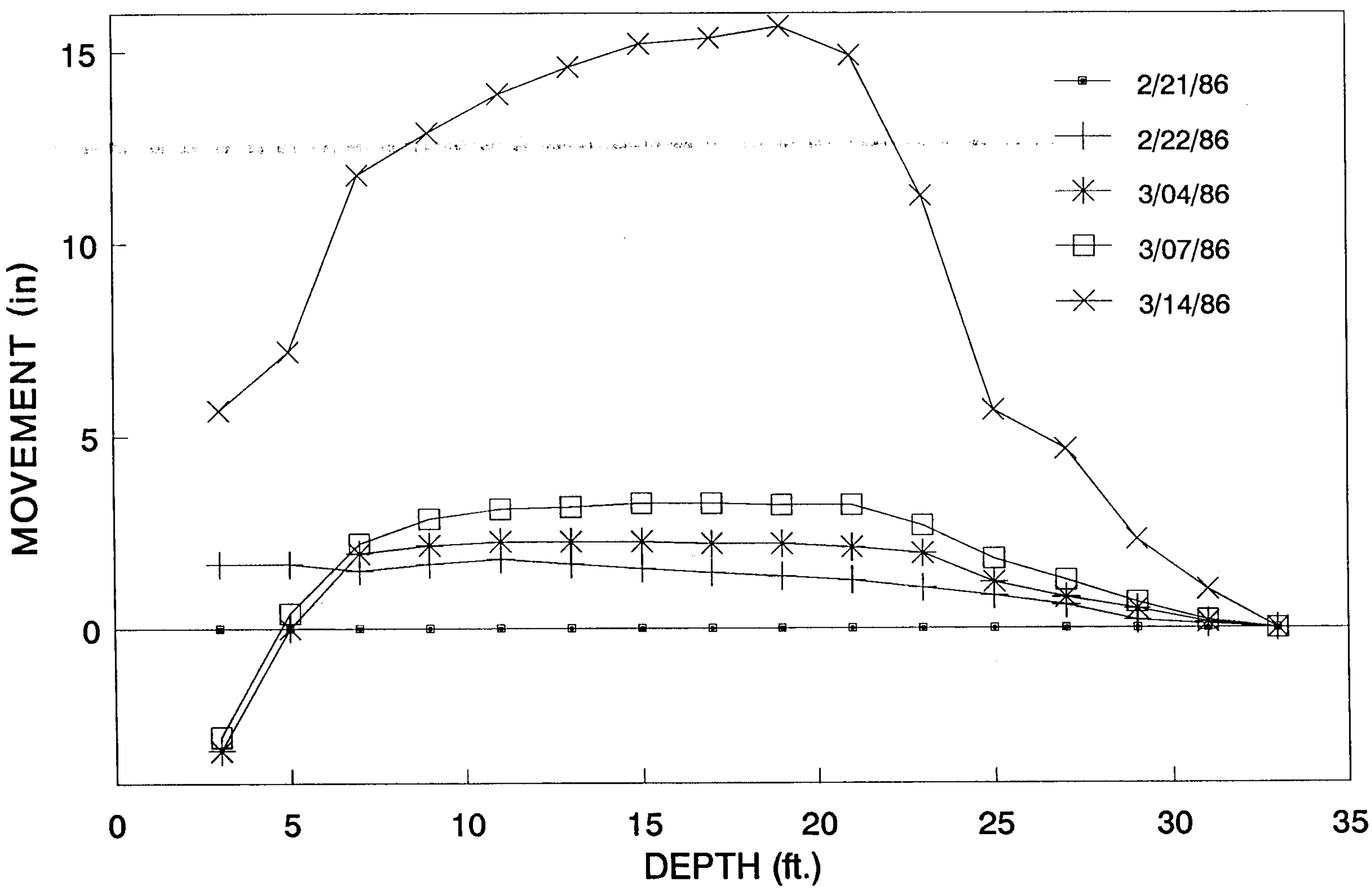

Figure 20. Lateral Movement (S.I.2) of Foundation at Abutment 1.

