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U.S. Department of Transportation
Urban Mass Transportation Administration

Field Evaluation of Advanced Methods of Geotechnical Instrumentation for Transit Tunneling

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September 1983
Final Report

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16. Abstract

This report presents the result of a field evaluation of advanced methods of geotechnical instrumentation on an ongoing urban rapid transit tunneling project.

Numerous methods of geotechnical instrumentation, including surface and building settlement points, deep settlement points, inclinometers, piezometers and observation wells were used to monitor ground movements and groundwater levels within a test section on an urban rapid transit project under construction. The performance of the instrumentation methods are evaluated in terms of accuracy, costs, and engineering and construction advantages.

In addition, predictions of stratigraphy presented in an earlier report are compared with the stratigraphy observed during tunnel construction. Advanced methods of explorations used for these predictions are evaluated.

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PREFACE

This report documents a field evaluation of advanced methods of geotechnical instrumentation for transit tunneling. It was conducted by Bechtel Civil & Minerals, Inc., San Francisco, California, in association with Haley & Aldrich, Inc., Cambridge, Massachusetts. "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling" (UMTA-MA-06-0100-80-1, DOT-TSC-UMTA-80-1) was also conducted by Bechtel, Inc., in conjunction with Haley & Aldrich, Inc.

The work was completed under Contract DOT-TSC 1570 for the Transportation Systems Center (TSC) on behalf of the Office of Systems Engineering of the Urban Mass Transportation Administration (UMTA), Office of Technical Assistance, U.S. Department of Transportation (DOT). The assistance and support throughout this study by Philip Mattson, Technical Monitor for TSC, is greatly appreciated. The Project Manager for the work was Harry Sutcliffe, Manager of Bechtel's Boston Office.

This study was conducted with the concurrence and assistance of the Massachusetts Bay Transportation Authority (MBTA). The assistance of Mr. Francis M. Keville, Director of Construction for the MBTA, is acknowledged and appreciated.

The assistance of Andrew F. McKown of Haley & Aldrich, Inc. for his contributions to the report is greatly appreciated. Acknowledgement is given to Timothy R. Cullen of Bechtel Civil & Minerals for the geologic mapping of the Test Section, and initial drafts of Sections 4, 7, and 11. Assisting in the mapping of the Test Section were Carl Bock and Mark Johnson of Bechtel Civil & Minerals, and Zenas F. Bliss of Haley & Aldrich, Inc. The final report could not have been produced without the very skillful efforts of Graphic Illustrator, Acey Welch; Senior Typist, Marion Keegan; and Word Processor, Kathy Howell of Haley & Aldrich, Inc.

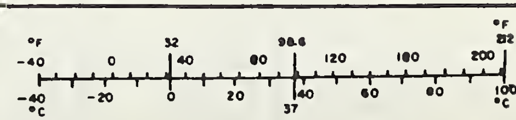
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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LIST OF ABBREVIATIONS

ORGANIZATIONS

ASCE	- American Society of Civil Engineers
ASTM	- American Society for Testing and Materials
CME	- Central Mine and Equipment Company
DCDMA	- Diamond Core Drill Manufacturer's Association
FHWA	- Federal Highway Administration
MBTA	- Massachusetts Bay Transportation Authority
NARETC	- North American Rapid Excavation and Tunneling Conference
NSF	- National Science Foundation
TSC	- Transportation Systems Center
UMTA	- Urban Mass Transportation Administration
USC&GS	- United States Coast and Geodetic Survey
USDOT	- United States Department of Transportation
USGS	- United States Geological Survey

EXPLORATION AND INSTRUMENTATION REFERENCES

DW	- Dewatering Well
I	- Inclinometer
N	- Number of Blows (Standard Penetration Testing)
OW	- Observation Well
PZ	- Piezometer
RQD	- Rock Quality Designation
SPT	- Standard Penetration Test
SSPD	- Deep Settlement Point
SSPS	- Shallow Settlement Point
TSC	- Study Exploration Numbers
TW	- Test Well

EXPLORATION EQUIPMENT

FX	- Flush Coupled Casing
FJ	- Flush Joint Casing
GRO	- Grout/Reinforcing/Orientating Tube
HXWL	- Sprague & Henwood Triple Tube Wire - Line Core Barrel

EXPLORATION EQUIPMENT (continued)

NWD3	- Christensen Double Tube Core Barrel
NWM3	- Acker Triple Tube Core Barrel
PVC	- Polyvinyl Chloride
RWT	- Sprague & Henwood Single Tube Core Barrel
XH	- Extra Heavy Duty Steel Casing
ABS	- Acrilo Butadiene Styrene

GEOPHYSICAL SURVEYING

CM/SEC	- Centimeter Per Second
GG	- Gamma-Gamma
NATG	- Natural Gamma
NG	- Neutron-Gamma
NN	- Neutron-Neutron
P	- Compressional Wave
R	- Resistivity
S	- Shear Waves
SP	- Spontaneous Potential

MISCELLANEOUS

BM	- Bench Mark
CL	- Center Line
ID	- Inside Diameter
MSL	- Mean Sea Level
ND	- Not Determined
OD	- Outside Diameter
RPM	- Revolutions Per Minute
TM	- Trademark or Brand Name
cu.ft./ft.	- Cubic Foot per Foot
gpm	- Gallons per Minute
$\pm$	- Plus or Minus

1. SUMMARY

1.1 GENERAL

The construction of new rail rapid transit systems and additions to existing systems has greatly increased the amount of tunneling performed in the United States. Since these transit systems are generally located in urban areas, tunneling is used to minimize the impact of the construction and operation on the community.

Prior to construction, a thorough evaluation must be made of the geotechnical parameters determined by the subsurface investigation program to select tunnel alignment, to evaluate methods of construction and groundwater control, and temporary or permanent support structures. It is equally important that additional information be obtained on a continuing basis as the tunnel is constructed, so that geotechnical parameters, ground performance and construction procedures may be continuously evaluated and refined.

A variety of advanced exploration techniques and instrumentation programs are available that can provide subsurface and ground movement data relevant to tunnel construction. Optimization of the tunnel lining system and construction techniques depends on the accuracy and the completeness of the subsurface exploration data. Evaluation of the need to protect structures along the alignment or to modify construction procedures in critical areas depends on the magnitude and distribution of ground movements. Therefore, increasing attention is being given to advanced exploration techniques and instrumentation programs.

1.2 OBJECTIVE

The objective of this study is to evaluate, through the use of a field demonstration program, the feasibility, applicability, reliability, and cost effectiveness of several advanced methods of subsurface exploration and geotechnical instrumentation to produce data usable for rapid transit tunnel design and construction within the time, cost, and schedule constraints common to the industry.

A Test Section on the Massachusetts Bay Transportation Authority (MBTA) Red Line Extension-Northwest, Cambridge, Massachusetts, was selected to evaluate methods of subsurface exploration that investigate geotechnical parameters, and instrumentation used to monitor ground movements due to tunnel construction. The Test Section was located in an area of rock, soft ground, and mixed face tunneling approximately 100 feet below ground surface. Overburden soils consist primarily of a saturated, very dense glacial till containing cobbles and boulders, with the bedrock a weakly metamorphosed shale that is severely fractured and intruded by igneous dikes. The site represents a typical urban setting with the Test Section located under a major, four-lane divided street, with structures adjacent on both sides. Excavation during construction of the tunnels permitted observation of actual stratigraphy and a comparison with predicted stratigraphy.

Because of the natural time lag between the investigation and construction, two stages of investigation were pursued and reported. Stage I, entitled, "Field Evaluation of Advanced Methods of Subsurface Explorations for Transit Tunneling" (I-7) included an evaluation of various types of field explorations, geophysical measurements, in situ testing, and procedures necessary to predict probable subsurface conditions. Stage II (the subject of this report), entitled, "Field Evaluation of Advanced Methods of Geotechnical Instrumentation for Transit Tunneling" includes an evaluation of geotechnical instrumentation procedures employed and geologic mapping necessary to evaluate the accuracy and applicability of predictions made during Stage I and to document the actual effects of tunneling on ground movements and groundwater levels within the study area.

1.3 RESULTS

Results of field measurements are presented in Section 8 and compared to predicted settlements in Section 9. Section 10 discusses instrument performance, based upon accuracy, cost, and benefits to engineering and construction.

The geologic mapping performed during tunnel excavation is presented in Section 11 and Appendix H. A comparison of the observed stratigraphy and the stratigraphy predicted from Stage I (I-7) explorations is presented in Section 12. Section 13 evaluates the advanced methods of explorations used for the Stage I predictions.

Section 14 presents conclusions on the performance of the geotechnical instrumentation and the advanced methods of explorations used in this research.

1.4 CONCLUSIONS

1.4.1 Ground Movements

1. The location of the Test Section in an area of such a high degree of geologic variability provided a unique opportunity to monitor and compare ground movements due to different subsurface conditions (rock, soft ground, and mixed face) and construction procedures.

2. In general, ground movements at the Test Section were small, in many cases approaching the limits of accuracy of the instrumentation.

3. The ground surface settlements measured at the Test Section were slightly less than the ground surface settlements predicted during Stage I studies. In view of uncertainties regarding subsurface conditions and details of tunneling methods, equipment, and workmanship at the time the predictions were made, the agreement between predicted and measured settlement is excellent.

4. Detailed comparison of the predicted settlement volumes in soft ground (V_S = 0.8 percent for single tunnels, ΔV_S = 0.4 percent for interference between twin tunnels) with the measured settlement volumes indicated excellent agreement. However, the settlement troughs were much wider than would be inferred from the relationships published by Cording et al. (I-2), Figure 9-11.

5. The surface settlements at the mixed face section show some interesting effects of mixed face conditions. The first tunnel (inbound) excavation, mostly in rock, caused slightly greater surface settlements than were observed at the section in rock, indicative of greater ground losses due to soil disturbance. The second (outbound) tunnel had more of its cross section in the soft ground and caused a settlement trough with a greater volume (V_S = 1.28 percent) than was measured for the single tunnel in soft ground (V_S = 0.76 percent), Figure 9-3. The settlement trough in the mixed face section caused by excavation of the second (outbound) tunnel was entirely symmetrical about the tunnel. This indicates that there was little, if any, interference between the first (inbound) and second (outbound) tunnels at this section.

6. At the Test Section, data from the deep settlement points and inclinometers show that large losses of soft ground at the tunnel heading propagated no more than 1 or 2 tunnel diameters from the heading (Figures 8-9 and 8-14).

1.4.2 Performance of Instruments

1.4.2.1 Surface and Building Settlement Points - These instruments, in general, performed very well. Because of their relatively low cost, it was possible to provide many of these points and read them often. They were reliable although some measurements were lost because of surface construction activities.

1.4.2.2 Deep Settlement Points - There was no clear difference in the observed performance of the electrical settlement system compared to the mechanical (telescoping settlement/inclinometer casing) system. On the other hand, the electrical system has a higher listed accuracy and a corrugated casing which probably follows ground movements more closely than the smooth extruded PVC casing used for the mechanical system. The extruded PVC casing also exhibited a great deal of spiral, complicating interpretation of inclinometer data.

1.4.2.3 Inclinometers - Even when the horizontal ground displacements are very small, as at the Test Section, the shapes of the inclinometer profiles (lateral displacement vs. depth) provide an indication of where ground loss and soil deformation are occurring. This suggests that plots of sensor inclination vs. depth, Figure 10-7, can sometimes clearly define the zones of largest deformation around the tunnel.

When very accurate measurements of lateral displacements are required, then sources of error can be corrected for, as discussed in Section 10. For example, casing spiral should be measured during installation and, if low, can be corrected for by means of Equations 10-1 and 10-2. In this respect, the interior grooved ABS casing exhibited much less spiral, in general, than the extruded PVC casing.

The computerized inclinometer data handling system utilized by this research (Section 6.5.1.4) proved to be of great value. It minimized tedious hand calculations and facilitated the interpretations described above. It would be an essential ingredient for providing rapid feedback to the tunneling contractor for modifications of construction procedures based on measurements of ground movements.

1.4.3 Value of Instrumentation for Engineering and Construction of Tunnels

1.4.3.1 Surface and Building Settlement Points - These measurements are relatively inexpensive and very reliable. The surface settlement data at the Test Section provided significant information on the size and shape of settlement troughs for the single and twin tunnels in glacial till (Figures 9-3 and 9-6) and for the tunnels in a mixed face condition (Figure 9-10).

1.4.3.2 Deep Settlement Points and Inclinometers - At the Test Section, these instruments demonstrated their value in identifying the sources and extent of lost ground and in indicating face instabilities (Figures 8-9 and 8-14).

1.4.4 Advanced Methods of Exploration

Detailed descriptions of advanced methods of exploration used at the Test Section are presented in the Stage I report (I-7). Evaluations of these methods are presented in Section 13 of this Stage II report, with conclusions in Section 14.5.

2. INTRODUCTION

2.1 GENERAL

The expansion of rapid rail transportation in urban areas throughout the United States has caused a careful examination of alternatives to surface transportation systems. Construction of rail systems in an urban environment, with surface space already limited due to existing development and the necessity to minimize impact on the community, has created increased demands on the use of underground space.

The use of underground space for rail transportation involves the below grade construction of stations and interconnecting tunnels. The basis for the successful design and construction of these facilities is accurate information followed by sound decision making on alternatives such as alignment, construction methods, and physical design of tunnels, stations and appurtenant facilities. The acquisition of comprehensive subsurface information as early as possible in the planning and design process can do much to improve this decision-making process. The EIS Process requires detailed excavation and construction cost estimates of alternative routes, but the expense and development of a complete geotechnical exploration program for each route is prohibitive. Hence, economical and non-descriptive exploratory methods will assist in making correct evaluations early in the planning process.

With tunnel construction, the potential cost and schedule impacts of variable subsurface conditions along the alignment are much greater than with surface construction. Therefore, improvements in data acquisition by using advanced and mutually supportive exploration techniques should result in cost-effective improvements in planning, design, and construction.

The construction costs associated with tunneling are largely dependent on the medium through which the tunnel is driven. The cost of the permanent structure itself may vary considerably, depending on the geological unit within which it is constructed. Most of the construction cost is incurred for shaft and tunnel excavation, temporary and permanent ground stabilization and support, utility relocation or support, and protection of existing structures.

Selection of ground support systems, optimization of excavation and construction processes, underpinning of structures, utility relocation, and other significant construction cost items are influenced or dictated by data developed by the geotechnical exploration program.

It is of critical importance that the tunnel design engineer obtain as much detailed information as possible about the subsurface geological conditions several months prior to the issue of tunnel construction bid documents. A thorough evaluation of the geotechnical parameters, determined from the subsuface investigation program, must be accomplished during the selection of the tunnel alignment, to evaluate methods of construction, groundwater control, and temporary or permanent support structures.

It is equally important that information on actual subsurface conditions and ground behavior be obtained as the tunnel is constructed. This information is usually supplied as part of a monitoring program using various kinds of instrumentation supplemented by visual inspection of the conditions in the tunnel and on the surface. The value of such performance data is readily apparent with regard to controlling on-going work and providing a basis to predict future behavior on a project. In addition, monitoring data provide a valuable resource for future projects where ground behavior and the effects of various construction techniques may be predicted with greater confidence and reliability.

A variety of instrumentation methods are available for the tunnel designer to use in obtaining performance records from tunnel construction. However, the selection of an appropriate instrumentation program, the selection and installation of hardware, and the interpretation of the instrumentation results must be done with careful consideration of both the instrumentation capabilities and monitoring program objectives.

2.2 PURPOSE AND SCOPE

The opportunity for cost savings in the design and construction of transit tunneling has generally been applied to tunnel support and lining systems and, to a lesser extent, to construction techniques. Optimization of both the tunnel lining system and the construction technique depends on the accuracy and completeness of subsurface exploration data. Therefore, increasing attention is being given to relatively new and advanced methods of exploration and monitoring techniques.

The U. S. Department of Transportation, through its modal elements, has sponsored several research studies on the subject of subsurface exploration and instrumentation methods applicable to tunneling projects. Although these studies provided excellent recommendations for new exploration methodologies and procedures, none had been thoroughly tested and evaluated under actual field conditions in association with a current transit tunneling project.

The Massachusetts Bay Transportation Authority (MBTA) is presently in the latter stages of construction of an extension of its Red Line subway system from Harvard Square Station to Alewife Brook Station in Cambridge, Massachusetts. This Northwest Extension project is a 3.1 mile addition to the MBTA's existing network of 120 miles of rapid transit lines which serve the greater metropolitan Boston area. The design of the tunnels was by Bechtel Civil & Minerals, Inc. The Contractor for the construction of the tunnels was a joint venture of Morrison-Knudsen, J.F. White and the Mergentime Corporation, under contract to the MBTA. This tunnel construction project presents an excellent opportunity to fully evaluate the more promising advanced techniques and equipment for subsurface exploration and instrumentation from inception to conclusion of transit tunnel construction (Figure 2-1).

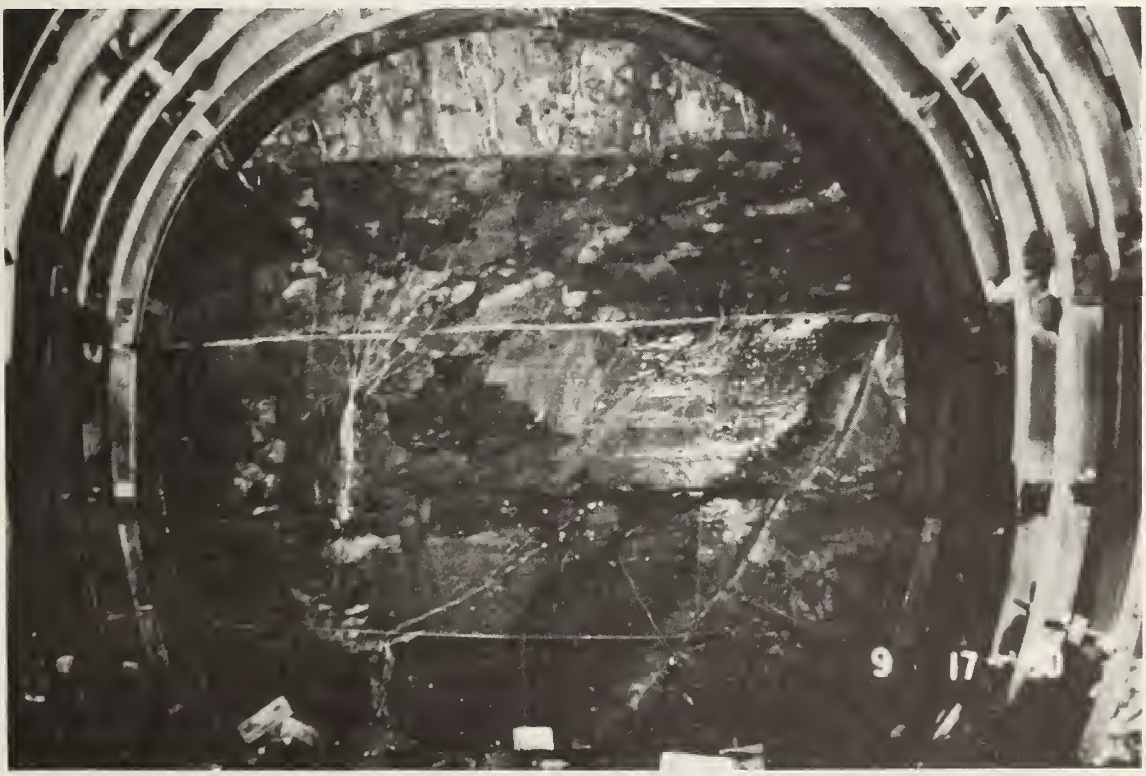


FIGURE 2-1. MBTA RED LINE EXTENSION NORTHWEST,
FULL FACE ROCK EXCAVATION, 220+06 IB

This field demonstration project was executed in two stages due to the natural time lag between the subsurface investigation for design and the constructon of the tunnels. The purpose and scope of the two stages is described in the following sections.

2.2.1 Stage I - Advanced Methods of Subsurface Exploration

The work conducted in Stage I was previously presented in a report entitled, "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling" (I-7). The scope included an evaluation of various methods of field exploration used to predict probable subsurface conditions. Stage I, which was completed in early 1980, consisted of the following primary activities:

a. To employ selected procedures for borings, soil and rock sampling, in-situ testing, and geophysical methods on an ongoing transit tunnel project.

b. To evaluate the feasibility, applicability, reliability, and cost effectiveness of the selected exploration techniques.

c. To use the selected techniques to predict the real and relevant geotechnical unknowns in a test section along the route of the MBTA Red Line Extension.

2.2.2 Stage II - Advanced Methods of Geotechnical Instrumentation

The Stage II studies, documented in this report, were conducted throughout the construction period of the rapid transit tunnels located between Stations 203+00 and 206+00 OB. This Test Section was selected during the Stage I phase of the work as a particularly advantageous area for intensive study. With its complex subsurface conditions and tunnel alignment consisting of soft ground, mixed face and rock tunneling, this area provided the opportunity to evaluate not only the predictions made for subsurface conditions during Stage I but the performance of the geotechnical instrumentation.

The primary objectives of Stage II were:

a. To evaluate the accuracy of the geotechnical predictions previously made with appropriate geotechnical instrumentation, monitoring, and geologic mapping during tunnel construction.

b. To evaluate the performance and reliability of selected geotechnical instrumentation installed for the project.

c. To demonstrate the effectiveness of instrumentation and monitoring during construction in documenting the effects of tunneling on adjacent structures.

d. To provide additional case history data, for use by designers and constructors in evaluating tunneling procedures and their effects on ground deformations for future projects.

e. To provide, in report form, an assessment of the applicability, reliability and cost effectiveness of the geotechnical instrumentation methods beyond those traditionally used on projects of this type.

This report, entitled, "Field Evaluation of Advanced Methods of Geotechnical Instrumentation for Transit Tunneling" documents the Stage II investigations.

3. DESCRIPTION OF TEST SECTION

3.1 SITE CONDITIONS

The Massachusetts Bay Transportation Authority (MBTA) Red Line Extension Northwest is a continuation of the original Boston Elevated Railroad, Cambridge Main Street Subway, which was completed in 1912, employing cut-and-cover construction techniques. The Northwest Extension project, shown in Figure 3-1, will extend from the reconstructed Harvard Square Station to new stations at Porter Square, Cambridge and Davis Square, Somerville, and terminate at the new Alewife Brook Station in North Cambridge, a distance of approximately 3.1 miles.

Most of the original topography along the alignment has been altered over the years and completely masked by heavy commercial and residential development. Although ground surface varies from elevation 145 to 126 MBTA Red Line Datum, the area along the alignment is relatively flat with the exception of one abrupt topographic change due to a depressed railroad right-of-way at Porter Square. MBTA Red Line Datum is 105.87 feet below United States Coast and Geodetic Survey [USC&GS] Mean Sea Level of 1929.

The portion of the tunnel alignment between Harvard and Porter Squares generally follows Massachusetts Avenue for its entire length of approximately 4400 feet. The alignment passes beneath a depressed section of the Boston & Maine Railroad in Porter Square and proceeds cross country into Davis Square, Somerville. The length of the alignment between Porter Square and Davis Square is approximately 2900 feet. The alignment leaves the Davis Square Station through a cut-and-cover section toward Alewife Brook Station.

Initial widely spaced reconnaissance borings were drilled in 1976, followed by detailed subsurface explorations and geophysical surveys during 1977 and 1978. Data from these explorations were used to determine the final vertical tunnel alignment between Harvard Square and Porter Square, shown on the generalized subsurface profile, Figure 3-2.

In the preliminary design phase (1976-1977), more detailed subsurface exploration lead to the recommendation to lower the tunnels' vertical profile to permit construction in the two most competent materials available (rock and glacial till) and to avoid unacceptable community disturbance from what would have been a largely mixed face tunnel.

FIGURE 3-1. MBTA RED LINE EXTENSION NORTHWEST AND TEST SECTION

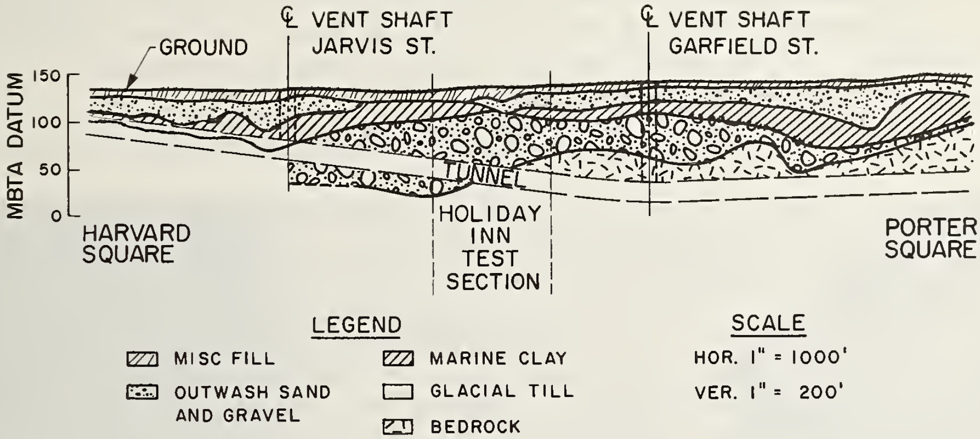


FIGURE 3-2. GENERALIZED SUBSURFACE PROFILE, HARVARD SQUARE TO PORTER SQUARE

In order to satisfy the basic intent of the Stage I study, to evaluate advanced methods of subsurface explorations for transit tunneling, areas with maximum geotechnical variability were sought. Based on the vertical tunnel alignment adopted for construction, two portions of the Harvard to Davis Square tunnels were to be driven in mixed face conditions; with the face partially in soil and partially in rock. In these sections, the definition of stratigraphy and other geotechnical parameters is of particular practical importance.

One of the mixed face areas occurs near Davis Square, Somerville, but this area was eliminated as a potential test section due to access limitations. The second mixed face area occurs beneath Massachusetts Avenue in Cambridge, opposite a Holiday Inn Motor Lodge, referred to hereinafter as the Test Section. This site offered similar geological conditions, but logistically and environmentally was judged to be more suited for the planned studies.

The Test Section is located between outbound (OB) tunnel stations 203+00 and 206+00, beneath Massachusetts Avenue in Cambridge approximately 3/4 of a mile north of Harvard Square (Figures 3-1 and 3-2). The 300-foot Test Section is occupied by a heavily traveled, major, four-lane artery. Commercial and residential structures varying from one to six stories in height-which are supported on shallow, soil bearing foundations-abut both sides of Massachusetts Avenue. A large in-ground concrete swimming pool at the Holiday Inn is approximately twelve feet horizontally from the eastern edge of the outbound tunnel. Numerous surface and subsurface utilities line the avenue, and pedestrian traffic in the area is exceptionally heavy. Although the site area imposed some surface access restrictions, it is considered to be representative of transit tunneling in an urban area (Figure 3-3).

FIGURE 3-3. TEST SECTION (Looking North)
CAMBRIDGE, MASSACHUSETTS

The geological conditions, environmental considerations, and engineering and construction parameters are considered to be comprehensive at the Test Section. The rapidly dipping bedrock surface allowed the detailed investigations of the Stage I study to be conducted within the 300-foot long, mixed face area. All three tunneling conditions (rock, soft ground, and mixed face) were anticipated to be encountered within the site area. In addition, a highly intruded bedrock complex, glacial till, and two water table conditions combine to give a high degree of geologic variability to this part of the alignment.

The Holiday Inn Test Section was approved by the Transportation Systems Center as the study area and field work commenced on 29 March 1979 for Stage I with Stage II work commencing 24 March 1980.

3.2 TUNNEL CROSS SECTIONS

The Northwest Extension project consists of twin single-track transit tunnels which connect cut-and-cover stations at Harvard and Davis Squares with a deep underground station mined in bedrock at Porter Square.

The design tunnel configurations for soil and rock conditions are shown in Figure 3-4, with the inside diameter of 19.2 feet selected for the design clearance envelope. Ventilation shafts are located intermittently along the alignment, serving initially as construction access shafts and, later during operation, as ventilation and emergency exit shafts.

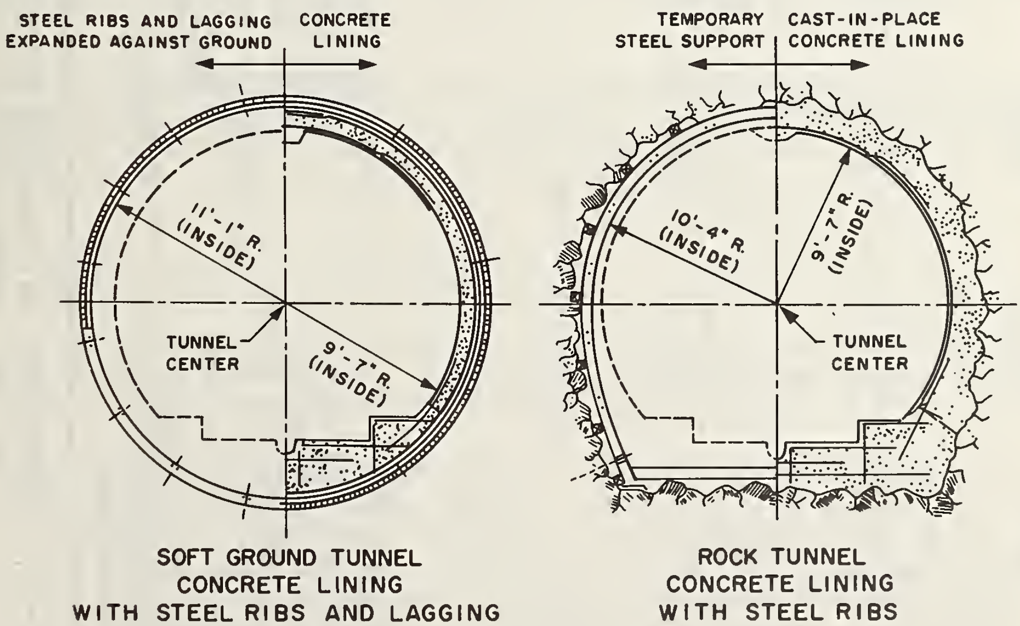


FIGURE 3-4. TYPICAL RED LINE TUNNEL SECTIONS

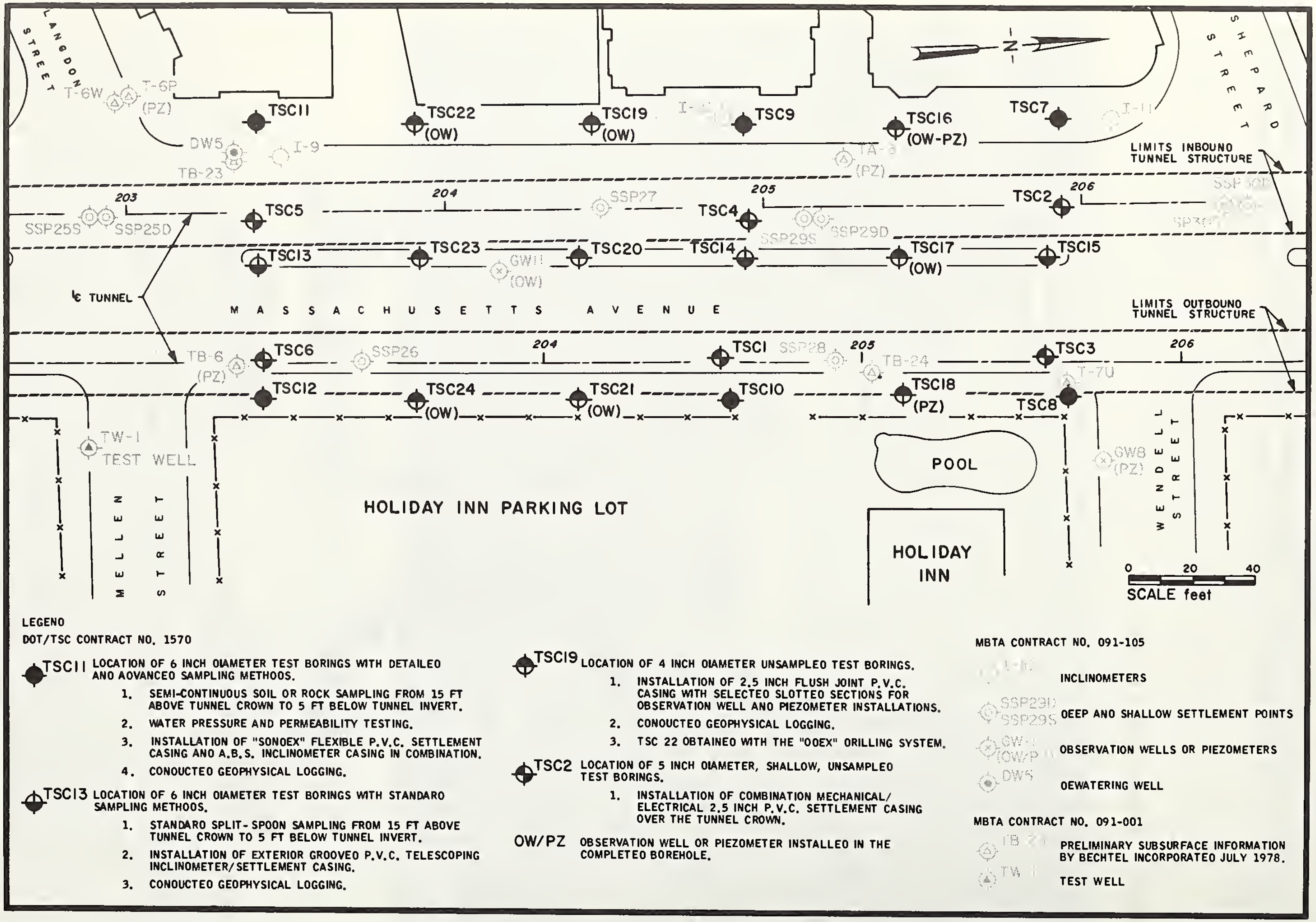


FIGURE 3-5. SUBSURFACE EXPLORATION LOCATION PLAN, TEST SECTION, CAMBRIDGE, MASSACHUSETTS

3.3 TEST SECTION DESCRIPTION

3.3.1 Explorations (Stage I)

A detailed exploration plan was developed at the Test Section to permit the implementation of a wide variety of exploratory methods and to facilitate the installation of geotechnical instrumentation to be evaluated during Stage II. The exploration program for the Test Section is summarized in Figure 3-5. The exploration program consisted of six parallel rows of boreholes spaced approximately 50 feet apart and oriented perpendicular to the tunnel alignment within the limits of the Test Section.

The exploratory holes were advanced using contract-specified drilling equipment and sampling procedures. The completed holes were subsequently used for electrical and nuclear surveys and seismic studies, and, finally, for placement of geotechnical instrumentation. Borehole spacing was maintained close to 50 feet to facilitate cross-hole seismic velocity studies and to define the complex stratigraphy expected in the area.

Detailed descriptions and evaluations of these explorations are presented in the Stage I report (I-7). Subsurface conditions determined from these explorations are presented in Appendix B and discussed in Section 4 of this report.

3.3.2 Instrumentation (Stage II)

At the end of the Stage I explorations, instrumentation for the Stage II work was installed in the completed boreholes. This advantage minimized the installation cost of the instruments.

Three rows of instrumentation spaced approximately 100 to 150 feet apart (one at each end and one approximately in the middle of the Test Section) were designed as high intensity instrumentation sections. These high intensity sections, are identified as the north, middle, and south sections on Figure 3-6. They provided the opportunity to monitor the performance of instruments and the results of tunnel excavation techniques in three ground conditions-rock, soft ground, and mixed face.

At the north section, Station 205+62 OB, the tunnel headings were entirely in rock. The tunnel crowns at the section were approximately 15 feet below the bedrock surface. The twin tunnels passed through this section from the north

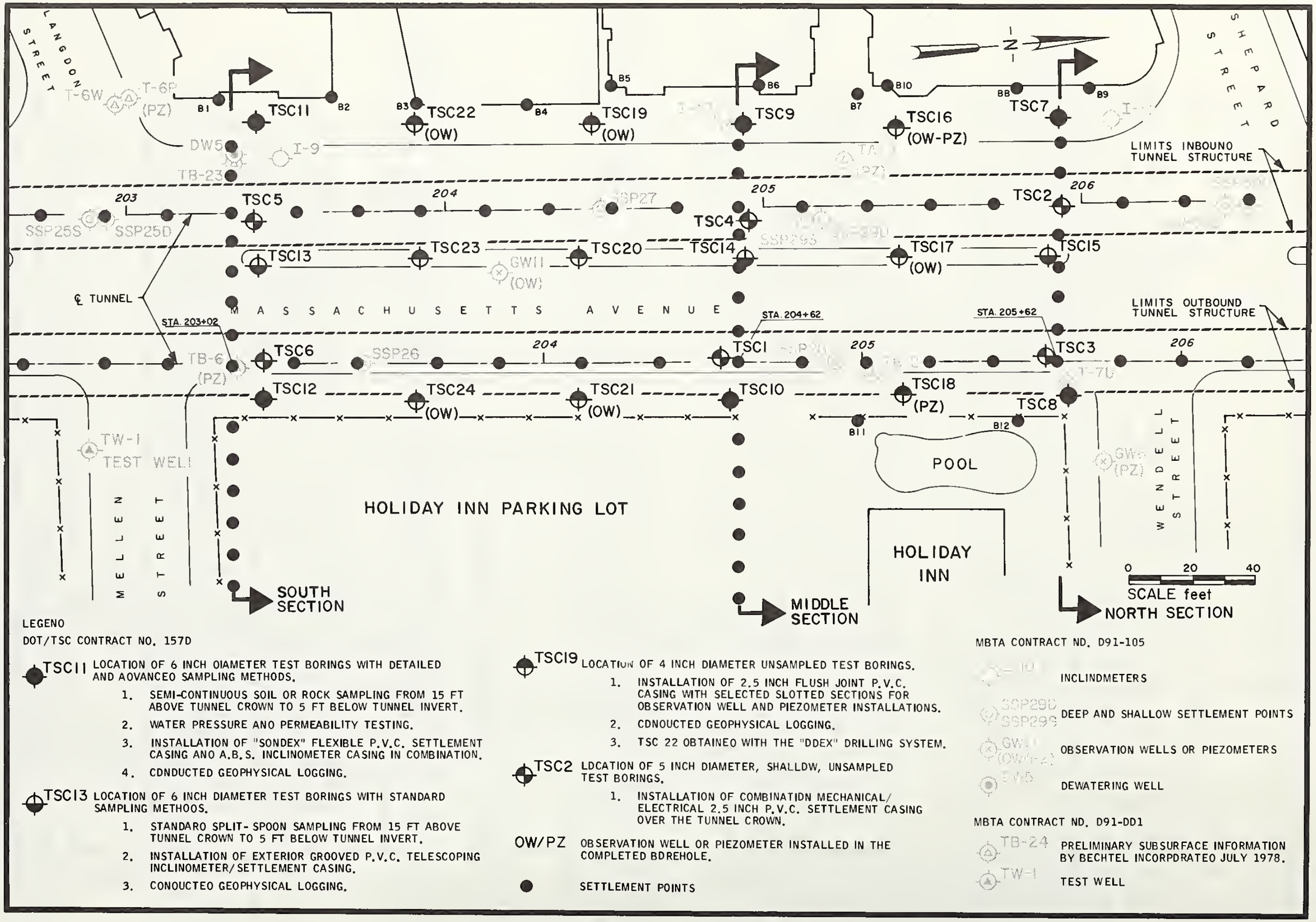


FIGURE 3-6. INSTRUMENTATION LOCATION PLAN, TEST SECTION, CAMBRIDGE, MASSACHUSETTS

during August through October 1980, using drill-and-blast excavation techniques. The outbound heading preceded the inbound by about 150 feet.

The south section, Station 203+02 OB, has most of the tunnel heading in glacial till and is designated as the soft ground section. The tunnel headings passed through this section from the south by means of soft ground shield excavation procedures in August 1981 (inbound) and October 1981 (outbound).

The middle section, Station 204+62 OB, is in a mixed face area, with the tunnel heading in glacial till overlying rock. This section was first passed by a 10-foot by 10-foot pilot drift in the invert of the outbound tunnel in October 1980, followed by the full faced excavation for the inbound tunnel in December 1980. Both of these excavations were advanced from the north using drill-and-blast rock excavation techniques. The outbound tunnel was completed in late November 1981 as the soft ground excavation advanced from the south.

The typical instrumentation at these three high intensity sections is shown on Figure 3-7. Instrumentation consisted of the following:

a. Surface settlement points (SSPS) installed at approximately 10-foot intervals across each section to clearly define the ground surface settlements.

b. Deep settlement points (SSPD) installed over the tunnel crown (just outside of the tunnel springlines) and in the pillar between the inbound and outbound tunnels to measure vertical displacement of the soil mass at various depths.

c. Inclinometer (I) casings installed just outside of the tunnel springlines, and in the pillar between the inbound and outbound tunnels, to measure horizontal movements of soil and rock at various depths. Horizontal movements both parallel and perpendicular to the tunnel axes were measured. The combined measurements of both horizontal and vertical movements at depth indicate the source of ground surface settlements and the mechanisms by which they develop.

Between the high intensity instrumentation sections, rows of unsampled holes were advanced for the installation of observation wells (OW) and piezometers (PZ). Groundwater level and piezometric data from these instruments are helpful in interpreting settlement and inclinometer data and in determining the effects of the construction dewatering systems, drainage into the heading, and recharge after liner placement.

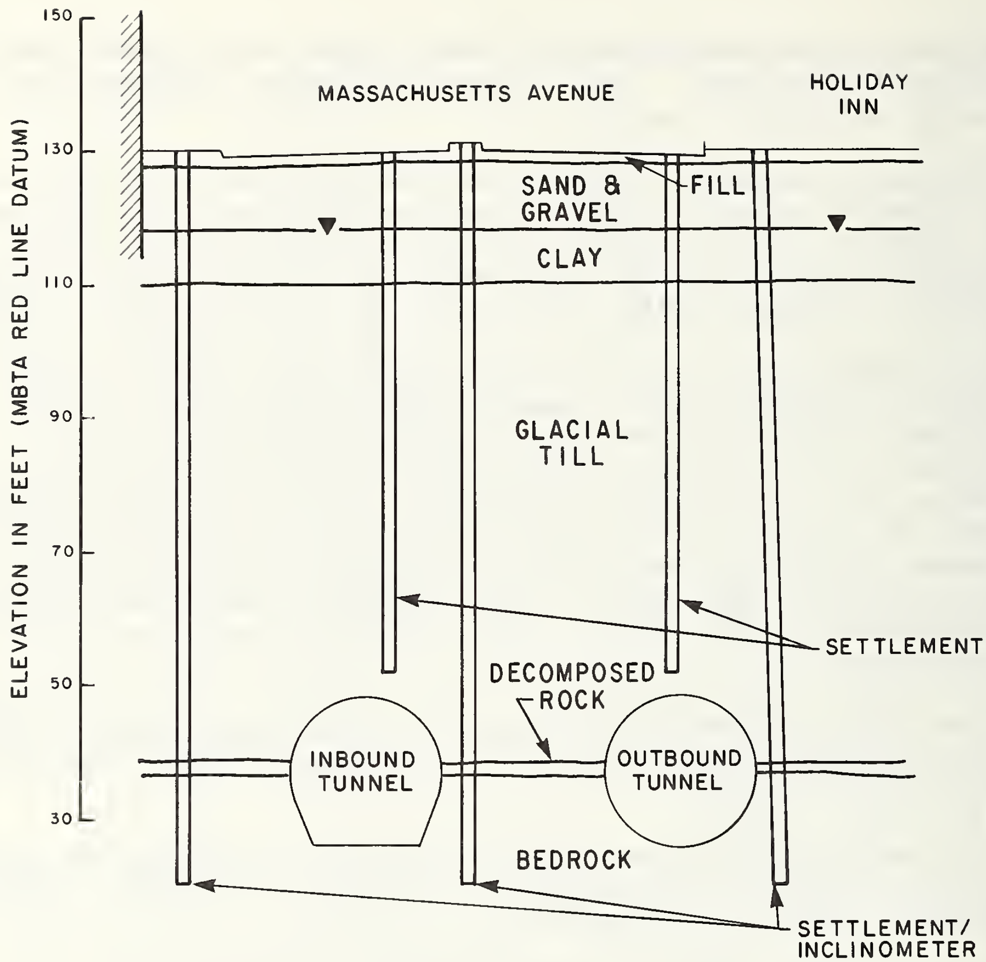


FIGURE 3-7. TYPICAL HIGH-INTENSITY INSTRUMENTATION SECTION

Surface settlement points were placed above the tunnel centerlines throughout the Test Section. At some locations, physical restrictions, such as buildings, limited the extent of the surface settlement points. Building settlement points were established on buildings on both sides of Massachusetts Avenue to monitor the effect of the tunnel excavations on these structures.

Instruments were installed in various combinations in order to achieve greatest economies and to compare the performance of different types of instruments. For example, deep settlements were measured with both electrical induction probe and mechanical probe systems. Inclinometer measurements were made in two different types of casing, each having a different cross section. Finally, many of the boreholes incorporated combinations of instruments for the measurement of both vertical and horizontal movements.

Table 3-1 summarizes the instruments installed at the Test Section. Section 6 presents detailed descriptions of the north (rock), south (soft ground), and middle (mixed face) sections. Section 6 also presents detailed descriptions of the instrumentation equipment, installation, monitoring, and data processing.

TABLE 3-1. INSTRUMENTATION SUMMARY

HOLIDAY INN TEST SECTION
CAMBRIDGE, MA.

INSTRUMENT NO.	LOCATION		ELEVATION		INSTRUMENTATION			
	OUTBOUND							
	STATION	OFFSET	GROUND SURFACE	BOTTOM OF CASING	INCLINOMETER	DEEP SETTLEMENT	OBSERVATION WELL	PIEZOMETER
TSC 1	204 + 56	1.4' L	129.4	--		Inoperable		
TSC 2	205 + 63	49.0' L	130.0	55.0		X		
TSC 3	205 + 58	2.0' L	129.5	54.5		X		
TSC 4	204 + 64.5	45.0' L	130.0	53.5		X		
TSC 5	203 + 09.5	45.2' L	129.7	58.2		X		
TSC 6	203 + 12.4	1.1' L	128.8	58.8		Inoperable		
TSC 7	205 + 62.5	77.5' L	130.2	17.8	X	X		
TSC 8	205 + 65	10.0' R	130.0	17.8	X	X		
TSC 9	204 + 63	75.5' L	130.0	27.2	X	X		
TSC 10	204 + 59	11.5' R	130.0	21.5	X	X		
TSC 11	203 + 10.5	76.5' L	129.3	28.5	X	X		
TSC 12	203 + 12	10.5' R	129.4	35.1	X	X		
TSC 13	203 + 11	31.0' L	130.7	29.5	X	X		
TSC 14	204 + 64.5	33.0' L	131.0	23.9	X	X		
TSC 15	205 + 58.5	34.0' L	131.3	30.9	X	X		
TSC 16	205 + 11	74.0' L	130.1	25.1			X	X
TSC 17	205 + 11.5	32.5' L	131.1	26.6			X	
TSC 18	205 + 13	10.5' R	130.1	31.1				X
TSC 19	204 + 16	76.0' L	129.9	99.9			X	
TSC 20	204 + 12	33.4' L	130.9	20.9				
TSC 21	204 + 11	11.0' R	129.8	19.8			X	
TSC 22	203 + 60	75.5' L	129.5	59.5			X	
TSC 23	203 + 62	33.5' R	130.8	25.8				
TSC 24	203 + 60.4	12.0' R	129.6	103.0			X	
BUILDING POINTS								
B1	203 + 01	86' L	133.7					
B2	203 + 38	86' L	133.7					
B3	203 + 61	83' L	130.1					
B4	203 + 97	83' L	130.3					
B5	204 + 26	88' L	133.6					
B6	204 + 69	88' L	133.7					
B7	205 + 00	84' L	130.3					
B8	205 + 53	87' L	130.7					
B9	205 + 72	85' L	133.7					
B10	205 + 10	85' L	133.6					
B11	204 + 99	19' R	130.1					
B12	205 + 49	19' R	130.1					

NOTES:
1. ELEVATIONS REFER TO MBTA RED LINE DATUM.
2. R = RIGHT, L = LEFT OF OUTBOUND CENTERLINE LOOKING NORTH.
3. SEE FIGURE 3-6 FOR INSTRUMENT LOCATION PLAN.

4. TEST SECTION SUBSURFACE CONDITIONS

4.1 REGIONAL GEOLOGY

4.1.1 General

A more detailed discussion of the subsurface conditions at the Test Section is presented in the Stage I report (I-7).

The project is within the New England Physiographic Province of the Appalachian Highlands. The entire province was subjected to glaciation during the Pleistocene Epoch. Thick deposits of glacial till and outwash sand and gravel were deposited throughout the area overlying bedrock.

The project is located in the northern portion of the Boston Basin (Figure 4-1), a structural-as well as topographic-depression which is filled with late Paleozoic rocks covered by Pleistocene glacial and post-glacial deposits.

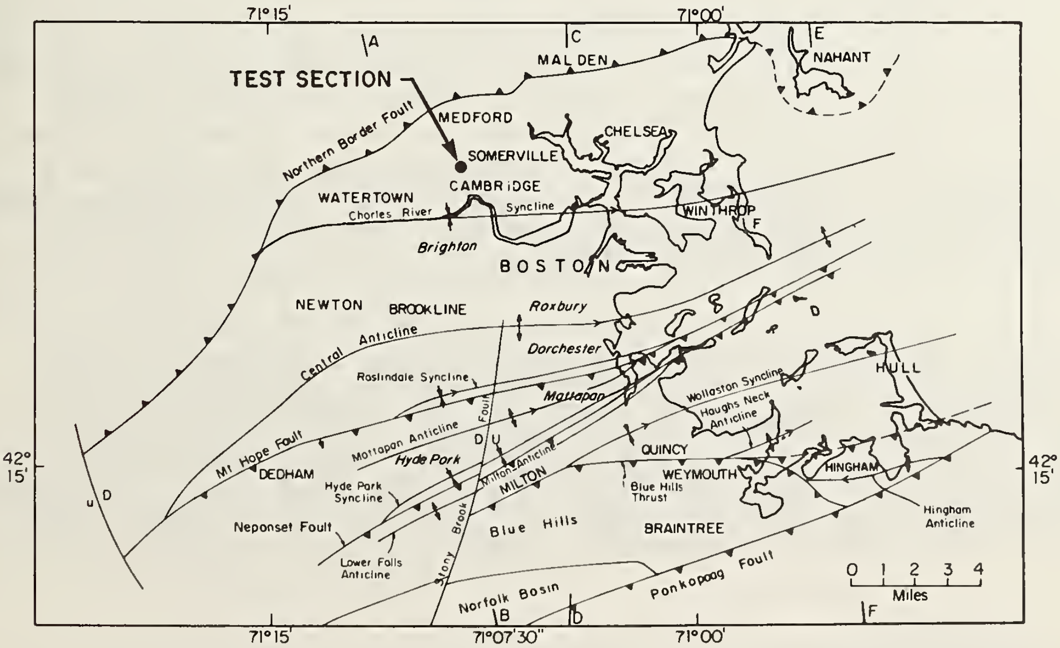


FIGURE 4-1. TECTONIC MAP OF BOSTON BASIN (BILLINGS, I-1)

4.1.2 Bedrock

The Boston Bay Group is the major stratigraphic bedrock sequence occurring in the portion of the basin where the project is located. This group has been divided into the Cambridge Argillite (upper unit) and the Roxbury Conglomerate (lower unit). The Cambridge Argillite (which is present within the Test Section), is a shale that is locally, weakly metamorphosed, with reworked tuffaceous material found throughout. The sediments that formed the Cambridge Argillite were originally fresh water deposits.

Light greenish-gray sedimentary beds of volcanic tuff and tuffaceous rock occur within the argillite. The thickness of the tuff beds ranges from a few inches to tens of feet.

Dikes, composed of igneous rocks such as dacite, felsite, diabase, basalt, and alterations thereof, are found throughout the Boston Basin. Regionally, the dikes are known to vary in size, form, structure, and degree of weathering. They range in thickness from a few inches to more than 500 feet and range in length from a few feet to several miles.

4.1.3 Structure

The project is located on the north limb of the Charles River Syncline, a regional bedrock structure which trends nearly east-west (Figure 4-1). The Red Line Extension tunnel alignment trends in a north-south direction, approximately perpendicular to the axis of this syncline. Most of the dikes and related intruded rocks dip steeply and have preferred strikes of east and east-northeast.

Large regional faults are not known to cross the tunnel alignment, although minor faulting can be expected anywhere along the alignment. The minor faults are generally young, normal faults. These faults regionally occur parallel to joints and along dikes. Regionally, the minor faults are narrow zones ranging from tight fractures to zones 3 feet wide and characterized in some cases by slickensides, gouge, and breccia. The dip is generally steep.

4.2 SITE GEOLOGY

4.2.1 Surficial Deposits

Surficial deposits at the Test Section consist of a thin layer of fill overlying a sequence of Pleistocene glacial and post glacial deposits which rest upon the bedrock surface. Based on the geologic origin and the characteristics of the soils encountered, the surficial materials have been divided into four (4) principal strata overlying the bedrock. In order of increasing depth from ground surface, they are:

1. Miscellaneous Fill
2. Outwash Sand and Gravel
3. Marine Clay
4. Glacial Till

Figure 3-2 shows the general geology along Massachusetts Avenue between Harvard Square and Porter Square, which includes the Test Section.

4.2.1.1 Miscellaneous Fill - The fill generally consists of silty fine to coarse sand with varying amounts of gravel, clay, organic materials and man-made materials such as bricks, concrete, and asphalt surface materials.

The thickness of the fill stratum within the Test Section averages about 3 feet, varying to a maximum thickness measured in test borings of about 6 feet. The relative density of the fill material ranges from medium dense to dense. The corresponding range of standard penetration resistance is from 14 to 45 blows per foot.

4.2.1.2 Outwash Sand and Gravel - Underlying the miscellaneous fill is a stratum of medium dense to very dense brown sand, gravelly sand, and silty sand. These deposits originated as glacial outwash deposited toward the end of the last glacial period (Lexington Substage).

The thickness of the deposit within the Test Section is variable, averaging about 9 feet, with a maximum thickness of about 15 feet. The relative density of the outwash sand and gravel varies widely with standard penetration resistances ranging from 14 to 55 blows per foot. The average standard penetration resistance in the stratum is in the range of 30 to 40 blows per foot.

4.2.1.3 Marine Clay - Underlying the outwash sand and gravel is a deposit of marine clay which contains varying proportions of silt and sand.

The thickness of the marine clay stratum within the Test Section ranges up to a maximum of 18 feet, with an average of approximately 8 feet. Standard penetration resistances average about 20 to 30 blows per foot in the upper portion of the stratum and 10 to 20 blows per foot in the less overconsolidated lower portion of the stratum.

4.2.1.4 Glacial Till - Underlying the marine clay is a relatively thick deposit of glacial till. The till generally consists of dense to very dense, silty, fine to coarse sand with varying amounts of clay, gravel, cobbles, boulders, and rock fragments of argillite and granite. The stratum was deposited directly over the bedrock surface in thicknesses varying from 51 to 83 feet within the Test Section.

Within the Test Section, the relative density of the glacial till ranges from dense to very dense with standard penetration resistance ranging from 45 to over 400 blows per foot. The average standard penetration resistance is over 150 blows per foot.

4.2.2 Bedrock and Structure

The principal rock type in the Test Section is altered and light grayish-green to greenish-gray argillite. The alteration is apparently caused by the intrusion of large igneous dikes, consisting primarily of diabase (Figure 4-2). A major diabase dike, estimated to be approximately 140 feet in the horizontal dimension, was encountered within the area. Additional details regarding the bedrock type and structure are summarized in Sections 4.3 and 11.

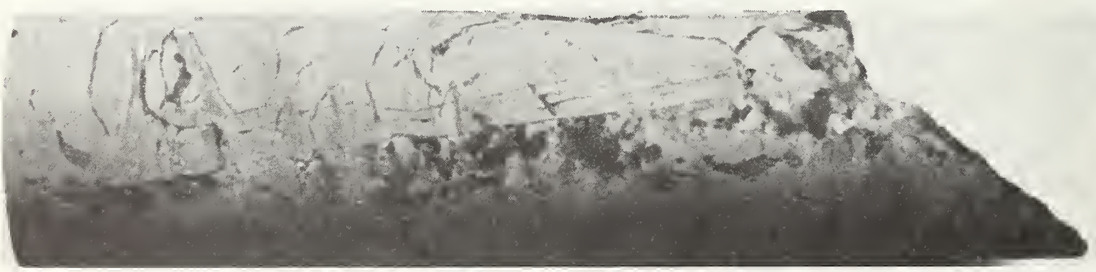


FIGURE 4-2. ALTERED ARGILLITE WITH DIABASE INTRUSIONS

4.2.3 Groundwater

The principal water bearing materials are the outwash sand and gravel and the zone along the bedrock/glacial till interface. The marine clay and glacial till together constitute an aquiclude between the overlying outwash sand and gravel and underlying bedrock/glacial till interface, resulting in two relatively independent water bearing zones.

Before tunnel construction, the normal depth to the water table in the outwash sand and gravel ranged from about 9 to 14 feet below ground surface. The normal groundwater gradient indicated by measurements in observation wells was from north to south.

Water levels in piezometers constructed in the bedrock before tunnel construction indicated artesian conditions. Based on measurement of the elevation of the piezometric surface, the general groundwater gradient was from north to south.

4.3 PREDICTED STRATIGRAPHY

This section will summarize stratigraphic evaluations and predictions made during Stage I of this study for the Test Section. These predictions will be compared with the field mapping of the tunnel face in Section 12.

The stratigraphic interpretations and predictions, based on an evaluation of all of the exploration techniques, are summarized in the Geologic Profiles in Appendix B. These profiles indicate two methods of interpretation for comparison purposes. The solid line interpretations are based on the physical data recovered from the test borings and the dotted line interpretations are based on the borehole nuclear logging surveys.

Table 4-1 summarizes the major geological units and the elevations at which they were anticipated to be encountered, relative to tunnel crown and invert, within the Test Section.

The subsurface conditions will be described in the following sequence:

4.3.1 Overburden Soils

4.3.2 Bedrock

4.3.3 Groundwater

TABLE 4-1. SUMMARY OF GEOLOGICAL CONDITIONS
HOLIDAY INN TEST SECTION
CAMBRIDGE, MA.

HOLIDAY INN TEST SECTION
CAMBRIDGE, MA.

TUNNEL STATION	TUNNEL CROWN	TUNNEL INVERT	APPROXIMATE ELEVATION OF			MAJOR ROCK TYPE	OVERALL AVERAGE ROCK QUALITY (RQD)
			TOP OF TILL	TOP OF DECOMPOSED ROCK	TOP OF NON-DECOMPOSED ROCK		
203+00 OB	51.5	29.5	105.5	33.5	31	ARGILLITE	FAIR
203+50 OB	50	28	108	N.D.	38.5	ARGILLITE	N.D.
204+00 OB	49	27	108	42	41	DIABASE	N.D.
204+50 OB	47	25	113	42	40	DIABASE	POOR TO EXCELLENT
205+00 OB	45.5	23.5	111	N.D.	50	DIABASE	VERY POOR TO FAIR
205+50 OB	44	22	116	60	58.5	ARGILLITE/DIABASE	POOR TO EXCELLENT
203+50 IB	52	29.5	112	32.5	30	ARGILLITE	VERY POOR
204+00 IB	50	28	107	N.D.	32	ARGILLITE	N.D.
204+50 IB	48	26.5	114	N.D.	32 - 42	ARGILLITE	N.D.
205+00 IB	47	25	118	42 - 49	41 - 46	ARGILLITE/DIABASE	POOR TO EXCELLENT
205+50 IB	45.5	23.5	114	N.D.	49.5	DIABASE	POOR
206+00 IB	44	22	111	57	55.5	ARGILLITE/DIABASE	POOR TO GOOD

NOTES:
1. N.D. = SAMPLES NOT OBTAINED AND NO DETERMINATION MADE.
2. ELEVATIONS REFER TO MBTA RED LINE DATUM.

4.3.1 Overburden Soils

The overburden materials which were encountered at the Test Section have been divided into four principal strata. In order of increasing depth below ground surface, they are:

1. Miscellaneous Fill,
2. Outwash Sand and Gravel,
3. Marine Clay, and
4. Glacial Till.

The glacial till was the major soil unit investigated in detail in the area and the only overburden unit through which the tunnel was constructed in the study area. As the overlying sediments have been discussed in Section 4.2, only the glacial till deposits will be described in more detail.

The glacial till unit consists of a thick deposit of generally a very dense, glacially consolidated mass which directly overlies the bedrock. Ranging in particle size from silt and clay to boulders, it is generally neither stratified nor sorted according to size. However, several lenses and pockets of clay, silt, or sand were encountered in the explorations. Boulders and cobbles, up to 3 feet in diameter, were encountered in the explorations. It was anticipated that boulders in excess of 3 feet would occasionally exist within the glacial till unit.

The test boring data indicated the presence of the glacial till unit described above. Nuclear logging techniques also indicated that the glacial till unit (Unit C) may be further divided into four subunits. The subunits (C_1, C_2, C_3 and C_4) are shown on the profiles in Appendix B, with general descriptions of their inferred composition.

Subunit C_3 was indicated, based on nuclear logs, to be a semi-continuous clay layer with an average thickness of 5 feet within the glacial till stratum. The test borings conducted throughout the site area, including the design phase explorations, did not reveal the presence of this relatively uniform clay layer, but rather occasional and isolated pockets of clay. This clay layer, Subunit C_3, is not a part of the overlying marine clay unit (Unit B) identified on the Geologic Profiles.

4.3.2 Bedrock

4.3.2.1 Description of Rock Mass - The geological and structural classification of the rock mass was determined by visual analysis of the recovered rock core utilizing the classification systems and terminology summarized in Appendix A. Photographs of rock core and more detailed geological information regarding the petrographic characteristics of the rock are presented in the Stage I report.

The host rock type in the Test Section, referred to as the Cambridge Argillite, is a slightly metamorphosed greenish-gray shale which varies from soft, severely fractured and weathered near its surface to very hard and fresh (the quality generally improving with increasing depth).

Secondary alteration, which probably accompanied later tectonic movements and intrusions of major igneous dikes, has further altered the argillite. Severe fracturing and brecciation, and, in many instances, subsequent healing to a competent rock, has created a highly intruded rock mass of varying composition.

Numerous veins and stringers of secondary minerals, which include calcite, pyrite, epidote, and quartz, were found in the argillite. Calcite, the most predominant mineral, has also cemented ancient joints and shears which are no longer considered as structural discontinuities in the rock mass. Occasional transition zones to sandstone and volcanic tuff were also noted within the argillite unit. Typically, these zones varied from a few inches to a few feet in thickness.

An igneous rock consisting primarily of green, highly fractured diabase intruded the argillite in the vicinity of Stations 204+00 to 205+50 OB and 204+50 to 206+00 IB. The diabase is believed to be one continuous dike. The greatest vertical thickness of the dike encountered in the explorations was 20.5 feet in TSC 10. However, previous design phase explorations in the same vicinity penetrated a vertical thickness of approximately 40 feet of diabase.

The horizontal extent of the diabase dike, measured along the centerline of the proposed tunnels, was anticipated to vary from 80 feet in the inbound tunnel to 110 feet in the outbound tunnel. Several isolated and smaller igneous dikes were indicated immediately north of the major dike. There was also much interfingering of the diabase and argillite near their associated contact zones.

The estimated contacts between the argillite and diabase dikes are shown on the Geologic Profiles, Appendix B. The diabase was expected to constitute the majority of the rock in the mixed face section of the outbound tunnel and in areas where some rock cover exist. Argillite was expected to be the principal rock type excavated in the mixed face portion of the outbound tunnel.

Surficial weathering and hydrothermal alteration have further altered both bedrock types near their surface and along mineralized zones or contacts. In some instances, this severe alteration has created kaolinization and formed clay or clay seams within the rock mass.

A zone of decomposed rock was identified on the subsurface exploration summaries and geologic profiles. This zone has lost its structural integrity and more closely resembled a soil. The decomposed rock zone consisted primarily of a clayey sand with pieces and fragments of intermixed un-weathered rock.

4.3.2.2 Bedrock Structure

a. Argillite - The Test Section is located on the north flank of the Charles River syncline. The regional strike of the bedrock structure is about N70°W and the strata dips to the south at about 5 to 15 degrees.

An important feature to note is the presence of planes of weakness or separations along which the rock either has been broken by past geological forces or along which it would be likely to break during tunnel excavation. These planes of weakness or fractures are controlled by bedding planes and rock mass discontinuities such as joints, shears, and faults (Figure 4-3).

Joints made up the vast majority of the discontinuities observed and are in sufficient number to impart a blocky nature to the rock mass. Jointing was ubiquitous in the rock core from the Test Section and usually occurred in sets. The joint surfaces ranged from smooth to rough, and the joint planes ranged from planar to curvilinear to very irregular. The joints were usually coated with calcite or other minerals from secondary mineralization. Some joints were open and not coated, while others were closed tight and cemented with secondary minerals. The joints generally dipped between 20 and 80 degrees.

The joint spacing ranged from less than 1 inch to over 3 feet. Orientations of the joints could not be determined

FIGURE 4-3. STRUCTURAL DISCONTINUITIES, PORTER SQUARE PILOT TUNNEL

from the borings since orienting techniques were not used. The regional joints are known to dip steeply and to strike approximately north to northeast, while a second set strikes nearly east and dips south.

Joints which had been opened due to ice jacking and subsequently filled with sediment were reportedly observed in the excavations south of Porter Square and, for the Pusey Library just south of Harvard Square. Some blocks of rock were entirely separated from the bedrock mass by a few inches of reworked till and the joints were filled with sediment. This condition, where identified in the Test Section, has been grouped in the decomposed rock zone previously described.

Some shears (naturally occurring rock discontinuities along which minor displacement has occurred) may have been classified as joints, since evidence of displacement was not apparent. Calcite mineralization, or occasional clay gouge material, is usually associated with shear fractures. The majority of the rock has been crushed and brecciated by former tectonic movements and subsequently healed by secondary mineralization.

Various zones of extremely fractured rock which were identified on the summary logs may be zones of severe jointing or may indicate fault zones. It was anticipated that faulting of the bedrock occurred in association with later tectonic movements when igneous dikes intruded the argillite.

The fracture zones and open joints are principal avenues for high volume water transport in the rock mass. The continuity of these fractures could not be determined, but the frequency of fracturing which was encountered would indicate water transmissibility. In-situ testing indicated an average permeability in the upper portions of the rock mass of about 10^{-5} cm/sec, decreasing to 10^{-6} cm/sec in the lower, more competent portions. It was expected that local fracture zones which display continuity would yield significant quantities of water during excavation.

The rock quality designation (RQD) and average length of core were measured in the borings to help define the overall rock mass quality. These parameters were generally lowest near the bedrock/glacial till interface. The RQD and average length of core generally increased from depths of 10 to 30 feet below the bedrock surface and were usually higher for the argillite than for the igneous rocks.

b. Diabase - Igneous intrusions in the form of diabase dikes have intruded the argillite within the Test Section. These intrusions are steeply dipping to the southwest and generally strike in a northwest-southeast direction. They have been exposed to surficial weathering and hydrothermal alterations along open joints and shears.

The diabase is a much harder, more brittle rock than the argillite. Due to its coarser texture and mineralogy, it contains much less clay and gouge material associated with its structural discontinuities. However, the diabase dikes display a higher percentage of discontinuities due to their tendency to more easily rupture during regional or local disturbances.

The massive nature of the igneous dikes and their total lack of bedding or laminations has created very rough and highly irregular fracture patterns. Although highly fractured, the discontinuities tend to be randomly oriented and were believed to lack continuity over long distances.

The various structural features determined from the recovered rock core are summarized in Appendix B.

4.3.3 Groundwater

Measurements made during Stage I in groundwater observation wells and piezometers installed within the Test Section indicated that there are two distinct piezometric surfaces. The upper surface was associated with hydrostatic heads in the near surface outwash sand and gravel stratum. The lower surface reflected water levels in the permeable zone in the upper surface of the fractured bedrock.

The two piezometric surfaces are essentially isolated from each other by the relatively impermeable marine clay and glacial till strata. Consequently, water levels measured in the shallow wells may not be the same as water levels measured in deep piezometers.

During the execution of the exploration program at the Test Section, the tunnel construction contractor initiated dewatering operations from a deep well installed in the fractured rock zone near the site. Measurements made in the Test Section piezometers and observation wells are summarized in Table 4-2.

TABLE 4-2. SUMMARY OF GROUNDWATER OBSERVATIONS (IN FEET BELOW GROUND SURFACE)

OBSERVATION DATE	NOTE	TSC16 (OW) (4)	TSC16 (PZ)	TSC17 (OW)	TSC18 (PZ)	TSC19 (OW)	TSC21 (OW)	TSC22 (OW)	TSC24 (OW)	TW1 (TW)	TA3 (PZ)	TB6 (PZ)	GW8 (PZ)	GW11 (OW)
May 1979	(1)	11.7	--	12.0	12.6	11.3	13.0	10.3	10.5	11.3	10.4	10.9	--	--
7/26/79	(2)	8.3	38.7	16.8	40.5	10.9	49.2	10.6	18.4	60.6	37.6	62.0	36.4	42 +
8/02/79	(2)	9.5	39.3	20.1	42 +	10.9	48.6	10.5	26.3	60.3	38.0	61.8	37.6	42 +
8/13/79	(2)	8.5	39.5	23.2	41.5	11.2	49.7	10.9	30.3	60.4	33.5	61.7	36.5	50.5
8/20/79	(2)	8.6	40.4	24.0	41.7	11.2	50.5	10.8	30.8	60.9	39.0	--	38.5	51.0
8/28/79	(2)	8.3	40.8	24.7	42.3	11.1	50.7	10.9	31.7	61.0	39.3	61.8	38.9	50.9
9/05/79	(2)	8.7	40.5	25.1	43.0	11.5	51.2	11.0	32.1	--	40.2	62.5	39.5	51.4
9/27/79	(3)	8.7	21.9	25.7	22.3	11.8	21.0	11.1	20.0	--	20.7	20.5	19.7	21.5
10/15/79	(2)	8.7	41.0	23.4	43.5	11.5	51.4	11.0	32.5	62.0	39.5	63.2	39.6	51.7
10/25/79	(2)	8.7	41.4	22.9	43.6	11.6	51.9	11.0	32.5	--	--	62.6	39.8	51.6

NOTES:
(1) Initial water reading obtained at completion of the test boring.
(2) Red Line construction pumping from dewatering well DW5 significantly lowered water table in site area.
(3) Pumping from DW5 terminated between 9/14/79 and 10/02/79.
(4) Observation well TSC16 plugged and unreliable observations.
(5) Subsurface Exploration Number TSC16
Observation Well (OW)
Piezometer (PZ)
Pump Test Well (TW)

Initial readings taken in May 1979 indicated a relatively constant depth to groundwater (10 to 13 feet) at all observation wells and piezometers. Subsequent to the start of pumping from dewatering well DW5, the water levels in the deep piezometers dropped to depths of 30 to 40 feet (TSC 16, 18, 24) while shallow observation wells remained at depths of 8 to 12 feet (TSC 16, 19, 22). When dewatering operations were terminated during the period 14 September to 2 October 1979, rapid recovery was evident in deep piezometers (TSC 16, 18, 24) while little or no response was evident in shallow observation wells (TSC 16, 19, 22).

The data from instruments TW1, TA3, TB6, GW8, and GW11 installed by others, are incomplete. The available data, presented in Table 4-2, indicate trends that are consistent with the groundwater observations described in the preceding paragraphs.

The static piezometric surface in both zones ranges from elevation 116 to 121 feet. The lower surface was depressed by pumping from the fractured rock, whereas, the upper surface remained relatively stable. Thus the two piezometric surfaces are isolated and do not necessarily respond together. It was considered unlikely that all water levels at this Test Section would approach static conditions until after construction dewatering for the project is complete.

5. PREDICTED SETTLEMENTS

5.1 GENERAL

As part of the initial work for the Stage II studies, predictions were made of the anticipated ground surface settlements due to tunneling through the Test Section. Although the prediction of an end result, such as settlement, is not normally part of a demonstration type study, the prediction of ground settlement is part of the engineering and geotechnical evaluations conducted on every urban tunneling project. It is well known that over prediction of settlement may cause extra costs in building protection that may be unnecessary. Under prediction of settlement may result in unsafe conditions developing during construction. The effects and potential costs of either situation are undesireable.

The predictions of ground settlement were made prior to tunneling within the Test Section. The predictions are based on the subsurface conditions developed during the Stage I explorations and available information about anticipated tunneling methods and equipment.

This section summarizes, and Appendix C presents, the original settlement predictions. Section 9 presents a detailed discussion of these predictions, compares them with the measured settlements, and suggests changes to the procedures based on these comparisons.

5.2 NORTH SECTION - ROCK

The tunnel alignment through the northern portion of the Test Section is entirely within the Cambridge Argillite bedrock. It was anticipated that the tunnels would be driven into the Test Section from the north using drill-and-blast methods for rock excavation. Temporary support would be provided by steel ribs.

Potential ground movements that occur over tunnels driven through rock are difficult to predict based on empirical techniques. Since the movements may be caused by stress relief that occurs around the tunnel opening or by movement of blocks of rock along joints, the total ground deformation is highly dependent on the construction technique. This includes the method of excavation, amount of rock cover over the tunnel, amount of rock excavated with each blast, the effectiveness of the temporary support, and the time of support installation.

Based on judgement concerning the amount of rock movement (let down) caused by the construction processes, the magnitude of ground surface settlement over the rock tunnels was estimated to be less than 1/4 inch.

5.3 SOUTH SECTION - SOFT GROUND

Based on information available at the time of the prediction, it was anticipted that the tunnels through the soft ground portion of the Test Section would be shield driven. Starting at Flagstaff Park, the tunnels would be driven northward into the Test Section, stopping when mixed face conditions were reached. The soft ground tunnels would have temporary support consisting of steel ribs and wood lagging assembled in the tail of the shield, with the ribs and lagging expanded hard against the ground after leaving the shield. Following excavation and temporary support, the tunnels would be lined with an 18-inch thick, cast-in-place concrete liner.

Cording et al. (I-3) separate the ground movements due to soft ground tunneling into two categories:

a. Large sudden ground loss due to raveling, flowing, or running of the ground that progresses above the tunnel crown This ground loss generally develops at the tunnel face.

b. Ground loss under normal conditions. Normal conditions include consideration of subsurface conditions, construction methods, and workmanship.

Predictions of ground settlement in this study are based on the second category of ground loss causing settlement. The first category-large, sudden ground loss-is excluded due to its random and unpredictable nature.

The magnitude of settlements was determined from case history studies by Cording et al. (I-3), which relate the estimated volume of ground loss at the tunnel heading to the volume and geometry of the settlement trough at the ground surface. The resulting prediction for the soft ground settlement in the Test Section was estimated to be approximately 0.6 inch for a single tunnel. When consideration was made for twin tunnels, the settlement of the centerline of the settlement trough was estimated to be approximately one inch.

5.4 MIDDLE SECTION - MIXED FACE

The anticipated construction sequence for the mixed face section was to excavate pilot drifts southward from the full face rock tunnels. The drifts would extend to the point where soil was encountered in the crown. The tunneling shields would then be driven through this area from the south.

The prediction of ground deformations in mixed face tunneling conditions is not readily quantifiable. Although the volume of soil encountered in the tunnel section is less than for a soft ground tunnel, the support problems are much more difficult. In general, any excavation sequence will result in a greater volume of ground loss and, therefore, potentially greater surface settlement.

With due consideration of anticipated construction methods and the subsurface conditions, the ground surface settlement over the tunnels in the mixed face area was estimated to be similar to or slightly greater (approximately 1/4-inch greater) than the settlement over the soft ground tunnels.

6. DESCRIPTION OF INSTRUMENTATION

6.1 GENERAL

Surface and deep settlement points and inclinometers were installed throughout the Test Section to monitor vertical and horizontal movements about the tunnel excavations. Piezometers and observation wells were installed to monitor changes in groundwater levels due to tunnel construction.

Except for the surface settlement points, the instrumentation was installed in completed Stage I boreholes during the period from 5 April to 29 June 1979. Surface and building settlement points were established 15 to 18 April 1980.

Instruments were selected for the Test Section on the basis of their respective economies, history of field performance, compatibility with Stage I boreholes, and ability to monitor desired parameters. Different types of settlement instruments and inclinometer casings were utilized in order to compare instrument performance.

The following sections describe the instrument locations and, for each type of instrumentation, the equipment, installation, monitoring, and data processing.

6.2 LOCATION OF INSTRUMENTATION

Section 3-3 describes the rationale for instrument placement. Figure 3-6 shows the instrumentation location plan for the Test Section, and Table 3-1 details the location, type and elevation for each instrument. Figures 6-1, 6-2 and 6-3 show the profiles of each of the three high intensity instrumentation sections.

6.3 SURFACE AND BUILDING SETTLEMENT POINTS

6.3.1 Equipment and Installation

A total of 87 settlement points were installed within the Test Section (see Figure 3-6), as follows:

1. Twelve building points, most as file cuts in masonry and stone, established on structures adjacent to the Test Section.

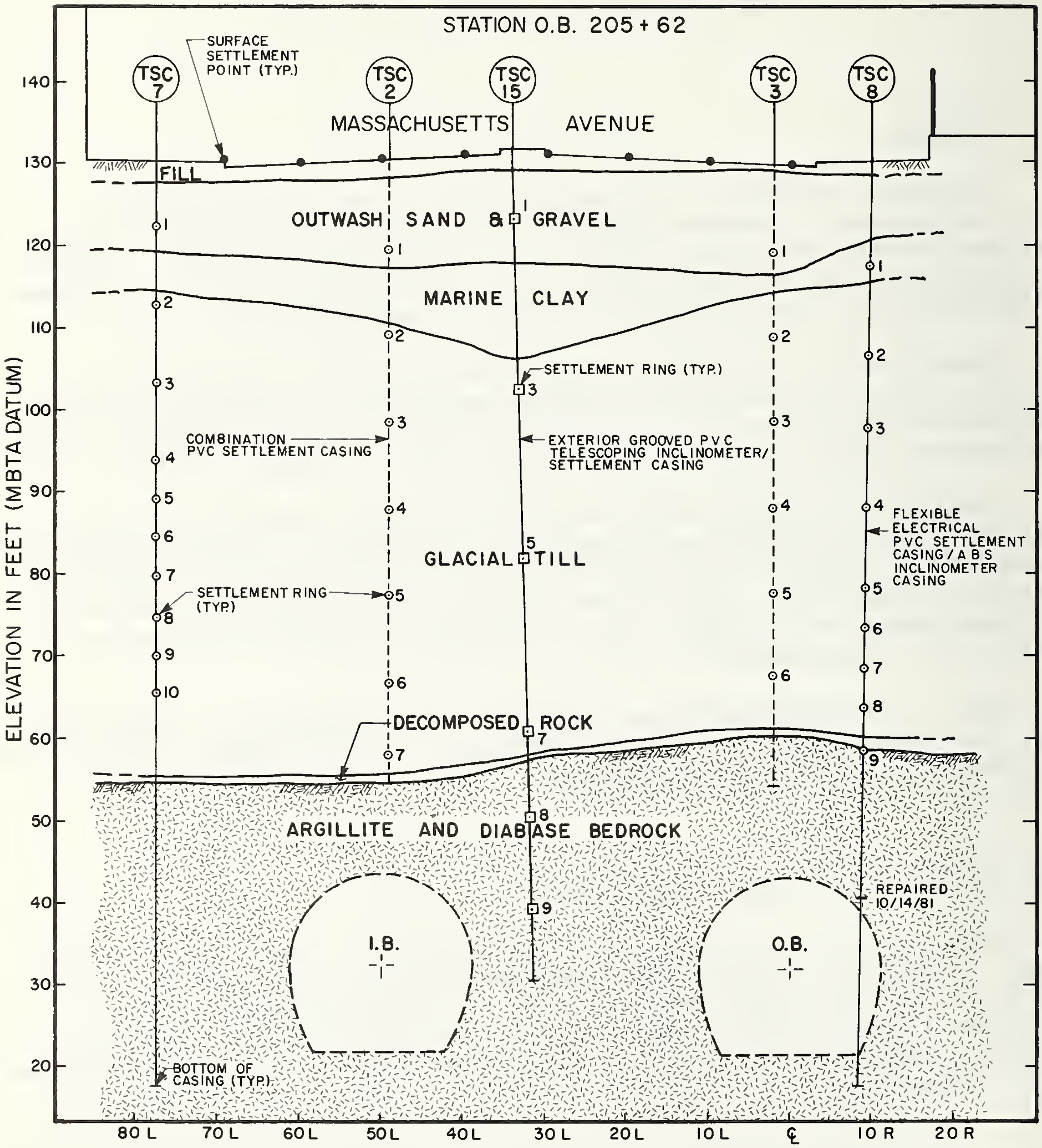


FIGURE 6-1. NORTH SECTION PROFILE

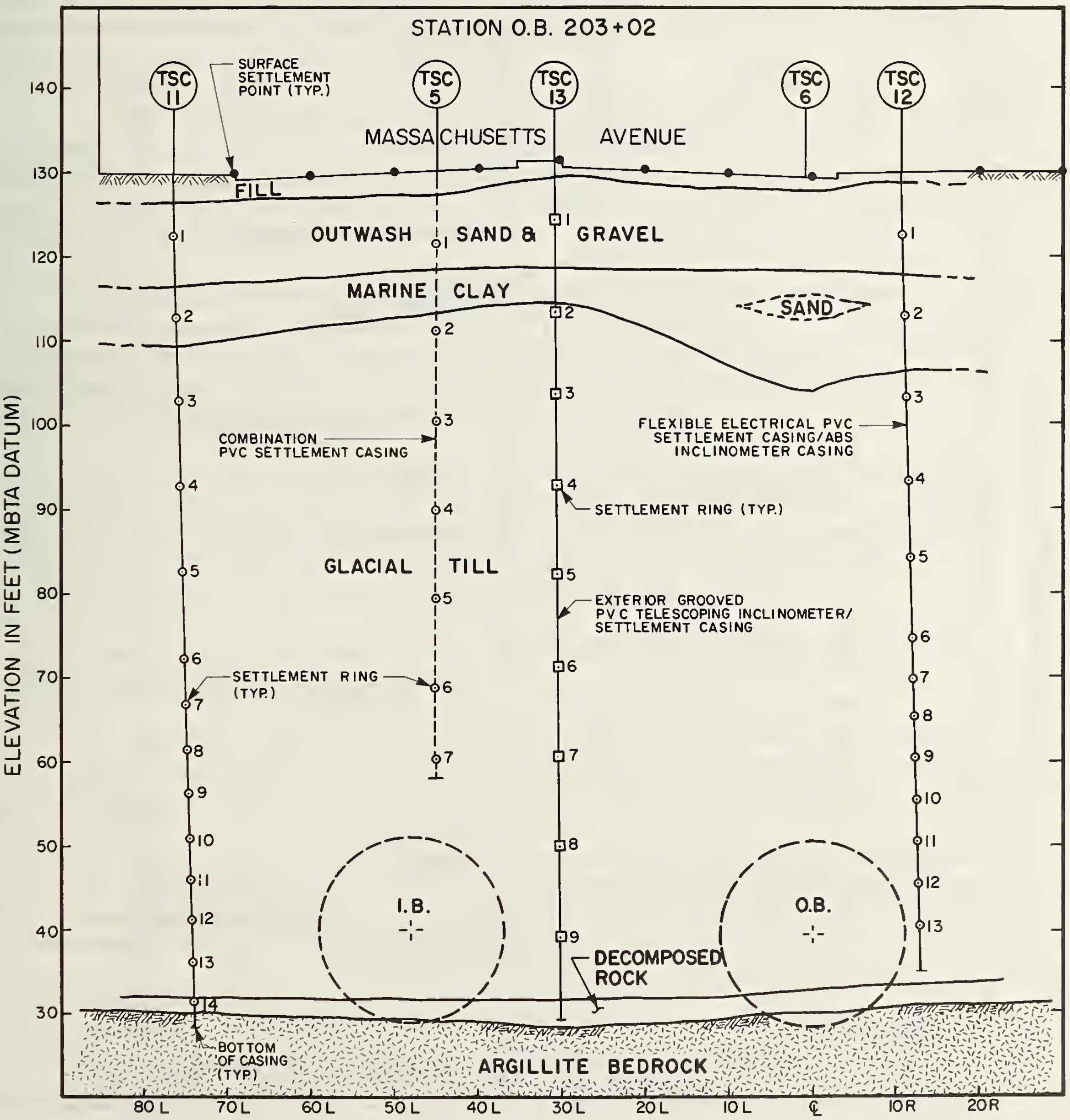


FIGURE 6-2. SOUTH SECTION PROFILE

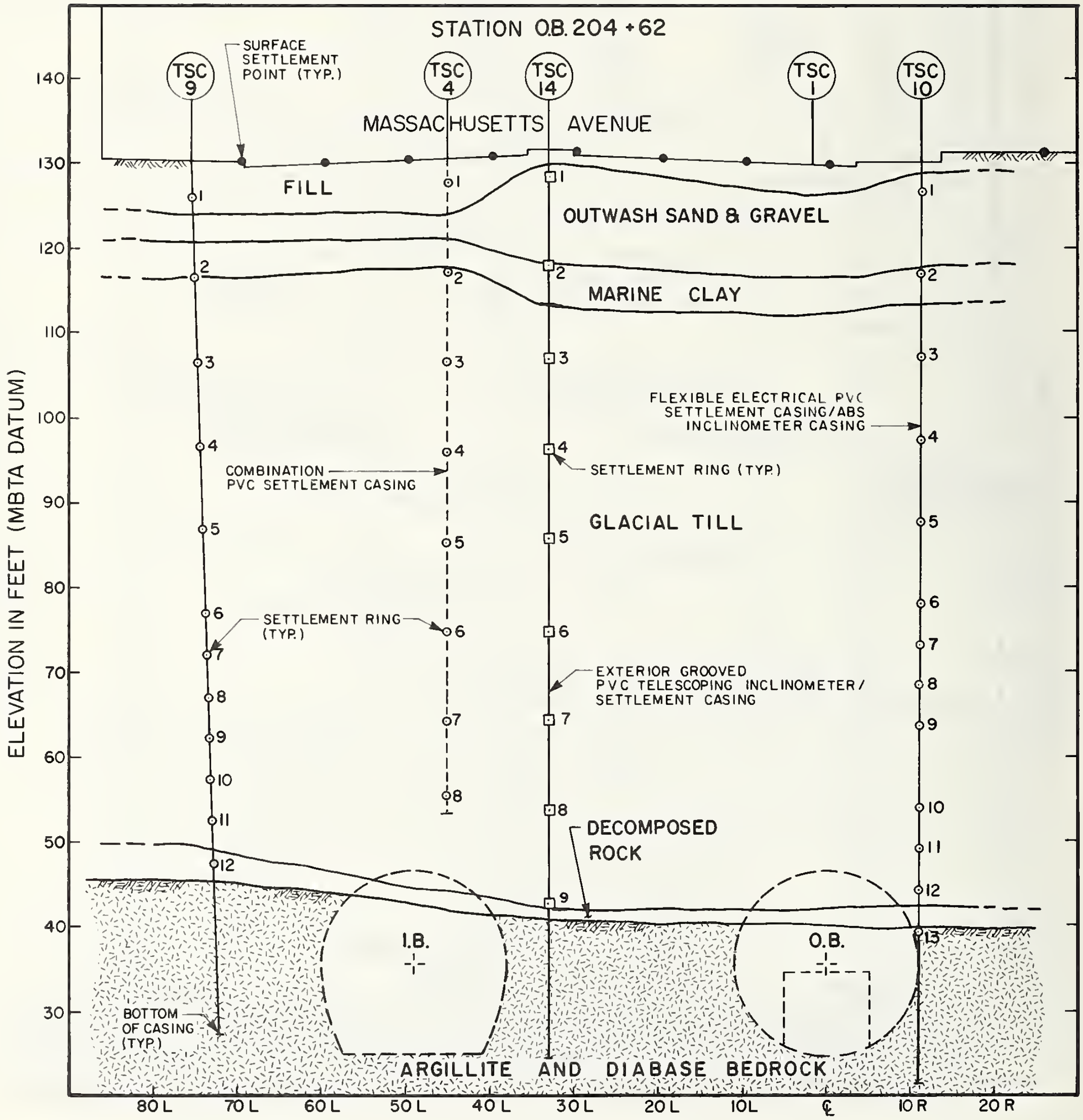


FIGURE 6-3. MIDDLE SECTION PROFILE

2. Two lines of concrete nails in the ground surface above each of the tunnel centerlines, at approximately 20 foot spacings. Over the inbound tunnel, these nails ran from stations 203+32 to 206+72 and over the outbound centerline, from Stations 202+07 to 206+62.

3. Three lines of concrete nails in the ground surface, perpendicular to the tunnel centerlines, at each of the three high intensity sections. These points were spaced at approximately 10 feet. The extent of these lines was limited by the presence of nearby structures.

The nails for the surface settlement points were driven into the pavement of Massachusetts Avenue and concrete sidewalks and paved areas within the Test Section. Nails were installed with the nail head extending just above the adjacent ground surface making a distinct level point.

The reference benchmark for the surface and building settlement points was a fire hydrant approximately 300 feet west of Massachusetts Avenue on Shepard Street (near north section). This remained as the level control for the duration of the project and was designated TBM-1.

The following equipment was used to survey surface and building settlement points:

a. Lietz Model B-1 Engineering Precision Automatic Level equipped with Parallel Plate Micrometer and Special "Wedge" Reticle (Lietz Nos. 7372-50 and 732-14, respectively).

b. Wild 10 Foot Invar Staff, Model GPLE 10, graduated in feet and hundredths, with precision machined base and bubble level.

6.3.2 Monitoring

The survey procedures used for optical leveling in the Test Section included the following:

a. All level circuits were started and closed to a permanent benchmark (TBM-1).

b. Turning points were established during the level circuit so that backsight and foresight were approximately equal and kept to 100 feet or less.

c. Settlement points were optically leveled as part of the survey using an Invar staff. No temperature corrections were required due to the small temperature coefficient of the Invar staff. The staff was maintained in a vertical position at each surveyed point by using a bubble level mounted on the staff. In some instances, when the staff could not be placed on the settlement point, an engineer's 6-foot wooden rule was used.

d. Staff readings were measured directly to thousandths (0.001) of a foot with use of the micrometer and estimated to ten thousandths (0.0001).

e. All level circuit closures were less than 0.005 feet. If a closure greater than this value was obtained, the circuit was surveyed again.

f. The optical survey data was adjusted for circuit closure error by dividing the number of setups by the error and distributing this difference equally among the turning points.

6.3.3 <u>Data Processing</u>

Optical level surveys of building and settlement points were reduced in tabular form (Figure 6-4). After initial elevations were determined, subsequent survey elevations were logged on the data summary sheet and the difference in elevations taken as the movement of the point.

6.3.4 <u>Surface Settlement Points By Others</u>

A grid of approximately 100 surface settlement points were installed and monitored by the construction contractor as partial requirement of the contract. These instruments were read periodically by the contractor, and the data was then provided to the Massachusetts Bay Transit Authority (MBTA).

6.4 DEEP SETTLEMENT POINTS

Three different types of deep settlement casings were installed throughout the Test Section for this study. They consisted of:

1. electrical settlement casing,
2. telescoping inclinometer/settlement casing, and
3. combination settlement casing.

PROJECT RED LINE TEST SECTION
CAMBRIDGE, MA
CLIENT DOT/TSC

LEVEL SURVEY DATA SUMMARY

FILE NO. 428401
DATUM MBTA RED LINE
BENCH MARK TBM-1

DATE	OB 202+62 ℄		OB 202+82 ℄		OB 203+22 ℄		OB 203+42 ℄		OB 203+67 ℄		OB 203+87 ℄		OB 204+12 ℄		OB 204+37 ℄		REMARKS	BY
	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF	ELEV	DIFF		
4-18-80											OBSTRUCTED		129.2147		129.2695			RFB
4-22	129.2704		129.2844		128.8775		128.9620		OBSTRUCTED									RFB
4-30	129.2691		129.2829		128.8750		128.9602		129.1256		OBSTRUCTED		129.2155		129.2093			RFB
5-2	129.2720		129.2864		OBSTRUCTED		128.9642		129.1287		129.2014		129.2184		129.2722			RFB
	129.2705		129.2846		128.8763		128.9621		129.1272		129.2014		129.2162		129.2703		INITIAL	
10-8-80	129.265	-.006	129.281	-.004	128.876	0	128.960	-.002	129.122	-.005	129.193	-.008	129.214	-.002	129.268	-.002		RFB
11-3-80	129.267	-.004	129.282	-.003	OBST	-	OBST	-	129.121	-.006	129.193	-.008	129.213	-.003	129.264	-.006		RFB
11-14-80	129.264	-.007	129.280	-.005	128.875	-.001	128.957	-.005	OBST	—	129.189	-.012	129.209	-.007	129.264	-.006		RFB
12-18-80	129.269	-.002	129.280	-.005	128.884	+.008	128.966	+.004	OBST.	—	OBST.	—	OBST.	—	129.262	-.008		RFB
5-19-81	129.266	-.005	129.281	-.004	128.876	0	OBST.	—	129.125	-.002	OBST	—	129.211	-.005	129.265	-.005		RFB
6-17-81	129.265	-.006	129.277	-.008	128.869	-.007	128.950	-.012	129.109	-.018	129.186	-.015	129.204	-.012	129.258	-.012		DST
7-30-81	129.266	-.005	129.278	-.007	128.870	-.006	OBST	—	129.107	-.020	OBST	—	129.207	-.009	129.258	-.012		RFB
10-13-81	129.242	-.029 / 0	129.260	-.025 / 0	128.857	-.019 / 0	128.938	-.024 / 0	129.096	-.031 / 0	OBSTRUCTED	—	129.200	-.016 / 0	129.253	-.017 / 0	FOR O.B. NORTH	JSM
10-16-81	129.231	-.040 / -.011	129.248	-.037 / -.012	128.853	-.023 / -.004	OBSTRUCTED	-	129.095	-.032 / -.001	129.176	-.045 / 0	129.199	-.017 / -.001	129.255	-.015 / +.002		JSM
10-19-81	129.226	-.046 / -.016	129.246	-.039 / -.014	128.851	-.035 / -.006	OBSTRUCTED	-	129.096	-.031 / 0	OBSTRUCTED	-	129.198	-.018 / -.002	129.253	-.017 / 0		JSM
10-21-81	129.222	-.049 / -.020	129.239	-.046 / -.021	128.847	-.029 / -.010	OBSTRUCTED	-	129.093	-.034 / -.003	129.174	-.027 / -.002	129.198	-.018 / -.002	129.254	-.016 / +.001		JSM
10-28-81	129.213	-.058 / -.029	129.227	-.050 / -.033	128.826	-.050 / -.031	128.910	-.052 / -.028	129.080	-.047 / -.016	129.167	-.034 / -.009	129.195	-.021 / -.005	129.252	-.018 / -.001		JSM
10-30-81	129.214	-.057 / -.028	129.228	-.057 / -.032	128.825	-.051 / -.032	128.904	-.058 / -.034	129.072	-.055 / -.024	129.166	-.035 / -.010	129.197	-.019 / -.003	129.255	-.015 / +.002		JSM
11-3-81	129.211	-.060 / -.031	129.226	-.059 / -.034	128.820	-.056 / -.037	128.900	-.062 / -.038	OBSTRUCTED	—	OBSTRUCTED	—	129.194	-.022 / -.006	129.253	-.017 / 0		JSM
11-9-81	129.211	-.060 / -.031	129.224	-.061 / -.036	OBSTRUCTED		OBSTRUCTED		129.062	-.065 / -.034	129.159	-.042 / -.017	OBSTRUCTED		OBSTRUCTED			JSM
11-11-81	129.206	-.065 / -.036	129.219	-.066 / -.041	OBSTRUCTED		OBSTRUCTED		129.032	-.095 / -.064	129.145	-.056 / -.031	129.188	-.028 / -.012	OBSTRUCTED			JSM
11-25-81	129.212	-.059 / -.030	129.226	-.059 / -.034	128.813	-.063 / -.044	128.857	-.105 / -.081	129.019	-.108 / -.077	129.136	-.065 / -.040	129.185	-.031 / -.015	129.252	-.010 / -.001		JSM
1-8-82	129.204	-.067 / -.038	129.215	-.070 / -.045	128.803	-.073 / -.054	128.829	-.133 / -.109	OBSTRUCTED		129.107	-.094 / -.067	129.153	-.063 / -.047	OBSTRUCTED			JSM
1-29-82	129.233	-.038 / -.007	129.219	-.066 / -.041	128.813	-.063 / -.044	OBSTRUCTED		129.031	-.096 / -.065	OBSTRUCTED		OBSTRUCTED		OBSTRUCTED			JSM
2-24-82	129.203	-.068 / -.037	129.217	-.068 / -.043	128.818	-.058 / -.039	128.841	-.121 / -.097	129.026	-.101 / -.070	129.116	-.085 / -.060	129.161	-.055 / -.039	129.190	-.080 / -.063		JSM

ELEV = ELEVATION IN FEET.
DIFF = DIFFERENCE FROM INITIAL ELEVATION.

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS

FIGURE 6-4. LEVEL SURVEY SUMMARY SHEET (TYPICAL)

Each system is described in detail in the following sections.

6.4.1 Electrical Settlement Casing (TSC 7, 8, 9, 10, 11, and 12)

6.4.1.1 Equipment - This casing is continuous 3.0-inch ID flexible corrugated PVC pipe, manufactured by Advanced Drainage Systems, Palmer, Massachusetts. Stainless steel wire rings can be bonded to the casing at any interval desired by the installer, 5 and 10-foot intervals were used at the Test Section. The casing's corrugations bond to the grouted borehole and it then moves with the adjacent ground. Settlements are determined by electrically monitoring the elevations of the stainless steel rings. Figure 6-5 schematically shows the casing installation and monitoring equipment.

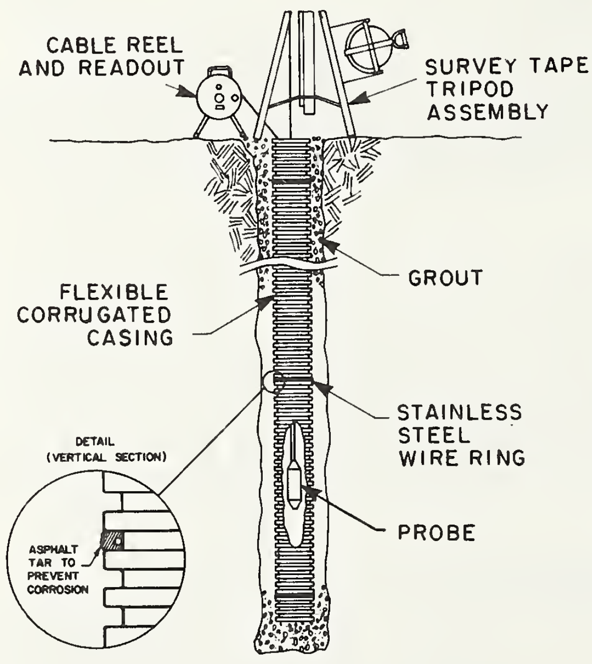


FIGURE 6-5. SCHEMATIC OF ELECTRICAL SETTLEMENT CASING INSTALLATION (SLOPE INDICATOR COMPANY)

These casings were monitored using the following equipment:

1. a tripod with an adjustable reference leg,

2. a 100-foot steel tape, graduated in hundredths, and

3. Sondex™ electrical settlement probe and readout/ control unit (Figure 6-6).

FIGURE 6-6. SONDEX™ SETTLEMENT MONITORING SYSTEM (SLOPE INDICATOR COMPANY)

Precise locations of the wire rings were determined by means of the electrical probe, which uses the induction principle, to register a maximum on the readout/control unit when it is centered on the wire ring. The electrical probe, (1.75 in. OD X 12 in. long) Model No. 50819, is manufactured by the Slope Indicator Company, Seattle, Washington. It is commonly designated as the Sondex™ probe. The probe is connected to a readout/control unit via an electrical cable. A 100-foot steel tape is attached to the probe to monitor depth of the ring sensors.

6.4.1.2 Installation - The settlement casings were installed in combination with ABS inclinometer casing, Section 6.5.1.1. Procedures used to install the electrical settlement casings, Figure 6-7, were as follows:

a. A 6.0-inch ID overburden borehole and 3.0-inch ID rock core hole were advanced to the depths required for exploration. Depending on soil conditions, 10 to 15 feet of SW (6.0-inch ID) casing was placed to stabilize the upper portion of the boring.

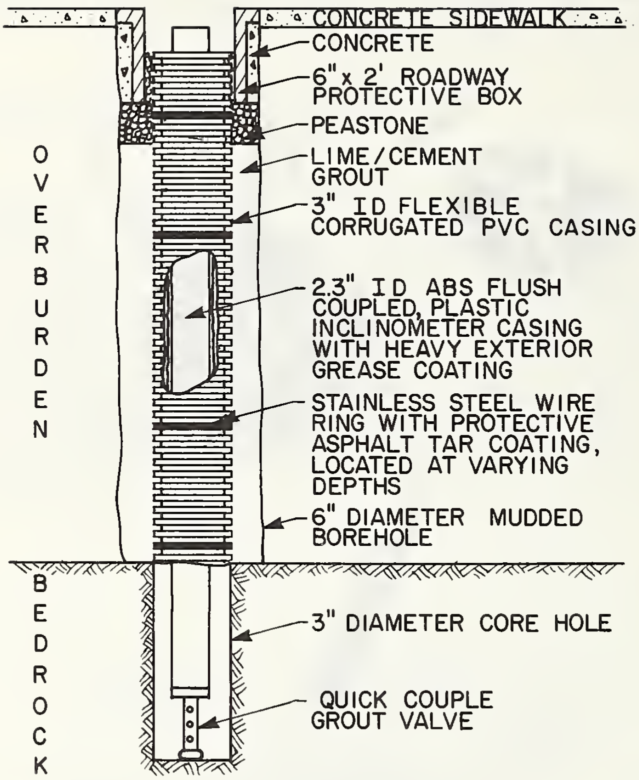


FIGURE 6-7. COMBINATION INCLINOMETER AND ELECTRICAL SETTLEMENT INSTALLATION

b. HW (4.0-inch ID) casing was then lowered to the top of bedrock to sleeve the mudded borehole to make settlement casing installation easier.

c. The continuous 3.0-inch ID electrical settlement casing was installed in the HW casing. Stainless steel wire rings had been placed at the desired locations and protected with asphalt mastic and electrical tape prior to installation.

d. 20-foot lengths of the ABS inclinometer casing were assembled on the ground surface with a pop riveted flush coupling that was sealed with caulk and electrical tape. The bottom casing section contained a quick couple grout valve connected to a 5/8-inch ID grout hose.

e. A heavy exterior grease coating was applied to the casing to keep it from "hanging up" in the corrugated settlement casing. The 20-foot ABS casing sections were then assembled and lowered into the borehole. The grout hose was carried through the ABS casing to the ground surface.

f. The system was then grouted using a 3 to 1 lime/cement grout. This grout was used since its hardened consistency was felt to be close to that of the till overburden. Upon the completion of grouting, the grout hose was pulled off the quick couple valve on the bottom casing section and removed.

g. The upper portion of the borehole was then backfilled with peastone and a protective roadway box installed.

6.4.1.3 Monitoring - The monitoring procedures for the electrical settlement system are summarized below:

a. After the roadway box cover was removed, the reference leg of the tripod was placed on the instrument box rim's predetermined reference mark. The reference leg was then centered over the casing and plumbed with the bulls-eye level attached to it.

b. The electrical probe was attached to the 100-foot tape and lowered into the casing.

c. The control unit meter deflection was observed as the probe was lowered. The meter deflected to a maximum and "buzzed" when the probe was close to the ring. By adjusting the control unit's sensitivity and the depth of the probe, the ring was precisely located. Once the probe zeroed in on the ring, the steel tape was clamped to the reference leg (Figure 6-8).

d. The reference leg was checked to see if it was still plumb and on the reference mark. If not, the leg was properly adjusted and step c was repeated.

e. The tape was read on the reference leg's vernier to 0.001 foot.

f. The clamp was released and the tape and electrical probe were lowered to the approximate level of the next lowest ring. Readings were taken in this manner until the probe was at the casing bottom.

g. The control/readout unit was then turned off, the reference leg was raised off the box rim, so as not to damage it, and the steel tape, cable and probe were removed from the casing. The probe can also sound the casing proceeding from bottom to top. This may be done after a downward survey to check the accuracy of the readings.

h. The casing cap and roadway box cover were replaced.