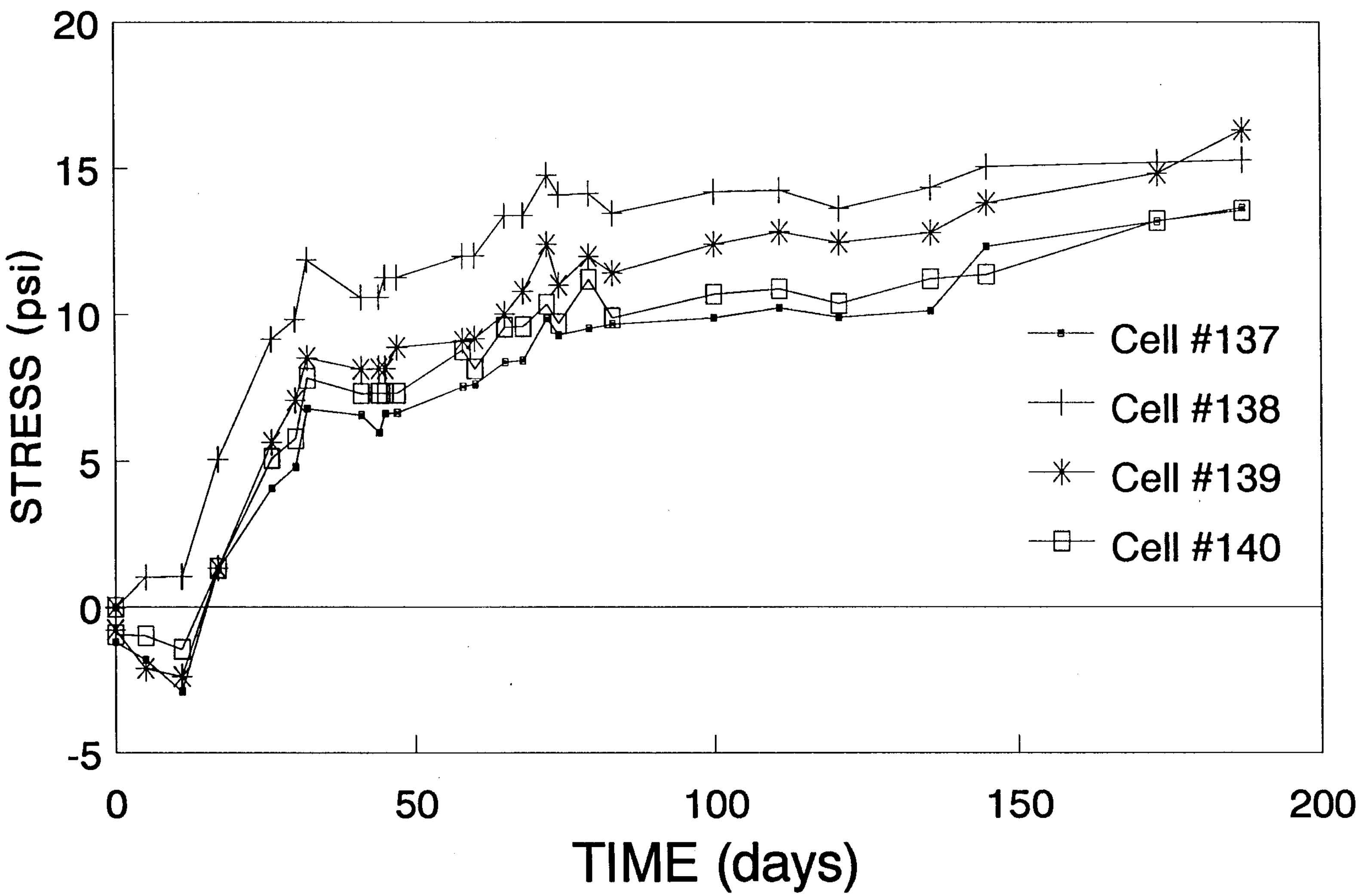

Figure 21. Earth Pressure Data on the Foundation at Abutment 1.