FIGURE 6-8. MONITORING SONDEX^{TM} SETTLEMENT CASING

6.4.1.4 Data Processing - Data from the electrical settlement surveys were processed similar to level survey data. After initial elevations were determined for each ring, subsequent survey elevations were logged and the difference in elevations taken as the movement of the ring. The instrument reference elevation was the elevation of the reference mark on the roadway box rim. This elevation was obtained from optical level surveys (Figure 6-9).

6.4.2 Telescoping Inclinometer/Settlement Casing (TSC 13, 14, and 15)

6.4.2.1 Equipment - This casing is 2.5-inch ID extruded PVC telescoping casing, Model No. C-4, manufactured by Terra Technology Corporation, Redmond, Washington, compatible for both inclinometer (See Section 6.5.2) and settlement applications. The casing is supplied in 10-foot lengths and was connected in the field with a 15-inch coupling. This coupling allows each 10-foot casing section to move up or down independently.

Settlements were monitored by means of a mechanical probe or hook, Model No. 50801, manufactured by Slope Indicator Company, Seattle, Washington, which locates the bottom of each casing section (Figure 6-10). The depth of the probe is then measured with a 100-foot steel tape connected to the probe.

PROJECT RED LINE TEST SECTION
CAMBRIDGE, MA
CLIENT DDT/TSC

SETTLEMENT PROBE DATA SUMMARY
CASING NO. TSC 12

FILE NO. 428401
DATUM MBTA RED LINE
BENCH MARK TBM-1

DATE	REF ELEV	RING NO. 1			RING NO. 2			RING NO. 3			RING NO. 4			RING NO. 5			REMARKS	BY
		DEPTH	ELEV	DIFF	DEPTH	ELEV	DIFF	DEPTH	ELEV	DIFF	DEPTH	ELEV	DIFF	DEPTH	ELEV	DIFF		
3-13-80		7.535			17.223			26.843			36.419			45.896				JHC
4-11-80		7.527			17.211			26.847			36.412			45.893				ALH
5-7-80		7.534			17.216			26.837			36.418			45889				RFB
	129.370	7.532	122.538	0	17.217	112.853	0	26.842	103.228	0	36.416	93.654	0	45.893	84.177	0		
10-8-80	129.369	7.532	122.537	-.001	17.214	112.855	+.002	26.844	103.225	-.003	36.409	[illegible]	+.006	45.889	[illegible]	+.003		RFB
11-3-80	129.369	7.532	122.537	-.001	17.214	112.855	+.002	26.846	103.223	-.005	36.412	93.657	+.003	45.888	84.181	+.004		RFB
11-14-80	[illegible]	7.534	122.532	-.006	17.211	112.855	+.002	26.845	103.221	-.007	36.412	93.654	0	45.891	84.175	-.002		RFB
12-18-80	129.368	7.531	122.537	-.001	17.219	112.849	-.004	26.844	103.224	-.004	36.420	93.648	-.006	45.891	84.177	0		RFB
6-19-81	129.365	7.541	122.524	-.014	17.221	112.844	-.009	26.846	103.219	-.009	36.424	93.641	-.013	45.903	84.162	-.015		DST
8-12-81	129.368	7.538	122.530	·008	17.219	112.849	-.004	26.844	103.224	-.004	36.416	93.652	-.002	45.893	84.175	-.002		DST
8-24-81	129.368	7.535	122.533	-.005	17.223	112.845	-.008	26.846	103.222	-.006	36.415	93.653	-.001	45.902	84.166	-.011		DST
10-8-81	129.354	7.545	122.509	-.029 / 0	17.223	112.831	-.022 / 0	26.849	103.205	-.023 / 0	36.424	93.630	-.024 / 0	45.908	84.146	-.031 / 0		JSM
10-14-81	129.354	7.546	122.508	-.030 / -.001	17.234	112.820	-.033 / -.011	26.854	103.200	-.028 / -.005	36.426	93.628	-.032 / -.002	45.928?	84.126?	-.051 / -.020		JSM
10-16-81	129.350	7.545	122.505	-.033 / -.004	17.217	112.833	-.020 / +.002	26.855	103.195	-.033 / -.010	36.420	93.630	-.024 / 0	45.904	84.146	-.031 / 0		JSM
10-19-81	129.356	7.542	122.514	-.025 / +.004	17.231	112.825	-.028 / -.006	26.856	103.200	-.028 / -.005	36.422	93.634	-.020 / +.004	45.916	84.140	-.037 / -.006		JSM
10-21-81	129.344	7.542	122.502	-.036 / -.007	17.223	112.821	-.032 / -.010	26.846	103.198	-.030 / -.007	36.419	93.625	-.029 / -.005	45.898	84.146	-.031 / 0		JSM
10-23-81	129.344	7.538	122.506	-.032 / -.003	17.228	112.816	-.037 / -.015	26.847	103.197	-.031 / -.008	36.418	93.626	-.028 / -.004	45.896	84.148	-.029 / +.002		JSM
10-27-81	129.329	7.536	122.493	-.045 / -.016	17.210	112.819	-.034 / -.012	26.846	103.183	-.045 / -.022	36.427	93.602	-.052 / -.028	45.907	84.122	-.055 / -.024		JSM
11-2-81	129.321	7.539	122.482	-.036 / -.007	17.226	112.795	-.058 / -.036	26.852	103.169	-.059 / -.036	36.421	93.600	-.054 / -.030	45.906	84.115	-.062 / -.031		JSM
11-9-81	129.319	7.543	122.476	-.062 / -.033	17.224	112.795	-.058 / -.036	26.849	103.170	-.058 / -.035	36.422	93.597	-.057 / -.033	45.908	84.111	-.066 / -.035		JSM
11-13-81	129.314	7.539	122.475	-.063 / -.034	17.216	112.798	-.055 / -.033	26.846	103.168	-.060 / -.037	36.422	93.592	.057 / -.033	45.901	84.113	-.064 / -.033		JSM
11-19-81	129.314	7.544	122.470	-.068 / -.039	17.225	112.789	-.064 / -.042	26.849	103.165	-.063 / -.040	36.424	93.590	-.064 / -.040	45.916	84.098	-.071 / -.048		JSM
11-30-81	129.317	7.548	122.469	-.069 / -.040	17.222	112.795	-.058 / -.036	26.855	103.162	-.066 / -.043	36.420	93.597	-.057 / -.033	45.914	84.103	-.074 / -.043		JSM
12-3-81	129.317	7.544	122.473	-.065 / -.036	17.226	112.791	-.062 / -.040	26.847	103.170	-.058 / -.035	36.417	93.600	-.054 / -.030	45.911	84.106	-.071 / -.040		JSM
12-10-81	129.315	7.542	122.473	-.065 / -.036	17.229	112.786	-.067 / -.045	26.840	103.175	-.053 / -.030	36.418	93.597	-.053 / -.029	45.900	84.115	-.060 / -.031		[illegible]

REF ELEV = ELEVATION OF ROAD BOX RIM, IN FEET.
DEPTH = TAPE READING, IN FEET.
ELEV = RING ELEVATION IN FEET = REF ELEV - DEPTH + CF

CF = DEPTH CORRECTION FACTOR = 0.7 FEET.
DIFF = DIFFERENCE FROM INITIAL ELEVATION.

CASING NO. TSC 12

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS

FIGURE 6-9. SETTLEMENT PROBE SUMMARY SHEET (TYPICAL)

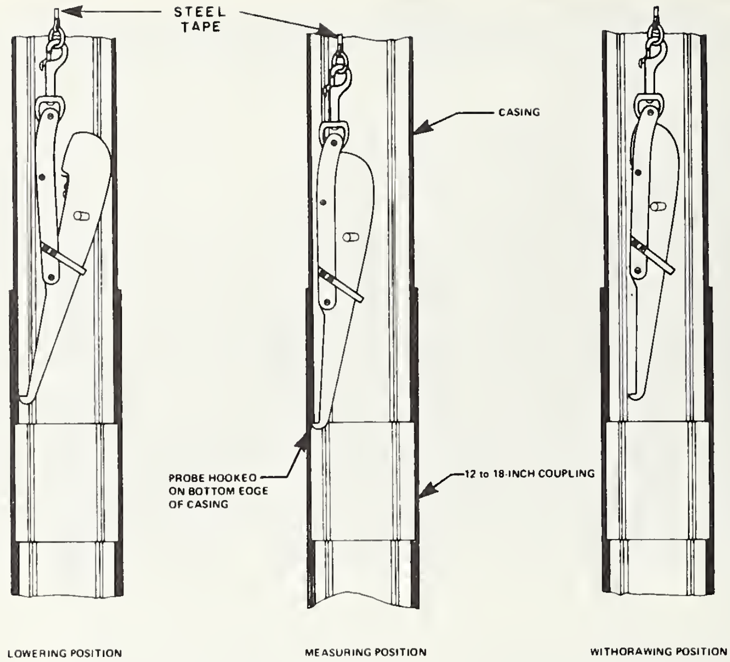


FIGURE 6-10. MECHANICAL PROBE OPERATION (SLOPE INDICATOR COMPANY)

These casings were monitored using the following equipment:

1. a tripod with an adjustable reference leg,
2. a 100-foot steel tape, graduated in hundredths, and
3. the mechanical hook probe.

6.4.2.2 Installation - Procedures used to install the telescoping inclinometer/settlement casings (Figure 6-11) were as follows:

a. The boring and HW (4.0-inch ID) casing were installed as in Section 6.4.1.2, paragraphs a. and b.

b. 20-foot lengths of the PVC casing were assembled on the ground surface with the telescoping coupling. The coupling was sealed with caulk and electrical tape to prevent grout from entering the casing. The bottom casing section contained a quick couple grout valve connected to a 5/8-inch ID grout hose.

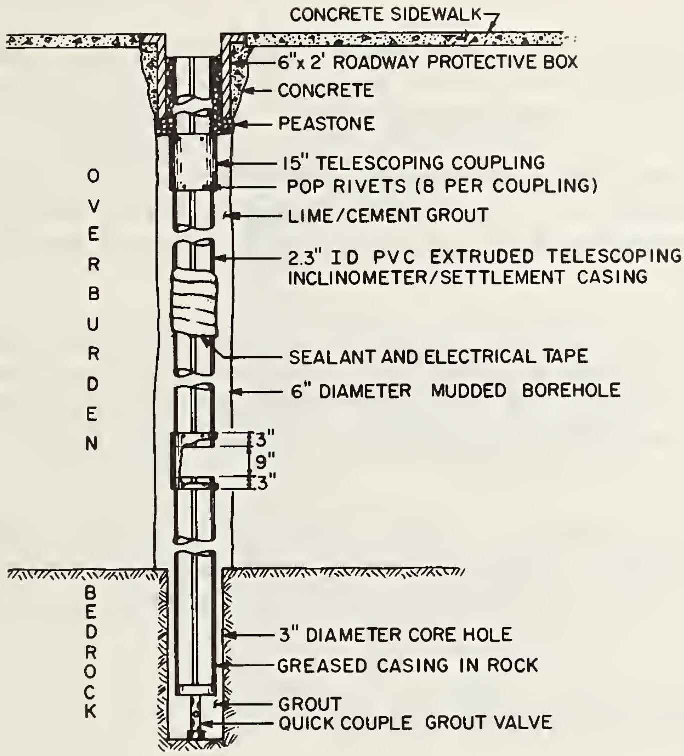


FIGURE 6-11. TELESCOPING INCLINOMETER/SETTLEMENT INSTALLATION

c. A grease layer was applied to casing sections to be installed in the rock, since the rock corehole ID was only 3.0 inches and the casing OD was 2.8 inches. The 20-foot long PVC casing assemblies were then lowered into the borehole. The grout hose was carried through the PVC casing to the ground surface.

d. The casing was then grouted, using a 3 to 1 lime/ cement grout, backfilled and protected with a roadway box as in Section 6.4.1.2, paragraphs f. and g.

6.4.2.3 Monitoring - The procedures for monitoring settlement in the telescoping inclinometer/settlement casing are summarized below:

a. The roadway box cover was removed and the tripod was placed on the box rim's reference mark as in Section 6.4.1.3, paragraph a.

b. The hook probe was then attached to the 100-foot tape, lowered into the casing and hooked to the bottom edge of the casing section being surveyed. The steel tape was tensioned and clamped to the reference leg. The reference leg was checked and the tape was read as in Section 6.4.13, paragraphs d. and e.

c. The clamp was released and the tape and probe lowered to survey the level of the next lowest casing section end. Readings were continued in this manner to the bottom of the casing.

d. After the bottom section of casing had been surveyed, the reference leg was raised off the box rim, so as not to damage it. The steel tape and probe were then removed from the casing and the cap and roadway box cover replaced.

6.4.2.4 Data Processing - Reduction of the settlement data from the telescoping inclinometer/settlement casing was similar to that for the electrical settlement casing, Section 6.4.1.4.

6.4.3 Combination Settlement Casing (TSC 1, 2, 3, 4, 5, and 6)

6.4.3.1 Equipment - This casing is Cresline™ 2.5-inch ID, schedule 40 PVC casing in 10-foot sections, attached by 12-inch flexible corrugated PVC couplings, each containing a stainless steel wire ring. Settlement in this casing may be measured either by the electrical or mechanical hook probe systems previously discussed, Sections 6.4.1 and 6.4.2 respectively.

6.4.3.2 Installation - Procedures used to install the combination settlement casings (Figure 6-12) were as follows:

a. A 5.0-inch ID overburden borehole and 3.0-inch ID rock corehole were advanced to the depth required for exploration. Depending on soil conditions, 10 to 15 feet of PW (5.0-inch ID) casing was placed to stabilize the upper portion of the boring.

b. HW (4.0-inch ID) casing was then lowered to the top of bedrock to sleeve the mudded borehole to make settlement casing installation easier

c. The PVC casing was assembled on the ground surface in 20-foot lengths with 12-inch flexible corrugated PVC couplings at 5 to 10-foot intervals as desired. A stainless steel sensor ring was installed at each coupling midpoint, coated with an asphalt mastic, and sealed with caulk and electrical tape to prevent grout from entering the casing.

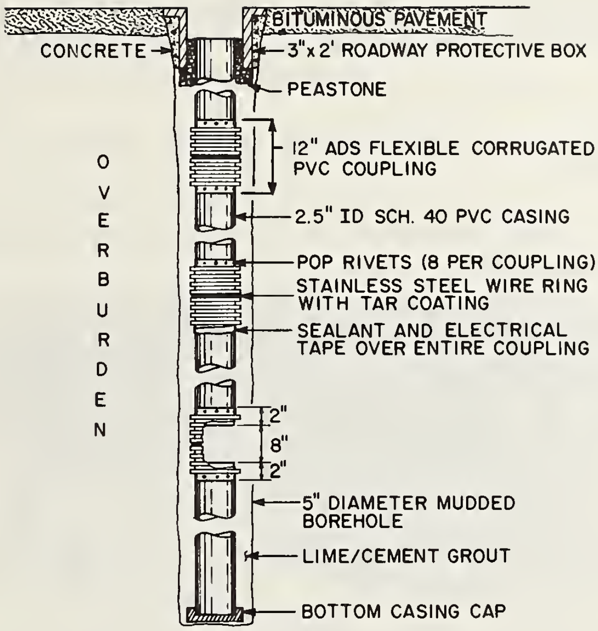


FIGURE 6-12. COMBINATION SETTLEMENT INSTALLATION

d. The 20-foot long assemblies were then lowered into the borehole. During installation, a 5/8-inch ID grout hose with its end at the casing bottom, was carried to the ground surface, taped to the outside of the PVC casing.

e. The settlement casing was then grouted, using a 3 to 1 lime/cement grout, backfilled, and protected with a roadway box as in Section 6.4.1.2, paragraphs f. and g.

6.4.3.3 Monitoring - The procedures for monitoring of the combination settlement casing were the same as described in Section 6.4.1.3 for electrical measurements, and Section 6.4.2.3 for hook probe measurements.

6.4.3.4 Data Processing - Reduction of the data obtained from this casing was similar to that for the electrical settlement casing, Section 6.4.1.4.

6 4.4 Deep Settlement Points by Others

Six "Borros type" settlement points were installed and monitored by the construction contractor as partial requirement of the contract. These instruments were at depths of approximately 45 to 80 feet and are identified as SSP 25 through 30 on Figure 3-6. These instruments were read periodically by the contractor and the data then provided to the MBTA.

6.5 INCLINOMETERS

Two types of inclinometer casings were installed in the Test Section as follows:

a. ABS inclinometer casing, and

b. telescoping inclinometer/settlement casing.

Each system is described in detail in the following sections.

6.5.1 ABS Inclinometer Casing (TSC 7, 8, 9, 10, 11, and 12)

6.5.1.1 Equipment - This casing is 2.3-inch ID flush coupled ABS inclinometer casing, Model No. 51111, manufactured by Slope Indicator Company, Seattle, Washington. The casing is supplied in 10-foot lengths with field connections made with a 2.55-inch ID coupling. The casing's grooves are machined inside of the casing and the outside of the casing is circular (Figure 6-13).

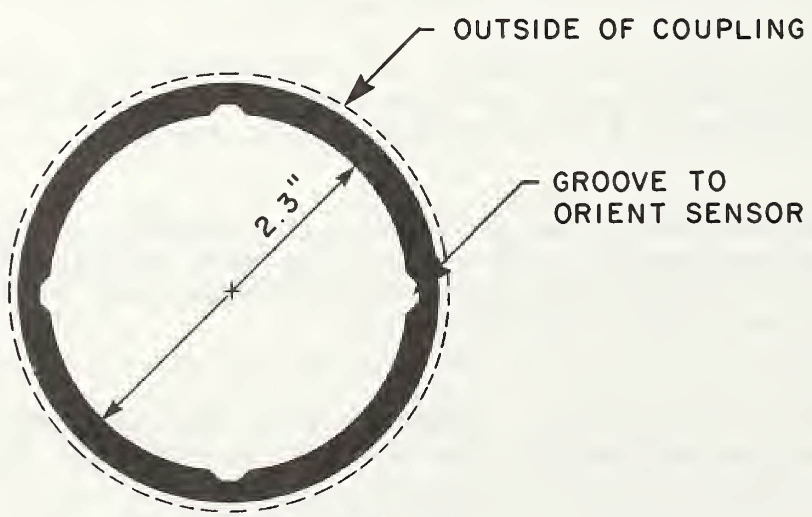


FIGURE 6-13. ABS INCLINOMETER CASING CROSS SECTION

These casings were monitored using the following equipment:

1. Digitilt™ inclinometer sensor,
2. inclinometer cable, marked in feet,
3. magnetic tape recorder/control unit (Figure 6-14), and
4. casing extension and pulley assembly.

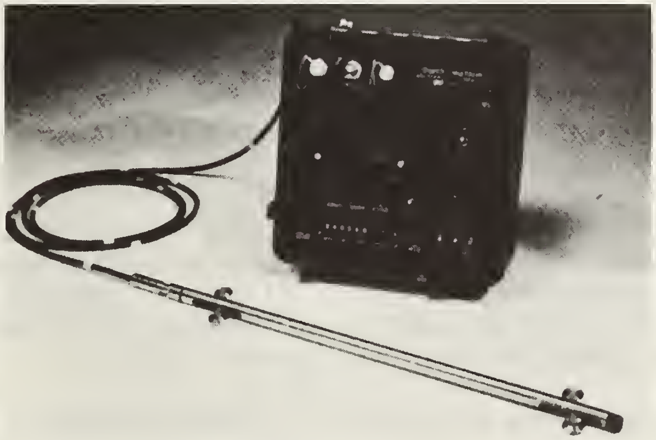


FIGURE 6-14. DIGITILT™ INCLINOMETER, CABLE AND MAGNETIC TAPE RECORDER/CONTROL UNIT (SLOPE INDICATOR COMPANY)

Inclinometer casings have four longitudinal grooves at their quarter points to guide the wheels of the 33-inch inclinometer sensor and control its orientation. The inclinometer sensor used in this study was the Digitilt™ inclinometer, Model No. 50325, manufactured by the Slope Indicator Company, Seattle, Washington. The inclinometer measured the inclination of the casing as it was raised from the bottom to top of the casing. A cable marked in feet (Model No. 50610, Slope Indicator Company) connected to the sensor indicated the depth of each reading. These inclination vs. depth readings were recorded on a magnetic tape cassette by a recorder control unit (Model No. 50308, Slope Indicator Company).

6.5.1.2 Installation - The ABS inclinometer casings were installed in combination with the electrical settlement casings, Section 6.4.1.2.

As standard procedure, the casing's grooves were installed parallel and perpendicular to the tunnel centerlines. The grooves were then designated A+, A-, B+, and B- as shown on Figure 6-15.

6.5.1.3 Monitoring - The monitoring procedures for the ABS inclinometer casing are summarized below:

a. The roadway box cover was removed and the casing extension and pulley assembly was placed on the inclinometer casing.

b. With the inclinometer sensor, cable and readout unit connected, the sensor was first lowered down the A+ and A- grooves with the upper wheels in the A+ groove. This gives inclinations in the A+ and B+ directions.

c. The sensor was lowered to the casing's datum point, referenced as 998 on the control unit's depth counter, near the casing bottom, and the cable was fixed in the extension cable clamp. Once the readout had stabilized, the reading was recorded on the recorder unit (Figure 6-16).

d. The cable was pulled up 2 feet, clamped on the extension assembly, and another reading taken. Reading were taken in this manner to the top of the casing. The control unit's counter automatically changed the depth component at 2-foot intervals to coordinate recorded readings with depth, and advanced the tape to allow space between sequential readings.

e. After the last reading (sensor near the top of the casing), the inclinometer sensor was removed from the casing, rotated 180 degrees, and replaced with the upper wheels in the A- groove. The instrument then monitored inclination in the A- and B- directions.

f. After the casing had been completely surveyed, the sensor and extension assembly were removed from the casing and the casing cap and roadway box cover replaced.

6.5.1.4 Data Processing - The field inclinometer data (inclinometer readings vs. depth) were automatically taken and stored on a magnetic tape cassette. The total lateral profile of the casing was found by integrating the recorded inclinations from the bottom of the casing to the top. For these calculations, the bottom of the casing was assummed fixed. Calculations, including corrections for casing spiral, Section 10.5, were made by means of a PDP-11 computer, manufactured by Digital Equipment Corporation, Maynard, Massachusetts. A reference file for each instrument was compiled from initial readings before

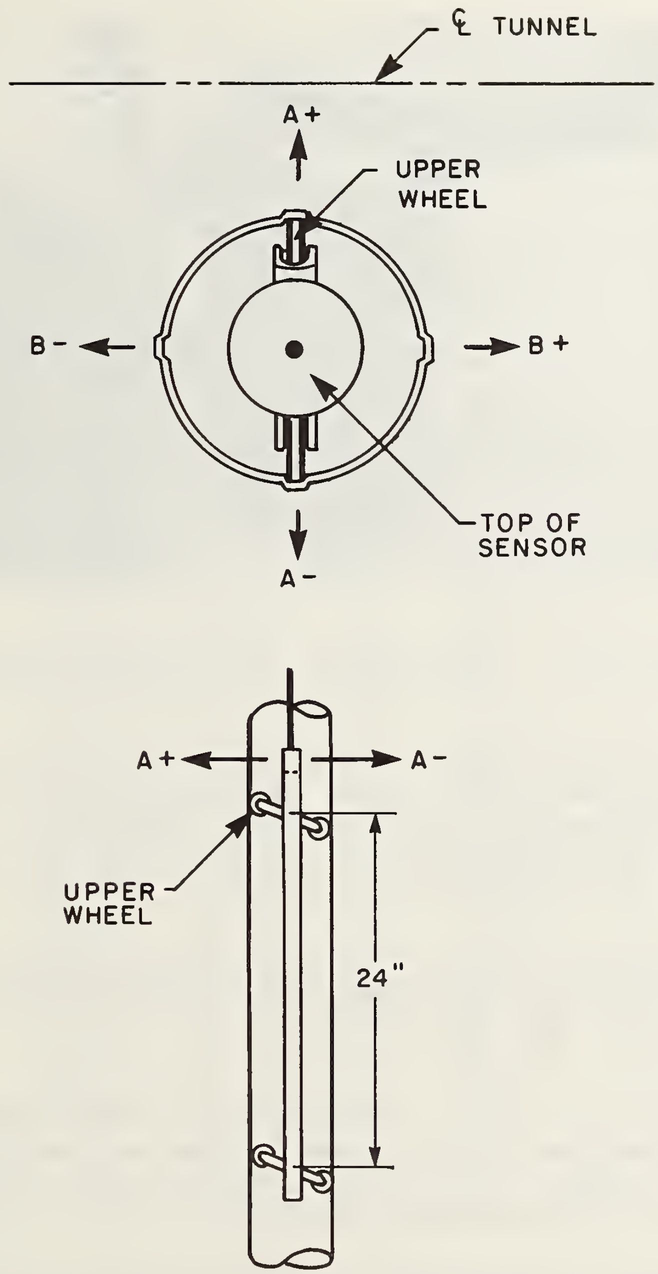


FIGURE 6-15. INCLINOMETER CASING ORIENTATION

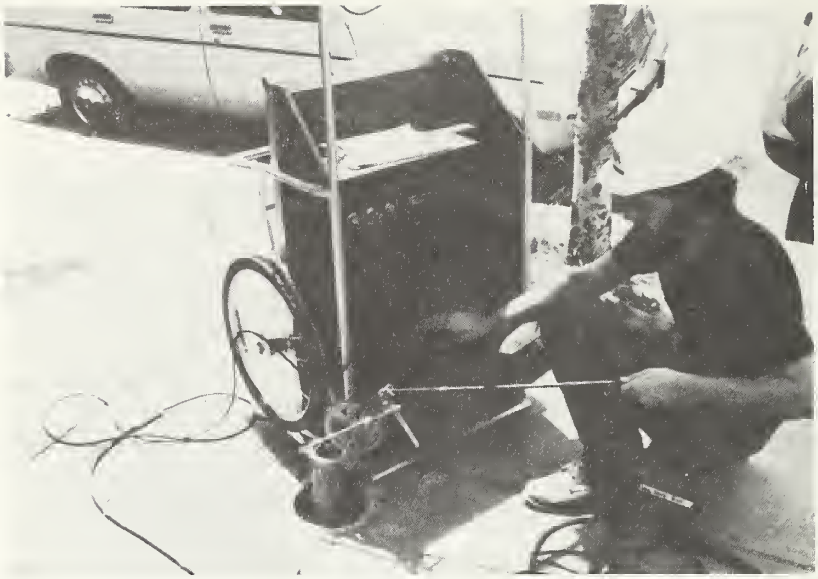


FIGURE 6-16. MONITORING INCLINOMETER CASING

tunnel construction began in the area of the Test Section. Lateral displacements were then determined, for each inclinometer survey, from the comparison with this reference file. Figure 6-17 shows typical numerical output, including the appropriate headings, for an inclinometer survey.

6.5.2 Telescoping Inclinometer/Settlement Casing (TSC 13, 14, and 15)

6.5.2.1 Equipment - This casing is the same casing as described in Section 6.4.2. The casing is connected with a telescoping coupling that maintains the orientation of the groove of the casing, (Figure 6-18).

This type of casing was monitored using the same equipment that was used for the ABS inclinometer casing, Section 6.5.1.1.

6.5.2.2 Installation - The installation of these casings is described in Section 6.4.2.2. As standard procedure, the casing's grooves were installed parallel and perpendicular to the tunnel centerlines. The grooves were then designated A+, A-, B+, and B- as shown on Figure 6-15.

HALEY & ALDRICH, INC.
CAMBRIDGE, MASSACHUSETTS

PROJECT: TSC/MBTA TUNNEL TEST SECTION
LOCATION: CAMBRIDGE, MASSACHUSETTS

INCLINOMETER NO.: 12

A AXIS ORIENTATION: PERPENDICULAR TO TUNNEL AXIS
ACTUAL A1 AZIMUTH: 270
DESIRED A1 AZIMUTH: 270
A1 AZIMUTH USED FOR COMPUTATION: 270
DATE OF REFERENCE READINGS: 05 19 80

FILE NO: 08 428401

READING DATE: 10 27 81
READING TIME: 12:00
READ BY: MOONEY

INCLINOMETER DATUM ELEVATION: 37.20

SURVEY DATUM BASE: MBTA RED LINE

PAGE 1 OF 2

COMPUTATION DATE: 07 09 82
COMPUTATION TIME: 13:16

TORPEDO SERIAL NO.: 25143
READOUT SERIAL NO.: 459

		------ A AXIS DATA --------					-------- B AXIS DATA -------				
ELEVATION (FEET)	DIST. ABOVE DATUM (FEET)	A1 READING	A2 READING	DIST. FROM VERT. (IN.)	INIT. DIST. (IN.)	DEFORM. A (IN.)	B1 READING	B2 READING	DIST. FROM VERT. (IN.)	INIT. DIST. (IN.)	DEFORM. B (IN.)
129.20	92.00	0.0945	-0.0934	22.327	22.801	-0.473	0.0094	-0.0143	-12.161	-12.015	-0.146
127.20	90.00	0.0872	-0.0837	21.200	21.752	-0.552	0.0061	-0.0132	-12.303	-12.221	-0.082
125.20	88.00	0.0902	-0.0874	20.174	20.720	-0.545	0.0072	-0.0158	-12.419	-12.380	-0.039
123.20	86.00	0.0833	-0.0824	19.109	19.659	-0.551	0.0040	-0.0110	-12.557	-12.539	-0.017
121.20	84.00	0.0767	-0.0751	18.115	18.681	-0.566	-0.0019	-0.0046	-12.647	-12.643	-0.004
119.20	82.00	0.0665	-0.0639	17.204	17.789	-0.585	-0.0096	0.0023	-12.663	-12.663	0.000
117.20	80.00	0.0622	-0.0599	16.421	17.014	-0.593	-0.0081	0.0017	-12.592	-12.593	0.001
115.20	78.00	0.0593	-0.0575	15.689	16.282	-0.593	-0.0121	0.0063	-12.533	-12.542	0.009
113.20	76.00	0.0522	-0.0476	14.988	15.589	-0.601	-0.0188	0.0134	-12.422	-12.435	0.013
111.20	74.00	0.0429	-0.0391	14.389	15.000	-0.611	-0.0253	0.0206	-12.229	-12.245	0.016
109.20	72.00	0.0276	-0.0261	13.897	14.520	-0.623	-0.0319	0.0265	-11.954	-11.968	0.014
107.20	70.00	0.0127	-0.0124	13.575	14.182	-0.607	-0.0373	0.0307	-11.603	-11.617	0.014
105.20	68.00	0.0066	-0.0053	13.424	14.033	-0.609	-0.0418	0.0356	-11.195	-11.198	0.002
103.20	66.00	0.0024	-0.0007	13.353	13.959	-0.606	-0.0390	0.0330	-10.731	-10.735	0.004
101.20	64.00	-0.0141	0.0142	13.334	13.948	-0.614	-0.0437	0.0372	-10.299	-10.303	0.004
99.20	62.00	-0.0096	0.0127	13.504	14.119	-0.615	-0.0472	0.0390	-9.814	-9.819	0.005
97.20	60.00	0.0092	-0.0073	13.638	14.241	-0.603	-0.0406	0.0329	-9.296	-9.294	-0.002
95.20	58.00	0.0293	-0.0278	13.539	14.126	-0.587	-0.0306	0.0239	-8.855	-8.848	-0.007
93.20	56.00	0.0463	-0.0460	13.196	13.773	-0.577	-0.0232	0.0174	-8.528	-8.517	-0.012
91.20	54.00	0.0505	-0.0483	12.643	13.208	-0.565	-0.0211	0.0133	-8.285	-8.274	-0.011
89.20	52.00	0.0415	-0.0423	12.050	12.596	-0.546	-0.0177	0.0106	-8.078	-8.068	-0.011
87.20	50.00	0.0339	-0.0351	11.547	12.083	-0.536	-0.0172	0.0132	-7.909	-7.893	-0.016
85.20	48.00	0.0277	-0.0258	11.133	11.657	-0.524	-0.0213	0.0148	-7.726	-7.683	-0.043
83.20	46.00	0.0324	-0.0308	10.812	11.335	-0.523	-0.0237	0.0152	-7.510	-7.461	-0.048
81.20	44.00	0.0329	-0.0318	10.433	10.948	-0.515	-0.0200	0.0147	-7.276	-7.246	-0.030
79.20	42.00	0.0395	-0.0388	10.045	10.521	-0.476	-0.0167	0.0110	-7.068	-7.044	-0.024
77.20	40.00	0.0366	-0.0361	9.575	10.013	-0.439	-0.0176	0.0116	-6.902	-6.863	-0.039
75.20	38.00	0.0353	-0.0340	9.139	9.560	-0.422	-0.0339	0.0332	-6.727	-6.674	-0.053
73.20	36.00	0.0348	-0.0325	8.723	9.127	-0.404	-0.0336	0.0278	-6.324	-6.293	-0.031
71.20	34.00	0.0384	-0.0369	8.319	8.698	-0.379	-0.0230	0.0165	-5.956	-5.954	-0.001
69.20	32.00	0.0341	-0.0376	7.867	8.255	-0.388	-0.0211	0.0123	-5.719	-5.707	-0.011
67.20	30.00	0.0306	-0.0316	7.437	7.832	-0.395	-0.0241	0.0164	-5.518	-5.504	-0.014
65.20	28.00	0.0138	-0.0148	7.064	7.442	-0.378	-0.0300	0.0254	-5.275	-5.269	-0.006
63.20	26.00	0.0177	-0.0163	6.892	7.070	-0.178	-0.0334	0.0276	-4.943	-4.964	0.021
61.20	24.00	0.0299	-0.0323	6.688	6.702	-0.014	-0.0326	0.0297	-4.577	-4.634	0.057
59.20	22.00	0.0258	-0.0299	6.315	6.343	-0.028	-0.0488	0.0375	-4.203	-4.237	0.034
57.20	20.00	0.0401	-0.0355	5.981	5.967	0.014	-0.0477	0.0411	-3.685	-3.781	0.096
55.20	18.00	0.0428	-0.0432	5.527	5.495	0.032	-0.0466	0.0417	-3.152	-3.254	0.102
53.20	16.00	0.0487	-0.0439	5.011	4.950	0.061	-0.0398	0.0324	-2.623	-2.723	0.101
51.20	14.00	0.0474	-0.0489	4.456	4.371	0.085	-0.0346	0.0282	-2.189	-2.282	0.093
49.20	12.00	0.0423	-0.0420	3.878	3.842	0.036	-0.0303	0.0244	-1.825	-1.914	0.089
47.20	10.00	0.0264	-0.0296	3.373	3.368	0.004	-0.0380	0.0328	-1.496	-1.573	0.077
45.20	8.00	0.0391	-0.0387	3.037	2.894	0.142	-0.0271	0.0216	-1.072	-1.177	0.106
43.20	6.00	0.0654	-0.0631	2.570	2.383	0.187	-0.0289	0.0206	-0.779	-0.869	0.090
41.20	4.00	0.0833	-0.0816	1.799	1.635	0.164	-0.0224	0.0158	-0.482	-0.581	0.099
39.20	2.00	0.0670	-0.0879	0.809	0.793	0.016	-0.0206	0.0216	-0.253	-0.323	0.070

FIGURE 6-17. INCLINOMETER DATA SHEET (TYPICAL)

6.5.2.3 Monitoring - Monitoring procedures for the telescoping inclinometer/ settlement casing was the same as that for the ABS inclinometer casing, Section 6.5.1.3.

6.5.2.4 Data Processing - Data processing for these instruments was the same as that for the ABS inclinometer casing, Section 6.5.1.4.

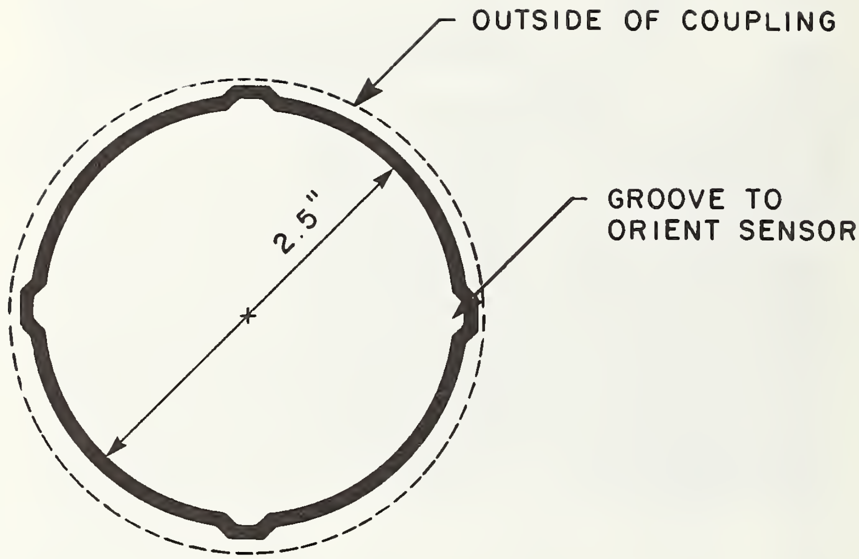


FIGURE 6-18. TELESCOPING INCLINOMETER/SETTLEMENT CASING CROSS SECTION

6.5.3 Inclinometers By Others

Three inclinometers were installed and monitored by the MBTA. These instruments are located on the west side of Massachusetts Avenue, and are identified as I-9, I-10, and I-11 on Figure 3-6. Data from these instruments were not available for use in this study.

6.6 PIEZOMETERS (TSC 16 and 18)

These instruments are flow or wellpoint type, open standpipe piezometers and were installed in bedrock. They consist of a piezometer point, a 1-3/8 inch OD x 12-inch long porous tube, connected to a 3/4-inch, Schedule 80 rigid PVC riser pipe (Figure 6-19).

Each point was placed in a quartz sand filter material (Ottawa™ sand) which extended 3 to 5 feet above the point. A 2 to 3-foot thick impervious seal of bentonite pellets was placed and compacted above the sand filter material. The remaining portions of the bedrock and overburden were then sealed with a lean cement grout to the ground surface or to the bottom section of the observation well. The instruments were secured at the ground surface with a protective roadway box.

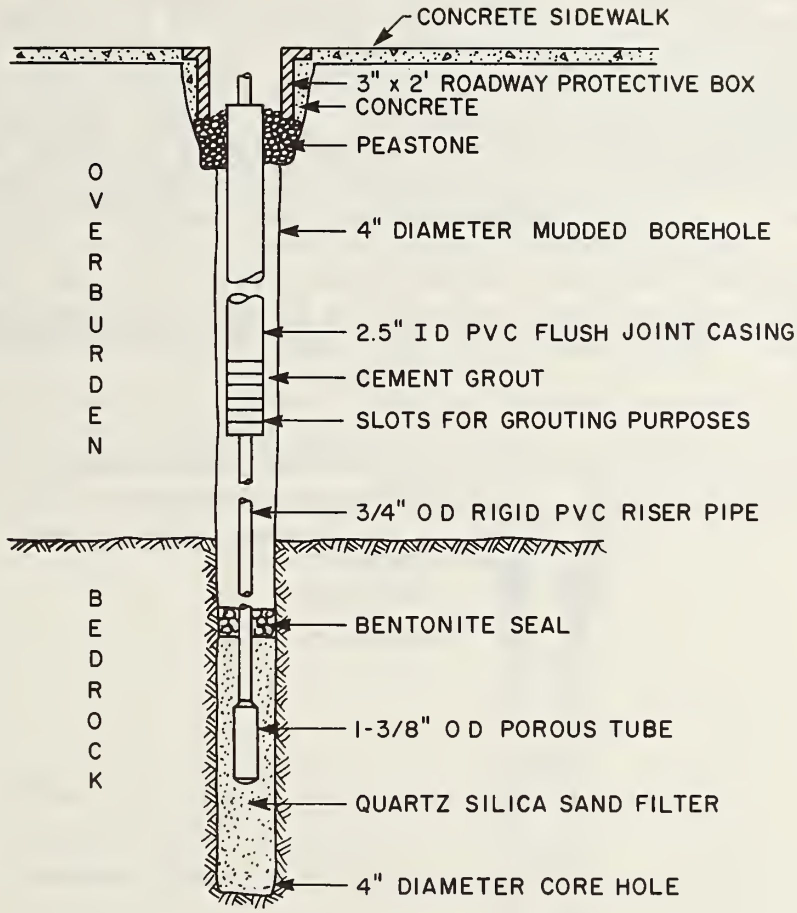


FIGURE 6-19. PIEZOMETER INSTALLATION

The piezometers were read by means of a graduated 100-foot long, 1/2-inch OD PVC tubing with two insulated wires inside. The bottom ends of the wires had both ends stripped of insulation. The top ends of the wires were connected to a control box, which had a battery power source, meter, and switch.

A complete circuit was made when the end of the tubing was at or below the water level in the riser pipe. The meter registered this and the depth to the water was measured on the tubing's graduations. Optical level surveys provided the elevation of the piezometer roadway box rim. Groundwater elevation data were continuously tabulated and changes were determined from comparisons with readings made before tunnel construction (Table 4-2).

6.7 OBSERVATION WELLS (TSC 16, 17, 19, 21, 22, and 24)

To insure borehole stability during geophysical surveying, each borehole was stabilized with Cresline™ 2.5-inch, Schedule 40, flush coupled PVC casing. In order to utilize existing plastic casing and to minimize project costs, 10-foot sections of preslotted Hydrophilic™ 2.5-inch, Schedule 40, PVC casing with forty-six 0.015-inch wide slots per foot, were selectively installed in the casing string for groundwater observations. At the completion of the borehole geophysical survey work, the boreholes were grouted to the base of the slotted wellpoint section, backfilled with a filter sand, and secured at the ground surface with a protective roadway box (Figure 6-20). Wells TSC 17, 19, and 21 eventually became obstructed.

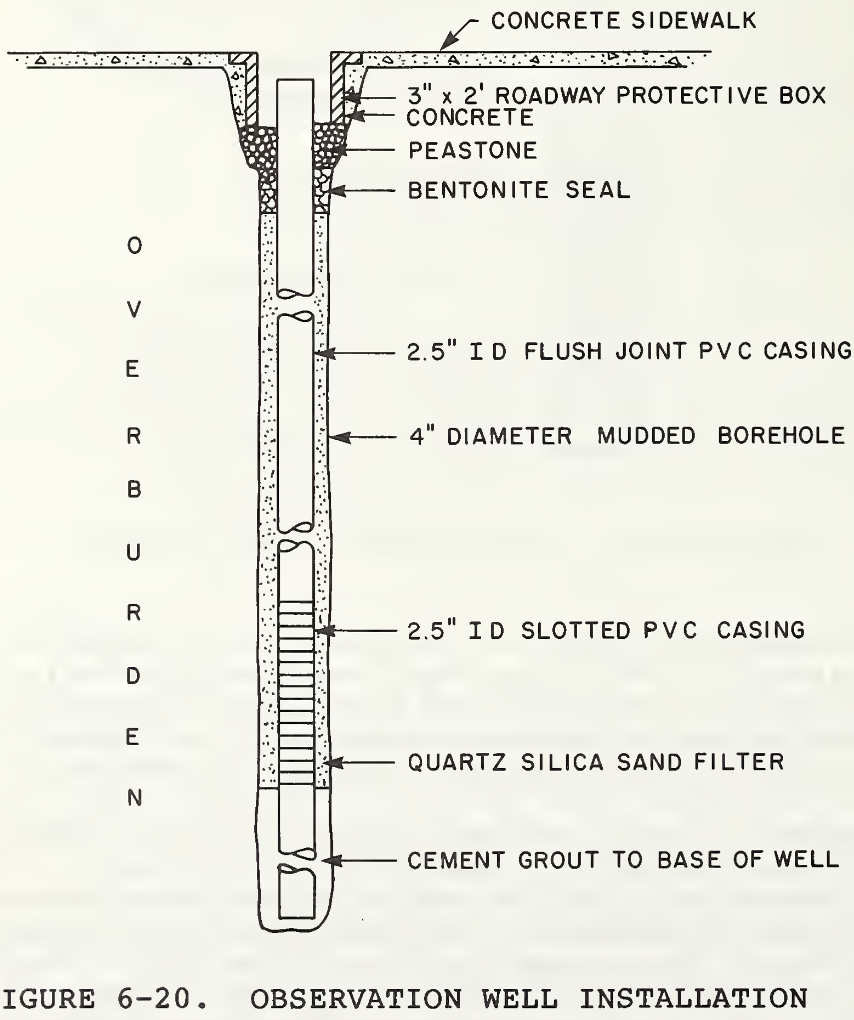


FIGURE 6-20. OBSERVATION WELL INSTALLATION

A short piece of pipe, closed on one end and open on the other, was attached to a 100-foot cloth tape and lowered into the casing to monitor the groundwater level. The pipe or plunker would pop upon encountering the water in the casing and the depth below ground surface registered. Optical level surveys provided the elevation of the observation well roadway box rim. Observation well data were continuously tabulated and changes were determined from comparisons with readings made before construction (Table 4-2).

6.8 PIEZOMETERS AND OBSERVATION WELLS BY OTHERS

Groundwater data from other instruments within the Test Section were supplied by the MBTA. These data were used to supplement data obtained from Test Section instrumentation, Section 8.

7. CONSTRUCTION PROCEDURES AND PROGRESS

7.1 GENERAL

Construction procedures are discussed for each of the tunnel environments encountered within the Test Section. These include rock, soft ground, and mixed face excavation. The support type for each tunneling condition is also discussed.

7.2 ROCK EXCAVATION AND SUPPORT PROCEDURES

Rock excavation in the Test Section was accomplished by full face and pilot tunnel methods. The excavation and support by these methods are discussed separately in the following sections. Specific tunneling terms are defined in the Glossary of Tunneling and Geologic Terms, Appendix J.

7.2.1 Full Face Rock Excavation and Support

The full face rock portion of the tunnel through the Test Section extended approximately from Station 204+80 IB to Station 206+31 IB; and Station 204+84 OB to Station 206+00 OB (Figure 3-5). In both tunnel headings, the face was advanced conventionally using drill-and-blast techniques. The length of the rounds within the Test Section was shorter (2 to 5 feet) than usual (10 to 12 feet) because of the diminished thickness of rock cover above the tunnel crown. Steel ribs (W8 x 30) were erected on 2-foot centers. Crown bars (spiles), consisting of 1-3/8 inch (No. 11) reinforcing bars, were used to provide immediate roof support in the tunnel. This was accomplished by placing the bars in the crown above the springline and advancing them 25 to 30 feet ahead of the working face. The crown bars provided roof support by cantilever action from the last rib (Figure 7-1). Generally, ribs were installed when the working face was four feet ahead of the last rib. Rock forces were transferred to the steel ribs by installing blocks (usually of wood) between the rock and the outside of the ribs. This blocking procedure was routinely accomplished in the tunnel crown between the quarter points (those points which are midway between the springline and the crown apex), and extended to the springlines or lower as necessary.

Full face rock excavation was stopped when till was encountered in the crown (Station 204+80 IB; Station 204+84 OB). At that location, a timber and steel bulkhead was constructed to stabilize the face in each heading (Figure 7-2). From this bulkhead locaton, a 10 by 10-foot invert drift was driven through the remaining rock.

FIGURE 7-1. CROWN SPILING IN FULL FACE ROCK TUNNEL, APPROXIMATELY STATION 205+57 OB

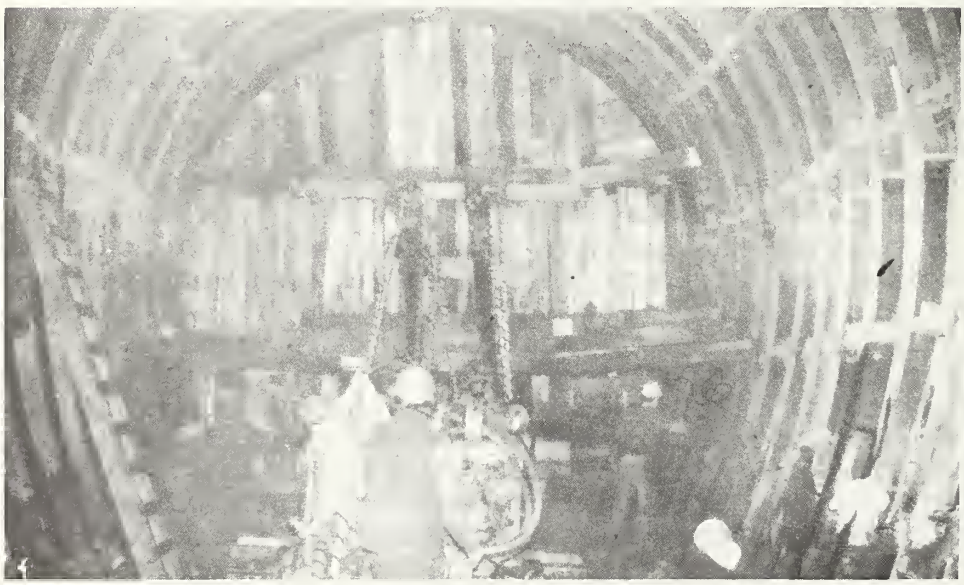


FIGURE 7-2. BULKHEAD CONSTRUCTED AT END OF FULL FACE ROCK TUNNEL PRIOR TO COMMENCEMENT OF INVERT DRIFT

7.2.2 Pilot Tunnel Excavation and Support

The pilot tunnel (invert drift) was driven southward from the end of full face rock excavation to a point where the till was exposed in the crown. The invert drift was a 10 by 10-foot opening which extended from Station 204+80 IB to Station 203+72 IB, and Station 204+84 OB to Station 203+20 OB.

The pilot tunnel was advanced by the drill-and-blast method. The average length of rounds was 4 to 6 feet. Crown bars (spiles) were installed in the pilot tunnel to provide increased stability. The bars were installed on 1-foot centers in the tunnel roof and the upper two feet of the side walls. They were advanced 12 to 15 feet ahead of the working face.

The tunnel opening was supported by 8 by 8-inch hardwood cap and post beams set on 4-foot centers. To stabilize the side walls, the posts were inclined slightly inward from invert to crown.

To maintain face stability, a temporary timber bulkhead was constructed after completing the pilot tunnel. The outbound pilot tunnel was completed November 11, 1980. The inbound pilot tunnel was completed on January 7, 1981.

7.3 SOFT GROUND AND MIXED FACE EXCAVATION AND SUPPORT PROCEDURES

The soft ground and mixed face portions of the Test Section were excavated using a shield (23 feet, 7-3/4 inches OD) manufactured to job specifications by Elgood-Mayo Corporation (Figure 7-3). The shield employed a hydraulic excavator arm mounted centrally in the nose, with twenty four 150-ton shove jacks, each with a 60-inch stroke. The shield design included a hydraulic ring expander, located in the tail section. The cutting edge of the shield was designed with 3/8-inch overcutters on the top 150 degrees of circumference.

The majority of the shield excavations in the Test Section were in mixed face, with rock encountered in the invert at Station 203+06 IB and Station 203+07 OB. Therefore, mixed face conditions existed until the excavations holed through, Station 203+06 IB to Station 204+80 IB, and Station 203+07 OB to Station 204+84 OB.

Various modifications had been made to the shield during earlier construction. A set of five hydraulically activated breast doors were installed above the springline across the hood of the shield. The doors were wedge-shaped and hinged at the point of attachment with the shield (Figure 7-4). Steel teeth (5 inches long) were welded to the cutting edge of the shield circumferentially on approximately 1-foot centers.

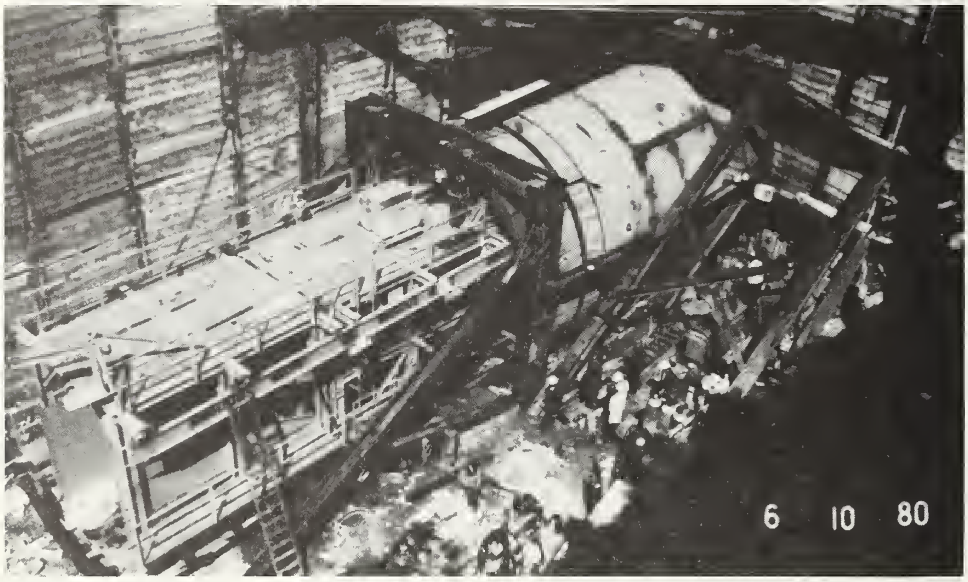


FIGURE 7-3. SHIELD ASSEMBLY AT INBOUND TUNNEL PORTAL (FLAGSTAFF PARK)

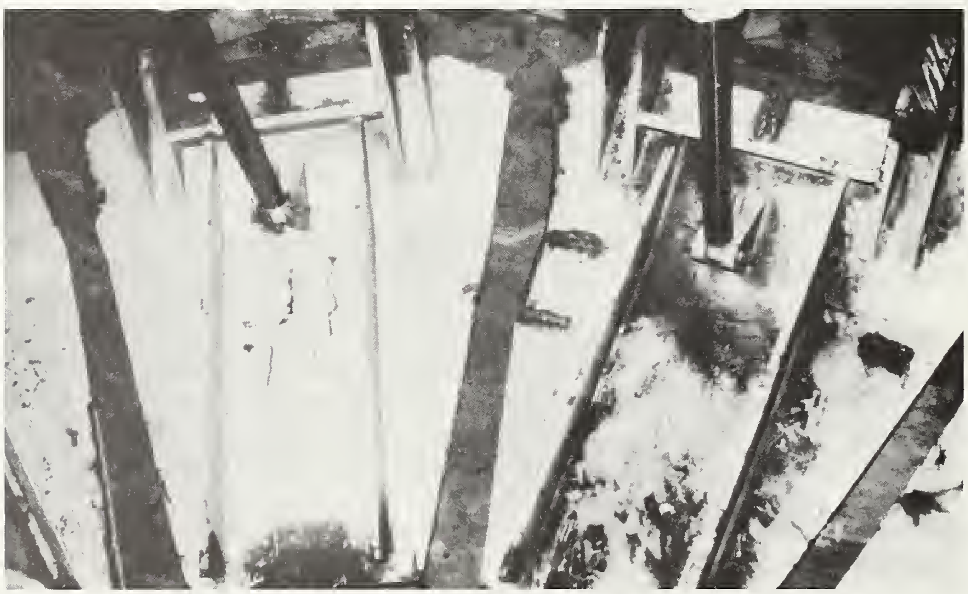


FIGURE 7-4. BREAST DOORS CLOSED AGAINST FACE

The shield was advanced by jacking against the ribs and lagging. The shield was advanced a maximum of four feet per shove. The face was excavated either by hand or with the aid of the excavator arm, depending on the difficulty and the amount of rock at the face. The center of the face was excavated first. Following this, material was excavated from approximately springline down to the invert. This procedure relieved the pressure on the cutting edge of the shield. As the shield advanced, the hood trimmed the remaining soil, which then collapsed into the invert. Laborers or an excavation arm pushed the muck from the face onto a conveyor belt, and the belt deposited it into a large muck bin behind the trailing gear of the shield. A Wagner™ loader emptied the muck bin and transported the muck to the portal where it was lifted to street level and loaded into trucks for disposal.

During the excavating procedure, four-piece steel ribs (W8 x 35) and lagging were assembled in the tail section. When the steel rib was clear of the tail section, it was expanded against the tunnel wall. Steel sections (Dutchmen) were inserted between the rib sections and welded into place. The Dutchmen aided in maintaining the ribs tight against the tunnel wall (Figure 7-5).

FIGURE 7-5. DUTCHMAN WELDED INTO PLACE AFTER RIB EXPANSION

The breast doors were maintained in a lowered position where required by lack of face stability. The excavator arm was then used to secure timbers across the face to increase stability. Timbers were also shoved ahead of the working face (forepoling) to aid in maintaining stability during excavation.

Because of soft rock quality within the Test Section, the hydraulic excavator arm was initially used to excavate the rock. As rock quality and quantity increased, the rate of advance was slowed. Explosives were used to fracture the rock when the excavator arm could not efficiently excavate the rock in the face. Excavation of the tunnel at the location of the invert pilot drift was eased because of the prior rock disturbance. Drilling and blasting of the rock was required in the inbound tunnel when the rock elevation approached the springline. However, in the outbound tunnel, continuous drilling and blasting were required from the time rock was encountered to the invert.

The inbound tunnel was excavated through the Test Section approximately seven weeks ahead of the outbound tunnel. The shield was dismantled after completing the tunnel drive. The shield's outer skin was left in place and used as part of the tunnel support.

7.4 DEWATERING PROGRAM

Construction dewatering in the area of the Test Section was accomplished initially using three deep wells (DW-3, DW-4, and DW-5). The wells were spaced over 900 feet of tunnel and pumped small quantities of water. Wells DW-3 and DW-4 pumped less than 10 gpm. Well DW-5, located in the mixed face area, pumped 30 to 35 gpm from early 1979 until September 1980, when the outbound rock tunnel was approaching Station 204+84. After September 1980, the output from well DW-5 dropped to 3 to 5 gpm. During November 1980, the well discharge dropped to zero. The outbound invert pilot drift was completed by November 1980 and probably contributed to dewatering the area. Piezometers and observation wells indicated that the groundwater level was approximately at the crown elevation of the tunnel (about elevation 50) by November 1980.

Due to problems encountered during excavation of the inbound soft ground tunnel (Station 186+86) about 1700 feet from the Test Section, a system of closely spaced ejector well points was installed on either side of the tunnel alignment. This system extended from the tunnel portals at Station 184+94, through the mixed face section to approximately Station 205+00. The system consisted of 3-inch diameter wells, 20 feet apart, set in holes drilled into bedrock. The Test Section portion of the system was installed and in operation by July 1981. Typical pumping rates were in the range of 0.3 to 0.6 gpm per ejector well.

7.5 PROGRESS OF EXCAVATION

Because the tunnel excavations from the north and south met within the Test Section, the tunnel excavation through the Test Section took place intermittently over a period of about 18 months. Various tunneling and temporary support methods were employed to drive the tunnel headings through this area. An excavation summary, including heading sequence, tunneling methods, advance rate, and support type is presented in Table 7-1. Figure 7-6 illustrates the sequence of tunneling.

The tunnel progress through the Test Section varied with the geologic conditions and method of excavation.

The rate of advance in the rock section was averaged 2.5 linear feet per day. This low production rate was a result of the conservative tunneling method which was employed because of the minimal rock cover through this section of tunnel (less than 1/2 tunnel diameter). The 2-foot spacing of the steel rib support and the crown (spile) bars installed while tunneling through this area contributed significantly to the amount of time needed to complete these headings. This section of the tunnel was not a critical schedule item.

The appearance of rock in the invert of both tunnels coincided closely with the southern limit of the Test Section. Outside the Test Section and prior to the appearance of rock in the invert, the shields were averaging about 18 linear feet per day.

The rate of advance in the mixed face was considerably less than that achieved in soft ground. Production was reduced to 7.1 linear feet per day in the inbound heading, and 4.2 linear feet per day in the outbound heading.

7.6 CONSTRUCTION PROBLEMS AND DELAYS

Figure 7-7 summarizes the progress of the shield driven tunnels through the Test Section. Pertinent construction notes are added to explain progress results.

7.6.1 North Section (Rock)

Many factors, both man-induced and natural, effect the construction progress in excavating large diameter tunnels in rock. These factors include the blasting design, stand up time of the unsupported tunnel, overall rock quality, presence and amount of groundwater, and type and spacing of temporary support elements. After installation of temporary support, the

TABLE 7-1. EXCAVATION SUMMARY

TUNNELING PHASE*	TUNNEL HEADING AND DIRECTION	EXCAVATION METHOD AND CONDITION	LENGTH (LINEAR FEET)	DATES EXCAVATED	ADVANCE RATE**	SUPPORT
I	OUTBOUND - SOUTH STA. 206+00 - STA. 204+84	FULL FACE: ROCK	114	JULY 1980- OCT. 1980	1.9 L.F./DAY ***	STEEL RIBS (W8 X 30) 2-FOOT CENTERS
II	INBOUND - SOUTH STA. 206+31 - STA. 204+80	FULL FACE: ROCK	150	OCT. 1980- DEC. 1980	2.7 L.F./DAY ***	STEEL RIBS (W8 X 30) 2-FOOT CENTERS
III	OUTBOUND - SOUTH STA. 204+84 - STA. 203+20	INVERT PILOT DRIFT 10' X 10': ROCK	166	OCT. 1980- NOV. 1980	5.0 L.F./DAY	8" TIMBER CAP AND POST 4-FOOT CENTERS
IV	INBOUND - SOUTH STA. 204+80 - STA. 203+72	INVERT PILOT DRIFT 10' X 10': ROCK	104	DEC. 1980- JAN. 1981	5.1 L.F./DAY	8" TIMBER CAP AND POST 4-FOOT CENTERS
V	INBOUND - NORTH STA. 203+31 - STA. 204+80	SHIELD DRIVEN	150	AUG. 1981- SEP. 1981	7.1 L.F./DAY	RIB & LAGGING 4-FOOT CENTERS
VI	OUTBOUND - NORTH STA. 203+00 - STA. 204+84	SHIELD DRIVEN	186	OCT. 1981- DEC. 1981	4.2 L.F./DAY	RIB & LAGGING 4-FOOT CENTERS

* REFERS TO FIGURE 7-6, SEQUENCE OF TUNNELING PROCEDURES.
** ASSUMES 6-DAY WORK WEEK.
*** NON-CRITICAL PATH WORK.

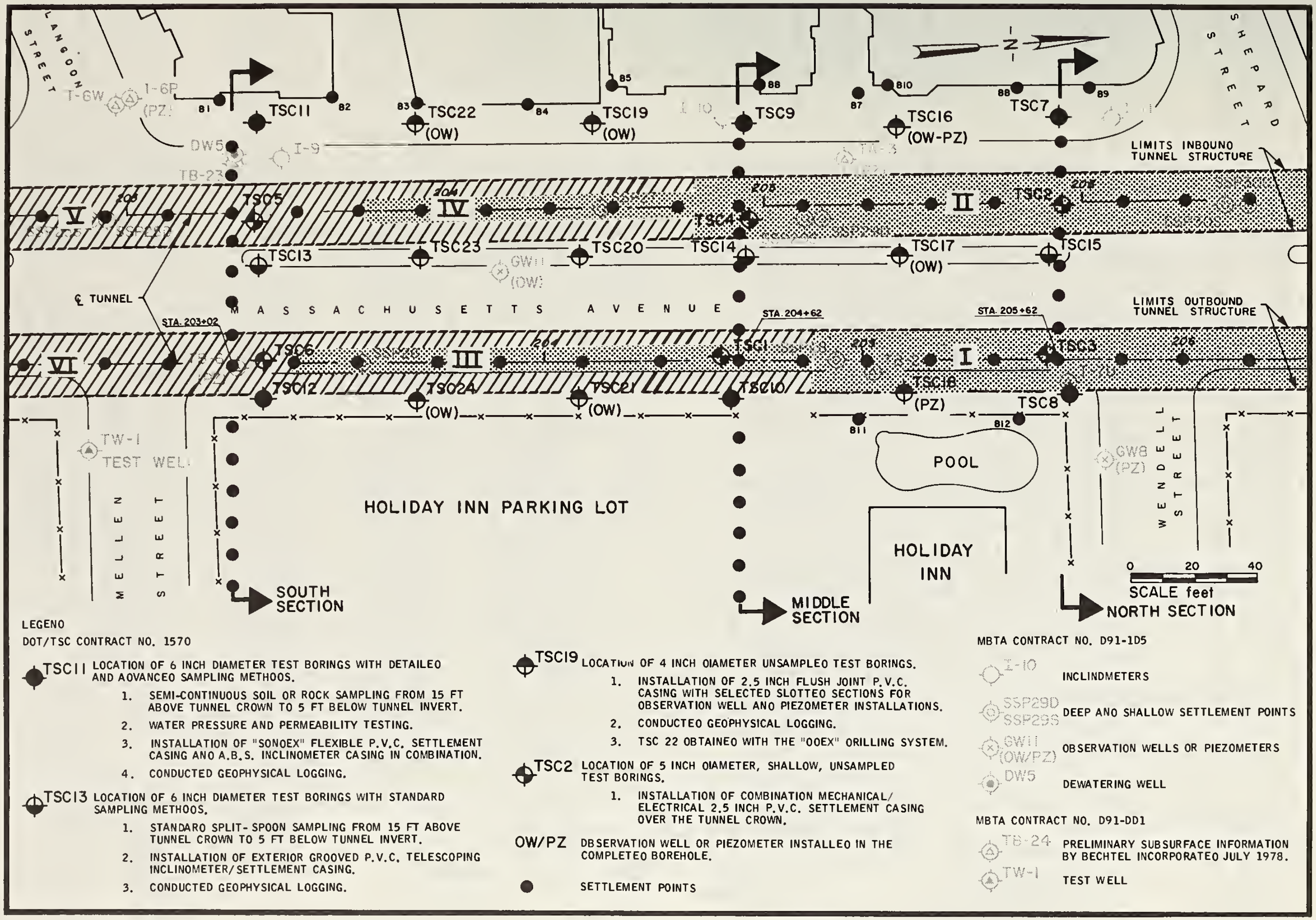


FIGURE 7-6. SEQUENCE OF TUNNELING PROCEDURES*

*ROMAN NUMERALS (I, II, III,....) INDICATE TUNNELING PHASE, REFER TO TABLE 7-1.

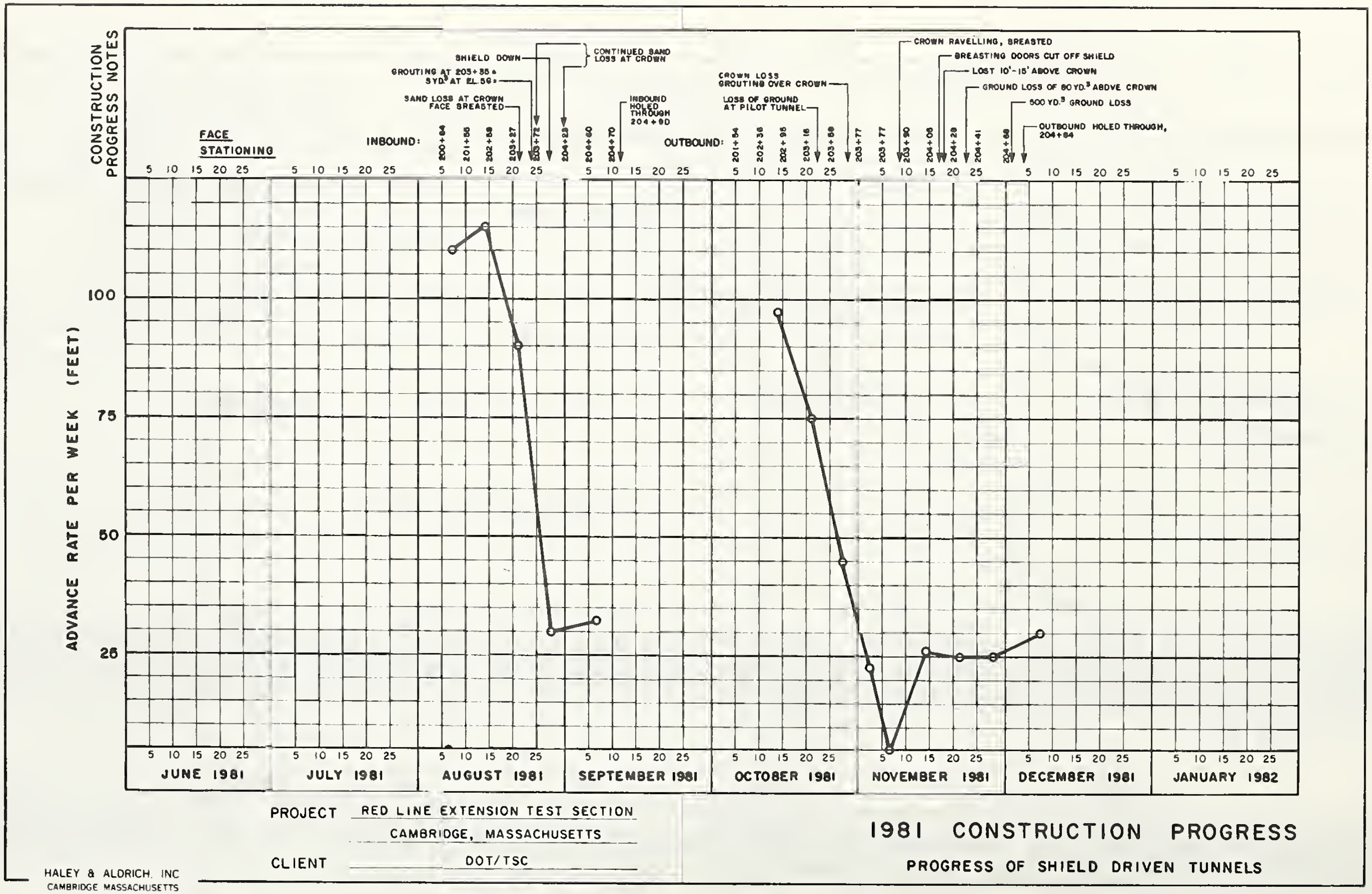


FIGURE 7-7. PROGRESS OF SHIELD DRIVEN TUNNELS

rock may continue to move and minor sloughing can occur. In the Test Section, adequate scaling after blasting and ventilation and proper blocking behind the ribs minimized this movement.

7.6.1.1 Overbreak - The quantity and quality of rock cover diminished as the tunnel headings advanced southward. To control overbreak, steel ribs were installed at 2-foot centers and were blocked tightly to the rock. Also, 30-foot crown bars were installed at low angles above the springline to springline on 1-foot centers. This forepoling procedure provided initial stability and personnel safety as the working face was advanced. It also provided a measure of the remaining distance to the soft ground/ rock interface. The presence of the crown (spile) bars limited the overbreak to 1 to 2 feet in the crown. Overbreak at the side walls was controlled by using short rounds. Scaling loose rock and blocking the ribs to the rock as soon after blasting as possible also reduced sloughing and overbreak. This conservative approach to a difficult tunneling situation eliminated the occurrence of significant overbreak or ground loss while excavating the full face rock in the Test Section.

7.6.2 South Section (Soft Ground)

Tunnel excavation using a shield requires several construction considerations. The face is controlled at all times by breasting or other means. Advance rates can be optimized to minimize ground deformation.

Rib expansion against the tunnel walls is achieved as soon as possible after the rib emerges from the tail section. If too much time elapses, the ground will relax, causing deformation and settlement at the surface.

7.6.2.1 Loss of Ground - During excavation of the inbound and outbound soft ground tunnel sections, the shield was advanced without appreciable ground loss.

7.6.2.2 Rib Spiral - The tunnel through the Test Section was on a tangent. This reduced the tendency for ribs to rotate due to the asymmetrical jacking usually employed on curved tunnel sections. Overexcavation at the sidewalls was minimized, which reduced the tendency for the rib to "roll" into a void created while excavating.

7.6.2.3 Corrections for Misalignment - The tunneling procedure included routine adjustments to maintain the alignment.

7.6.3 Middle Section (Mixed Face)

Tunneling in a mixed face environment is usually complex. The problems encountered in soft ground and rock tunneling are present and often magnified in the mixed face environment. Drilling and blasting, which is often required for rock excavation, has an adverse effect on the soft ground portion of the section. The presence of groundwater can be particularly detrimental to tunneling progress in the mixed face section.

7.6.3.1 Loss of Ground - The inbound shield excavated the mixed face section with only two episodes of unexpected ground loss. Both occurred within 60 feet of the bulkhead which marked the end of full face rock tunneling. The first incident of ground loss occurred at Station 204+25 IB, on August 31, 1981, immediately after blasting commenced at the rock face. The glacial till/rock interface was 2 to 3 feet above springline elevation at the face with fine to medium grained silty sand in the crown. Minor seepage occurred in the face. Blasting may have loosened the ground, causing it to break out about seven feet above and ahead of the shield.

The second episode occurred just prior to the final shove into the full face rock tunnel. The blast from the previous round caused the ground to break ahead of the shield into the unsupported tunnel where an indeterminate amount of ground was lost. The area had been affected by several blasting sequences and the soft ground apparently lost the ability to bridge unsupported lengths of tunnel. This resulted in a stand-up time of essentially zero and consequent failure.

Progress through the outbound tunnel of the Test Section was delayed by numerous ground loss episodes. Causes for the ground loss may be attributed to earlier use of explosives to remove rock from the tunnel heading; to ground losing its strength because of induced stresses caused by the passage of the adjacent inbound tunnel; and to blasting during excavaton of the invert pilot drift. Significant amounts of ground loss occurred at the following locations: Station 203+25 to Station 203+76, Station 204+12, Station 204+32, Station 204+40, Station 204+75, and Station 204+85 OB.

Ground loss usually occurred when blasting of the broken and weathered bedrock resulted in the rock breaking ahead of the cutting edge of the shield. When this occurred, the soft ground was left unsupported and ravelled, creating voids above the shield. As observed in the inbound ground loss occurrences, the overburden soils (soft ground) lost the ability to bridge unsupported lengths of tunnel.

The size of the voids and the extent of stoping associated with each ground loss occurrence dictated the type of remedial action taken. Smaller voids were treated from within the shield with timber, excelsior backpacking and, in some cases, drypacking. In addition, the larger voids were treated by probing and grout injection from ground surface.

7.6.3.2 Rib Spiral - Tunneling on a tangent through the Test Section minimized the uneven jacking pressures on the ribs. Asymmetrical pressures are used to maintain the shield on line and grade. In the mixed face tunnel environment, over-excavation of the rock or soil caused by blasting and rock points bearing against a rib could cause the rib to roll when jacked during shield advance.

7.6.3.3 Corrections for Misalignment - Continuous corrections for misalignment of the shield were made during excavation of the mixed face section. The presence of bedrock in the invert increased the difficulty of maintaining line and grade. Rock points tended to deflect the shield away from the desired course during its advance through the mixed face section.

7.6.4 Grouting Program

To assist in ground control in various locations, it was necessary to perform remedial grouting after the shield had passed. These locations are shown on the geologic tunnel maps in Appendix H.

8. RESULTS OF FIELD MEASUREMENTS

8.1 GENERAL

Because of the large amount of data which was collected during the field monitoring program, only selected data will be presented and analyzed in this report. Appendices D and E present plots of vertical movement vs. date for surface and building settlement points and selected deep settlement points, respectively. Appendix F presents plots of lateral displacement vs. elevation from selected inclinometer observations. Appendix G presents plots of groundwater observations vs. date.

Section 8.2 of this report presents and analyzes the vertical movement data; Section 8.3 the horizontal movement data; and Section 8.4 the groundwater level observations.

8.2 VERTICAL DISPLACEMENTS (SETTLEMENT)

8.2.1 Surface and Building Settlement Points

Figures 8-1 (north), 8-2 (south), and 8-3 (middle) present plots of measured settlement vs. date for some typical surface and building settlement points in the three high intensity instrumentation sections. The construction progress notes on these figures are based on the description of construction procedures and progress, Section 7.

In the north (rock) section, Figure 8-1, negligible settlements were observed prior to 20 July 1980. As the outbound (OB) and inbound (IB) tunnel headings passed through this section (from July 1980 to about the beginning of November 1980), some small settlements gradually developed. Typically, the incremental settlements during this time period were less than 0.015 feet, although a few points in the vicinity of this section showed incremental settlements up to 0.025 feet. Because these settlements were so small, it was not possible to make detailed comparisons between the time sequence of settlements and detailed construction operations. After November 1980 and throughout 1981 soft ground and mixed face tunneling operations to the south, observed settlements in the rock section were negligible.

Figure 8-2 summarizes the settlement observations for two selected surface settlement points in the soft ground (south) section. These points are located within 10 feet of

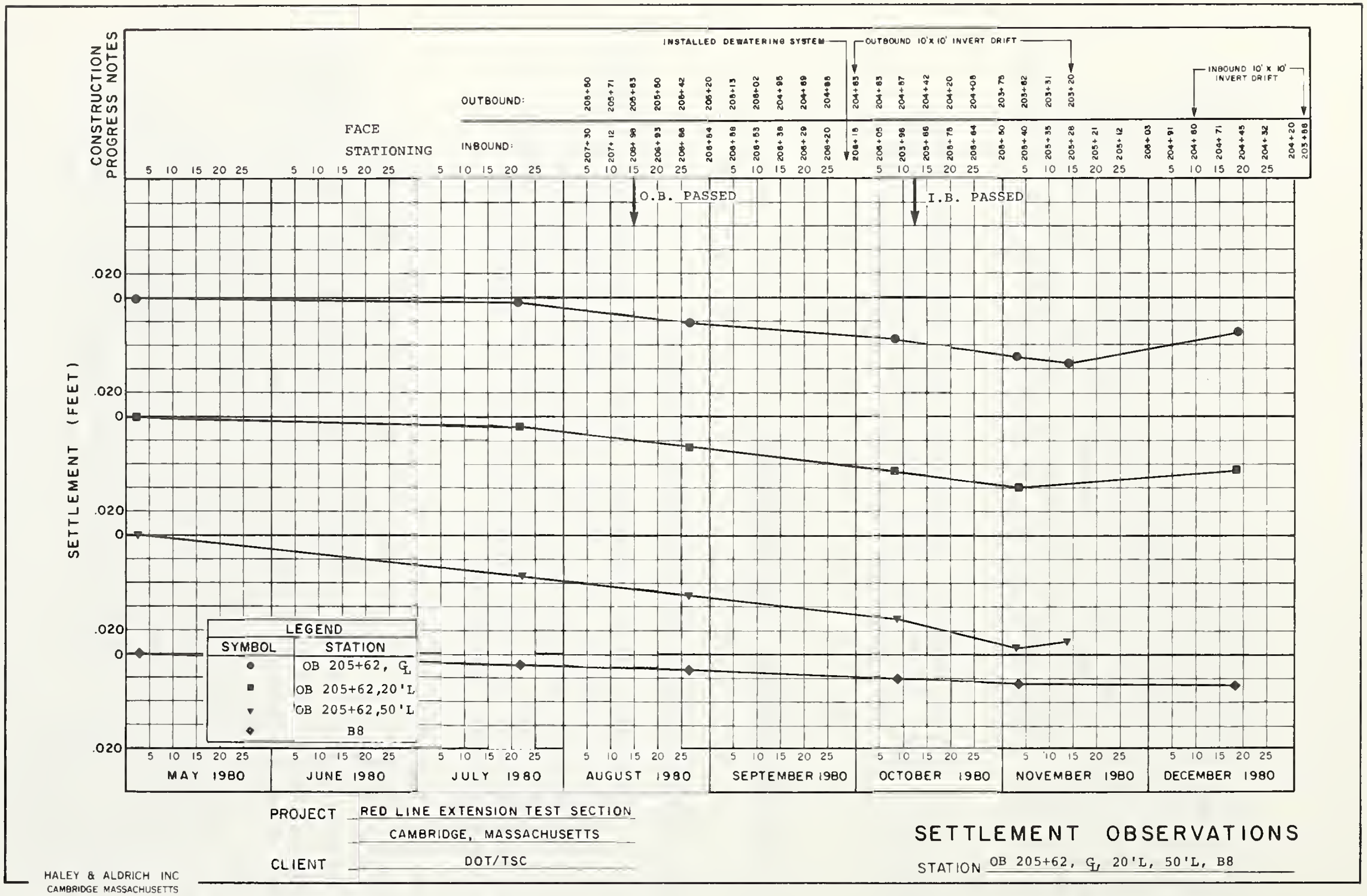


FIGURE 8-1. TYPICAL SURFACE SETTLEMENT POINT OBSERVATIONS - NORTH (ROCK) SECTION

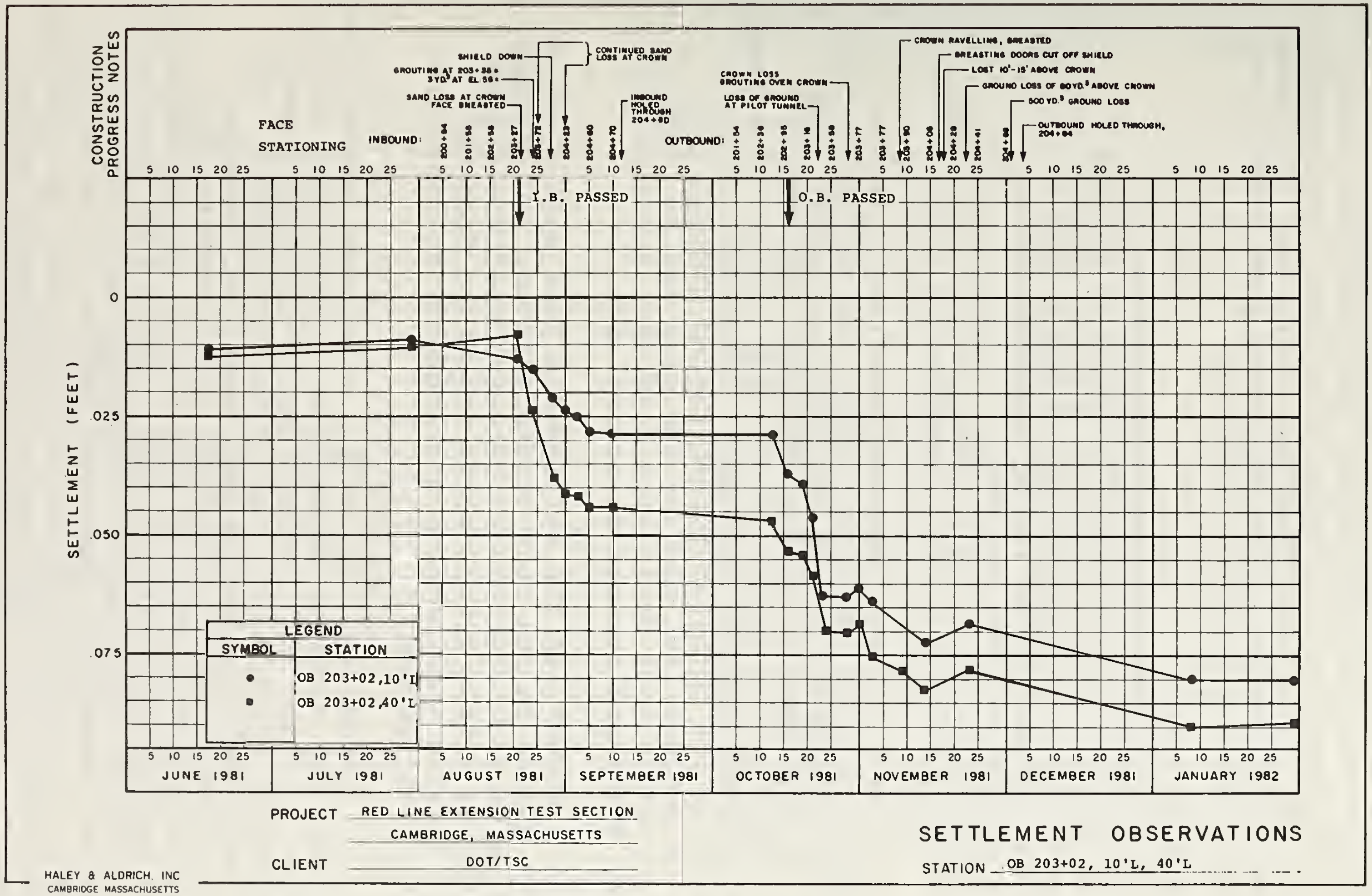


FIGURE 8-2. TYPICAL SURFACE SETTLEMENT POINT OBSERVATIONS - SOUTH (SOFT GROUND) SECTION

the centerlines of the inbound and outbound tunnels. The points directly over the centerlines had incomplete survey data due to obstructions and damage by surface construction operations. The settlements observed before June 1981 were small and were due mostly to factors other than soft ground tunnel construction operations, including:

1. survey error,
2. seasonal ground surface fluctuations,
3. disturbance of settlement points by surface construction operations (a utility trench excavation destroyed many of the survey points on the inbound centerline in November 1980),
4. groundwater lowering, and
5. possibly some ground movement toward the pilot tunnels which had been excavated in 1980.

The settlement points responded immediately as the inbound tunnel passed on 21 August 1981, and surface settlements continued to develop over the next two weeks. Then, from about 10 September to 15 October 1981, just before passage of the outbound tunnel, additional small time dependent surface settlements slowly developed. Settlement rates increased again as the outbound tunnel passed through this section on 16 October 1981 and for the next two to three weeks. Then, additional small time dependent surface settlements slowly developed through 8 January 1982.

Table 8-1 summarizes the settlements that were observed at these two points in the soft ground section. An attempt has been made here to distinguish between immediate settlements, occurring within 2 to 3 weeks after the face had passed, and delayed settlements, attributable to long term adjustments of the ground.

Figure 8-3 summarizes the surface settlement observations at a point over the outbound centerline in the middle (mixed face) section. It should be noted that the inbound tunnel passed the middle section as a full faced rock tunnel. The settlement was small as the inbound tunnel and outbound pilot drift, both in rock, passed the middle section from north to south in 1980. This is very similar to the behavior observed in the north section in rock, Figure 8-1. Settlement here remained small, about 0.015 feet, until the outbound tunnel passed the mixed face section at the end of November 1981. A total settlement of 0.075 feet was recorded on 8 January 1982, with 0.061 feet of it occurring as the outbound tunnel passed. There were insufficient surveys to make comparisons of the time sequence of settlements and the specific details of the outbound tunnel construction operations.

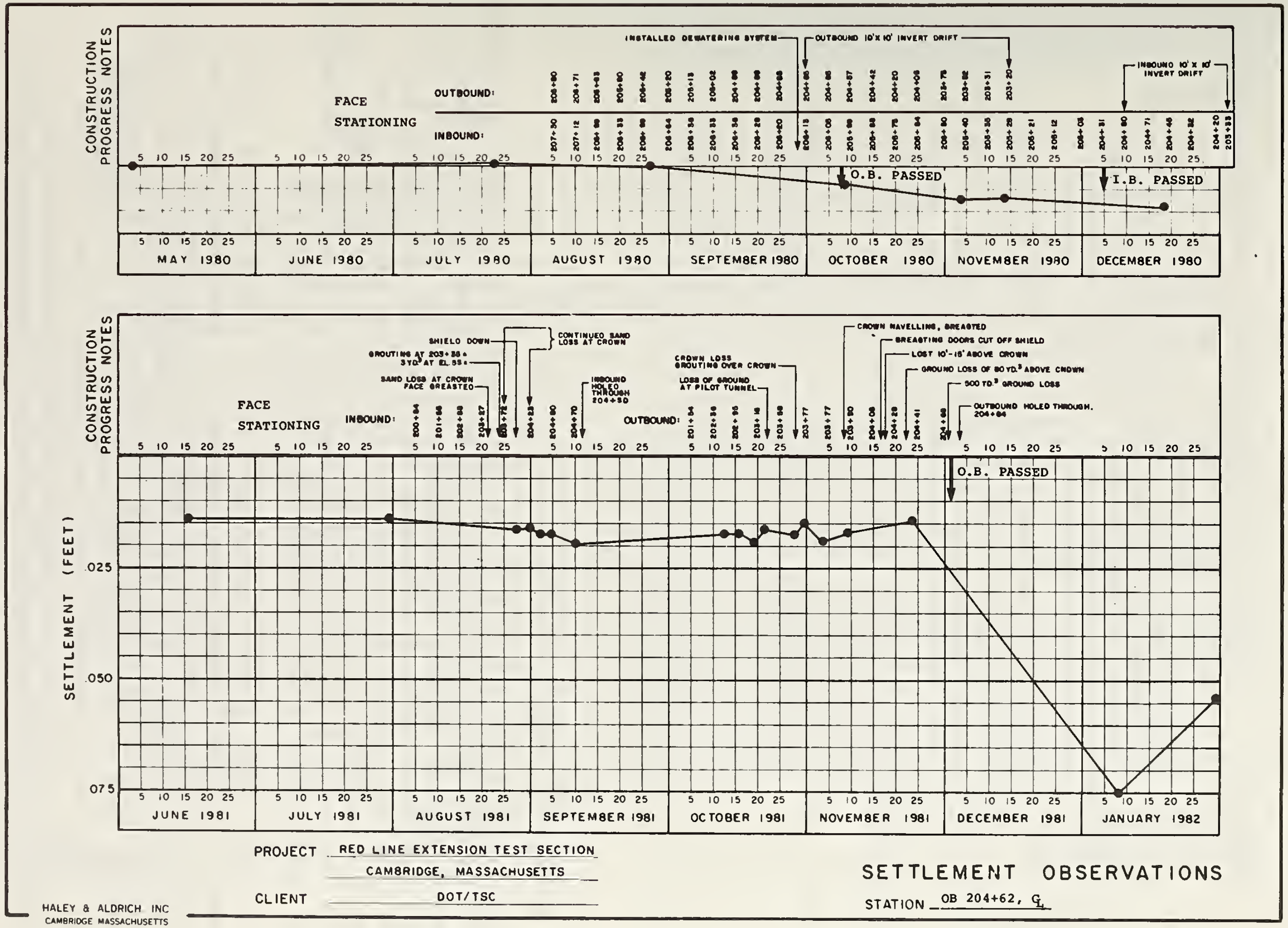


FIGURE 8-3. TYPICAL SURFACE SETTLEMENT POINT OBSERVATIONS - MIDDLE (MIXED FACE) SECTION

TABLE 8-1. SUMMARY OF GROUND SURFACE SETTLEMENTS: SOUTH (SOFT GROUND) SECTION

DATE	IB CENTERLINE (10 ft. right)	OB CENTERLINE (10 ft. left)	COMMENTS
17 June 1981	0 ft.	0 ft.	
10 September 1981	0.032 ft.	0.017 ft.	Incremental settlement, IB tunnel
13 October 1981	0.003 ft.	0.000 ft.	Delayed settlement, IB Tunnel
	0.035 ft.	0.017 ft.	TOTAL SETTLEMENT, IB TUNNEL
23 November 1981	0.031 ft.	0.040 ft.	Incremental settlement, OB tunnel
8 January 1982	0.012 ft.	0.012 ft.	Delayed settlement, OB tunnel
	0.043 ft.	0.052 ft.	TOTAL SETTLEMENT, OB TUNNEL
	0.078 ft.	0.069 ft.	TOTAL SETTLEMENT, IB AND OB TUNNELS

8.2.2 Deep Settlement Points

Figures 8-4 (north), 8-5 (south), and 8-6 (middle) present plots of measured settlement vs. date for some typical deep settlement points in the three high intensity instrumentation sections. The locations of these instruments are shown on Figures 3-6, 6-1, 6-2, and 6-3. As would be expected, the settlement behavior of the surface box rims, Figures 8-4, 8-5, and 8-6, was very similar to the behavior of the corresponding surface settlement points, Figures 8-1, 8-2, and 8-3.

For the north (rock) section, TSC 2 in Figure 8-4, the settlements at depth were very similar to the settlements at the ground surface. They were generally very small (less than about 0.015 feet) and developed gradually as the outbound and inbound tunnel headings were excavated in rock from July to November 1980. The apparent heave of ring 6, TSC 2 from 25 June 1980 to 27 August 1980, is a feature that did not appear in any of the other deep settlement points in the rock section, and is probably due only to measurement error.

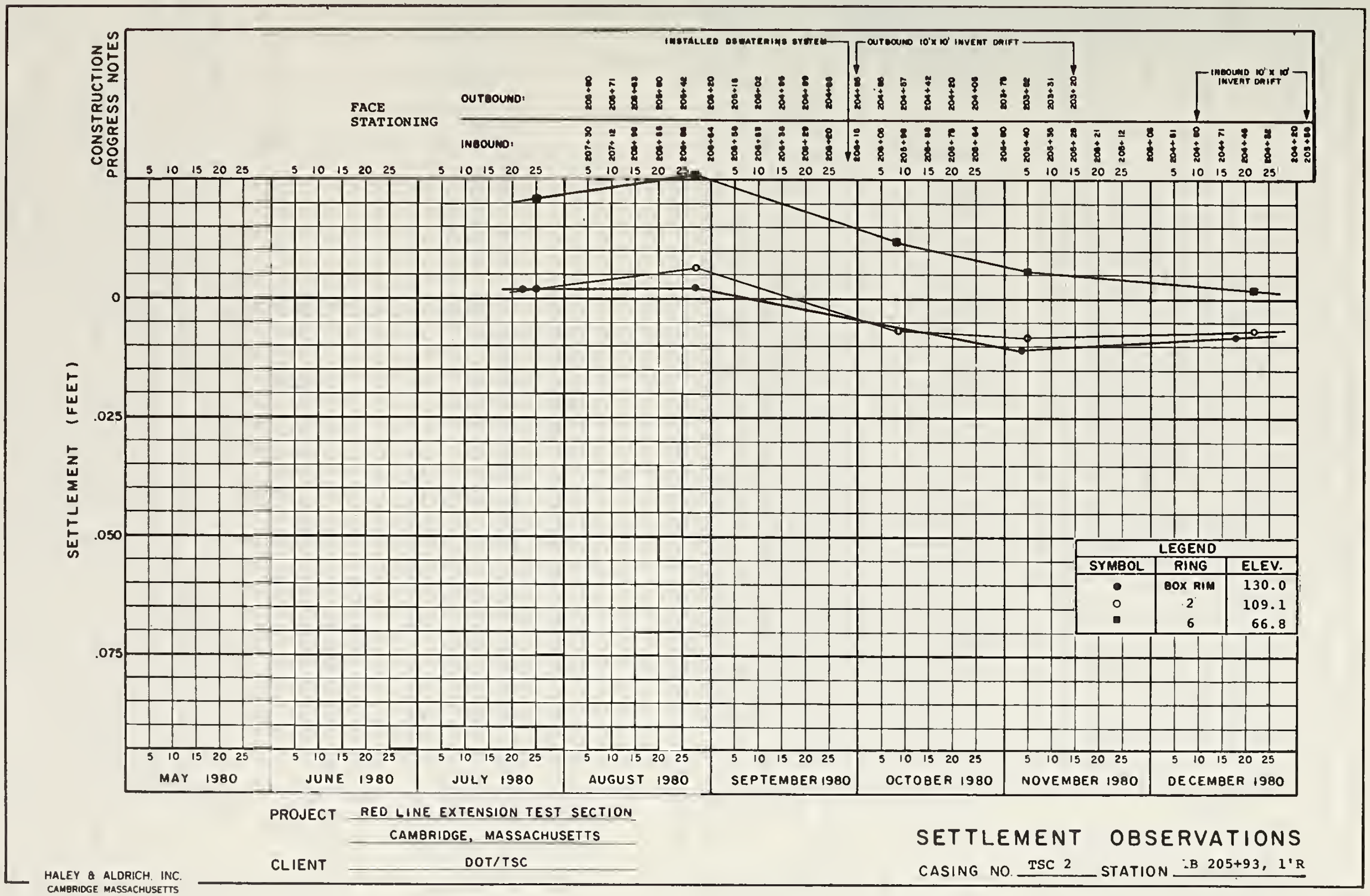


FIGURE 8-4. TYPICAL DEEP SETTLEMENT POINT OBSERVATIONS - NORTH (ROCK) SECTION

For the south (soft ground) section, TSC 5 in Figure 8-5, ring 7, located about 10 feet above the inbound tunnel crown, recorded a very large increment of settlement (0.354 feet) as the inbound heading passed in August 1981. There were substantial losses of ground at the tunnel face and crown during this time period. Despite this, these large ground movements did not propagate very far away from the tunnel. For example, the increment of settlement due to advance of the inbound tunnel, from early August through early September 1981, was only about 0.09 feet at ring 6, 18 feet above the tunnel crown. At the instrument box rim, the corresponding incremental settlement was only 0.033 feet.

For the middle (mixed face) section, the deep settlement points of TSC 10, Figure 8-6, recorded relatively small settlements, except for the time period when the outbound soft ground tunnel heading passed. This is similar to observations at the ground surface, Figure 8-3. For the time period from 30 November 1981 to 10 December 1981, rings 3 and 10 recorded increments of settlement of 0.028 and 0.040 feet, respectively. Because they were surveyed more frequently during this time period, the deep settlement point observations, Figure 8-6, show a more detailed relation with outbound tunneling operations than the surface settlement observations, Figure 8-3.

8.2.3 Settlement vs. Tunnel Face Stationing

Plots of settlement vs. the position of the tunnel's working face sometimes help to determine the source of ground loss. For example, Cording et al. (I-3) divides the settlements into four stages, as follows:

1. ahead of the face,
2. over the shield,
3. during erection of the lining, and
4. with time and with further advance of the heading.

However, this requires very frequent settlement observations, say with each advance of the shield. Figure 8-7 plots incremental settlement vs. the distance from the working face of the tunnel for three surface settlement points over the soft ground tunnel. These points have the greatest amount of available data, and the plots seem to indicate the presence of all of the four stages listed above as contributory. However, there are insufficient data to make detailed numerical breakdowns. It is interesting to note that both of the outbound surface settlement points plotted on Figure 8-7, Station 203+22 and Station 203+67, show the effects of the crown loss at Station 203+77 OB, 55 and 10 feet away from those settlement points, respectively.

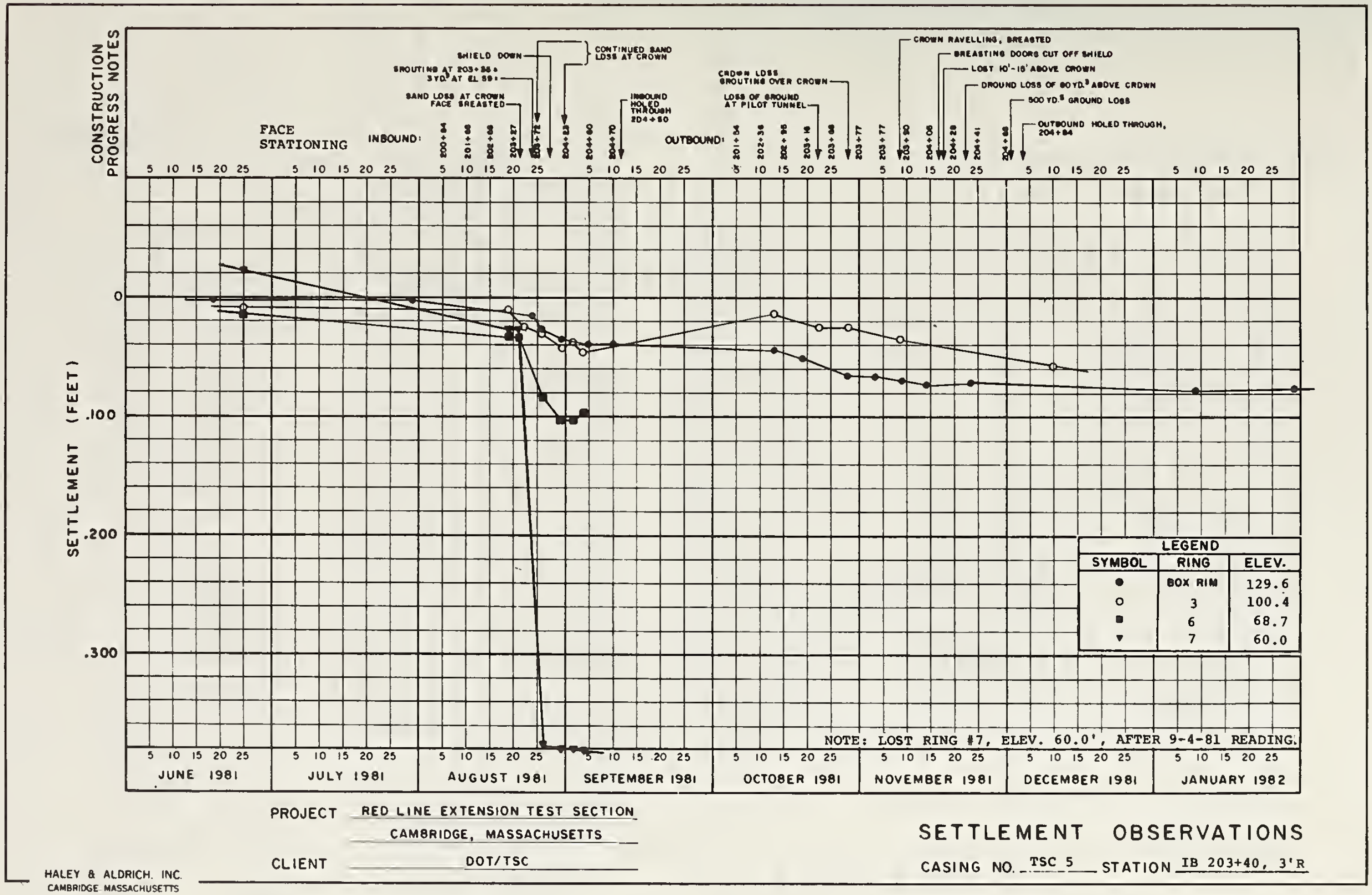


FIGURE 8-5. TYPICAL DEEP SETTLEMENT POINT OBSERVATIONS - SOUTH (SOFT GROUND) SECTION

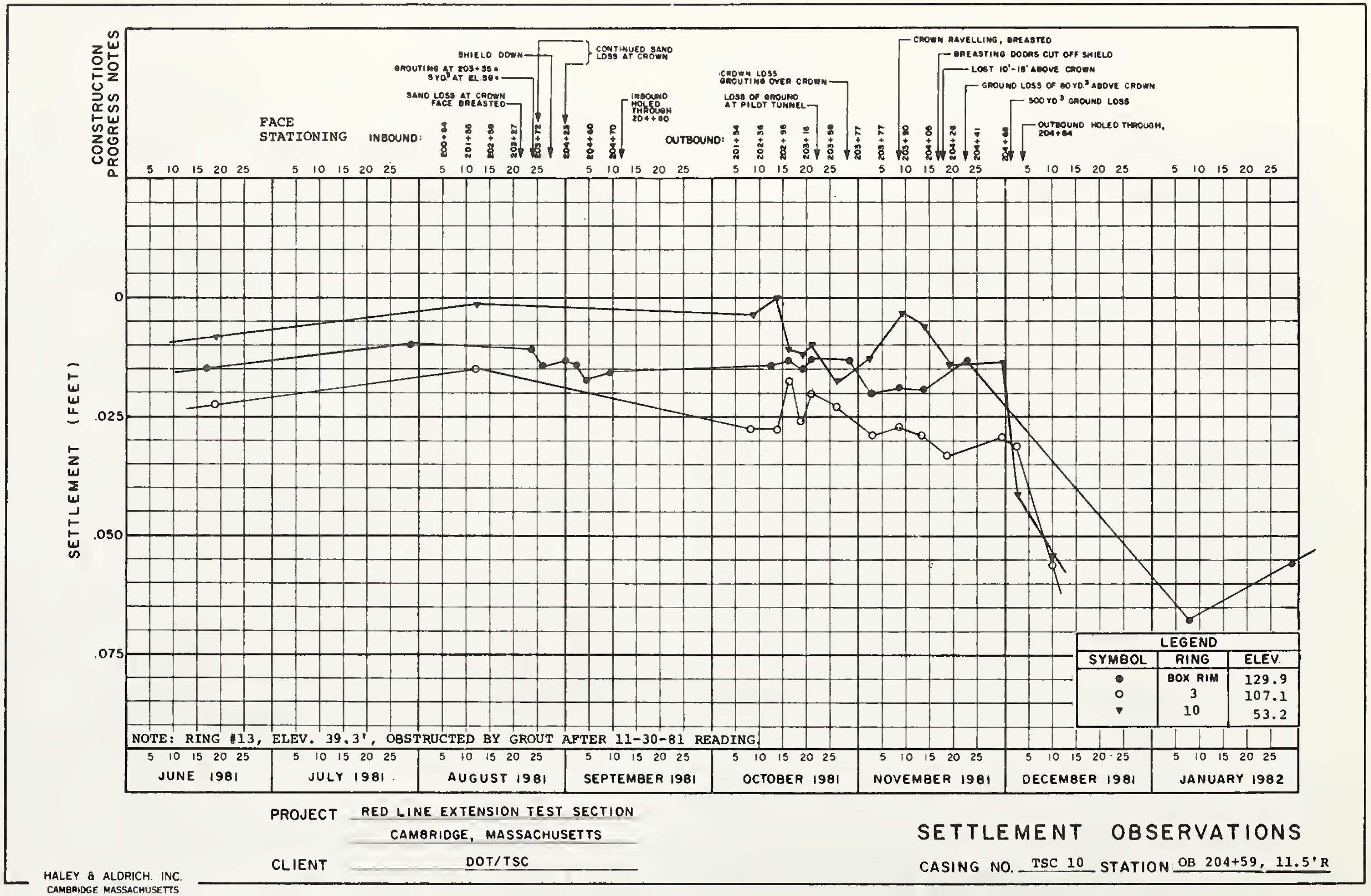


FIGURE 8-6. TYPICAL DEEP SETTLEMENT POINT OBSERVATIONS - MIDDLE (MIXED FACE) SECTION

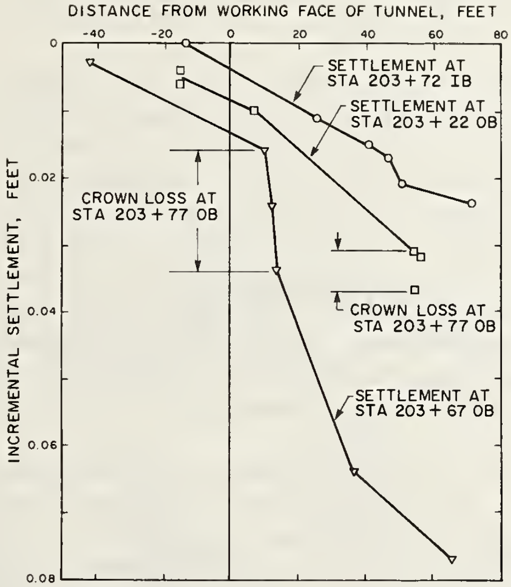


FIGURE 8-7. INCREMENTAL SURFACE SETTLEMENT VS. DISTANCE FROM WORKING FACE OF TUNNEL

8.2.4 Profiles of Settlement (Settlement Troughs)

Figures 8-8, 8-9, and 8-10 present summary profiles of observed settlements at the ground surface and at selected deep settlement points for the north, south, and middle sections, respectively. The generalized subsurface conditions are superimposed on these profiles as well as a graphic legend which relates the observed settlements to tunneling operations. These figures are discussed in detail in the following paragraphs.

Figure 8-8 summarizes the surface and deep settlements which developed in the north (rock) section as the outbound (16 August 1980) and inbound (11 October 1980) tunnels passed. Ground surface settlements were small, less than about 0.015 feet, and occurred fairly uniformly across the section. Deep settlements were of the same order as those observed at the ground surface. This indicates that there was no substantial compression or expansion of soil. Because the settlements and the number of observations were both small here (Figure 8.1), it is not possible to relate the observed settlements to the details of construction. If the survey of 8 October 1980 is used to distinguish between outbound and inbound tunneling operations, then about two-thirds of the total settlement occurred as the first tunnel (outbound) passed through the

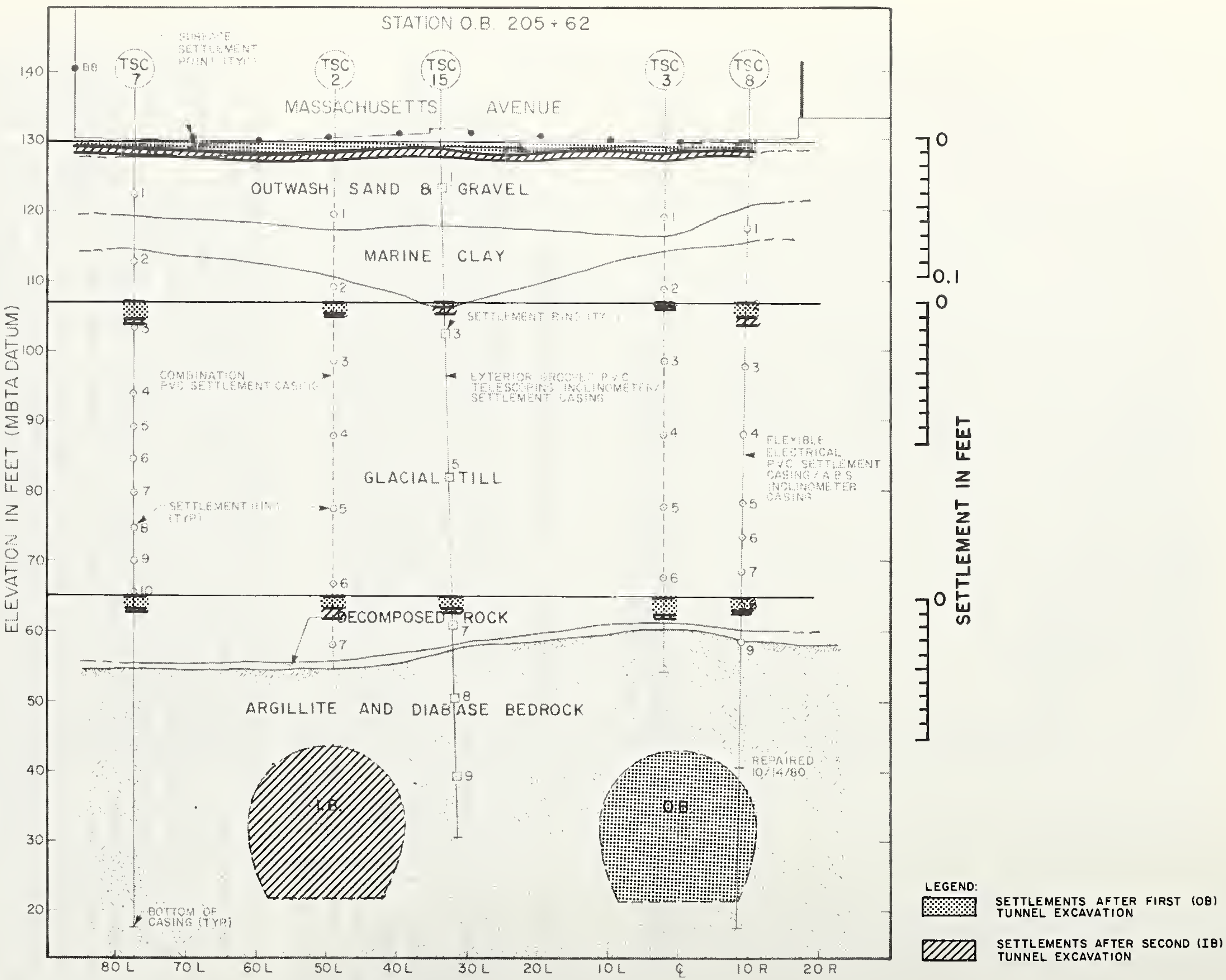


FIGURE 8-8. PROFILE OF SETTLEMENTS - NORTH SECTION

rock section and the remaining one-third, as the second tunnel (inbound) passed. However, for the reasons stated above, this observation must be regarded as highly speculative.

Figure 8-9 summarizes the surface and deep settlements which developed in the south (soft ground) section as the inbound (21 August 1981) and outbound (16 October 1981) tunnels passed. Settlements which developed between 18 June 1981 and the 15 October 1981 survey are presumed to occur due to the inbound tunnel construction. Some of this settlement, however, may represent ground movement toward the advancing outbound face rather than delayed settlement caused by the inbound tunnel. Settlements which developed between 15 October 1981 and 8 January 1982 are presumed to occur due to passage of the second tunnel, in the outbound direction.

The surface settlement trough caused by the inbound tunnel construction extended beyond the limits of Massachusetts Avenue, a total width of more than 150 feet. A maximum surface settlement of about 0.035 feet was observed over the centerline of the inbound tunnel. As the outbound tunnel passed through the south section, a maximum incremental surface settlement of 0.052 feet was observed 10 feet to the left of the outbound centerline. The combined surface settlement trough, showing the effects of both tunnels, showed a maximum total surface settlement of 0.078 feet, over the inbound tunnel. The maximum total surface settlement in the vicinity of the outbound tunnel was about 0.069 feet. The settlement trough has a very gradual slope over its 150-foot width.

The deep settlement points, Figure 8-9, show settlements of the same order of magnitude as observed at the ground surface, with one exception. The deep settlement point over the inbound centerline showed a settlement of 0.418 feet, mostly due to some substantial losses of ground at the tunnel face. Settlements of this order were not observed at any other instrument locations in the soft ground section because of the ability of the dense glacial till to carry load, or arch, over any opening caused by ground loss.

Figure 8-10 summarizes the surface and deep settlements which developed in the middle (mixed face) section. Settlements between the surveys of 26 August 1980 and 30 July 1981 are assumed to occur as a result of 1980 rock tunneling operations. These include construction of the inbound tunnel here using full face drill-and-blast procedures and advancement of a 10-foot square invert drift along the outbound tunnel alignment. Settlements between 30 July 1981 and 8 January 1982 are assumed to occur as a result of the completion of the outbound tunnel in mixed face conditions in late November 1981.

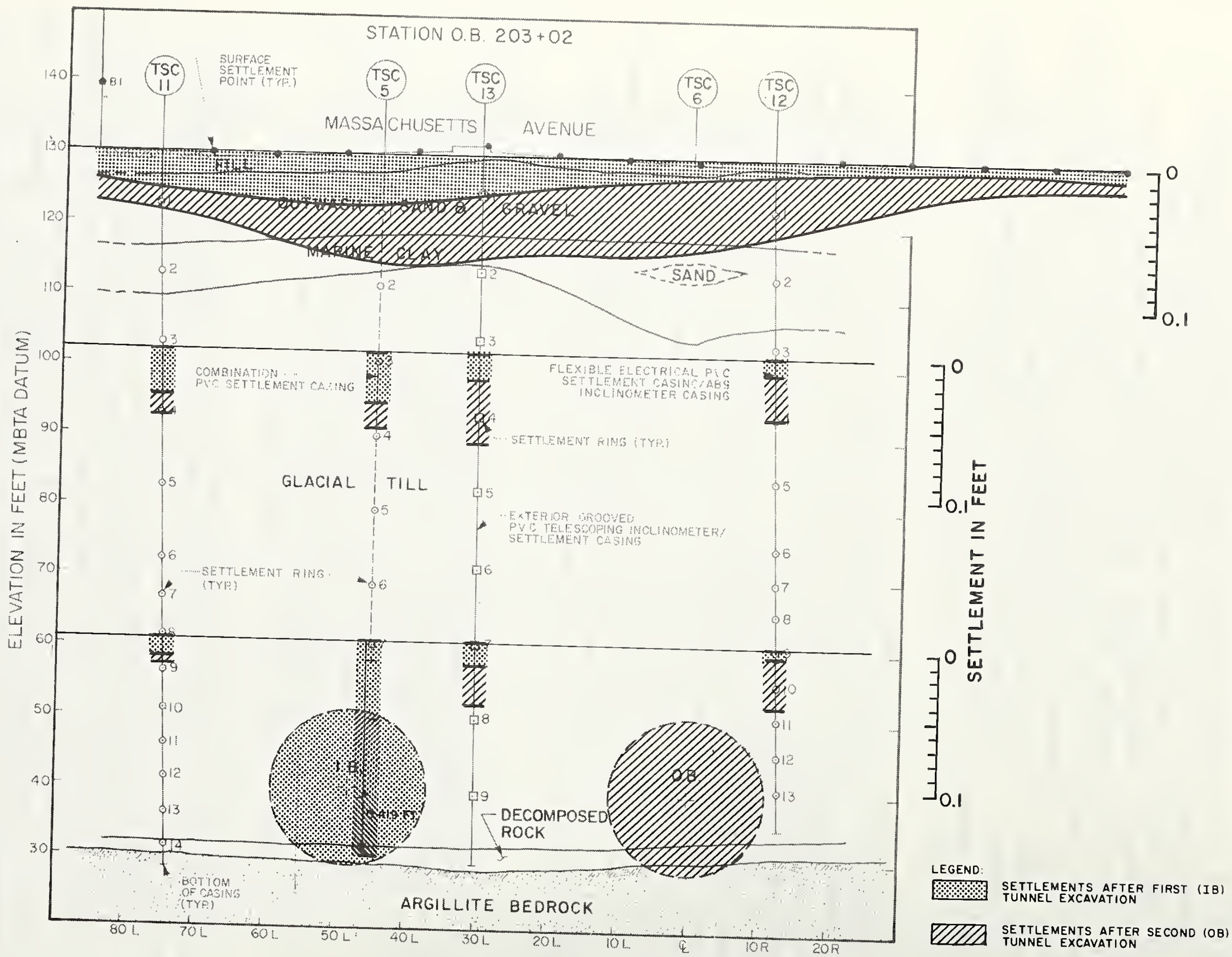


FIGURE 8-9. PROFILE OF SETTLEMENTS - SOUTH SECTION

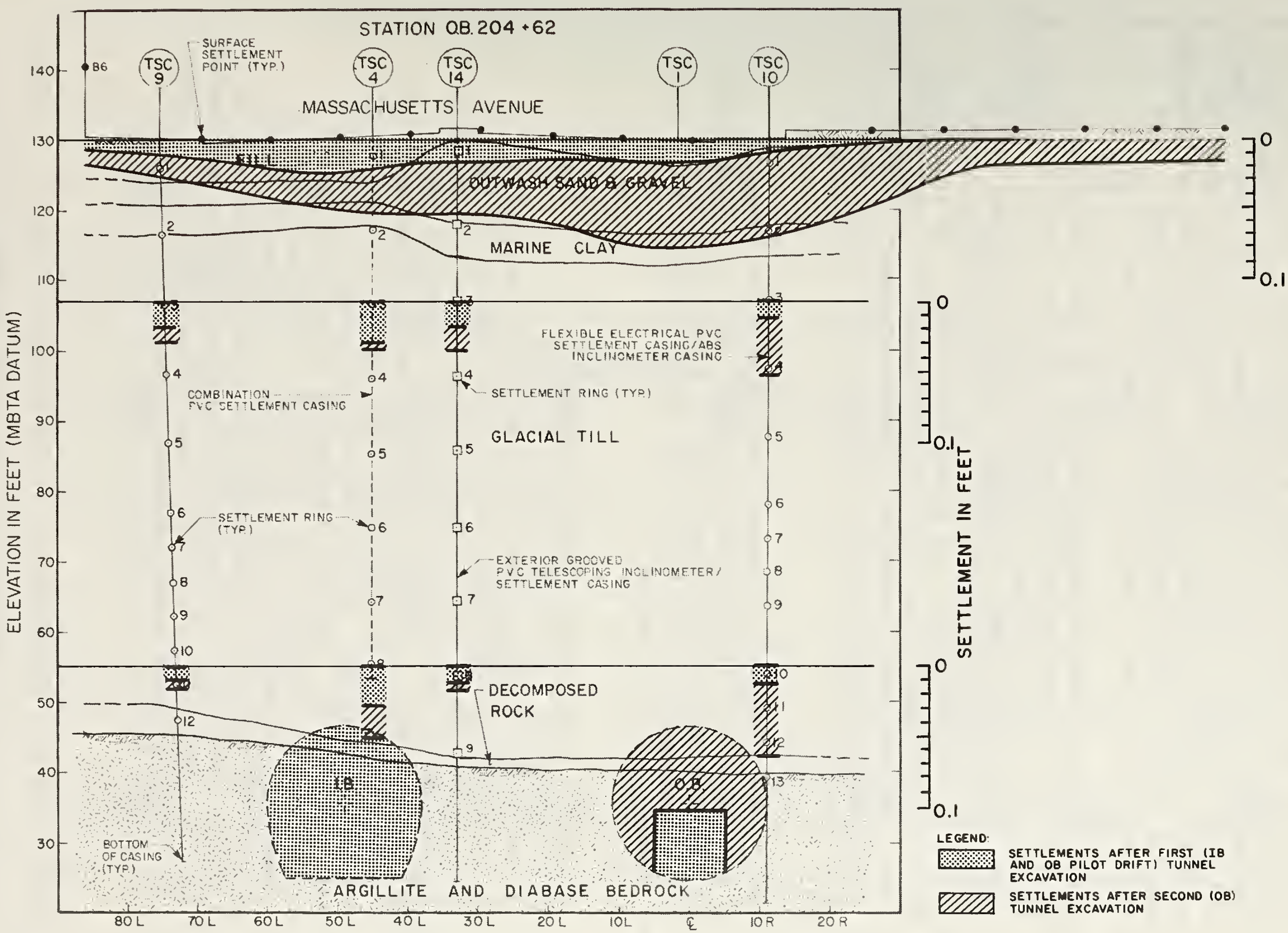


FIGURE 8-10. PROFILE OF SETTLEMENTS - MIDDLE SECTION

The surface settlement trough caused by the 1980 rock tunneling operations extended roughly to the limits of Massachusetts Avenue, a total width of approximately 120 feet. The trough has very gradual slopes, with a maximum settlement of 0.022 feet over the inbound centerline. The surface settlements are in general small, although perhaps somewhat larger than those observed in the north section in rock, Figure 8-8.

The surface settlement trough caused by the outbound mixed face tunnel is considerably larger, deeper, and steeper. Much of this was caused by some major losses of ground, including a crown loss in the outbound mixed face tunnel heading in November 1981. The maximum increment of surface settlement caused by the outbound tunnel is 0.061 feet over the outbound tunnel, and the maximum total settlement caused by both tunnels is 0.076 feet at the same location. The settlement trough extended well beyond Massachusetts Avenue, with 0.013 feet of settlement observed 75 feet to the right of the outbound centerline.

The deep settlement points in the mixed face section (Figure 8-10) show settlements of the same order of magnitude as observed at the ground surface. As in the soft ground section, this shows the ability of the dense glacial till to arch over severe ground losses.

8.2.5 Settlement Monitoring Data by Others

The construction contractor provided the results of surface settlement surveys made on 1 August 1981, 31 August 1981, 30 September 1981, and 31 March 1982. These data were provided in the form of surface settlement contours at an interval of 0.05 foot. For the 30 September 1981 survey, the settlements measured within the Test Section by the contractor did not exceed 0.05 feet. The 31 March 1982 survey (Figure 8-11) shows a 0.05-foot contour encompassing most of the south and approaching the middle sections. Surface settlements greater than 0.10 feet were observed in a small area above the outbound tunnel, approximately from Station 203+30 to Station 203+60. North of the middle section, surface settlements measured by the contractor were all less than 0.05 feet. These observations are compared with the data from this research project in the following paragraphs.

8.2.6 Summary of Settlements

Figure 8-12 plots contours of the total measured surface settlements due to the twin tunnel construction on a

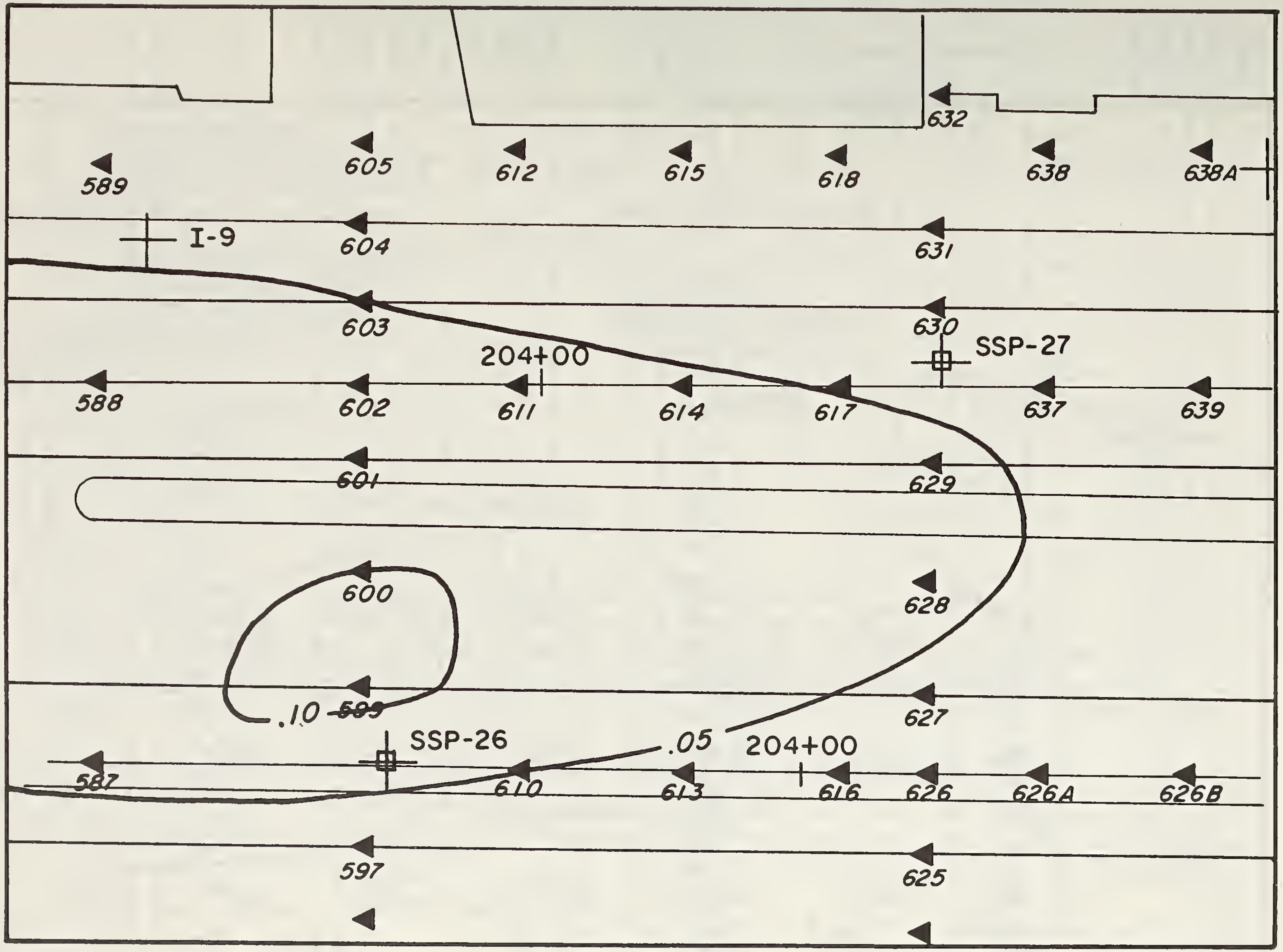


FIGURE 8-11. CONTOURS OF SURFACE SETTLEMENTS OBSERVED BY CONTRACTOR, 31 MARCH 1982

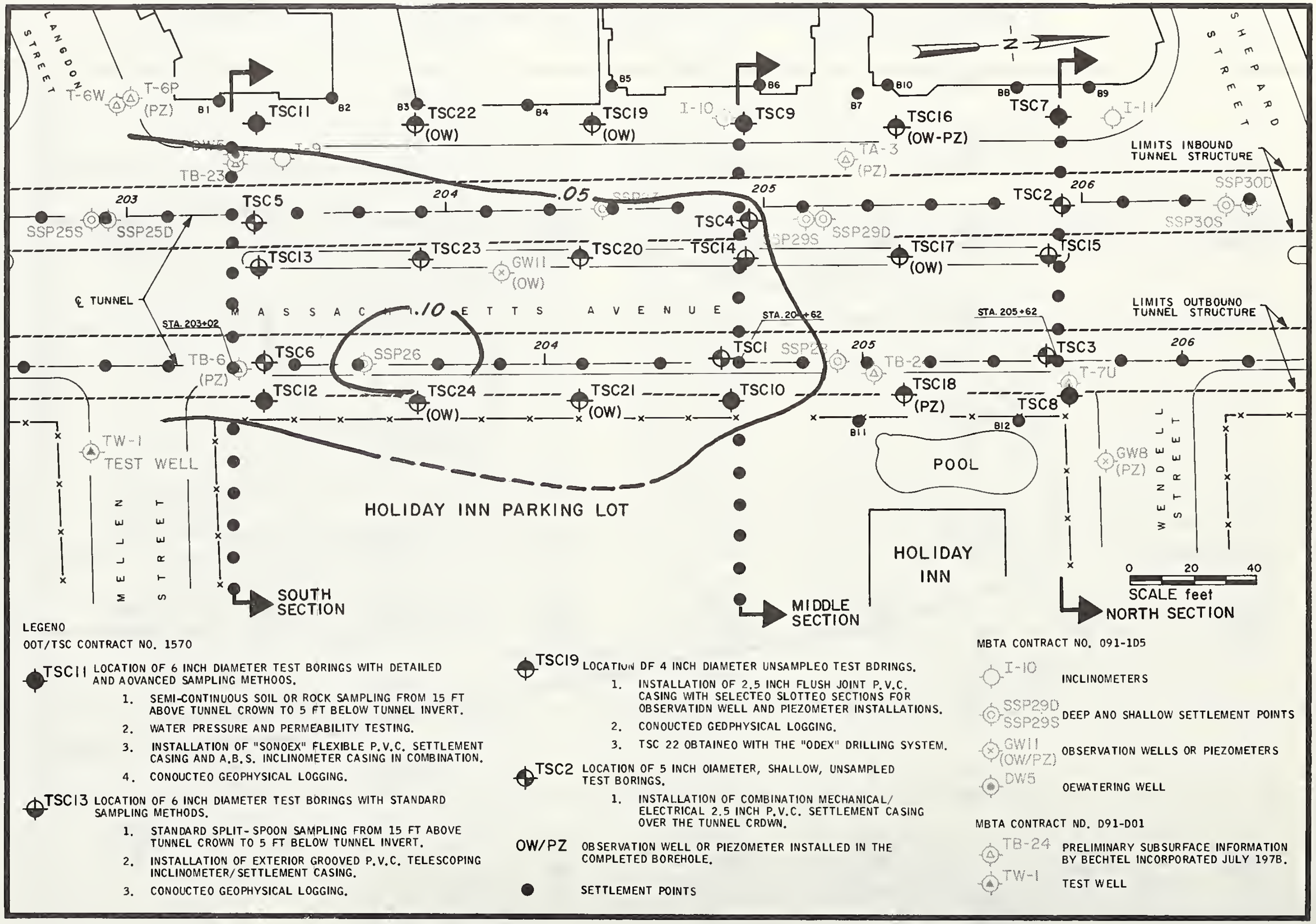


FIGURE 8-12. SUMMARY OF SURFACE SETTLEMENTS OBSERVED DURING STUDY

plan of the Test Section. The 0.05 and 0.10-foot settlement contours shown on this plan are in general agreement with the settlement data provided by the Contractor, Figure 8-11. Table 8-2 summarizes the measured surface settlements at the deepest portions of the settlement troughs.

TABLE 8-2 SUMMARY OF MEASURED SURFACE SETTLEMENTS

SECTION	TUNNELING CONDITIONS	MEASURED (ft.)	SETTLEMENT (in.)
South Section	Soft ground	0.078	0.94
Middle Section	Mixed face	0.076	0.91
North Section	Rock	0.015	0.18

8.3 HORIZONTAL DISPLACEMENTS

Borehole inclinometer casings were installed at the locations shown on Figure 3-6 for the purpose of measuring horizontal ground movements caused by the tunnel construction. These data can sometimes identify sources of lost ground and the limits of soil strains resulting from tunneling procedures.

Casings TSC 7, 9, and 11 were located outside of the springline of the inbound tunnel, casings TSC 8, 10, and 12 outside of the springline of the outbound tunnel, and casings TSC 13, 14, and 15 in the pillar between the tunnels. All casings were drilled into bedrock within a few feet of the elevation of the tunnel inverts, except for casing TSC 12, which terminated in the glacial till about 5 feet above the bedrock surface. The locations of these inclinometer casings are shown in profile in Figures 6-1, 6-2, and 6-3. Detailed descriptions of the equipment, and installation and monitoring procedures are presented in Section 6 of this report.

Appendix F presents plots of lateral displacements vs. elevation for selected inclinometer observations. These displacements are all occurring transverse to the tunnel centerlines and are referenced to initial inclinometer surveys made in May and June 1980, before the tunnels were advanced into the Test Section. These plots were prepared by integrating the measured angular displacements from the bottom of the casing to the top, assuming that the bottom of the casing remains fixed and does not move laterally.

The measured horizontal displacements are very small. Except for TSC 9 and 14, the horizontal displacements accumulated at the ground surface are less than one inch, and in two inclinometer casings (TSC 7 and 12) less than 0.5 inches. TSC 9 shows a maximum horizontal displacement at the ground surface of about 1.5 inches, and TSC 14 of about 1.2 inches (see Appendix F).

The development of these inclinometer profiles with time shows some interesting behavior. For example, the inclinometer observations at TSC 10 (Station 204+59 OB) on 31 October and 22 November 1981 (Figure 8-13) show displacements away from the tunnels and do not seem to be logically related to the observatins made on nearby dates. Furthermore, the outbound tunnel did not reach this station until 1 December 1981.

These measured displacements at TSC 10 may be accurate and a result of tunneling operations. They may be due to the relatively severe ground losses experienced downstation in the outbound tunnel. Outward movement may be caused by grouting or jacking of tunnel liners into place.

On the other hand, these measured displacements may be mostly the result of inclinometer survey error. Sources of error, discussed in detail in Section 10, include:

1. lateral displacement of the bottom of casing,
2. casing spiral,
3. non-verticality of the casing transverse to the measurement plane, and
4. sensor instrument error.

For example, the repeatability limits of the sensor are superimposed on the data of Figure 8-13. These limits are based upon normal laboratory calibration checks of the Digitilt™ inclinometers used and correspond to a repeatability level of ± one unit of angular displacement in 1600 (± 0.75 in. over 100 ft.). Figure 8-13 shows that many of the measured displacements, including the displacements away from the tunnel, fall within the repeatability limits of Digitilt™ inclinometer. Thus, it may be that the lateral ground movements were so small as to be obscured by the normal accuracy of the instrument.

Inclinometer survey errors frequently accumulate as the measurements are integrated up the casing. Because of this, the position of the inclinometer profiles in space, Figure 8-13 and Appendix F, are somewhat speculative. On the other hand, the shapes of the inclinometer profiles are not so greatly affected by errors which accumulate up the casing. The shapes also provide an excellent indication of where ground loss is occurring.

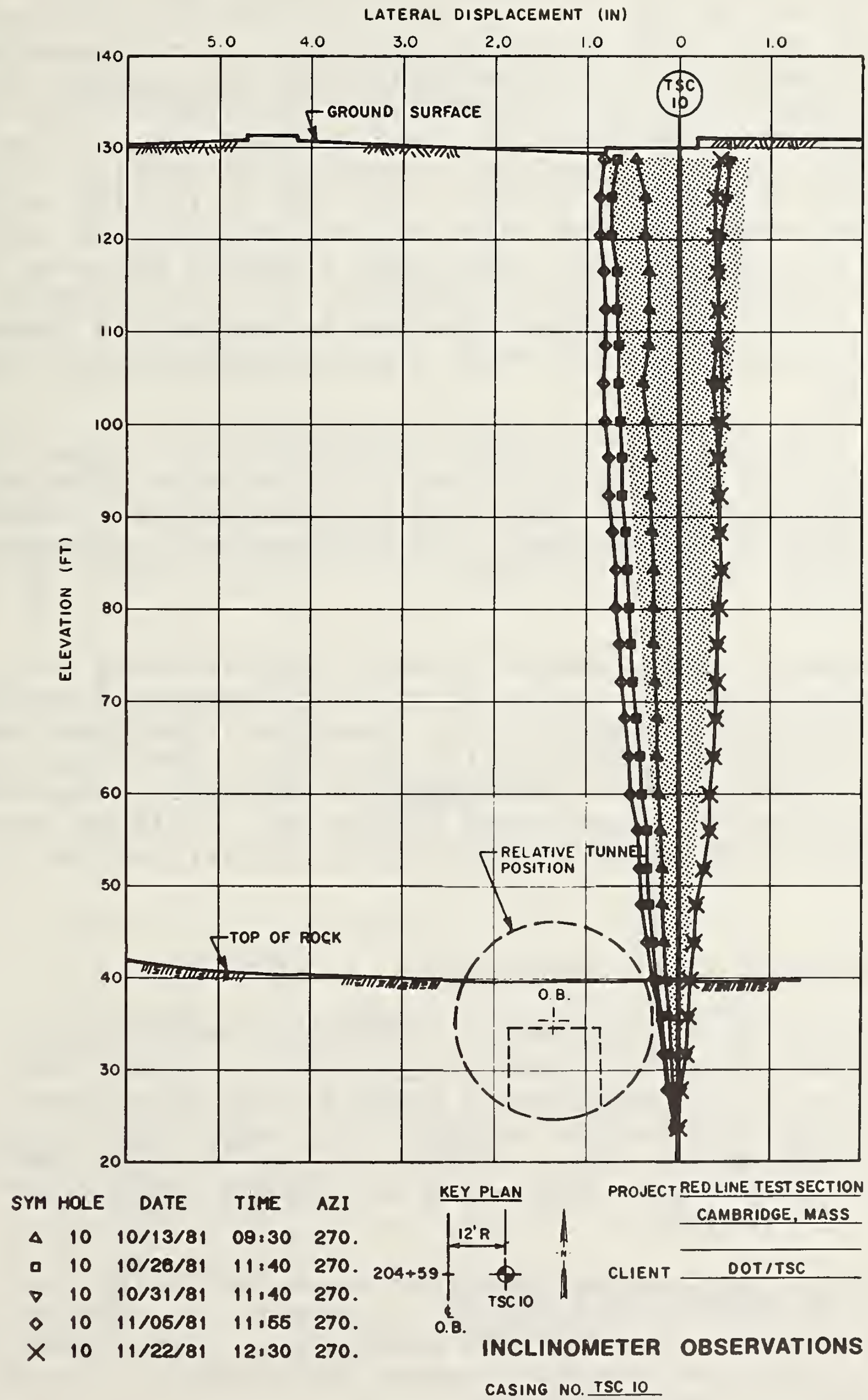


FIGURE 8-13. SELECTED INCLINOMETER OBSERVATIONS - TSC 10

Figure 8-14 shows selected inclinometer profiles at casing TSC 12. As in the previous Figure 8-13, the positions of these profiles in space show surficial horizontal displacements away from the tunnel, and are chronologically, not necessarily related to each other or to tunneling operations. Note also that because the casing was not carried deep enough, its bottom moved laterally. Nevertheless, the shape of these profiles clearly show that a zone of soil near the springline of the outbound tunnel experienced significant deformations as the tunnel passed on 16 October 1981. This zone extended to about 18 feet above the tunnel crown, elevation 68, by 27 October 1981. In this manner, these data can be useful for identifying sources of ground loss and zones which experience the greatest defomations.

This suggests that for small deformations, it may be more appropriate to interpret these inclinometer surveys by plots of inclination vs. depth rather than by the traditional plots of lateral displacement. The development of these plots, including their use in correcting for survey errors, is discussed in Section 10.

In summary, the measured lateral displacements are small, approaching the limits of the inclinometer measuring system. As shown by Figure 8-14, these data indicate that for normal tunneling operations, with no large, sudden ground loss, the zone of greatest soil deformations extended no more than about one tunnel diameter above the tunnel. This is consistent with the data from the deep settlement points, Section 8.2.2.

8.4 GROUNDWATER LEVEL OBSERVATIONS

As discussed in Section 4, the Stage I site studies indicated the presence of two piezometric surfaces, one associated with the hydrostatic heads in the near surface outwash sands and gravels, and the other associated with the permeable zone in the upper surface of the fractured bedrock. The static piezometric surface in both zones range from elevation 116 to 121.

Plots of groundwater elevation observed during construction vs. date are presented as Appendix G. The location of the instruments are shown on Figure 3-6. Also shown in Appendix G, are the bottom of screen elevations, if known, and the tunnel crown elevations.

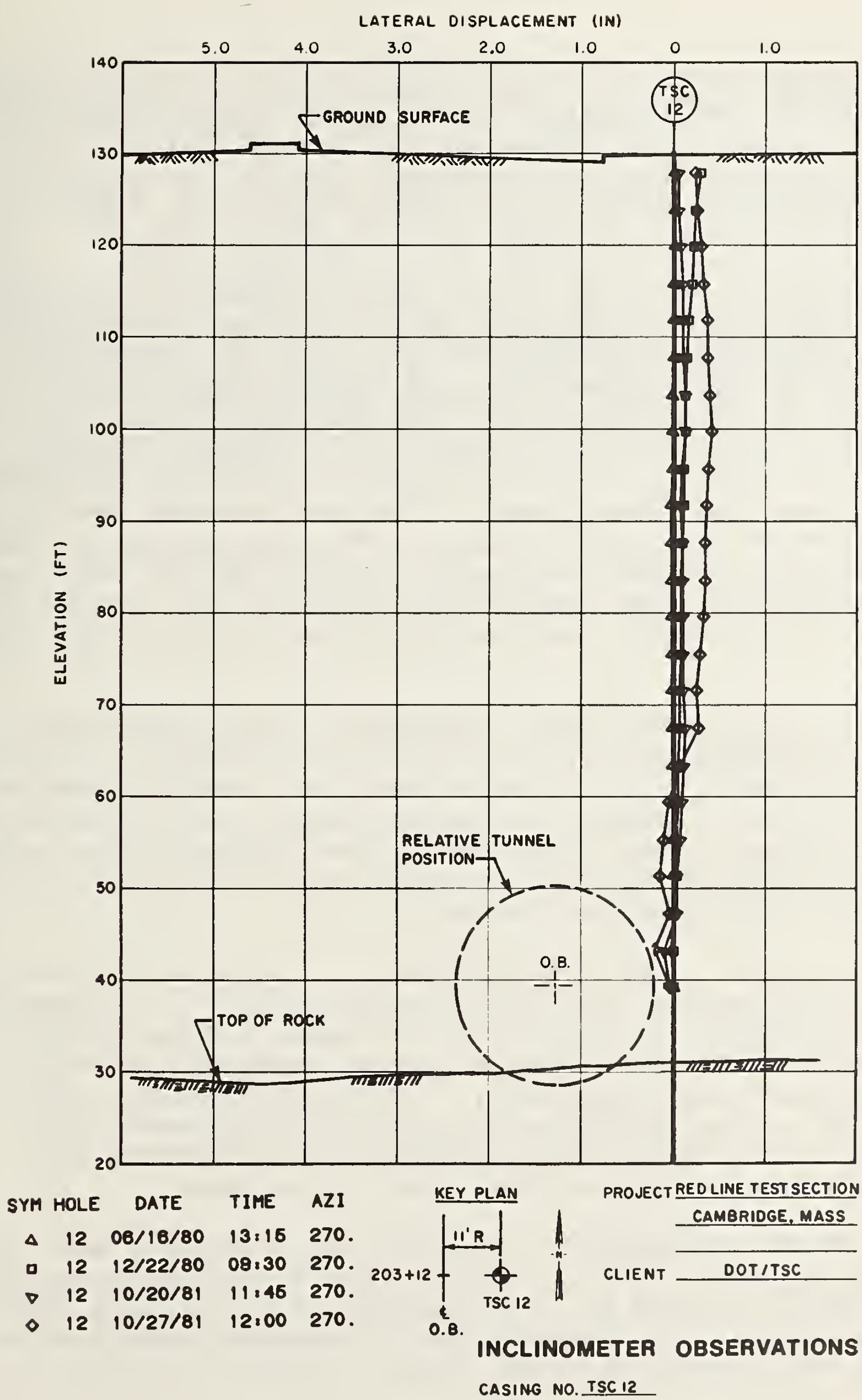


FIGURE 8-14. SELECTED INCLINOMETER OBSERVATIONS - TSC 12

Instruments TSC 16, 18, 22, and 24 were installed and read as part of the research study. Instruments T6-P and GW-11 were installed by others as part of an early tunnel design exploration program, but were included as part of this study. Instruments M-25, M-26, M-28, M-29, P-2, and P-5 were installed and read by the construction contractor.

Observation wells TSC 22 and T6-P (Appendix G) recorded no water level changes and were either clogged, improperly sealed, or had their tips located in zones of impervious soils which did not respond to construction operations. Piezometers TSC 16 and 18, near the rock tunnel part of the Test Section and with their tips in the bedrock, showed drops in water level, to about elevation 40 to 45 feet during 1980 rock tunneling operations. This corresponds roughly to the elevation of the tunnel crown. These groundwater level changes occurred before the dewatering system for soft ground tunnel construction was installed, probably as a result of natural drainage toward the tunnels through discontinuities in the rock.

In 1981, after the rock tunnels had been completed and lined, the groundwater level in TSC 18 recovered to about elevation 88, still below preconstruction levels (Section 4). TSC 16 continued to show a low water level, about elevation 30, possibly because of drainage toward the inbound pilot tunnel or the approaching inbound soft ground tunnel. On the other hand, it may simply have become clogged.

Water levels in TSC 24 (Appendix G) with its screen located in the marine clay, are representative of changes in the upper piezometric surface due to construction operations, even at depth. TSC 24 showed a steady decrease in water level throughout 1980 and 1981. It reached a water level of elevation 80 at the last reading made in September 1981. However, this was before the outbound tunnel passed and water levels here may have continued to go down.

Well GW-11 (Appendix G) was read only three times, in August and September 1981. These water level readings, of about elevation 30 and 40, are indicative of reduced piezo-etric pressures in the bedrock due to construction dewatering or natural drainage toward the nearby inbound tunnel.

Of the wells installed by the contractor, only M-25 and M-29 are located within the Test Section. They both show water levels of about elevation 40 (± 10 feet) during 1981 soft ground tunneling operations, indicative of deep pressure relief. The water level in M-25 began to rise in early

November 1981, for no apparent reason, as the outbound tunnel heading was still 33 feet away. This rise may, however, be related to details of construction dewatering operations.

In summary, these groundwater observations show that the tunneling operations reduced the piezometric heads in the upper zone of the bedrock to about elevation 40 ($\pm$ 10 feet) for long periods of time. The level of the upper piezometric surface was lowered to elevation 80 feet in one observation well (TSC 24), but was observed to be unchanged in two others (TSC 22 and T6-P). This groundwater lowering is due to both construction dewatering and natural drainage toward the unlined tunnels through the soil and discontinuities in the rock.

9. COMPARISON OF PREDICTED AND MEASURED SETTLEMENTS

9.1 GENERAL

Predictions of ground surface settlements due to tunneling through the Test Section were made as described in Section 5 and Appendix C of this report. The predictions were made in August 1980, before any major tunneling work had yet passed through the Test Section. They are based on knowledge of the subsurface conditions developed during Stage I subsurface explorations, and information available at the time, concerning tunneling methods, equipment, and workmanship. These predictions consider ground settlements caused by tunneling under normal conditions and exclude the effects of unusually large, sudden ground losses, equipment breakdowns, or significant variations from normally expected tunneling technology.

Table 9-1 compares the predicted ground surface settlements with those measured at the Test Section, Section 8.2. These values represent maximum settlements measured at the ground surface at the indicated sections. Table 9-1 shows that at all sections, the predicted settlements were slightly larger than the measured settlements. For engineering purposes, the predictions are slightly conservative. In view of uncertainties regarding subsurface conditions and details of tunneling methods, equipment, and workmanship at the time the predictions were made, the agreement between predicted and measured settlements is excellent.

TABLE 9-1. COMPARISON OF PREDICTED AND MEASURED GROUND SURFACE SETTLEMENTS

SECTION	GROUND SURFACE SETTLEMENT (FEET)	
	PREDICTED	MEASURED
North section (rock)	0.021	0.015
South section (soft ground)		
After first tunnel	0.047	0.035
After second tunnel	0.083	0.078
Middle section (mixed face)		
After first tunnel	--	0.022
After second tunnel	0.083	0.076

Detailed comparisons of the predictions with the measured settlement troughs are instructive, particularly regarding the geometry of ground losses over soft ground tunnels. These detailed comparisons are made in the following sections.

9.2 NORTH SECTION (ROCK)

Ground movements over tunnels in rock may be caused by stress relief at the tunnel opening or by ground losses due, for example, to movement of blocks of rock along joints. These movements depend on the details of construction and are very difficult to predict. However, if the stability of the tunnel heading is maintained, then the ground surface movements will likely be very small, considerably less than over soft ground tunnels.

Figure 8-8 shows that the measured surface settlements were very uniform over the north section, with a maximum of 0.015 feet over the inbound tunnel. Figure 8-1 shows that typically, these settlements developed gradually with time, perhaps partly due to groundwater lowering and subsequent consolidation of the soils above the rock. This may, however, only reflect the frequency of surveys made during this time period.

The settlement of the ground surface in the rock tunneling area was predicted to be less than 0.021 feet (0.25 inch). This shows excellent agreement with the measured settlements.

9.3 SOUTH SECTION (SOFT GROUND)

9.3.1 Single Tunnel (Inbound)

The prediction of ground surface movements was based upon a collection of case history data first published by Peck (I-6) later supplemented by Cording et al. (I-3). The procedure first requires that an estimate be made of the volume of lost ground at the tunnel, V_L(%). This estimate is usually made based upon experiences on similar projects, Table 9-2.

The volume of the settlement trough at the surface, V_S(%) is usually less than the volume of ground loss at the tunnel. This is due to volume expansion or dilation of dense granular soils over the crown of the tunnel when the tunnel is excavated. Figure 9-1 shows a relationship between V_L and V_S developed by Cording et al. (I-3).

TABLE 9-2. LOST GROUND AROUND SINGLE TUNNELS (CORDING AND HANSMIRE, I-2)

Case	Diameter, 2R, m	Depth to axis, z, m	$\frac{z}{2R}$	Lost Ground: Before Face	Lost Ground: Over Shield	Lost Ground: At Tail	Lost Ground: Lining Deflection and Time Dependent Movements	Lost Ground: Total	Comments
1a. Washington, D.C. Metro, Section A-2, 1st tunnel, C line. (Hansmire, 1975) (Hansmire and Cording, 1972)	6.4	14.6	2.3	δ = 12 mm V_L = 0.06 $\frac{m^3}{m}$ $\%V_L$ = 0.2%	250 mm 1.82 $\frac{m^3}{m}$ 5.6%	41 mm 0.29 $\frac{m^3}{m}$ 0.9%	42 mm 0.30 $\frac{m^3}{m}$ 0.9%	345 mm 2.46 $\frac{m^3}{m}$ 7.6%	Settlement point 0.45 m above crown. Expanded lining
1b. Washington, D.C. Metro, Section A-2, 2nd tunnel, C line. (Hansmire, 1975)	6.4	14.6	2.3	δ = 14 mm V_L = 0.06 $\frac{m^3}{m}$ $\%V_L$ = 0.2%	58 mm 0.43 $\frac{m^3}{m}$ 1.4%	24 mm 0.18 $\frac{m^3}{m}$ 0.6%	28 mm 0.21 $\frac{m^3}{m}$ 0.7%	124 mm 0.92 $\frac{m^3}{m}$ 2.9%	Settlement point 0.45 m above crown.
17. Washington, D.C. Metro, Section D-9, 2nd tunnel	6.4	16.1	2.5	δ = 26 mm V_L = 0.21 $\frac{m^3}{m}$ $\%V_L$ = 0.7%	51 mm 0.41 $\frac{m^3}{m}$ 1.3%	13 mm 0.11 $\frac{m^3}{m}$ 0.3%	13 mm 0.11 $\frac{m^3}{m}$ 0.3%	103 mm 0.83 $\frac{m^3}{m}$ 2.6%	Settlement point 1 m above crown
3a. Washington, D.C. Metro, Section F-2a, L Route, 1st tunnel, line 1	5.5	20.1	3.7	δ = 3 mm V_L = 0.02 $\frac{m^3}{m}$ $\%V_L$ = 0.1%	15 mm 0.15 $\frac{m^3}{m}$ 0.6%	3 mm 0.03 $\frac{m^3}{m}$ 0.1%	10 mm 0.08 $\frac{m^3}{m}$ 0.4%	30 mm 0.28 $\frac{m^3}{m}$ 1.2%	Settlement point 1.8 m above crown
3b. 1st tunnel, line 3	5.5	20.1	3.7	δ = 1 mm V_L = 0.01 $\frac{m^3}{m}$ $\%V_L$ = 0.04%	10 mm 0.09 $\frac{m^3}{m}$ 0.4%	3 mm 0.03 $\frac{m^3}{m}$ 0.1%	8 mm 0.07 $\frac{m^3}{m}$ 0.3%	23 mm 0.2 $\frac{m^3}{m}$ 0.8%	
3c. 1st tunnel, line 9	5.5	22.3	4.1	δ = 8 mm V_L = 0.07 $\frac{m^3}{m}$ $\%V_L$ = 0.3%	30 mm 0.28 $\frac{m^3}{m}$ 1.2%	5 mm 0.05 $\frac{m^3}{m}$ 0.2%	10 mm 0.08 $\frac{m^3}{m}$ 0.4%	53 mm 0.47 $\frac{m^3}{m}$ 2.0%	
3d. 2nd tunnel, line 10	5.5	21.6	3.9	δ = 3 mm V_L = 0.03 $\frac{m^3}{m}$ $\%V_L$ = 0.1%	5 mm 0.05 $\frac{m^3}{m}$ 0.2%	3 mm 0.03 $\frac{m^3}{m}$ 0.1%	10 mm 0.09 $\frac{m^3}{m}$ 0.4%	20 mm 0.2 $\frac{m^3}{m}$ 0.8%	
3e. 1st tunnel, line 11	5.5	21.9	4.0	δ = 3 mm V_L = 0.02 $\frac{m^3}{m}$ $\%V_L$ = 0.1%	10 mm 0.08 $\frac{m^3}{m}$ 0.4%	5 mm 0.05 $\frac{m^3}{m}$ 0.2%	3 mm 0.02 $\frac{m^3}{m}$ 0.1%	18 mm 0.15 $\frac{m^3}{m}$ 0.6%	
4. Frankfurt, Shield (Fahrgasse)(Chambosse, 1972, Sauer and Lama, 1973, Breth and Chambosse, 1972)	6.5	12.4	1.9	δ = 8 mm V_L = 0.08 $\frac{m^3}{m}$ $\%V_L$ = 0.2%	37 mm 0.37 $\frac{m^3}{m}$ 1.0%	25 mm 0.25 $\frac{m^3}{m}$ 0.6%	*	70 mm 0.69 $\frac{m^3}{m}$ 1.8%	Settlement point 1.7 m above crown * Final readings not included.
16. Frankfurt, no shield, Baulos 25 (Chambosse, 1972; Sauer and Lama, 1973).	6.5	14.6	2.2	δ = 33 m V_L = 0.33 $\frac{m^3}{m}$ $\%V_L$ = 0.8%	(No shield)	12 mm 0.12 $\frac{m^3}{m}$ 0.3%	13 mm 0.13 $\frac{m^3}{m}$ 0.3%	56 mm 0.55 $\frac{m^3}{m}$ 1.5%	Settlement point 1.7 m above crown
10. London Transport (Attewell and Farmer, 1974)	4.1	29.3	7.1	δ = 8 mm V_L = 0.07 $\frac{m^3}{m}$ $\%V_L$ = 0.5%	4 mm 0.04 $\frac{m^3}{m}$ 0.3%	2 mm 0.02 $\frac{m^3}{m}$ 0.15%	4 mm 0.04 $\frac{m^3}{m}$ 0.3%	18 mm 0.17 $\frac{m^3}{m}$ 1.3%	
11. Heathrow Cargo Tunnel (Muirwood and Gibb, 1971, Smyth and Osbourne, 1971)	10.9	13.3	1.2	δ = 10 mm V_L = 0.13 $\frac{m^3}{m}$ $\%V_L$ = 0.14%	-5 + 8 = 3 mm -0.07+0.12 = 0.05 $\frac{m^3}{m}$ 0.05%	1 mm 0.01 $\frac{m^3}{m}$ 0.01%	0 0	14 mm 0.19 $\frac{m^3}{m}$ 0.2%	
12. Boa Vista, Sao Paulo, (Costa, et al, 1974)	5.5	11.8	2.1	δ = 2 mm V_L = 0.03 $\frac{m^3}{m}$ $\%V_L$ = 0.1%	18 mm 0.2 $\frac{m^3}{m}$ 0.8%	20 mm 0.3 $\frac{m^3}{m}$ 1.3%	30 mm 0.4 $\frac{m^3}{m}$ 1.7%	70 mm 0.9 $\frac{m^3}{m}$ 3.8%	Settlement cross-section. 4 m above crown
14. Mexico City, Siphon II Manuel Gonzales (Tinajero and Vieitez, 1971)	2.9	11.7	4.0	δ = 40 mm V_L = 0.2 $\frac{m^3}{m}$ $\%V_L$ = 3%	30 to 80 mm, est. 0.1 to 0.4 m^3/m 2% to 6%		100 to 50 mm, est. 0.5 to 0.2 $\frac{m}{m}$ 7% to 3%	170 mm 0.8 $\frac{m^3}{m}$ 12%	Settlement point 1.2 m above crown; Total at 28 days

NOTE: δ = Vertical settlement of deep settlement point

V_L = Volume lost into tunnel

$\%V_L$ = $\frac{\text{Volume lost into tunnel}}{\text{tunnel volume}}$

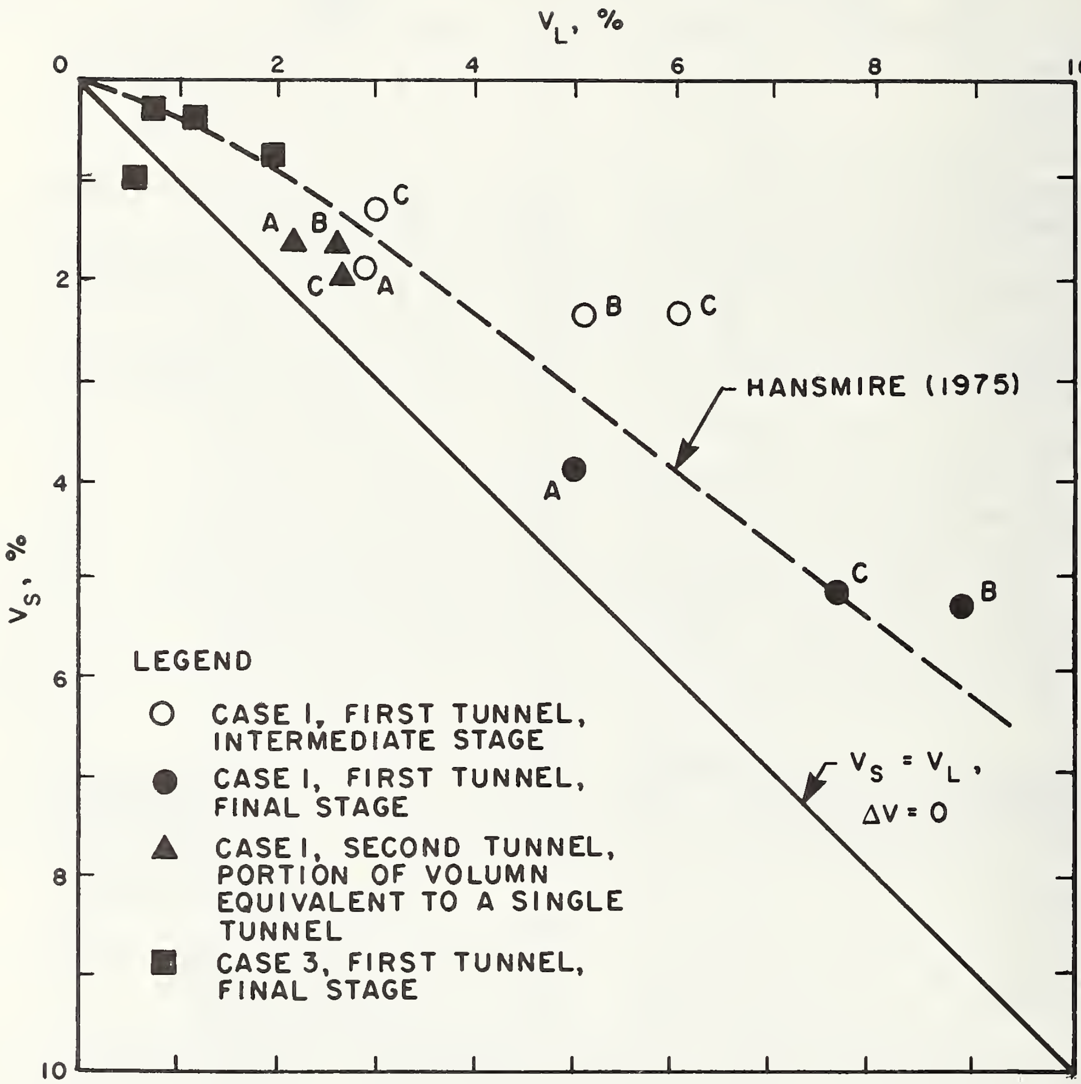


FIGURE 9-1. COMPARISON OF VOLUME OF LOST GROUND AND VOLUME OF SURFACE SETTLEMENT, SINGLE TUNNEL CASE, GRANULAR SOILS (Cording et al., I-3)

After the volume of the settlement trough has been estimated, then an assumption must be made about its geometry in order to compute maximum settlements. The assumption made by Cording et al. (I-3) in developing the relationships shown in Figure 9-2 is that the settlement trough can be represented by a normal probability curve, with width i to its point of inflection. The triangle which approximates the normal probability curve has a half-width, W, which equals 2.5 i. Then, the volume of the settlement trough, V_S, and the maximum settlement, δ_{max}, are related by:

$$V_S = W \delta_{max} = 2.5 i \delta_{max} \qquad \text{Equation 9-1}$$

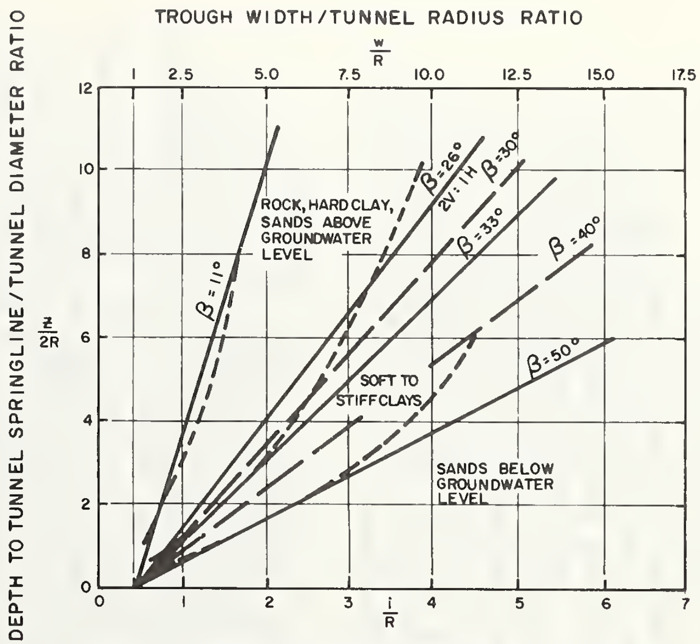
The relationship between tunnel depth and size and width of the settlement trough for various subsurface conditions (Figure 9-2a) provides an estimate of W and i. Then knowing V_S, δ_{max} can then be computed from Equation 9-1. However, Cording et al. (I-3) caution that when ground movements are small, soil deformations are elastic rather than plastic, and a settlement trough wider than would be inferred from Figure 9-2a develops, with less maximum settlement above the tunnel.

For this prediction V_L was taken as 2 percent (Table 9-2), and then Figure 9-1 gives a V_S of 0.8 percent (3.5 cu.ft./ft.). For Z/2R equal to 3.8, Figure 9-2a yields i/R equal to 1.5. However, i/R of 2.5 was used for the prediction in recognition that soil behavior would be mostly elastic. Equation 9-1 then gave a predicted δ_{max} of about 0.047 feet (0.6 inches) over the single tunnel excavated in soft ground.

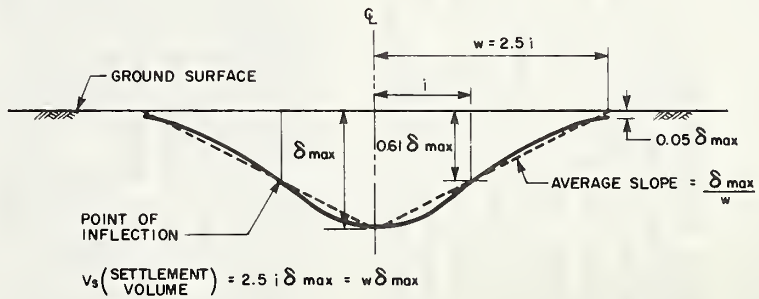
Figure 9-3 shows the settlement trough at the south section after the first (inbound) tunnel was excavated. Since the settlements were small, the data points indicate some scatter due to survey error. The settlement trough was extrapolated to the left symmetrically about the inbound tunnel centerline based upon the available data to the right. Figure 9-3 also shows the measured parameters V_S, δ_{max}, W, i/R, β, the angle of draw, and δ_{max}/W, the average slope of the settlement trough. Also shown are the idealized triangles, based upon the assumptions of i/R equals 1.5 and 2.5.

The predicted volume of 3.5 cu. ft./ft. (0.8 percent) shows excellent agreement with the measured value of 3.32 cu. ft./ft. (0.76 percent). A triangle of width W of 92.2 feet computed from Equation 9-1 provides a reasonable approximation to the shape of the settlement trough. The corresponding i/R of 3.1 is slightly greater than the value of 2.5 used for predicting settlements, accounting for most of the difference between δ_{max} of 0.036 feet, measured, and δ_{max} of 0.047 feet, predicted. Note that the V_S of 0.8 percent (predicted), with i/R of 3.1 (measured) leads to a computed δ_{max} of 0.038 feet, only 0.002 feet greater than the measured value.

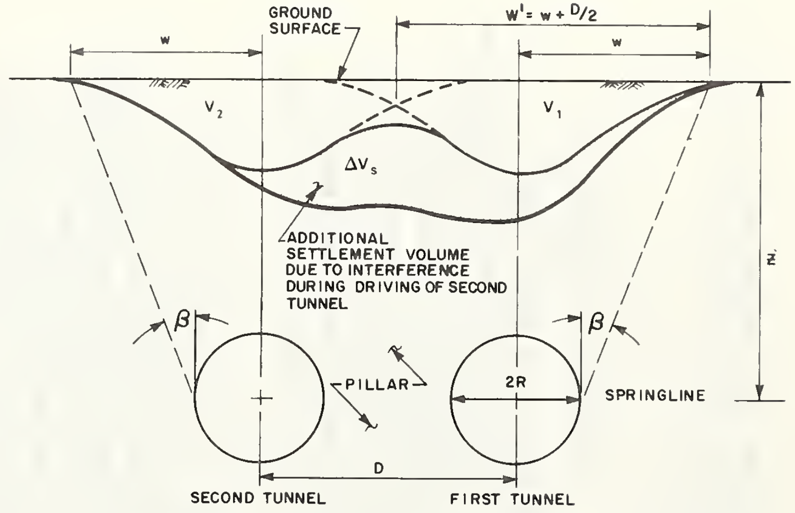
Direct application of Figure 9-2a, using i/R of 1.5, computes a settlement trough that is much narrower and a maximum settlement that is much greater than measured values.



a. RELATIONSHIP OF TUNNEL DEPTH AND SIZE TO SETTLEMENT TROUGH WIDTH FOR VARIOUS SUBSURFACE CONDITIONS



b. PROPERTIES OF IDEALIZED SETTLEMENT TROUGH (NORMAL PROBABILITY CURVE)



c. TYPICAL SETTLEMENT TROUGH OVER TWIN TUNNELS

N.T.S.

NOTES:

1. THE INFORMATION SHOWN ON THIS PLATE WAS DEVELOPED AND PRESENTED IN PUBLISHED PAPERS BY PECK (I-6), AND CORDING AND HANSMIRE (I-2).
2. DEFINITIONS OF TERMS.

D	=	CENTER TO CENTER DISTANCE BETWEEN TUNNELS
R	=	RADIUS OF TUNNEL
z	=	DEPTH BELOW GROUND SURFACE TO TUNNEL SPRINGLINE
V_s	=	SETTLEMENT VOLUME
V_1	=	SETTLEMENT VOLUME ASSOCIATED WITH FIRST TUNNEL
V_2	=	SETTLEMENT VOLUME ASSOCIATED WITH SECOND TUNNEL
ΔV_s	=	SETTLEMENT VOLUME ASSOCIATED WITH INTERFERENCE DUE TO DRIVING OF SECOND TUNNEL
δ_{max}	=	ESTIMATED SETTLEMENT AT ℄ OF SETTLEMENT TROUGH
i	=	THE HORIZONTAL DISTANCE EITHER SIDE OF THE TUNNEL CENTERLINE WHICH DEFINES THE POINTS OF INFLECTION OF AN IDEALIZED SETTLEMENT TROUGH (NORMAL PROBABILITY CURVE).
W	=	WIDTH OF SETTLEMENT TROUGH FOR A SINGLE TUNNEL WHERE W = 2.5 i
W'	=	WIDTH OF SETTLEMENT TROUGH FOR TWIN TUNNELS WHERE W' = W + D/2
β	=	THE VERTICAL ANGLE DRAWN FROM THE PERIMETER OF THE TUNNEL AT THE SPRINGLINE TO THE DEFINED WIDTH, W, OF THE SETTLEMENT TROUGH AT THE SURFACE.

FIGURE 9-2. RELATIONSHIPS FOR ESTIMATING SETTLEMENTS OVER SOFT GROUND TUNNELS (PECK, I-6)

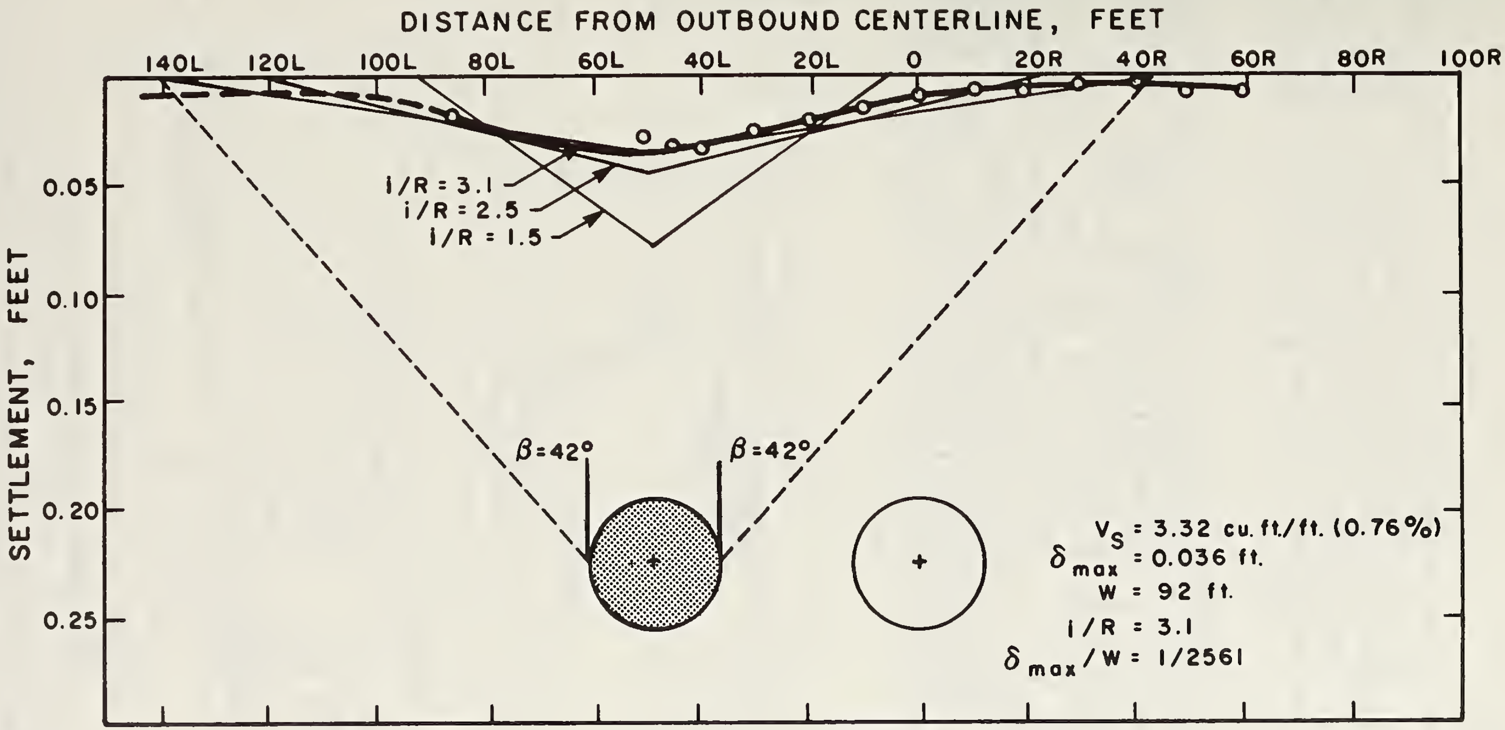


FIGURE 9-3. SURFACE SETTLEMENT AFTER INBOUND TUNNEL EXCAVATION - SOUTH SECTION

9.3.2 Twin Tunnels

For twin tunnels, an additional ground loss is caused by the interference of one tunnel with the other. This interference may cause an additional deflection of the lining of the first tunnel, compression of the pillar between the two tunnels, and a volume decrease in the previously expanded region over the first tunnel.

The resulting interference volume, ΔV_S, is portrayed on Figure 9-4, with some field data relating the interference volume to pillar width at the Washington, DC Metro, Cording et al. (I-3). The interference volume at the south section was estimated to be about 0.4 percent ($\Delta V_S/V_2$ = 0.5) based on these data. (This estimate used a pillar width of 29 feet. The actual pillar width here of about 24 feet yields a slightly larger interference volume). When distributed over the twin tunnels, as shown in Figure 9.2c (i/R = 2.5), the total volume losses give a predicted δ_{max} of 0.083 feet (about 1 inch).

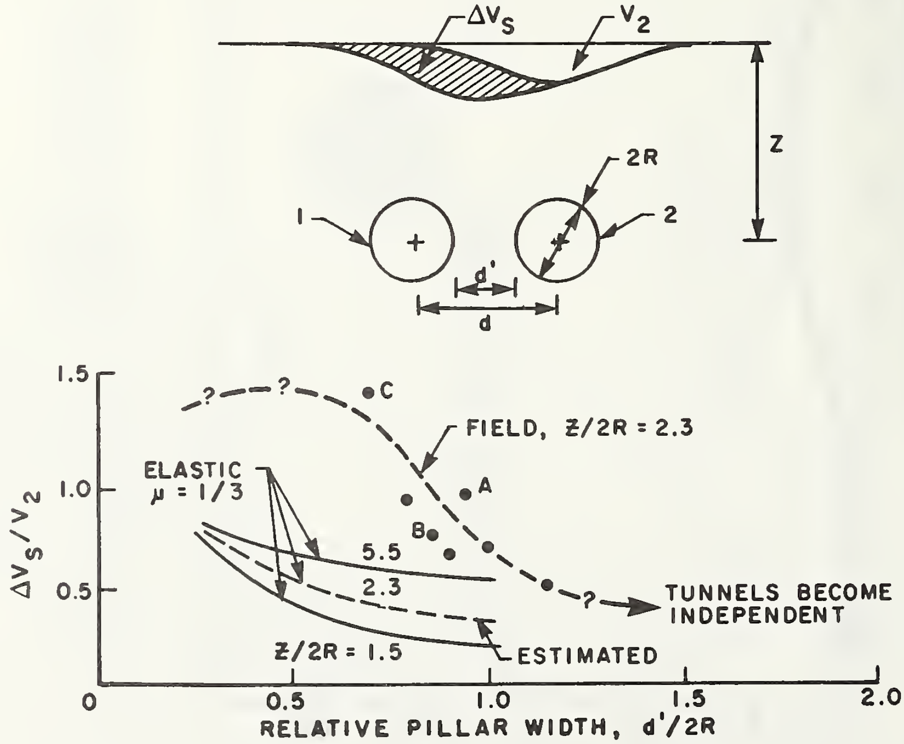


FIGURE 9-4. PILLAR WIDTH AND VOLUME OF SETTLEMENTS AT GROUND SURFACE FOR CONSTRUCTION OF A SECOND TUNNEL (Cording et al., I-3)

Figure 9-5 shows the trough of incremental surface settlements after the second (outbound) tunnel was excavated at the south section. This trough is non-symmetrical and shifted toward the first tunnel, consistent with the idealization of Figure 9-4. The surface settlements that would occur if the second (outbound) tunnel were excavated as a single tunnel, with no interference effects, were estimated from the settlement trough for the first (inbound) tunnel, Figure 9-3. This settlement trough, V_2, centered over the second (outbound) tunnel, is superimposed on the data of Figure 9-5. The difference between the measured incremental settlement volume, and V_2, Figure 9-5, is the interference volume, ΔV_S. The measured ΔV_S of 1.74 cu. ft./ft. (0.4 percent) is identical to the 1980 prediction, Appendix C.

Figure 9-6 shows the total surface settlements at the south section after both the inbound and outbound tunnels were excavated. The settlement volumes corresponding to single tunnel excavations ($V_1 = V_2$), and the interference volume (ΔV_S), δ_{max}, W', W, i/R, and δ_{max}/W' are also shown.

The predicted total settlement volume, V_S, of 2.0 percent shows excellent agreement with the measured value of 1.92 percent (8.38 cu. ft./ ft.). The predicted settlement of 0.083 feet is slightly larger than the measured value of 0.78 feet. This difference occurs mostly because the measured settlement trough is wider (i/R = 2.8) than was assumed for the prediction (i/R = 2.5).

This is shown more clearly on Figure 9-7, which compares the idealized triangles based on the relationships of Figure 9-2 with the measured data. The triangle with i/R of 1.5, corresponding to a direct application of the Cording et al. (I-3) relationships, Figure 9-2a, computes a settlement trough that is much narrower, and a maximum settlement that is much greater than measured values. The predicted settlement trough, i/R = 2.5 in recognition that soil deformations will be small and the settlement triangle wide, much more closely approximates the measured data. Note that the V_S of 2.0 percent (predicted), with i/R of 2.8 (measured) leads to a computed δ_{max} of 0.081 foot, only 0.003 feet greater than the measured value.

Finally, the very flat bottom of the settlement trough for the twin tunnels, Figure 9-6, compared to the settlement trough for the single tunnel, Figure 9-3, suggests that a trapezoid may be a more appropriate approximation than a triangle. Figure 9-8 compares the triangle with i/R of 2.8, and a trapezoid with uniform maximum settlement between the tunnel centerlines, with the measured settlements. Both shapes correspond to the measured δ_{max} of 0.078 feet and

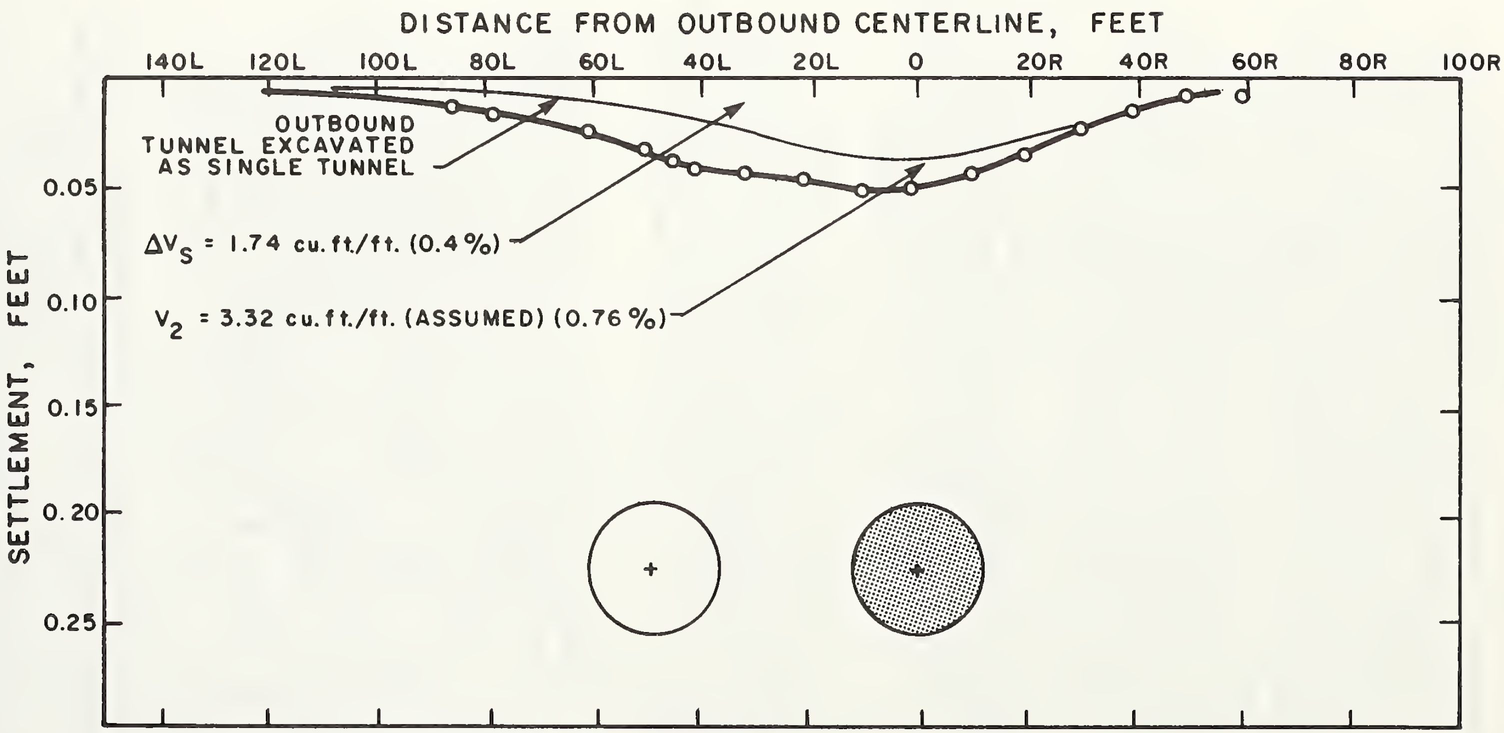


FIGURE 9-5. INCREMENTAL SURFACE SETTLEMENT AFTER OUTBOUND TUNNEL EXCAVATION - SOUTH SECTION

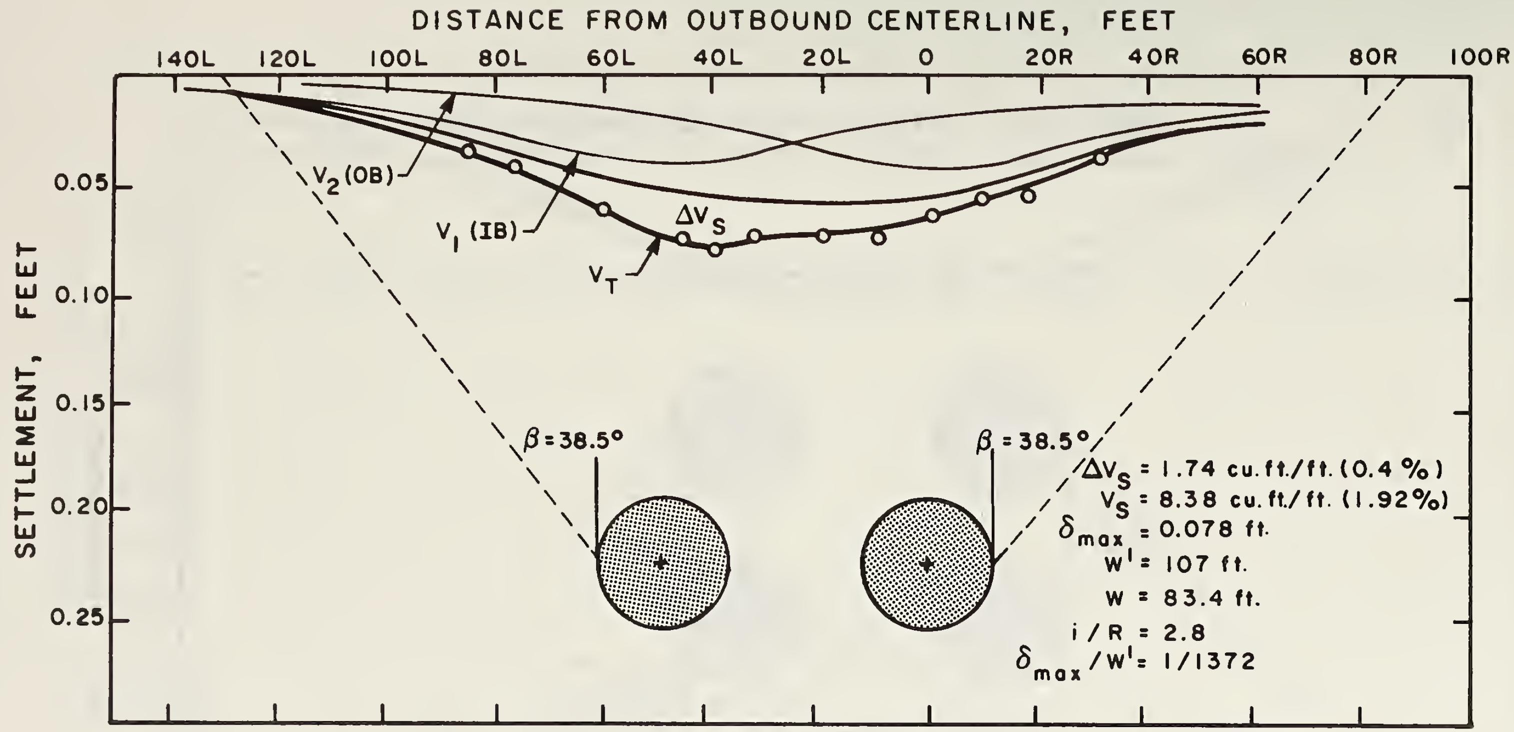


FIGURE 9-6. TOTAL SURFACE SETTLEMENT AFTER INBOUND AND OUTBOUND TUNNEL EXCAVATIONS - SOUTH SECTION

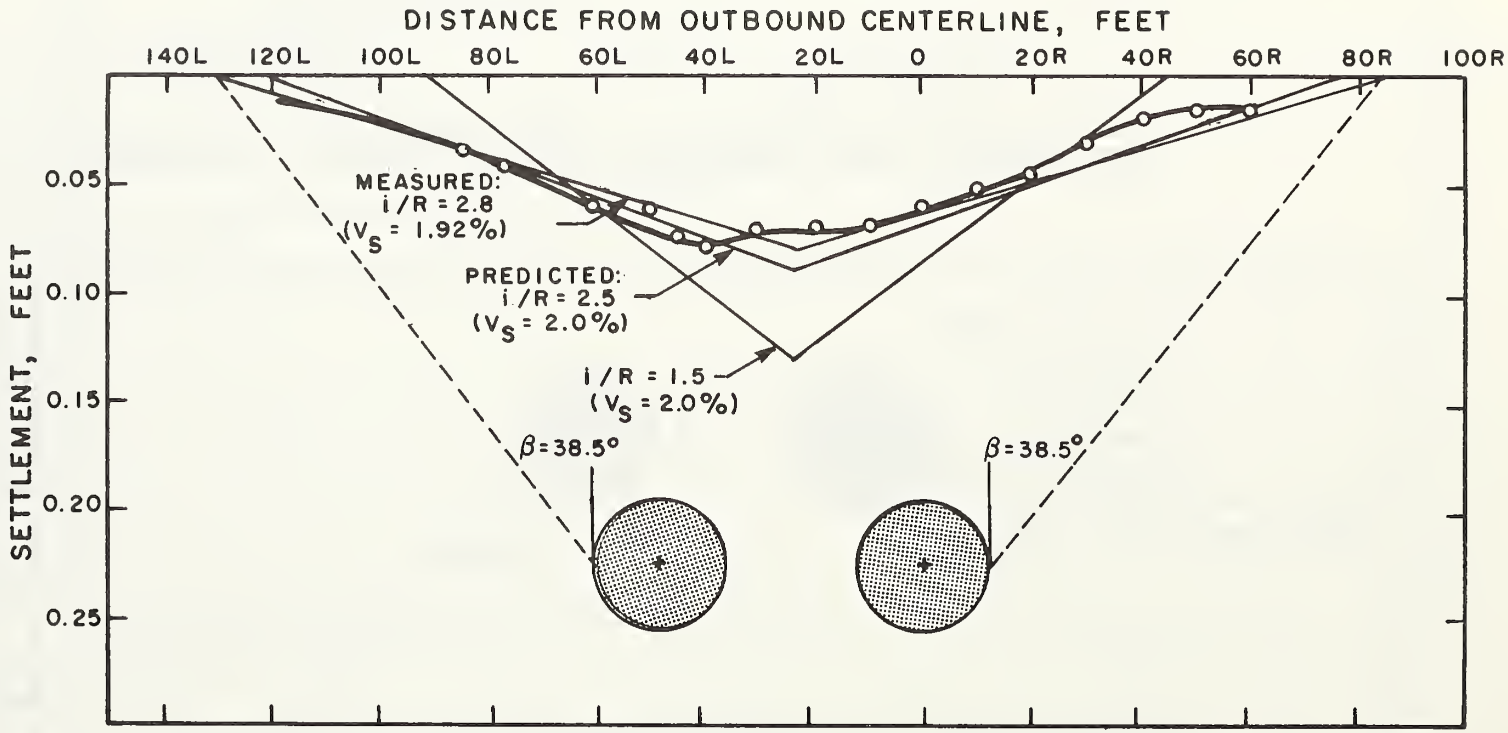


FIGURE 9-7. COMPARISON OF TOTAL SURFACE SETTLEMENT TROUGH WITH TRIANGLES OF DIFFERENT i/R VALUES

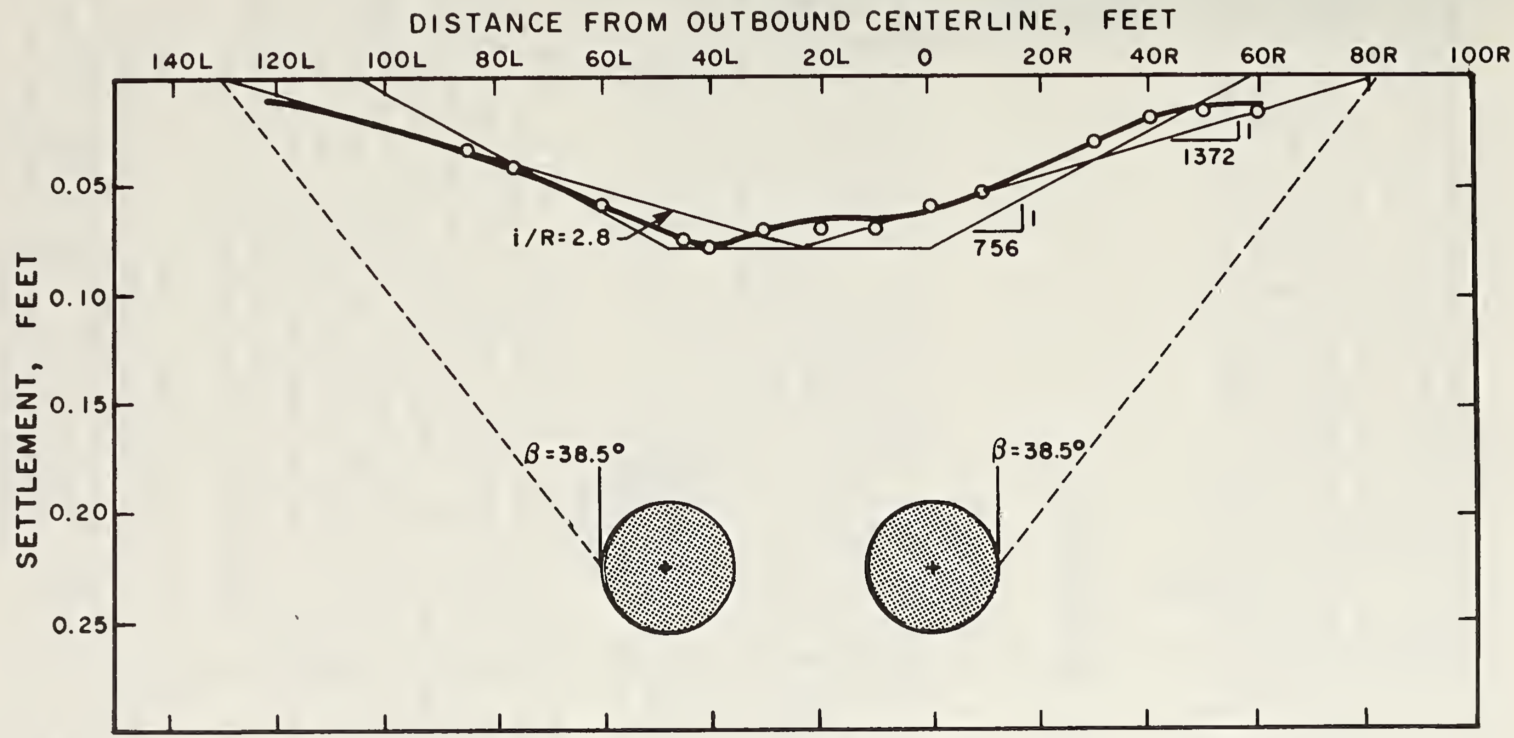


FIGURE 9-8. COMPARISON OF TOTAL SURFACE SETTLEMENT TROUGH WITH TRIANGULAR AND TRAPEZOIDAL APPROXIMATIONS

settlement volume of 8.3 cu. ft./ft. (1.92 percent). Figure 9-8 shows that both shapes reasonably approximate the measured settlements. The trapezoid is narrower than the triangle, but has a much steeper slope, 1/756 vs. 1/1372. Thus, the trapezoid may be a reasonably conservative representation for estimating the effects of ground surface movements on structures near the centerlines of the tunnels, in this case within about 40 feet. For estimating the effects on structures farther away, the trapezoid may have an excessively steep slope and the triangle may be more appropriate.

9.4 MIDDLE SECTION (MIXED FACE)

9.4.1 Single Tunnel (Inbound)

Prediction of ground loss, and resulting settlement in mixed face tunnels is extremely difficult. Compared to soft ground tunnels, the degree of disturbance is likely to be greater due to the more difficult construction environment. The magnitude of ground losses is highly dependent on construction methods used in tunneling and the speed and effectiveness with which the soil or rock is supported. The magnitude of ground settlement over the centerline of the mixed face tunnels in the Test Section was predicted to be slightly greater than the settlement predicted for soft ground tunnels (0.083 feet).

Figure 9-9 shows the surface settlements after the first (inbound) tunnel had passed the middle section. Here, all but a small part of the inbound tunnel crown was in rock and the tunnel was excavated as a full face using rock tunneling drill-and-blast procedures. The measured δ_{max} of 0.022 feet is slightly greater than the δ_{max} of 0.015 feet measured at the north section in rock after both tunnels had passed. Compared to the settlement trough for the north section, Figure 8-8, the settlements here are more concentrated over the tunnel centerline. These comparisons indicate greater ground losses at the tunnel crown, as compared to the tunnels in rock.

The δ_{max} of 0.022 feet and volume loss of 1.53 cu. ft./ft. (0.39 percent) for the single mixed face tunnel, Figure 9-9, are about half the corresponding values for the single tunnel in soft ground, Figure 9-3. This is indicative of reduced compression in the rock outside of the springline of the tunnel, relative to the tunnel in soft ground.

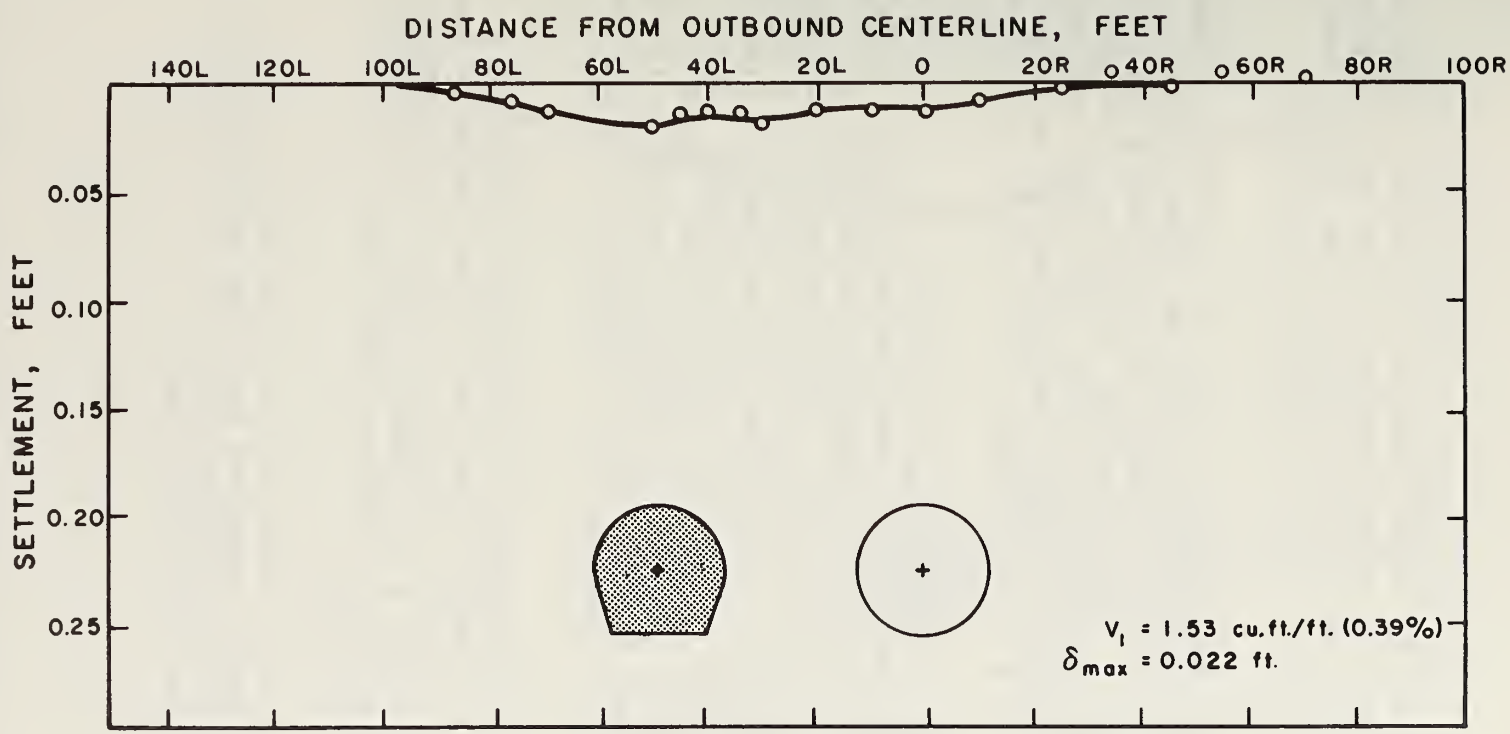


FIGURE 9-9. SURFACE SETTLEMENT AFTER INBOUND TUNNEL EXCAVATION - MIDDLE SECTION

9.4.2 Twin Tunnels

Figure 9-10 shows the incremental surface settlement after the second (outbound) tunnel passed the middle section. This tunnel had more of its cross section in the soft ground and except for a 10-foot by 10-foot invert pilot drift, was excavated using soft ground shield tunneling procedures, Figure 8-10.

The settlement trough, Figure 9-10, is remarkably symmetrical about the outbound tunnel centerline, as shown by the asterisks, which are the mirror images of settlement data from the right of the tunnel. This symmetry indicates that the interference volume is negligible. Thus, additional compression of the lining of the first (inbound) tunnel mostly in rock, and compression of the rock pillar between the tunnels must have been small.

On the other hand, the settlement volume, V_S, of 5.58 cu. ft./ft. (1.28 percent) is considerably larger than the settlement volume for the single tunnel in soil, 3.32 cu. ft./ft. (0.76 percent), Figure 9-3. This may be reflective of the more difficult construction environment, some major ground losses, and a crown failure, described in Section 7. Also, the till overlying the second (outbound) tunnel may have been somewhat disturbed by the excavation of the first (inbound) tunnel, even though there was no interference volume evident here. This would reduce the ability of the till to expand and arch over the outbound tunnel opening, resulting in greater ground losses.

The geometry of the settlement trough for the mixed face outbound tunnel is almost identical to the geometry for a single tunnel in soft ground. This is shown by the comparison of the measured parameters $W = 92$ feet, $i/R = 3.1$, and $\beta = 40$ degrees, Figure 9-10; with the corresponding values for the soft ground tunnel, Figure 9-3.

The maximum total settlement due to excavation of the twin tunnels is 0.076 feet, measured over the springline of the outbound tunnel. This compares well with the predicted value of slightly greater than 0.083 feet.

9.5 SUMMARY

Comparison of the predicted settlements with the settlements measured at the Test Section, Table 9-1, shows that the predicted settlements were slightly larger than the measured settlements. In view of uncertainties regarding subsurface conditions and

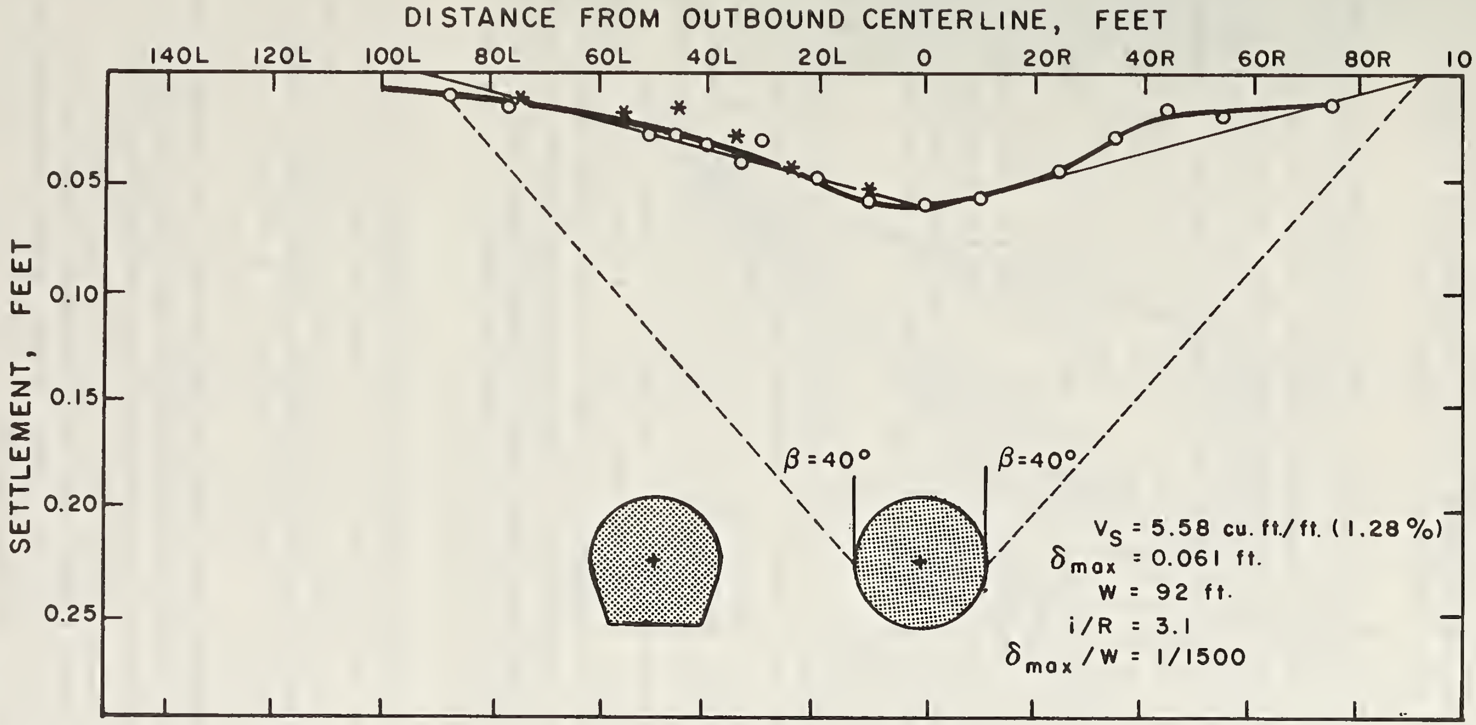


FIGURE 9-10. INCREMENTAL SURFACE SETTLEMENT AFTER OUTBOUND TUNNEL EXCAVATION - MIDDLE SECTION

details of tunneling methods, equipment, and workmanship at the time the predictions were made, the agreement between predicted and measured settlements is excellent.

Detailed comparison of the predicted settlement volumes in soft ground, V_S = 0.8 percent for single tunnels, ΔV_S = 0.4 percent for interference between twin tunnels, with the measured settlement volumes indicates excellent agreement. However, the settlement troughs were much wider than would be inferred from the relationships between tunnel depth and size to settlement trough width for various subsurface conditions published by Peck (I-6) and Cording et al. (I-3), Figure 9-2a. Figure 9-11 plots the i/R values measured at the Test Section on these relationships. The three plotted points represent the south section (soft ground), single and twin tunnels, and the middle section (mixed face) outbound tunnel treated as a single tunnel. The three points are very consistent with one another, showing i/R values from 2.8 to 3.1 and β values from 38.5 to 42 degrees.

Although these tunnels were excavated in the glacial till above the groundwater level (dewatered), and the soil profile is predominantly granular, the measured values plot in the region for tunnels excavated in soft to stiff clays, Figure 9-11. This may be because for small settlements such as these, the soil deformations are elastic rather than plastic and hence a wider trough develops. Also, the response of the soils above the alignment to the disturbance caused by the excavation and the groundwater lowering may have effected the measured values. Similar observation was made by Cording et al. (I-3) in their studies of the Washington, DC, Metro Section F2a, also plotted on Figure 9-11.

In anticipation of this effect, the settlement predictions were based on an i/R of 2.5. Even using this value of i/R, the width of the settlement troughs were slightly underpredicted, resulting in predicted settlements that were higher than measured settlements.

The surface settlements at the middle section show some interesting effects of mixed face conditions. The first tunnel (inbound) was mostly in rock and was excavated full faced using rock tunneling drill and blast procedures. It caused slightly greater surface settlements than were observed at the north section in rock, indicative of greater ground losses due to soil disturbance.