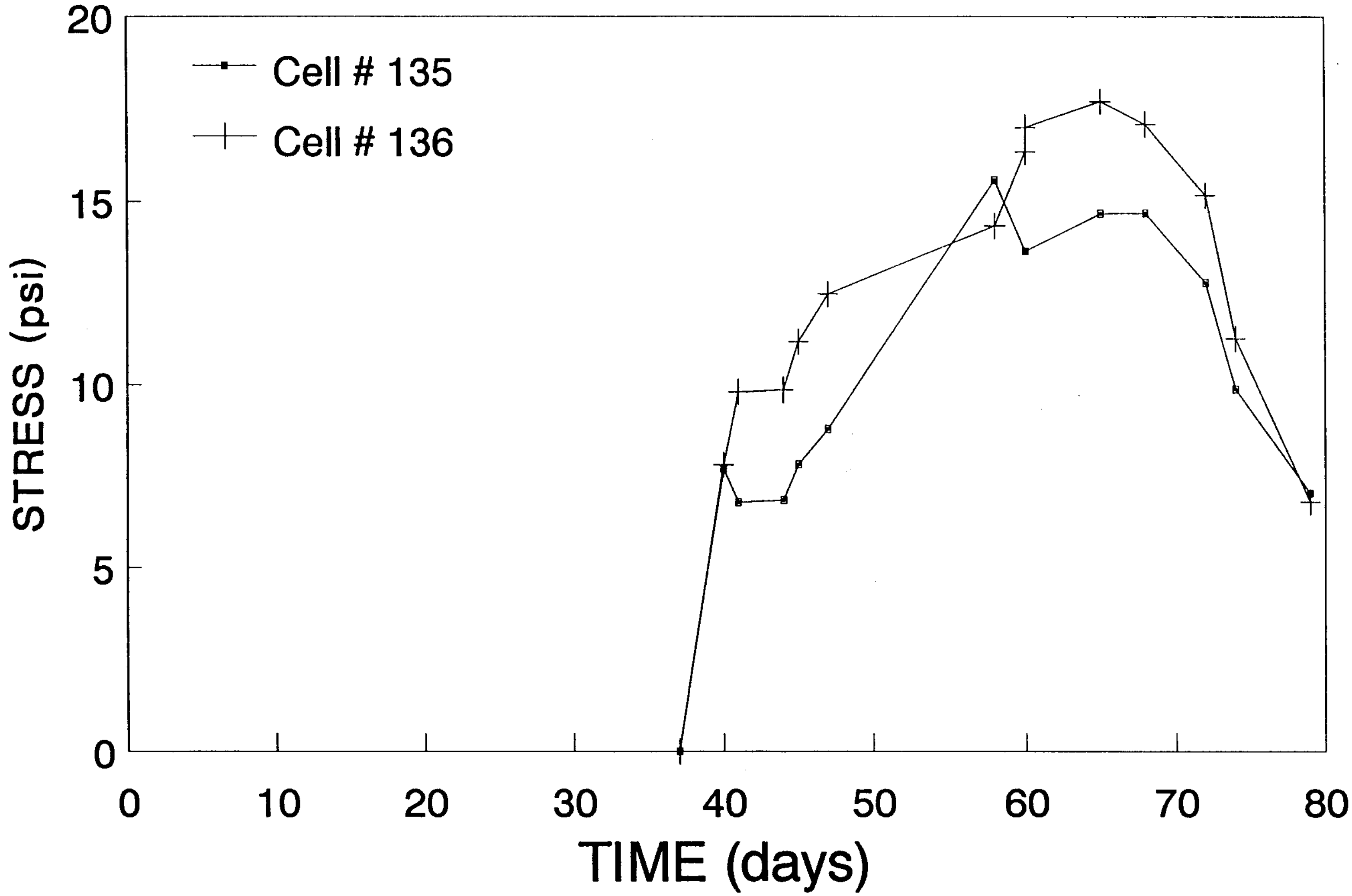

Figure 22. Earth Pressure Data on the Foundation at Abutment 2.

APPENDIX A

Design procedure for the stone column treated foundation

II. ORIGINAL DESIGN OF STONE COLUMN TREATED FOUNDATION ...

The geotechnical engineering report recommending that stone columns be utilized to stabilize the foundation for a two span structure with reinforced earth abutments was produced in June, 1984. Slope stability, bearing capacity and settlement calculations were made for the reinforced earth abutments with and without stone column treatment. The design involved stone columns 3.25 feet in diameter on 6 foot center to center triangular spacing. The soil strength parameters chosen were based on soil tests of samples obtained at the proposed wall site on the original design project and the updated soil information prior to the 1984 report. The effective stress (indrained strength) parameters for the foundation were estimated to be:

$$\bar{c} = 260 \text{ psf}$$

$$\bar{\phi} = 27 \text{ degrees}$$

while the foundation was assumed to be composed of three distinct soil layers of the following total stress shear strength parameters:

0 - 10 ft. depth	800 psf
10 - 20 ft. depth	1200 psf
20+ ft. depth	800 psf

See Appendix D for a graphical depiction of the original soil test data from which these soil strength parameters were derived. Also included in the Appendix is slope stability sections for the north and south proposed reinforced earth abutments.