The second (outbound) tunnel had more of its cross section in the soft ground and except for a 10-foot by 10-foot invert pilot drift, was excavated using soft ground shield

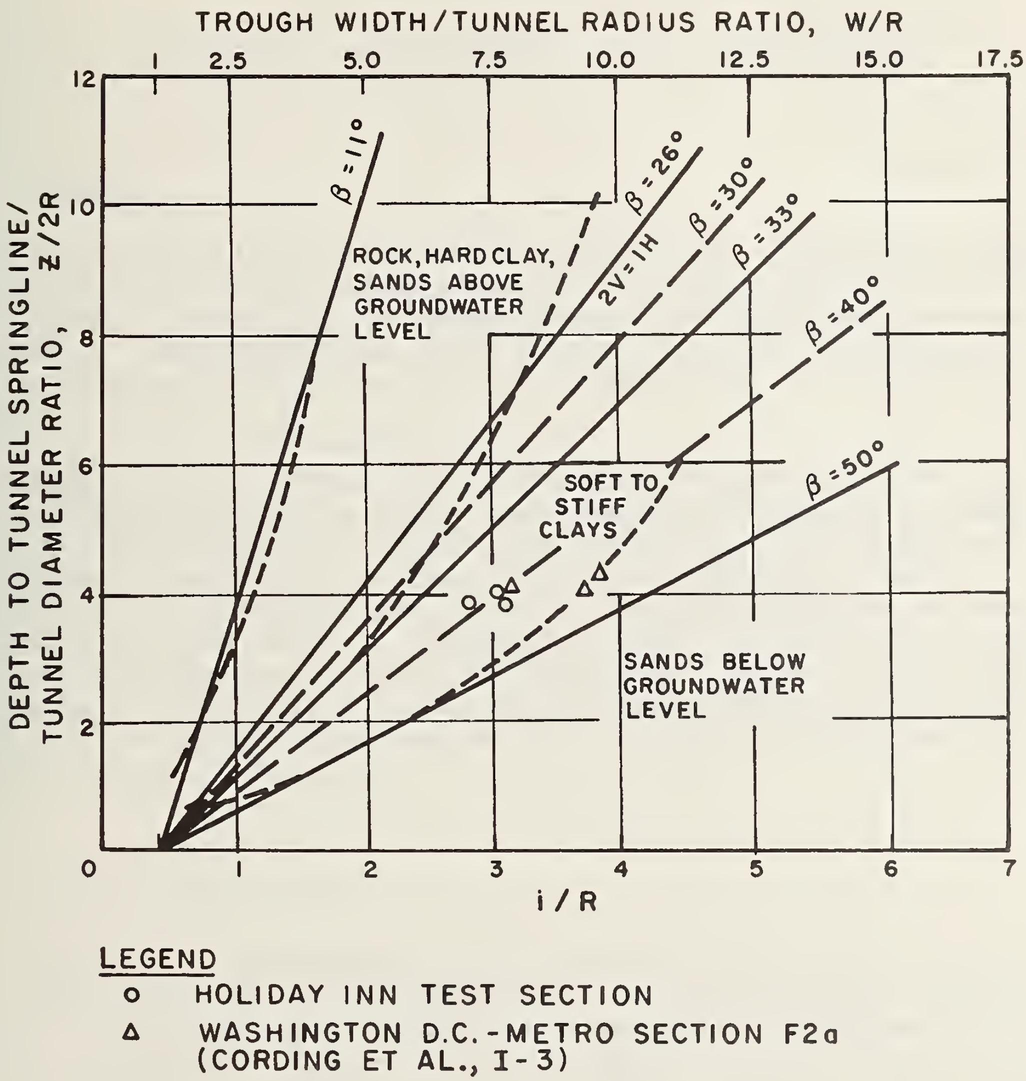


FIGURE 9-11. WIDTH OF SETTLEMENT TROUGHS, TEST SECTION

tunneling procedures, Figure 8-10. It caused a settlement trough here with a greater volume (V_S = 1.28 percent) than was measured for the single tunnel in soft ground (V_S = 0.76 percent), Figure 9-3. This may be reflective of the more difficult construction environment under mixed face conditions. However, the settlement trough in the middle section (mixed face) caused by excavation of the second (outbound) tunnel was entirely symmetrical about the tunnel. This indicates that there was little, if any, interference between the first (inbound) and second (outbound) tunnels at this section. The geometry of the settlement trough caused by the second (outbound) tunnel, as measured by i/R and β, was almost identical for the single tunnel in soft ground, Figure 9-3.

10. INSTRUMENT PERFORMANCE, COST, AND BENEFIT

10.1 GENERAL

The complicated subsurface conditions, different construction procedures, and small measured ground movements at the Test Section make direct comparisons of instrument performance difficult. Nevertheless, this study acquired significant data on the accuracy, costs, and benefits of the instruments used.

Section 10.2 summarizes the costs of installation of the deep settlement casings, inclinometer casings, piezometers and observation wells. These cost data have already been discussed in detail in Section 10 of the Stage I report. Sections 10.3 through 10.6 discuss the performance of the Test Section instrumentation, including their accuracy, monitoring and data processing costs, and benefits. Section 10.7 presents a summary of the performance of the instruments at the Test Section, including their value for the engineering and construction of transit tunnels.

10.2 INSTALLATION COSTS

Table 10-1 summarizes the costs of making the boreholes and installing the instrumentation casings for the deep settlement points, inclinometers, observation wells and piezometers at the Test Section. These costs are based upon the cost analysis and unit prices presented in the Stage I report, increased by 10 percent per year, to reflect 1982 costs.

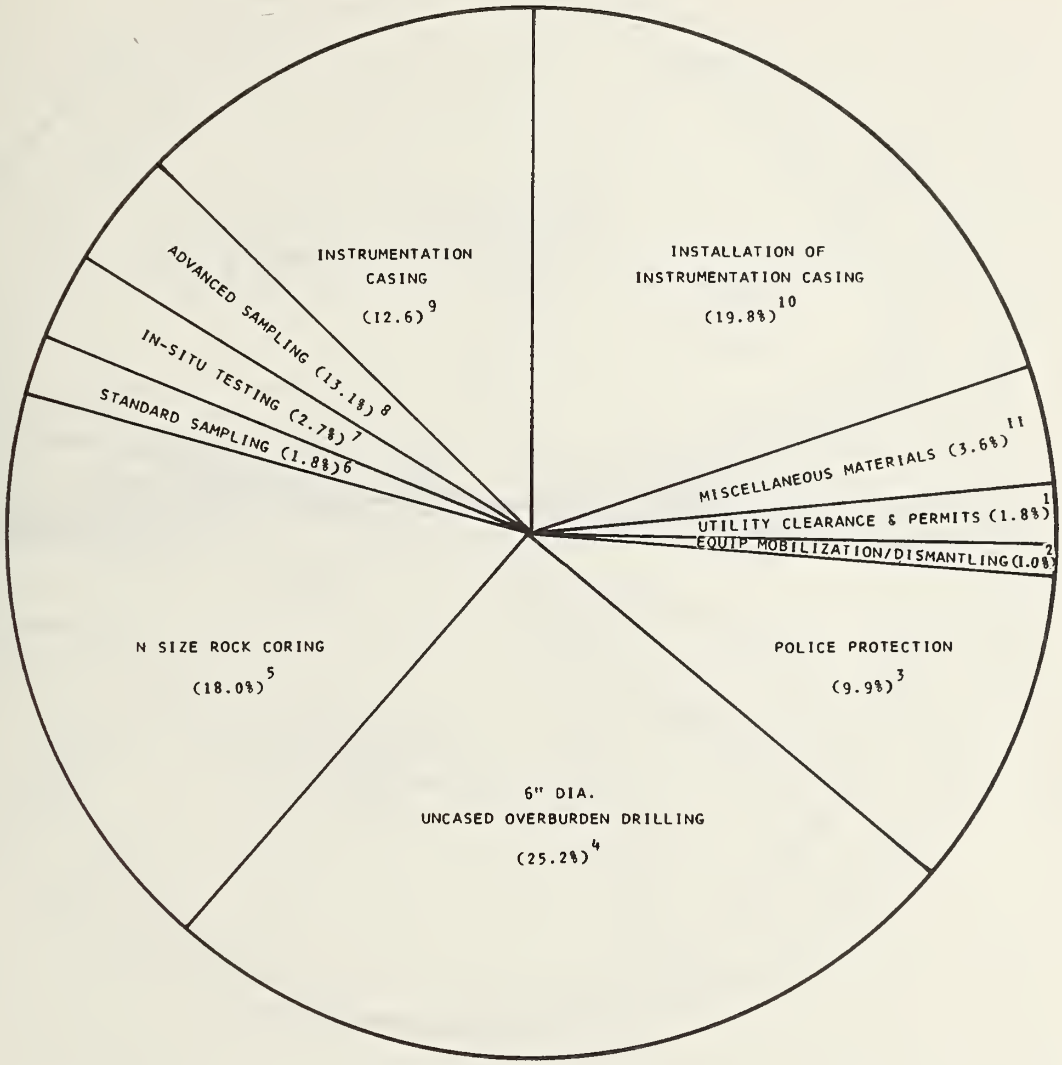
The distribution of borehole costs for each type of instrumentation is summarized graphically on Figures 10-1, 10-2, 10-3, and 10-4. All four types of boreholes involved the expenditure of 11.9 percent (Figure 10.2) to 20.9 percent (Figure 10-4) of the costs toward technically non-productive items, such as utility clearances, permits, insurance, bonds, equipment mobilization and dismantling, and the services of police officers for traffic control. Actual drilling costs account for 29.4 percent (Figure 10-4) to 43.2 percent (Figure 10-1) of the total with sampling costs ranging from zero percent (Figures 10-3 and 10-4) to 14.9 percent (Figure 10-1). The supply and installation of instrumentation and/or geophysical casing accounts for 28.3 percent (Figure 10-3) to 42.7 percent (Figure 10-2) of the total borehole costs.

TABLE 10-1. SUMMARY OF STUDY BOREHOLE COSTS

INSTRUMENT DESCRIPTION	BORING DESCRIPTION	DEPTH - FEET			COST PER BOREHOLE		COST PER FOOT
		SOIL	ROCK	TOTAL	TOTAL(1)	AVERAGE(1)	AVERAGE(1)
ELECTRICAL SETTLEMENT / ABS INCLINOMETER	6 IN. DIAMETER BORING WITH DETAILED SOIL AND ROCK SAMPLING	75 95	40 5	115 100	\$4870 \$4519	\$4702	\$43.74
TELESCOPING INCLINOMETER/SETTLEMENT	6 IN. DIAMETER BORING WITH STANDARD SOIL AND ROCK SAMPLING	75 100	40 5	115 105	\$5733 \$4652	\$5090(2)	\$42.27(2)
PIEZOMETERS AND OBSERVATION WELLS	4 IN. DIAMETER DEEP BORING WITHOUT SAMPLING	80 100	30 5	110 105	\$3342 \$2510	\$2778	\$25.83
COMBINATION SETTLEMENT	5 IN. DIAMETER SHALLOW BORING WITHOUT SAMPLING	75 70	0 0	75 70	\$2895 \$2304	\$2529	\$34.87

NOTES:

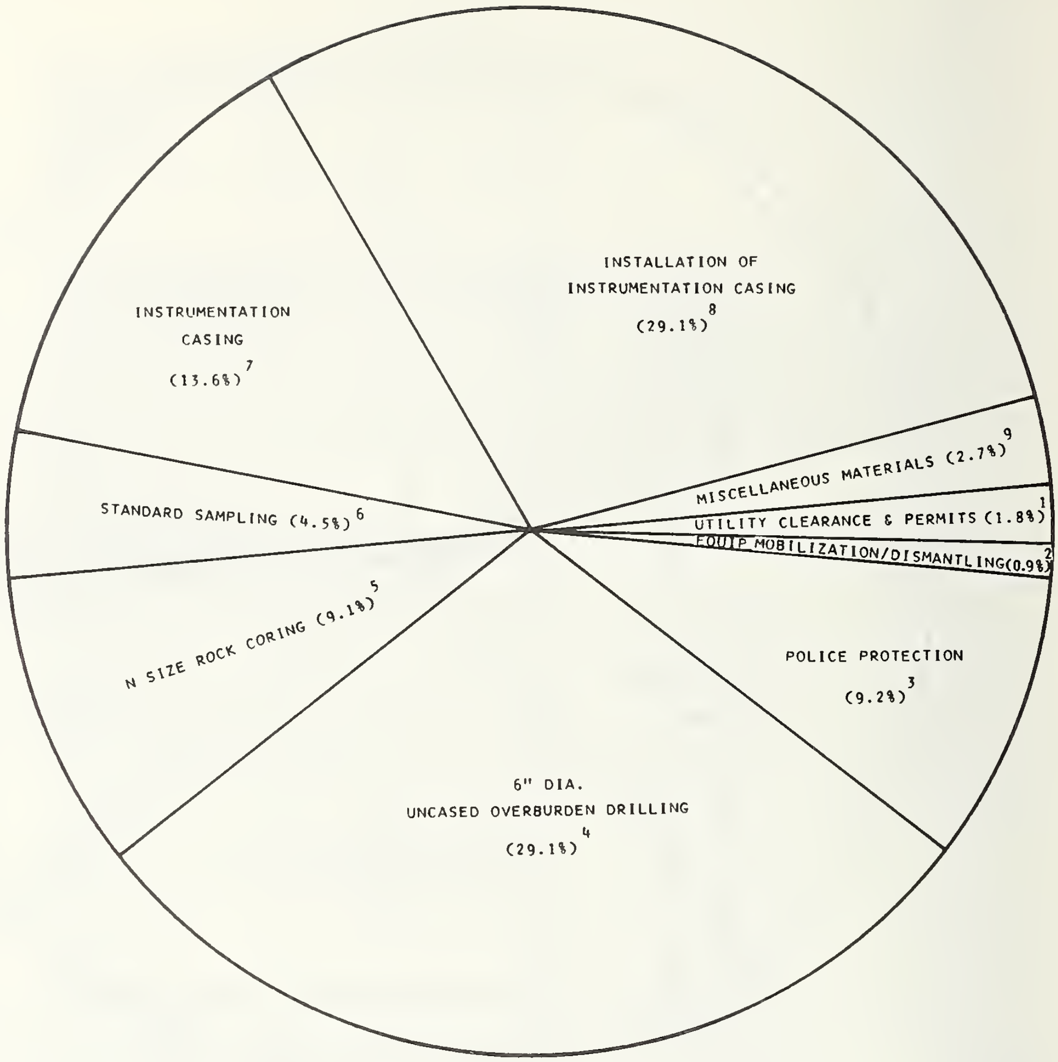
1. TOTAL AND AVERAGE BOREHOLE COSTS INCLUDE ALL DRILLING, SAMPLING, TESTING, CASING AND PRO-RATED MOBILIZATION AND DISMANTLING, UTILITY CLEARANCE, PERMITS, INSURANCE AND BONDS, POLICE PROTECTION AND INSTRUMENTATION CASING.
2. BORINGS FOR TELESCOPING INCLINOMETER/SETTLEMENT CASINGS WERE COMPLETED NEAR THE CENTER OF MASSACHUSETTS AVENUE AND INCLUDED MORE STAND-BY OR DELAY TIME ASSOCIATED WITH TRAFFIC RELOCATION AND INSTRUMENTATION INSTALLATION.
3. AN INCREASE OF 10 PERCENT PER YEAR WAS USED TO CONVERT 1979 COSTS TO 1982 COSTS.



NOTES:

1. PRO-RATED UTILITY CLEARANCES, PERMIT COSTS, INSURANCE, BONDS, ETC.
2. PRO-RATED EQUIPMENT MOBILIZATION AND DISMANTLING.
3. SERVICES OF POLICE OFFICERS FOR TRAFFIC CONTROL.
4. SIX INCH DIAMETER, UNCASED, MUD STABILIZED OVERBURDEN DRILLING.
5. NWM AND NWD DOUBLE TUBE, SOLID AND SPLIT INNER LINER ROCK CORING.
6. SPLIT SPOON SAMPLING (TWO AND THREE INCH OD).
7. BOREHOLE PERMEABILITY TEST AND WATER PRESSURE TESTS.
8. DENISON SAMPLING, PITCHER SAMPLING AND NWM OVERBURDEN CORE SAMPLING.
9. ELECTRICAL SETTLEMENT CASING IN COMBINATION WITH INCLINOMETER CASING AND ASSOCIATED MATERIALS.
10. EQUIPMENT AND CREW TIME ASSOCIATED WITH INSTALLATION OF INSTRUMENTATION CASING.
11. CEMENT, BENTONITE, OTTAWA SAND, COMMON SAND, GRAVEL, ROADWAY BOXES, WATERPROOF SEALANT, ETC.
12. SEE TABLE 10-1 FOR DEFINITION OF EXPLORATION TYPES.

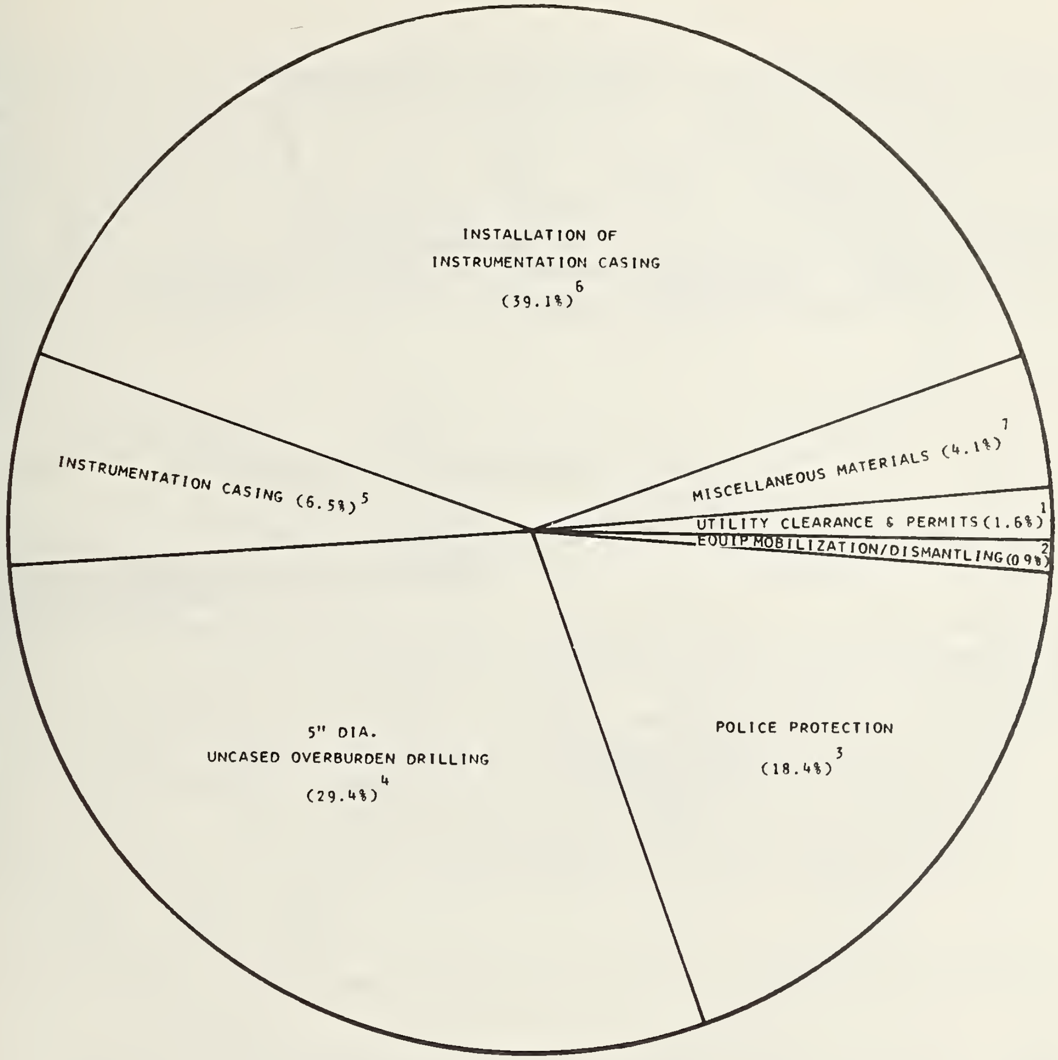
FIGURE 10-1. COST DISTRIBUTION - ELECTRICAL DEEP SETTLEMENT CASING/ABS INCLINOMETER CASING



NOTES:

1. PRO-RATED UTILITY CLEARANCES, PERMIT COSTS, INSURANCE, BONDS, ETC.
2. PRO-RATED EQUIPMENT MOBILIZATION AND DISMANTLING.
3. SERVICES OF POLICE OFFICERS FOR TRAFFIC CONTROL.
4. SIX INCH DIAMETER, UNCASED, MUD STABILIZED, OVERBURDEN DRILLING.
5. NWM AND NWD DOUBLE TUBE, SOLID AND SPLIT INNER LINER ROCK CORING.
6. SPLIT SPOON SAMPLING (TWO INCH OD).
7. TELESCOPING INCLINOMETER/SETTLEMENT CASING.
8. EQUIPMENT AND CREW TIME ASSOCIATED WITH INSTALLATION OF INSTRUMENTATION CASING.
9. CEMENT, BENTONITE, COMMON SAND, GRAVEL, ROADWAY BOXES, WATERPROOF SEALANT, ETC.
10. SEE TABLE 10-1 FOR DEFINITION OF EXPLORATION TYPES.

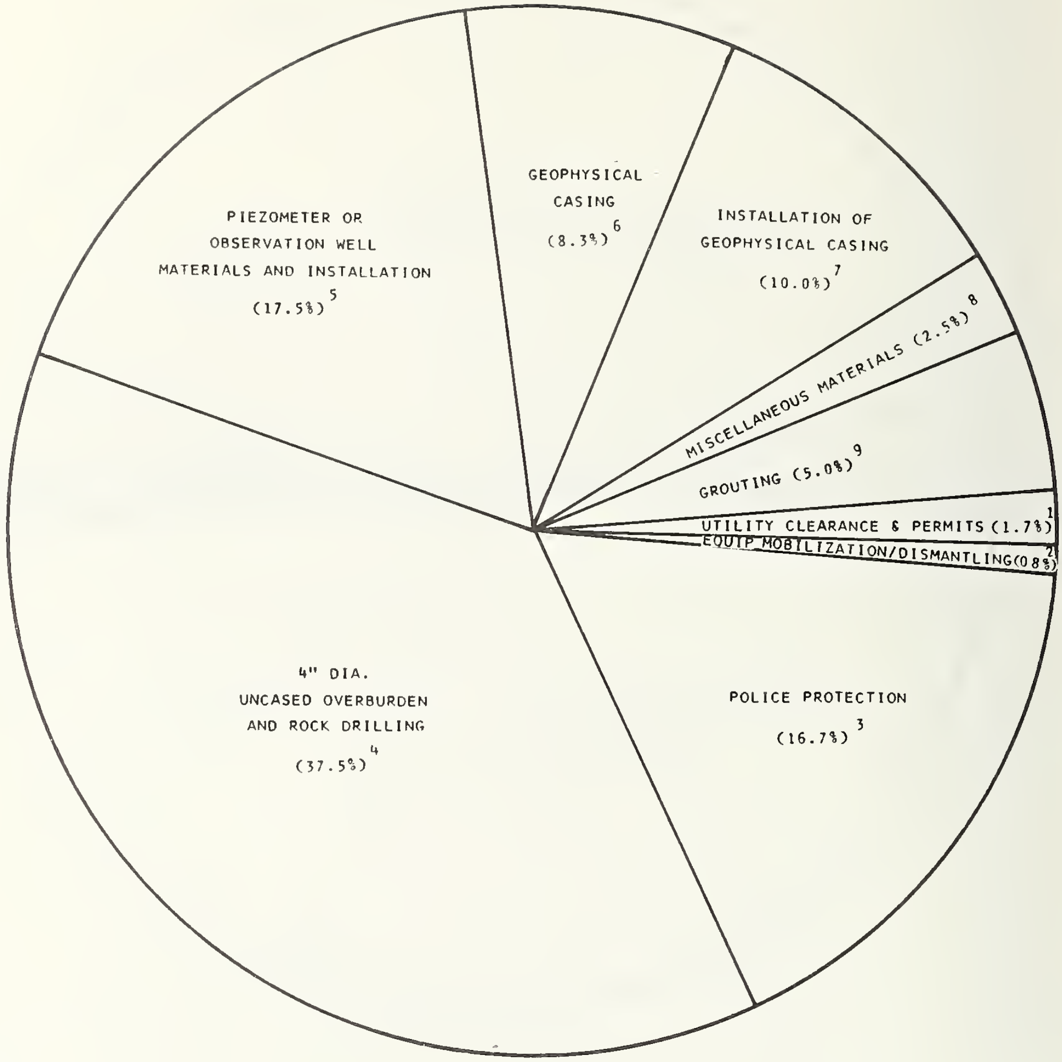
FIGURE 10-2. COST DISTRIBUTION - TELESCOPING INCLINOMETER/SETTLEMENT CASING



NOTES:

1. PRO-RATED UTILITY CLEARANCES, PERMIT COSTS, INSURANCE, BONDS, ETC.
2. PRO-RATED EQUIPMENT MOBILIZATION AND DISMANTLING.
3. SERVICES OF POLICE OFFICERS FOR TRAFFIC CONTROL.
4. FIVE INCH DIAMETER UNSAMPLED, UNCASED, MUD STABILIZED, OVERBURDEN DRILLING.
5. PVC MECHANICAL/ELECTRICAL SETTLEMENT CASINGS AND ASSOCIATED MATERIALS.
6. EQUIPMENT AND CREW TIME ASSOCIATED WITH INSTALLATION OF INSTRUMENTATION CASING.
7. CEMENT, BENTONITE, COMMON SAND, GRAVEL, ROADWAY BOXES, WATERPROOF SEALANT, ETC.
8. SEE TABLE 10-1 FOR DEFINITION OF EXPLORATION TYPES.

FIGURE 10-3. COST DISTRIBUTION - COMBINATION SETTLEMENT CASING



NOTES:

1. PRO-RATED UTILITY CLEARANCES, PERMIT COSTS, INSURANCE, BONDS, ETC.
2. PRO-RATED EQUIPMENT MOBILIZATION AND DISMANTLING.
3. SERVICES OF POLICE OFFICERS FOR TRAFFIC CONTROL.
4. FOUR INCH DIAMETER UNSAMPLED, UNCASED, MUD STABILIZED, OVERBURDEN AND ROCK DRILLING.
5. GROUNDWATER OBSERVATION WELL AND PIEZOMETER MATERIALS, ASSEMBLY AND INSTALLATION.
6. TEMPORARY PVC CASING INSTALLED TO FACILITATE GEOPHYSICAL SURVEYS.
7. EQUIPMENT AND OVERTIME ASSOCIATED WITH INSTALLATION OF GEOPHYSICAL CASING.
8. CEMENT, BENTONITE, OTTAWA SAND, COMMON SAND, GRAVEL, ROADWAY BOXES, WATERPROOF SEALANT, ETC.
9. CEMENT GROUTING OF PIEZOMETERS, OBSERVATION WELLS, AND ABANDONED GEOPHYSICAL HOLES.
10. SEE TABLE 10-1 FOR DEFINITION OF EXPLORATION TYPES.

FIGURE 10-4. COST DISTRIBUTION - PIEZOMETERS AND OBSERVATION WELLS

By installing instrumentation casing in completed boreholes for use at a later date for construction monitoring, the costs for redrilling the borehole or drilling a new hole were eliminated. As may be noted from the cost distributions, Figures 10-1 through 10-4, drilling costs and associated technically non-productive costs are a substantial portion of the total exploration costs. To incur these drilling costs twice would substantially increase the total cost of the program without increasing its value.

10.3 SURFACE AND BUILDING SETTLEMENT POINTS

10.3.1 Accuracy

Level survey procedures, including the use of the automatic precision level and Invar staff, Section 6.3.2, were developed to minimize errors and obtain accurate information. Surveys of surface and building settlement points were recorded to 0.0001 feet, with any closure error distributed over the level net. Level survey error was usually on the order of $\pm$ 0.0010 feet. Obvious spurious data was noted and edited during data reduction. It was felt that the overall accuracy of the survey was $\pm$ 0.0025 feet. This value is within the limits noted by Cording et al. (I-4), Table 10-2.

Factors which may have contributed to survey inaccuracies were:

a. Random or systematic error in instrument or rod setup or in level reading.

b. Seasonal movements due, for example, to the effects of frost heave.

c. Effects of surface construction or traffic. Points that were destroyed by surface construction were re-established.

10.3.2 Cost

Costs for the initial setup of the 12 building points and 75 surface settlement points are shown in Table 10-3.

TABLE 10-2. SUMMARY OF METHODS AND INSTRUMENTS FOR MONITORING GROUND MOVEMENTS (CORDING ET. AL., I-4)

INSTRUMENT	RANGE	ACCURACY	ADVANTAGES	LIMITATIONS AND PRECAUTIONS	RELIABILITY
OPTICAL LEVELING WITH PRECISE-PARALLEL PLATE MICROMETER ATTACHMENT, SPECIAL ROD, 1ST ORDER TECHNIQUES.		± 0.002 to ± 0.004 ft.	SIMPLE, FAST (PARTICULARLY WITH SELF-LEVELING INSTRUMENTS). PRECISE.	REQUIRES GOOD BENCH MARK AND REFERENCE POINTS, AND CAREFUL ADHERENCE TO STANDARD PROCEDURES.	EXCELLENT
DEEP SETTLEMENT POINT - MULTIPOINT MAGNETIC SWITCH TYPE. (ELECTRICAL)		± 0.003 to ± 0.01 in.	SIMPLE, CAN BE COMBINED WITH INCLINOMETER CASING.		
DEEP SETTLEMENT POINT - TELESCOPING CASING - USING INCLINOMETER CASING WITH SLIDING JOINTS. (MECHANICAL)	6 in.	± 0.2 to 0.8 in.	SIMPLE, CAN ALLOW MEASUREMENTS OF BOTH HORIZONTAL MOVEMENTS (WITH INCLINOMETER) AND VERTICAL MOVEMENTS OF INCLINOMETER CASING.	LACK OF POSITIVE DRIVING FORCE BETWEEN CASING AND SOIL.	FAIR
PORTABLE BOREHOLE INCLINOMETER - ACCELEROMETER TYPE	± 30° optional to ± 90°	± 1 in. in 500 to 1000 ft.	READS TWO AXES AT A TIME. AUTOMATIC READOUT AND RECORDING PROVISIONS.	LENGTHY CALCULATIONS WITHOUT AUTOMATIC READOUT. WITH AUTOMATIC READOUT, MANUAL CHECK OF DATA FOR ERRORS STILL REQUIRED.	GOOD

TABLE 10-3. SURFACE AND BUILDING SETTLEMENT POINTS INITIAL COSTS (1982*)

DESCRIPTION		COST
Establishment of benchmark		$ 225
Layout and installation of points (Total for 87 points)		1,000
Setup of data handing system		125
	TOTAL	$1,350

*An increase of 10 percent per year was used to convert 1979 costs to 1982 costs.

A complete survey of building and surface settlement points had an average cost of $400, distributed as shown in Table 10-4.

TABLE 10-4. SURVEY COST OF SURFACE AND BUILDING SETTLEMENT POINTS

DESCRIPTION		COST
Equipment Rental		$ 16
Field Labor		320
Data Reduction/Plotting		64
	TOTAL	$ 400

NOTES:

1. Costs of police traffic control and travel time to and from site not included.
2. Costs are in 1982 dollars.
3. Costs are based on optical survey of all points.

After a number of complete surveys had been performed during construction operations, it was apparent that partial surveys, omitting points far away from the tunnel headings, could provide adequate information regarding ground movement. Usually, partial surveys reduced the number of points in the survey by 25 to 50 percent. Approximately half of the surveys were partial surveys, realizing a savings of about 30 percent per survey.

10.3.3 Benefit

Optical leveling of building and surface settlement points allowed this research to:

1. monitor surface settlements at the ground surface in a simple and reliable manner,

2. anticipate effects of surface settlements on buildings and other structures,

3. compare surface settlements with deep settlements,

4. check reference elevations of other instrumentation, and

5. provide data for the engineering analysis of ground surface settlement troughs.

10.4 DEEP SETTLEMENT POINTS

Deep settlements were monitored by the following combination of systems:

1. the electrical probe, Section 6.4.1, with the electrical settlement casing, Section 6.4.1, and combination settlement casing, Section 6.4.3;

2. the mechanical hook probe with the telescoping inclinometer/settlement casing, Section 6.4.2.

Table 10-1 shows that the cost of installing the combination settlement casing, Section 6.4.3, was about one-half the cost of installing the electrical settlement casing, Section 6.4.1. However, the performance of these two casing systems appeared to be about the same when used with the electrical settlement probe. Therefore, these two systems are considered together in the following discussion.

The following sections discuss the accuracy, costs, and benefits of these electrical and mechanical deep settlement measurement systems.

10.4.1 Accuracy

All deep settlement measurements were recorded to 0.001 feet. The electrical measurement system has a manufacturer's rated accuracy of $\pm$ 0.004 feet, and from project experience, it is felt that this is a reasonable value. The manufacturer of the mechanical hook probe does not rate its accuracy but it was felt to be repeatable to ± 0.003 feet. Cording et al. (I-4), Table 10-2, rate the accuracy of the electrical system as ± 0.0003 to ± 0.0008 feet, and of the mechanical system ± 0.017 to ± 0.067 feet. However, from project experience, there was no discernable difference in the accuracy of the electrical and mechanical systems.

Factors which may have contributed to survey inaccuracies were:

a. deficiencies in grouting of casing, possibly resulting in some casing free play relative to the surrounding ground,

b. errors in the optical survey of the surface box rims, Section 10.3.1, since these were used as reference for the deep settlement measurements,

c. inconsistencies by personnel in the setup, operation, and reading of the electrical or mechanical probes,

d. electrical or mechanical probe error.

In addition to these, the mechanical hook probe measurements may also have been influenced by:

a. binding of the telescoping joint due to grout or excessive inclination,

b. variable tension in the measurement tape,

c. the possibility that the probe hooked different points on the casing section end for different readings. This may become important if the casing end is inclined out of a horizontal plane.

10.4.2 Cost

The costs of installing the deep settlement casings are summarized in Section 10.2. The costs of monitoring these casings are summarized on Table 10-5.

TABLE 10-5. SURVEY COSTS OF DEEP SETTLEMENT MEASUREMENTS

TYPE OF SYSTEM	NUMBER OF CASINGS	SETUP OF DATA HANDLING/ INSTRUMENT CALIBRATION	SURVEY COST			TOTAL COST	SURVEY COST PER CASING
			EQUIPMENT RENTAL	FIELD LABOR	OFFICE LABOR		
ELECTRICAL SYSTEM	9	$125	$10	$205	$75	$290	$32.20
MECHANICAL SYSTEM	4	$125	$ 5	$ 30	$35	$ 70	$17.50

NOTES:

1. COSTS OF POLICE TRAFFIC CONTROL AND TRAVEL TIME TO AND FROM SITE NOT INCLUDED.
2. COSTS ARE IN 1982 DOLLARS.
3. COSTS ARE BASED ON READING AN AVERAGE OF TEN SETTLEMENT POINTS FOR EACH CASING.

These costs include $125 to set up the data handling system, including forms, and instrument calibration. The equipment cost represents the costs of renting the Sondex™ probe for the electrical surveys and the depreciation of the hook probe (mechanical survey) which was purchased for this project. Many of the surveys made for this study were partial, omitting settlement casings far away from the tunnel headings. This resulted in savings of survey costs generally in proportion to the number of casings omitted.

Table 10-5 shows that the cost of surveying the nine casings with the electrical probe was over four times the cost of surveying four casings with the mechanical probe. The survey cost per casing for the electrical system, $32.20, was approximately double the cost per casing for the mechanical system, $17.50. However, Table 10-1 shows that the installation costs were slightly less for the electrical casings compared to the telescoping (mechanical) casings. Based upon these data, there seems to be no clear overall cost advantage to either of the two deep settlement measurement systems used on this project.

10.4.3 Benefit

The benefits of the deep settlement measurements were identification of the sources of lost ground near the tunnels surface settlements, and to relate them to tunnel construction activities. In particular, these measurements showed that:

a. Small displacements at depth propagated relatively undiminished to the ground surface, Figure 8-8.

b. Because of the ability of the glacial till to arch and carry load, large displacements at depth extended no more than about one tunnel diameter up from the tunnel crown, Figure 8-9.

c. There was very little time lag between surface and deep settlements, Figure 8-5. These settlements were observed to begin almost immediately with the passage of the tunnel headings.

10.5 INCLINOMETERS

10.5.1 Accuracy

The inclinometer measurements were recorded on magnetic tape to four significant places, corresponding to an angular displacement of 1 in 10,000. The Digitilt™ inclinometer sensor used has a rated accuracy of about 1 part in 10,000 (+ 0.05 inches over 100 feet) when used with properly installed and nearly vertical Slope Indicator Company casing. Cording et al. (I-4) report a slightly lower level of accuracy, of 1 part in 3000 to 6000 (± 1.0 inches over 500 to 1000 feet), Table 10-2. During this study, the inclinometer sensor was read in a calibration casing mounted in the laboratory as part of the daily checkout procedure. These measurements, made over more than 1 year, show a level of accuracy of about 1 part in 800 (± 0.75 inches over 100 feet).

This lower observed performance level, relative to the listed accuracy of the Digitilt™, may reflect calibration slope shifts, the effects of humidity within the sensor, temperature changes, or sensor rotation errors. The procedure of determining the inclinometer profile from the average of readings from two casing surveys, with the torpedo sensor rotated 180 degrees between them, should minimize errors due to sensor zero shifts.

As already discussed with reference to Figure 8-13, many of the measured lateral displacements are very small and fall within the accuracy limits of the Digitilt™ inclinometer. Thus, these small displacements may show only measurement inaccuracies. A method for interpreting these data is described later in this Section.

Casing related factors which may have contributed to inclinometer survey scatter were:

1. lateral displacement of the bottom of the casing,
2. non-verticality of the casing transverse to the measurement plane, and
3. casing spiral.

These sources of error and corrections which have been developed for them are discussed in the following paragraphs.

10.5.1.1 Lateral Displacement of the Bottom of the Casing - The inclinometer profiles, Appendix F, are determined from an integration of the measured angular displacements from the casing bottom up. This procedure requires that the bottom of the casing remains fixed. If the bottom of the casing does move laterally, then all of the integrated lateral displacements will be in error by the amount of lateral movement of the casing bottom.

All but one of the inclinometer casings at the Test Section were grouted into bedrock, generally at or below the tunnel inverts. Inclinometer casing TSC 12 was terminated about 5 feet above the bedrock surface, Figure 3-8. This is evident in the resulting inclinometer profiles from TSC 12, Figure 8-14. These show significant angular displacements in the lowest 5 feet of casing because it was not fixed.

The lateral displacement below the bottom of the casing introduces an error in the measured lateral displacements from the bottom of the top of the casings. This lateral displacement, if large and known to exist, can be determined by an independent optical location survey of the top of casing. However, as Figure 8-14 shows, this error still does not effect the determination of the zones of greatest deformations in the overburden soils.

10.5.1.2 Casing Non-verticality - The inclinometer casings were installed with initial non-verticalities, typically 1 to 2 feet (0.5 to 1.1 degrees) on both the A and B axes (Figure 6-14). This occurred partly in order to avoid the tunnels, and partly because of drift of the boreholes.

Wilson and Mikkelsen (I-8) point out, Figure 10-5, that casing non-verticality in one plane will introduce measurement errors in the other perpendicular plane if the sensor alignment changes. Sensor alignment changes, say on the order of 1 to 2 degrees, may occur "because of one or several factors, such as wheel play in the groove, wear of the sensor carriage (particularly wheel assemblies), internal change in the sensor itself, and change in the alignment between sensor and carriage", (Wilson and Mikkelsen, I-8).

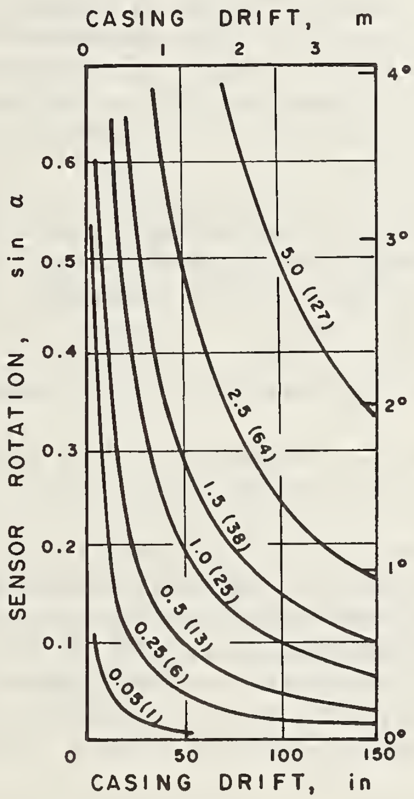


FIGURE 10-5. MEASUREMENT ERROR AS A RESULT OF CASING INCLINATION AND SENSOR ROTATION (Wilson and Mikkelsen, I-8)

Figure 10-5 shows that B-axis drift, typically of 1 to 2 feet at the Test Section, with sensor rotations of 1 to 2 degrees, may introduce A-axis measurement errors on the order of 0.25 to 1 inch. A suggested correction for this source of error is presented in Section 10.5.1.4.

10.5.1.3 Casing Spiral - The spirals of the installed inclinometer casings were measured in-situ by means of a spiral checking instrument available from the Slope Indicator Company. The results of these measurements show that five of the six ABS casings (TSC 8 through TSC 12) had very low spirals, less than

20 degrees, with TSC 7 having a total spiral of about 48 degrees. The ABS casings had interior milled grooves, Figure 6-13, and were installed within the corrugated PVC electrical settlement casing.

On the other hand, the telescoping settlement/inclinometer casings (TSC 13, 14, and 15) with extruded grooves, Figure 6-18, showed much greater spirals. The measured spirals here, from 66 to 77 degrees, may be a result of manufacturing defects. Possibly, the exterior grooves of the PVC casing tend to screw the casing into the ground as it passes restrictions in the borehole.

The equations for the transformation of rectilinear coordinate axes were applied to some inclinometer data to develop a suitable correction for spiral. The equations are:

$$\delta'_A = \delta_A \cos\chi - \delta_B \sin\chi \qquad (10\text{-}1)$$

$$\delta'_B = \delta_A \sin\chi + \delta_B \cos\chi \qquad (10\text{-}2)$$

where:

δ'_A and δ'_B are the corrected position coordinates of a point on the inclinometer casing, δ_A and δ_B the uncorrected position coordinates, and χ the spiral angle from the top of casing to the point. See Figure 10-6. δ_A and δ_B are the positions obtained on the rotated axes, A and B. δ'_A and δ'_B are the positions of the A' and B' axes at the top of the casing.

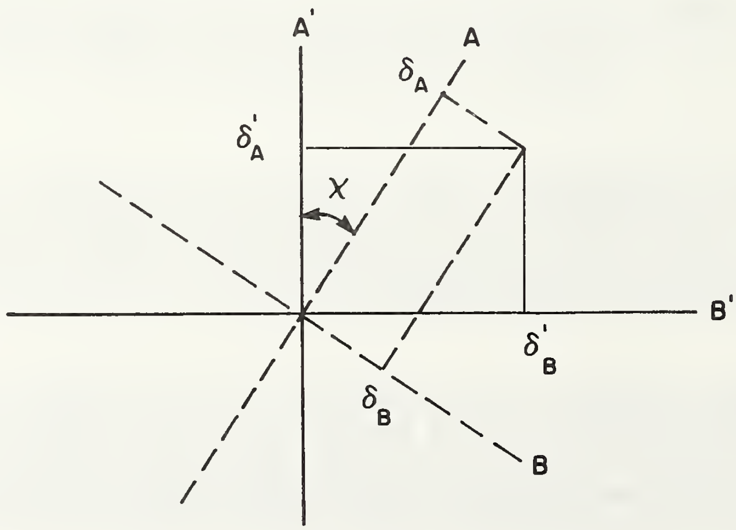


FIGURE 10-6. SPIRAL CORRECTION

Application of Equations 10-1 and 10-2 to selected data from the casings with spirals less than 20 degrees gave reasonable results. The effects of the spiral correction on the data from these casings were very small.

Application of Equations 10-1 and 10-2 to selected data from TSC 13 and 14, two of the high spiral casings, gave unreasonably large corrected lateral displacements. These large corrected displacements are geometrically impossible and inconsistent with general observations at the ground surface. The difficulty occurs because the simple correction, Equations 10-1 and 10-2, does not consider the complex three-dimensional geometry of the problem. The vertical distance between read points on the cable is not adjusted for the nonvertical, spiralled path which the cable takes. Also, the effect of casing spiral on the reading of the sensor itself may be significant, but is not known. It was not feasible to explore these points within the scope of the current research. These, however, may be appropriate subjects for future studies.

Based on these considerations, the final inclinmeter profiles, Appendix F, are uncorrected for spiral. For the ABS inclinometer casings with small spiral, the corrections would be very small. For the four highly spiralled inclinometer casings, the inclinometer profiles provide a good measure of the dates and elevations of large horizontal displacements. However, the direction of these displacements in the horizontal plane is confused by the need to correct for large casing spiral.

10.5.1.4 Profiles of Inclination - Inclinometer survey errors, such as those due to lateral displacement of the bottom of the casing, casing non-verticality, and casing spiral, frequently accumulate as the measurements are integrated up the casing for profiles of lateral displacement, Appendix F. On the other hand, the shapes of the profiles are not so greatly affected by these errors. The shapes also provide an excellent indication of where ground loss and soil deformations are occurring. As indicated in Section 8-3, this suggests that it may be appropriate to interpret these inclinometer surveys by plots of inclinations vs. depth as compared to the conventional displacement vs. depth.

For example, Figure 10-7 presents profiles of inclination vs. depth for inclinometer TSC 11. Zones of ground movement as the tunnel passed are very clearly defined at elevations 30 to 38 and 42 to 50. Movements at higher elevations are also shown, due perhaps to the effects of ground loss or groundwater lowering as well. The corresponding profiles of lateral displacement, Figure 10-8, even with the very large scales, have much poorer resolution.

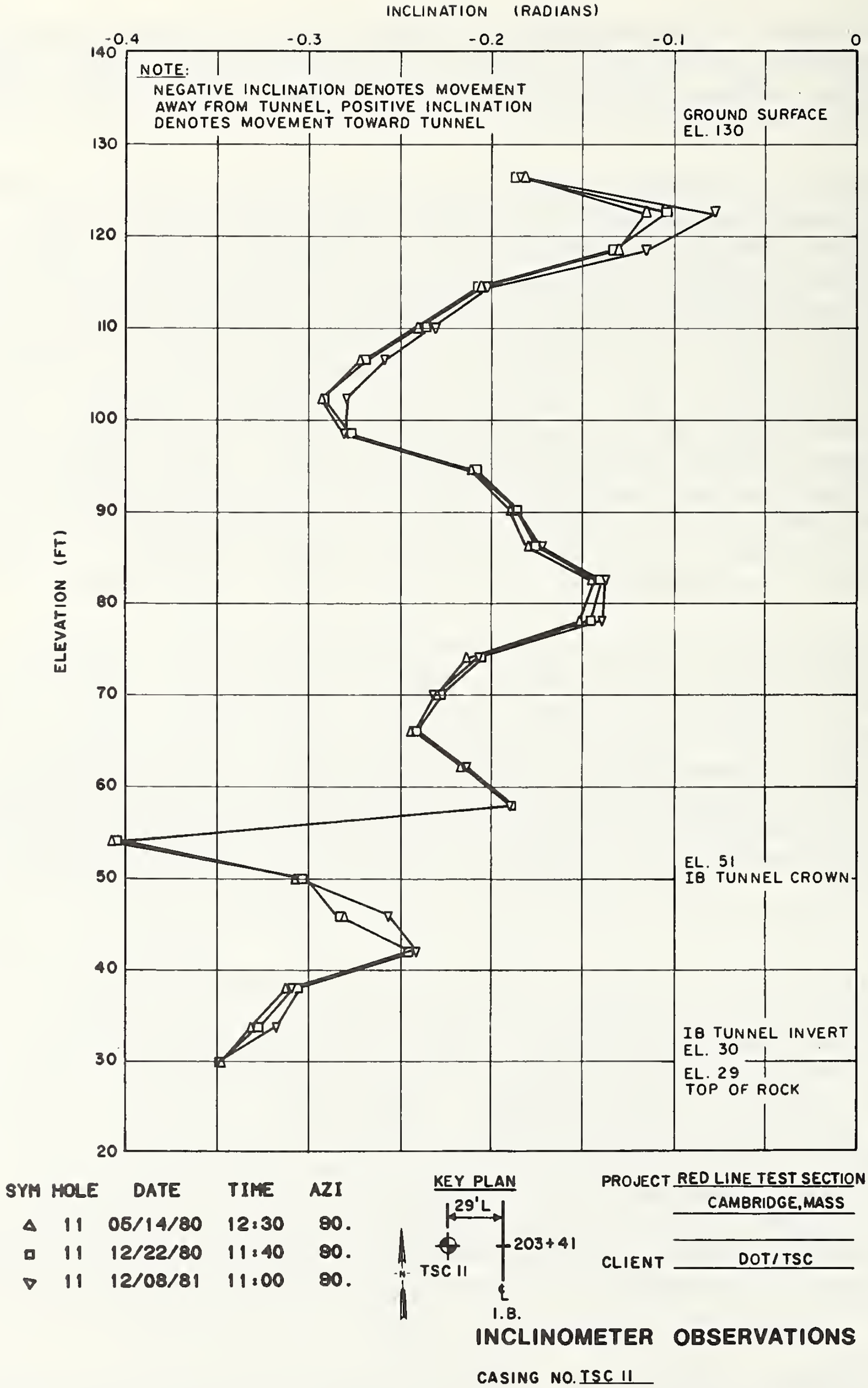


FIGURE 10-7. PROFILE OF INCLINATION VS. DEPTH, TSC 11

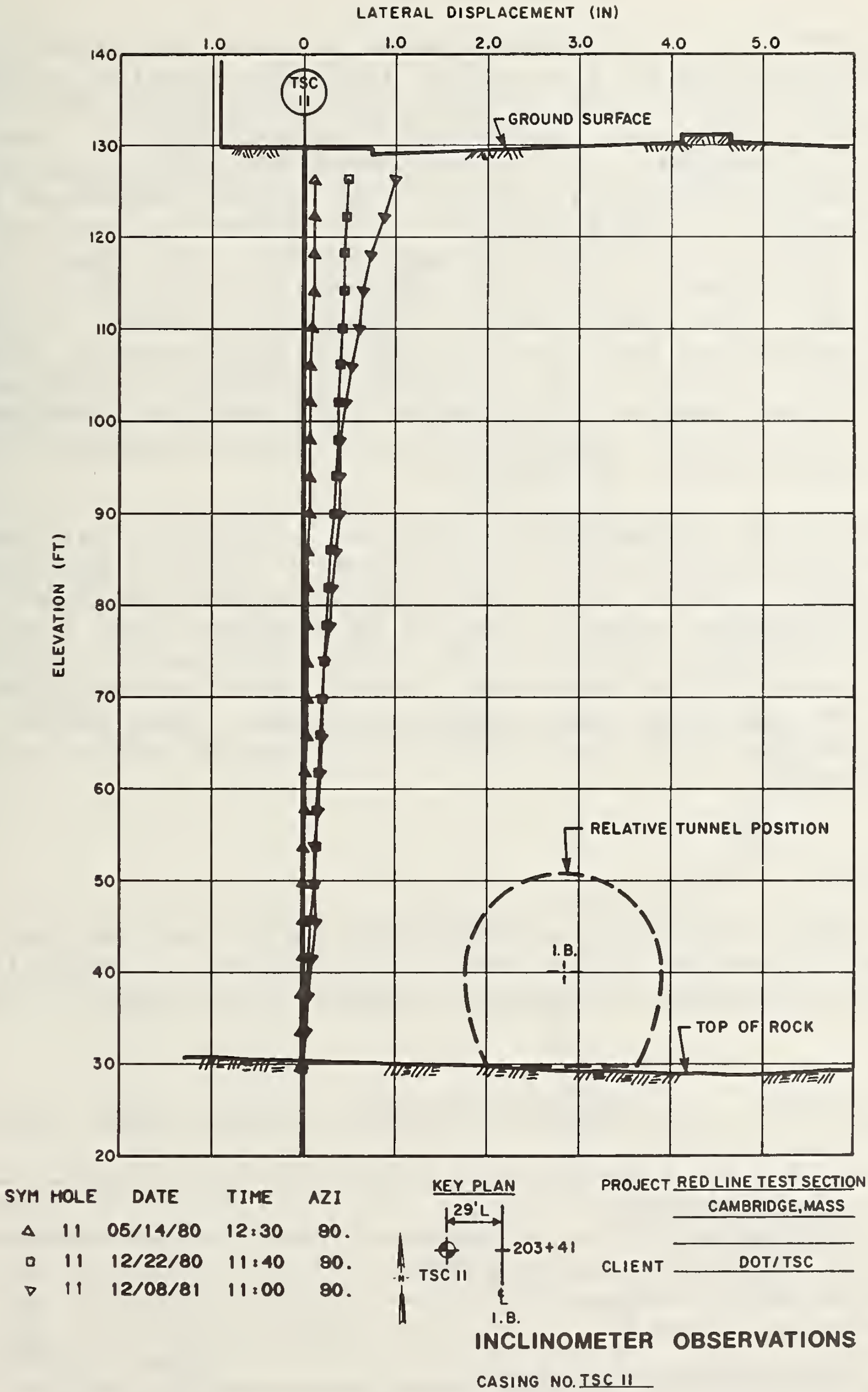


FIGURE 10-8. PROFILE OF LATERAL DISPLACEMENT VS. DEPTH, TSC 11

The profiles of inclination also provide some useful corrections for some of the errors discussed earlier in this section. For example, the inclinations measured at TSC 10 for three selected dates are plotted on Figure 10-9. If, for the moment, it is assumed that the bottom of the casing was fixed, then the inclination at elevation 25 should be unchanged for all three surveys. The differences in angles here may be due to casing non-verticality and sensor errors. If, then, the angles are adjusted so that the readings at the bottom of the casing are all the same, this will cause a rotation in space of the corresponding profiles of lateral displacement, Figure 10-10. If the survey of 16 June 1981 is taken as reference, then the inclinations of 22 December 1980 must be translated slightly to the right, Figure 10-9, and the inclinations of 22 November 1981 slightly to the left.

Figure 10-10 compares the profiles of lateral displacement which have been corrected in this manner, with the uncorrected profiles. The profile for 22 November 1981 now has been rotated towards the tunnel, that of 22 December 1980 away from the tunnel, and they appear to be correctly related to one another. However, the corrected lateral displacements for 22 December 1980 now show some negative values. This may be because of movement of the bottom of the casing which has not been measured.

10.5.2 Cost

The costs of installing the inclinometer casings are summarized in Section 10.2. The initial costs of establishing the monitoring system for the inclinometers are shown in Table 10-6.

TABLE 10-6. INCLINOMETER SURVEY INITIAL COSTS (1982*)

DESCRIPTION		COST
Data Sheets and Plotting Forms		$ 626
Computer Data Storage		400
Computer File Setup		450
	TOTAL	$1,475

* An increase of 10 percent per year was used to convert 1979 costs to 1982 costs.

A complete survey of the nine 100-foot inclinometers cost approximately $360 (1982). Distribution of these costs are

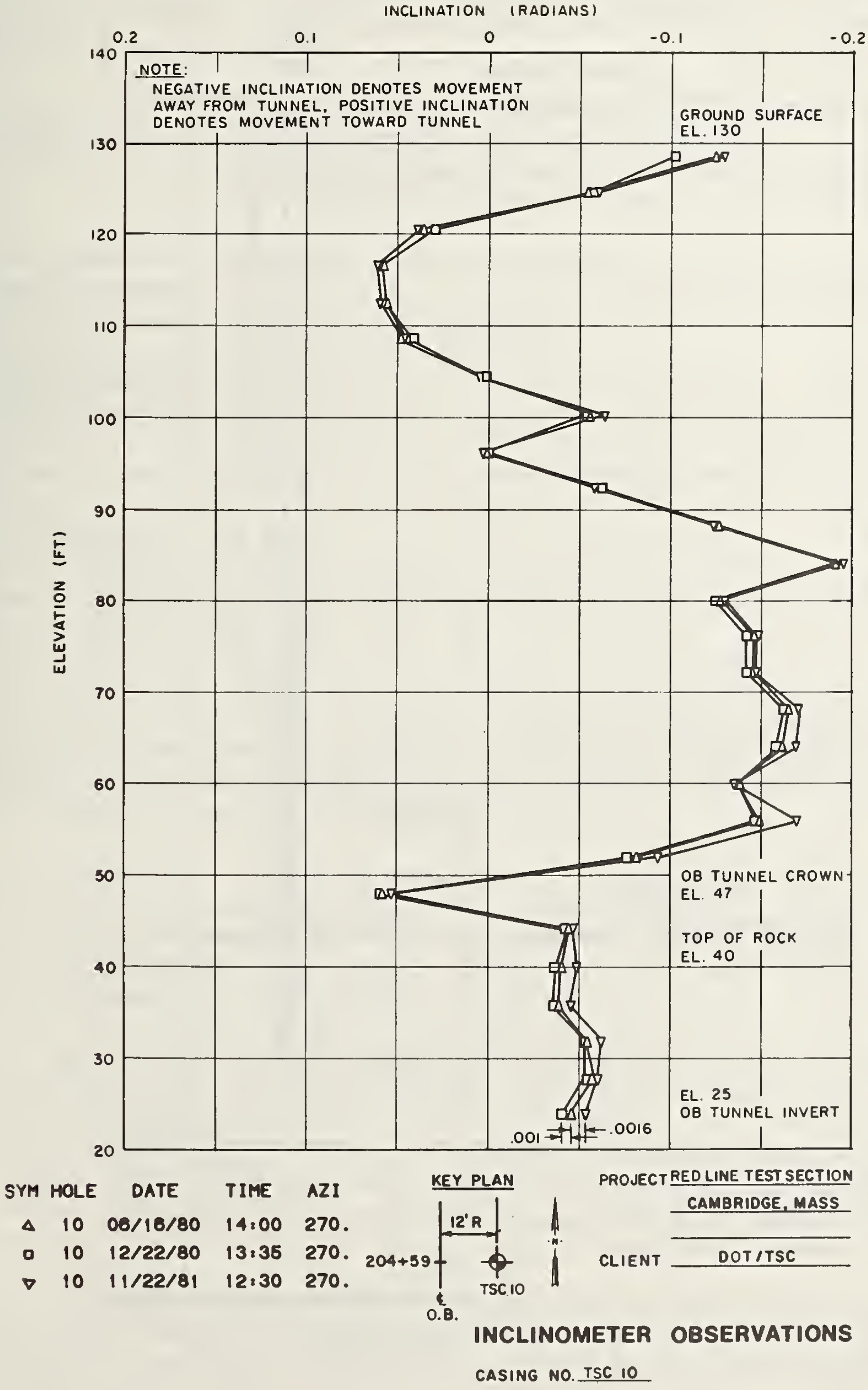


FIGURE 10-9. PROFILES OF INCLINATION VS. DEPTH, TSC 10

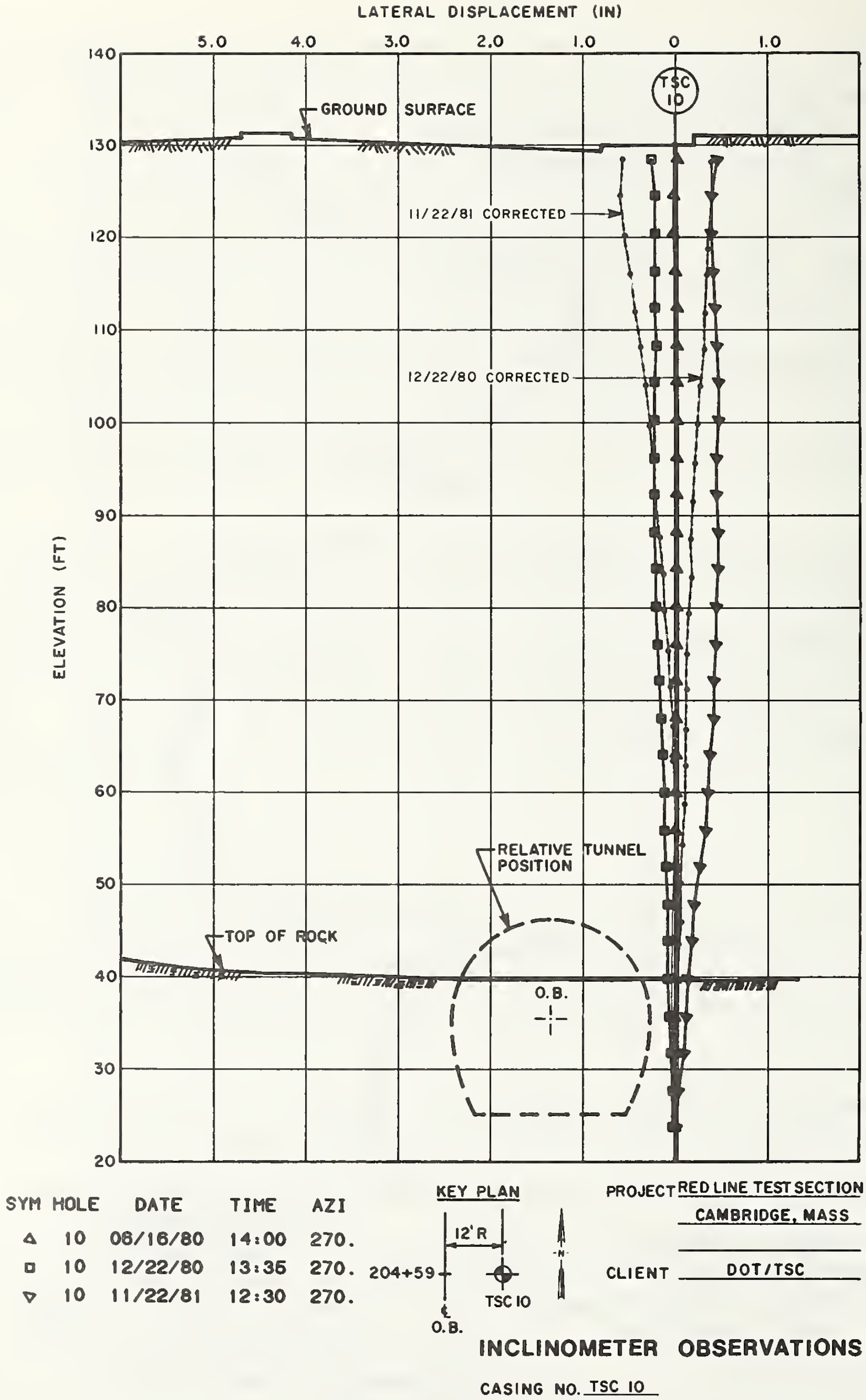


FIGURE 10-10. PROFILES OF INCLINATION VS. DEPTH, WITH CORRECTION, TSC 10

shown in Table 10-7. Partial surveys were often performed, monitoring only six of the nine casings, giving a proportional reduction in survey costs.

TABLE 10-7. SURVEY COSTS OF INCLINOMETER MEASUREMENTS (Nine 100-foot casings)

DESCRIPTION		COST
Equipment Rental		$ 30
Field Labor		200
Data Reduction/Plotting		130
	TOTAL	$360

10.5.3 Benefit

The benefits of the inclinometer observations during this research were as follows:

a. Provided a measure of lateral displacements at depth, and, therefore, the locations of largest ground movement and ground loss. For example, TSC 12, Figure 8-14, shows the zones of largest ground movements as the outbound tunnel passed to be at elevations 46 to 55 and 60 to 68.

b. Provided an immediate indication of ground loss due to soft ground tunneling operations. For example, the inclinometer survey of 20 October 1981, TSC 12, Figure 8-14, shows that lateral displacements at depth developed almost immediately with the passage of the tunnel heading.

c. Showed that soil deformations due to ground losses at the tunnel heading did not extend very far horizontally. For example, the soil deformations (i.e. change in inclination) measured at TSC 11, 15 feet away from the inbound tunnel, Figure 10-8, are considerably smaller than the soil deformations measured at TSC 12, two feet away from the outbound tunnel, Figure 8-14.

d. Showed that the soil deformations due to ground losses at the tunnel heading did not extend very far vertically. The deformations measured at TSC 12, Figure 8-14, extended no more than about one tunnel diameter above the tunnel. The measured lateral displacements accumulated at the ground surface were small, within the limits of accuracy of the inclinometer system.

The computerized inclinometer data handling system utilized by this research proved to be of great value. It minimized tedious hand calculations and facilitated the interprations described above. It would be an essential ingredient for providing rapid feedback to the tunneling contractor for modifications of construction procedures based upon measurements of ground movements.

10.6 PIEZOMETERS AND OBSERVATION WELLS

Observation wells were read to 0.5 foot, piezometers to 0.1 foot. It was felt that these instruments were repeatable to these accuracies. Some of the wells became clogged or obstructed during the almost two years of research, making them unreadable.

The cost of installing the observation wells and piezometers are summarized in Section 10.2. The cost of reading one observation well or piezometer and reducing the data was about $10.

The benefit of these instruments was to monitor shallow groundwater levels, piezometric levels in the rock, and to note their change with tunnel heading excavations and dewatering operations.

10.7 SUMMARY

The following sections provide summary comments on instrument performance and their value for the engineering and construction of tunnels, particularly for predicting the effects of tunneling on adjacent facilities. These comments must be considered in view of the following factors:

a. In general, ground movements at the Test Section were small, in many cases approaching the limits of accuracy of the instrumentation.

b. The location of the Test Section in an area of such a high degree of geologic variability provided a unique opportunity to monitor and compare ground movements due to different subsurface conditions (rock, soft ground, and mixed face) and construction procedures. This, however, makes a direct comparison of instrument performance difficult.

10.7.1 Instrument Performance

10.7.1.1 Surface and Building Settlement Points - These instruments, in general, performed very well. Because of their relatively low cost, it was possible to provide many of these points, and read them often. They were reliable although some measurements were lost because of surface construction activities. However, the degree of redundancy that could be included in the surface measurements allowed a complete analysis of ground surface movements, Section 9.

10.7 1.2 Deep Settlement Points - There was no clear difference in the observed performance of the electrical settlement system compared to the mechanical (telescoping settlement/inclinometer casing) system. On the other hand, the electrical system has a higher listed accuracy and a corrugated casing which probably follows ground movements more closely than the smooth extruded PVC casing used for the mechanical system. The extruded PVC casing also exhibited a great deal of spiral, complicating interpretation of inclinometer data. There was no clear overall cost differential between these two systems and, therefore, the electrical system is the preferred system.

The combination settlement casing system, Figure 6-9, has the disadvantage that it cannot accommodate inclinometer measurements.

10.7.1.3 Inclinometers - Even when the horizontal ground displacements are very small, as at the Test Section, the shapes of the inclinometer profiles (lateral displacement vs. depth) provide an indication of where ground loss and soil deformation are occurring. This suggests that plots of sensor inclination vs. depth, Figure 10-7, can sometimes more clearly define the zones of largest deformation around the tunnel, and are less affected by measurement errors than the traditional profiles of lateral displacement.

When very accurate measurements of lateral displacements are required, then sources of error can be corrected for as discussed in Section 10. For example, if there is the possibility that the bottom of the casing has moved, then this shoud be checked by an independent optical survey of the horizontal position of the casing top. Casing spiral should be measured during installation and, if low, can be corrected for by means of Equations 10-1 and 10-2. In this respect, the interior grooved ABS casing exhibited much less spiral, in general, than the extruded PVC casing. Plots of sensor inclination vs. depth can sometimes be used to correct for the effects of casing non-verticality and sensor errors, as discussed in Section 10.5.1.4.

10.7.2 Value for the Engineering and Construction of Tunnels

Construction monitoring of ground movements shows the effects of ground loss due to tunnel construction, both near the tunnel and near the ground surface. The engineering of tunnels, particularly in soft ground or mixed face conditions, emphasizes the effects of ground movements on adjacent or overlying structures. During construction, the stability of the face is of paramount importance. Research on the effects of tunnel construction benefits greatly from measurements of the complete three dimensional pattern of movements around the tunnel opening. The value of the instrumentation studied in this project for engineering, construction, and research of tunnels is discussed in the following paragraphs.

10.7.2.1 Surface and Building Settlement Points - These measurements are relatively inexpensive and reliable. Often, surface settlements are a direct cause of building damage. Therefore, settlement surveys at the ground surface by optical leveling are important for engineering purposes. These data are also very important relative to the contractor's liability for damage to structures. Surface settlement surveys can be of value for research purposes. The surface settlement data at the Test Section provided significant information on the size and shape of settlement troughs for the single and twin tunnels in glacial till, Figures 9-3 and 9-6 and for the tunnels in a mixed face condition, Figure 9-10.

10.7.2.2 Deep Settlement Points - For engineering purposes, measurements of deep settlements can identify the sources of lost ground. At the Test Section, data from the deep settlement points show that large losses of soft ground at the tunnel heading propagated no more than 1 or 2 tunnel diameters from the heading, Figure 8-9. For construction purposes, deep settlement points ahead of the tunnel may forewarn of impending face instabilities. However, this requires very frequent observations at critical times and immediate feedback to the contractor. This is expensive (although not so expensive as a major ground loss) and, hence, is not frequently done. For research purposes, data from deep settlement points are most valuable when combined with measurements of horizontal movements, such as inclinometer data.

10.7.2.3 Inclinometers - For engineering purposes, measurements of horizontal movements at depth can identify the sources and extent of lost ground. (See Figure 8-14 for an example at the Test Section). They can be very valuable for identifying the effects of the tunnel construction on adjacent structures since buildings are often more sensitive to horizontal movement than vertical movement. For construction purposes, inclinometer surveys ahead of the tunnel opening may forewarn of impending face instabilities. However, as discussed above with reference to deep settlement points, this requires very frequent observations at critical times and immediate feedback to the contractor. In this respect, the computerized inclinometer data handling system utilized in this research is essential. For research purposes, data from inclinometer surveys are most valuable when combined with measurements of vertical movements, such as deep settlement point data.

11. DESCRIPTION OF GEOLOGIC MAPPING

11.1 GENERAL

The geologic mapping program consisted of daily visits to the working face. Each face exposure was sketched and mapped, recording the significant geologic features. This information was then added to the as-built geologic map being compiled during tunnel excavation. Photography was an integral part of the mapping program, when conditions permitted. Observations were recorded regarding excavation procedures, mining, and temporary support techniques used in each heading. In the shield driven headings, rib spacing and rib expansion was measured and recorded. In the rock tunnels and invert drifts, the distance to the working face from the last rib and the advance rate was recorded. The geologic mapping is shown in Appendix H.

11.2 FACE MAPPING - SOIL AND ROCK

The ability to map at a working face depended on the nature of the construction activity at the time of the visit. Drilling, scaling and mucking activity made work close to the face hazardous. During the loading of explosives, all non-essential personnel were restricted from the face. Working at the shield face was generally difficult due to the confined conditions and limited visability, especially when the breast doors were closed. The excavated material was left in place below the springline wherever possible to assist in stabilizing the face. This hindered face mapping of the lower areas of the face.

Face maps were made daily in the full face tunnel sections at a scale of 1 inch equals 10 feet. Mapping in the invert pilot drift was performed at a scale of 1 inch equals 5 feet. The data were taken from the field sketches and incorporated onto the as-built map at a scale of 1 inch equals 20 feet.

All significant geologic features which were thought to have an effect on the stability of the tunnel were mapped. The bedrock was classified by lithology and any internal structural features, such as foliation, schistosity, flow structure and bedding, were recorded. The position of formation boundaries (rock or soil), bedding thickness and attitude, and areas of soft or unstable rock were noted. Physical properties, such as hardness, brittleness, color, degree(s) of weathering or alteration, and cementation were also recorded. Veins, sills, or dikes were described in terms of mineralogy, size, form (tabular and irregular), contacts (sharp, gradational, and sheared), and any mineralization accompanying the intrusion.

Features or conditions which affected the stability of the rock were recorded. These included joints, shear zones, slip planes, and faults. Joints or joint systems were described according to spacing, attitude and type (whether open or tight, slickensided, cemented, wet or dry). Shear zones were described by severity of shearing and the physical condition of the rock in and surrounding the zone. The internal condition of the shear zone was investigated for crushing, fragmentation or gouge. The extent of fracturing in the rock on either side of the shear zone was described. A slip plane was considered as a joint which showed evidence of movement, but direction or amount was not measurable. Faults were described showing attitude, width of the gouge, amount of disturbance or fracturing of the rock on each side and, when possible, amount and direction of displacement.

Estimates of groundwater inflow were made at each face mapped. When tunneling through soft ground sections, the stratigraphy was described as well as the standup time. The presence of groundwater and the effect of it on the face stability was also described. Mapping in the mixed face sections required both soft ground and rock mapping methods, in addition to noting the position of the top of rock surface in each exposure.

11.3 PHOTOGRAPHY - SOIL AND ROCK

Photographs were taken at the tunnel heading as conditions permitted to provide a permanent record of the conditions encountered, to supplement the geologic mapping, and to illustrate particular features or conditions. Photographs are used throughout this report to illustrate or give special emphasis to the discussion.

11.4 OTHER

The long-term effects of the tunnel on the rock or soil were closely observed. In tunnel projects with multiple headings, it is important that a distance be maintained between two adjacent tunnel headings. The passage of the first heading will normally cause some localized deformation. When the second heading passes, the additional ground deformation can be minimized by proper spacing of the working faces and prompt installation of temporary support elements.

Newly exposed rock can react in an adverse way. Air slaking, blast vibration or the effect of adjacent tunnel headings can all contribute to deteriorating conditions. Estimates of the amount of overbreak or fallout were made at each working face.

The size and spacing of ribs varied with type of tunnel and tunneling environment. Measurements of the amount of expansion achieved on the steel ribs were made. The size of the Dutchmen installed in the shield driven tunnels was noted. The type and effectiveness of the temporary support was recorded in the full face rock tunnels and the invert pilot drifts.

12. COMPARISON OF PREDICTED AND OBSERVED STRATIGRAPHY

12.1 GENERAL

This section will compare the stratigraphic predictions made in the Stage I report (Section 9) with the as-built geologic conditions observed at the time of tunnel excavation. The as-built conditions are shown as a series of maps, sections, and tables presented in Appendix H of this report.

The profiles presented in the Stage I report have been reproduced and included in Appendix B. The Stage I profiles combined conventional core logging with seismic crosshole and nuclear borehole logging techniques to establish the presence and trend of major stratigraphic changes, weathered horizons, shear zones, and areas of poor quality rock along the proposed tunnel alignment. These logging techniques provide details on the distribution of major changes in the rock mass, providing a basis for making general assessments of the rock mass as related to tunneling.

The geologic mapping program carried out during tunnel excavation as part of the Stage II program confirmed the location of major shears, weathered areas, zones of poor quality rock, and geologic contacts predicted in Stage I. Additionally, the Stage II mapping program provided the opportunity to achieve a greater degree of control on the extent and attitude (strike and dip) of discontinuities and stratigraphic changes observed in the tunnel excavation.

12.2 AS-BUILT GEOLOGIC MAPS

Appendix H of this report contains the as-built geologic maps that were compiled during the tunnel excavation. The geologic maps are at true scale of 1 inch equals 20 feet to avoid any distortion of detail. The major features that can be correlated include poor quality rock zones, strata changes, and the decomposed rock zone predicted in the Stage I report. The only restriction imposed upon this comparison is that the as-built data is limited to the area within the tunnel excavation. No attempt has been made to project geologic features more than several feet beyond the tunnel sidewalls. The extent of shearing, or presence of intrusive bodies, which results in sudden changes in lithology and attitude shown within the tunnels, justifies this choice.

The plan view (H-4, H-5) shows the geology along the tunnel alignment projected to springline elevation. Two longitudinal sections, each showing the geology along the inbound tunnel (H-8, H-9) and the outbound tunnel (H-6, H-7) have been included. The geology in these sections has been projected to the centerline of each tunnel, compared to the longitudinal profiles in the Stage I report that were located west of the inbound tunnel (profile A-A), in the pillar between tunnels (profile B-B) and along the east edge of the outbound tunnel (profile C-C). Appropriate notes about tunnel conditions observed at the indicated face stations are shown above and below the tunnel area. These notes focus on the quality of rock or soil in the face, groundwater inflows, and stability of the crown and sidewalls.

For comparison, cross sections A-A', B-B', and C-C' shown in Appendix H, are located at the approximate locations of Stage I profiles D-D, G-G, and I-I, respectively.

12.3 SUMMARY OF SITE GEOLOGY AND STRUCTURE

Section 4 of this report describes the predicted stratigraphy in the Test Section. The principal bedrock type along the tunnel alignment is the Cambridge Argillite, which contains diabase dike intrusions of varying thicknesses. The tunnels in the Test Sections were driven partially through argillite and partially through the C_4 subunit of the glacial till. Table 12-1 summarizes the major geologic units that were encountered during tunneling.

12.3.1 Glacial Till

The tunnel excavation through the Test Section penetrated the basal section of the glacial till. This corresponds to subunit C_4 described in the Stage I report. In general, the till consists of dense to very dense, silty, fine to coarse grained sand with varying amounts of clay, gravel, cobbles, boulders and lithic fragments including argillite, granitic and mafic rocks.

The glacial till observed in the tunnel headings revealed that it is characteristically poorly sorted and unstratified, lacking horizons that possessed any great degree of lateral continuity. There were, however, zones, lenses, or pockets of sand, gravel, silty sand, that were continuous across the tunnel face, and sometimes mappable between adjacent headings.

The glacial till zones mapped ranged from massive greenish-gray to gray silty sands and sandy silts to a grayish-green

TABLE 12-1. SUMMARY OF GEOLOGIC CONDITIONS FOUND DURING TUNNEL MAPPING

TUNNEL STATION	TUNNEL CROWN	TUNNEL INVERT	APPROXIMATE ELEVATION OF			MAJOR ROCK TYPE	OVERALL AVERAGE ROCK QUALITY (RQD)
			TOP[1] OF TILL	TOP OF DECOMPOSED ROCK	TOP OF NON-[1] DECOMPOSED ROCK		
203+00 OB	54.5[3]	30.5	105.5	33.5	31.5	ARGILLITE	POOR
203+50 OB	50	28	108	41	39	ARGILLITE	POOR - FAIR
204+00 OB	49	27	108	45	44	MIXED ROCK[2]	VERY POOR - POOR
204+50 OB	47	25	113	47.5	46	DIABASE	FAIR
205+00 OB	45.5	23.5	111	N.D.*	N.D.*	MIXED ROCK	POOR
205+50 OB	44	22	116	N.D.*	N.D.*	MIXED ROCK	FAIR
206+00 OB	43	19	N.D.*	N.D.*	N.D.*	ARGILLITE	POOR - FAIR
203+50 IB	53.5	29.5	112	34	32.5	ARGILLITE	VERY POOR
204+00 IB	52	28	107	40.5	39	ARGILLITE	POOR - FAIR
204+50 IB	50	26	114	47.5	45.5	MIXED ROCK	FAIR
205+00 IB	48	24	118	N.D.*	N.D.*	MIXED ROCK	FAIR
205+50 IB	47	23	114	N.D.*	N.D.*	MIXED ROCK	POOR - FAIR
206+00 IB	45	21	111	N.D.*	N.D.*	MIXED ROCK	POOR - FAIR
206+37 IB	43	19	N.D.*	N.D.*	N.D.*	MIXED ROCK	POOR - FAIR

NOTES:
(1) FROM TABLE 9-1, STAGE I REPORT.
(2) SEE APPENDIX H.
(3) MBTA RED LINE DATUM (105.87 FEET BELOW USC & GS, MSL 1929).
(4) N.D. = SAMPLES NOT OBTAINED AND NO DETERMINATION MADE.
(5) * = OUTSIDE TUNNEL LIMIT.

till with light gray laminae of silty, fine grained sand. All were poorly sorted and contained gravel, pebbles and cobbles. These clasts varied in size and lithology. Locally, throughout the glacial till, pockets of dense, well sorted sands and gravels were observed. Many of these pockets were damp, some seeped slightly, and others were dry.

At the base of the glacial till, a zone of broken and weathered bedrock was encountered. This zone was interpreted as either weathered and broken in place or deposited by glacial action. At some localities, this zone was clearly part of the overlying glacial till. The blocks of argillite were randomly oriented relative to the underlying bedrock, and, therefore, a depositional feature. Conversely, when the structural elements such as bedding planes and jointing in the overlying blocks were continuous with the bedrock, they were interpreted as part of the bedrock which weathered in place. The location of this zone is comparable to the decomposed rock zone predicted in the Stage I report. It did not, however, match the description of the predicted zone. This inconsistency in predicted versus observed condition will be explored in more detail in Section 12.6. This zone, never more than two feet thick, was a source of groundwater seepage in the mixed face section.

12.3.2 Cambridge Argillite

The Cambridge Argillite, the dominant bedrock lithology in the Test Section, is a greenish-gray, banded, slightly metamorphosed mudstone. The hardness ranges from soft, severly fractured and weathered near its surface, to hard and fresh at greater depths below the interface zone. There are local exceptions adjacent to shear zones or igneous intrusions. Typically, the argillite is bedded with alternating layers of dark gray to black silty clay and light gray silty, fine sand. Bed thickness typically ranges from 1/8 to 5 inches. Overall, it demonstrates a rhythmically layered or varved appearance.

Throughout the argillite sequences exposed during tunnel excavation, thin sandstone and volcanic tuff interbeds were encountered. These zones ranged from several inches to three feet in thickness.

12.3.3 Diabase Dikes

Diabase dikes of varying thickness were observed intruded into the host argillite. The mineralogy was generally altered amphiboles and/or pyroxenes and feldspars with traces of opaques, commonly pyrite. The color of the dikes ranged from dark green in fresh exposures to light green in weathered zones. Dike thickness ranged from less than a foot to several tens of feet. The maximum continuous thickness exposed (at springline) was approximately 50 feet in the outbound tunnel excavation from about Station 204+11 to Station 204+59. The dikes were usually bounded with sheared contacts with dips ranging from 2 to 90 degrees, indicating tectonic emplacement. An extensive mixed rock zone exists in both tunnels where the argillite and diabase are in contact either by sheared zones or their interfingering of diabase into the argillite. In areas where the igneous contact with the host argillite was preserved, a number of contact alterations were observed. Secondary mineralization within the contact zone was usually in the form of pyritization. The argillite underwent contact metamorphism adjacent to the intrusions. The color was altered from the usual grayish green to purple or green from the combined effects of thermal and hydrothermal fluid activity. The argillite in the contact zones was more brittle than the argillite unaffected by the intrusive body.

12.3.4 Structure

The discontinuities along which the rock tended to break during tunneling were controlled by bedding planes, jointing and faulting. None of these features were observed to extend into the overburden soils.

12.4 POOR QUALITY ROCK ZONES

The Stage I report predicted generally poor quality rock at the northern end of the Test Section, which was projected towards the pillar between the two tunnels. The Stage II data confirms this relationship. While local reversals in attitude within the poor quality rock zones exist, the general trend of the intensity of shearing is in agreement. Also, the construction summaries (H-10,H-11) indicate that the rock quality is poorer in the inbound tunnel than in the outbound tunnel, due to intensity of shear zones.

The poor quality rock zones were primary avenues for groundwater seepage into the tunnel. The rock in the Test Section was generally moderately weathered. The blocks of rock were stained, and the joint surfaces bounding them were altered and commonly clay filled. Few of the discontinuities were clean and tight in the Test Section.

The flow of groundwater was not extensive at the time the tunnels were excavated. There were localized areas of groundwater seepage where the full face rock tunnel or the invert drift reached the interface with the overlying glacial till. The flow of groundwater gradually diminished in these areas.

12.5 STRATIGRAPHIC CHANGES

The major bedrock stratigraphic changes predicted in the Stage I report occurred at locations where diabase dikes were intruded. The tunnel mapping confirmed this. The predicted attitudes of the dikes were not always in agreement with the mapped conditions. This was probably because the large dike mass predicted by Stage I methods was in fact found to be several distinct and separate bodies, usually bounded by shear zones of varying widths. These field relations suggest a tectonic origin of emplacement. The geologic mapping indicates that in both the inbound and the outbound tunnels, several separate dikes are either in contact with one another or in close proximity. The host rock between the diabase dikes is usually sheared and altered argillite. The Stage I report predicted extensive zones of altered argillite between dike occurrences.

Minor changes in lithology that were indicated by the core logs in the Stage I report were observed during tunnel excavation. The argillite consists of interbeds of volcanic tuff and occasional argillaceous sandstone. The thickness of these beds ranged from several inches to three feet.

12.6 DECOMPOSED ROCK ZONE

The Stage I report identified a zone of decomposed rock located at the glacial till/rock interface. This zone was characterized by a loss of structural integrity and closely resembles a soil. It consists primarily of a clayey sand with fragments of intermixed unweathered rock. The thickness of the zone varied but was never greater than two feet (Table 12-1).

During Stage II mapping, a zone of rock unlike the underlying bedrock, but not resembling the overlying till, was mapped at the glacial till/rock interface. The zone appeared to consist of discrete blocks of fresh bedrock bounded by seams or open joints filled with glacial till like material. The structural elements within the bedrock, such as bedding and jointing, had no continuity with similar features in the blocks above.

The predicted zone of soil-like material described in the Stage I report closely resembles the description of a saprolite zone. The presence of a saprolite zone, while not impossible, is rather unlikely to occur in this geologic environment due to the extensive and probable multiple glacial events that took place in this area. Local pockets or zones of saprolite might be expected to be preserved in special situations. However, the extent of the zone mapped is far greater than what could be expected to a preserved saprolite zone, considering the geologic history of this area.

The zone observed in the Stage II mapping would appear to be the result of an ice jacking mechanism. Ice formed in partially open joints close to the bedrock surface during glaciation, expanding and actually separating blocks of bedrock from the rock mass below. As the ice melted, till washed into the voids. The maximum thickness of these till filled seams was one to two inches. The ice jacking mechanism generally does not cause the blocks of rock to move very far. Movement is confined to several inches or feet at most. Therefore, although orientation of bedding planes and joint surfaces are no longer consistent with the underlying bedrock, the differences are not great.

Conversely, these blocks could have been deposited as part of the till. If this were the case, the thickness of till bounding the blocks would be much thicker and the individual blocks of rock would most likely be randomly oriented.

Because of the thickness (relatively thin) of till seams bounding the blocks of rock and the subtle but distinct reorientation of the rock blocks, ice jacking rather than glacial transport is considered to be the mechanism responsible for the formation of this decomposed rock zone.

This condition has been observed and documented in other areas of the Red Line Extension tunnels (i.e., south of Porter Square) and in other excavations in the areas, such as the foundation of Harvard University's Pusey Library.

12.7 ENGINEERING GEOLOGY

As part of the geologic mapping program conducted during tunnel excavation, geologic features (discontinuities) were observed with the intent to analyze groups of similar elements and the effect on tunnel stability. In addition to stratigraphic changes, faults and shears, major factors controlling tunnel stability are discontinuity spacing, and attitude relative to the tunnel alignment and degree of weathering.

The geologic factors which affected tunnel excavation were planes of weakness or discontinuities controlled by such features as bedding, jointing, shear, and fault zones. These elements making up the fabric of the rock mass are the surfaces along which the rock is most likely to break during tunneling. Jointing is probably the most common type of rock discontinuity. It can be controlled by a variety of pre-existing qualities in the rock mass. In the Cambridge Argillite, bedding plane joints were the most ubiquitious of the joint sets mapped. Joint spacing ranged from a few inches to three feet or more in the massive rock zones. Except for local disruptions by shears or igneous intrusions, the bedding plane joint set was oriented nearly perpendicular to the tunnel alignment and dipped less than 45 degrees to the south.

The next most common joint set was a series of discontinuities oriented nearly parallel to the tunnel alignment, dipping at angles greater than 45 degrees. This set of joints was spaced greater than one foot apart. When this joint set bounded a block of rock and was inclined into the tunnel opening, a potentially unstable situation resulted.

Most shear zones were filled with clay and crushed argillite with some local secondary mineralization, usually calcite. These discontinuities were normally oriented north-northeast with dips nearly vertical. A distinctive set of joints was found very close (within two feet) to the shear zone(s). These joints were spaced less than one foot apart and created a randomly fractured surface on the tunnel walls. The orientation, being nearly parallel to the tunnel alignment, made the crown and sidewalls locally unstable where the two intersected.

The diabase dike intrusions were often bounded by shear zones. These occurrences created a set of joints parallel to the shear zones that were oriented from 60 to 90 degrees to the tunnel alignment. The dip angles varied from each location.

The strength of the argillite rock mass was somewhat less than that of the diabase. For this reason, the diabase was more difficult to excavate. The diabase arched well after blasting, except where extensively sheared. The diabase ravelled after exposure. The argillite had a tendency to ravel continuously. Scaling after blasting, tight blocking and the use of crown spiling minimized ravelling when tunneling in either diabase or argillite.

The argillite and diabase, located near the interface with the glacial till, were weathered and blocky. This created uncertain support conditions at the mixed face. Some rounds would break ahead of the shield, while some would not. In the cases where the till was unsupported ahead of the shield and when the rock broke too far ahead of the shield, the till ravelled into the shield face. This created voids above the hood.

Groundwater inflows in the full face rock section of the tunnel excavation generally increased as the tunnel heading approached the till interface. In both headings, the last 10 to 15 feet had initial inflows of approximately 20 gallons per minute. This volume diminished gradually with time. At no time were large volume inflows observed. Also, during the excavation of the 10-foot square invert drifts, water volumes increased slightly closer to the interface.

Groundwater seepage into the face of the shield driven headings in the mixed face section was limited to local flows from the sandy or gravelly zones. These flows were of short duration and low volume. The interface zone was an intermittent source of groundwater inflow as well. A weir and settling basin at the portal collected water pumped from the tunnel face. The weir at the portal to the tunnels indicated 2 to 5 gallons per minute volume being pumped from the face of each heading.

13. EVALUATION OF ADVANCED METHODS OF EXPLORATION

13.1 GENERAL

Detailed descriptions and evaluations of various advanced exploration methods have been presented in the Stage I report (I-7). In this Section, a brief evaluation of these methods is presented considering the results of the geologic tunnel mapping. The predicted stratigraphy is shown on the geologic profiles presented in Appendix B; while the mapped geologic profiles are presented in Appendix H.

13.2 DRILLING AND SAMPLING

Boreholes with detailed sampling were drilled along both tunnel alignments. These boreholes, of course, allow excellent stratigraphic control and identification of structural discontinuities at the borehole site. This type of borehole information is an absolute prerequisite for tunneling exploration. The preferred sampling method is one which provides the highest recovery percentage of the sampled material. In fractured rock, this is a core barrel with a split inner barrel, either double-tube or triple-tube. In soil, the preferred sampling method depends on the grain size and density of the material.

The character of the soil and rock between sampled boreholes not is known and can only be approximated by straight-line interpolation. Problems could result from this interpolation when the geologic structure is parallel to the borehole axis and cannot be inferred by other means. In the Test Section, a reasonably close approximation of top of rock was obtained from the sampled boreholes. The undulations of this contact were further clarified by the unsampled boreholes. Geologic mapping of the tunnels showed that the rock was severely fractured throughout the Test Section. This is supported by the sampled borehole logs, which all indicate zones of major fracturing. The extent of this fracturing between the boreholes was inferred by the various geophysical exploration methods.

13.3 GEOPHYSICAL METHODS

Three types of geophysical logging techniques were used during the Stage I work. They are nuclear, seismic, and electrical.

13.3.1 Electrical Methods

It was concluded in the Stage I report that electrical borehole logging may not be applicable in an urban environment, where most transit tunneling is located. The excessive interference caused by man made sources such as electric lines and underground utility pipes and conduits is a severe limitation in obtaining accurate, repeatable log responses.

13.3.2 Seismic Methods

Seismic crosshole and uphole surveys were conducted during the Stage I investigation. Details of this investigative method are presented in the Stage I report (I-7).

The radius of investigation is limited only by the amount of energy available for impulse generation and the bulk densities of the investigated materials. In urban areas, the maximum energy may be limited by the necessity of protecting adjacent buildings and utilities.

Seismic methods are often used in surface surveys to determine depth to bedrock. Similar applications are possible within a borehole provided that the velocity contrast is large enough to discriminate between bedrock and overlying soil.

Bedrock quality may be ascertained using seismic borehole methods. A decrease in wave velocity may indicate a decrease in rock quality. Also, correlations of the elastic constants (Poisson's Ratio and Young's Modulus) with compressional and shear wave amplitudes are used to define zones of poor quality rock.

In the Test Section, predicted zones of poor quality rock are shown on the Geologic Profiles in Appendix B. The geologic profiles showing the mapped geology (Appendix H) indicate a zone of concentrated shearing and jointing at the north end of the Test Section, principally in the inbound tunnel. This zone correlates well with the zones of poor quality rock inferred from the seismic crosshole survey and shown on geologic profiles B and I in Appendix B. Geologic Profile B also shows a zone of poor quality rock at top of rock, which thickens upstation. This zone is confirmed by the geologic mapping shown in Appendix H.

13.3.3 Nuclear Methods

Nuclear borehole logging was conducted during the Stage I work. Details of the logging procedure, theory and results are presented in the Stage I report (I-7). Four types of nuclear logging were employed, including natural gamma (NATG), neutron epithermal neutron (NN), neutron gamma (NG), and gamma gamma (GG). The radius of investigation is typically 6 to 24 inches.

The nuclear logging method provided excellent correlation with top of rock. The correlation with soil stratigraphy is discussed in the Stage I report. The tunnel, within the Test Section, cut through only the lower glacial till and bedrock.

Within the bedrock, detection of zones of poor quality rock was not particularly successful. A few major fracture zones were detected but many others were not, including zones of clay 6 inches thick. The success of nuclear logging as a stratigraphic tool has been demonstrated in the Stage I report. However, its use as a structural tool is apparently limited by detection radius as well as the careful control necessary for complete quantitative analyses, which was not available during the test program.

The nuclear instruments could not be calibrated for borehole size because several different borehole diameters and casing schedules were used. This variability was necessitated by the variety of exploration, testing, and monitoring techniques used in the test program. This variability would not occur in a nonresearch oriented exploration program.

Also, breakouts in the borehole wall interfere with the interpretation of nuclear logs. The boreholes were cased and a measure of the borehole roughness could not be obtained.

14. CONCLUSIONS

14.1 GENERAL

This section presents principal conclusions with respect to the application and performance of the geotechnical instrumentation studied during this investigation. The conclusions are based on performance at a Test Section through which twin rapid transit tunnels were excavated in rock, soft ground, and mixed face tunneling conditions. Conclusions are also presented with respect to advanced methods of subsurface exploration based upon comparison of the stratigraphy predicted by these methods with the stratigraphy observed during tunnel construction.

The site is considered to represent a typical urban area with the Test Section located under a major four-lane divided street, with masonry and wood frame structures abutting both sides of the street. The geotechnical instruments were evaluated in relation to rock, soft ground, and mixed face tunneling conditions at a depth of approximately 100 feet below ground surface. Overburden soils consisted primarily of saturated, very dense, glacial till containing cobbles and boulders. The bedrock unit consisted of a partially metamorphosed shale (argillite) which has been intruded by major igneous dikes (diabase).

The instruments included surface and building settlement points to clearly define the ground surface settlements. Instruments installed in completed exploratory boreholes included deep settlement points and inclinometer casings, to measure vertical and horizontal movements, respectively, at various depths. Piezometers and observation wells were installed to measure changes in groundwater conditions due to tunnel construction.

Section 3 of this report describes the Test Section, Section 4 the subsurface conditions, Section 6 the geotechnical instrumentation, and Section 7, the construction procedures and progress. Despite complicated subsurface conditions, different construction procedures, and small measured ground movements at the Test Section, this study acquired significant data regarding the performance of the instruments used. Conclusions from this study are presented in the following sections.

14.2 GROUND SURFACE SETTLEMENTS

Measured ground surface settlements at the Test Section were compared to predictions of ground surface settlements made prior to tunneling within the Test Section. The predictions were based on the collection of case history data first published by Peck (I-6), later supplemented by Cording et al. (I-3).

Comparison of the predicted settlements with the settlements measured at the Test Section, Table 9-1 shows that the predicted settlements were slightly larger than the measured settlements. In view of uncertainties regarding subsurface conditions and details of tunneling methods, equipment, and workmanship, at the time the predictions were made, the agreement between predicted and measured settlement is excellent.

The settlement of the ground surface in the rock tunneling area was predicted to be small, less than 0.021 ft. This shows excellent agreement with the measured settlements. Detailed comparison of the predicted settlement volumes in soft ground, V_S = 0.8 percent for single tunnels, ΔV_S = 0.4 percent for interference between twin tunnels, with the measured settlement volumes indicates excellent agreement. However, the settlement troughs were much wider than would be inferred from the relationships between tunnel depth and size to settlement trough width for various subsurface conditions published by Peck (I-6) and Cording et al. (I-3), Figure 9-11. Although these tunnels were excavated in glacial till above the groundwater level (dewatered), and the soil profile is predominantly granular, the measured i/R values plot in the region for tunnels excavated in soft to stiff clays, Figure 9-11. This may be because for small settlements such as these, the soil deformations are elastic rather than plastic and hence a wider trough develops. The identical observation was made by Cording et al. (I-3) in their studies of the Washington, DC, Metro Section F2a, also plotted on Figure 9-11.

The surface settlements at the middle section show some interesting effects of mixed face conditions. The first tunnel (inbound) was mostly in rock here and was excavated full faced using rock tunneling drill-and-blast procedures. It caused slightly greater surface settlements than were observed at the section in rock, indicative of greater ground losses due to soil disburbance. The second (outbound) tunnel had more of its cross section in the soft ground and except for a 10-foot by 10-foot invert pilot drift, was excavated using soft ground shield tunneling procedures, Figure 8-10. It caused the settlement trough here to have a greater volume (V_S = 1.28 percent) than was measured for the single tunnel in soft ground (V_S = 0.76 percent), Figure 9-3. This may be reflective of the more difficult construction environment under mixed face conditions. However, the settlement trough in the mixed face section caused by excavation of the second (outbound) tunnel was entirely symmetrical about the tunnel. This indicates that there was little, if any, interference between the first (inbound) and second (outbound) tunnels at this section. The geometry of the settlement trough caused by the second (outbound) tunnel (Figure 9-10), as measured by i/R and β, was almost identical for the single tunnel in soft ground (Figure 9-3).

14.3 PERFORMANCE OF INSTRUMENTS

These summary comments on instrument performance must be considered in view of the following factors:

a. In general, ground movements at the Test Section were small, in many cases approaching the limits of accuracy of the instrumentation.

b. The location of the Test Section in an area of such a high degree of geologic variability provided a unique opportunity to monitor and compare ground movements due to different subsurface conditions (rock, soft ground and mixed face) and construction procedures. This, however, makes a direct comparison of instrument performance difficult.

14.3.1 Surface and Building Settlement Points (Section 6.3)

These instruments in general, performed very well. Because of their relatively low cost, it was possible to provide many of these points and read them often. They were reliable although some measurements were lost because of surface construction activities. However, the degree of redundancy that could be included in the surface measurements allowed a complete analysis of ground surface movements, Section 9.

14.3.2 Deep Settlement Points (Section 6.4)

There was no clear difference in the observed performance of the electrical settlement system compared to the mechanical (telescoping settlement/inclinometer casing) system. On the other hand, the electrical system has a higher listed accuracy and a corrugated casing which probably follows ground movements more closely than the smooth extruded PVC casing used for the mechanical system. The extruded PVC casing also exhibited a great deal of spiral, complicating interpretation of inclinometer data. There was no clear overall cost differential between these two systems and therefore, the electrical system is the preferred system.

The combination settlement casing system, Figure 6-9, has the disadvantage that it cannot accommodate slope inclinometer measurements.

14.3.3 Inclinometers (Section 6.5)

Even when the horizontal ground displacements are very small as at the Test Section, the shapes of the inclinometer profiles (lateral displacement vs. depth) provide an indication of where ground loss and soil deformation are occurring. This suggests that plots of sensor inclination vs. depth, Figure 10-7, can sometimes clearly define the zones of largest deformation around the tunnel, and are less affected by measurement errors than the traditional profiles of lateral displacement.

When very accurate measurements of lateral displacements are required, then sources of error can be corrected for as discussed in Section 10. For example, if there is the possibility that the bottom of the casing has moved, then this should be checked by an independent optical survey of the horizontal position of the casing top. Casing spiral should be measured during installation and if low, can be corrected for by means of Equations 10-1 and 10-2. In this respect, the interior grooved ABS casing exhibited much less spiral, in general, than the extruded PVC casing. Also, plots of sensor inclination vs. depth can sometimes be used to correct for the effects of casing non-verticality and sensor errors as discussed in Section 10.5.1.4.

The computerized inclinometer data handling system utilized by this research proved to be of great value. It minimized tedious hand calculations and facilitated the interpretations described above. It would be an essential ingredient for providing rapid feedback to the tunneling contractor for modifications of construction procedures based upon measurements of ground movements.

14.3.4 Piezometers and Observation Wells (Section 6.6)

The benefit of these instruments was to monitor shallow groundwater levels, piezometric levels in the rock, and to note their change with tunnel heading excavations and dewatering operations.

14.4 VALUE OF INSTRUMENTATION FOR THE ENGINEERING AND CONSTRUCTION OF TUNNELS

Construction monitoring of ground movements shows the effects of ground loss due to tunnel construction, both near the tunnel and the ground surface. The engineering of tunnels, particularly in soft ground or mixed face conditions, emphasizes the effects of ground movements on adjacent or overlying structures. During construction, the stability of the face is of paramount importance. Research on the effects of tunnel construction benefits greatly from measurements of the complete three dimensional pattern of movements around the tunnel opening. The value of the instrumentation studied in this project for engineering, construction, and research of tunnels is summarized in the following paragraphs.

14.4.1 Surface and Building Settlement Points

These measurements are relatively inexpensive and reliable. Often, surface settlements are a direct cause of building damage. Therefore, settlement surveys at the ground surface by optical leveling are very important for engineering purposes. These data are also important relative to the contractor's liability for damage to structures. Surface settlement surveys can be of some value for research purposes. The surface settlement data at the Test Section provided significant information on the size and shape of settlement troughs for the single and twin tunnels in glacial till, Figures 9-3 and 9-6, and for the tunnels in a mixed face condition, Figure 9-10.

14.4.2 Deep Settlement Points

For engineering purposes, measurements of deep settlement can identify the sources of lost ground. At the Test Section, data from the deep settlement points show that large losses of ground at the tunnel heading propagated no more than 1 or 2 tunnel diameters from the heading (Figure 8-9). For construction purposes, deep settlement points ahead of the tunnel may forewarn of impending face instabilities. However, this requires very frequent observations at critical times and immediate feedback to the contractor. This is very expensive (although not so expensive as a major ground loss), and hence, is not frequently done. For research purposes, data from deep settlement points are most valuable when combined with measurements of horizontal movements, such as inclinometer data.

14.4.3 Inclinometers

For engineering purposes, measurements of horizontal movements at depth can identify the sources and extent of lost ground. (See Figure 8-14 for an example at the Test Section). They can be very vaulable for identifying the effects of the tunnel construction on adjacent structures since buildings are often more sensitive to horizontal movement than vertical movement. For construction purposes, inclinometer surveys ahead of the tunnel opening may forewarn of impending face instabilities. However, as discussed above with reference to deep settlement points, this requires very frequent observations at critical times and immediate feedback to the contractor. In this respect, the computerized inclinometer data handling system utilized in this research is essential. For research purposes, data from inclinometer surveys are most valuable when combined with measurements of vertical movements such as deep settlement point data.

14.5 ADVANCED METHODS OF EXPLORATION

14.5.1 Overburden Drilling and Sampling

The selection of the most economical and satisfactory method of overburden drilling involves an evaluation of numerous hole advancement and stabilization techniques,with consideration for the formations to be penetrated and the samples to be obtained. No single method of drilling will prove satisfactory and economical for all formations and sampling requirements.

For the subsurface conditions investigated during this study, a diamond bit rotary core barrel using double-tube, split inner liner core barrels was found to be an effective overburden sampling device.

14.5.2 Rock Drilling and Sampling

Detailed information about rock is necessary in a tunnel exploration program. This necessitates maximum core recovery and minimum core disturbance. Double-tube and triple-tube core barrels with split inner barrels provide the means of obtaining relatively undisturbed core. When proper drilling techniques are employed, near complete recovery is possible, even in completely weathered rock.

14.5.3 Geophysical Methods

14.5.3.1 Nuclear - Nuclear or radioactive logging measures and records the natural or artificially produced formation radiations in an open or cased, fluid filled borehole.

The nuclear logging information obtained as a part of this study closely substantiated the correlation explorations, in addition to further defining stratigraphy and local anomalies which were not detected by conventional explorations.

However, in defining zones of poor quality rock the nuclear method was not impressive. The reason for this is that careful control and specific information are required to obtain a quantitative analysis using the nuclear methods. These requirements include calibration of the the location and extent of breakouts in the borehole wall. However, the boreholes were installed for a multiplicity of purposes and the control and information needed was not available in the test program. Six different borehole environments (casing types and borehole diameters) were used and the casing prevented the use of a caliper log to determine borehole roughness.

Nuclear logging was found to be rapidly and easily executed in the urban environment. However, the method requires that a borehole be available. Considering the high fixed costs of mobilization, utility clearances, police protection, etc., as compared to the cost of borehole advancement and the minimal cost of sampling, it appears impractical to advance a hole without sampling. Therefore, nuclear logging must be justified based on its capability to detect anomalies between samples or to measure properties not otherwise determined, at least until a rapid and economical method is developed to advance an unsampled hole solely for use in geophysical borehole logging. (In a hard rock environment, air drilling may meet these requirements.)

14.5.3.2 Seismic - Seismic borehole investigations are initiated to obtain a measurement of the transit time of compressional and shear seismic waves between boreholes which are used in calculating the engineering properties (Poisson's Ratio and Young's Modulus) of the materials through which the waves pass. In addition, supplementary information is obtained for interborehole formation correlation purposes and locating major anomalies.

The seismic crosshole method provided good correlation with the zones of poor quality rock. The method provides a broad rather than bed specific picture of the rock quality. Because velocity contrast between the glacial till and weathered rock was not very large, it was difficult to predict top of rock from seismic methods alone. In urban areas, where more difficult noise conditions exist, signal enhancement capabilities could be a prerequisite for successful seismic surveys.

Each of the exploration methods used in the Test Section provides information which increases in value when integrated with the results of the other methods. All of the indirect methods require sampled boreholes as correlation guides. It is necessary, when designing an exploration program, to have a working knowledge of the attributes and limitations of the various methods. With this knowledge, a proper combination of exploration techniques can be applied to efficiently determine the subsurface conditions.

APPENDIX A

GENERAL LEGEND AND NOTES

DESCRIPTION AND CLASSIFICATION OF SUBSURFACE MATERIALS

SOIL

SOIL DESCRIPTIONS NOTED ON THE SUBSURFACE EXPLORATION SUMMARY SHEETS ARE BASED ON PENETRATION TEST RESULTS, VISUAL-MANUAL EXAMINATION OF SOIL SAMPLES AND THE RESULTS OF LABORATORY TESTING ON SELECTED SOIL SAMPLES. THE CRITERIA, DESCRIPTIVE TERMS AND DEFINITIONS USED ARE PRESENTED BELOW:

TYPICAL DESCRIPTIONS

BROWN MEDIUM DENSE COARSE TO FINE SAND, LITTLE GRAVEL, TRACE SILT WITH BRICK AND WOOD - FILL -

BLUE-GRAY VERY STIFF SILTY CLAY, TRACE FINE SAND AND GRAVEL WITH THIN LENSES OF SILTY FINE SAND (CL) - MARINE CLAY -

GRAY VERY DENSE SILTY FINE SAND, LITTLE COARSE TO FINE GRAVEL, TRACE COARSE SAND WITH COBBLES AND BOULDERS AND OCCASIONAL FINE SAND AND SILT LAYERS (SM) - GLACIAL TILL -

DENSITY OR CONSISTENCY

DENSITY OF COHESIONLESS SOILS	PENETRATION RESISTANCE (BLOWS PER FOOT)	CONSISTENCY OF COHESIVE SOILS	PENETRATION RESISTANCE (BLOWS PER FOOT)
VERY LOOSE	≤ 5	VERY SOFT	≤ 3
LOOSE	6 - 10	SOFT	4 - 5
MEDIUM DENSE	11 - 30	MEDIUM STIFF	6 - 10
DENSE	31 - 50	STIFF	11 - 15
VERY DENSE	≥ 51	VERY STIFF	16 - 30
		HARD	≥ 31

COLOR

BROWN, YELLOW-BROWN, GRAY, BLUE-GRAY, ETC.

COMPONENTS

MAJOR SOIL COMPONENT - FIRST SOIL COMPONENT USED

SECONDARY COMPONENT - ADJECTIVE USED (IF 20-50% OF TOTAL)

THIRD COMPONENT - "SOME" USED (IF THIRD COMPONENT COMPRISES 20-35% OF TOTAL)

OTHER COMPONENTS - "LITTLE" USED (IF 10-20% OF TOTAL) "TRACE" USED (IF <10% OF TOTAL)

GEOLOGICAL DESCRIPTIVE TERMS

FILL - MAN MADE DEPOSITS OF NATURAL EARTH MATERIALS OR WASTE MATERIALS

OUTWASH - STRATIFIED SEDIMENTS OF PRIMARILY SAND AND GRAVEL DEPOSITED IN RAPID MOVING GLACIAL MELT WATERS

MARINE CLAY - FINELY INTERBEDDED SEDIMENTS OF SILT AND CLAY WHICH SETTLED OUT OF CALM POSTGLACIAL SEAS

GLACIAL TILL - VERY DENSE HOMOGENEOUS SEDIMENTS RANGING IN PARTICLE SIZE FROM CLAY TO BOULDERS DEPOSITED DURING THE GLACIAL ADVANCE

COMPONENT DEFINITIONS BY GRADATION

MATERIAL		FRACTIONS	SIZE LIMITS (1) UPPER	LOWER
BOULDERS	MATERIAL TOO LARGE TO PASS THROUGH AN OPENING 8-INCHES SQUARE	-	-	-
COBBLES	MATERIAL PASSING THROUGH AN 8-INCH SQUARE OPENING AND RETAINED ON THE 3-INCH SIEVE	-	8-IN.	3-IN.
GRAVEL	MATERIAL PASSING THROUGH THE 3-INCH SIEVE AND RETAINED ON THE NO. 4 SIEVE	COARSE FINE	3-IN. 3/4-IN.	3/4-IN. NO. 4
SAND	MATERIAL PASSING THROUGH THE NO. 4 SIEVE AND RETAINED ON THE NO. 200 SIEVE	COARSE MEDIUM FINE	NO. 4 NO. 10 NO. 40	NO. 10 NO. 40 NO. 200
SILT	MATERIAL PASSING THROUGH THE NO. 200 SIEVE WHICH IS ALSO NON-PLASTIC IN CHARACTER AND EXHIBITS LITTLE OR NO STRENGTH WHEN DRIED		< NO. 200	-
CLAY	MATERIAL PASSING THROUGH THE NO. 200 SIEVE WHICH CAN ALSO BE MADE TO EXHIBIT PLASTICITY WITHIN A CERTAIN RANGE OF MOISTURE CONTENTS AND WHICH EXHIBITS CONSIDERABLE STRENGTH WHEN AIR DRIED		< NO. 200	-

(1) U.S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

GROUP SYMBOL	FINE-GRAINED SOILS
ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GROUP SYMBOL	COARSE-GRAINED SOILS
GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SM	SILTY SANDS, SAND-SILT MIXTURES
SC	CLAYEY SANDS, SAND-CLAY MIXTURES

SUPPLEMENTARY DESCRIPTIVE TERMS

LAMINAE - 0 TO 1/16 INCH THICKNESS (COHESIVE SOIL)

PARTING - 0 TO 1/16 INCH THICKNESS (GRANULAR SOIL)

SEAM - 1/16 TO 1/2 INCH THICKNESS

LAYER - 1/2 TO 12 INCH THICKNESS

POCKET - SMALL, ERRATIC DEPOSIT USUALLY LESS THAN 12 INCHES

STRATUM - GREATER THAN 12 INCH THICKNESS

LENS - LENTICULAR DEPOSIT, LARGER THAN POCKET

OCCASIONAL - ONE OR LESS PER FOOT OF THICKNESS

FREQUENT - MORE THAN ONE PER FOOT OF THICKNESS

INTERBEDDED - APPLIED TO STRATA OF SOIL LYING BETWEEN OR ALTERNATELY WITH OTHER STRATA OF A DIFFERENT NATURE

ZONE - A REGULAR OR IRREGULAR BELT, LAYER, BAND OR STRIP

PENETRATION RESISTANCE:
STUDY PENETRATION TEST: NUMBER OF BLOWS REQUIRED TO DRIVE A 2-INCH OUTSIDE DIAMETER SPLIT-SPOON SAMPLER 12 INCHES USING A 300 POUND WEIGHT FALLING FREELY THROUGH 24 INCHES. TESTING WAS TERMINATED AT A MAXIMUM OF 400 BLOWS REGARDLESS OF THE SAMPLER PENETRATION.

ROCK

ROCK DESCRIPTIONS ARE BASED ON VISUAL-MANUAL EXAMINATION AND SELECTED PETROGRAPHIC ANALYSES OF CORE SAMPLES. THE CRITERIA, DESCRIPTIVE TERMS AND DEFINITIONS USED ARE PRESENTED BELOW:

TYPICAL DESCRIPTION

ALTERED ARGILLITE, LIGHT GREENISH GRAY, FRESH, HARD, BRECCIATED, REHEALED MACRO-SHEARS WITH FRACTURES AND JOINTS: OCCASIONAL CLAY FILLINGS, CALCITE VEINS AND DIABASE STRINGERS.

LITHOLOGY

ALTERED ARGILLITE, DIABASE, ARGILLACEOUS SANDSTONE, ETC.

WEATHERING (THE ACTION OF THE ELEMENTS IN ALTERING THE COLOR, TEXTURE AND COMPOSITION OF THE ROCK.) DEGREE OF WEATHERING WAS DETERMINED USING THE FOLLOWING TERMINOLOGY:

FRESH - NO DISCOLORATION, SLIGHT STAINING IN JOINTS.

SLIGHT - SLIGHT DISCOLORATION UP TO 1" INTO CORE FROM THE JOINT; JOINTS STAINED WITH CLAY INFILLING.

MODERATE - SIGNIFICANT PORTION OF ROCK DISCOLORED OR STAINED.

SEVERE - ALL ROCK DISCOLORED OR STAINED, SEVERE LOSS OF STRENGTH, CLAY SEAMS

FIELD HARDNESS

DETERMINATION OF HARDNESS OF THE ROCK CORE WAS PERFORMED IN THE FIELD BY USING A STEEL KNIFE BLADE, AND THEN DESCRIBED IN ACCORDANCE WITH THE FOLLOWING CATEGORIES:

VERY HARD - CANNOT BE SCRATCHED WITH A KNIFE BLADE

HARD - CAN BE SCRATCHED BY A KNIFE BLADE BUT ONLY WITH GREAT DIFFICULTY

MEDIUM-HARD - CAN BE SCRATCHED BY A KNIFE BLADE

MEDIUM-SOFT - EASILY SCRATCHED BY A KNIFE BLADE

SOFT - CAN BE GOUGED 1/8 INCH TO 1/4 INCH WITH A KNIFE BLADE

VERY SOFT - CAN BE CUT IN HALF OR NEARLY SO WITH A KNIFE BLADE

DISCONTINUITIES

SURFACES REPRESENTING BREAKS OR FRACTURES SEPARATING THE ROCK MASS INTO DISCRETE UNITS. DISCONTINUITIES ARE SUMMARIZED IN THE GENERAL ROCK DESCRIPTIONS AND PRESENTED IN DETAIL ON THE GRAPHIC SUMMARY FOR EACH EXPLORATION. MAJOR FRACTURE ZONES ARE INDICATED BY CROSS HATCHING ON THE GRAPHIC LOG SUMMARIES.

CRACK - A PARTIAL OR INCOMPLETE FRACTURE.

JOINT - A SINGLE FRACTURE ALONG WHICH NO SHEAR DISPLACEMENT HAS OCCURRED. MAY FORM JOINT SETS.

SHEAR - A FRACTURE ALONG WHICH DIFFERENTIAL MOVEMENT HAS TAKEN PLACE SUFFICIENT TO PRODUCE SLICKENSIDES, STRIATIONS OR POLISHING. MAY BE ACCOMPANIED BY A ZONE OF FRACTURED ROCK.

FAULT - A MAJOR FRACTURE ALONG WHICH THERE HAS BEEN APPRECIABLE DISPLACEMENT. OFTEN ASSOCIATED WITH GOUGE AND/OR A SEVERELY FRACTURED ADJACENT ZONE OF ROCK.

SHEAR OF FAULT ZONE - A BAND OR ZONE OF PARALLEL, CLOSELY SPACED SHEARS OR FAULTS.

STRUCTURAL DESCRIPTION

FRACTURES	BEDDING AND FOLIATION	SPACING (*)
VERY CLOSE	VERY THIN	LESS THAN 2 INCHES
CLOSE	THIN	2 INCHES - 1 FOOT
MODERATELY CLOSE	MEDIUM	1 FOOT - 3 FEET
WIDE	THICK	3 FEET - 10 FEET
VERY WIDE	VERY THICK	MORE THAN 10 FEET

* SPACING REFERS TO PERPENDICULAR DISTANCE BETWEEN DISCONTINUITIES.

ATTITUDE DESCRIPTION

ATTITUDE	ANGLE
HORIZONTAL	0 - 5
SHALLOW OR LOW ANGLE	5 - 35
MODERATELY DIPPING	35 - 55
STEEP OR HIGH ANGLE	85 - 90

LEGEND AND NOTES

LEGEND

MAJOR SUBSURFACE STRATA

MISC FILL	BROWN LOOSE TO DENSE COARSE TO FINE SAND, LITTLE COARSE TO FINE GRAVEL, LITTLE SILT WITH INTERMIXED LOAM, BRICK AND WOOD
OUTWASH SAND & GRAVEL	BROWN LOOSE TO MEDIUM DENSE COARSE TO FINE SAND, LITTLE MEDIUM TO FINE GRAVEL, TRACE TO LITTLE SILT
MARINE CLAY	YELLOW-BROWN TO BLUE-GRAY VERY STIFF TO MEDIUM STIFF SILTY CLAY, TRACE COARSE TO FINE SAND AND GRAVEL WITH THIN LENSES OF SILTY FINE SAND AND FINE SANDY SILT
GLACIAL TILL	BROWNISH GRAY MEDIUM TO VERY DENSE COARSE TO FINE SILTY SAND, LITTLE GRAVEL, TRACE CLAY WITH OCCASIONAL SAND, CLAY AND SILT LAYERS WITH COBBLES AND BOULDERS
DECOMPOSED BEDROCK	GRAY VERY DENSE CLAYEY SAND, TRACE TO LITTLE GRAVEL WITH ROCK FRAGMENTS AND BOULDERS
ALTERED ARGILLITE	LIGHT GRAY TO GRAYISH GREEN, FRESH TO MODERATELY WEATHERED, HARD TO SOFT, BRECCIATED WITH REHEALED MACRO SHEARS. LOCAL FRACTURE ZONES, JOINTS AND SHEARS WITH OCCASIONAL CLAY AND GOUGE FILLINGS, QUARTZ, EPIDOTE AND CALCITE VEINS, AND DIABASE STRINGERS.
DIABASE	DARK GREEN TO OLIVE GRAY, FRESH TO SEVERELY WEATHERED, HARD TO SOFT, BRECCIATED WITH REHEALED MACRO SHEARS. LOCAL FRACTURE ZONES, JOINTS AND SHEARS WITH OCCASIONAL CLAY AND GOUGE FILLINGS, PITTING, EPIDOTE AND QUARTZ VEINS WITH IRON OXIDE STAINING AND ARGILLACEOUS INCLUSIONS.

ROCK QUALITY DESIGNATION (RQD)

RQD IS THE SUM IN INCHES OF ALL PIECES OF NX SIZE OR LARGER OF ROCK CORE, FOUR INCHES IN LENGTH AND LONGER, DIVIDED BY THE LENGTH IN INCHES OF THE CORE RUN, AND IS EXPRESSED AS A PERCENTAGE. IF THE CORE WAS BROKEN BY HANDLING OR DRILLING PROCEDURES, THE PIECES WERE FITTED TOGETHER AND CONSIDERED TO BE ONE PIECE, PROVIDING THEY CONSTITUTE THE REQUIRED FOUR INCH LENGTH. WHERE THE RECOVERY FOR A CORE RUN IS GREATER THAN 100 PERCENT, RQD VALUES WERE ADJUSTED TO ACCOUNT FOR THE PROPORTION OF THE CORE LEFT IN THE HOLE FROM THE PREVIOUS RUN. THE LENGTH DETERMINATION WAS MEASURED DOWN THE CENTERLINE OF THE CORE AND WAS CALCULATED FOR EACH CORE RUN.

SAMPLE TYPE

SAMPLE TYPE AND NUMBER (S2) IS INDICATED AT THE LOCATION IT WAS OBTAINED IN THE BOREHOLE.

S	-	2.0 INCH O.D. SPLIT SPOON SAMPLE
S*	-	3.0 INCH O.D. SPLIT SPOON SAMPLE
D	-	2-15/16 INCH O.D. DENISON SAMPLE
P	-	2-1/2 INCH O.D. PITCHER SAMPLE
B	-	BOULDER
R	-	ROCK CORE RUN

* WHERE ONE SAMPLE CONTAINED TWO DIFFERENT SOIL TYPES, IT WAS SPLIT INTO TWO SEPARATE SAMPLES AND DESIGNATED WITH LETTERS A AND B (S2A AND S2B)

TEST TYPE

TEST TYPE AND NUMBER (H3) IS INDICATED AT THE LOCATION IT WAS CONDUCTED OR OBTAINED FOR EITHER IN-SITU TESTS OR SELECTED LABORATORY TEST SAMPLES.

G - GRAIN SIZE ANALYSIS
H - HYDROMETER ANALYSIS
A - ATTERBERG LIMITS
  LL - LIQUID LIMIT
  PL - PLASTIC LIMIT
  PI - PLASTICITY INDEX
LP - LABORATORY PERMEABILITY TEST
FP - FIELD PERMEABILITY TEST
WP - FIELD WATER PRESSURE TEST

ABBREVIATIONS

TR	TRACE	MED	MEDIUM
LI	LITTLE	FRAC	FRACTURED
SME	SOME	SEV	SEVERE OR SEVERELY
V	VERY	LYR	LAYER
SL	SLIGHT OR SLIGHTLY	&	AND
FLEX	FLEXIBLE	W	WITH
OCC	OCCASIONAL OR OCCASIONALLY	OBS	OBSERVATION
MOD	MODERATE OR MODERATELY		

SAMPLE RECOVERY

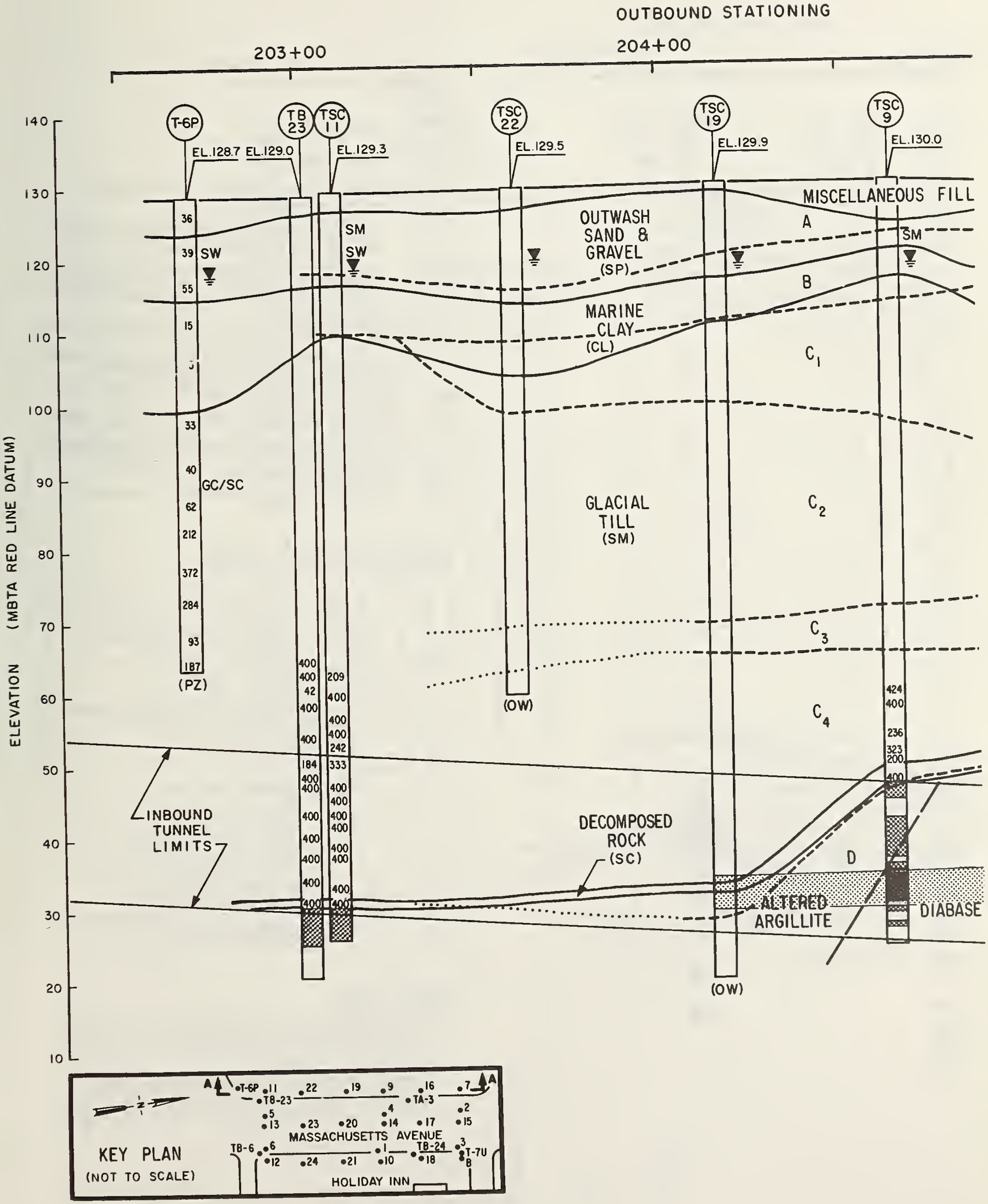
THE TOTAL LENGTH OF THE SOIL OR ROCK SAMPLE RECOVERED IS DIVIDED BY THE TOTAL LENGTH OF PENETRATION OR CORE RUN AND IS EXPRESSED IN PERCENT AND PRESENTED GRAPHICALLY ON THE SUMMARY SHEETS FOR EACH EXPLORATION.

GENERAL NOTES

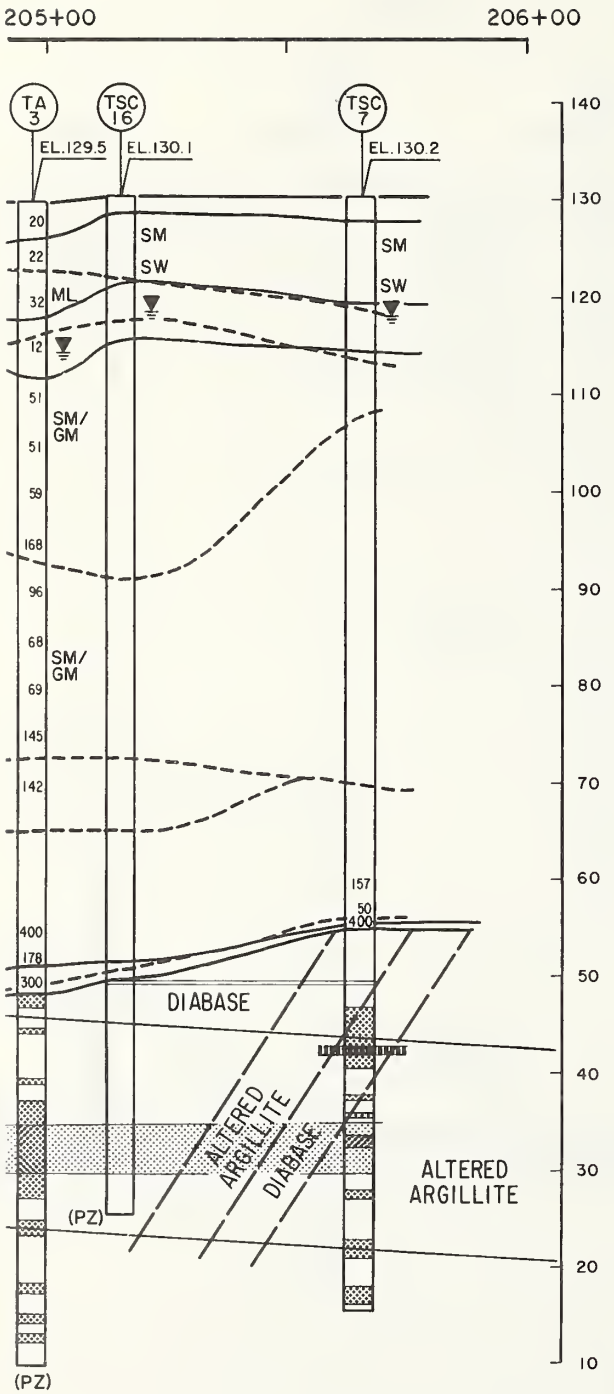
1. LINES DRAWN BETWEEN EXPLORATIONS DIVIDING SUBSURFACE SOIL AND/OR ROCK STRATA ARE INTERPOLATIONS BASED ON AVAILABLE INFORMATION AND MAY NOT AGREE WITH ACTUAL FIELD CONDITIONS.
2. ALL ELEVATIONS ARE IN FEET AND REFER TO MBTA RED LINE DATUM WHICH IS 105.87 FEET BELOW THE USC AND GS MEAN SEA LEVEL DATUM OF 1929.
3. REFER TO APPENDIX B FOR GEOLOGICAL PROFILES.
4. INITIAL WATER LEVEL OBSERVATIONS IN BOREHOLES WERE OBTAINED AT THE COMPLETION OF THE EXPLORATION AND MAY NOT REFLECT THE TRUE GROUND-WATER LEVEL AT BOREHOLE LOCATIONS.
5. OBSERVATION WELL AND PIEZOMETER READINGS ARE SUMMARIZED IN THE REPORT TEXT TABLE 4-2 AND APPENDIX G.
6. THE ELECTRICAL RESISTIVITY AND SPONTANEOUS POTENTIAL DATA ARE OF QUESTIONABLE RELIABILITY DUE TO INTERFERENCE FROM TRANSIENT ELECTRICAL CURRENTS AT THE PROJECT SITE.
7. THE GRAPHIC PRESENTATION OF THE DISCONTINUITIES INDICATES ONLY THE APPROXIMATE LOCATION AND DIP ANGLE OF THE DISCONTINUITY. IT DOES NOT INDICATE ORIENTATION OR BEARING.

APPENDIX B

GEOLOGIC PROFILES


OUTBOUND STATIONING
203+00
204+00
T-6P
TB 23
TSC 11
TSC 22
TSC 19
TSC 9
EL.128.7
EL.129.0
EL.129.3
EL.129.5
EL.129.9
EL.130.0
MISCELLANEOUS FILL
OUTWASH SAND & GRAVEL (SP)
MARINE CLAY (CL)
GLACIAL TILL (SM)
DECOMPOSED ROCK (SC)
ALTERED ARGILLITE
DIABASE
A
B
C1
C2
C3
C4
D
SW
SM
GC/SC
(PZ)
(OW)
INBOUND TUNNEL LIMITS
ELEVATION (MBTA RED LINE DATUM)
KEY PLAN (NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN

PROFILE A-A



SCALE

HOR. 1" = 40'
VER. 1" = 20'

LEGEND

TSC 1 — BORING NUMBER

EL.129.1 — GROUND SURFACE ELEVATION

SM — UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOLS

PENETRATION TEST

22 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES

400 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES WAS IN EXCESS OF 400

WATER LEVEL OBSERVED IN BOREHOLE UPON COMPLETION, MAY-JUNE 1979

WATER LEVEL OBSERVED IN BOREHOLE DURING PUMPING OF DEWATERING WELL DW-5, JULY-AUGUST 1979

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE DETERMINED FROM TEST BORINGS

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM GEOPHYSICAL NUCLEAR LOGGING

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM INCOMPLETE GEOPHYSICAL DATA

BOTTOM OF EXPLORATION

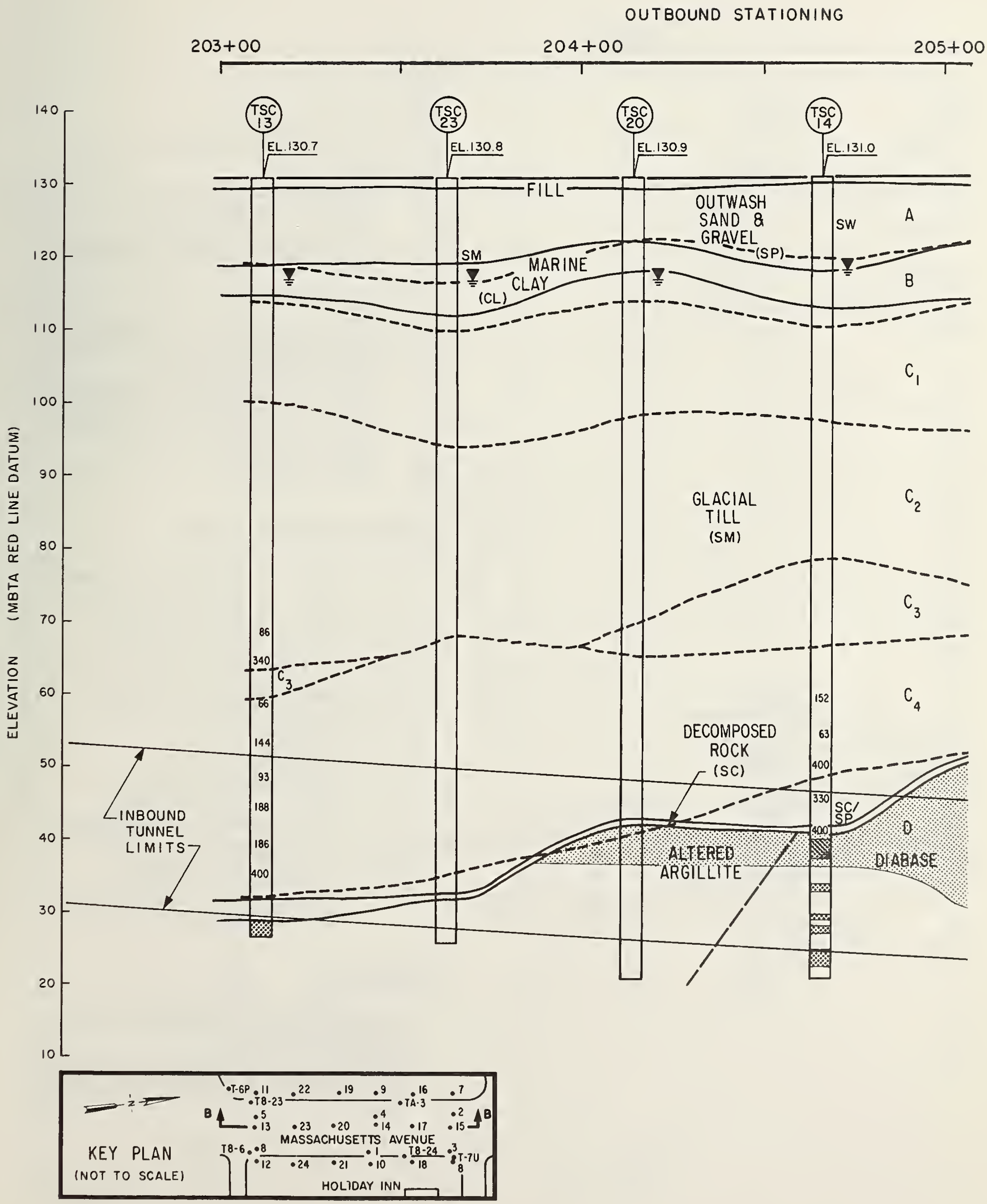
(OW) (PZ) — OBSERVATION WELL OR PIEZOMETER INSTALLED IN COMPLETED BOREHOLE

BOREHOLE GEOPHYSICAL LOGGING

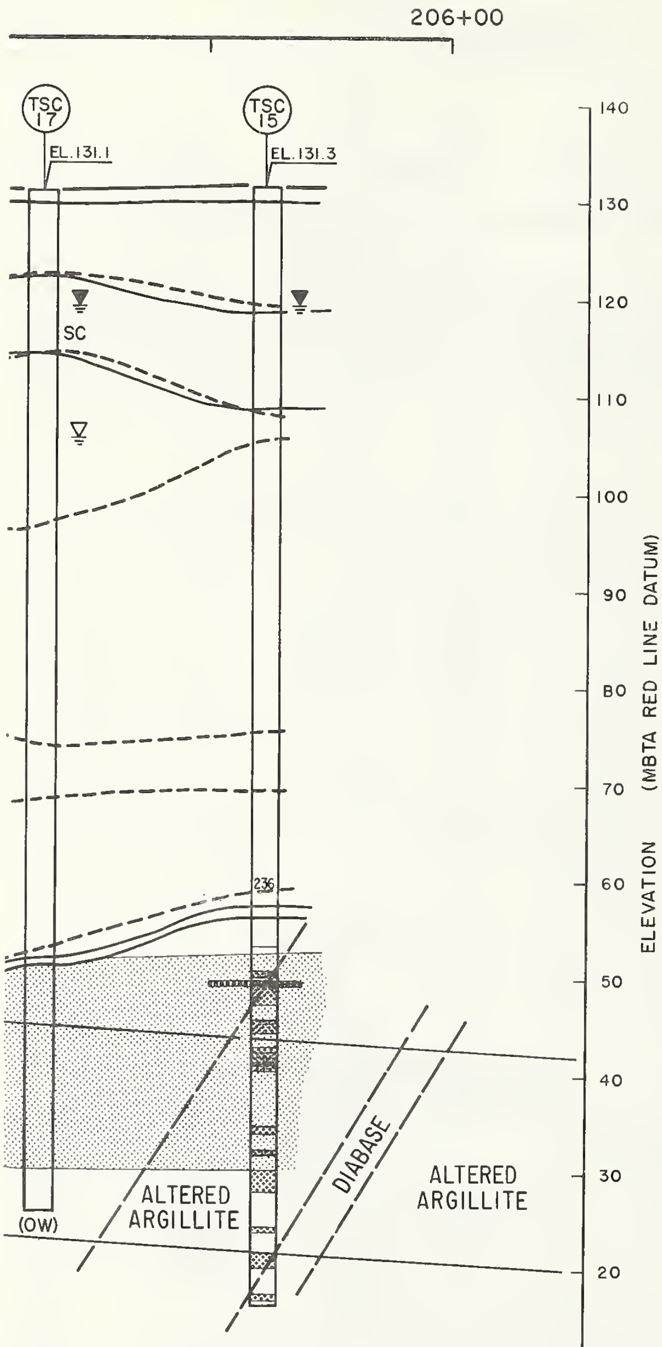
A	APPROXIMATE STRATA CHANGES INFERRED FROM NN AND NATG LOGGING		MISCELLANEOUS FILL AND SAND
B			MARINE CLAY INFERRED FROM NATURAL GAMMA LOGGING
C_1		GLACIAL TILL INFERRED FROM GG, NN AND NATG LOGGING	DENSE SANDY UNIT
C_2			CLAY LENS OR ZONE OF GRAVEL, COBBLES AND BOULDERS
C_3			CLAY UNIT
C_4			DENSE SAND UNIT
D			BEDROCK INFERRED FROM GAMMA-GAMMA LOGGING
	ZONES OF POOR QUALITY ROCK INFERRED FROM SEISMIC CROSS-HOLE SURVEY		
	FRACTURE ZONES INFERRED FROM NUCLEAR BOREHOLE LOGGING		
	MAJOR FRACTURE ZONES DETERMINED FROM RECOVERED BEDROCK CORE		

GENERAL NOTES

1. REFER TO APPENDIX A FOR DETAILED LEGEND AND NOTES.
2. ALL ELEVATIONS ARE IN FEET AND REFER TO MBTA RED LINE DATUM WHICH IS 105.87 FEET BELOW U.S.C. & G.S. MEAN SEA LEVEL 1929.
3. SUBSURFACE STRATIFICATION LINES BETWEEN EXPLORATIONS ARE NECESSARY INTERPOLATIONS OF ALL AVAILABLE INFORMATION AND MAY NOT AGREE WITH ACTUAL FIELD CONDITIONS AT LOCATIONS OTHER THAN AT THE EXPLORATIONS.


OUTBOUND STATIONING
203+00
204+00
205+00
ELEVATION (MBTA RED LINE DATUM)
TSC 13
TSC 23
TSC 20
TSC 14
EL.130.7
EL.130.8
EL.130.9
EL.131.0
FILL
OUTWASH SAND & GRAVEL
(SP)
SW
SM
MARINE CLAY
(CL)
A
B
C_1
C_2
C_3
C_4
GLACIAL TILL
(SM)
DECOMPOSED ROCK
(SC)
SC/SP
D
ALTERED ARGILLITE
DIABASE
INBOUND TUNNEL LIMITS
KEY PLAN
(NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN

PROFILE B-B



SCALE

HOR. 1" = 40'
VER. !" = 20'

LEGEND

- TSC 1 — BORING NUMBER
- EL.129.1 — GROUND SURFACE ELEVATION
- SM — UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOLS
- PENETRATION TEST
  - 22 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES
  - 400 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES WAS IN EXCESS OF 400
- WATER LEVEL OBSERVED IN BOREHOLE UPON COMPLETION, MAY-JUNE 1979
- WATER LEVEL OBSERVED IN BOREHOLE DURING PUMPING OF DEWATERING WELL DW-5, JULY-AUGUST 1979
- INDICATES APPROXIMATE LOCATION OF STRATA CHANGE DETERMINED FROM TEST BORINGS
- INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM GEOPHYSICAL NUCLEAR LOGGING
- INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM INCOMPLETE GEOPHYSICAL DATA
- BOTTOM OF EXPLORATION
- (OW) (PZ) — OBSERVATION WELL OR PIEZOMETER INSTALLED IN COMPLETED BOREHOLE

BOREHOLE GEOPHYSICAL LOGGING

A	APPROXIMATE STRATA CHANGES INFERRED FROM NN AND NATG LOGGING		MISCELLANEOUS FILL AND SAND
B			MARINE CLAY INFERRED FROM NATURAL GAMMA LOGGING
C_1		GLACIAL TILL INFERRED FROM GG, NN AND NATG LOGGING	DENSE SANDY UNIT
C_2			CLAY LENS OR ZONE OF GRAVEL, COBBLES AND BOULDERS
C_3			CLAY UNIT
C_4			DENSE SAND UNIT
D			BEDROCK INFERRED FROM GAMMA-GAMMA LOGGING
	ZONES OF POOR QUALITY ROCK INFERRED FROM SEISMIC CROSS-HOLE SURVEY		
	FRACTURE ZONES INFERRED FROM NUCLEAR BOREHOLE LOGGING		
	MAJOR FRACTURE ZONES DETERMINED FROM RECOVERED BEDROCK CORE		

GENERAL NOTES

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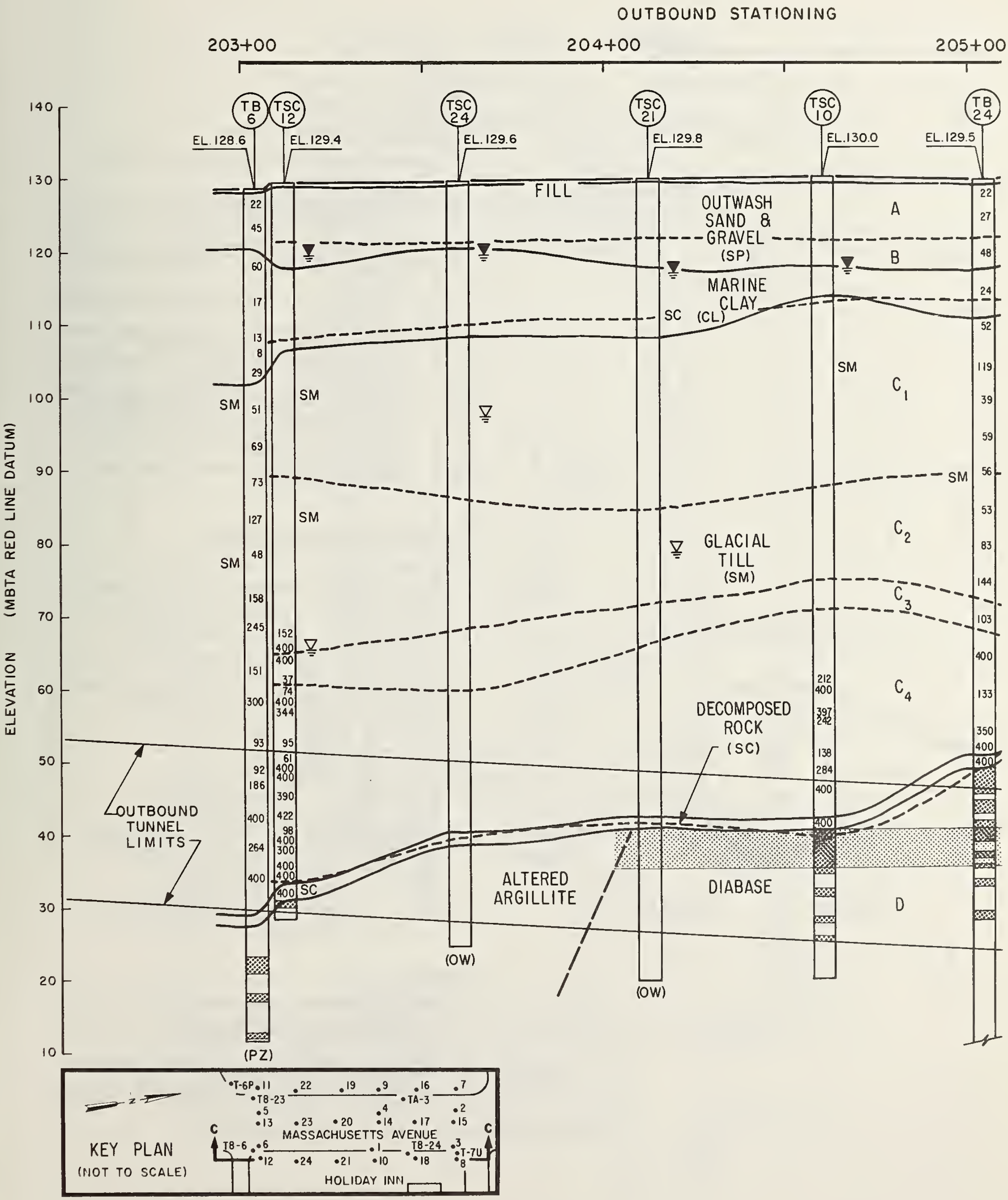
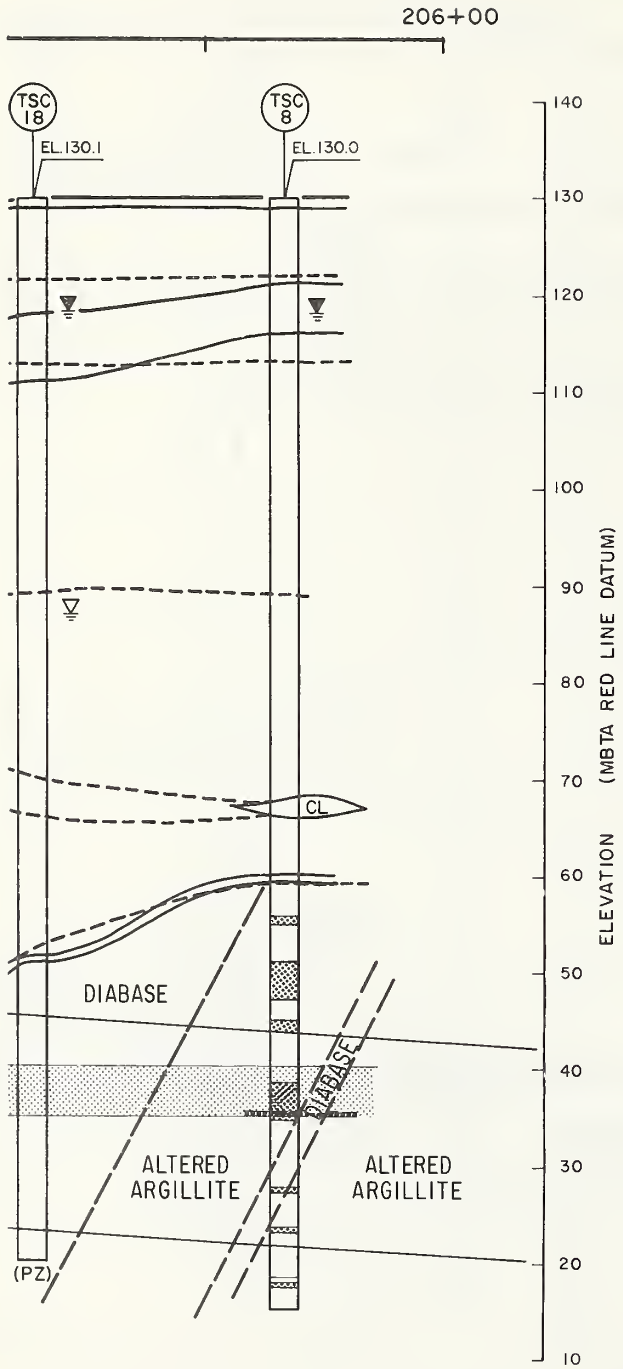


OUTBOUND STATIONING
203+00
204+00
205+00
ELEVATION (MBTA RED LINE DATUM)
TB 6
TSC 12
TSC 24
TSC 21
TSC 10
TB 24
EL. 128.6
EL. 129.4
EL. 129.6
EL. 129.8
EL. 130.0
EL. 129.5
FILL
OUTWASH SAND & GRAVEL (SP)
MARINE CLAY
SC (CL)
SM
GLACIAL TILL (SM)
DECOMPOSED ROCK (SC)
ALTERED ARGILLITE
DIABASE
A
B
C_1
C_2
C_3
C_4
D
OUTBOUND TUNNEL LIMITS
(PZ)
(OW)
KEY PLAN
(NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN

PROFILE C-C



SCALE

HOR. 1" = 40'

VER. 1" = 20'

LEGEND

TSC 1 — BORING NUMBER

EL.129.1 — GROUND SURFACE ELEVATION

SM — UNIFIED SOIL CLASSIFICATION SYSTEM SYMBOLS

PENETRATION TEST

22 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES

400 — NUMBER OF BLOWS REQUIRED TO DRIVE A 2 INCH O.D. SPLIT SPOON SAMPLER ONE FOOT WITH A 300 LB. HAMMER FREE FALLING 24 INCHES WAS IN EXCESS OF 400

WATER LEVEL OBSERVED IN BOREHOLE UPON COMPLETION, MAY-JUNE 1979

WATER LEVEL OBSERVED IN BOREHOLE DURING PUMPING OF DEWATERING WELL DW-5, JULY-AUGUST 1979

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE DETERMINED FROM TEST BORINGS

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM GEOPHYSICAL NUCLEAR LOGGING

INDICATES APPROXIMATE LOCATION OF STRATA CHANGE INFERRED FROM INCOMPLETE GEOPHYSICAL DATA

BOTTOM OF EXPLORATION

(OW) (PZ) — OBSERVATION WELL OR PIEZOMETER INSTALLED IN COMPLETED BOREHOLE

BOREHOLE GEOPHYSICAL LOGGING

A	APPROXIMATE STRATA CHANGES INFERRED FROM NN AND NATG LOGGING		MISCELLANEOUS FILL AND SAND
B			MARINE CLAY INFERRED FROM NATURAL GAMMA LOGGING
C_1		GLACIAL TILL INFERRED FROM GG, NN AND NATG LOGGING	DENSE SANDY UNIT
C_2			CLAY LENS OR ZONE OF GRAVEL, COBBLES AND BOULDERS
C_3			CLAY UNIT
C_4			DENSE SAND UNIT
D			BEDROCK INFERRED FROM GAMMA-GAMMA LOGGING
	ZONES OF POOR QUALITY ROCK INFERRED FROM SEISMIC CROSS-HOLE SURVEY		
	FRACTURE ZONES INFERRED FROM NUCLEAR BOREHOLE LOGGING		
	MAJOR FRACTURE ZONES DETERMINED FROM RECOVERED BEDROCK CORE		

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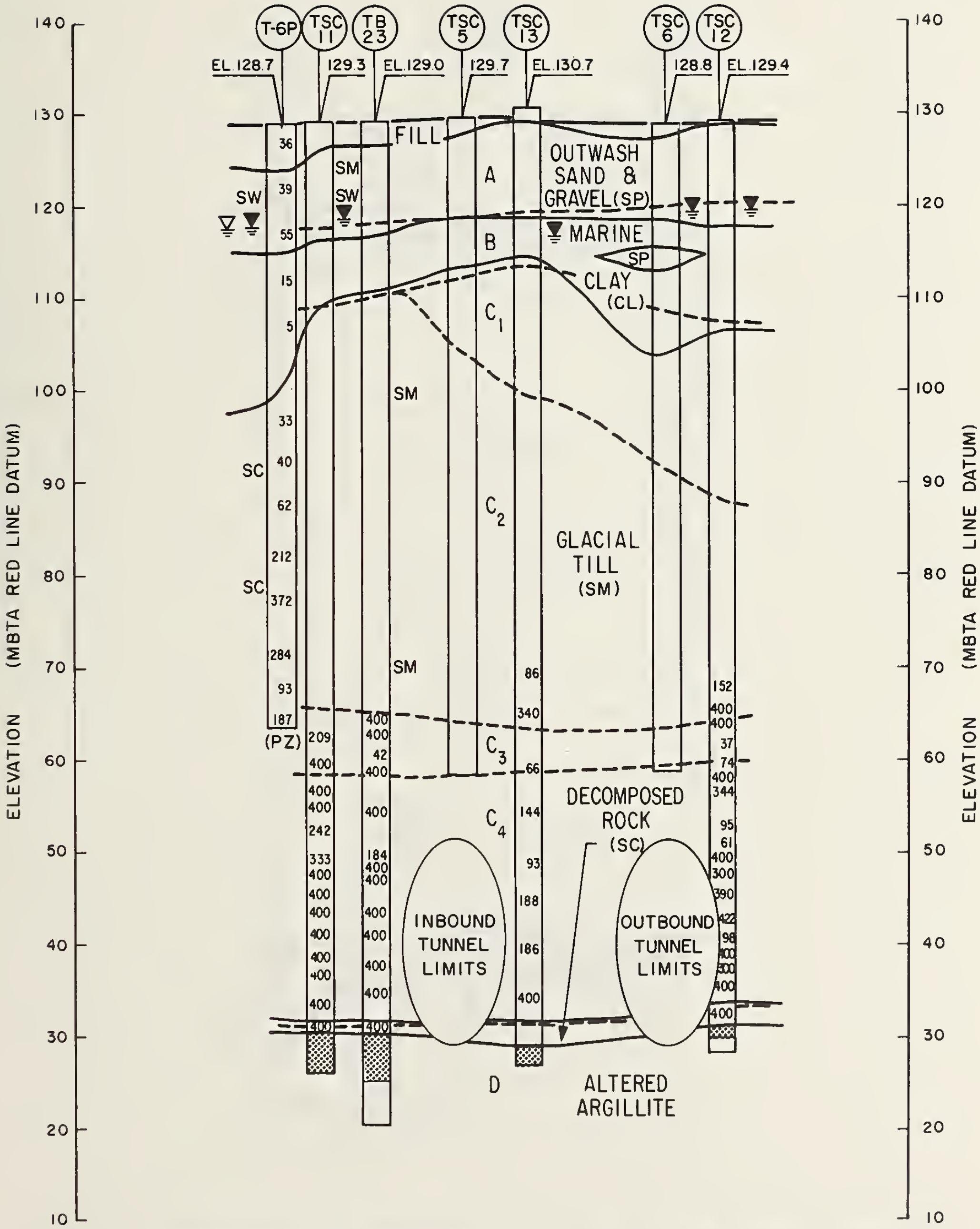
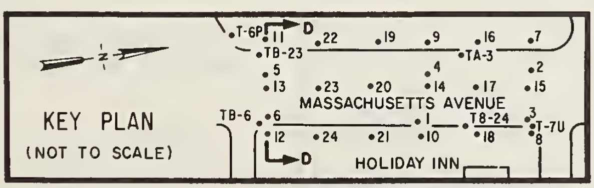
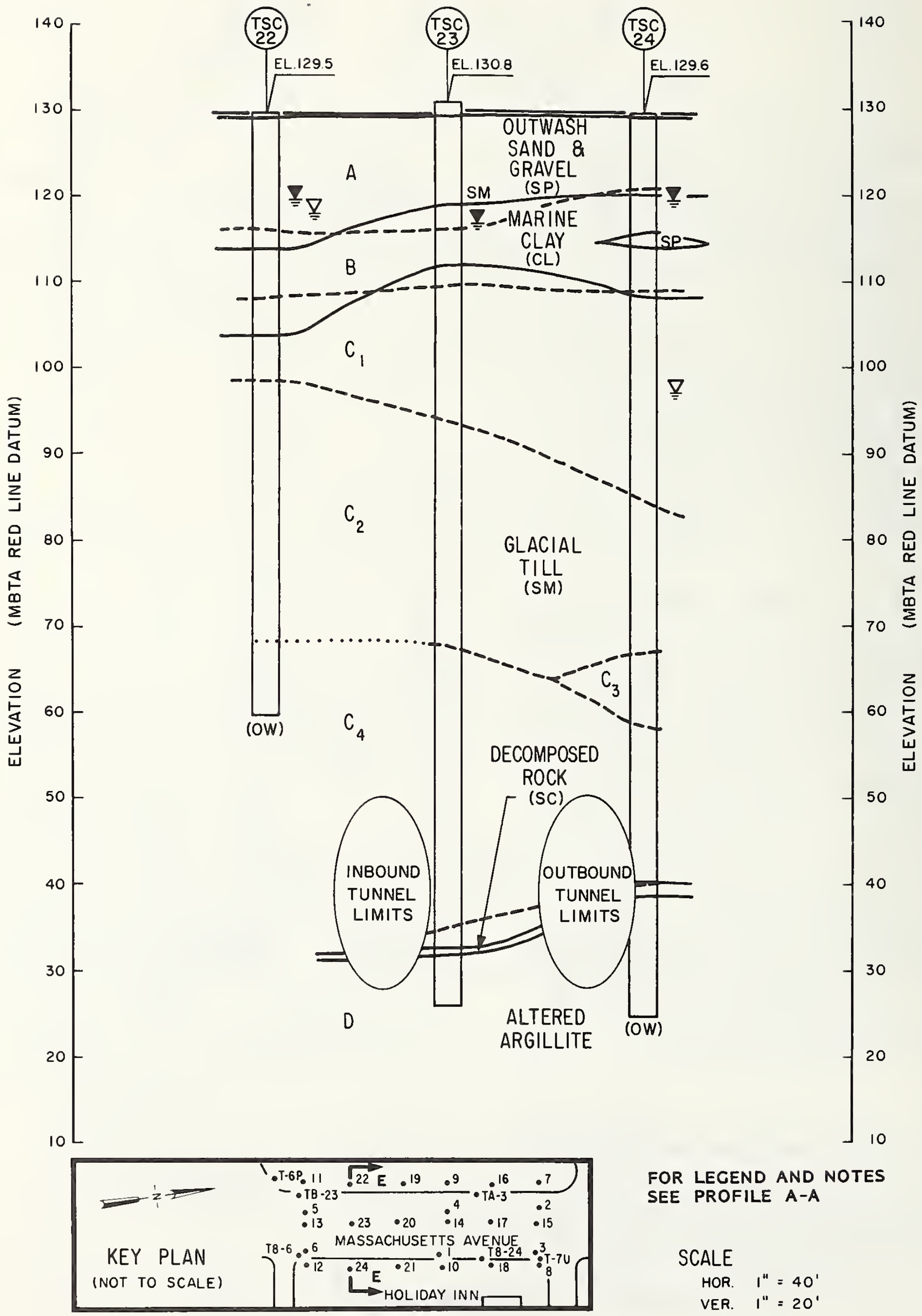




FOR LEGEND AND NOTES SEE PROFILE A-A

SCALE
HOR. 1" = 40'
VER. 1" = 20'

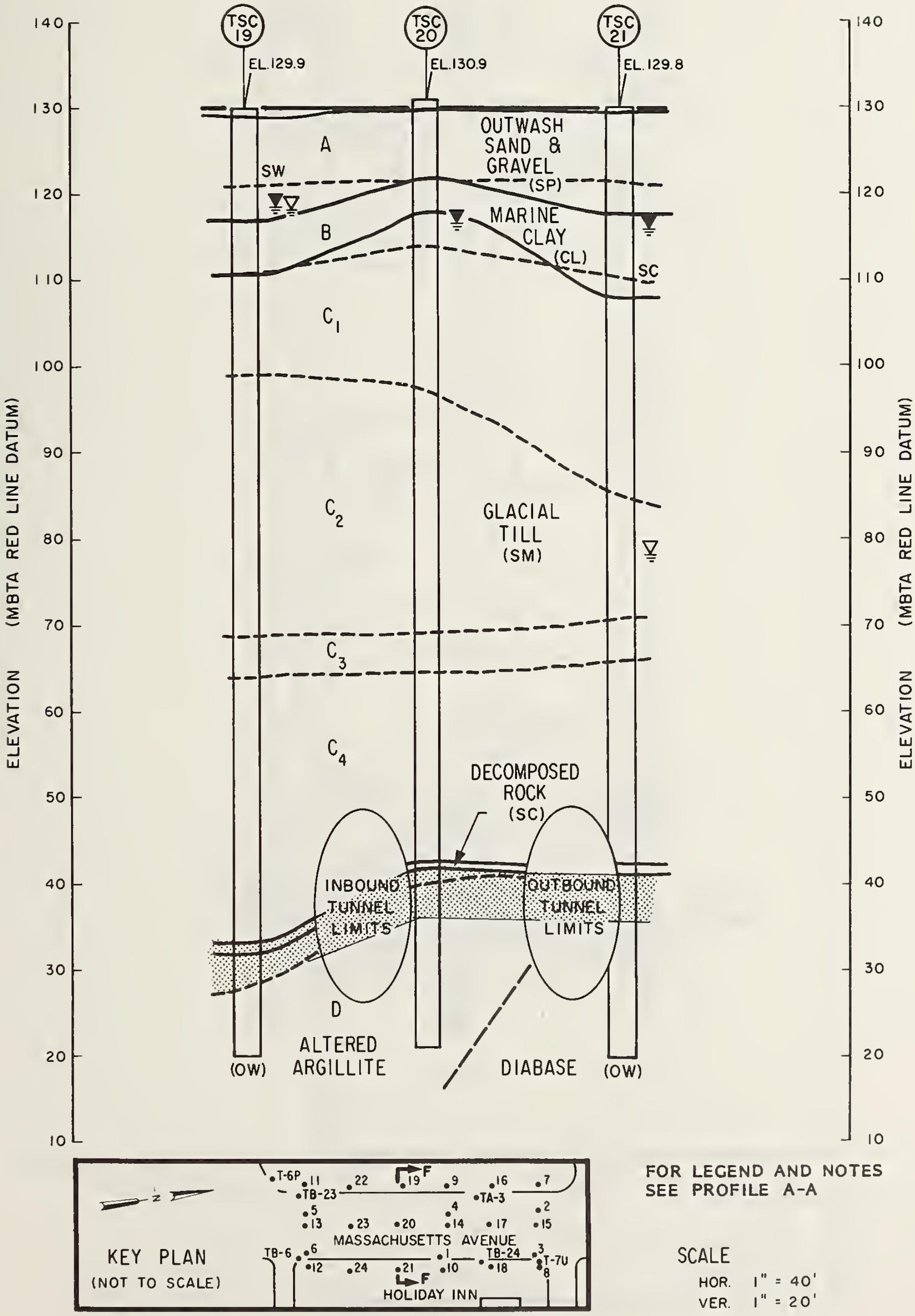
PROFILE D-D



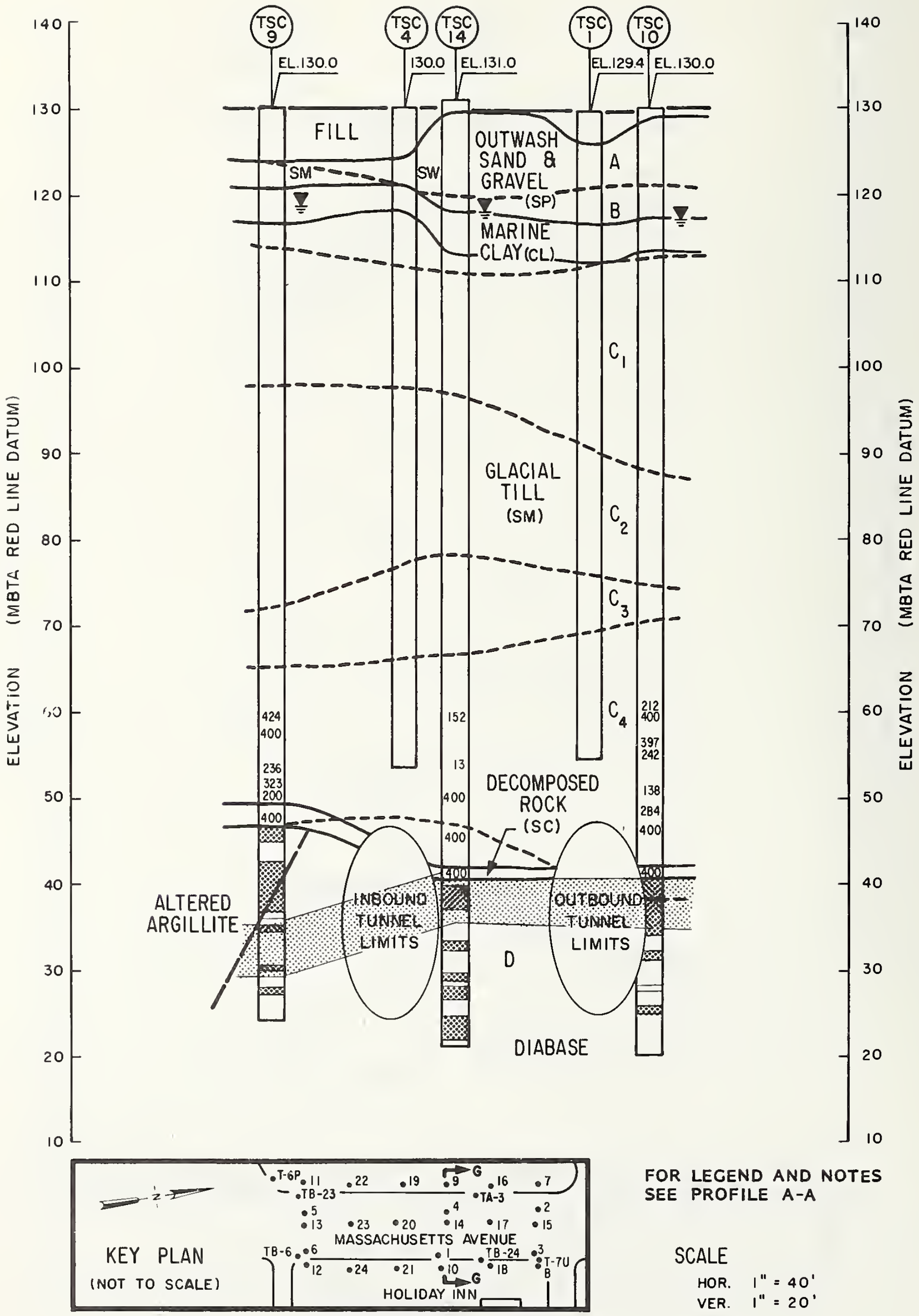
FOR LEGEND AND NOTES SEE PROFILE A-A

SCALE
HOR. 1" = 40'
VER. 1" = 20'

PROFILE E-E


TSC 19
EL.129.9
TSC 20
EL.130.9
TSC 21
EL.129.8
ELEVATION (MBTA RED LINE DATUM)
A
OUTWASH SAND & GRAVEL (SP)
SW
B
MARINE CLAY (CL)
SC
C1
C2
GLACIAL TILL (SM)
C3
C4
DECOMPOSED ROCK (SC)
INBOUND TUNNEL LIMITS
OUTBOUND TUNNEL LIMITS
D
ALTERED ARGILLITE
DIABASE
(OW)
(OW)
KEY PLAN
(NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN
FOR LEGEND AND NOTES SEE PROFILE A-A
SCALE
HOR. 1" = 40'
VER. 1" = 20'

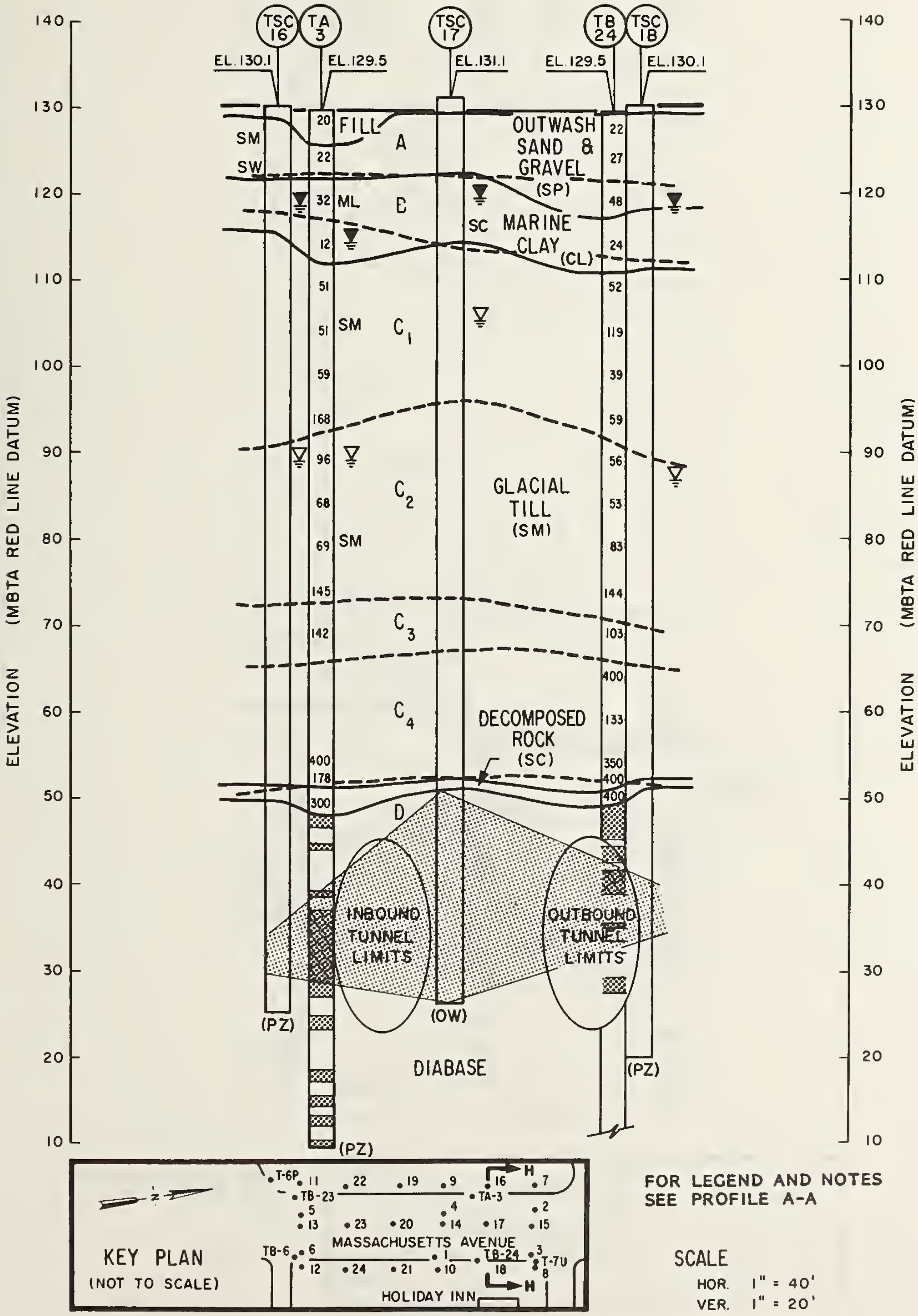
PROFILE F-F



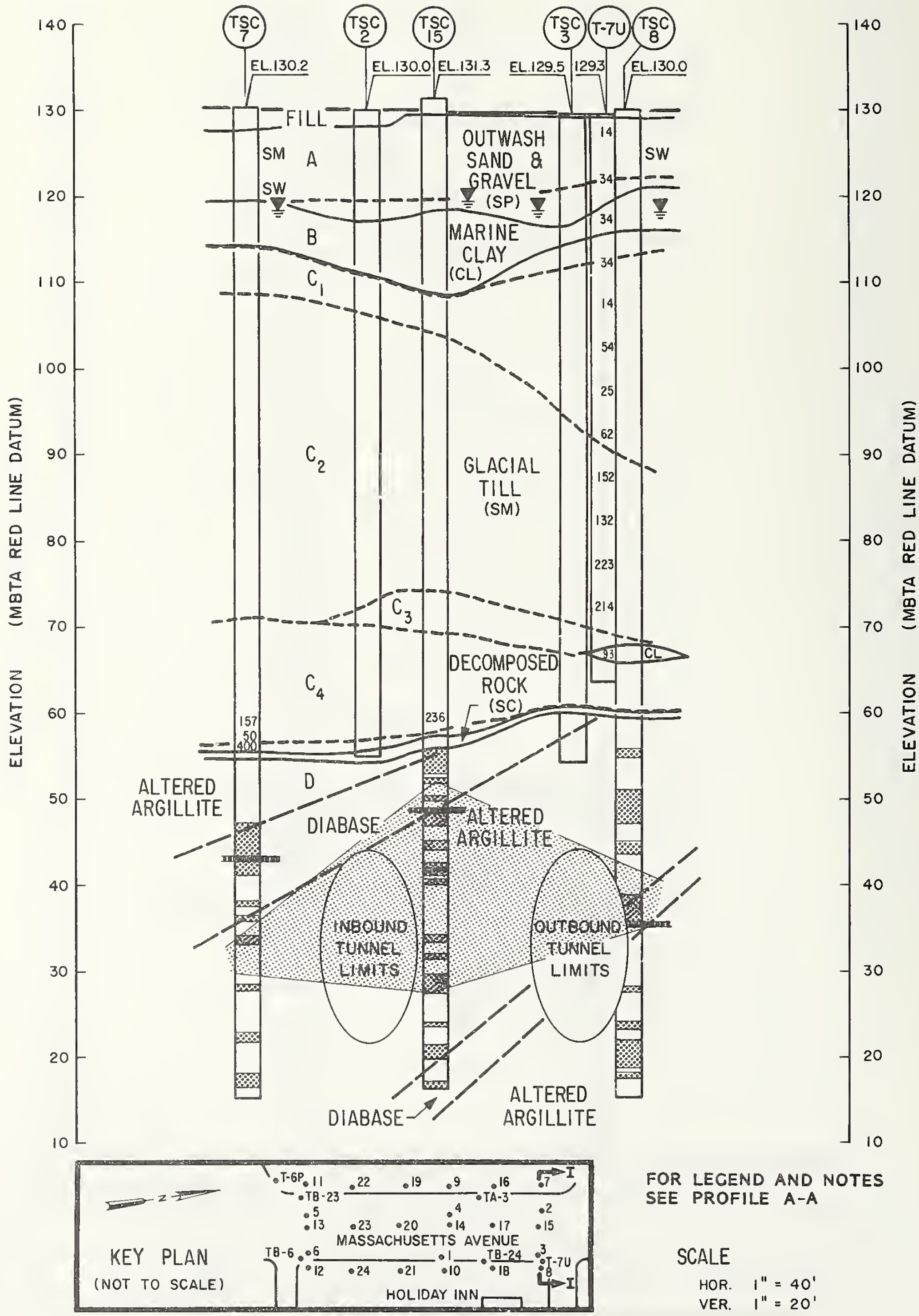
FOR LEGEND AND NOTES SEE PROFILE A-A

SCALE
HOR. 1" = 40'
VER. 1" = 20'

PROFILE G-G


TSC 16
TA 3
TSC 17
TB 24
TSC 18
EL.130.1
EL.129.5
EL.131.1
EL.129.5
EL.130.1
ELEVATION (MBTA RED LINE DATUM)
FILL
A
OUTWASH SAND & GRAVEL (SP)
B
MARINE CLAY (CL)
SC
ML
SM
SW
C_1
C_2
GLACIAL TILL (SM)
C_3
C_4
DECOMPOSED ROCK (SC)
D
INBOUND TUNNEL LIMITS
OUTBOUND TUNNEL LIMITS
DIABASE
(PZ)
(OW)
(PZ)
(PZ)
KEY PLAN
(NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN
FOR LEGEND AND NOTES SEE PROFILE A-A
SCALE
HOR. 1" = 40'
VER. 1" = 20'

PROFILE H-H



TSC 7
TSC 2
TSC 15
TSC 3
T-7U
TSC 8
EL.130.2
EL.130.0
EL.131.3
EL.129.5
129.3
EL.130.0
FILL
SM
A
SW
OUTWASH SAND & GRAVEL (SP)
B
MARINE CLAY (CL)
C_1
C_2
GLACIAL TILL (SM)
C_3
C_4
DECOMPOSED ROCK (SC)
CL
D
ALTERED ARGILLITE
DIABASE
ALTERED ARGILLITE
INBOUND TUNNEL LIMITS
OUTBOUND TUNNEL LIMITS
DIABASE
ALTERED ARGILLITE
ELEVATION (MBTA RED LINE DATUM)
KEY PLAN
(NOT TO SCALE)
MASSACHUSETTS AVENUE
HOLIDAY INN
FOR LEGEND AND NOTES SEE PROFILE A-A
SCALE
HOR. 1" = 40'
VER. 1" = 20'

PROFILE I-I

APPENDIX C

SETTLEMENT PREDICTION

Bechtel Incorporated
Engineers—Constructors

58 Day Street
P.O. Box 487
W. Somerville, MA. 02144
(617) 628-9600

August 29, 1980

U. S. Department of Transportation
Transportation Systems Center
Kendall Square
Cambridge, MA 02142

Attention: Mr. Philip Mattson
Technical Monitor, DTS-741

Subject: Contract DOT/TSC-1570
Innovative Methods of Exploration and
Instrumentation for Transit Tunneling
Soil Deformation Due to Tunneling

Gentlemen:

In accordance with Item 3b. (page 5 of 10) of the subject contract, enclosed is our report predicting ground surface deformation due to tunneling through the "Holiday Inn" test section.

These predictions are based on knowledge of the subsurface conditions as developed during our Stage I subsurface exploration, and available information concerning the tunneling methods and equipment. As pointed out in the enclosure, these predictions consider ground settlement caused by tunneling under "normal conditions". Normal conditions include consideration of subsurface conditions, anticipated construction methods, and workmanship, but excludes random occurrences such as large, sudden ground loss, particularly loss through the face; equipment breakdown; or significant variance from normally expected tunneling methodology. Where such events occur, settlements can be expected to exceed the predicted values. Because of the heavy influence of unpredictable occurrences on the amount of total settlement, the settlements predicted in this report are not appropriate for use as a measure of the effectiveness of the tunneling contractor or as a parameter to evaluate the effects of tunneling. The Stage II final report will more fully discuss the relationships between measured settlements and these predictions.

If you have any questions, please contact us.

Very truly yours,

Harry Sutcliffe
Project Manager

HS:mh

Enclosure

cc: Robert N. Nelson, DTS-852 w/o
D. Thompson, Haley & Aldrich w/a

DEPARTMENT OF TRANSPORTATION
TRANSPORTATION SYSTEMS CENTER

ADVANCED METHODS OF EXPLORATION AND
INSTRUMENTATION FOR TRANSIT TUNNELING
CONTRACT DOT-TSC-1570

PREDICTION OF SURFACE SETTLEMENT DUE TO TUNNELING

A. INTRODUCTION

The Massachusetts Bay Transportation Authority (MBTA) is presently constructing an extension of its Red Line subway system north of Harvard Square in Cambridge, MA. This project has provided the opportunity to evaluate certain aspects of subsurface exploration methods and instrumentation techniques during actual construction. A test section on the Red Line Extension was selected to evaluate innovative methods of subsurface exploration and instrumentation. The overall study is being sponsored by the Transportation Systems Center, Cambridge, Massachusetts.

The purpose of this study is to predict ground surface settlement over the tunnels being driven through the "Holiday Inn" test section. The test section is located between Sta. 203+00 and 206+00 O.B. along Massachusetts Avenue in Cambridge, MA. This settlement prediction will be analyzed using the results of instrumentation installed in the test section in order to better understand the causes and nature of settlements that occur during tunneling.

B. SUBSURFACE CONDITIONS

The subsurface conditions through which the tunnel will pass have been studied in an earlier phase of the work (Ref. 1). The conditions at the tunnel level consist of glacial till at the south end of the test section and bedrock, referred to as Cambridge Argillite, at the north end. Within the test section is a mixed-face transition zone where the tunneling conditions change from soil to rock.

The glacial till consists of a thick deposit of generally a very dense, glacially consolidated mass which directly overlies bedrock. The glacial till, ranging in particle size from silt and clay to boulders, is generally neither stratified nor sorted according to size. Ground water levels in the area, prior to construction, generally ranged from 10 to 15 feet below the ground surface.

The Cambridge Argillite bedrock is a slightly metamorphosed greenish-gray mudstone which varies from soft, severely fractured and weathered near its surface to very hard and fresh with depth. The argillite has been altered by the intrusion of large igneous dikes, consisting primarily of diabase. These dikes have caused extensive local fracturing.

Additional details on the subsurface conditions can be found in Reference 1.

C. TUNNELING METHODS

Based on available information, following is the anticipated tunneling sequence and method to be used in the test section.

1. Rock tunnels, starting from the Garfield Street vent shaft, will approach the test section from the north. These tunnels will be driven to a point at which 5 to 10 feet of rock cover exists over the crown. Construction method for these tunnels is drill and blast with construction support provided by steel ribs.

2. South from the stopping point of full-face rock tunneling an invert drift will be constructed through the mixed-face area. The planned size of the drift is approximately 8 feet by 8 feet. Supports will be placed in the floor of the drift to carry the shield approaching from the south.

3. Soft-ground shield-driven tunnels, starting from the Flagstaff Park portal, will be driven north towards the test section. The shields have the following characteristics:

. Diameter	-	23 ft., 7 3/4 in.
. Overall length	-	21 ft., 2 in.
. Hood length	-	4 ft. bottom 7 ft., 8 in. top
. Overcutter thickness	-	3/8 in.
. Face support	-	8 breasting jacks 4 table jacks 2 half-moon jacks
. Tail skin thickness	-	3/4 in.

The soft ground tunnels have construction support consisting of steel ribs and wood lagging assembled in the tail of the shield and expanded after leaving the shield. Following excavation, the soft ground tunnels will be lined with an 18-inch thick cast-in-place concrete liner.

D. TYPES AND SOURCES OF GROUND LOSS

Ground movements due to tunneling can be separated into two categories (Ref. 2):

. Large sudden ground loss due to raveling, flowing, or running of the ground that progresses above the tunnel crown. This ground loss generally develops at the tunnel face.

- Ground loss under normal conditions. Normal conditions include consideration of subsurface conditions, construction methods and workmanship.

Predictions of ground settlement in this study are based on the second category of ground loss causing settlement. The first category, large, sudden ground loss, is excluded due to its random and unpredictable nature.

The prediction of settlement is based on that ground loss at the tunnel is eventually manifested at the ground surface in the form of the development of a settlement trough. The maximum settlement of which can be estimated by a variety of techniques. The amount of ground loss at the tunnel can be estimated by considering potential void spaces that can occur around the tunnel. These void spaces are in addition to any ground losses at the tunnel face. Potential void spaces are caused by one or more of the following:

1. Pitch and yaw of the shield.

2. Overcutters.

3. Removal of boulders or cobbles present at the cutting edge of the shield, that are not backfilled.

4. Overexcavation beyond the perimeter of the cutting edge not being backfilled.

5. Ribs and lagging construction support not being fully expanded.

6. Downward deflection of the tunnel crown accompanied by lateral deflection at the springline and lateral compression of the soil. Most likely caused by voids developing outside the springline.

Due to the basically granular nature of the glacial till soils the tunnel is being driven through, the development of surface settlement should be relatively rapid. Long-term movements should be relatively small.

Determination of ground loss and resulting settlement in mixed-face or rock tunnels is more difficult to predict than in soft ground tunnels. The magnitude of any losses is highly dependent on construction methods used in tunneling and the speed and effectiveness in which the soil or rock is supported.

E. SETTLEMENT PREDICTION

The magnitude of surface settlement is generally determined by a stepwise process in which the input values for each step are based on observations obtained from case history studies. The major steps generally followed to estimate settlement are:

1. Estimate volume of potential void spaces (ground loss) due to causes listed on page 3.

2. Estimate volume of settlement trough.

3. Determine geometry of settlement trough.

4. Determine magnitude of settlement.

5. Consider the effects of multiple tunnels.

The following sections provide a summary of the steps in the prediction. Significant parameters are given so that they can later be compared with the results obtained from the monitoring of instrumentation installed in the test section.

a. Estimate of Ground Loss at Tunnel

Cording, et al. (Ref. 2), estimates that ground loss for well constructed tunnels is in the range of 1 to 2 percent of tunnel volume. For tunnels on the WMATA (Washington, D.C.) project average measured ground loss was 1.5 percent (from Ref. 2, pp. 3-4 and 3-5). A value of ground loss of 2 percent was selected.

b. Estimate of Volume of Settlement Trough

In granular soils, the volume of the settlement trough at the surface is usually less than the volume of ground loss at the tunnel. This is due to bulking or volume expansion in dense granular soils. For a ground loss of 2 percent at the tunnel, this results in a trough volume of 0.8 percent (Ref. 2, p. 4-7).

For twin tunnels, an additional amount of apparent ground loss is caused by the interference of one tunnel with the other. This interference may cause an additional deflection of the lining of the first tunnel, compression of the pillar between the two tunnels, and a volume decrease in the previously expanded region over the first tunnel (Ref. 2, p. 6-1). For the tunnels at Sta. 203+00, with pillar width (d') of 29 feet, diameter (2R) of 23.6 feet, and depth to centerline of tunnel of 89 feet, the interference settlement is about 50 percent (Ref. 2, p. 6-3). The following summarizes settlement trough volume estimates:

Volume first tunnel	- 0.8%
Volume second tunnel	- 0.8%
Volume interference	- 0.4%
Total volume	- 2.0%

c. Geometry of Settlement Trough

Based on observations of many tunnels which are reported in the literature, the shape of the settlement trough approximates the shape of a normal probability curve. The volume of this curve is defined as $V_S = 2.5 \times i \times \delta_{max}$

Where:

V_S - volume of settlement trough

i - distance from centerline of trough to point of inflection

δ_{max} - settlement at center of trough.

d. Magnitude of Settlement

The magnitude of settlement was determined by solving the equation for the normal probability curve with parameters correlated with tunnel radius, depth and type of soil. The resulting settlement for the soft ground settlement in the test section is approximately 0.6 inches for a single tunnel. When consideration is made for twin tunnels, the settlement of the centerline of the settlement trough is approximately 1 inch.

The estimated settlement of 1 inch for twin tunnels compares well with the measured settlement of almost 1 inch on the Edmonton, Canada, metro twin tunnels driven through glacial till (Ref. 3).

As discussed previously, the settlement of tunnels driven through mixed face conditions is not readily quantifiable. The perimeter of soft ground is smaller but the degree of disturbance is likely to be greater due to the more difficult construction environment.

Therefore, the magnitude of ground settlement over the centerline of mixed face tunnels in the test section area is estimated to be slightly greater (in the range of 1/4 inch greater) than for soft ground tunnels.

The settlement of the ground surface in the rock tunneling area is estimated to be less than 1/4 inch.

SUMMARY OF GROUND SETTLEMENT ESTIMATES OVER CENTERLINE OF TUNNELS

- Soft Ground

  Single tunnel - ≈ 0.6 inch
  Twin tunnels - ≈ 1 inch

- Mixed Face

  Similar to slightly greater than soft ground

- Rock - < 1/4 inch

REFERENCES

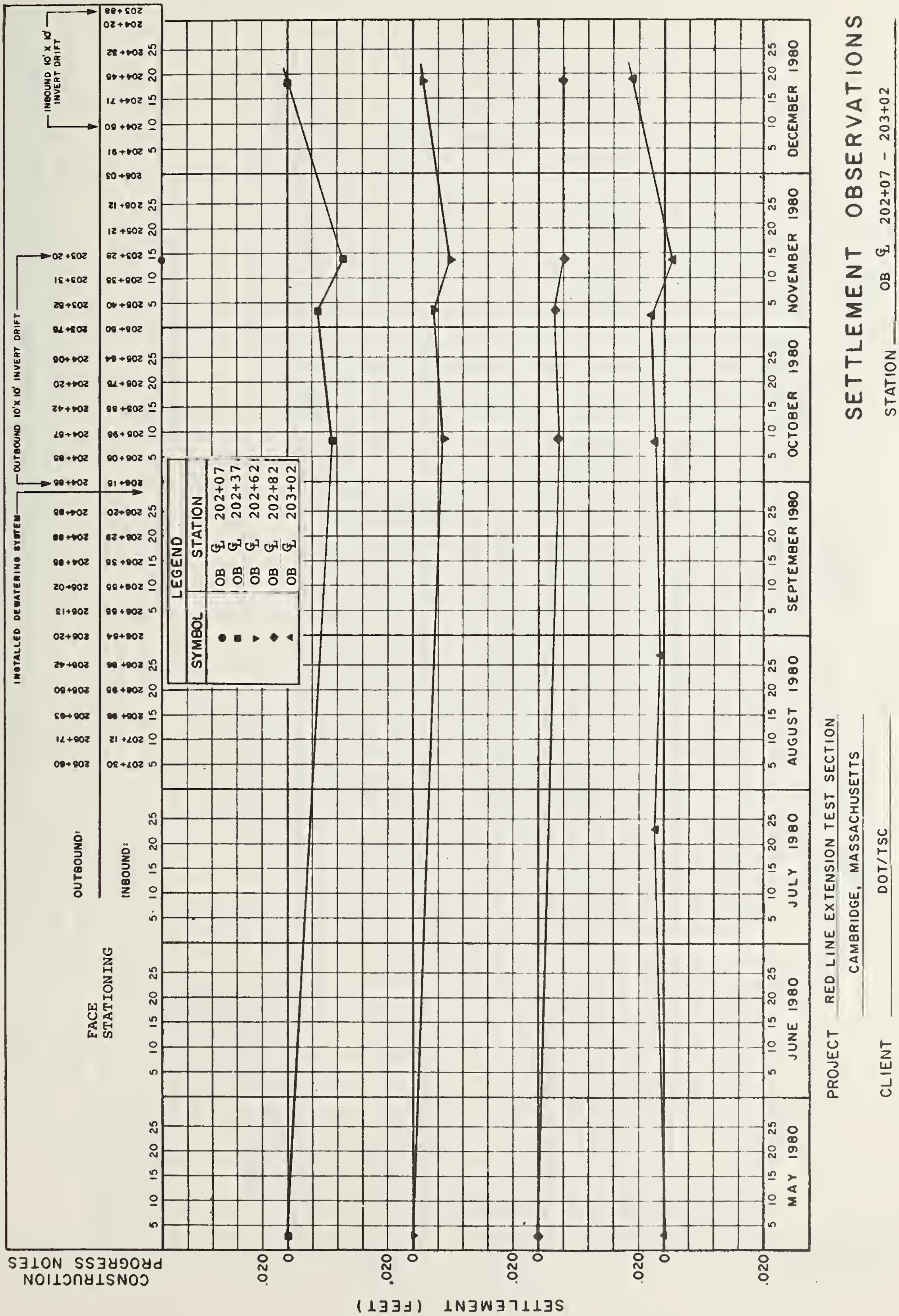
1. Thompson, D. E., et al., "Field Evaluation of Advanced Methods of Subsurface Exploration for Transit Tunneling," Report for U.S. Department of Transportation/Transportation Systems Center, Bechtel Incorporated, Report No. DOT-TSC-UMTA-80-1, March 1980.

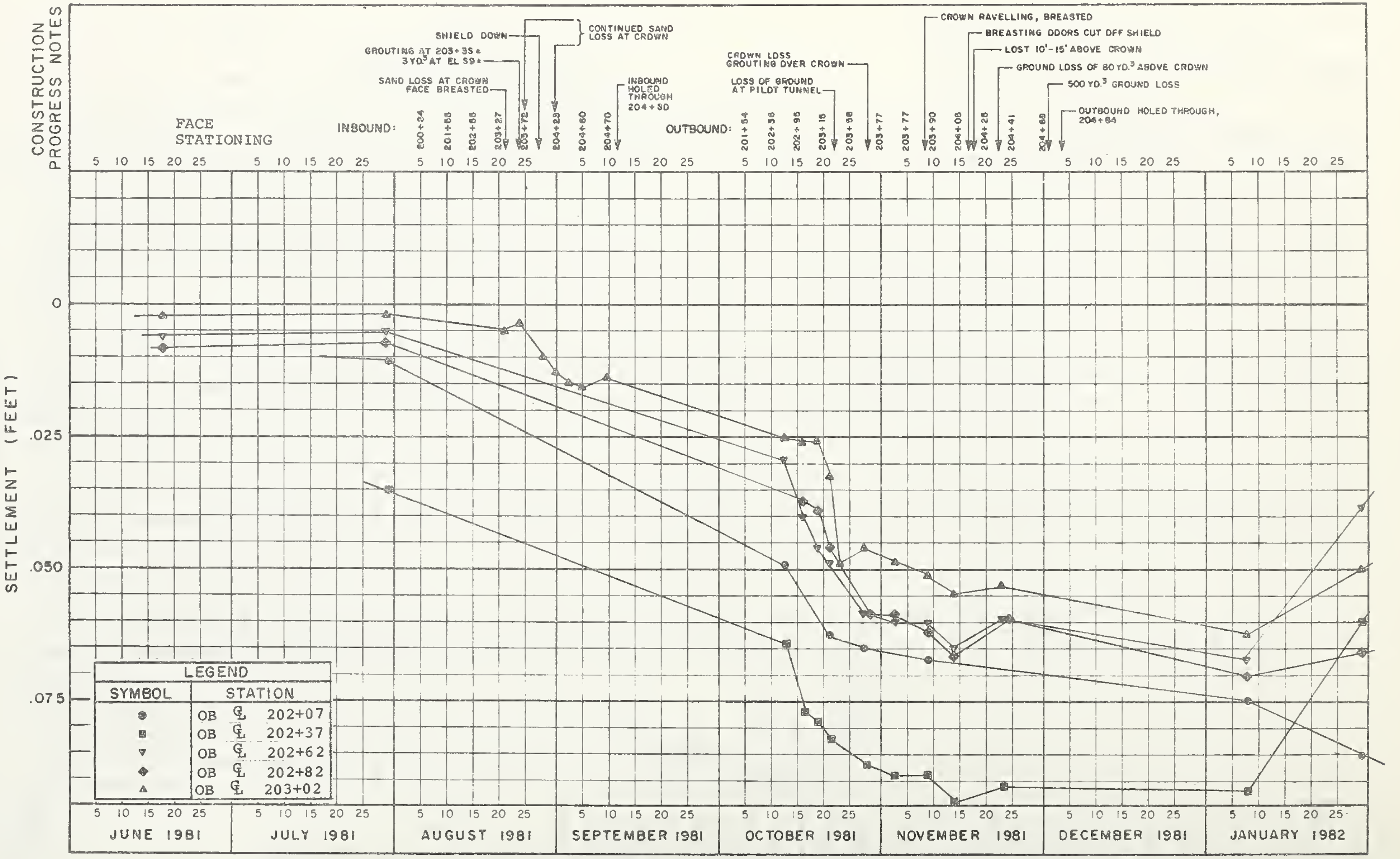
2. Cording, E. J., et al., "Displacements Around Tunnels in Soil," Report for U.S. Department of Transportation, University of Illinois, Report No. DOT-TST 76T-22, August 1976.

3. Eisenstein, Z. and S. Thamson, "Geotechnical Performance of a Tunnel in Till," Canadian Geotechnical Journal, Volume 15, Number 3, August 1978.