The design assumed that the stress concentration ratio (n) would equal 2.0 for the short and long term conditions. This value is consistent with those recommended in the Design and Construction of Stone Columns Manual (FHWA/RD-83/026) prepared for the Federal Highway Administration by the Georgia Institute of Technology.

In the stability calculations the parameter averaging method and the assumed stress concentration ratio were used to derive the long and short term soil strength parameters. The resulting average strength parameters for the stone column improved foundation were:

Total Stress:

0 - 10 ft.	c(avg) = 587 psf
10 - 20 ft.	c(avg) = 880 psf
20+ ft.	c(avg) = 587 psf

Effective Stress:

$$\bar{c}(avg) = 191 \text{ psf}$$

$$\bar{\phi}(avg) = 34 \text{ degrees}$$

The bearing capacity of the stone columns was calculated such that the ultimate bearing capacity of the stone columns (q_s) is:

$$q_s = (c)\ (N_s) = 800 \text{ psf} \times 20 = 16{,}000 \text{ psf} = 8 \text{ tsf}$$

The ultimate bearing capacity of the clay (q_c) was calculated by the formula:

$$q_c = (c)\ (N_c) = 800 \text{ psf} \times 5 = 4{,}000 \text{ psf} = 2 \text{ tsf}$$

The average ultimate bearing capacity (q avg) was then computed by proportioning the contributing bearing from the clay and the stone according to its percentage of the unit cell area. q average was computed to be:

$$q_{avg} = (As/A)\ (q_s) + (Ac/A)\ (q_c)$$

thus,

$$q\ avg = (8.3/31.17)\ (8 \text{ tsf}) + (22.87/31.17)\ (2 \text{ tsf})$$
$$q\ avg = 3.6 \text{ tsf}$$

The average ultimate bearing capacity, at 90 percent consolidation of the foundation was calculated to be:

$$q\ avg = (8.3/31.17)\ (12.3) + (22.87/31.17)\ (3.1)$$
$$q\ avg = 5.5 \text{ tsf}$$

The settlement of the foundation was computed by conventional methods and then reduced using the Equilibrium Method, where s, the amount of settlement in the treated area, is:

$$s = s \times \log \frac{Po + (Mc)\ (Pe)}{\frac{Po}{(Pf/Po)}}$$

For stress concentration factor, n, equal to 2.0,

$$Mc = (1)\ /\ (1 + [(n - 1) \times (As)] = 0.48$$

This method estimated to produce a reduction of approximately 40 percent of the expected settlement and was considered to be the upper bound of the anticipated ultimate settlement.

The basis for calculating the rate of settlement was to use the conventional sand drain theory assuming a reduced stone column diameter of 3.0 feet to account for some infiltration of soil fines that would reduce the efficiency of the stone columns' drainability. This method had been seen by previous designers to underestimate the time required for consolidation to occur in stone column treated areas.

A summary of the stability analyses is:

Stability Analyses Of The South Wall

Short Term Safety Factor = 0.9 (w/o stone columns)
Long Term Safety Factor = 1.3 (w/o stone columns)

Short Term Safety Factor = 1.4 (w/stone columns)
Long Term Safety Factor = 1.4 (w/stone columns)

Stability Analyses Of The North Wall

Short Term Safety Factor = 1.5 (w/o stone columns)
Long Term Safety Factor = 2.1 (w/o stone columns)

Short Term Safety Factor = 2.3 (w/stone columns)
Long Term Safety Factor = 2.3 (w/stone columns)

The settlement analyses for each wall were performed and indicated that the soil foundation would settle extremely slow. This slow rate of consolidation of the soil would pose long term disruptions in the grade of the approaches to the bridge. Stone columns were analyzed for their effect on the long term settlement problem. The computed settlements for the south wall showed an average long term settlement of 9.4 inches over 1.4 years. The stone column treated foundation was computed to settle about 6.0 inches over 35 days. Both magnitudes of settlement are for the 90 percent of the total expected settlement.

The settlement analyses for the north wall showed that the wall would settle an average of about 13 inches over 5.2 years and would decrease in settlement with stone column treatment to about 8 inches over 46 days. These times and magnitudes of settlement are also for 90 percent consolidation.

SETTLEMENT ANALYSES OF THE SOUTH WALL

9.4 inches in 1.4 years - without stone columns
6 inches in 35 days - with stone columns

SETTLEMENT ANALYSES OF THE NORTH WALL

13 inches in 5.2 years - without stone columns
8 inches in 46 days - with stone columns