APPENDIX D

SURFACE AND BUILDING POINT SETTLEMENT OBSERVATIONS


SETTLEMENT OBSERVATIONS
STATION OB ℄ 202+07 - 203+02
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
CONSTRUCTION PROGRESS NOTES
SETTLEMENT (FEET)
FACE STATIONING
OUTBOUND:
INBOUND:
INSTALLED DEWATERING SYSTEM
OUTBOUND 10'x10' INVERT DRIFT
INBOUND 10' x 10' INVERT DRIFT
LEGEND
SYMBOL
STATION
OB ℄ 202+07
OB ℄ 202+37
OB ℄ 202+62
OB ℄ 202+82
OB ℄ 203+02
MAY 1980
JUNE 1980
JULY 1980
AUGUST 1980
SEPTEMBER 1980
OCTOBER 1980
NOVEMBER 1980
DECEMBER 1980
0
.020

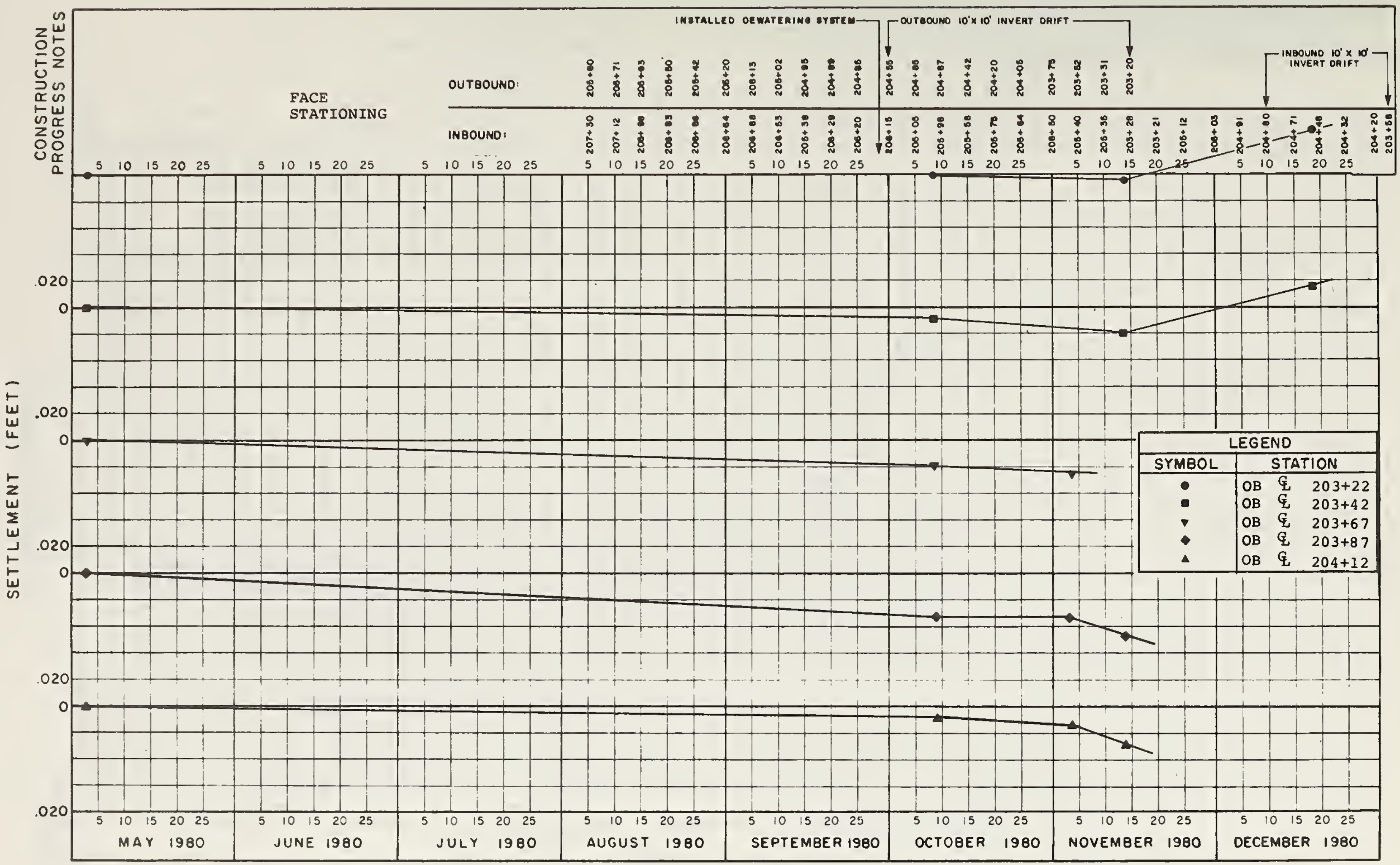


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 202+07 – 203+02



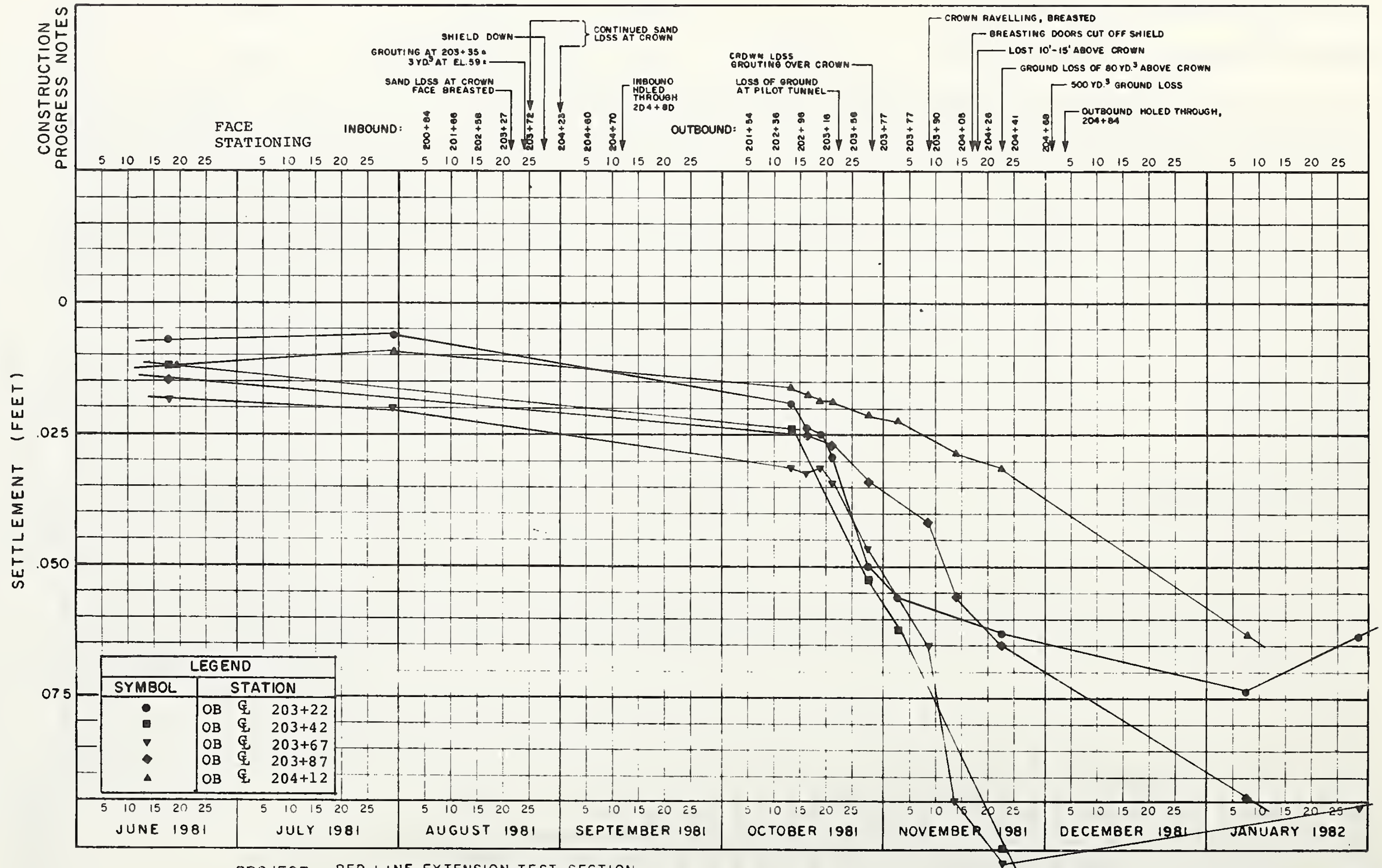
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 203+22 - 204+12



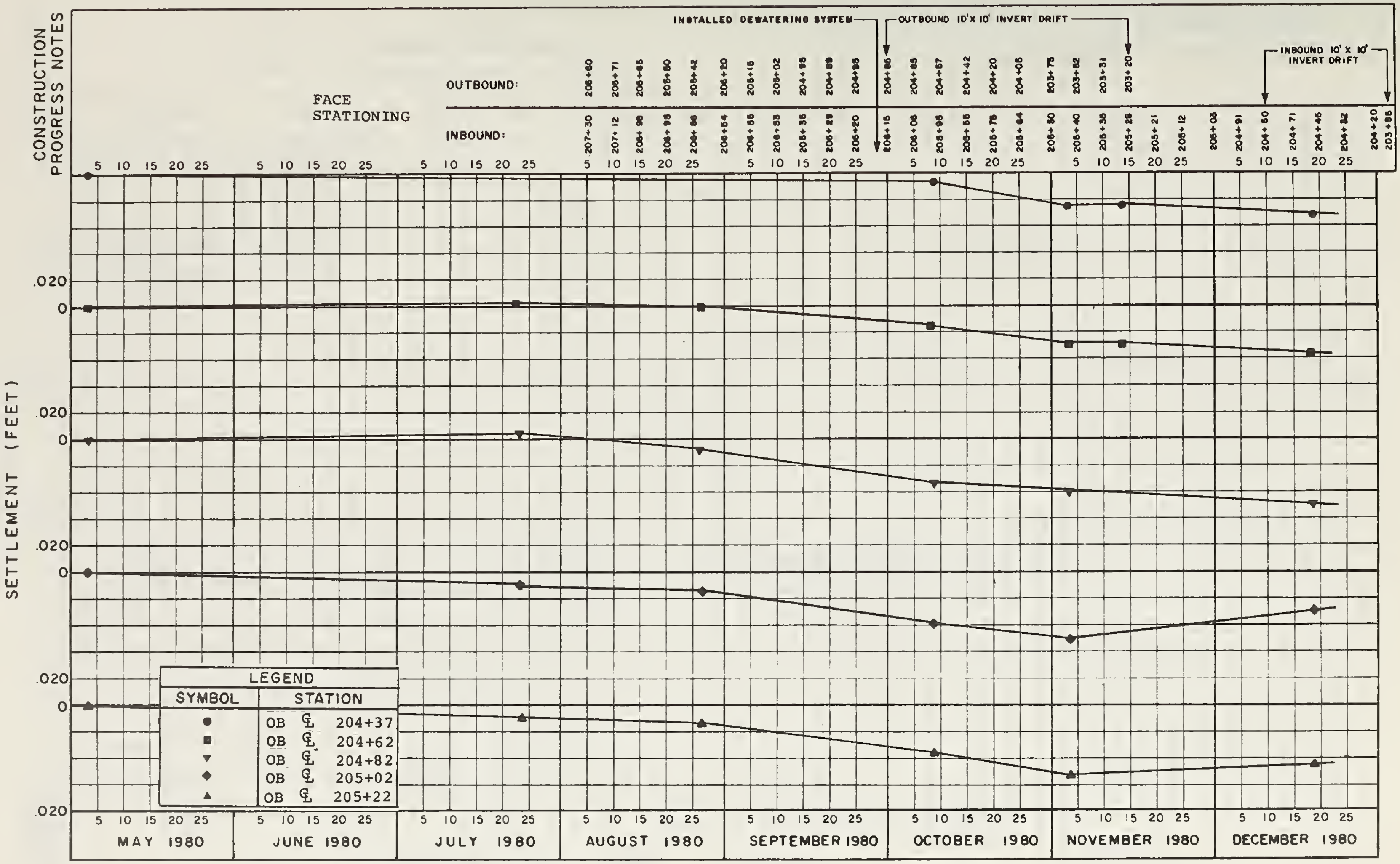
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 203+22 - 204+12

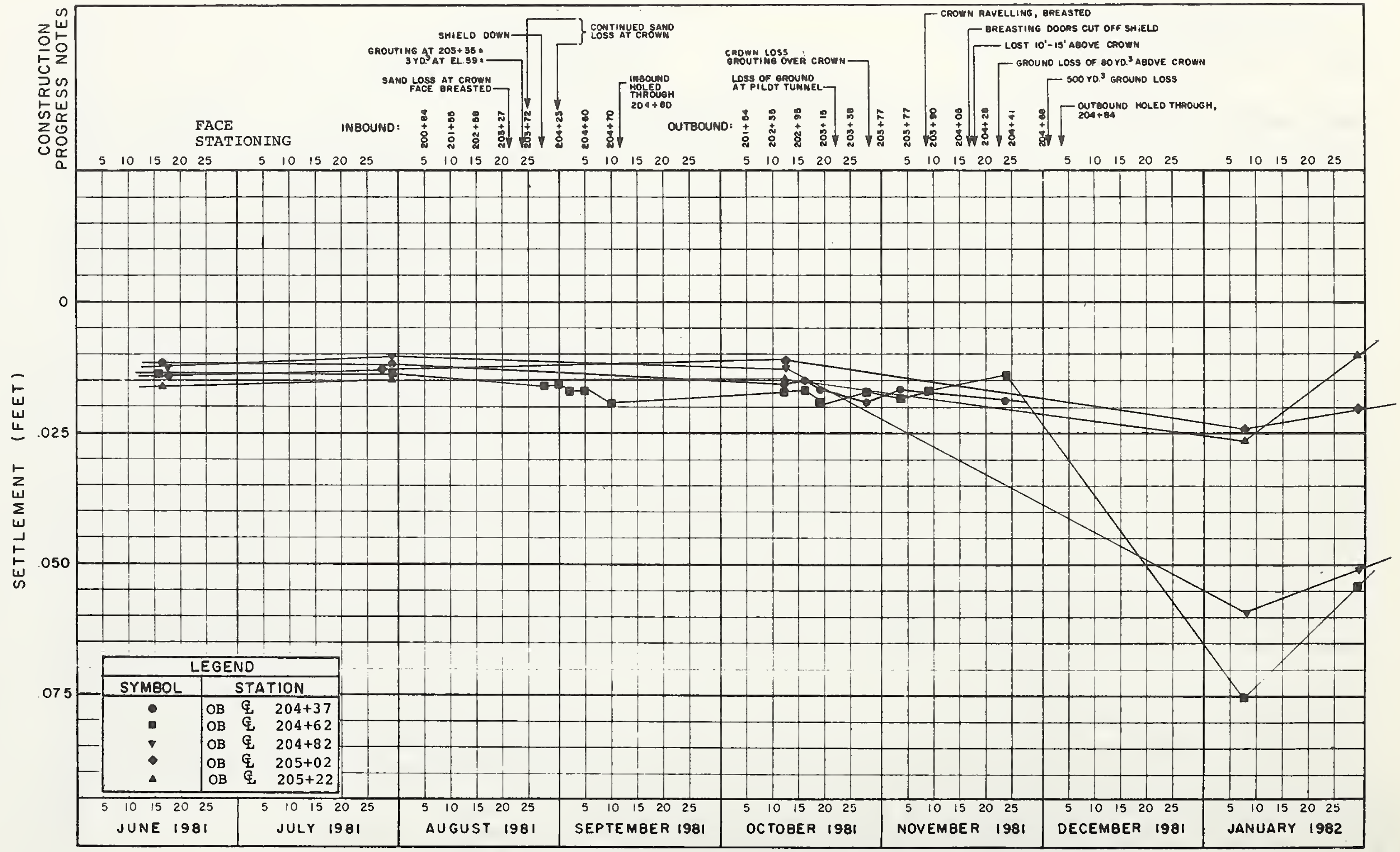


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 204+37 - 205+22

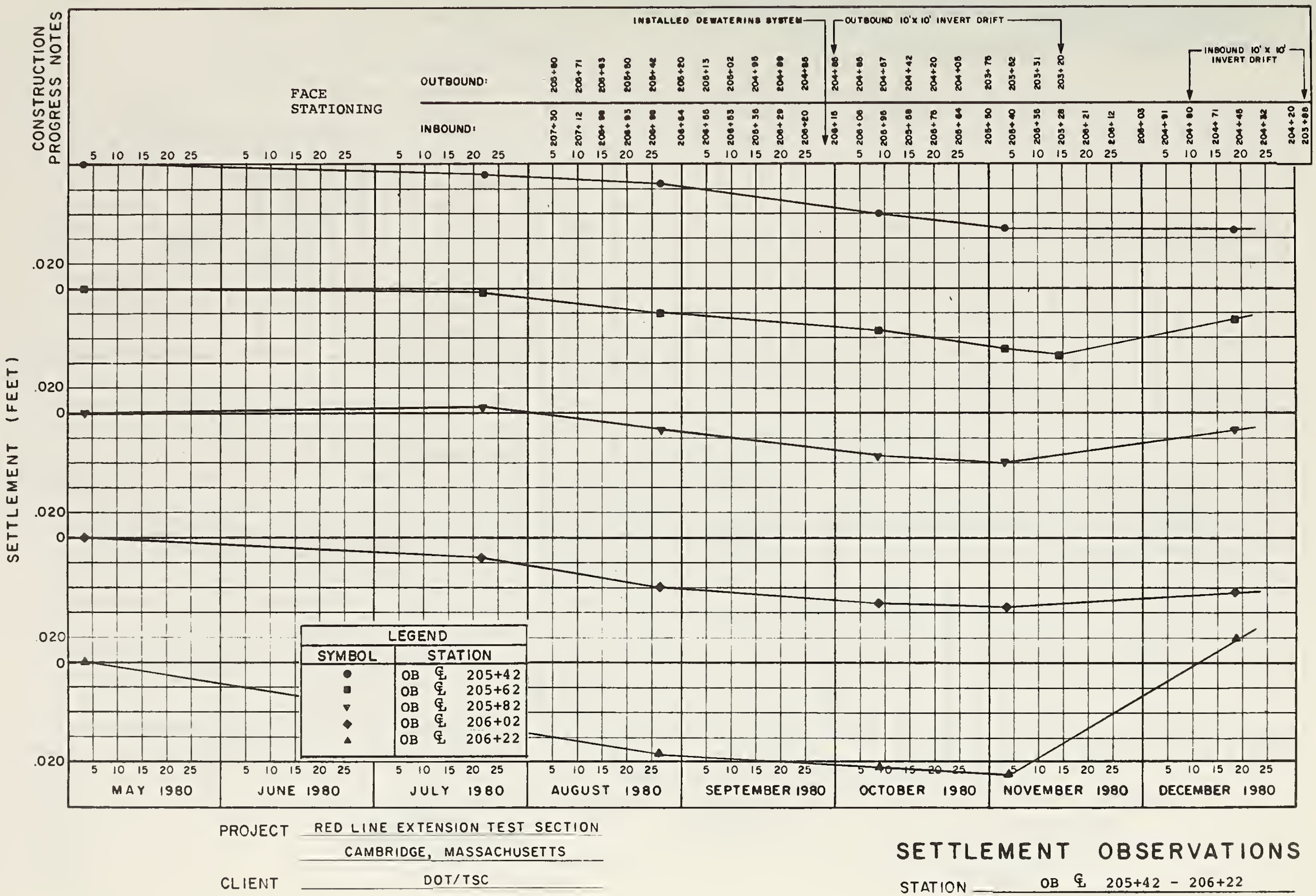


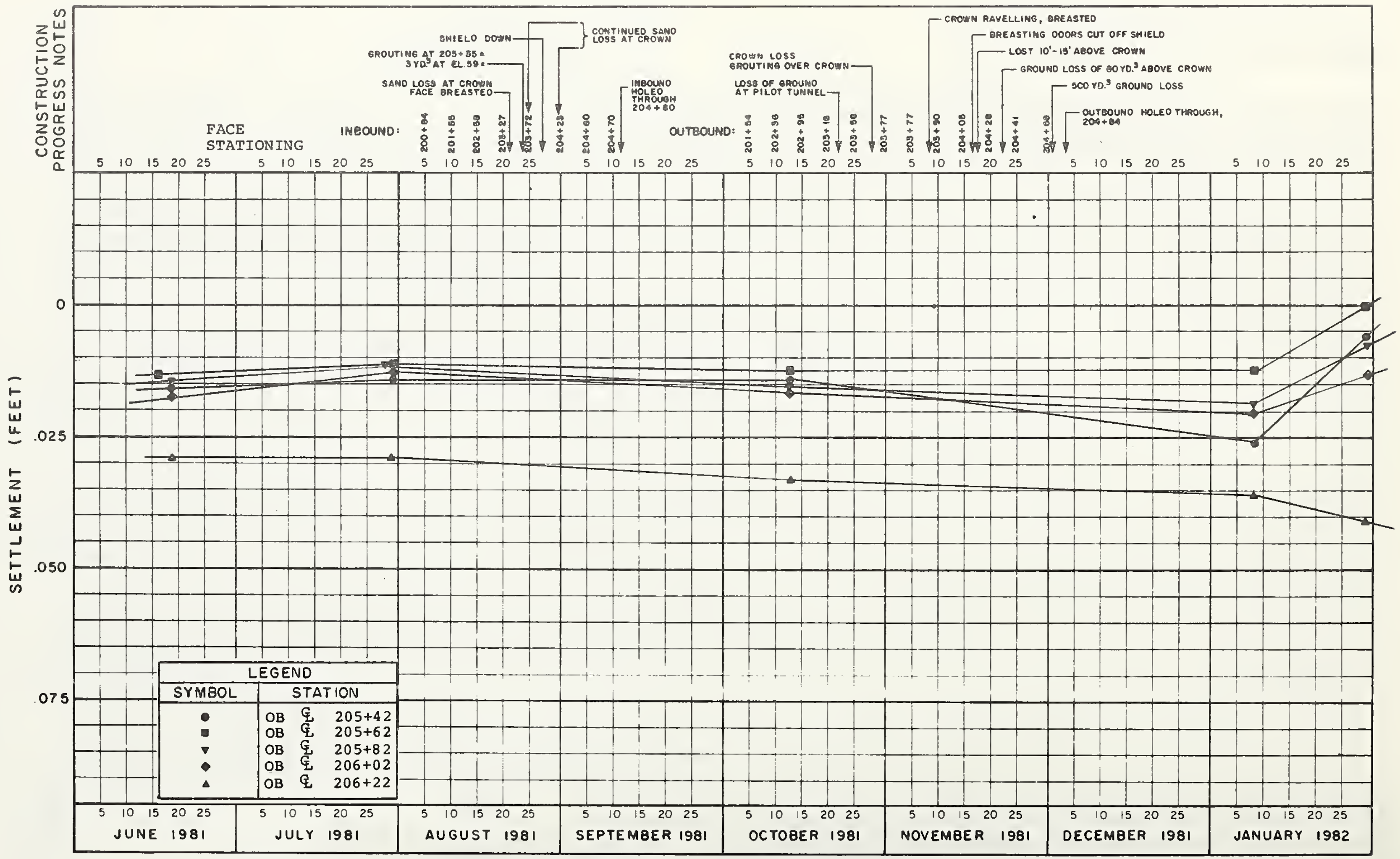
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 204+37 - 205+22


CONSTRUCTION PROGRESS NOTES
FACE STATIONING
OUTBOUND:
INBOUND:
INSTALLED DEWATERING SYSTEM
OUTBOUND 10'x 10' INVERT DRIFT
INBOUND 10'x 10' INVERT DRIFT
SETTLEMENT (FEET)
LEGEND
SYMBOL
STATION
OB ℄ 205+42
OB ℄ 205+62
OB ℄ 205+82
OB ℄ 206+02
OB ℄ 206+22
MAY 1980
JUNE 1980
JULY 1980
AUGUST 1980
SEPTEMBER 1980
OCTOBER 1980
NOVEMBER 1980
DECEMBER 1980
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
SETTLEMENT OBSERVATIONS
STATION OB ℄ 205+42 - 206+22

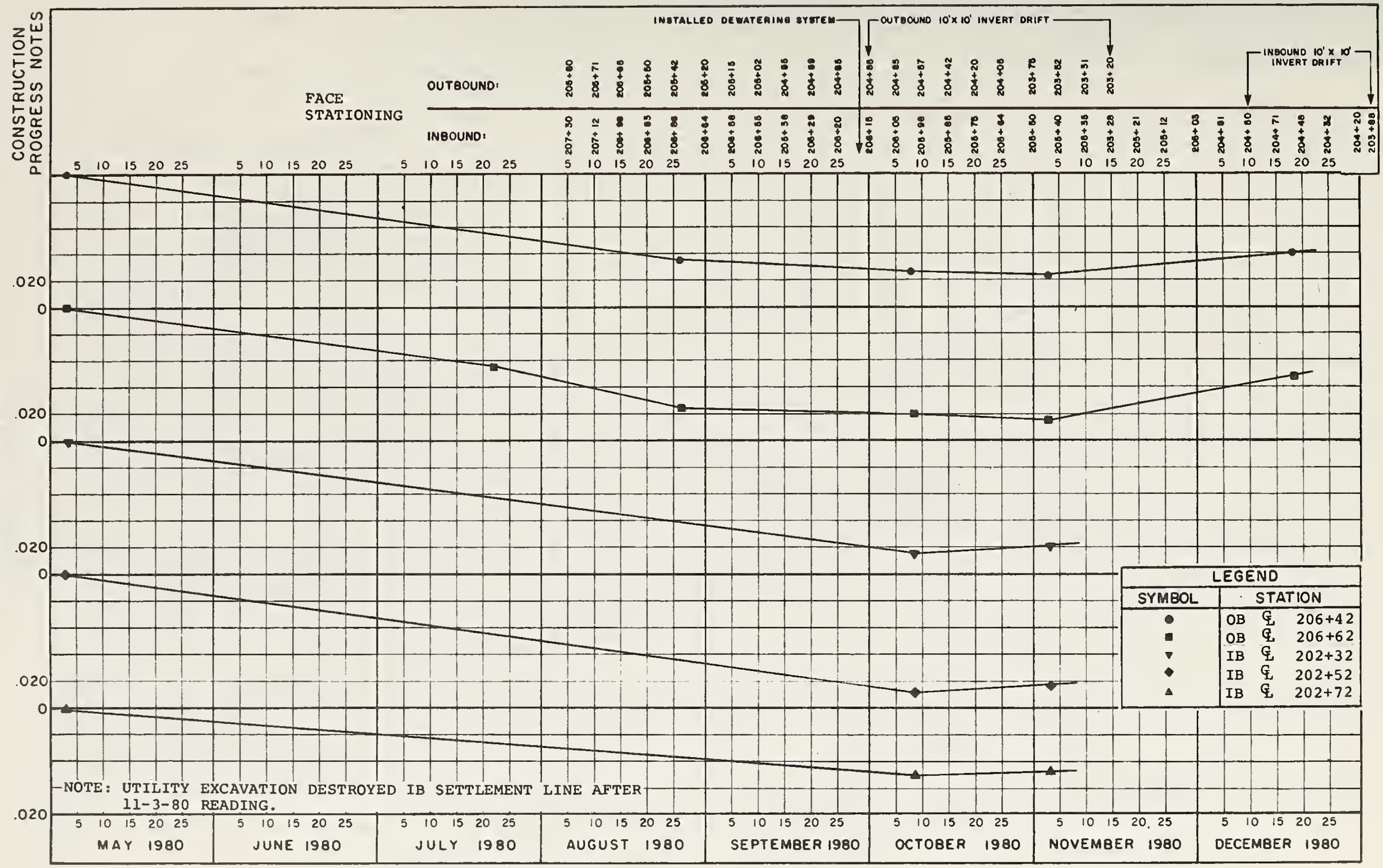


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 205+42 - 206+22

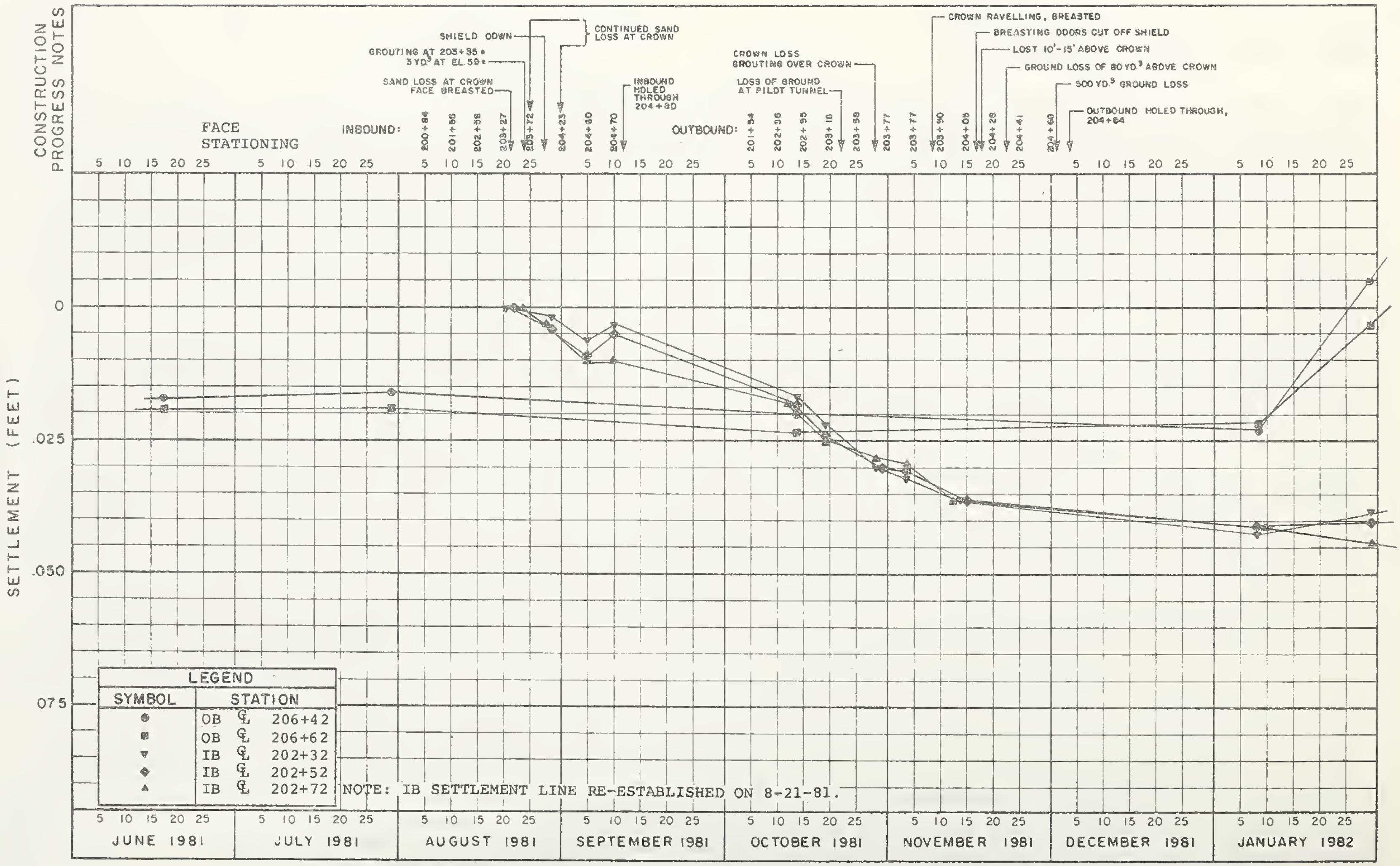


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 206+42 - 206+62, IB ℄ 202+32 - 202+72

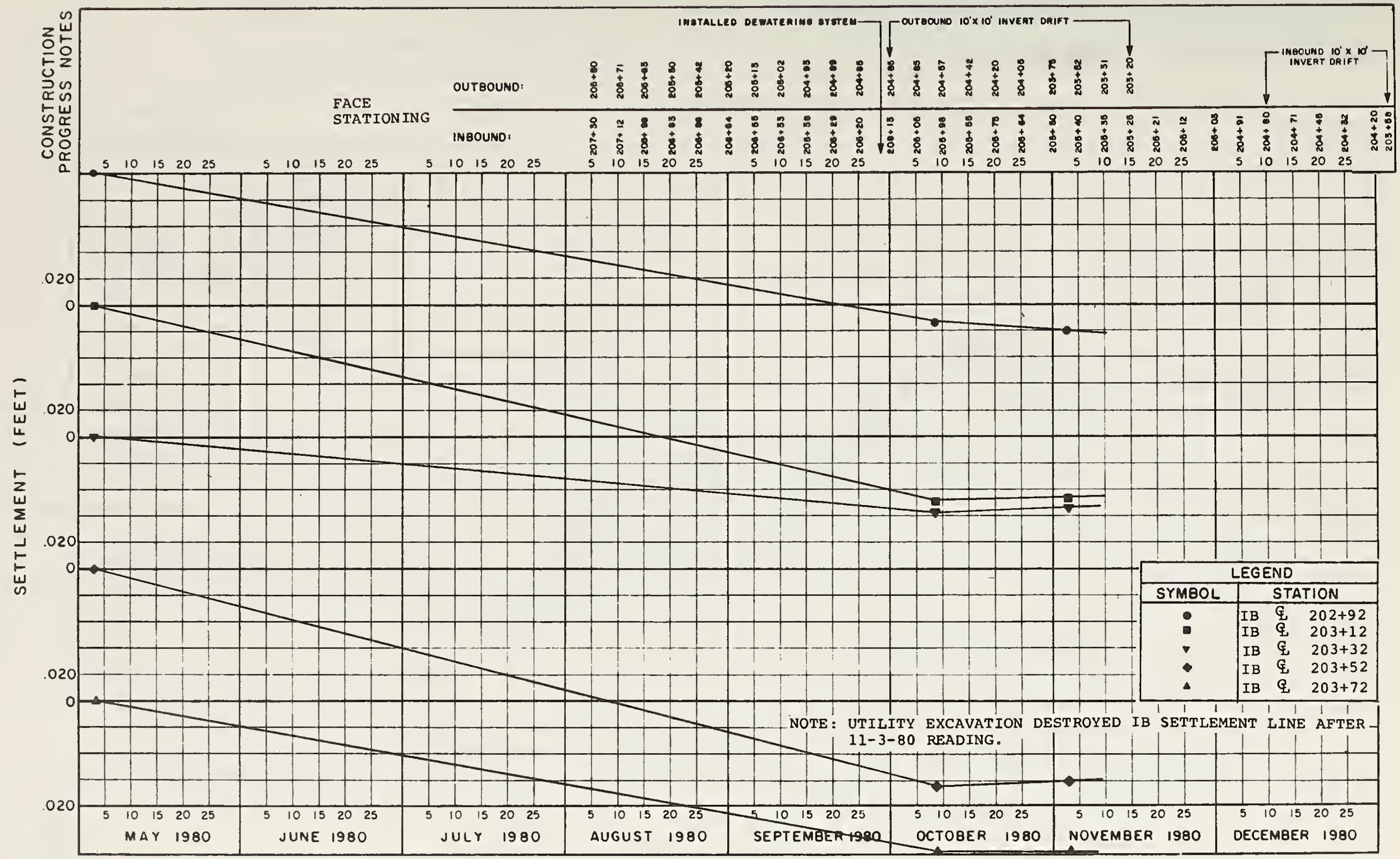


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB ℄ 206+42 - 206+62, IB ℄ 202+32 - 202+72

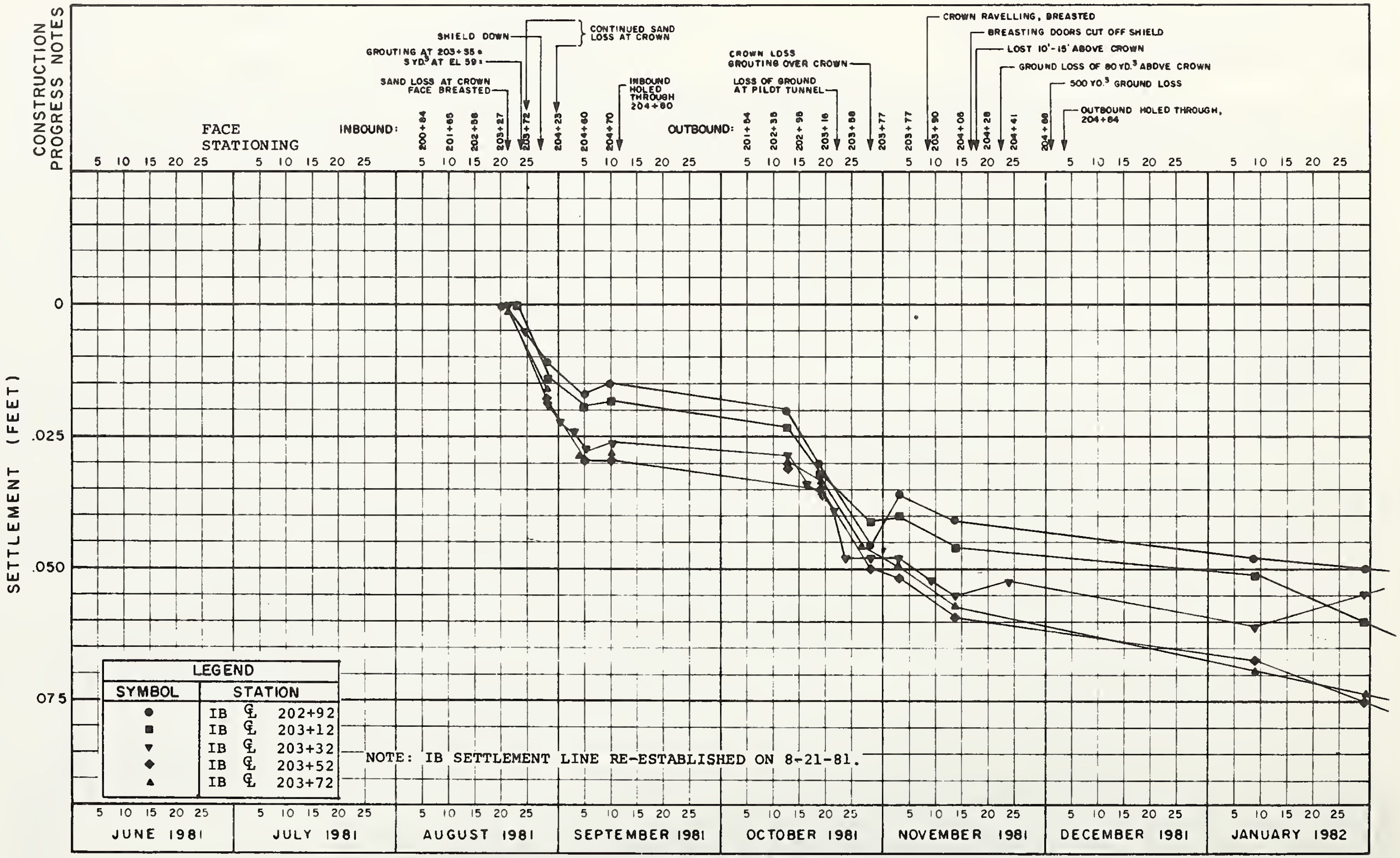


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION IB ℄ 202+92 - 203+72

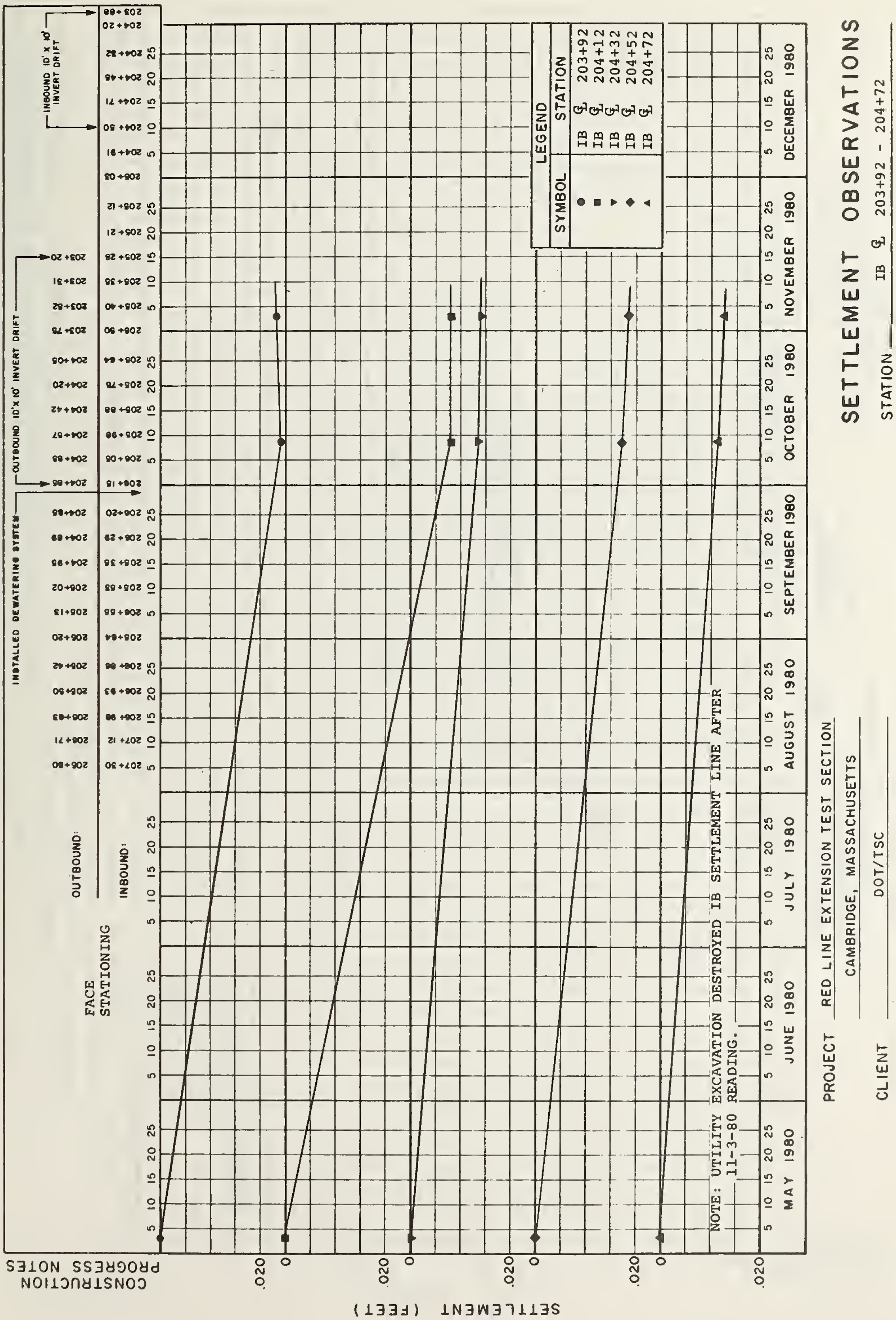


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

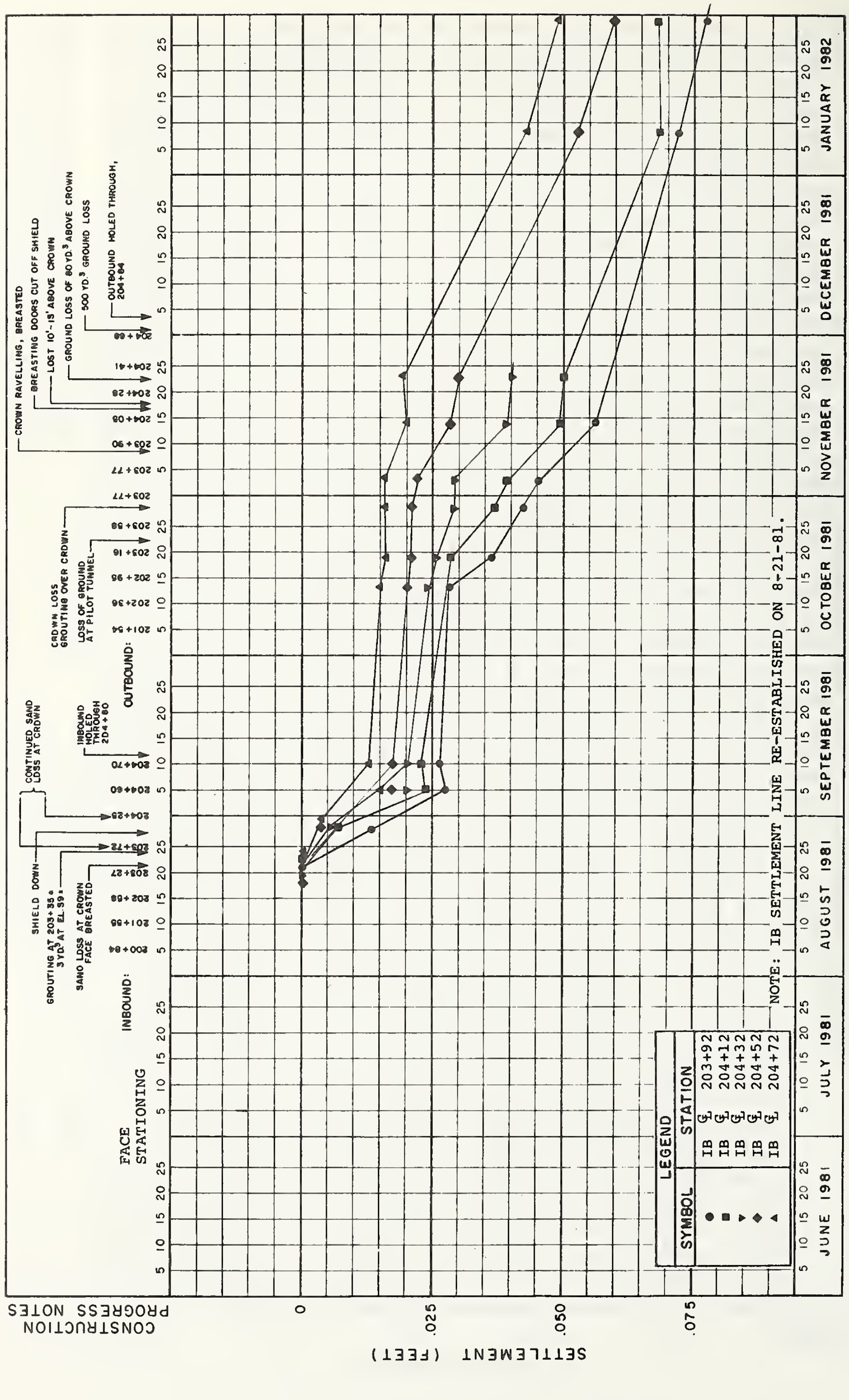
CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION IB ℄ 202+92 - 203+72



SETTLEMENT OBSERVATIONS
STATION IB ℄ 203+92 - 204+72
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
NOTE: UTILITY EXCAVATION DESTROYED IB SETTLEMENT LINE AFTER 11-3-80 READING.
LEGEND
SYMBOL STATION
IB ℄ 203+92
IB ℄ 204+12
IB ℄ 204+32
IB ℄ 204+52
IB ℄ 204+72
SETTLEMENT (FEET)
CONSTRUCTION PROGRESS NOTES
FACE STATIONING
OUTBOUND:
INBOUND:
INSTALLED DEWATERING SYSTEM
OUTBOUND 10'x10' INVERT DRIFT
INBOUND 10'x10' INVERT DRIFT
MAY 1980
JUNE 1980
JULY 1980
AUGUST 1980
SEPTEMBER 1980
OCTOBER 1980
NOVEMBER 1980
DECEMBER 1980

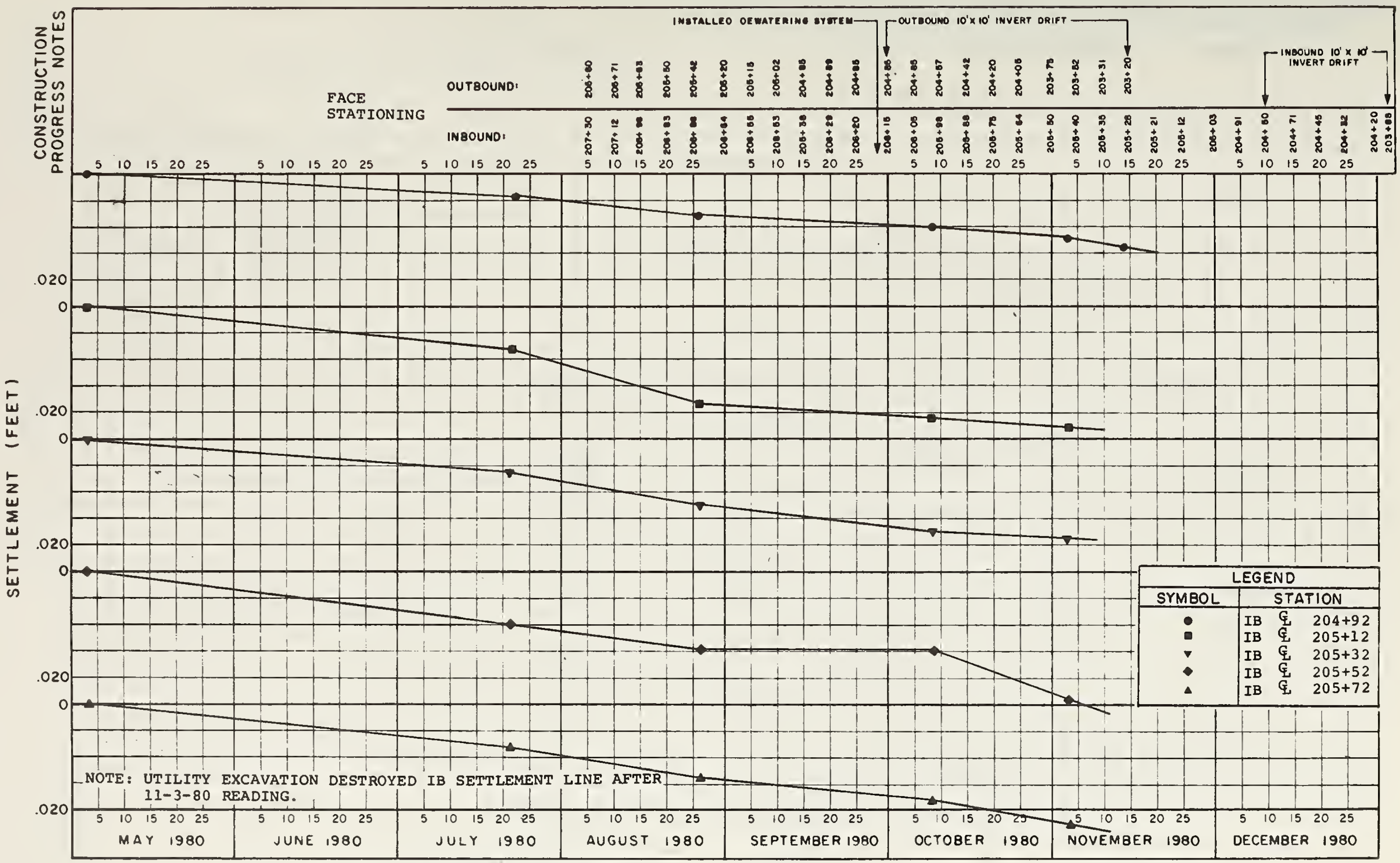


SETTLEMENT OBSERVATIONS

STATION IB ℄ 203+92 - 204+72

PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC



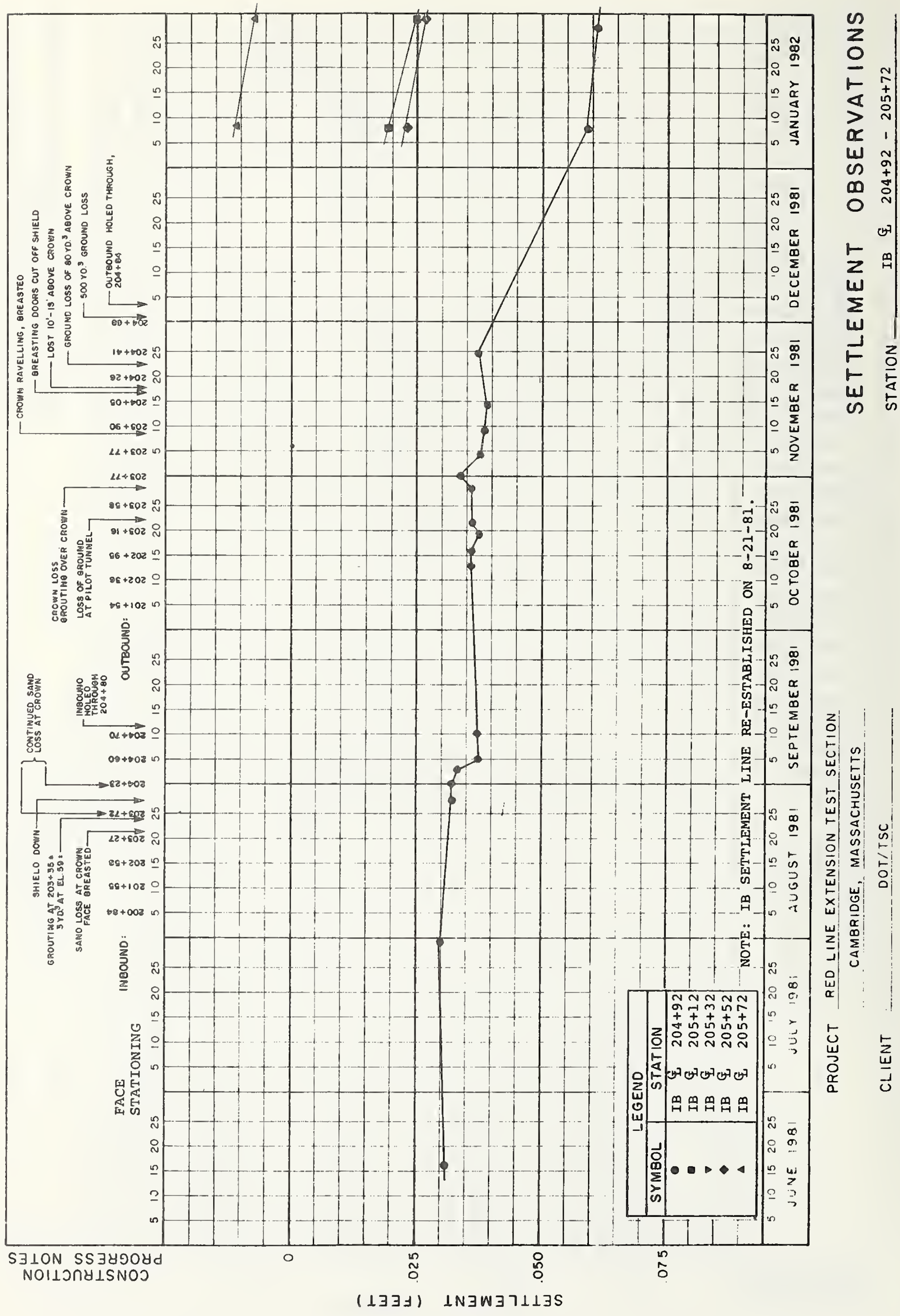
PROJECT RED LINE EXTENSION TEST SECTION

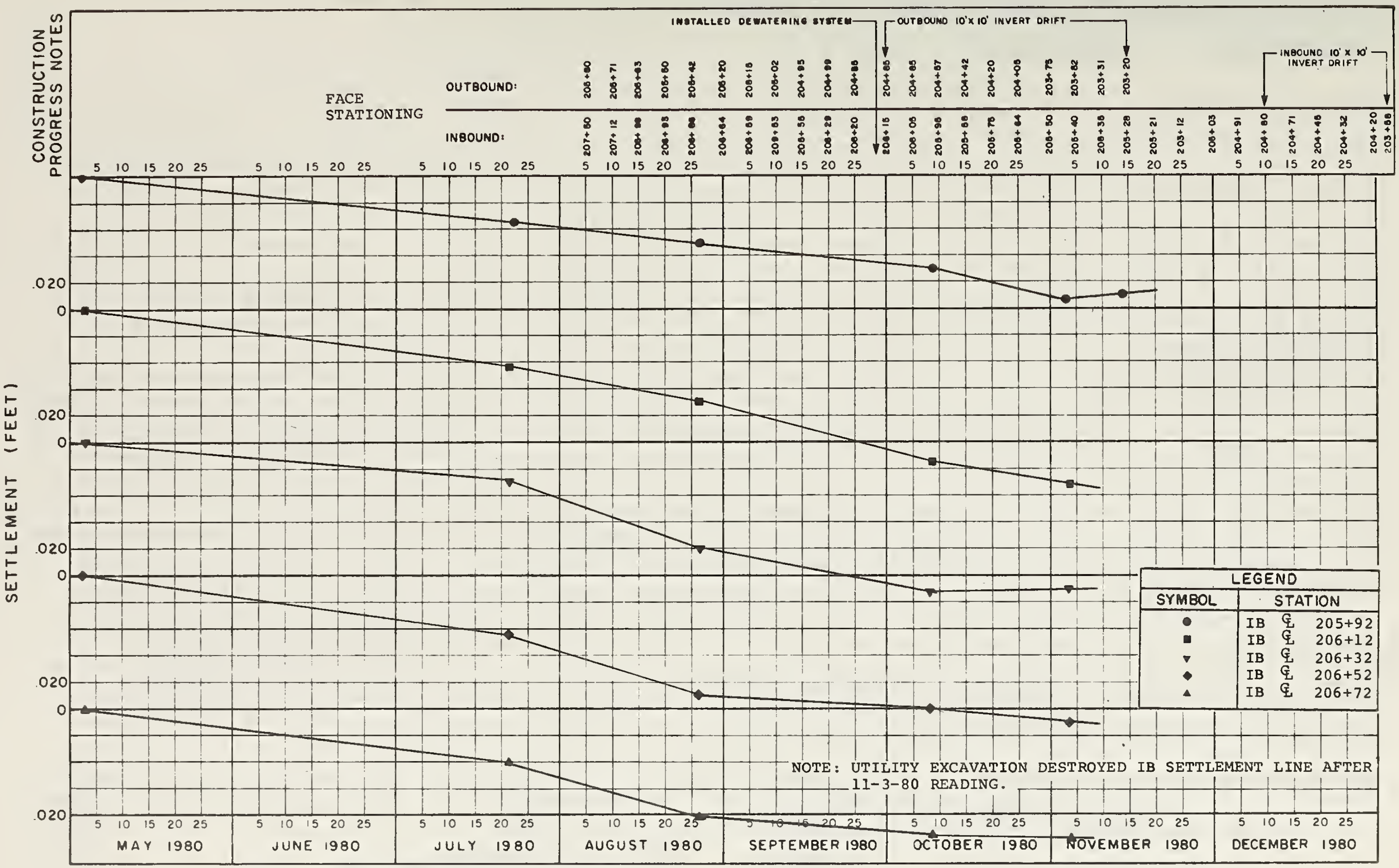
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION IB ℄ 204+92 - 205+72


SETTLEMENT OBSERVATIONS
STATION IB ℄ 204+92 - 205+72
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
SETTLEMENT (FEET)
0
.025
.050
.075
CONSTRUCTION PROGRESS NOTES
FACE STATIONING
INBOUND:
OUTBOUND:
JUNE 1981
JULY 1981
AUGUST 1981
SEPTEMBER 1981
OCTOBER 1981
NOVEMBER 1981
DECEMBER 1981
JANUARY 1982
SHIELD DOWN
GROUTING AT 203+35 ± 3YD³ AT EL 59±
SAND LOSS AT CROWN FACE BREASTED
CONTINUED SAND LOSS AT CROWN
INBOUND HOLED THROUGH 204+80
CROWN LOSS GROUTING OVER CROWN
LOSS OF GROUND AT PILOT TUNNEL
CROWN RAVELLING, BREASTED
BREASTING DOORS CUT OFF SHIELD
LOST 10'-15' ABOVE CROWN
GROUND LOSS OF 80YD³ ABOVE CROWN
500 YD.³ GROUND LOSS
OUTBOUND HOLED THROUGH, 204+84
200+84
201+55
202+53
203+27
203+72
204+23
204+60
204+70
201+54
202+36
202+95
203+16
203+58
203+77
203+77
203+90
204+05
204+26
204+41
204+69
NOTE: IB SETTLEMENT LINE RE-ESTABLISHED ON 8-21-81.
LEGEND
SYMBOL
STATION
IB ℄ 204+92
IB ℄ 205+12
IB ℄ 205+32
IB ℄ 205+52
IB ℄ 205+72

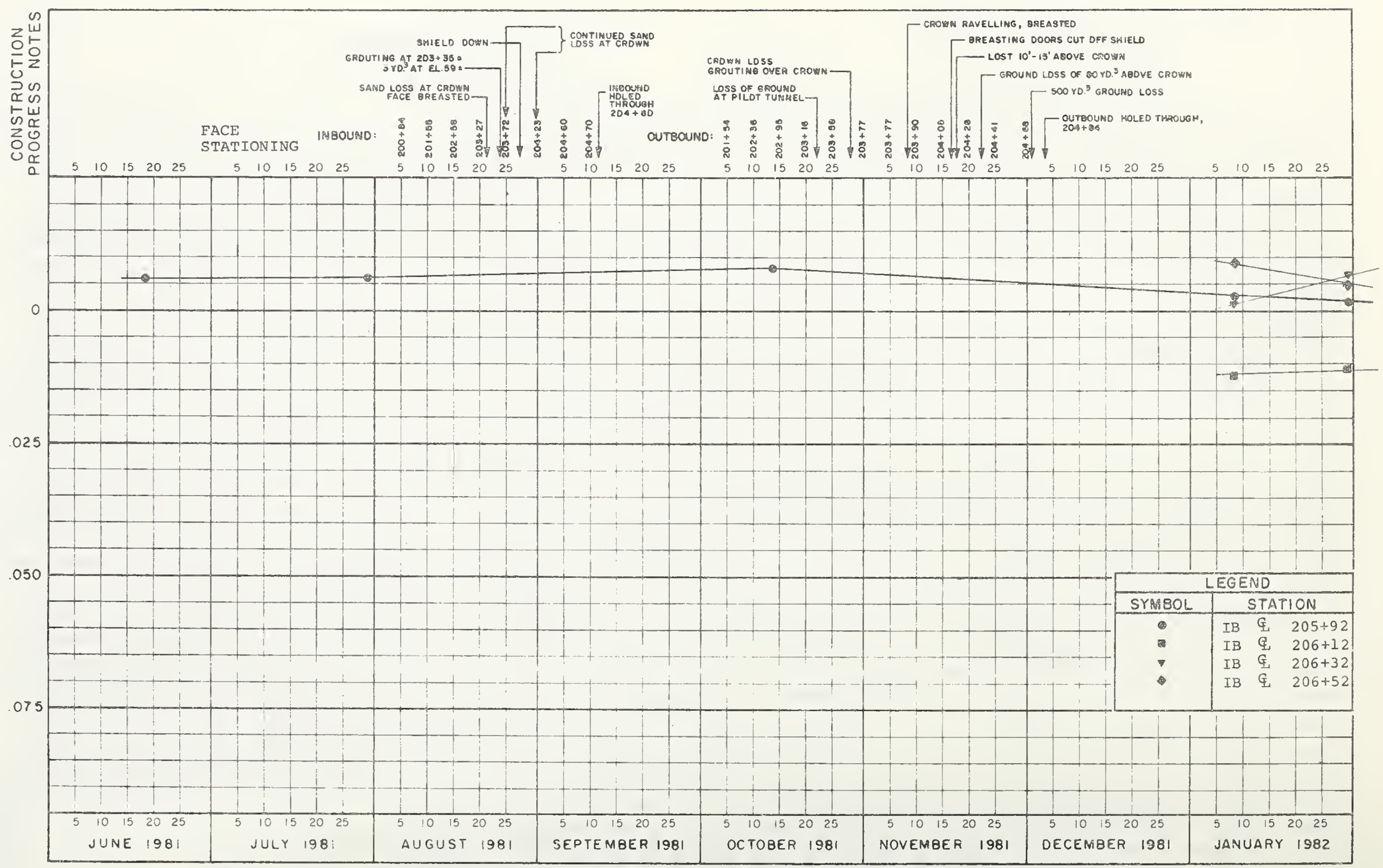


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION IB ℄ 205+92 - 206+72

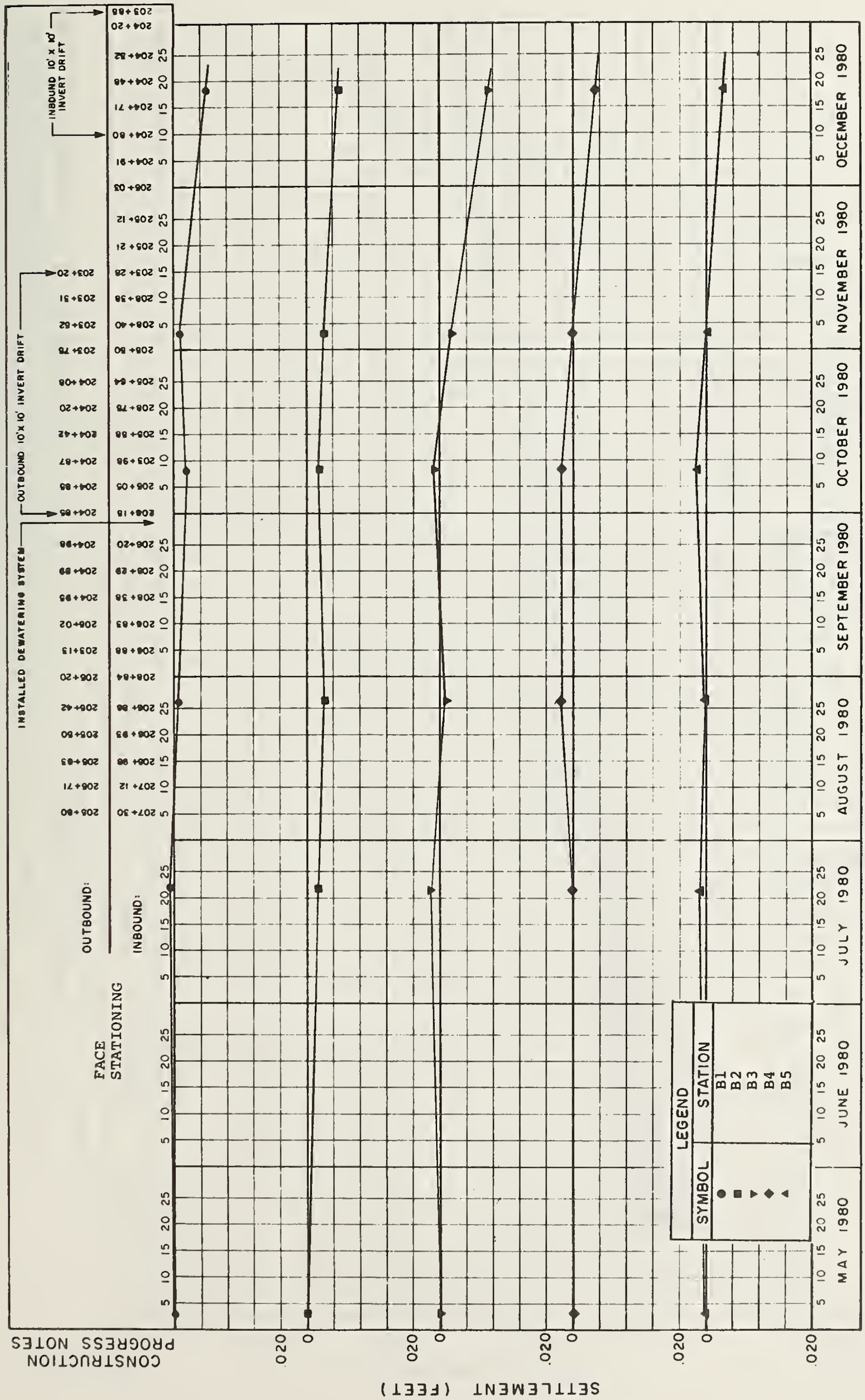


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION IB ℄ 205+92 - 206+52



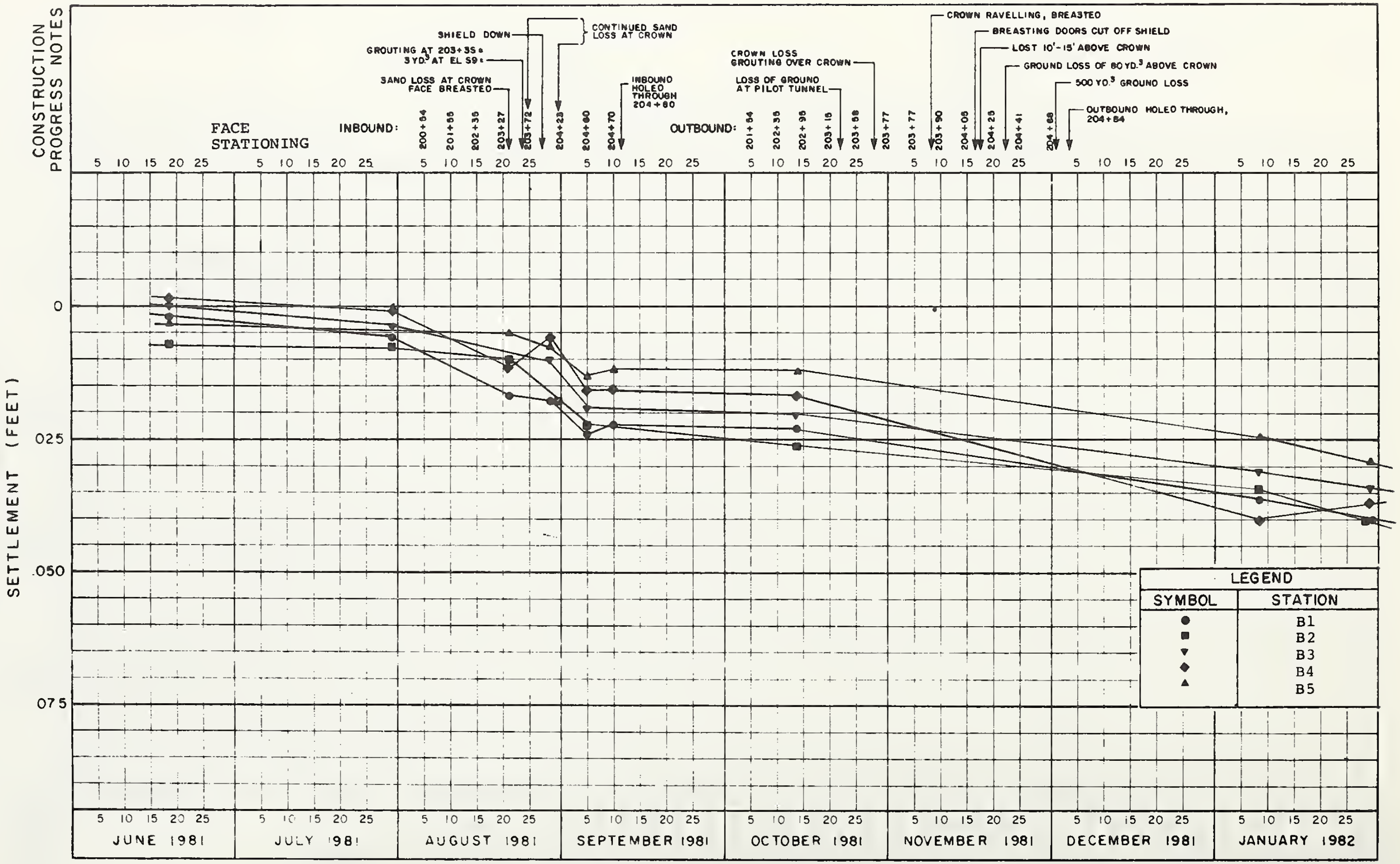
SETTLEMENT OBSERVATIONS

STATION BUILDING POINTS B1,B2,B3,B4 & B5

PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

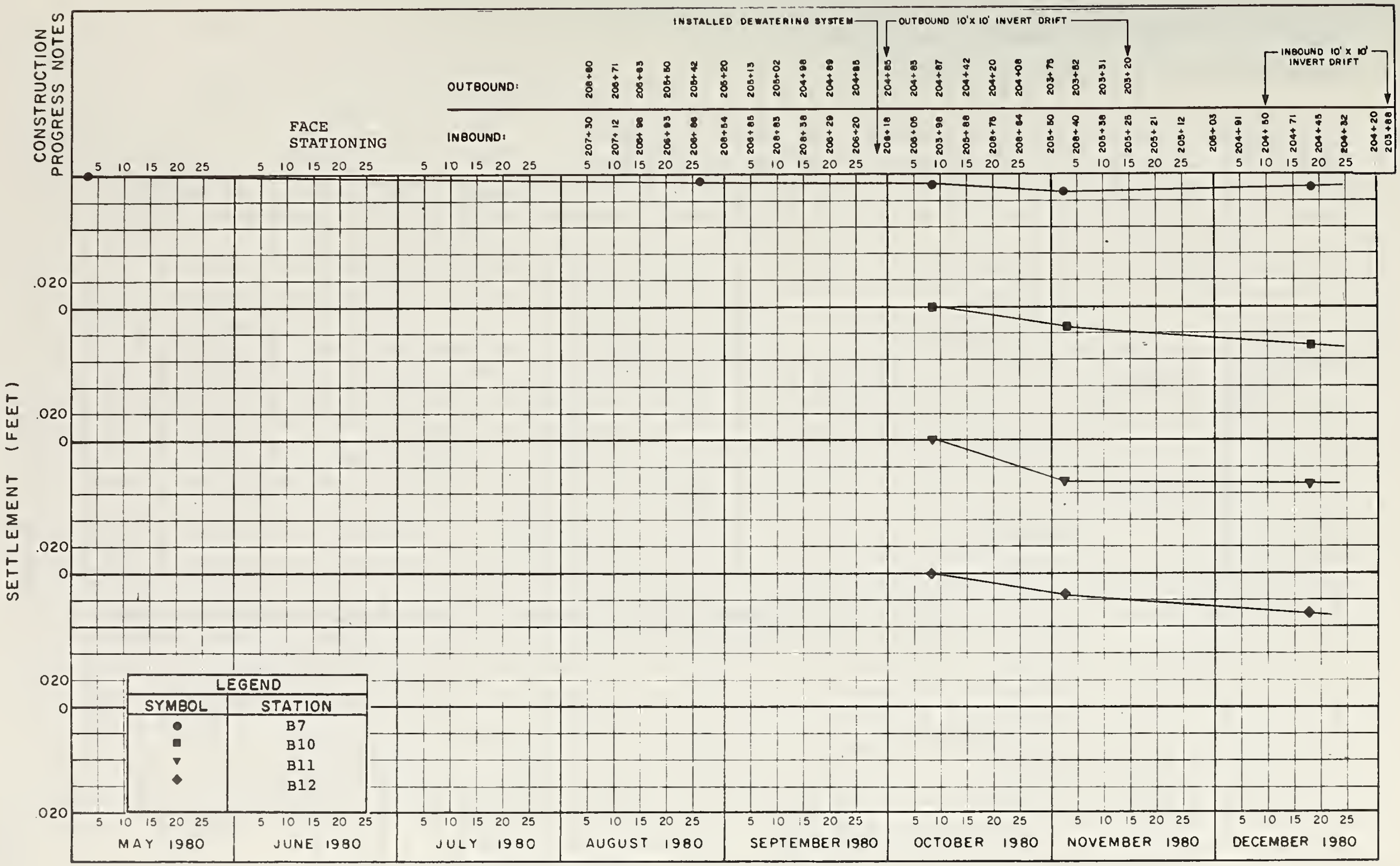


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION BUILDING POINTS B1, B2, B3, B4 & B5



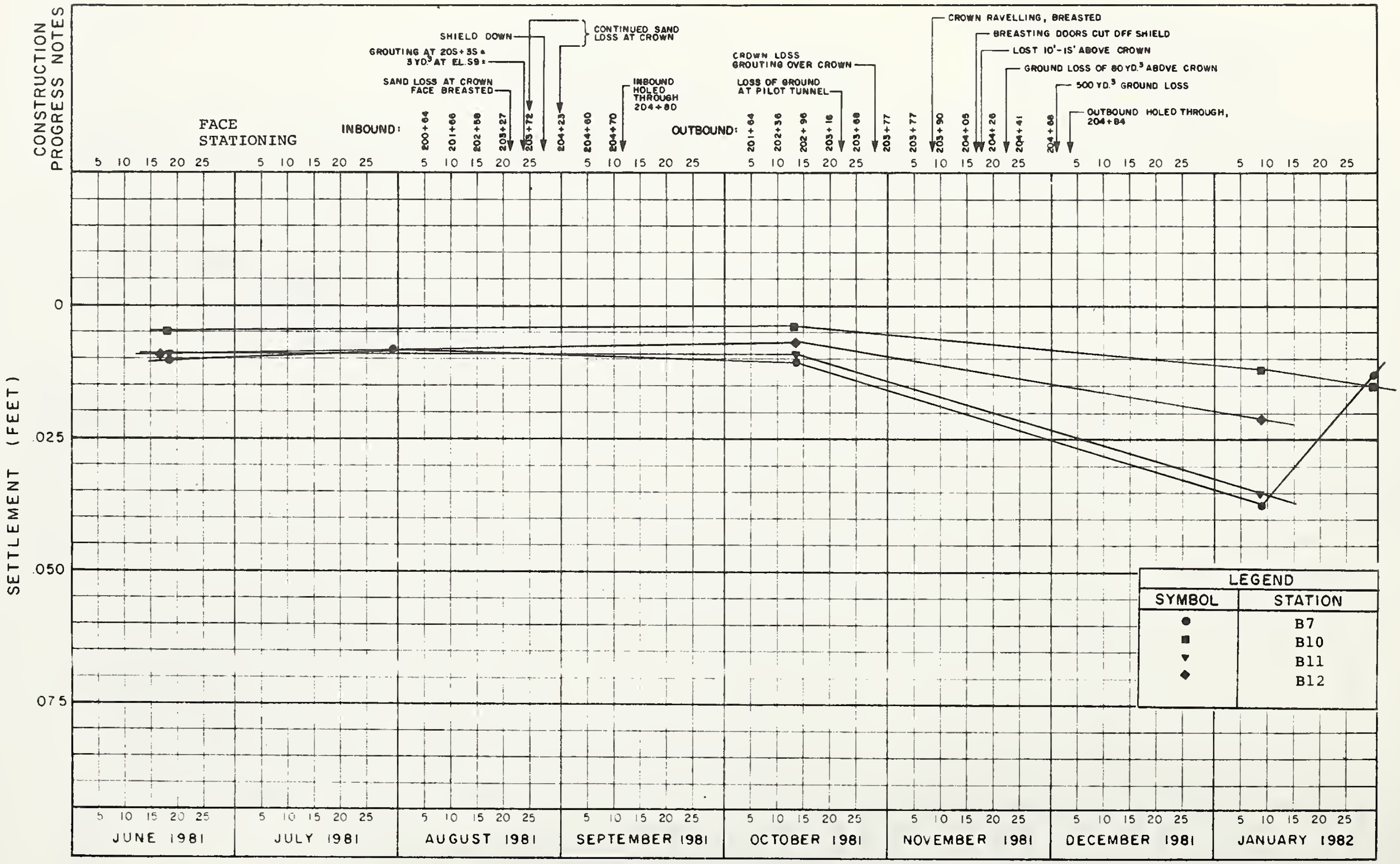
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION BUILDING POINTS B7, B10, B11 & B12

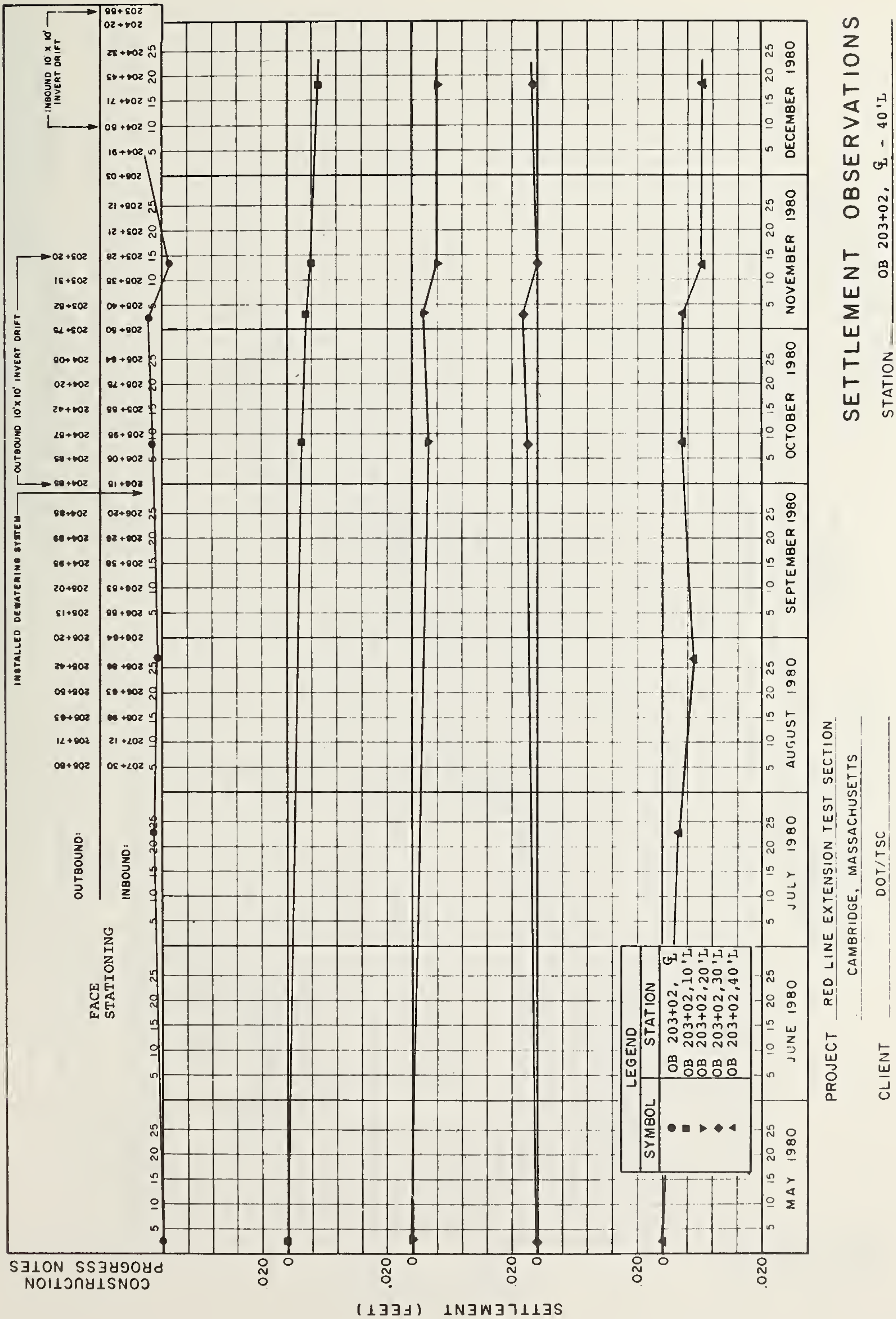


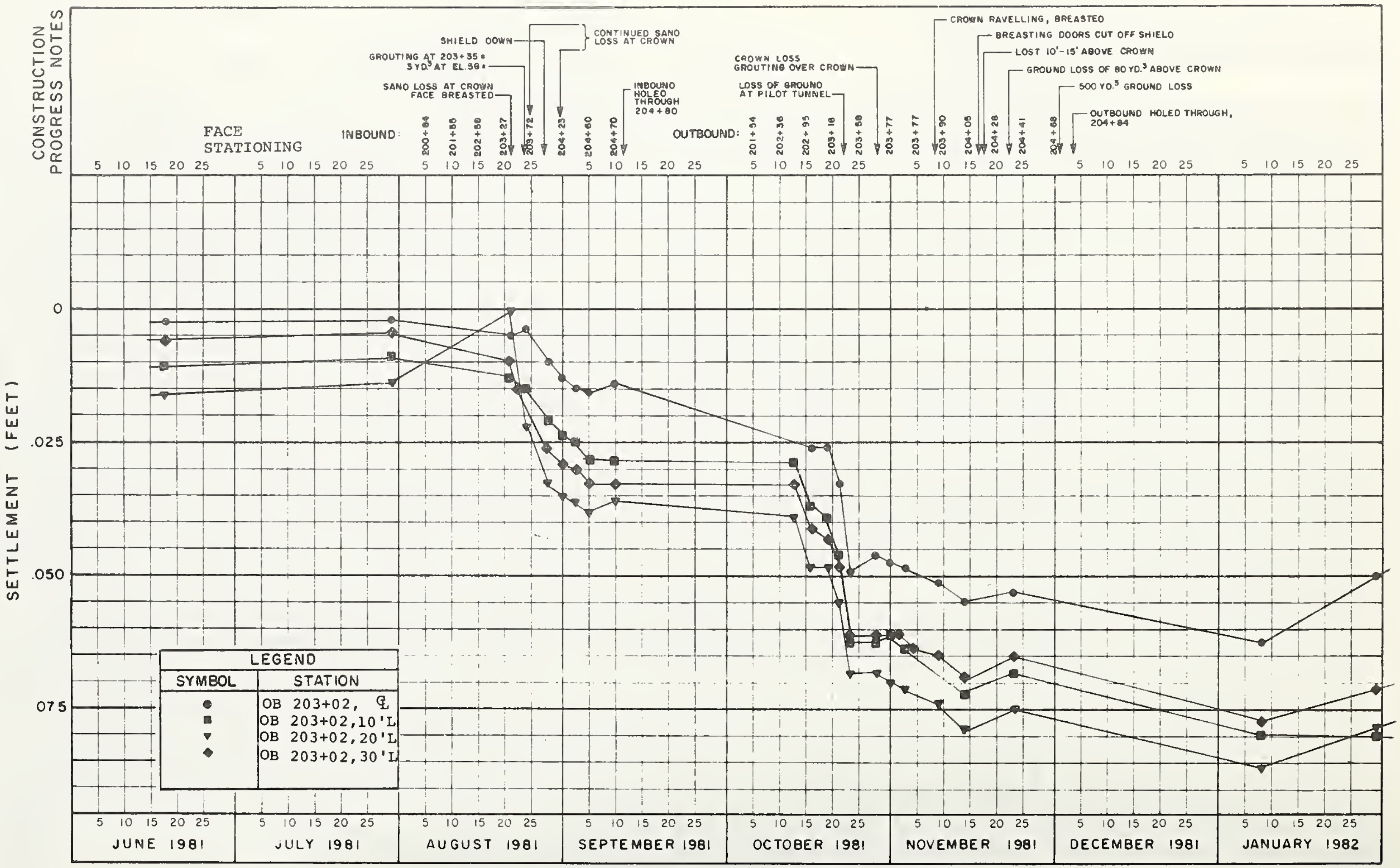
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION BUILDING POINTS B7, B10, B11 & B12


SETTLEMENT OBSERVATIONS
STATION OB 203+02, ℄ - 40'L
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
LEGEND
SYMBOL STATION
OB 203+02, ℄
OB 203+02,10'L
OB 203+02,20'L
OB 203+02,30'L
OB 203+02,40'L
SETTLEMENT (FEET)
CONSTRUCTION PROGRESS NOTES
FACE STATIONING
OUTBOUND:
INBOUND:
INSTALLED DEWATERING SYSTEM
OUTBOUND 10'x10' INVERT DRIFT
INBOUND 10'x10' INVERT DRIFT
MAY 1980
JUNE 1980
JULY 1980
AUGUST 1980
SEPTEMBER 1980
OCTOBER 1980
NOVEMBER 1980
DECEMBER 1980



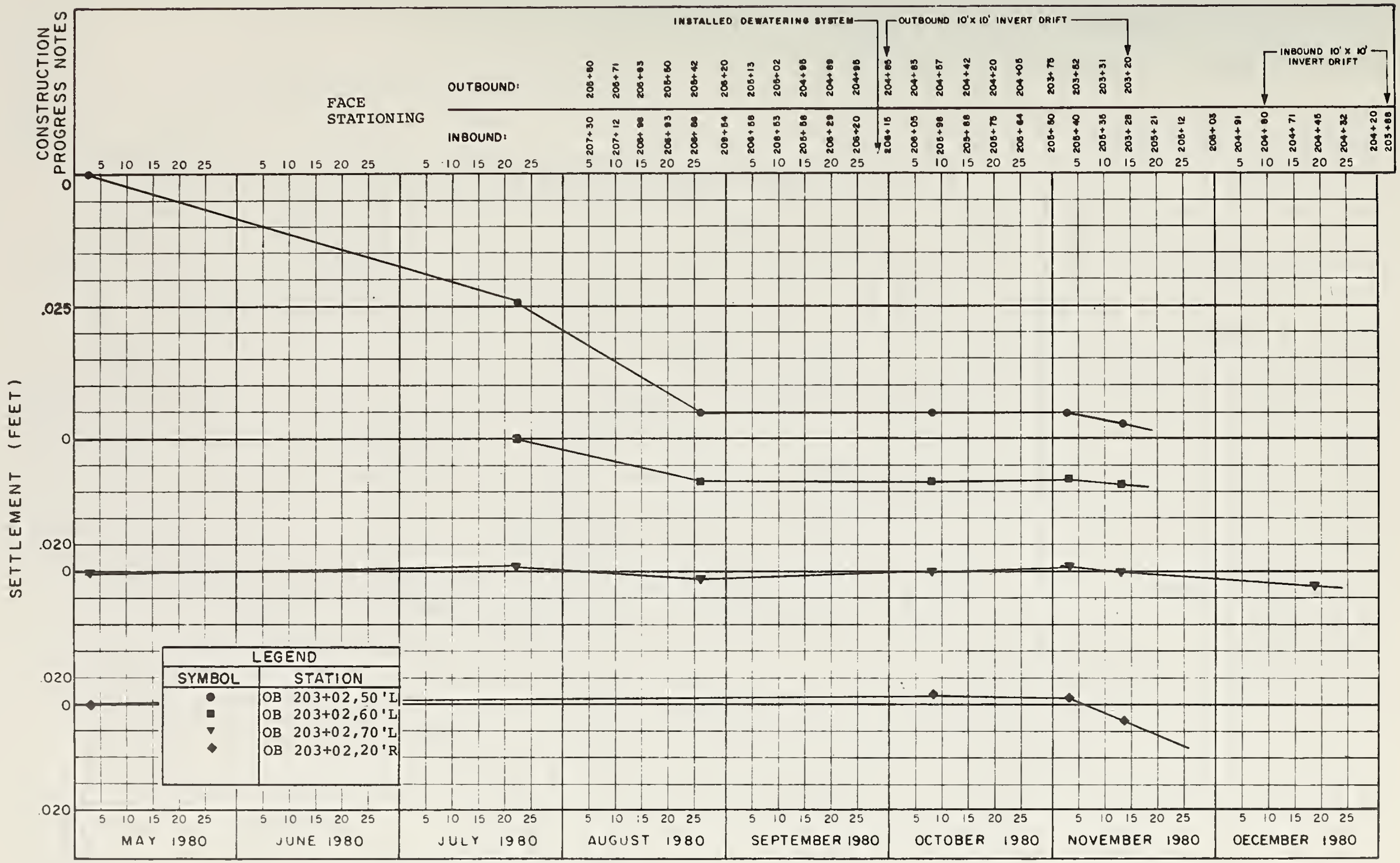
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 203+02, ℄ - 30'L



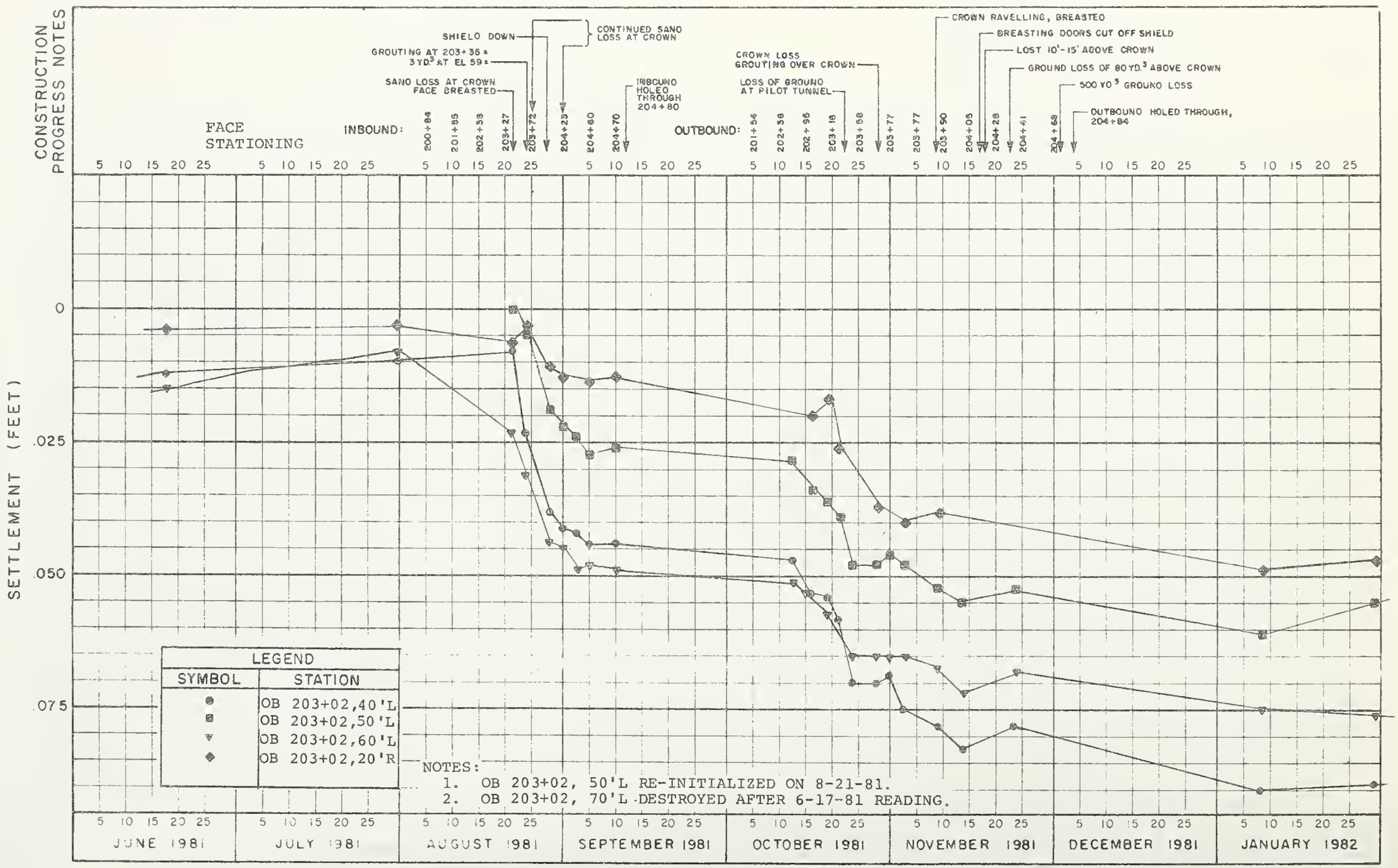
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 203+02, 50'L - 70'L, 20'R

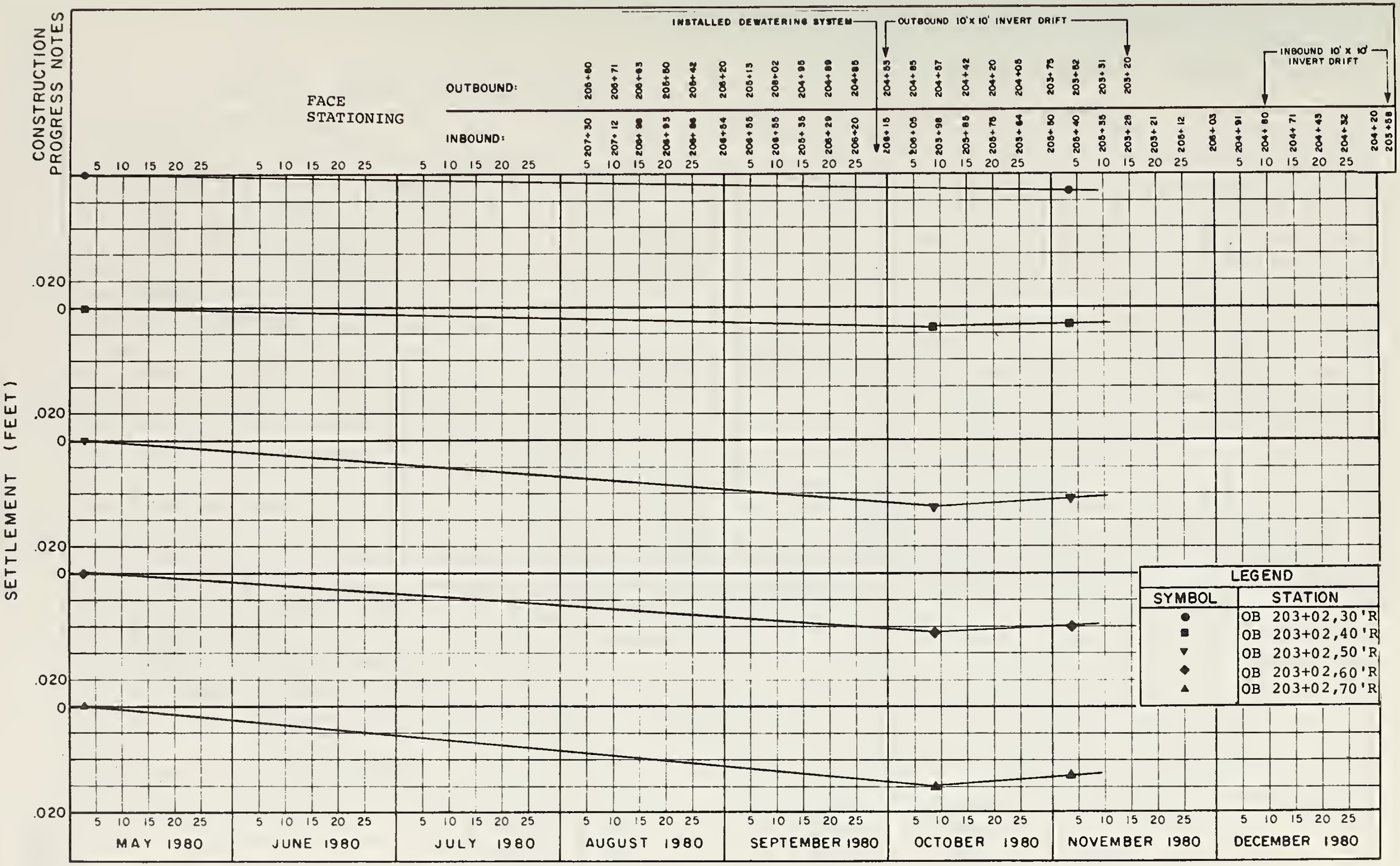


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 203+02, 40'L - 60'L, 20'R

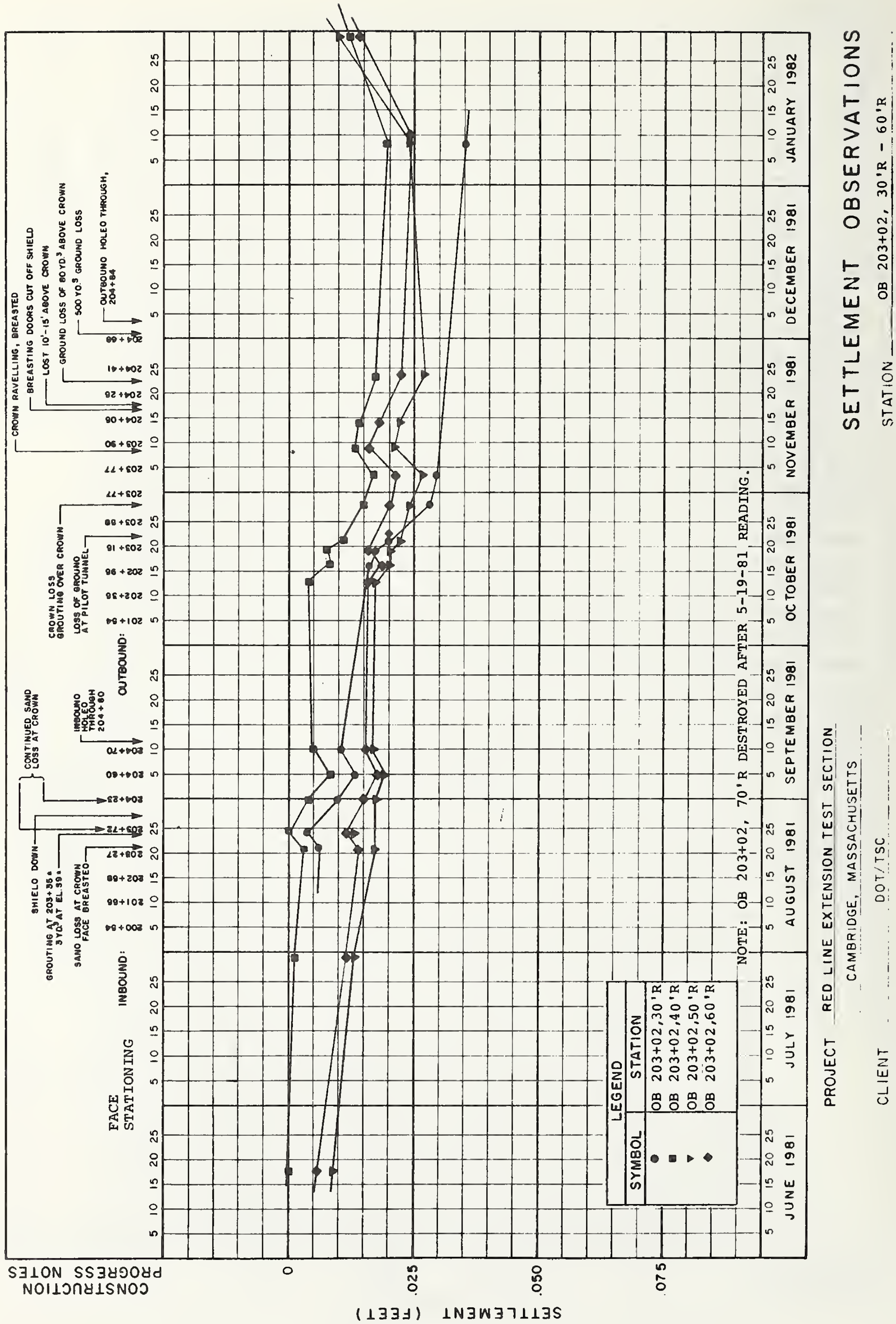


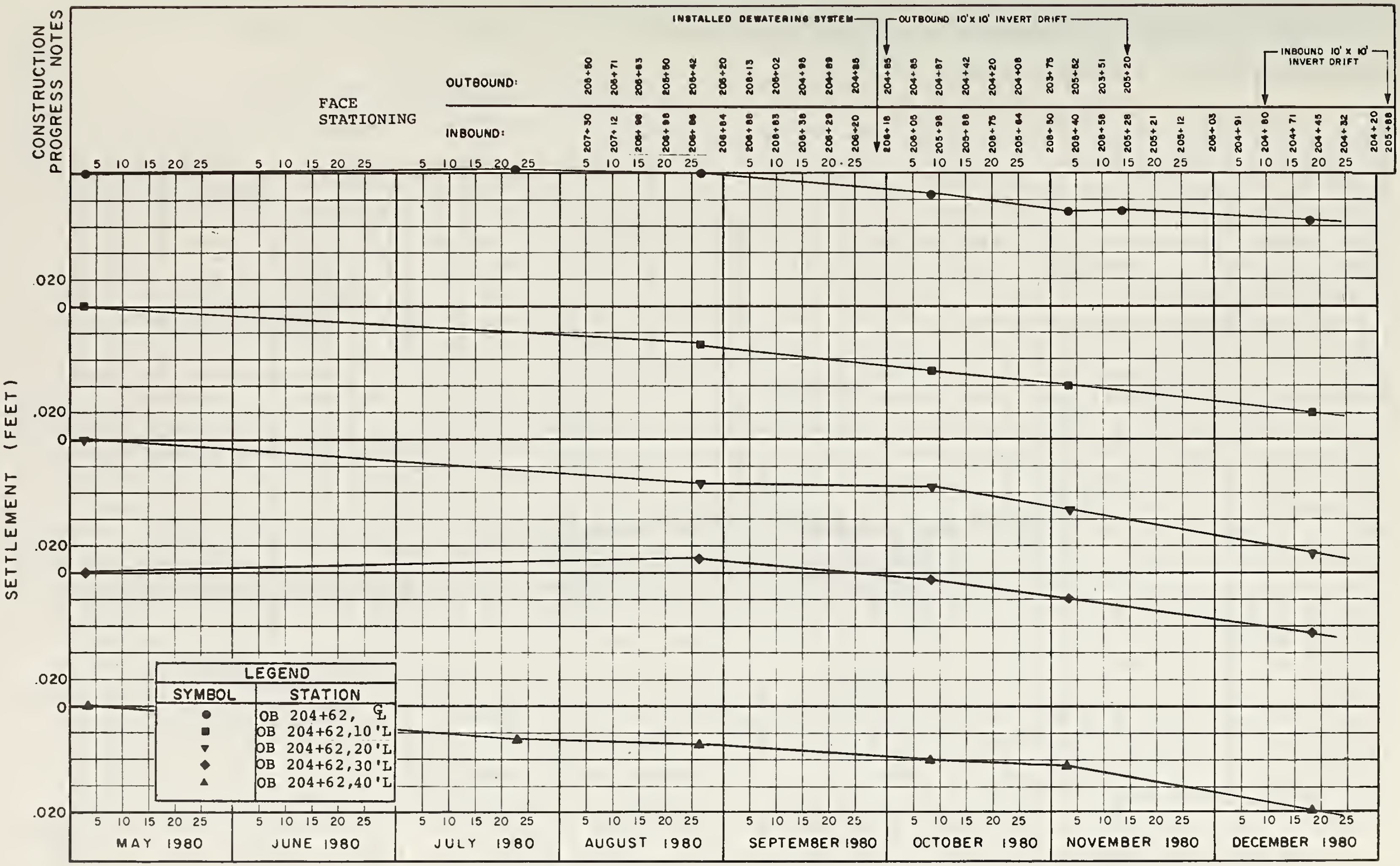
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 203+02, 30'R - 70'R


SETTLEMENT OBSERVATIONS
PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC
STATION OB 203+02, 30'R - 60'R
SETTLEMENT (FEET)
CONSTRUCTION PROGRESS NOTES
FACE STATIONING
INBOUND:
OUTBOUND:
LEGEND
SYMBOL
STATION
OB 203+02,30'R
OB 203+02,40'R
OB 203+02,50'R
OB 203+02,60'R
NOTE: OB 203+02, 70'R DESTROYED AFTER 5-19-81 READING.
JUNE 1981
JULY 1981
AUGUST 1981
SEPTEMBER 1981
OCTOBER 1981
NOVEMBER 1981
DECEMBER 1981
JANUARY 1982
0
.025
.050
.075



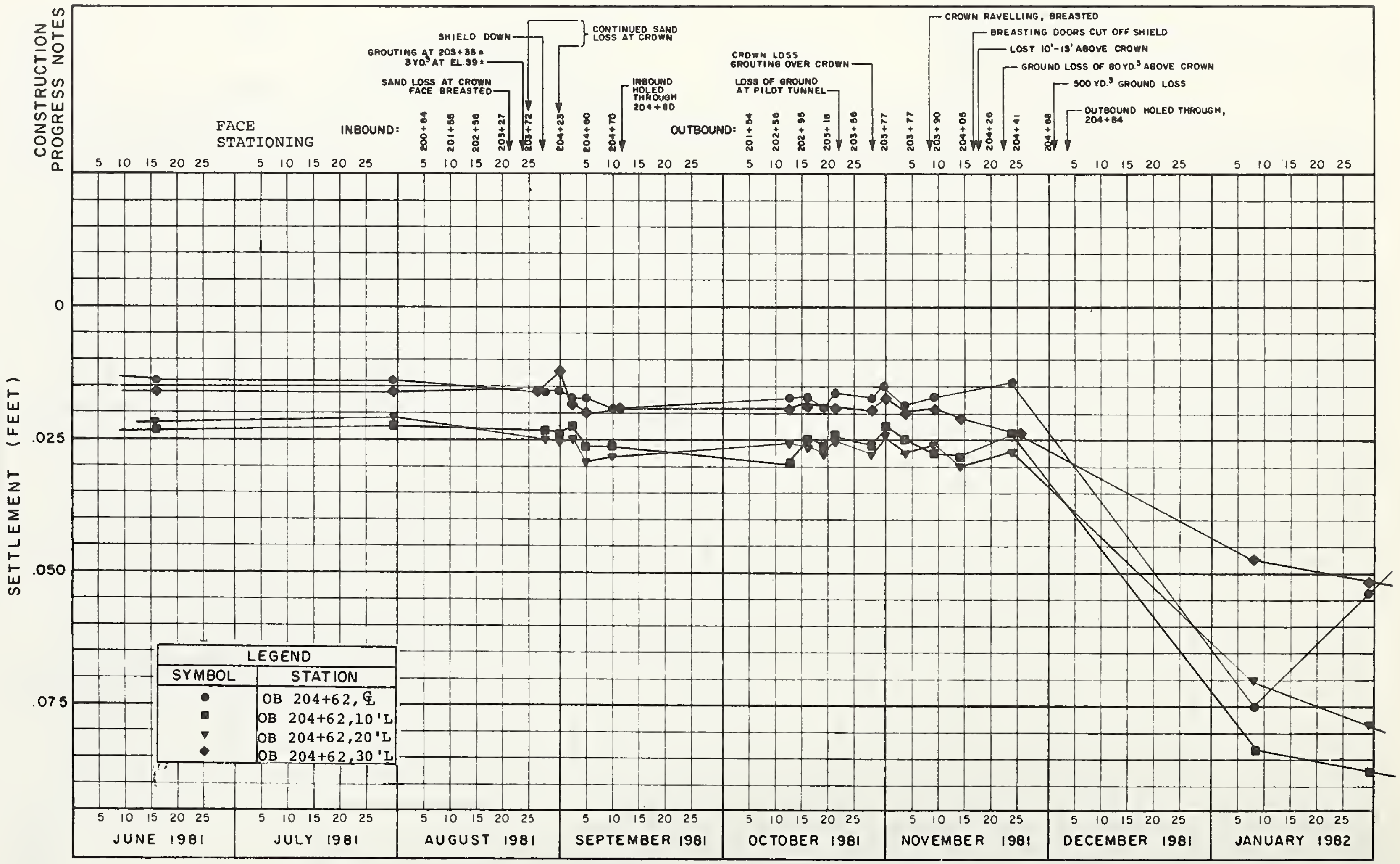
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 204+62, ℄ - 40'L

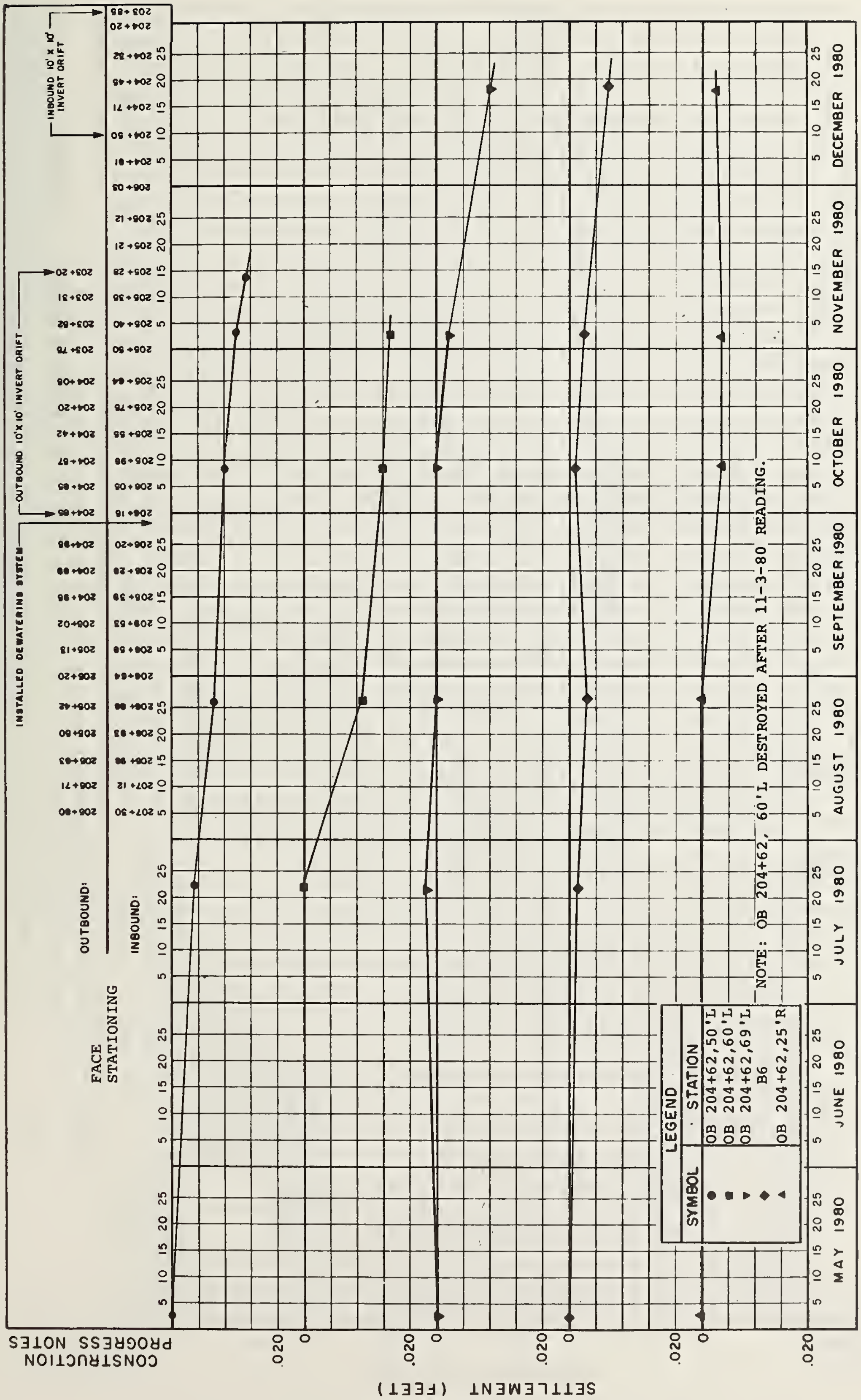


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 204+62, ℄ - 30'L



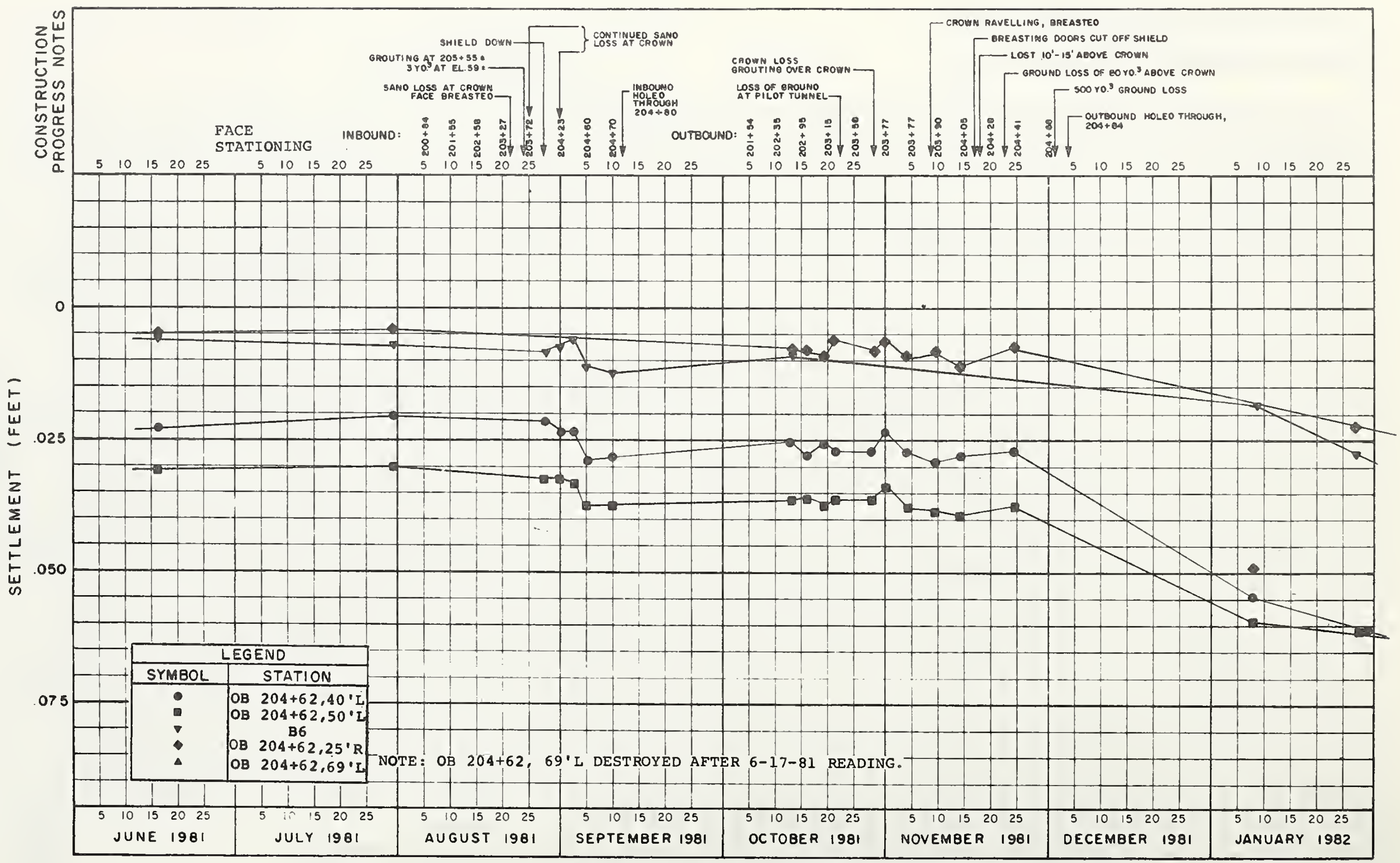
SETTLEMENT OBSERVATIONS

STATION OB 204+62, 50'L - 69'L, 25'R; B6

PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

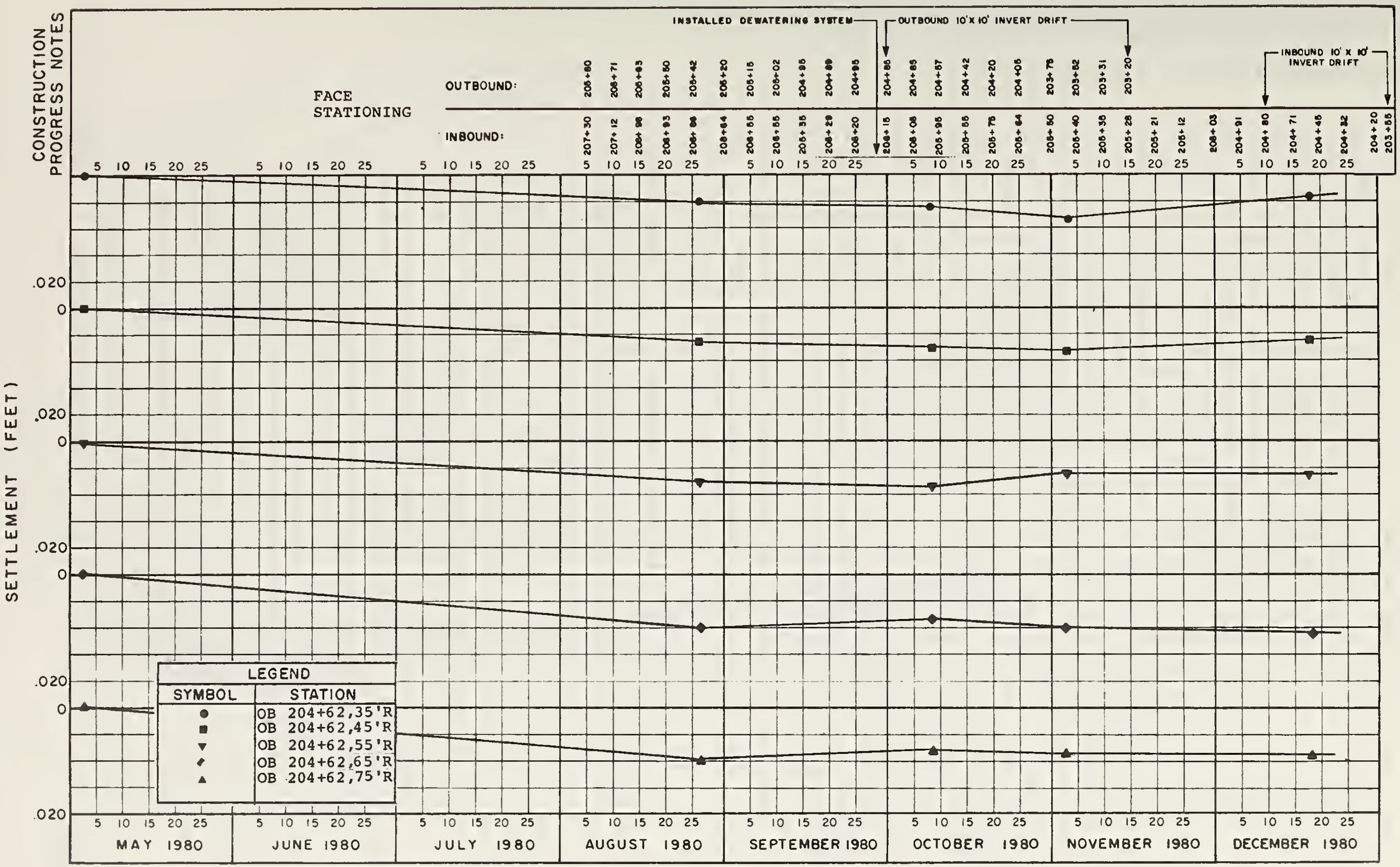


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 204+62, 40'L - 50'L, 25'R; B6



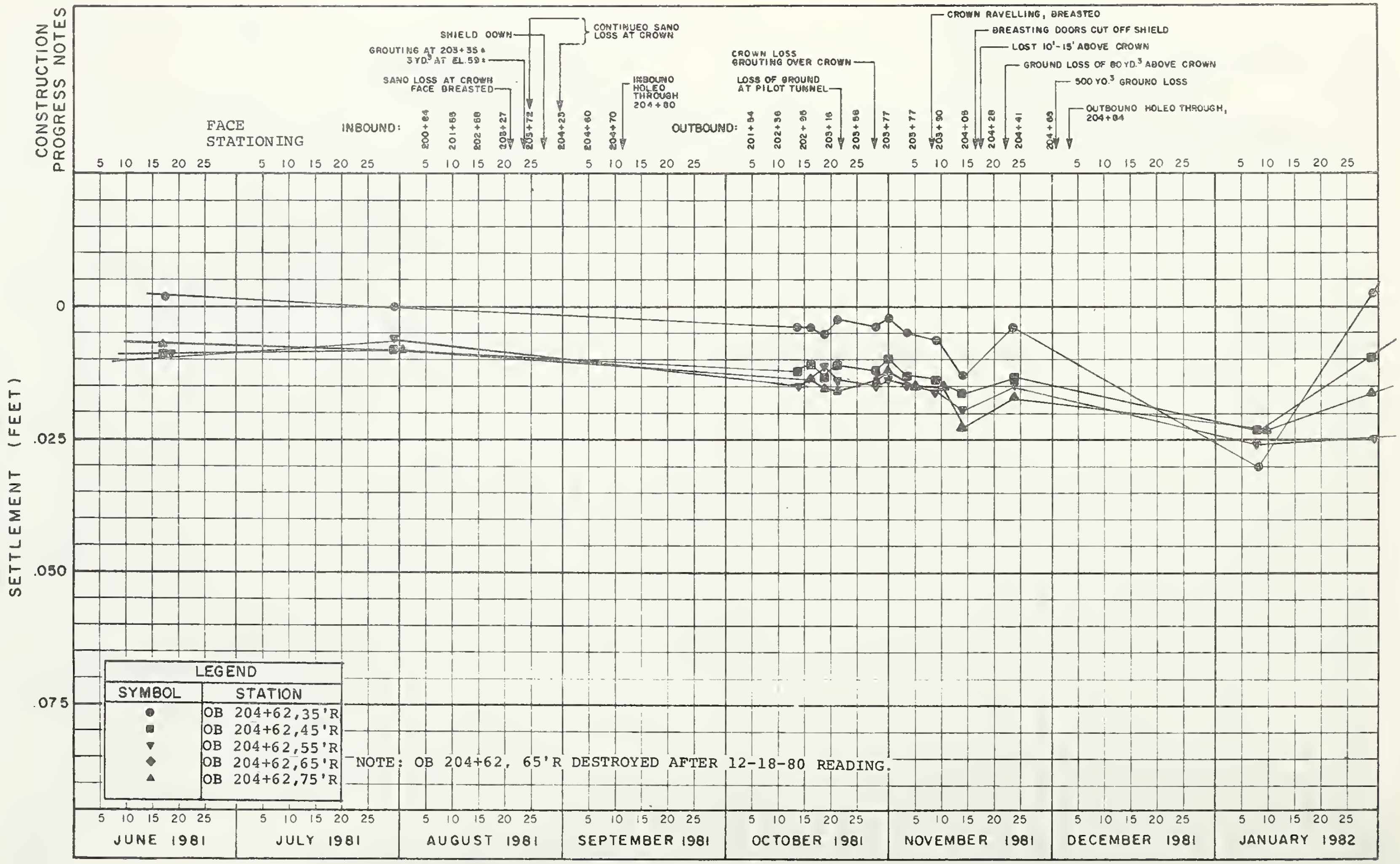
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 204+62, 35'R - 75'R

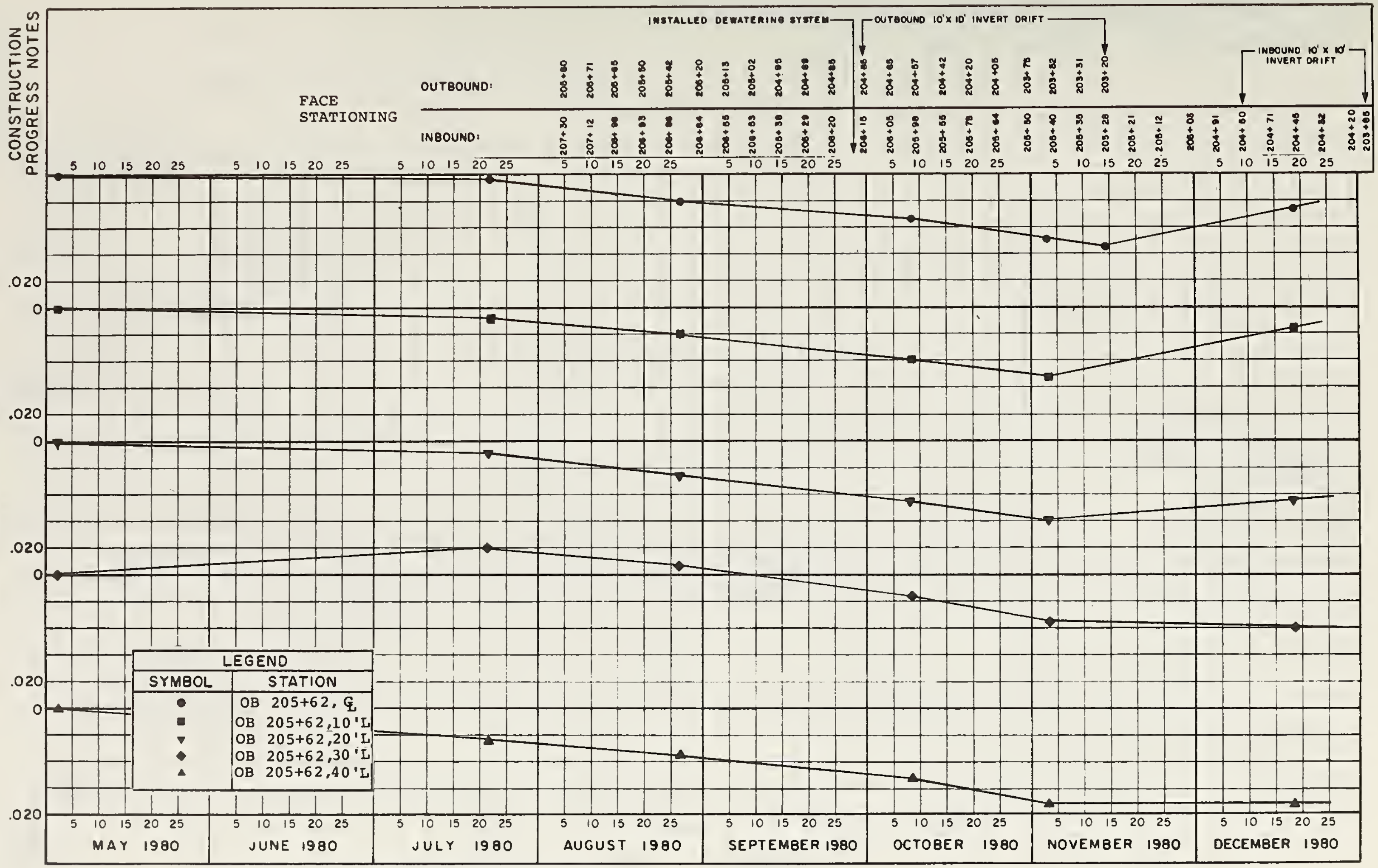


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 204+62, 35'R - 75'R

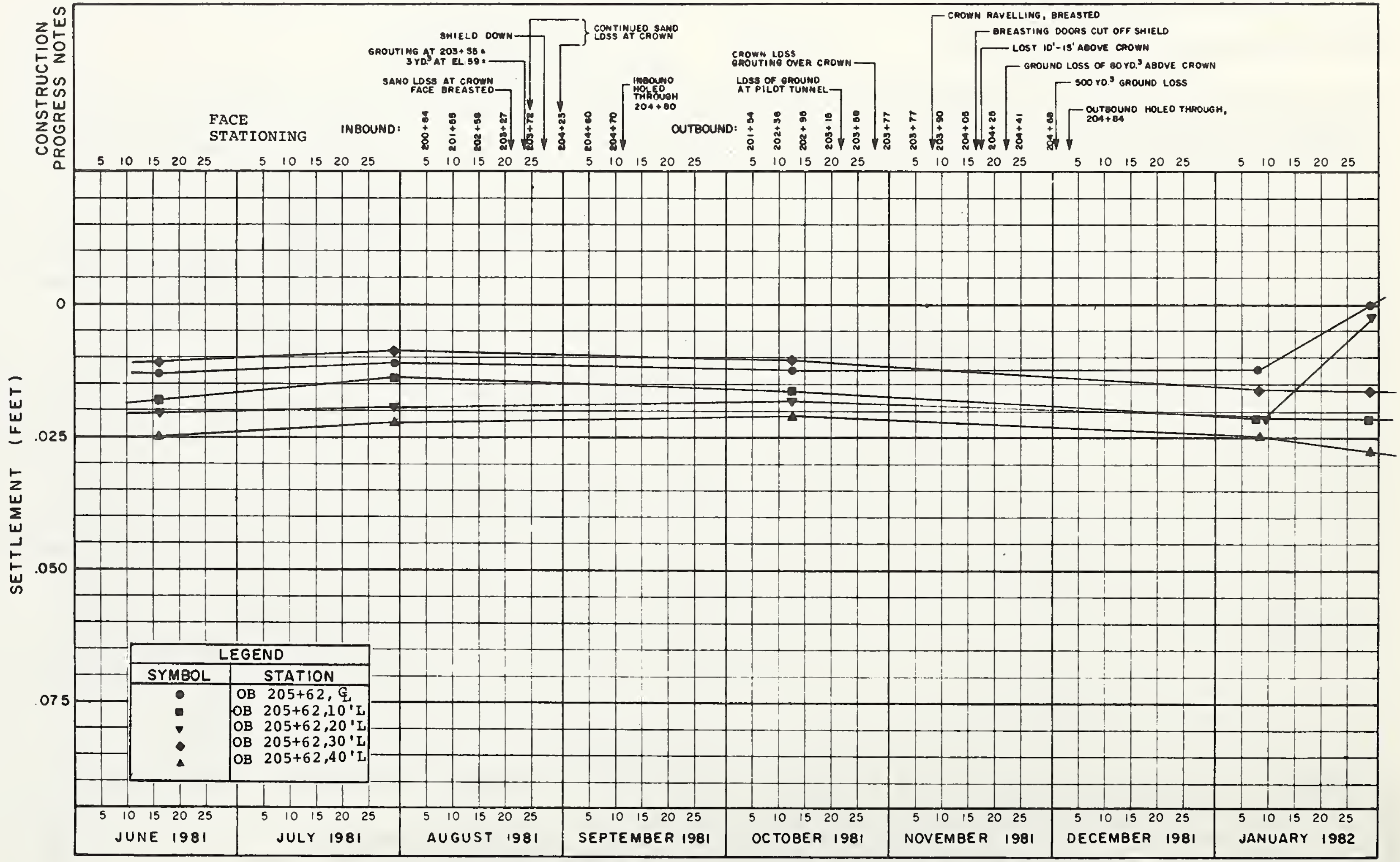


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 205+62, ℄ - 40'L



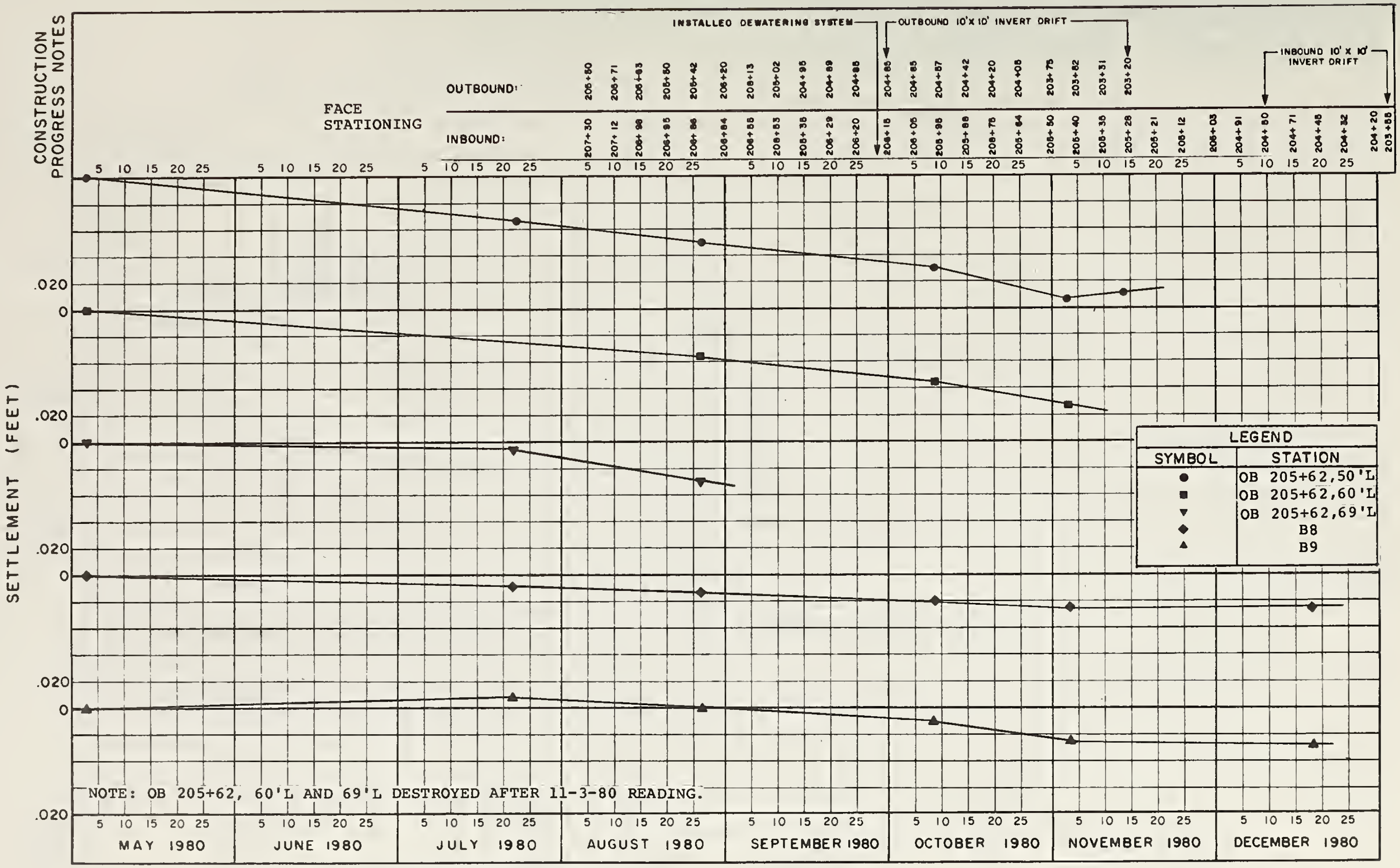
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 205+62, ℄ - 40'L

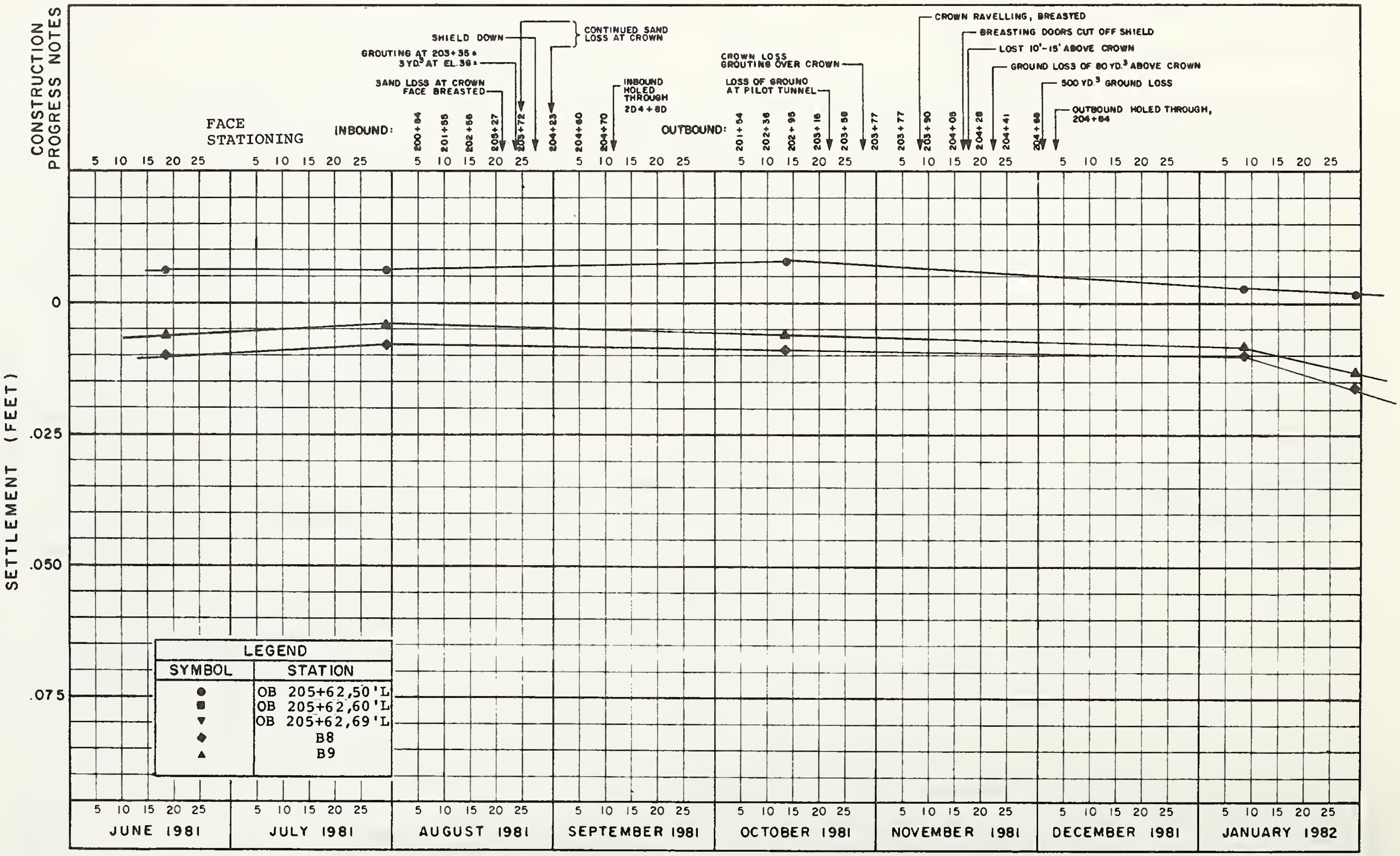


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

STATION OB 205+62, 50'L - 69'L; B8, B9



PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

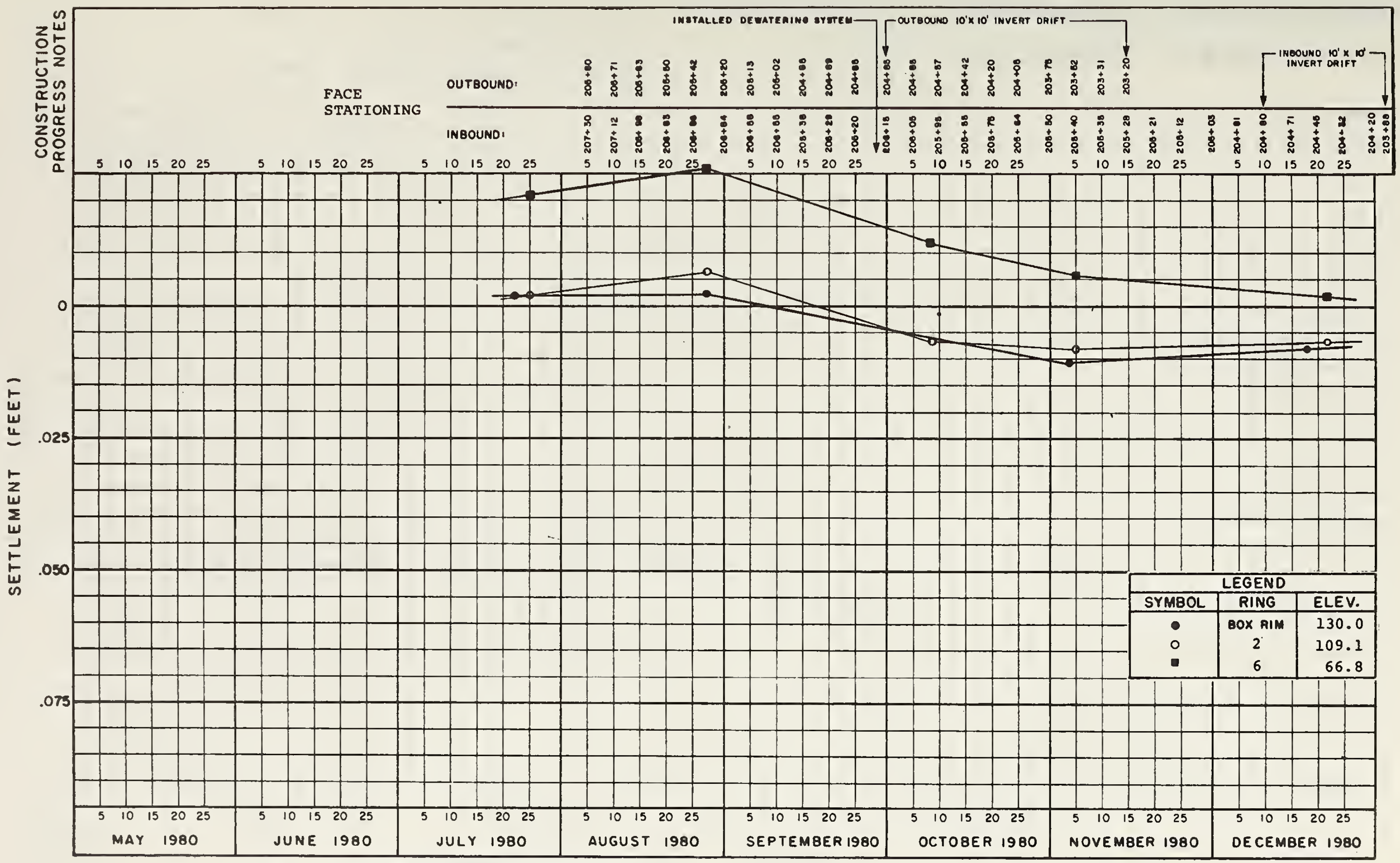
STATION OB 205+62, 50'L; B8, B9

APPENDIX E

DEEP SETTLEMENT POINT OBSERVATIONS

APPENDIX E

DEEP SETTLEMENT POINT OBSERVATIONS

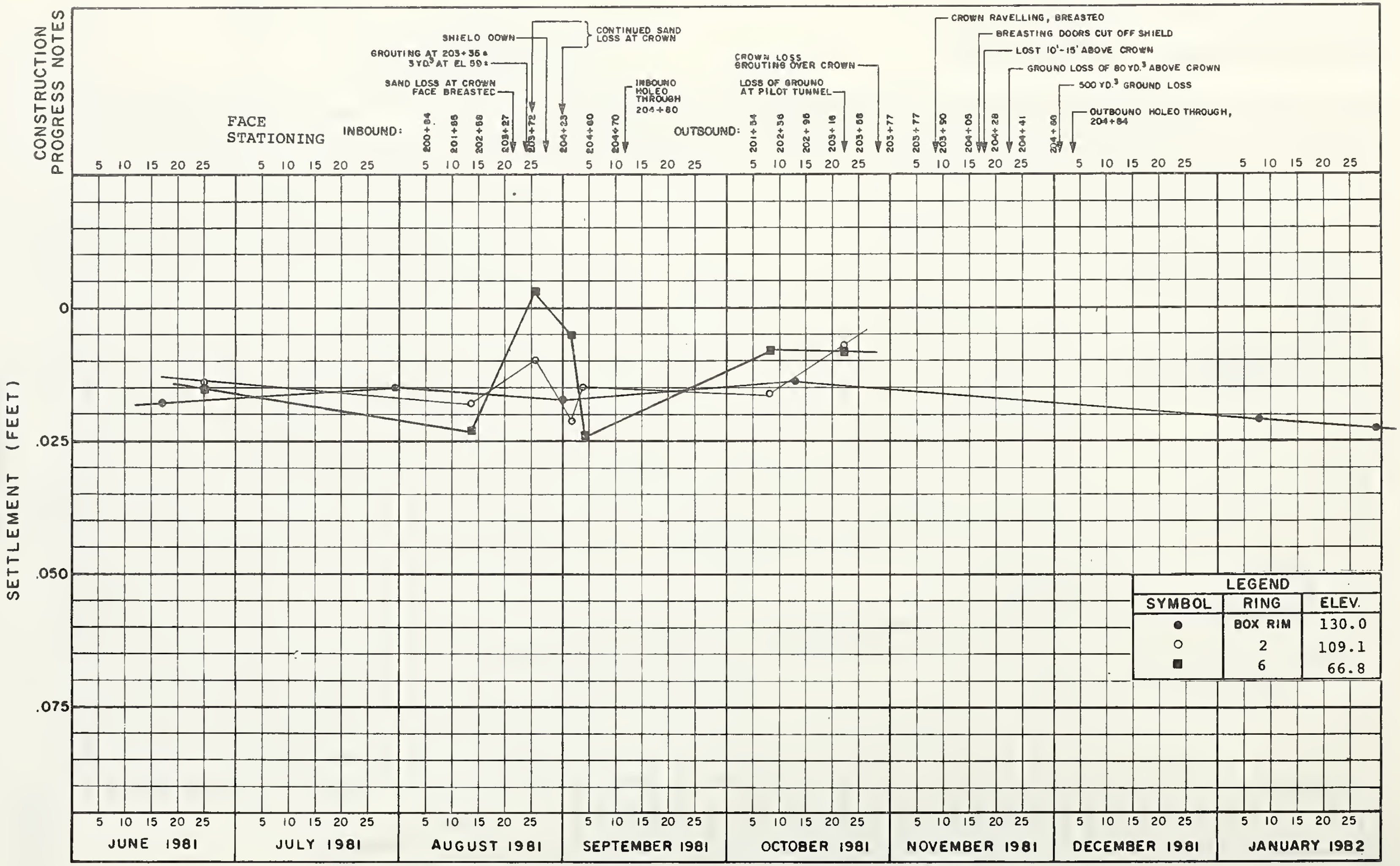


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 2 STATION IB 205+93, 1'R



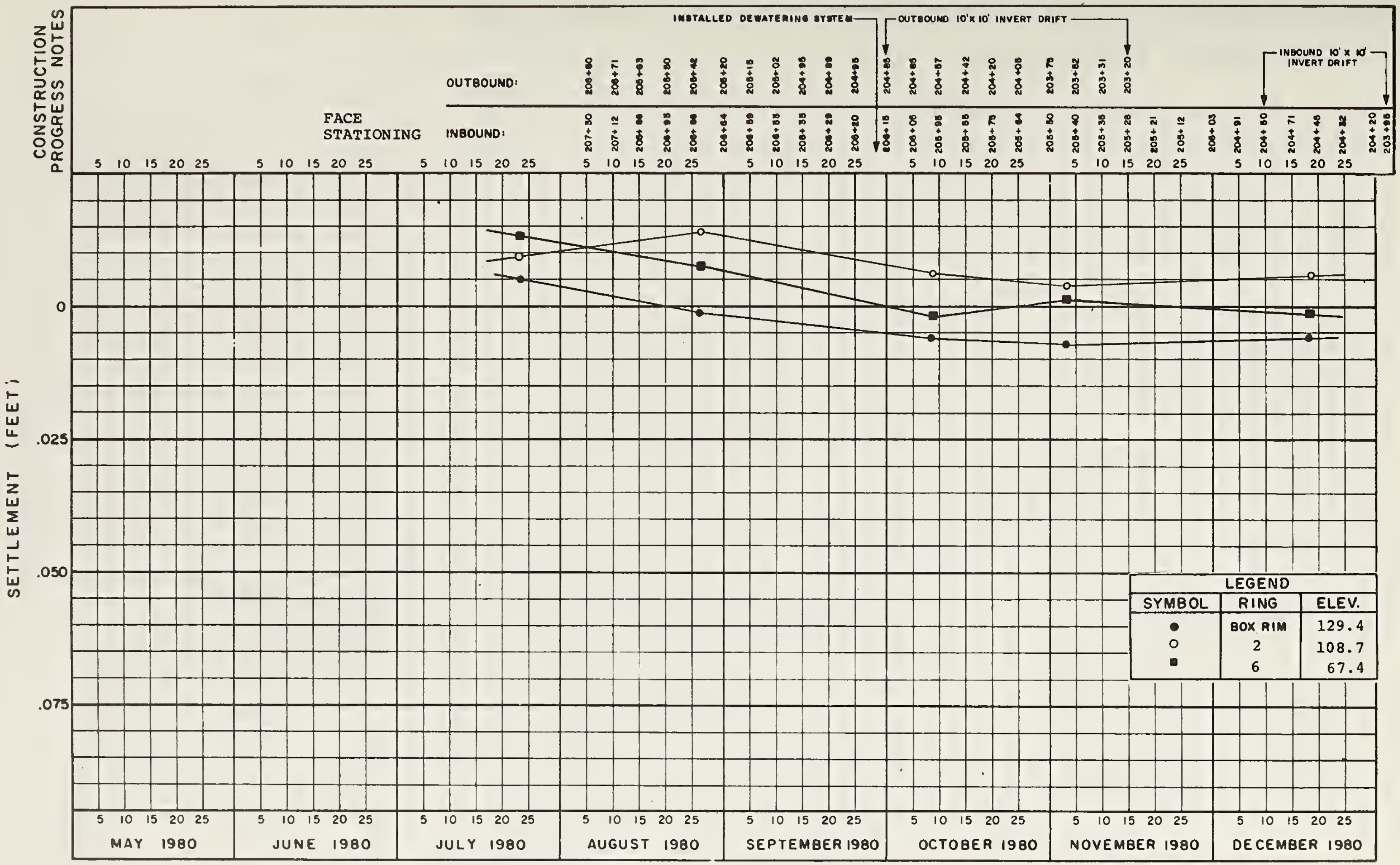
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 2 STATION IB 205+93, 1'R



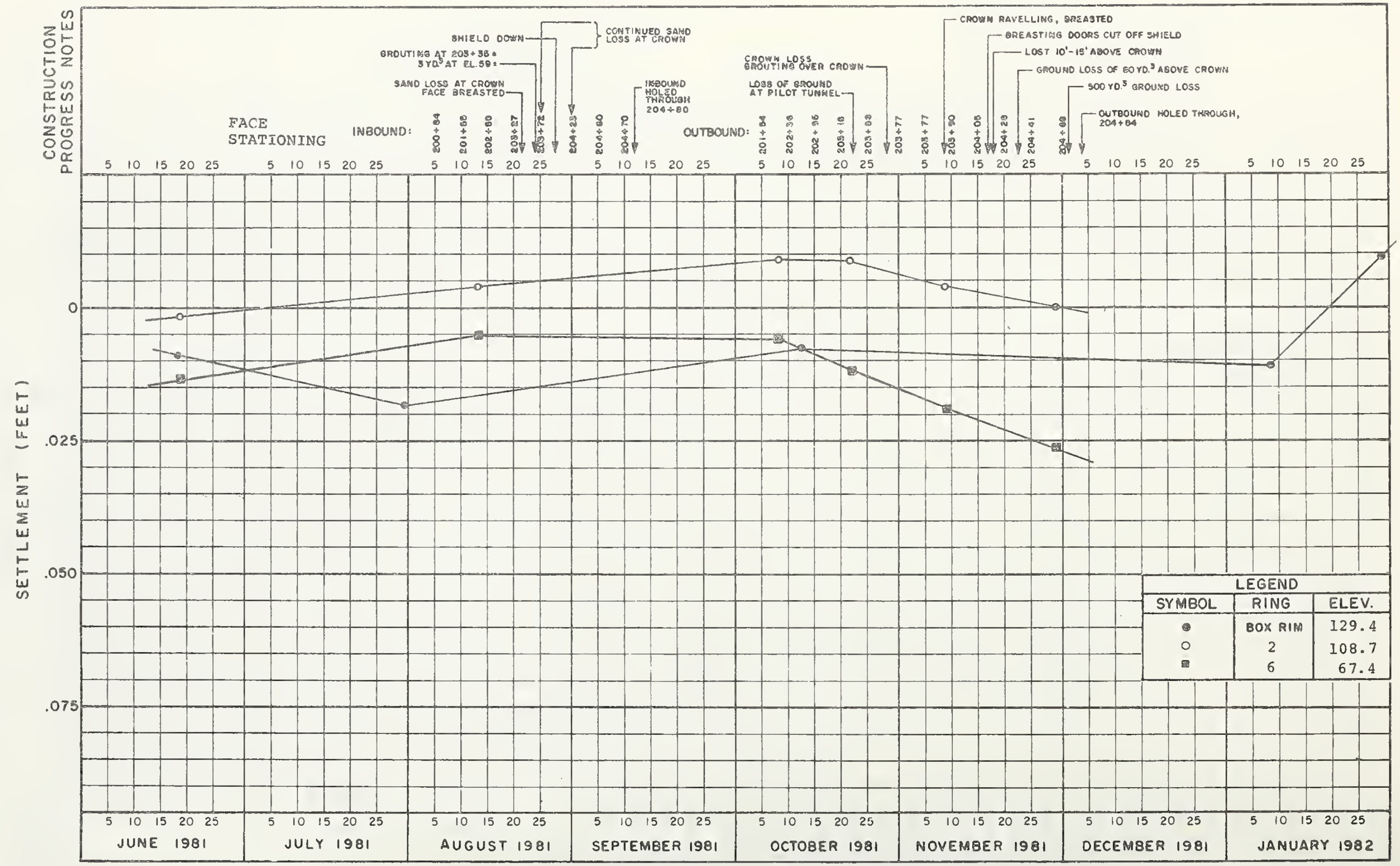
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 3 STATION OB 205+58, 2'L

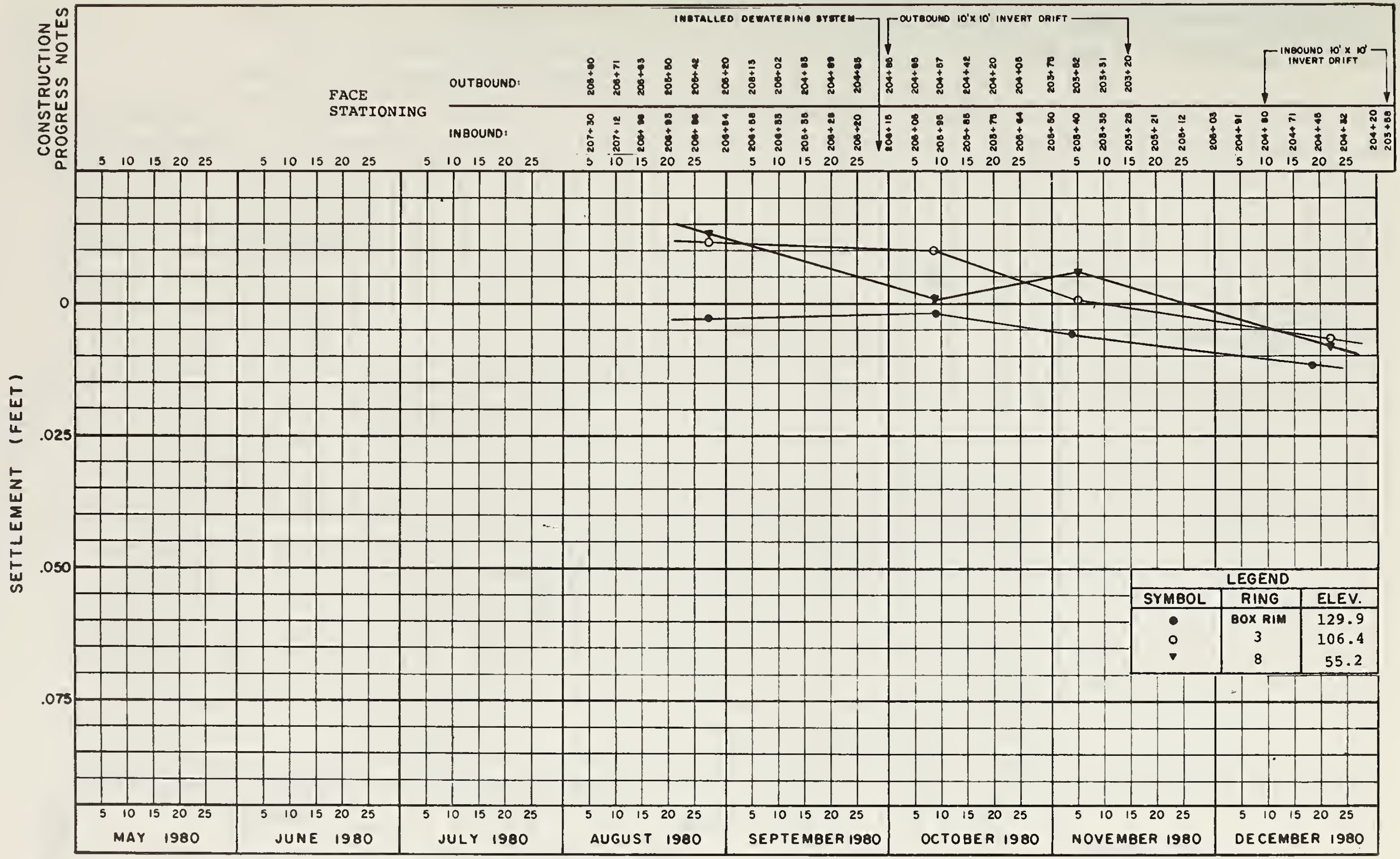


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 3 STATION OB 205+58, 2'L



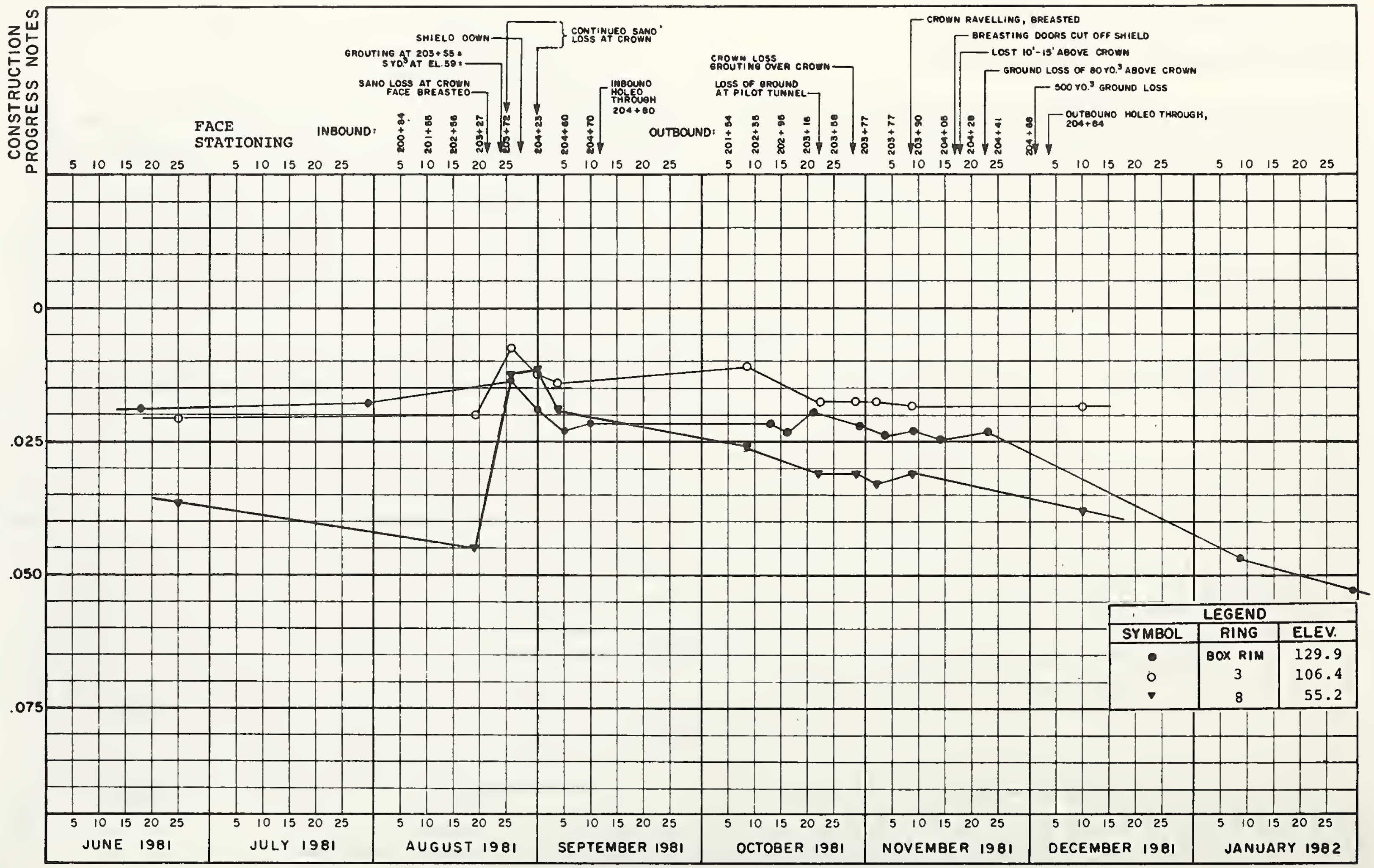
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 4 STATION IB 204+95, 4'R

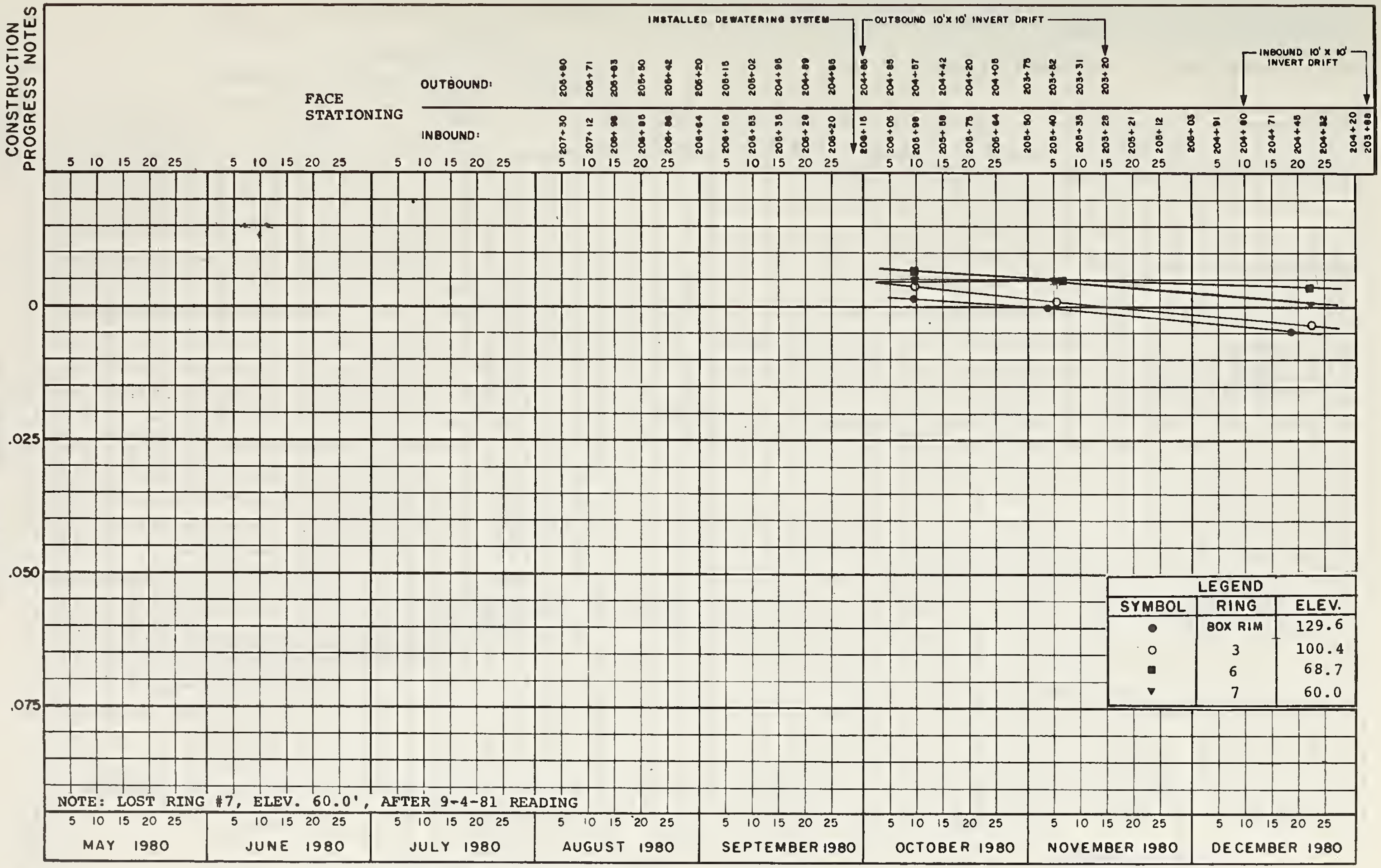


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 4 STATION IB 204+95, 4'R

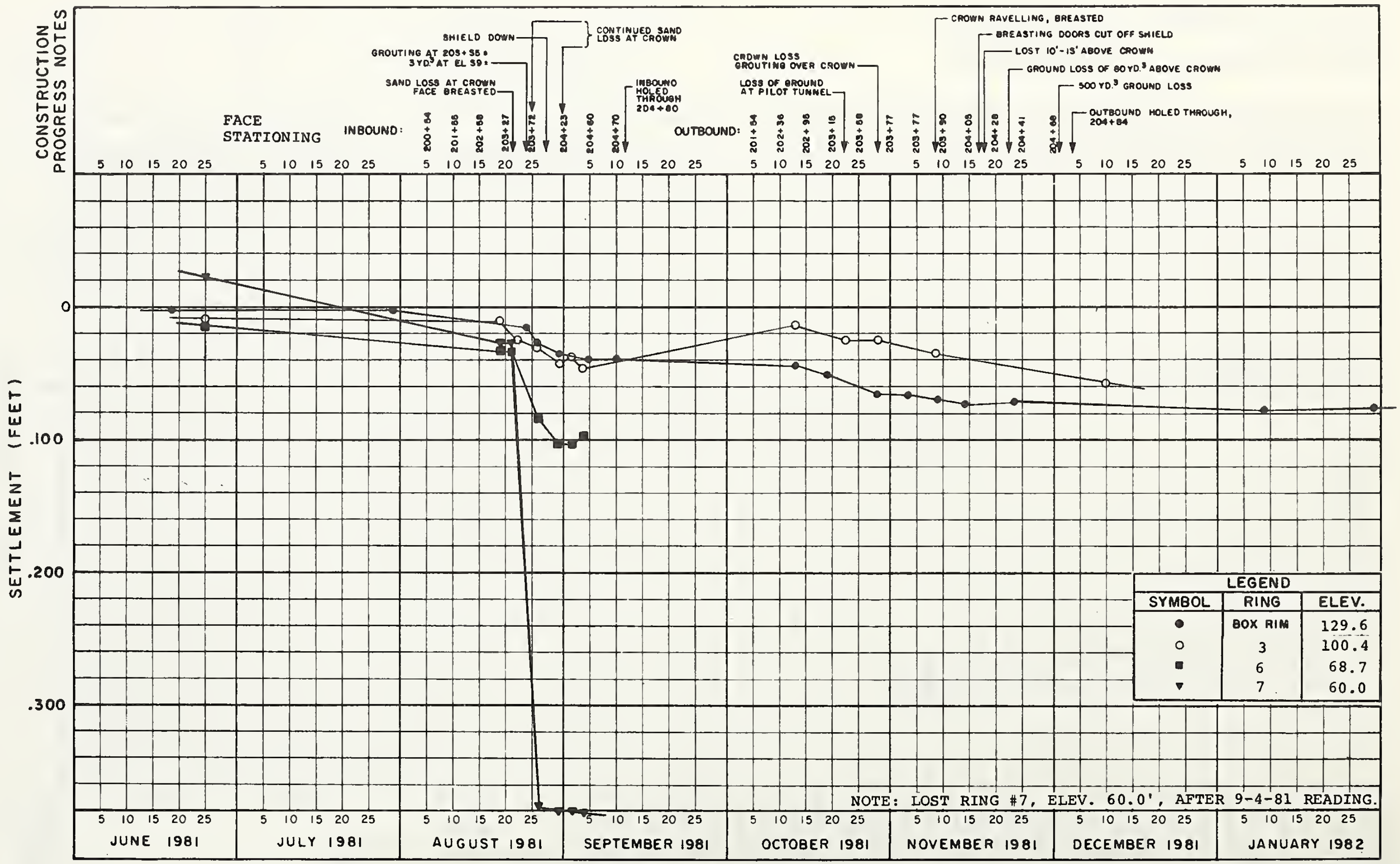


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 5 STATION IB 203+40, 3'R

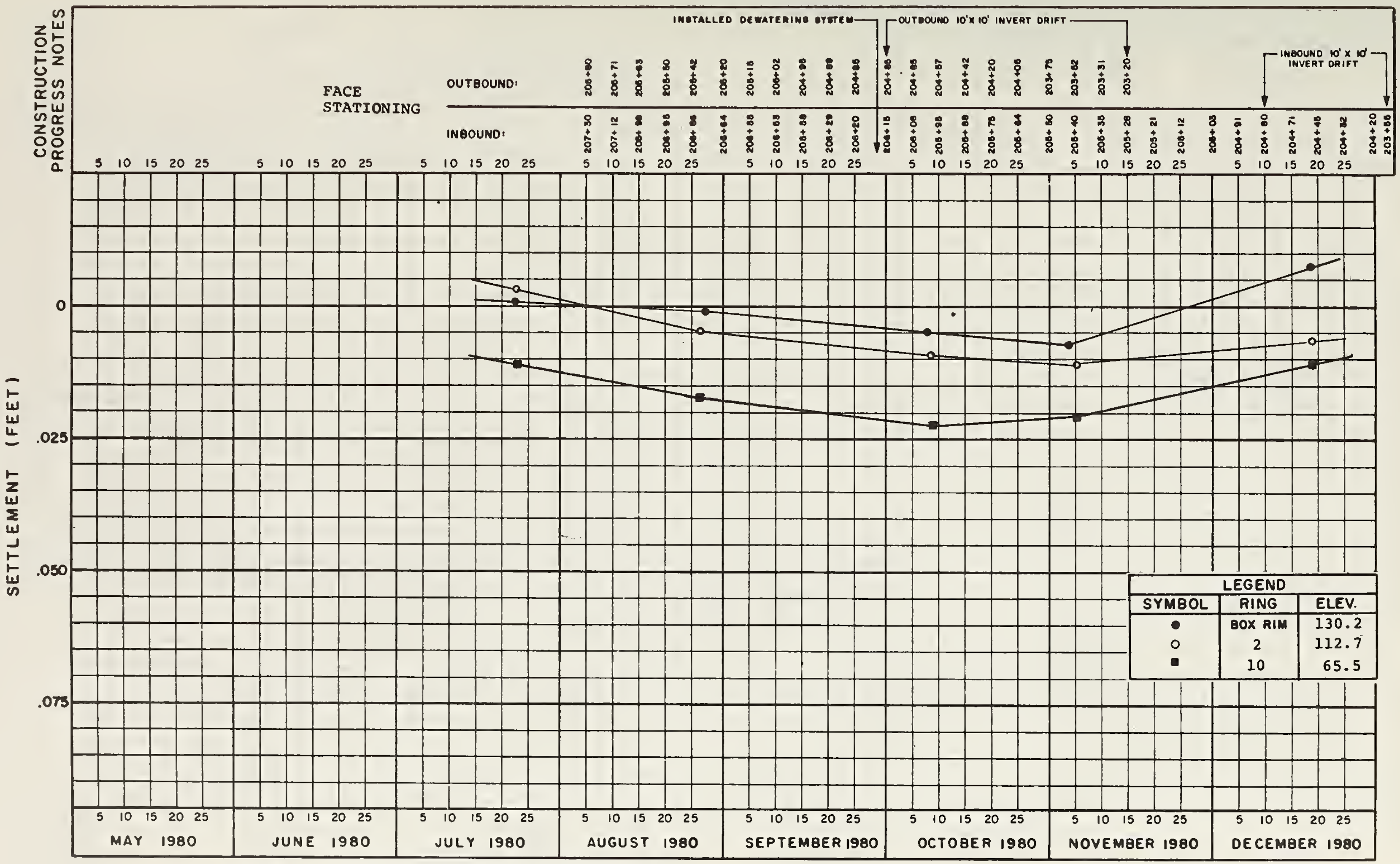


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 5 STATION IB 203+40, 3'R

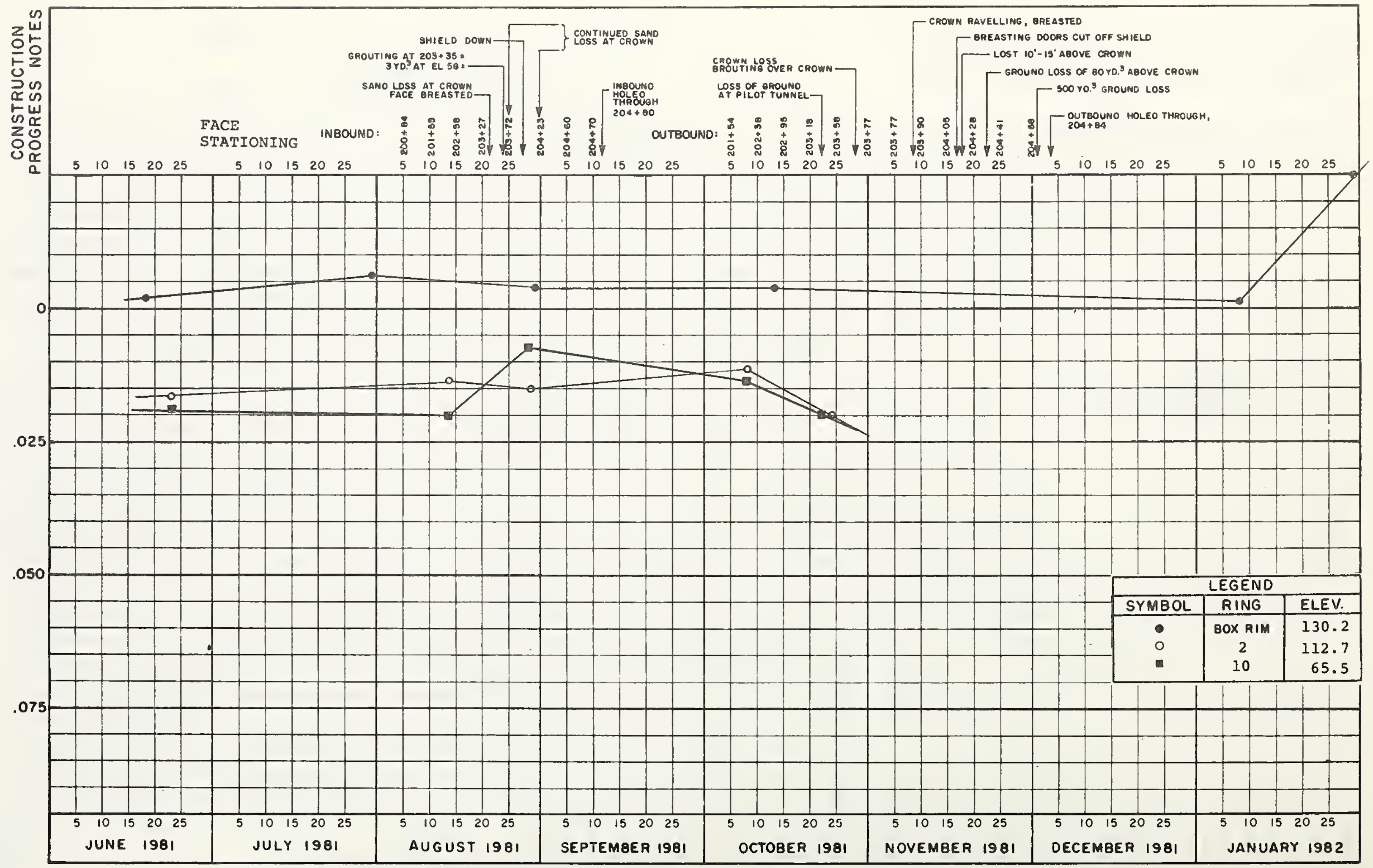


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 7 STATION IB 205+92, 28'L

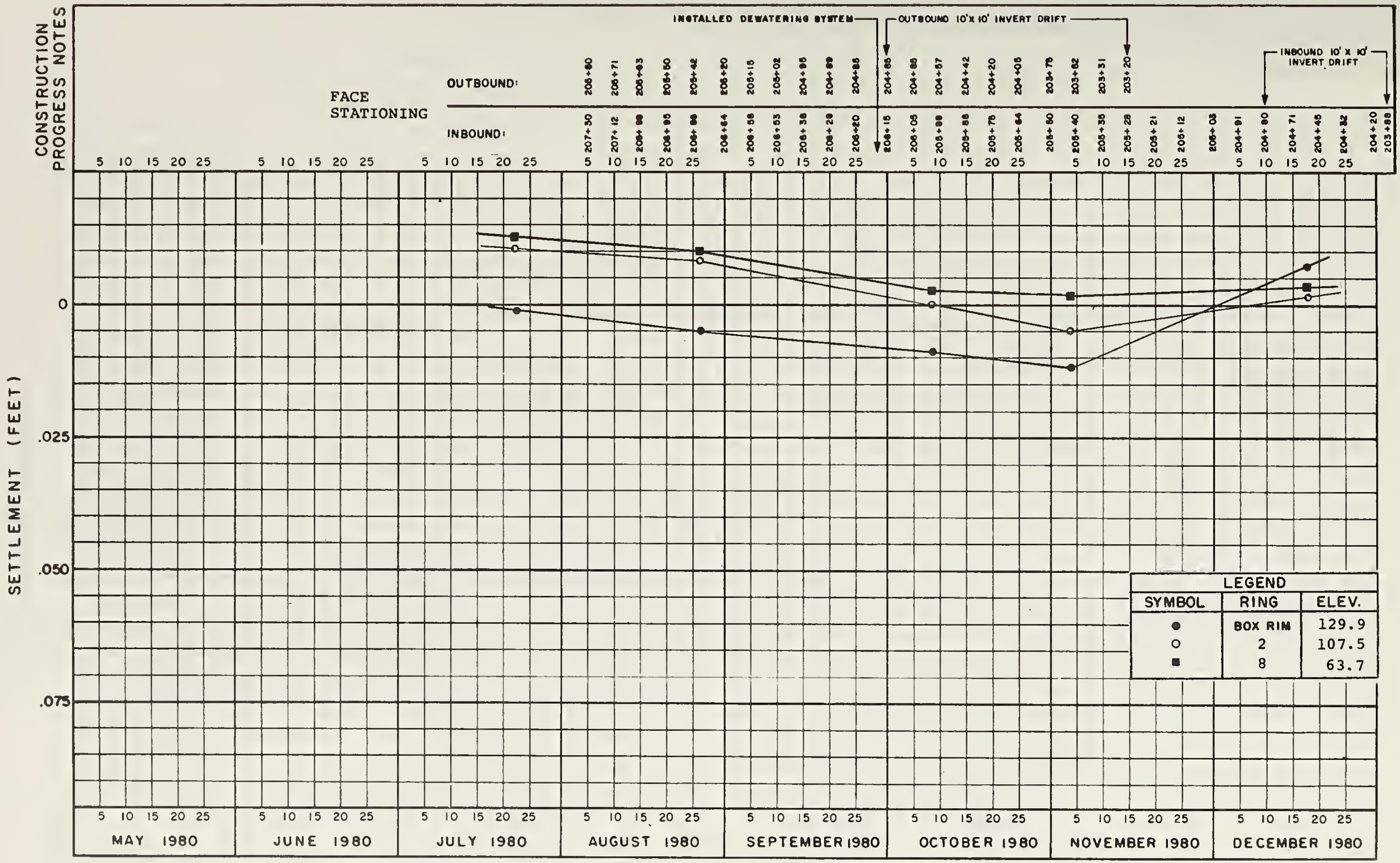


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 7 STATION IB 205+92, 28'L



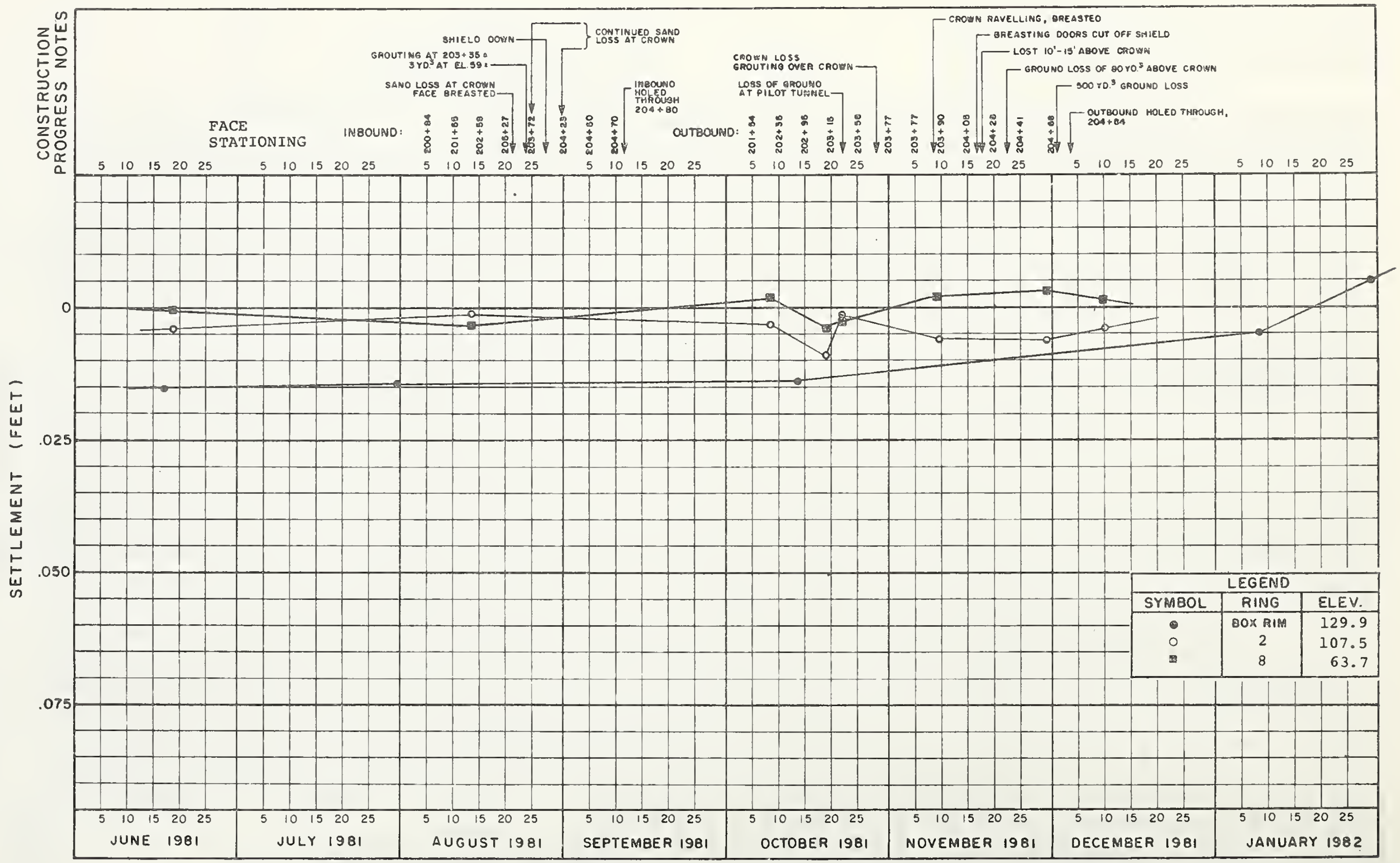
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 8 STATION OB 205+65, 10'R

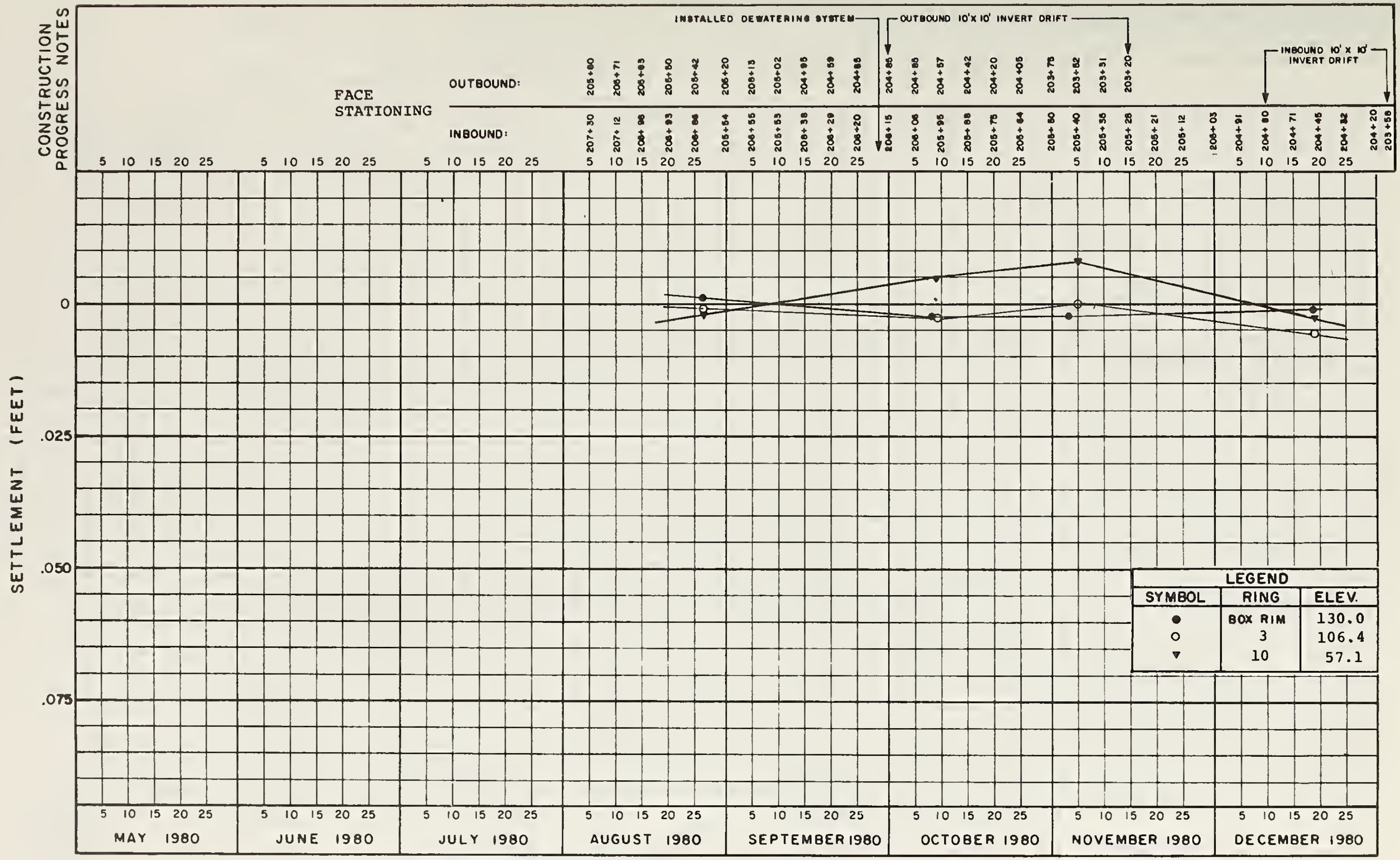


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 8 STATION OB 205+65, 10'R

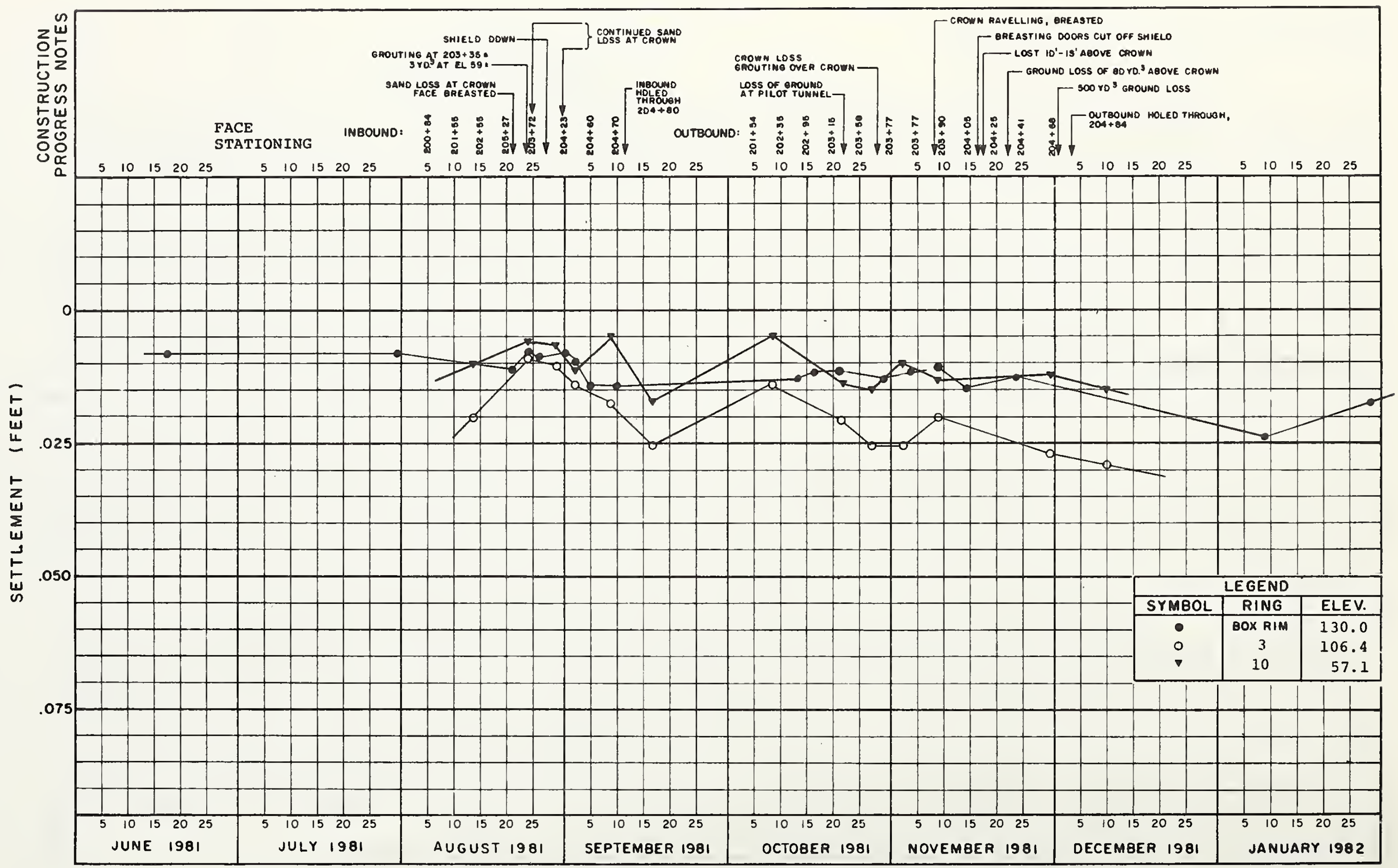


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO TSC 9 STATION IB 204+93, 27'L



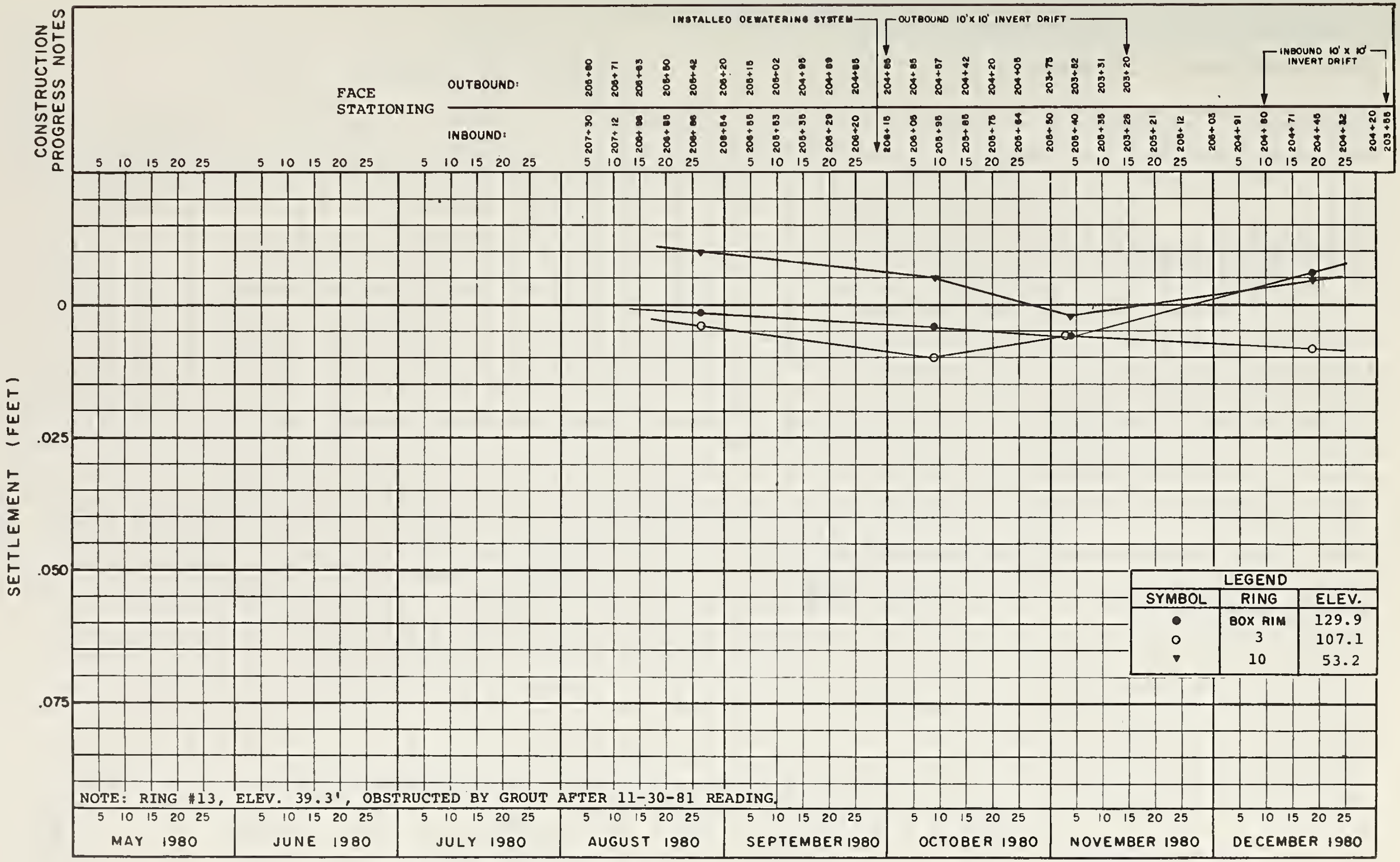
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 9 STATION IB 204+93, 27'L



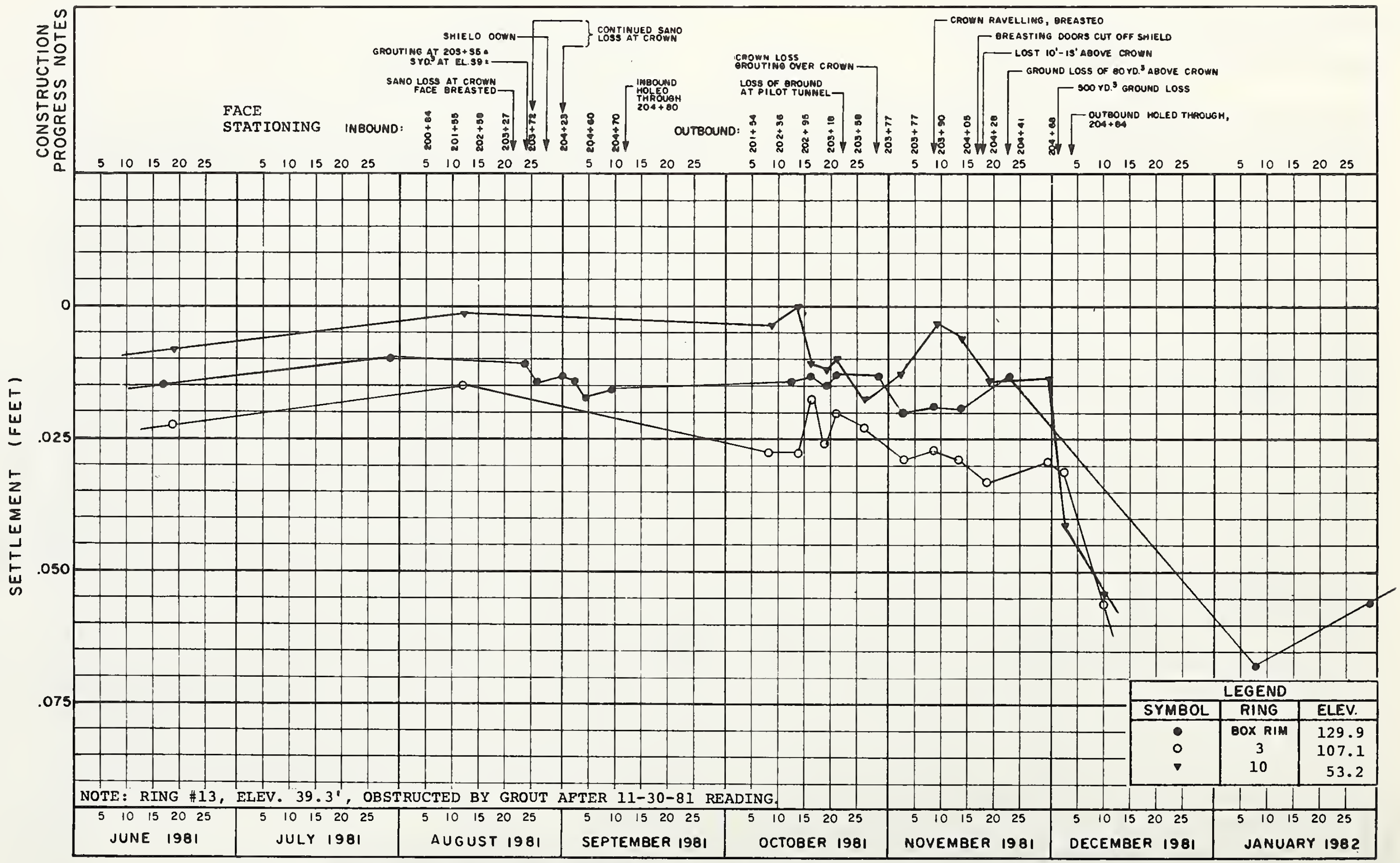
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 10 STATION OB 204+59, 11.5'R



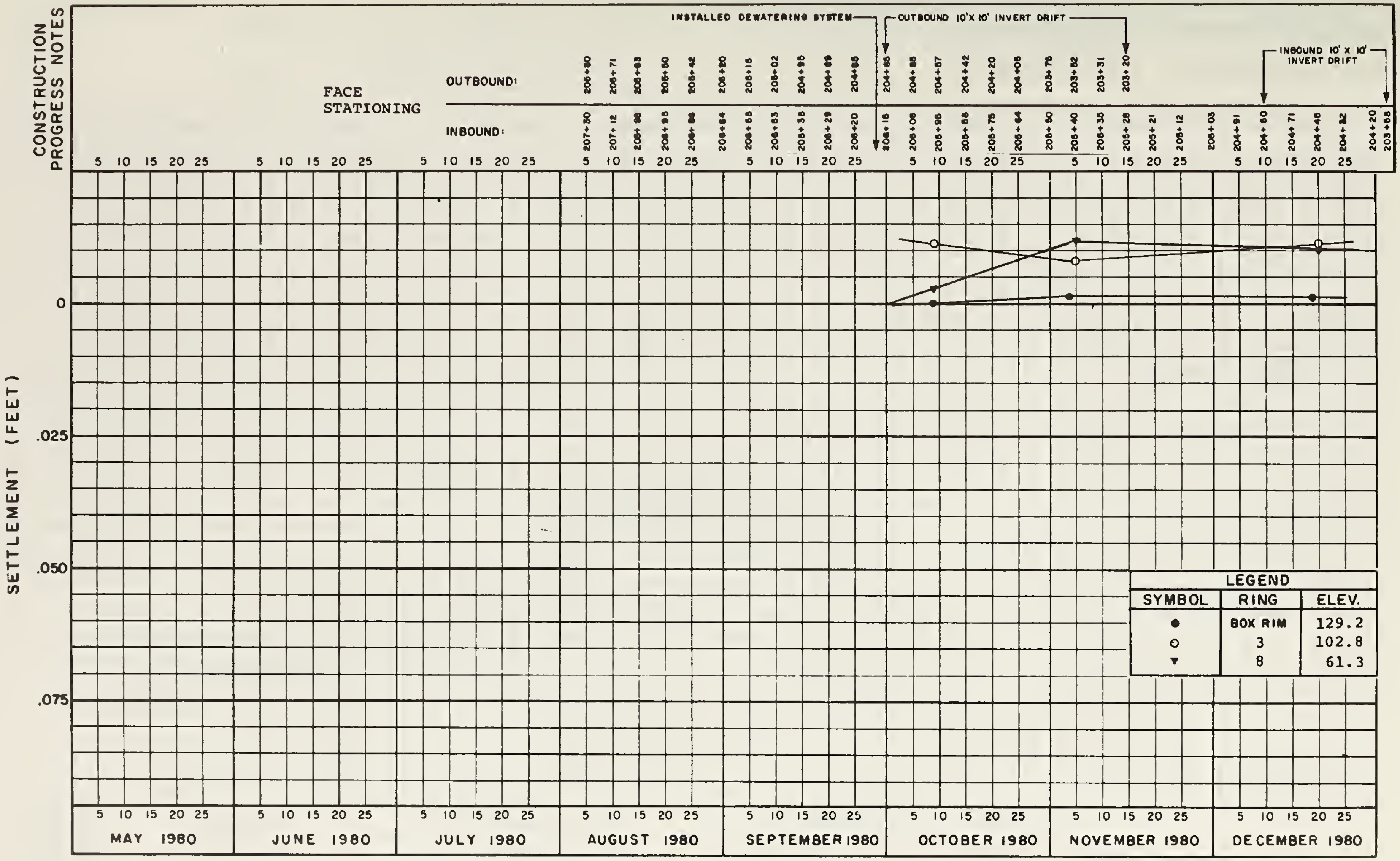
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 10 STATION OB 204+59, 11.5'R



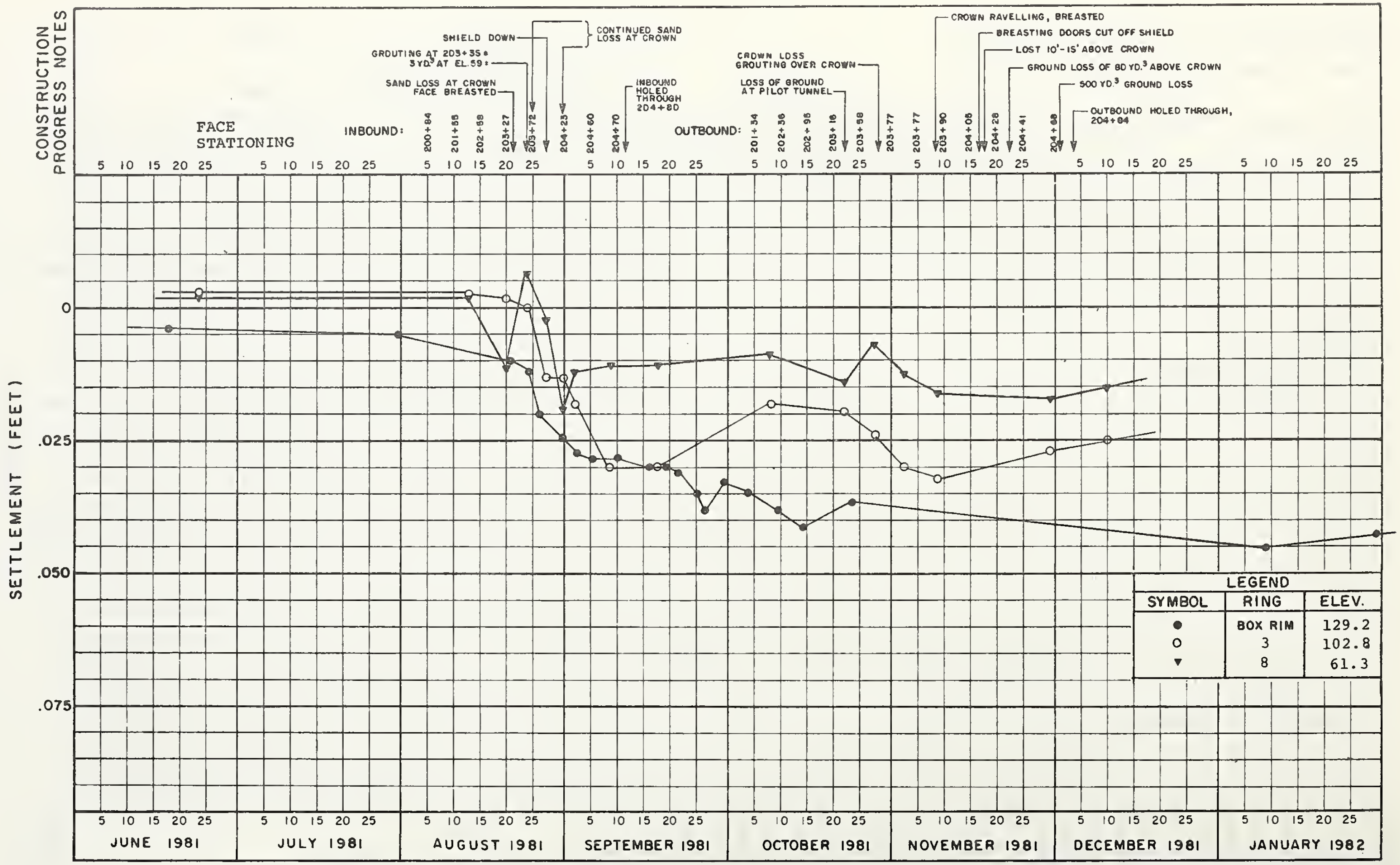
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 11 STATION IB 203+41, 29'L

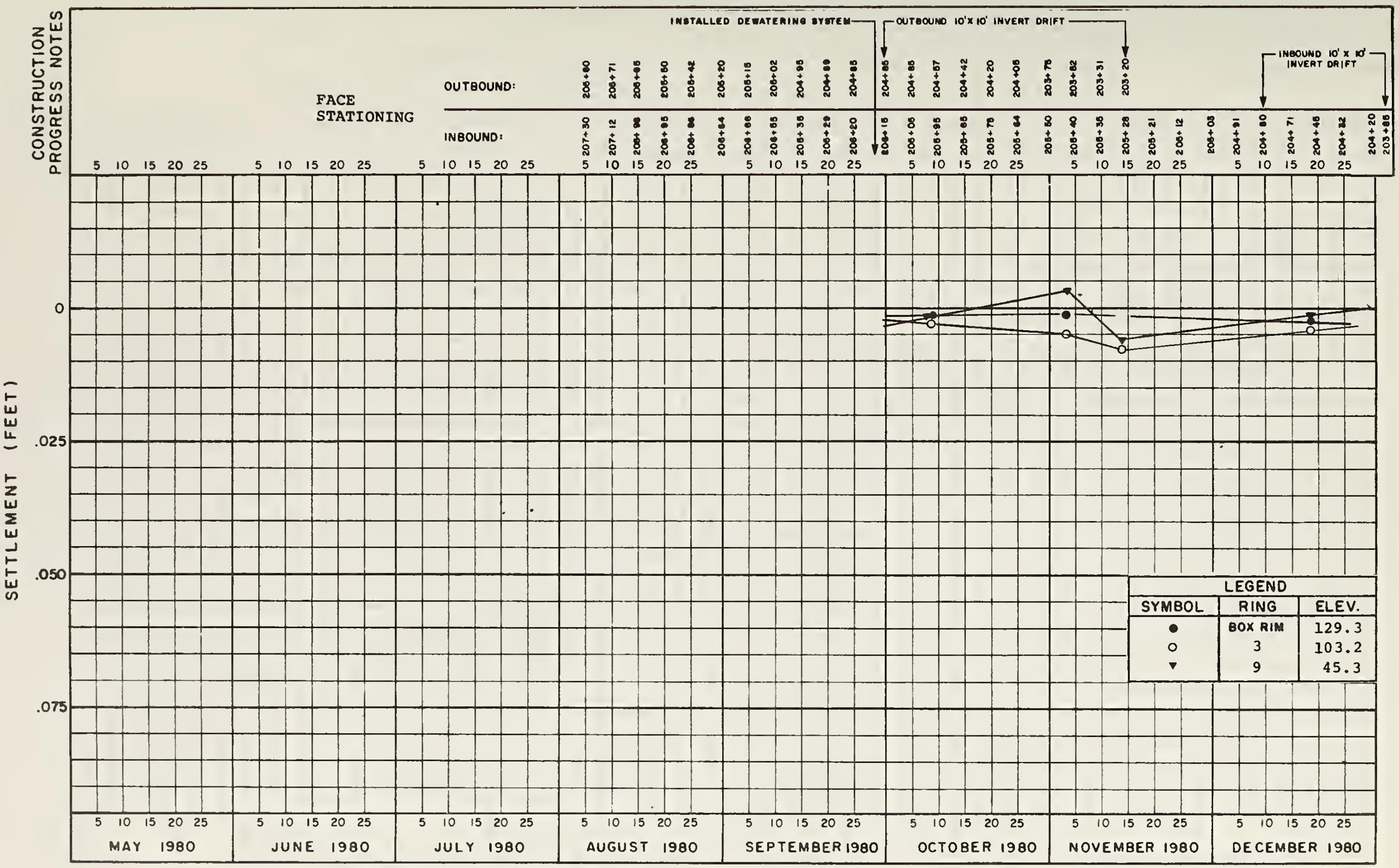


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 11 STATION IB 203+41, 29'L



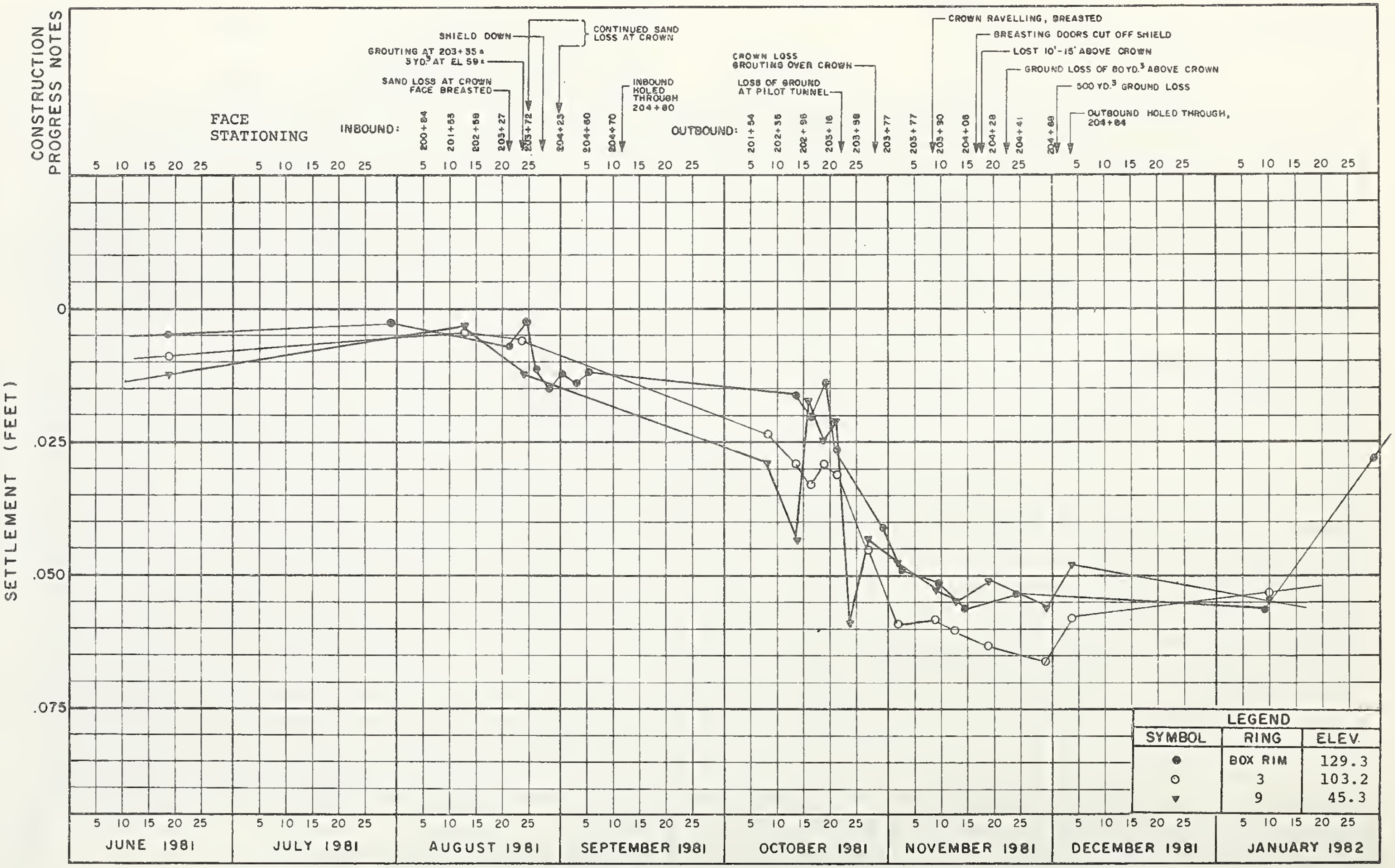
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 12 STATION OB 203+12, 10.5'R



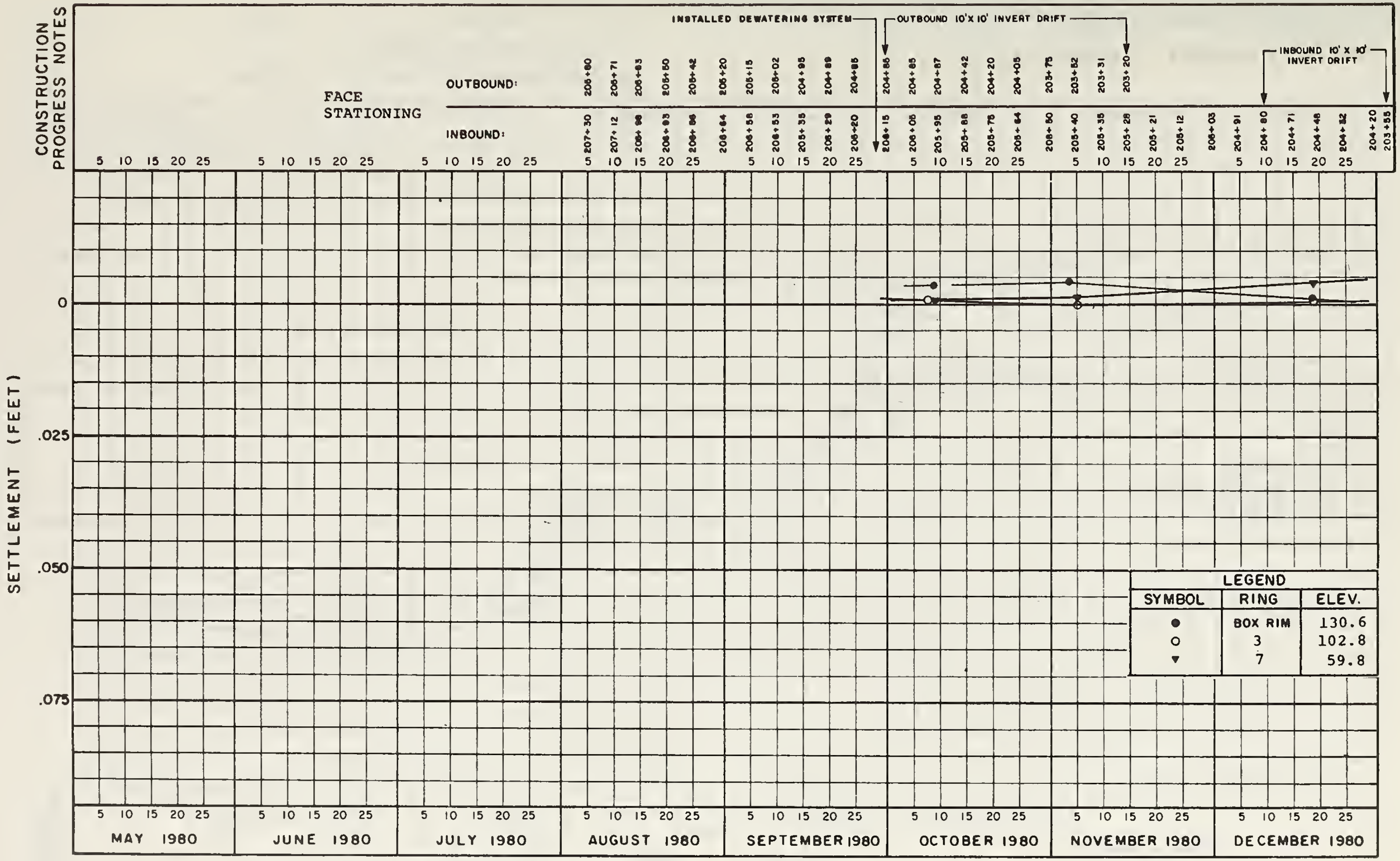
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 12 STATION OB 203+12, 10.5'R

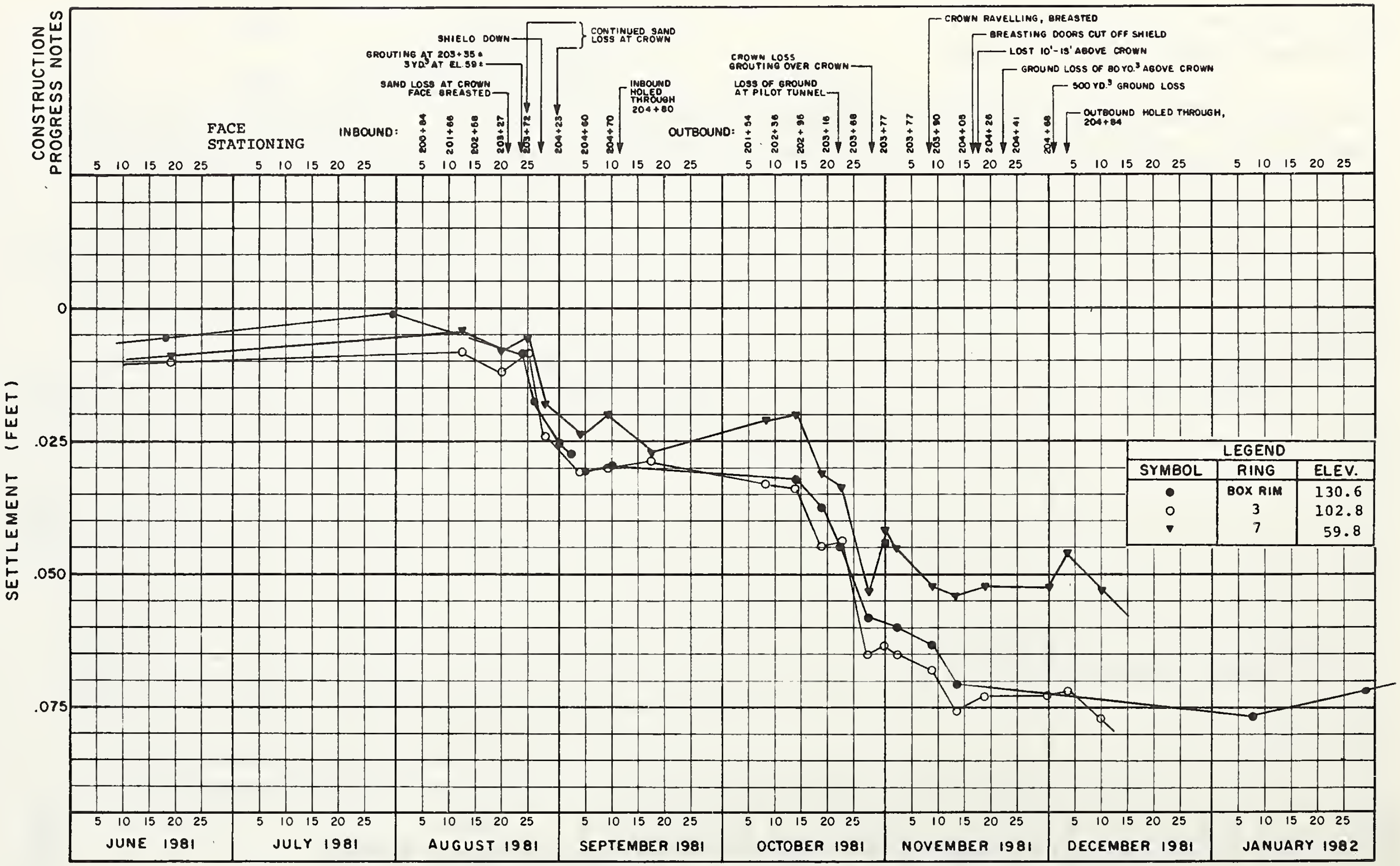


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 13 STATION IB 203+41, 17'R

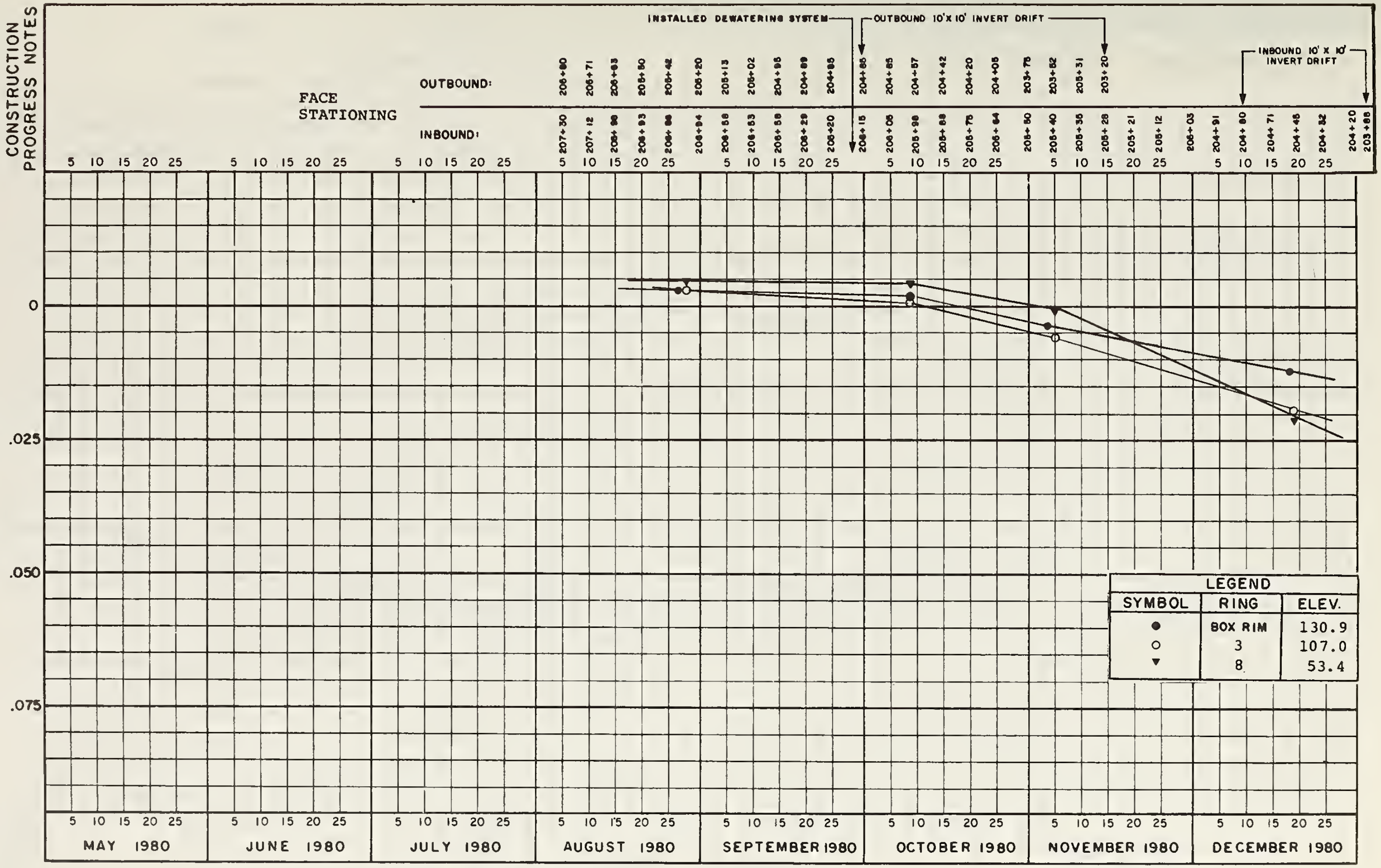


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 13 STATION IB 203+41, 17'R

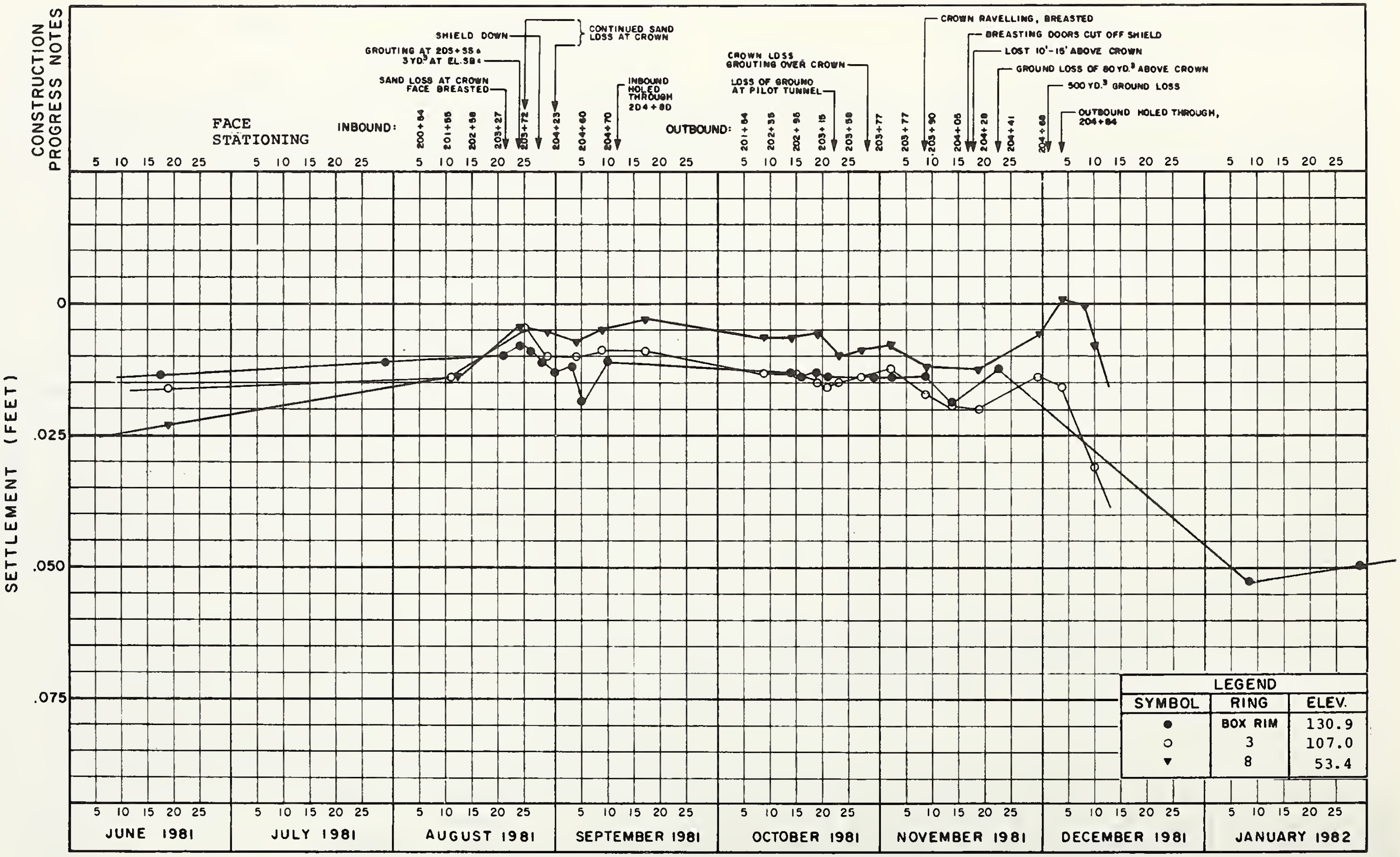


PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 14 STATION IB 204+95, 16'R



LEGEND		
SYMBOL	RING	ELEV.
●	BOX RIM	130.9
○	3	107.0
▼	8	53.4

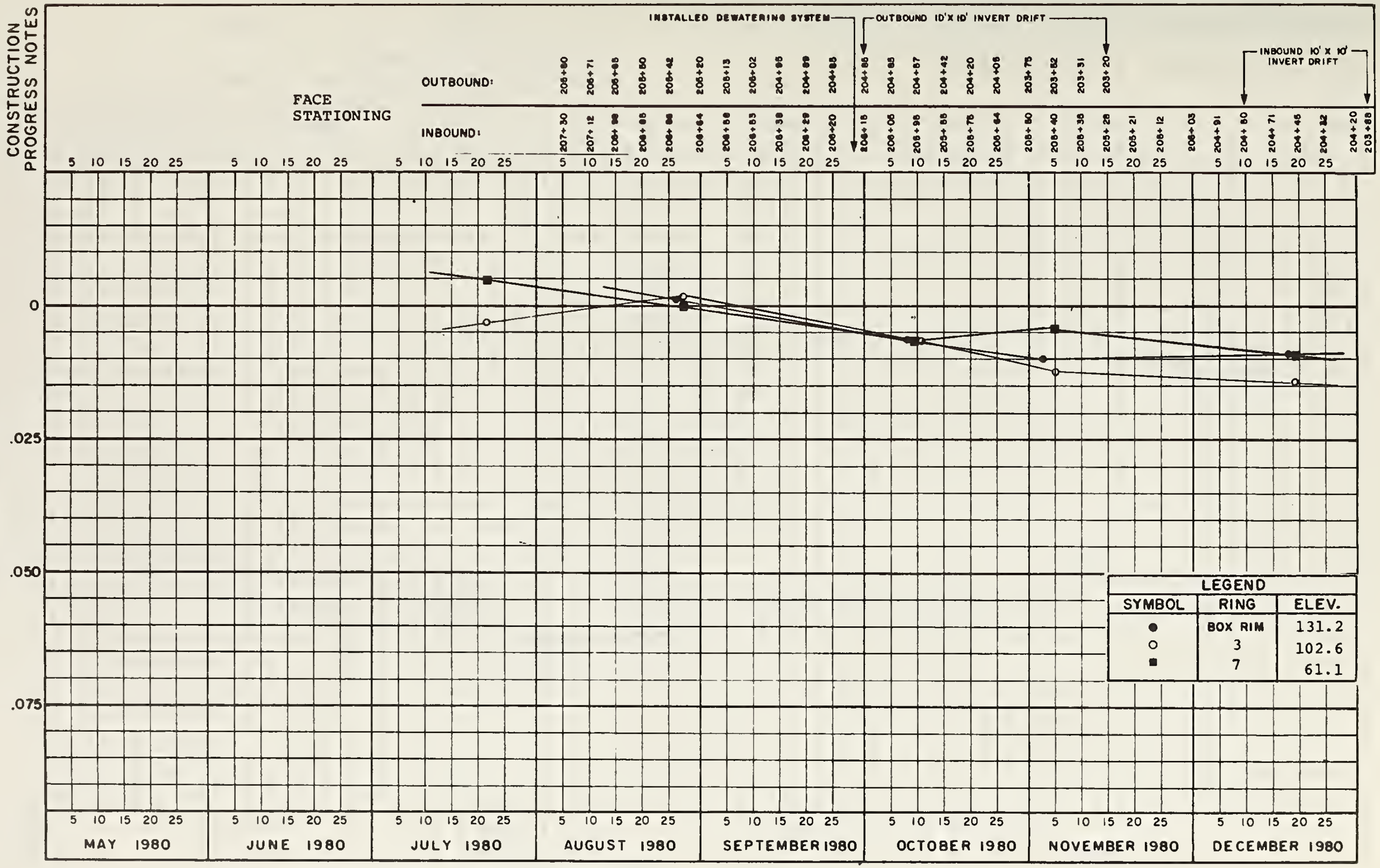
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 14 STATION IB 204+95, 16'R



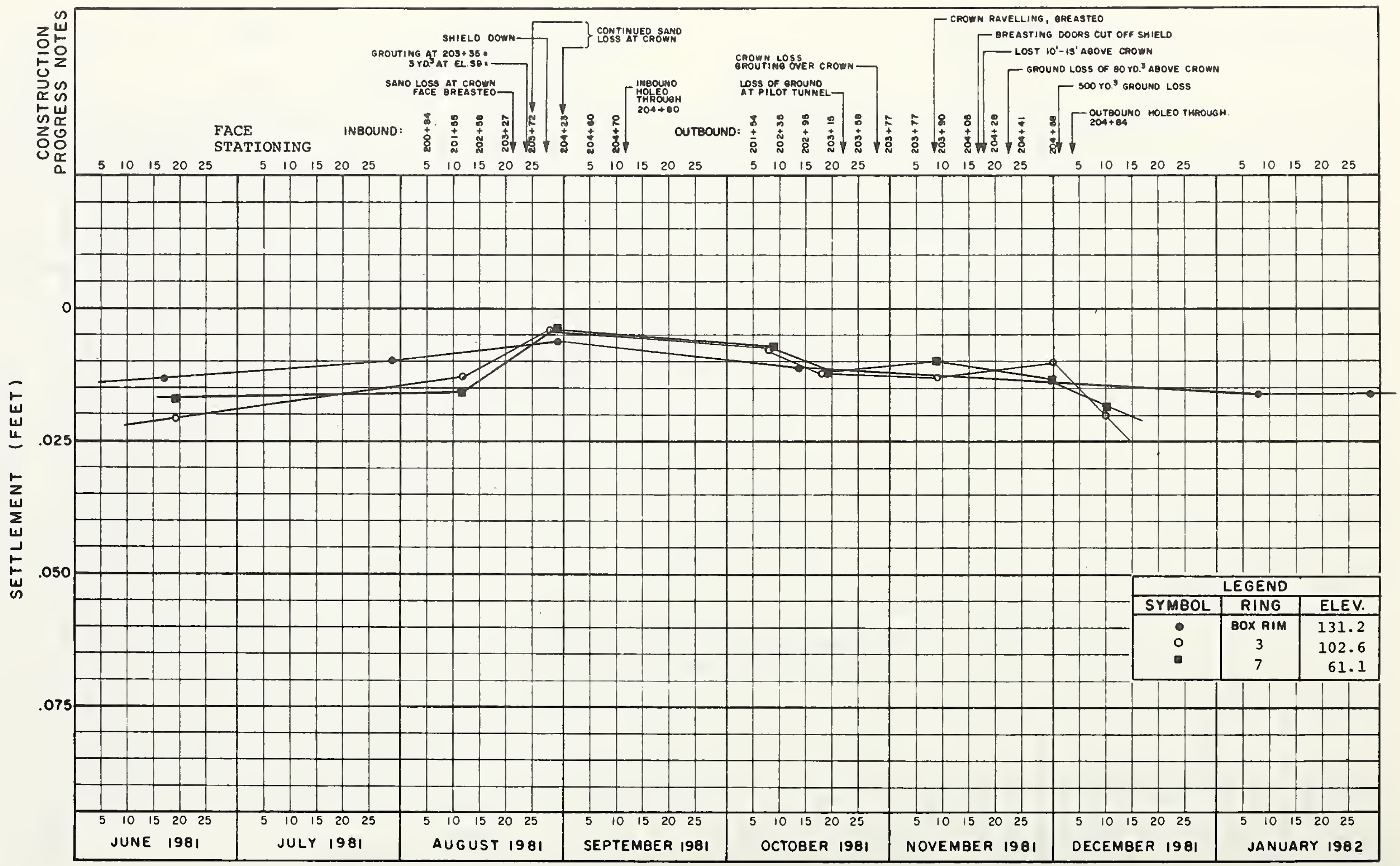
PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 15 STATION IB 205+89, 16'R



PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS

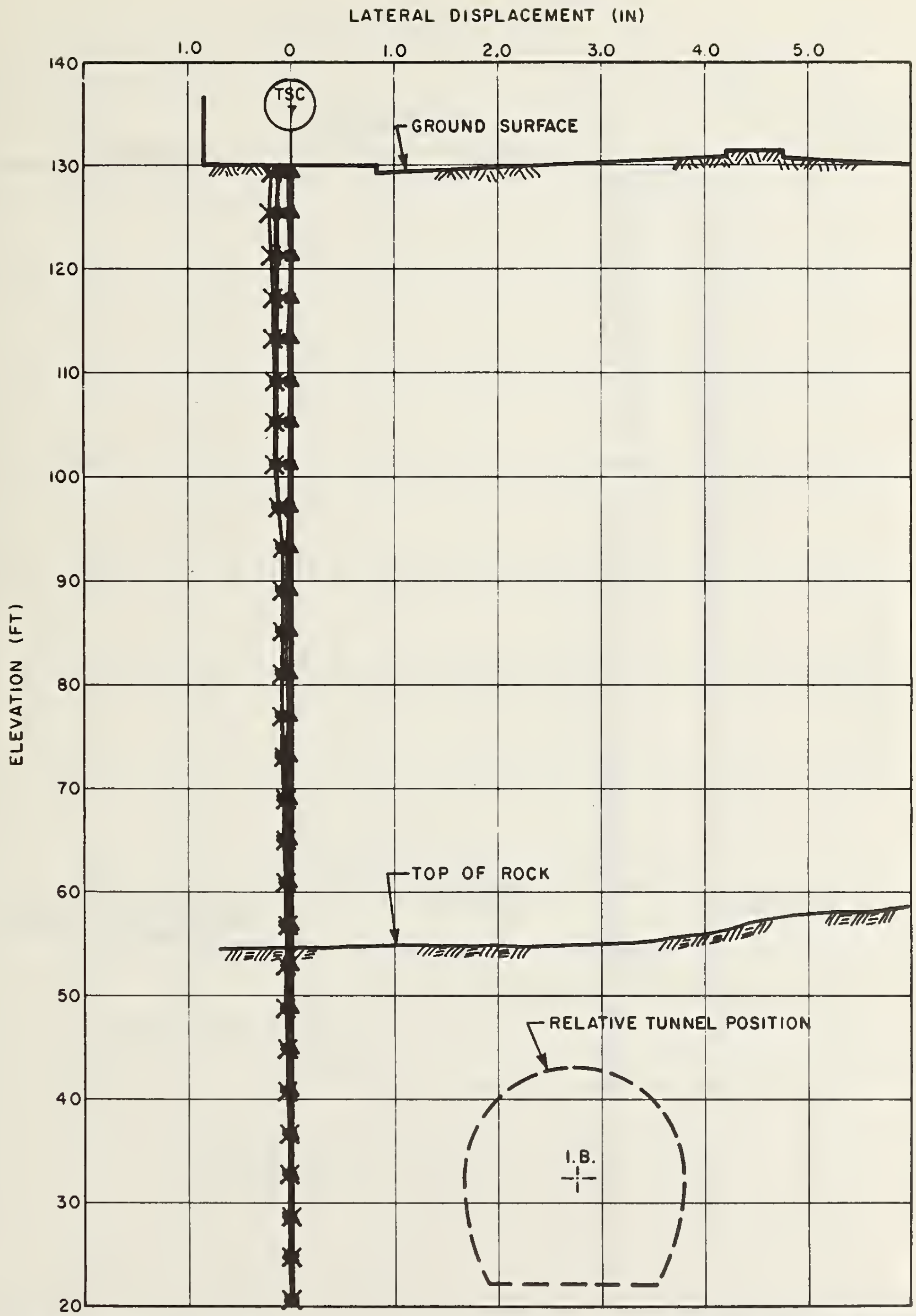
CLIENT DOT/TSC

SETTLEMENT OBSERVATIONS

CASING NO. TSC 15 STATION IB 205+89, 16'R

APPENDIX F

INCLINOMETER OBSERVATIONS



SYM	HOLE	DATE	TIME	AZI
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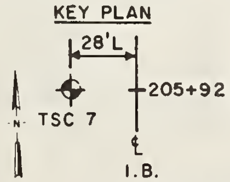
□	7	07/22/80	15:15	90.
▽	7	08/28/80	11:15	90.
◇	7	10/09/80	12:45	90.
X	7	11/13/80	11:15	90.

KEY PLAN

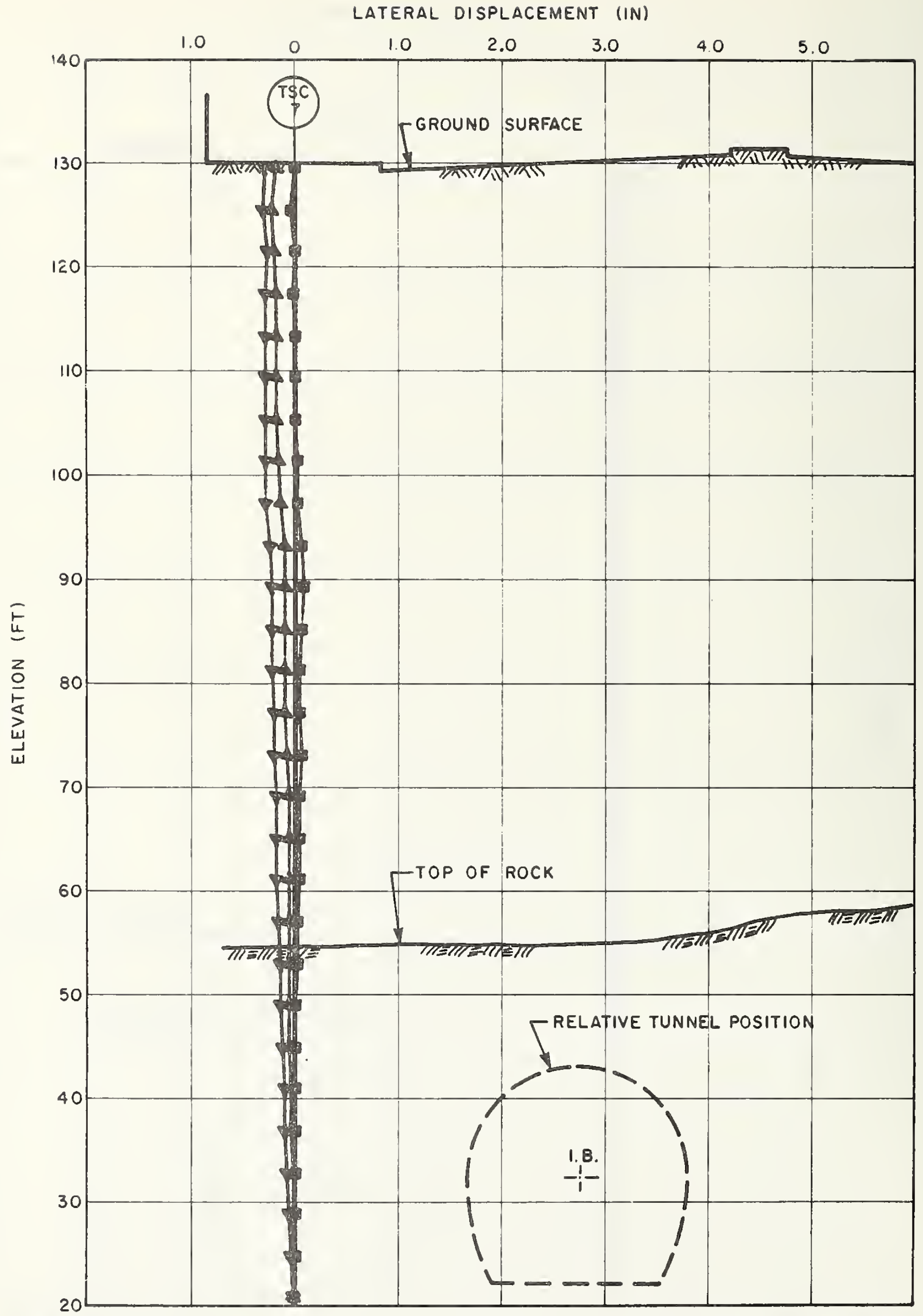


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 7



SYM	HOLE	DATE	TIME	AZI
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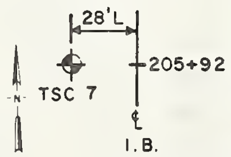
□	7	12/22/80	14:40	90.
▽	7	10/21/81	14:00	90.

KEY PLAN



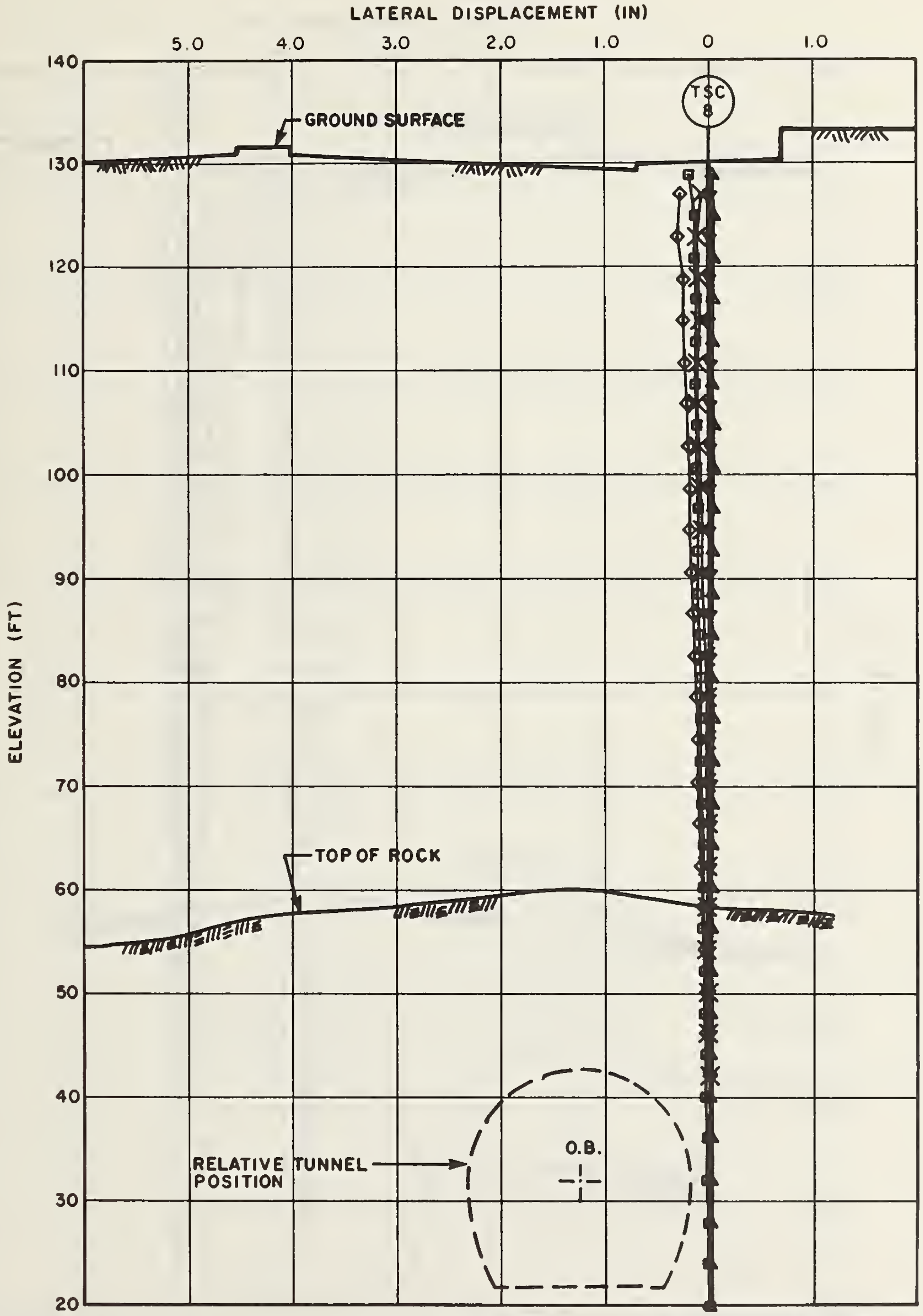
PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 7



SYM	HOLE	DATE	TIME	AZI
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▽	8	10/21/80	14:00	270.
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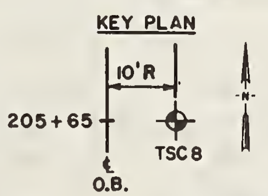
X	8	12/22/80	14:15	270.

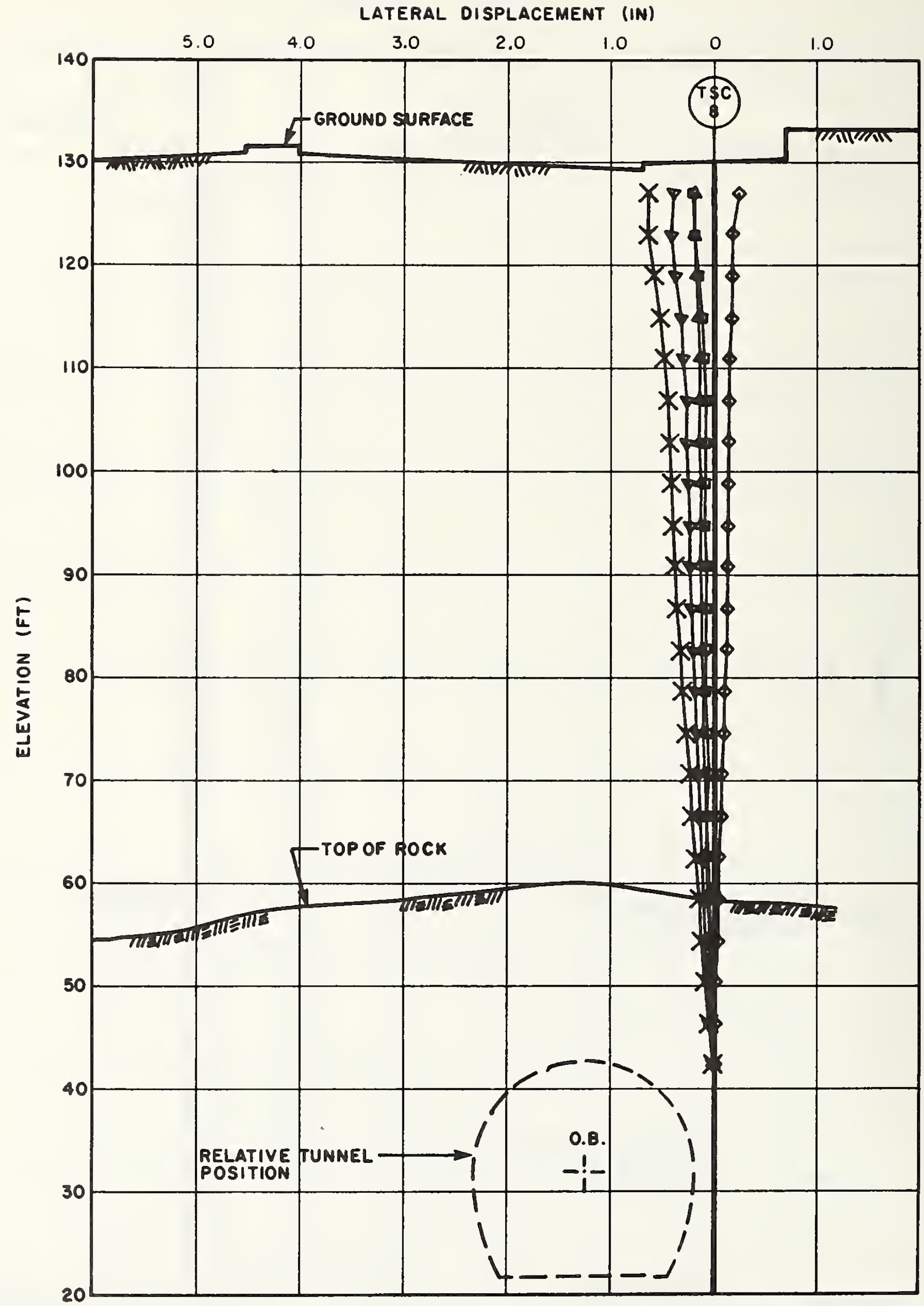


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 8



SYM	HOLE	DATE	TIME	AZI
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□	8	10/26/81	09:45	270.
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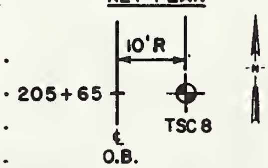
KEY PLAN

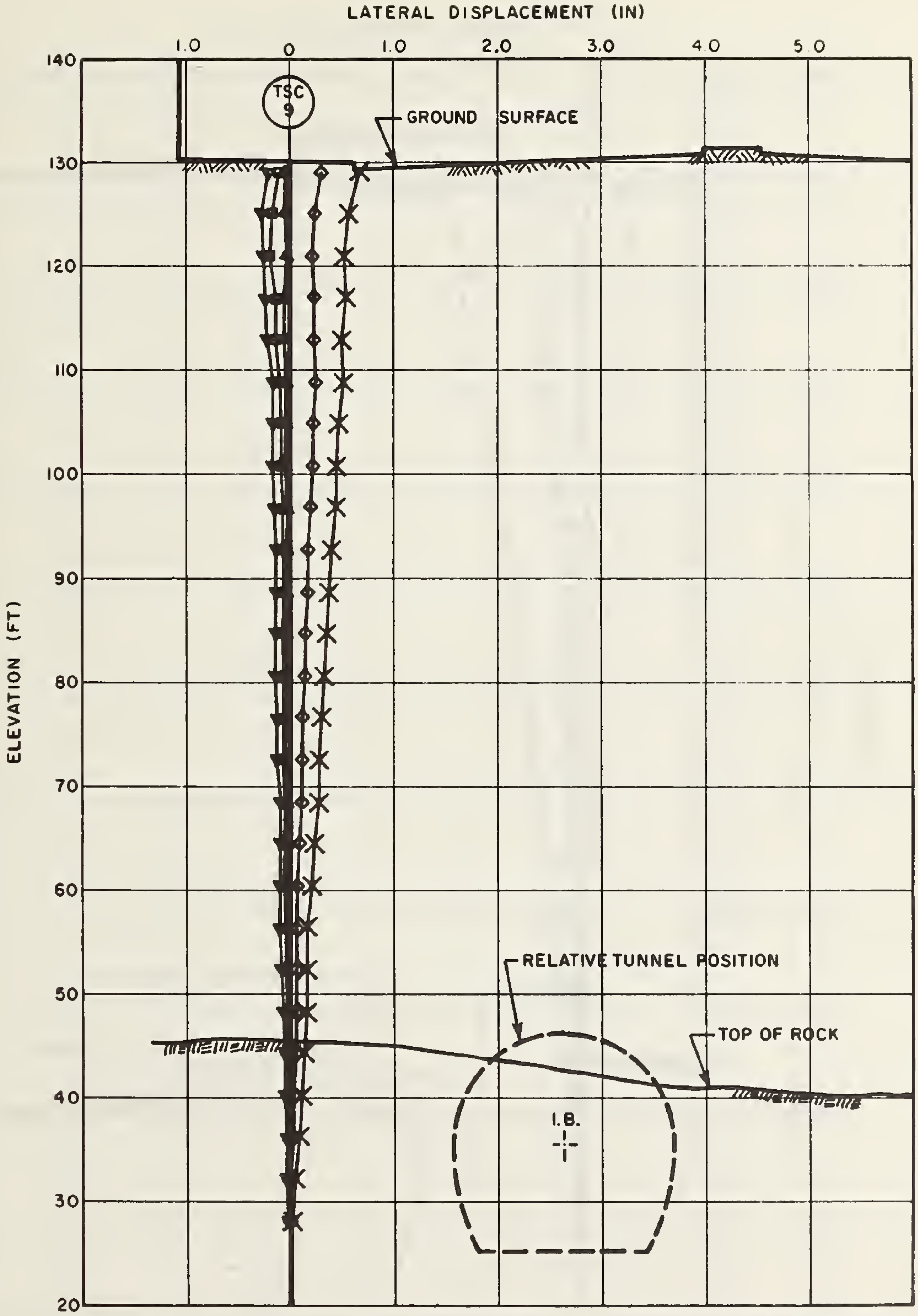


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 8



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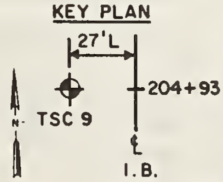
X	9	12/22/80	12:20	90.

KEY PLAN

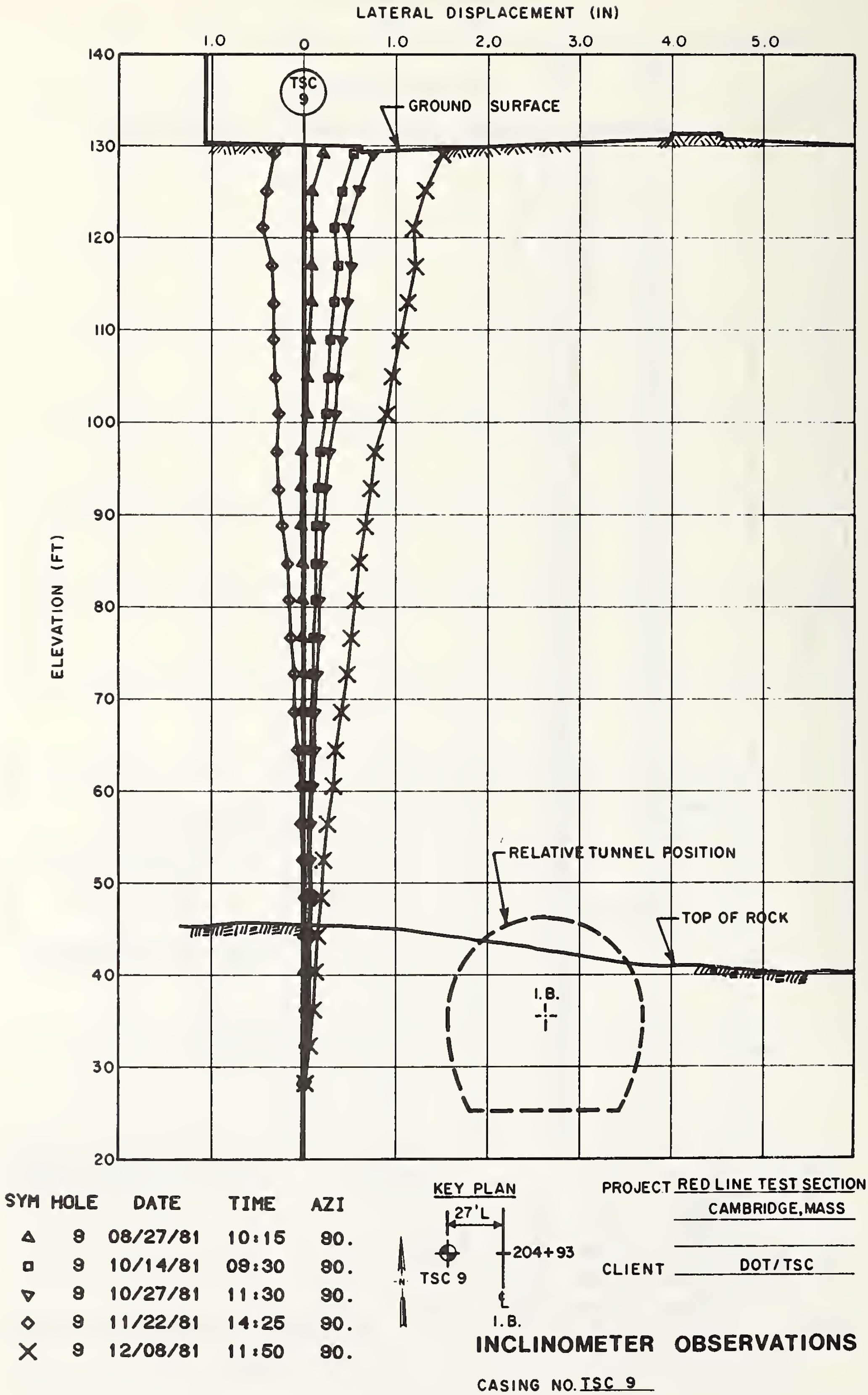


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

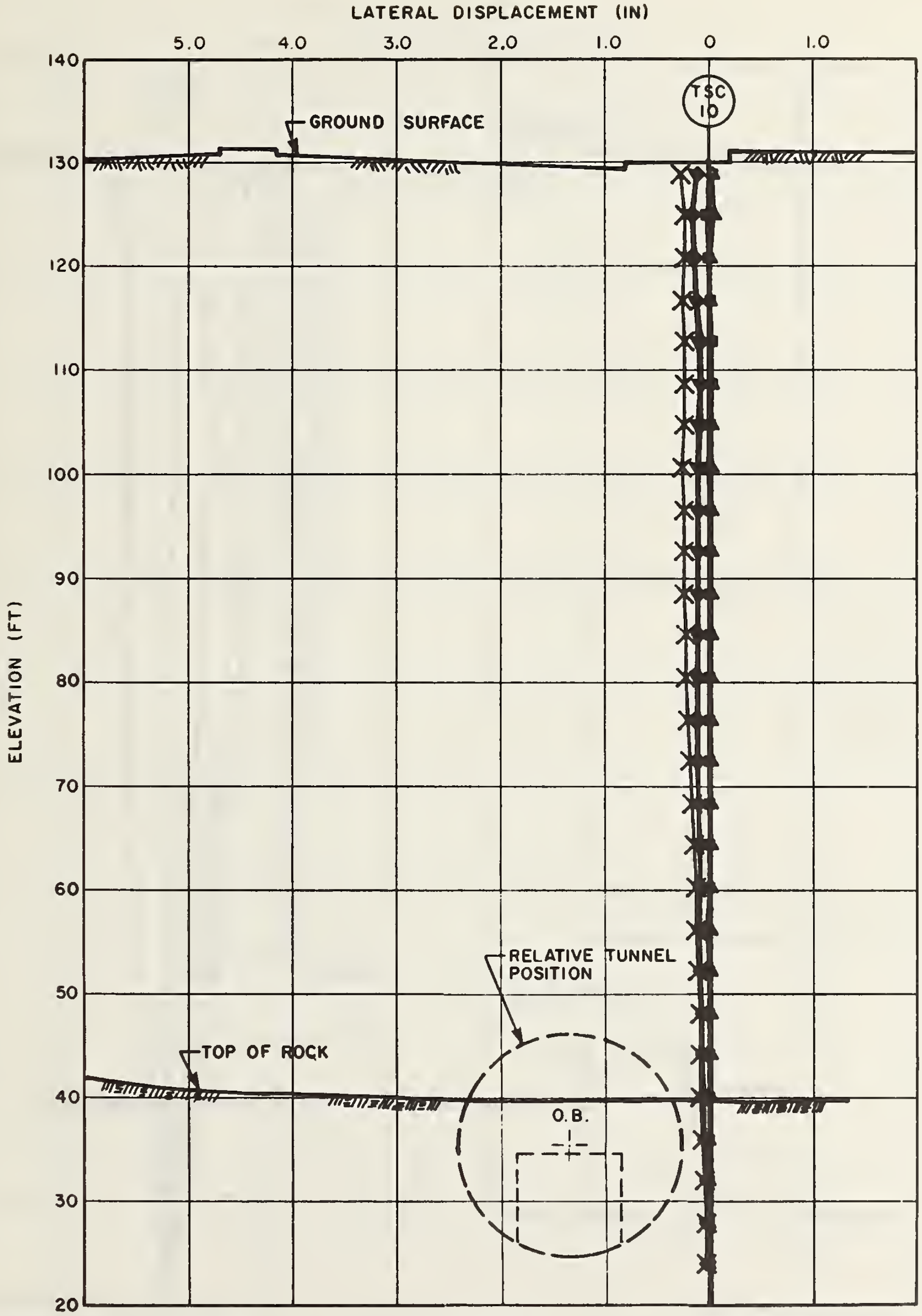
CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 9



LATERAL DISPLACEMENT (IN)
1.0
0
1.0
2.0
3.0
4.0
5.0
ELEVATION (FT)
140
130
120
110
100
90
80
70
60
50
40
30
20
TSC 9
GROUND SURFACE
RELATIVE TUNNEL POSITION
TOP OF ROCK
I.B.
SYM HOLE DATE TIME AZI
9 08/27/81 10:15 90.
9 10/14/81 09:30 90.
9 10/27/81 11:30 90.
9 11/22/81 14:25 90.
9 12/08/81 11:50 90.
KEY PLAN
27'L
204+93
TSC 9
N
I.B.
PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS
CLIENT DOT/TSC
INCLINOMETER OBSERVATIONS
CASING NO. TSC 9



SYM	HOLE	DATE	TIME	AZI
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□	10	08/28/80	11:55	270.
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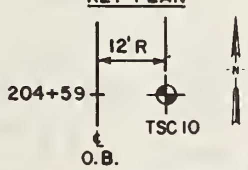
KEY PLAN

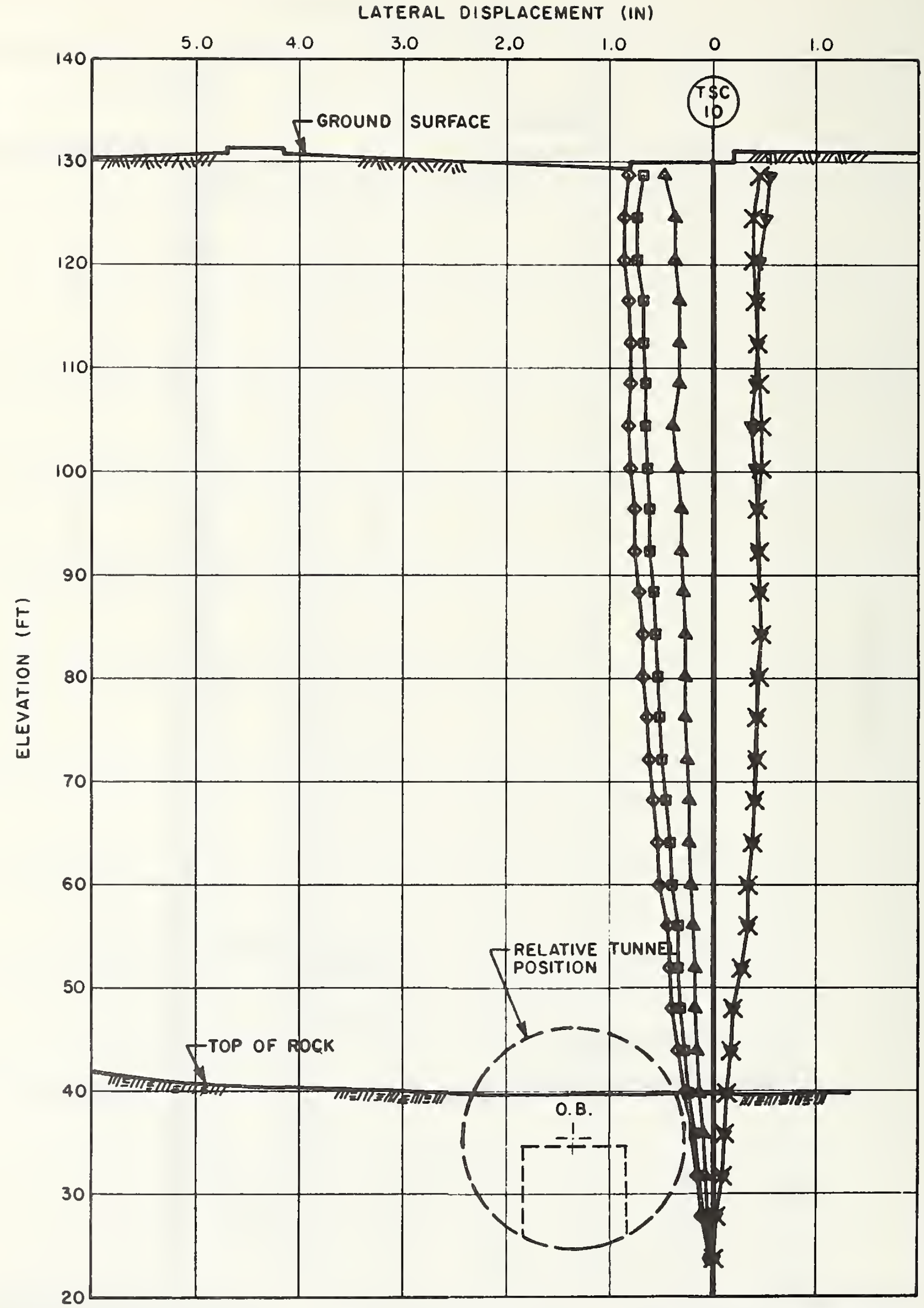


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 10



SYM	HOLE	DATE	TIME	AZI
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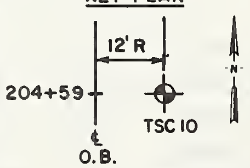
KEY PLAN



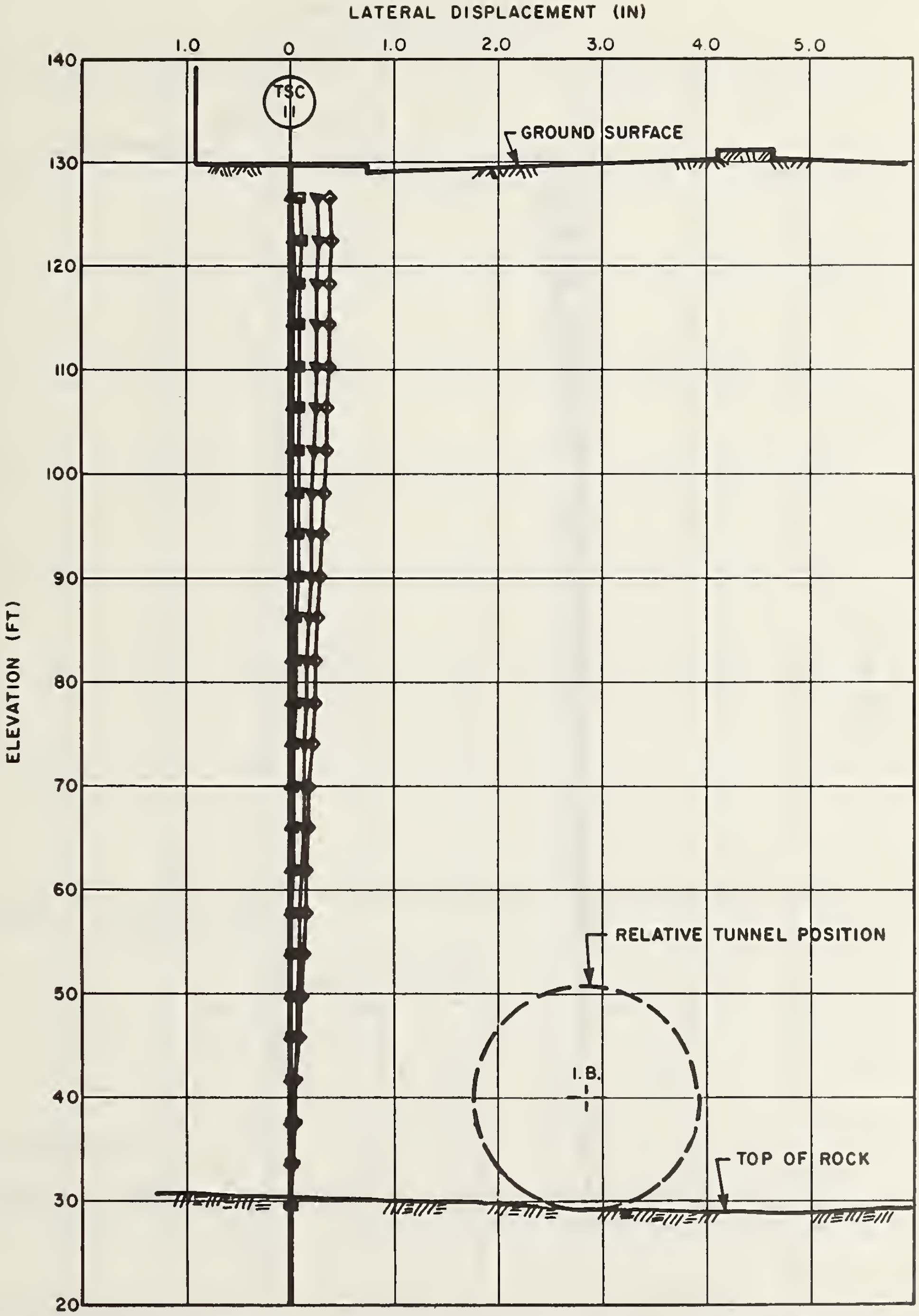
PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 10



SYM	HOLE	DATE	TIME	AZI
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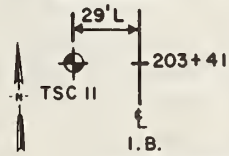
▽	11	11/13/80	13:15	90.
◇	11	12/22/80	11:40	90.

KEY PLAN

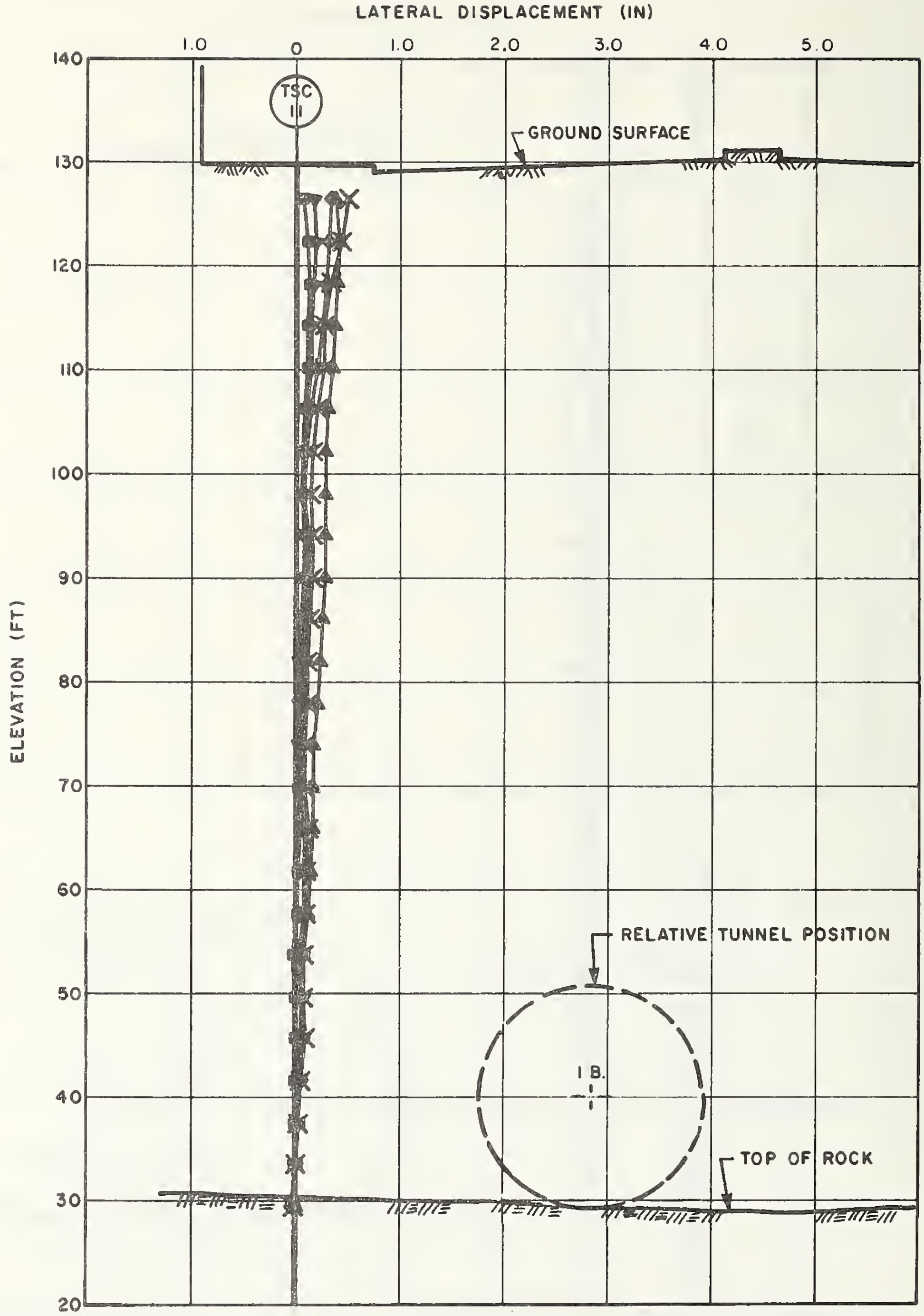


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 11



SYM	HOLE	DATE	TIME	AZI
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□	11	08/20/81	12:30	90.
▽	11	09/03/81	08:40	90.
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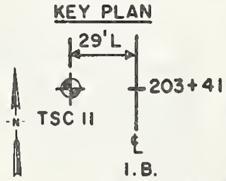
×	11	10/21/81	13:10	90.

KEY PLAN



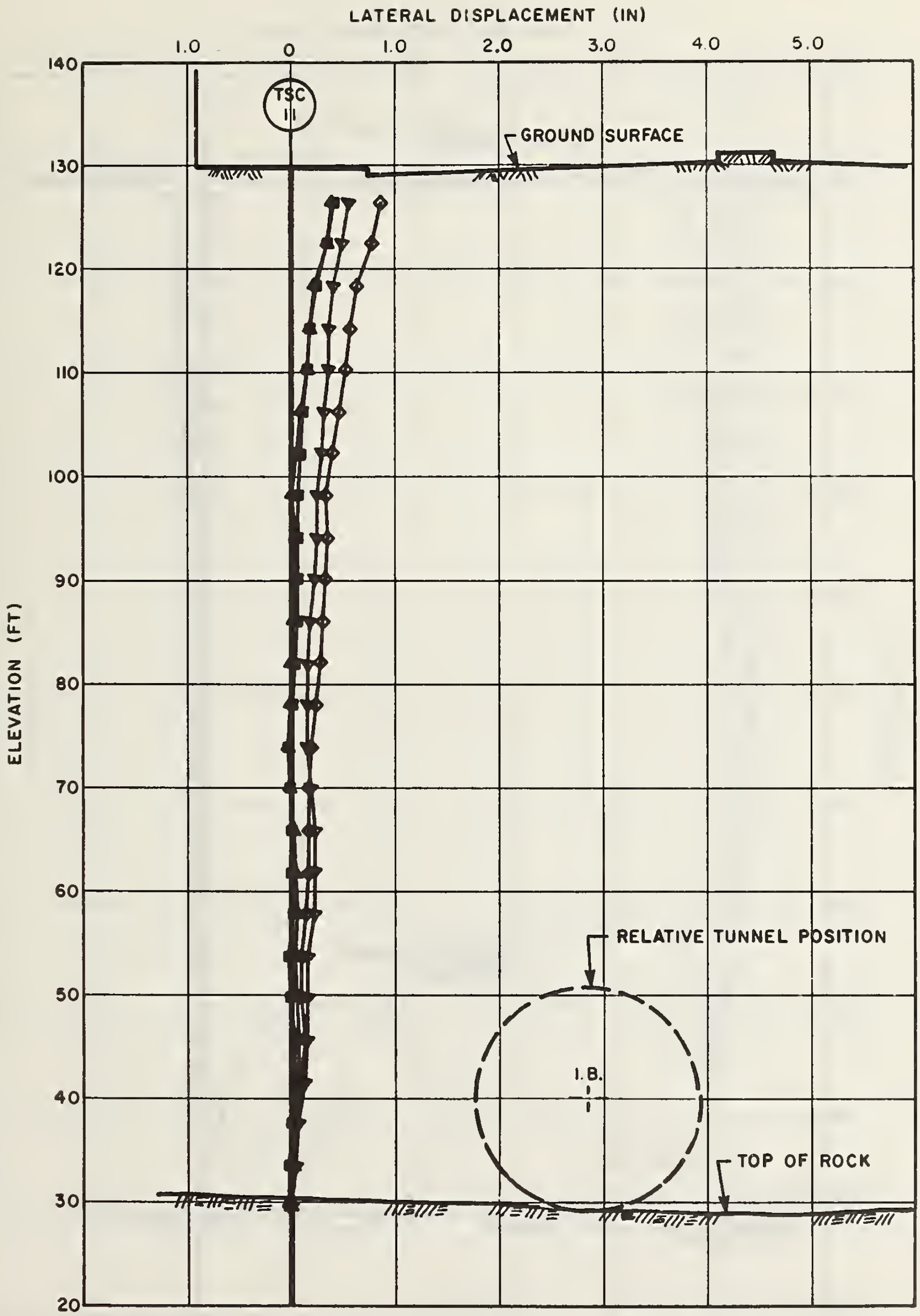
PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 11



SYM	HOLE	DATE	TIME	AZI
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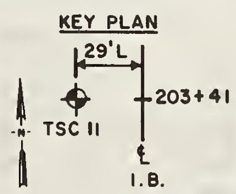
◇	11	12/08/81	11:00	90.

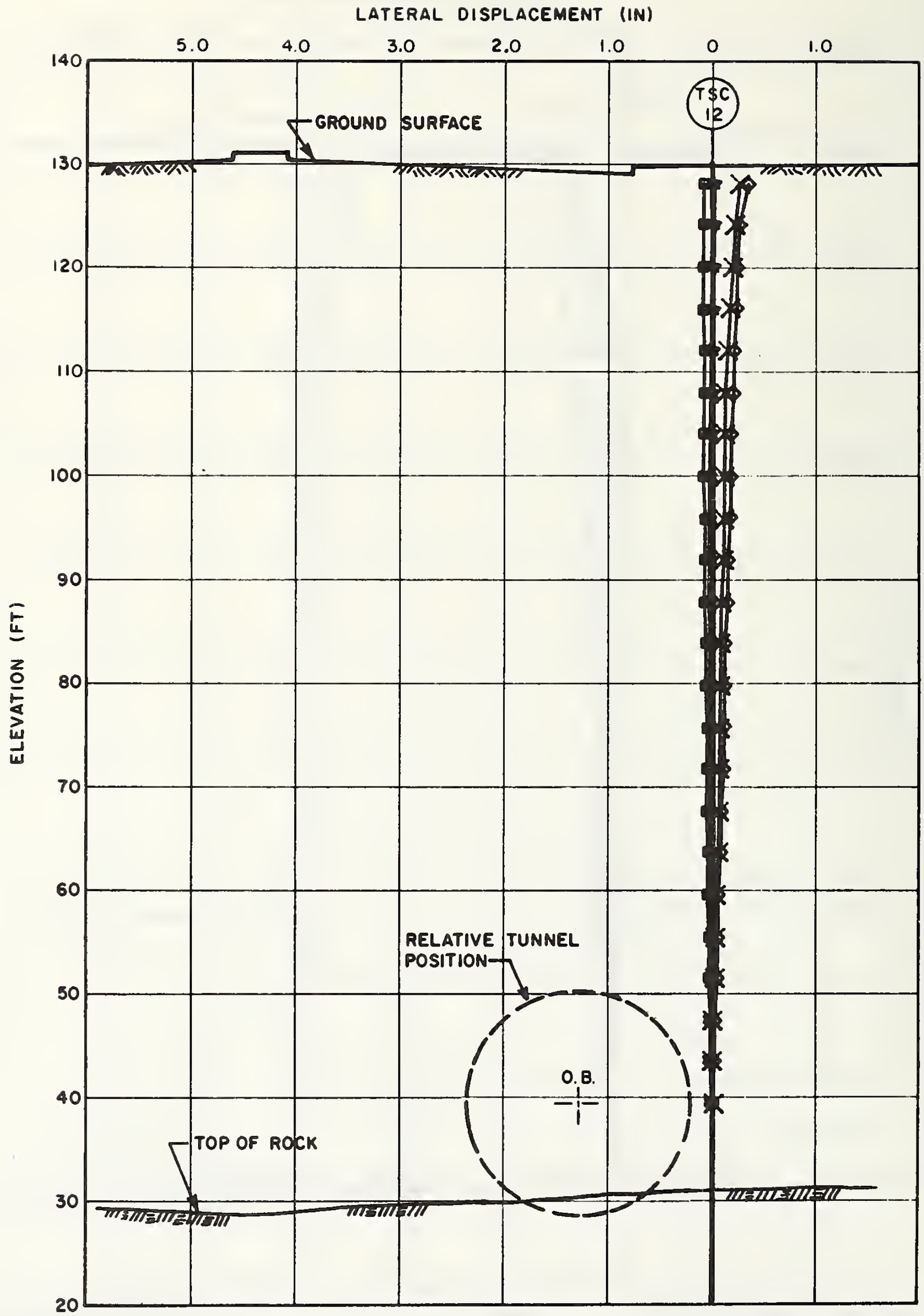


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 11



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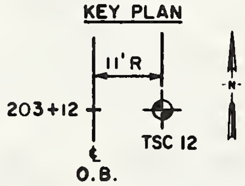
×	12	12/22/80	09:30	270.

KEY PLAN

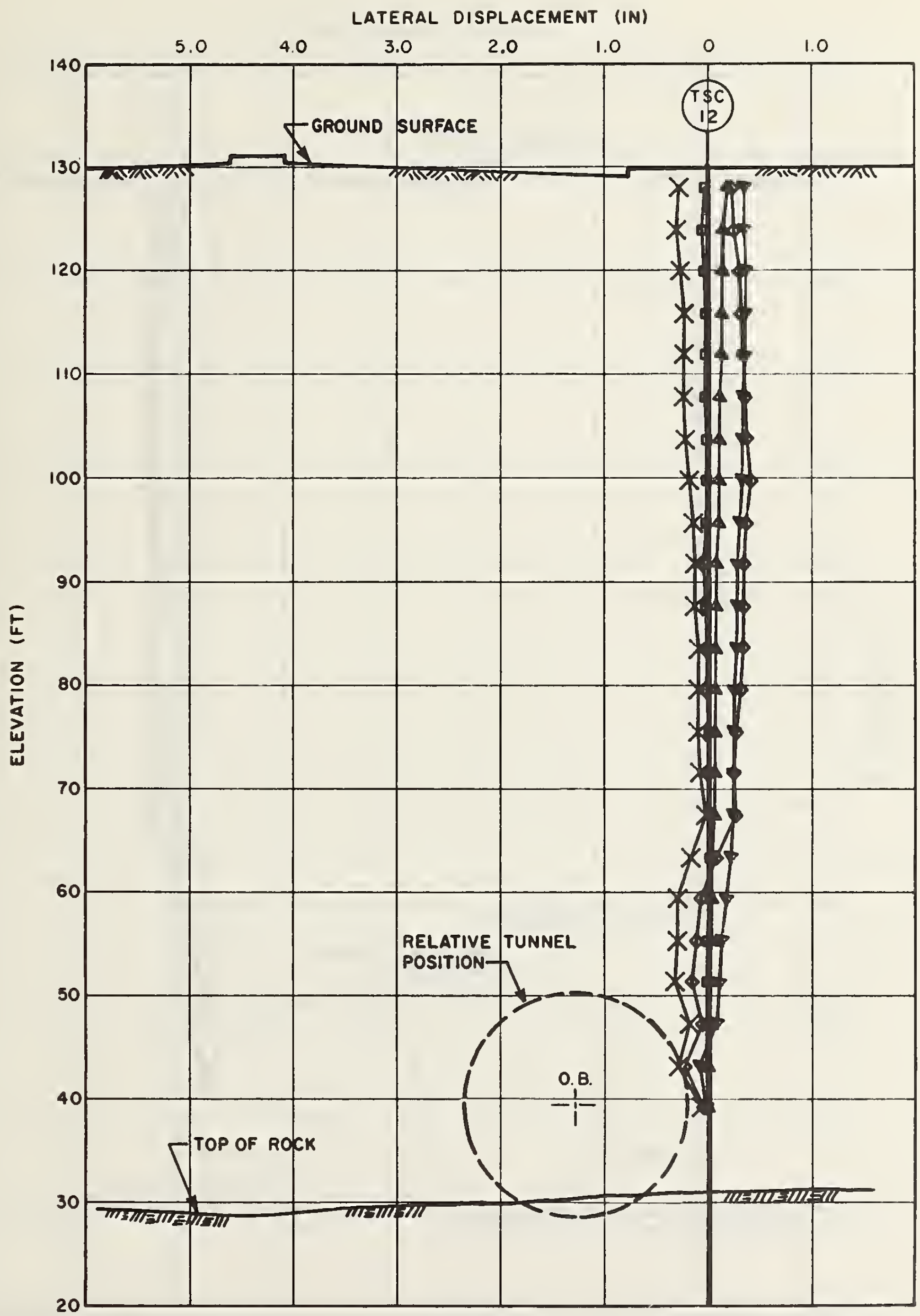


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 12



SYM	HOLE	DATE	TIME	AZI
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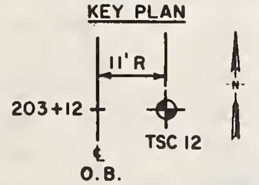
X	12	10/31/81	10:20	270.

KEY PLAN

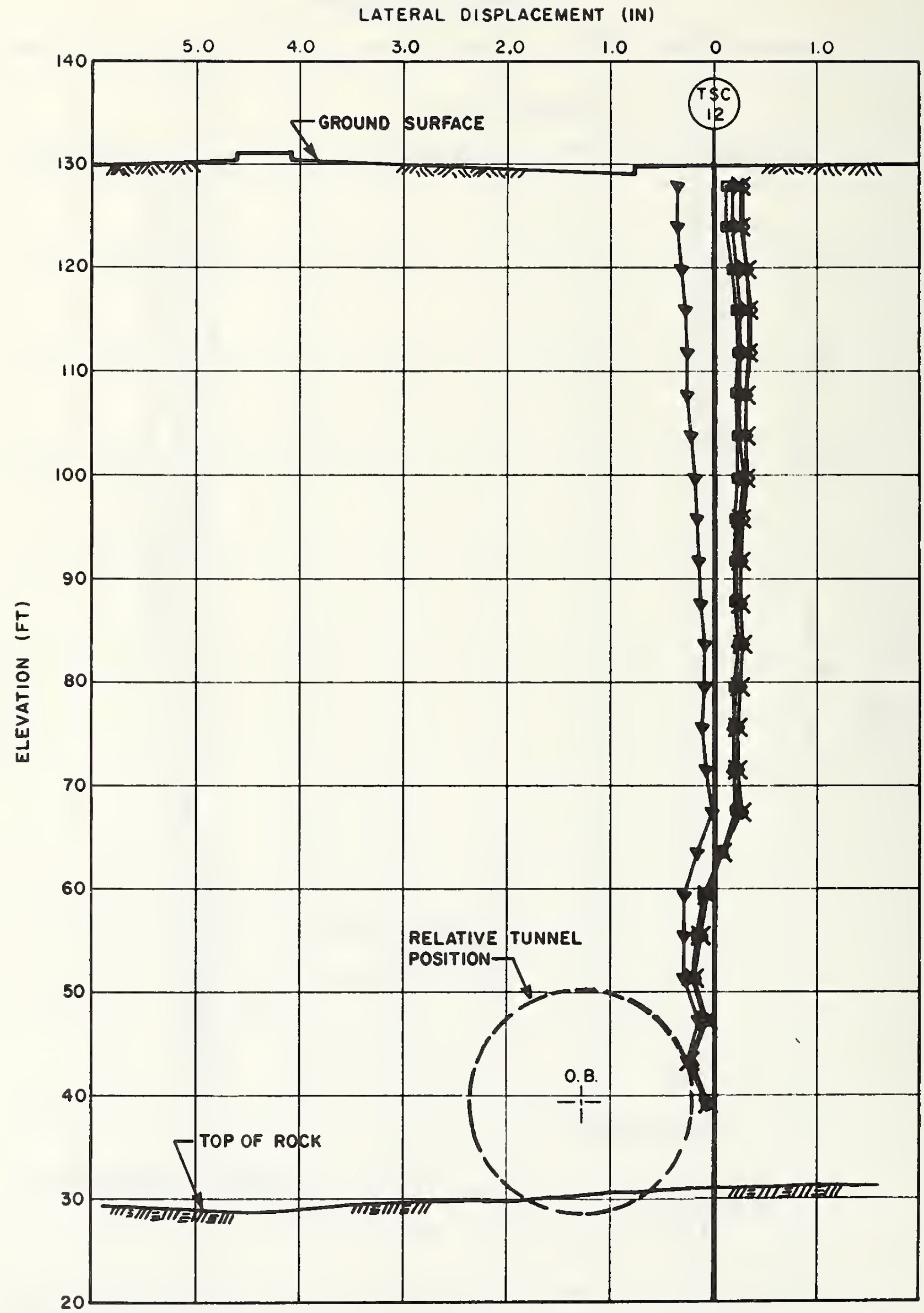


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 12



SYM	HOLE	DATE	TIME	AZI
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□	12	11/12/81	11:10	270.
▽	12	11/22/81	12:00	270.
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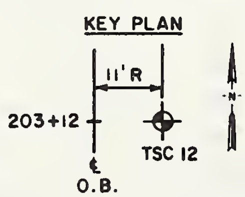
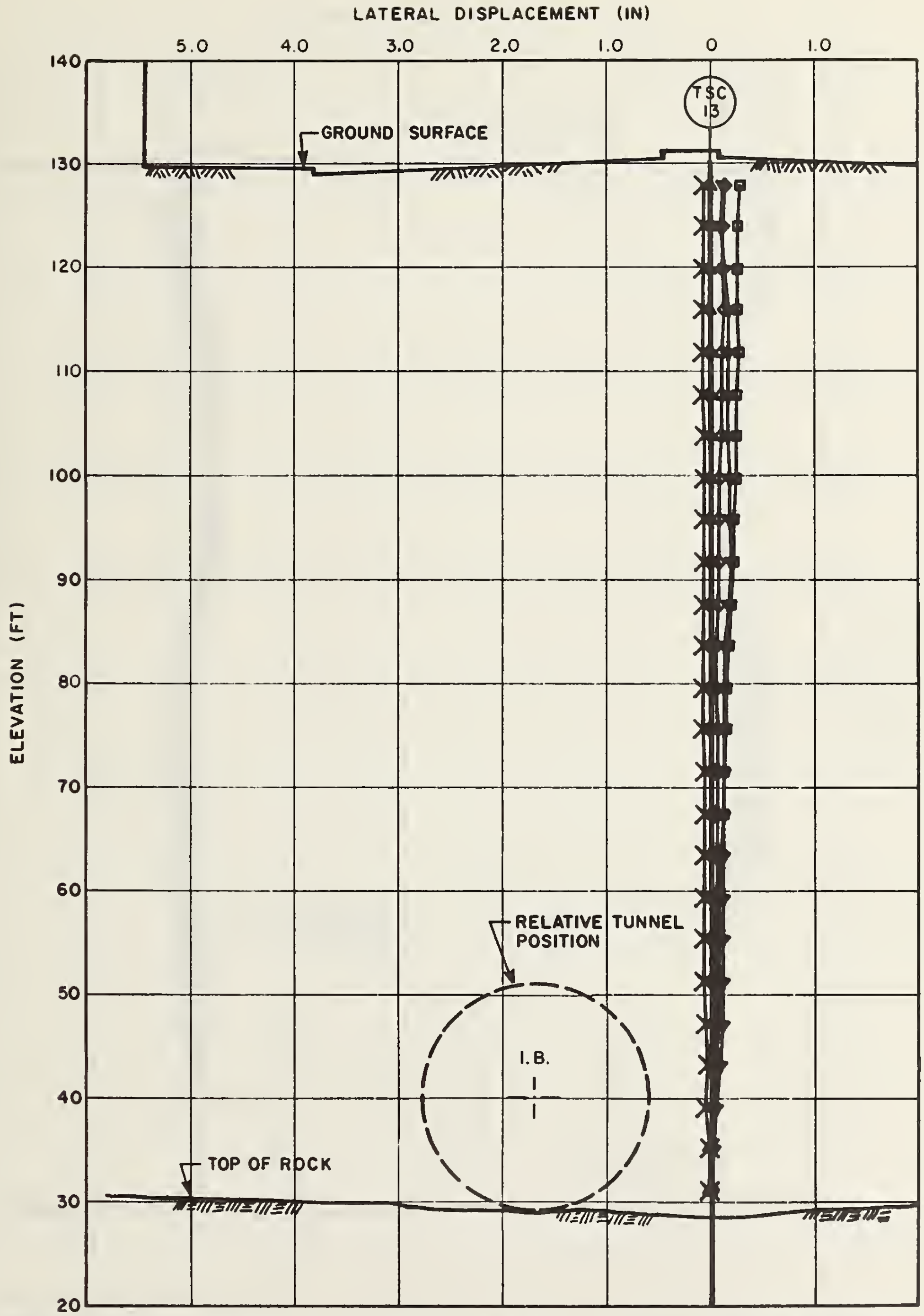


PROJECT RED LINE TEST SECTION CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 12



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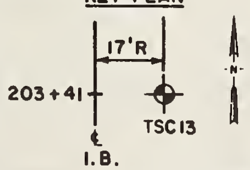
X	13	12/22/80	10:40	270.

KEY PLAN



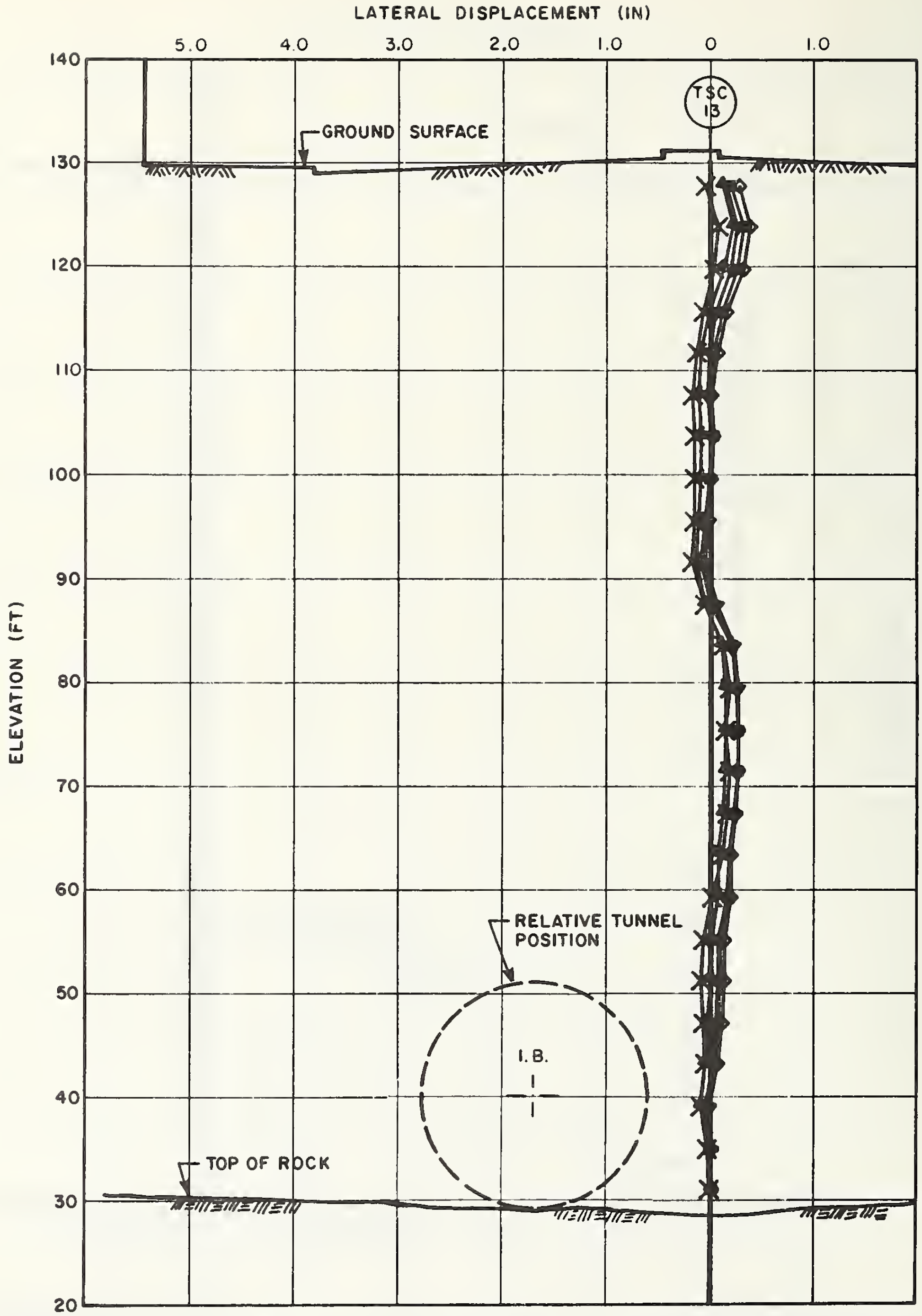
PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 13



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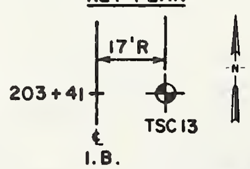
KEY PLAN

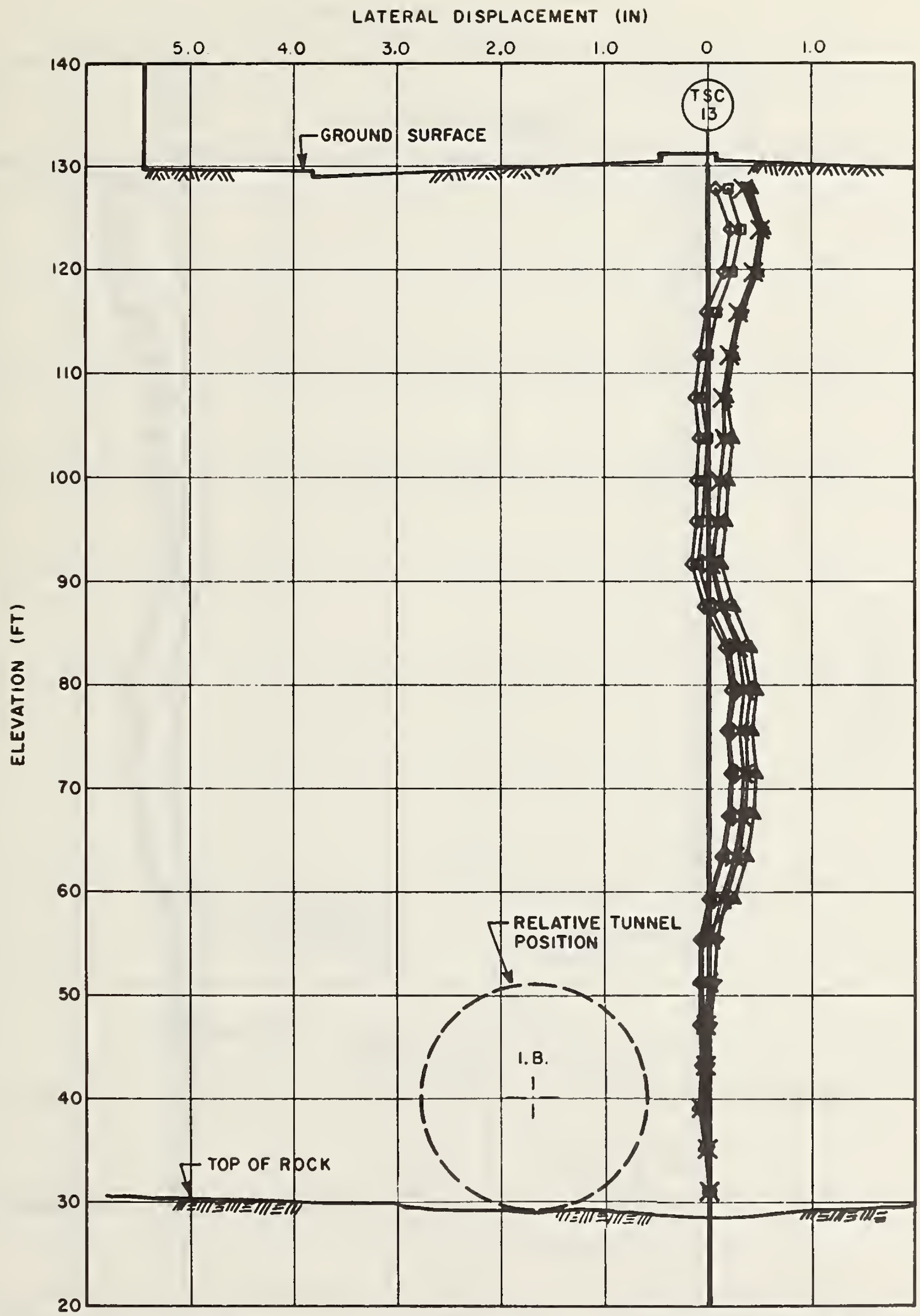


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 13



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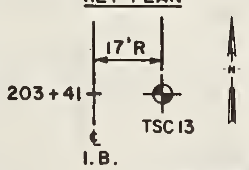
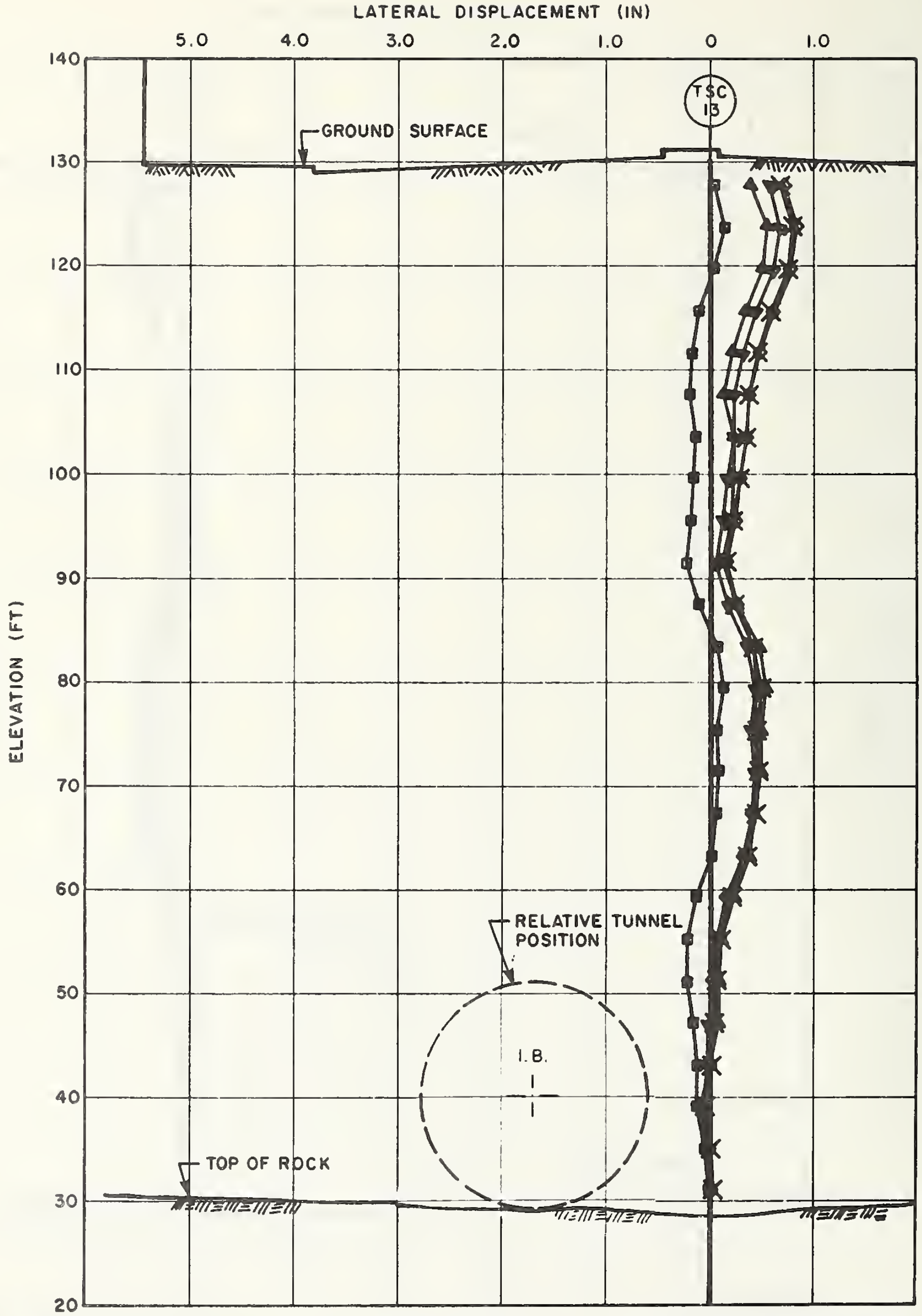


PROJECT RED LINE TEST SECTION CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 13



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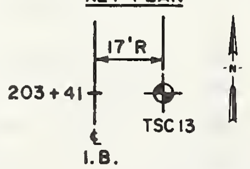
KEY PLAN

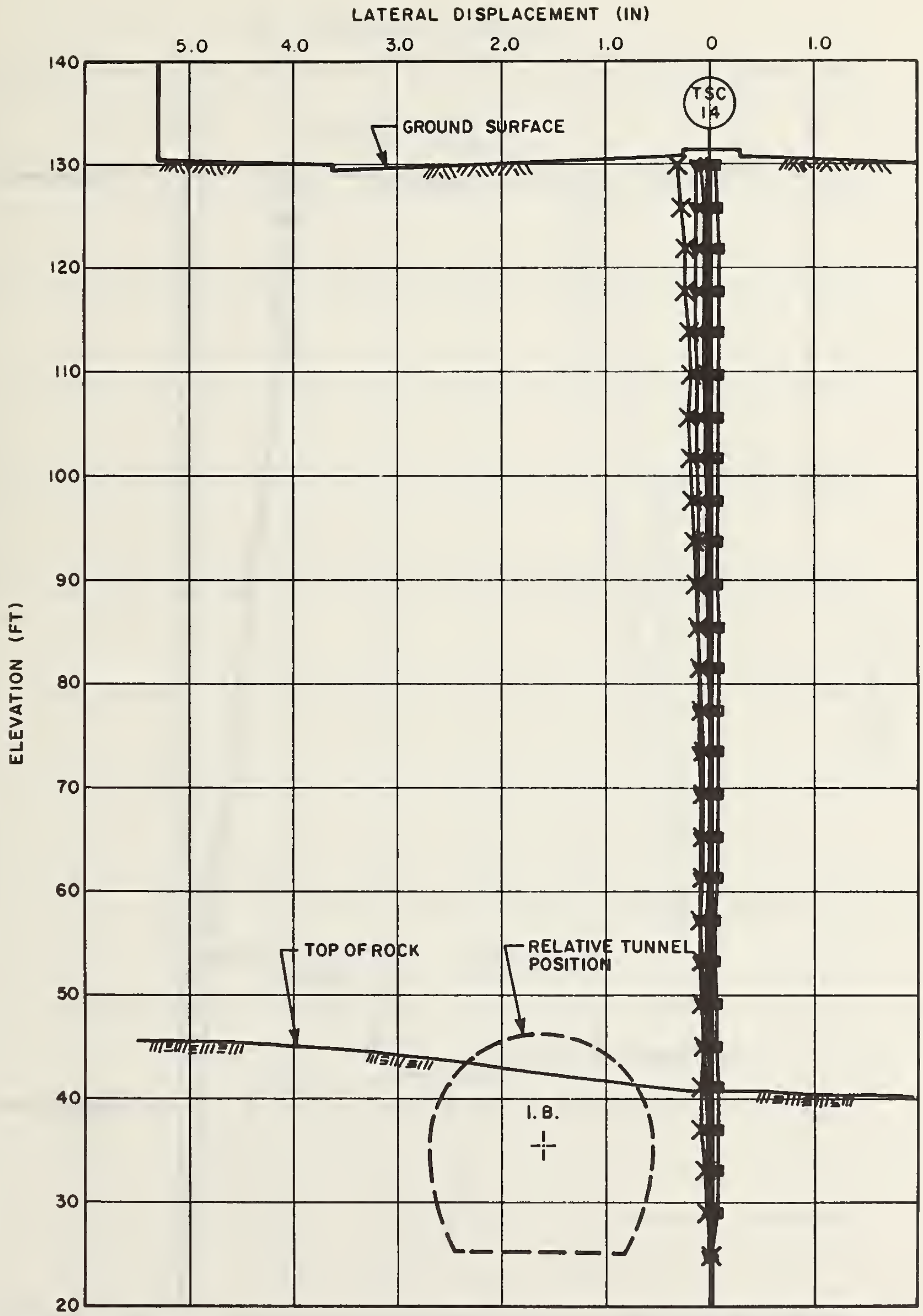


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 13



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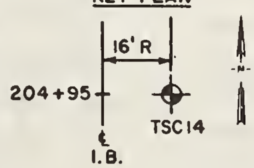
◇	14	11/13/80	09:45	270.
X	14	12/22/80	12:50	270.

KEY PLAN

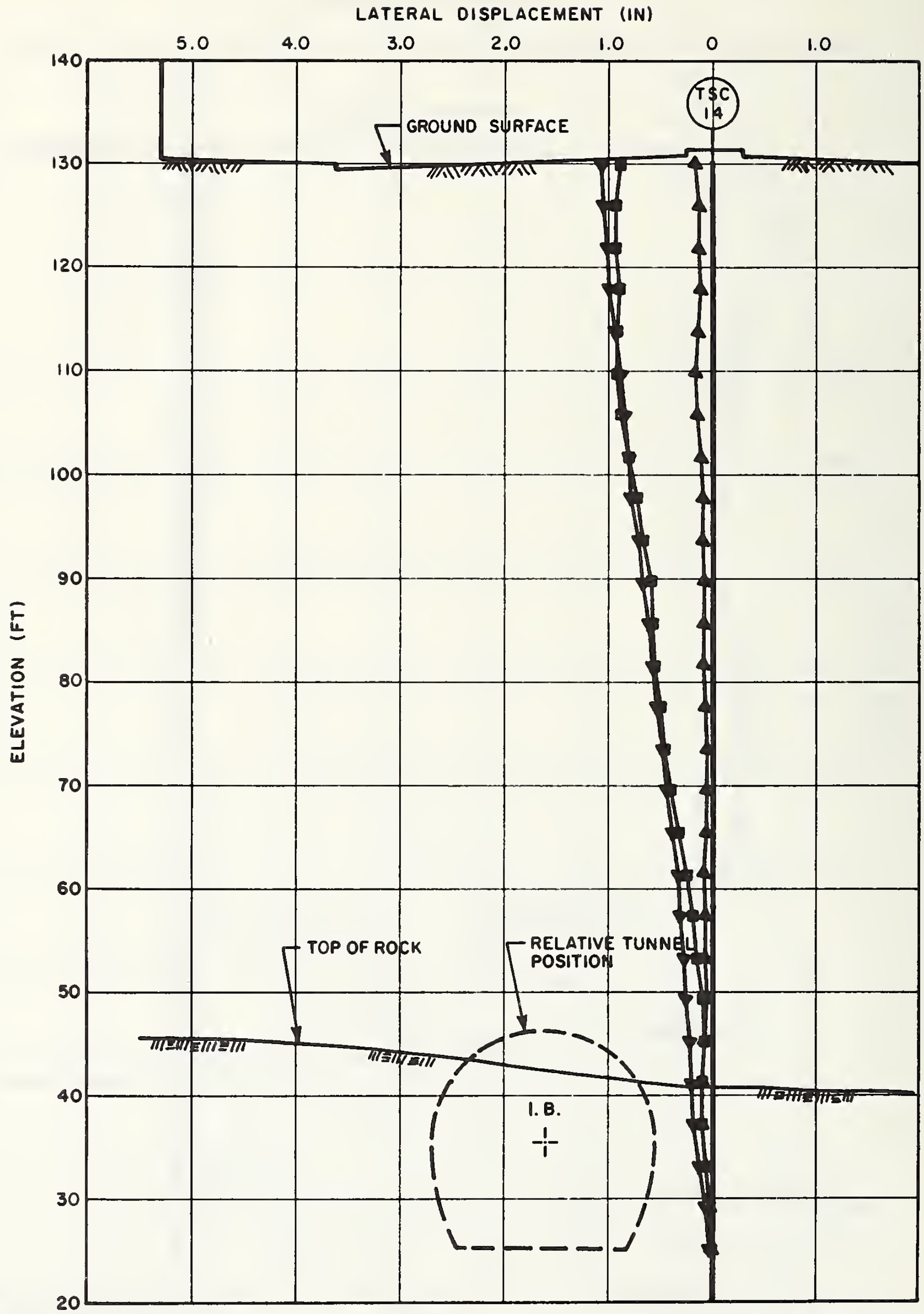


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 14



SYM	HOLE	DATE	TIME	AZI
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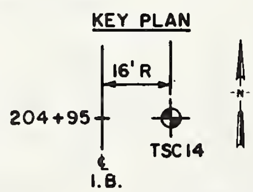


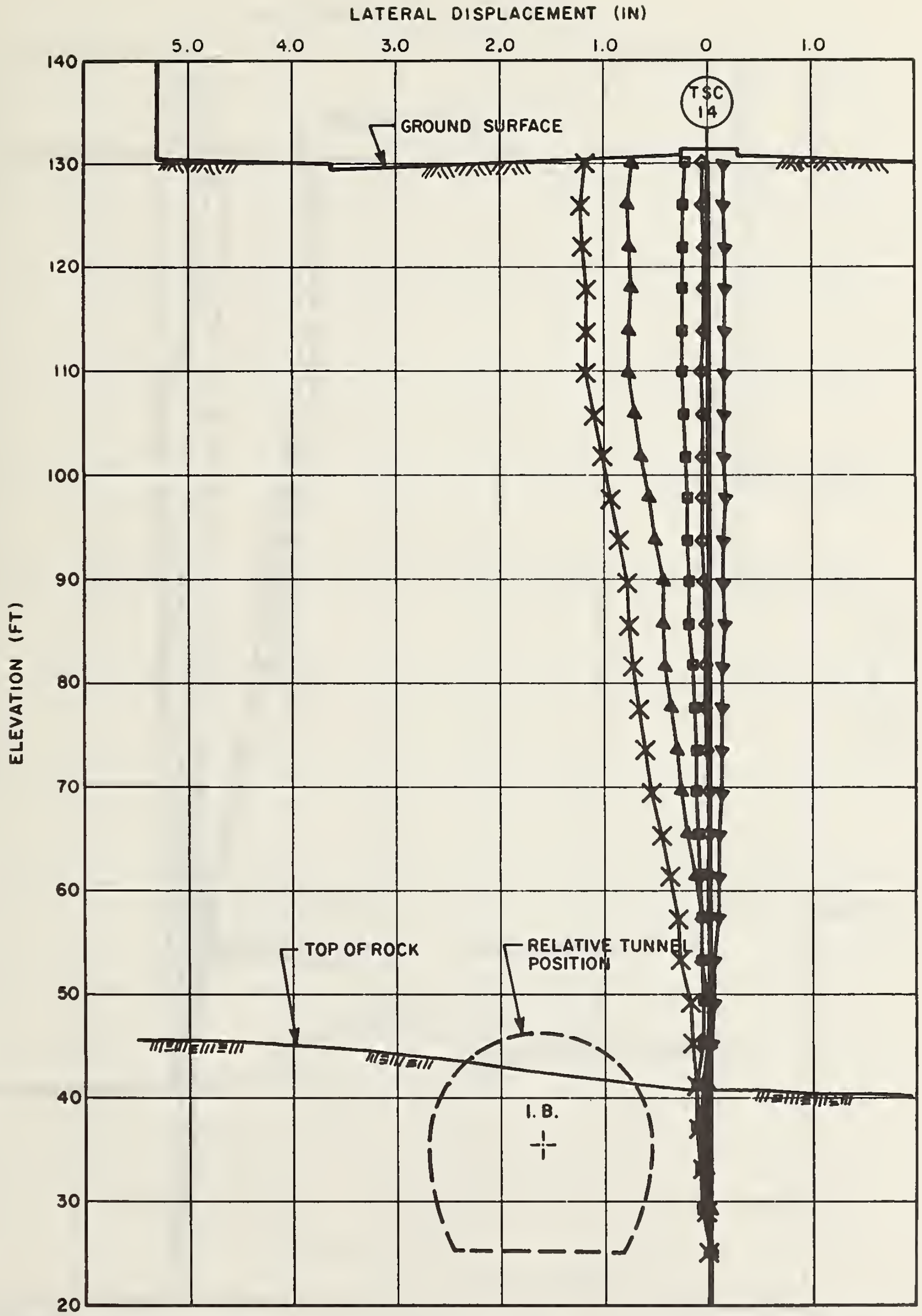
PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 14



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□	14	10/16/81	12:15	270.
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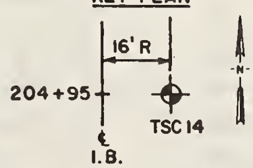
◇	14	10/22/81	11:30	270.
X	14	10/26/81	11:00	270.

KEY PLAN

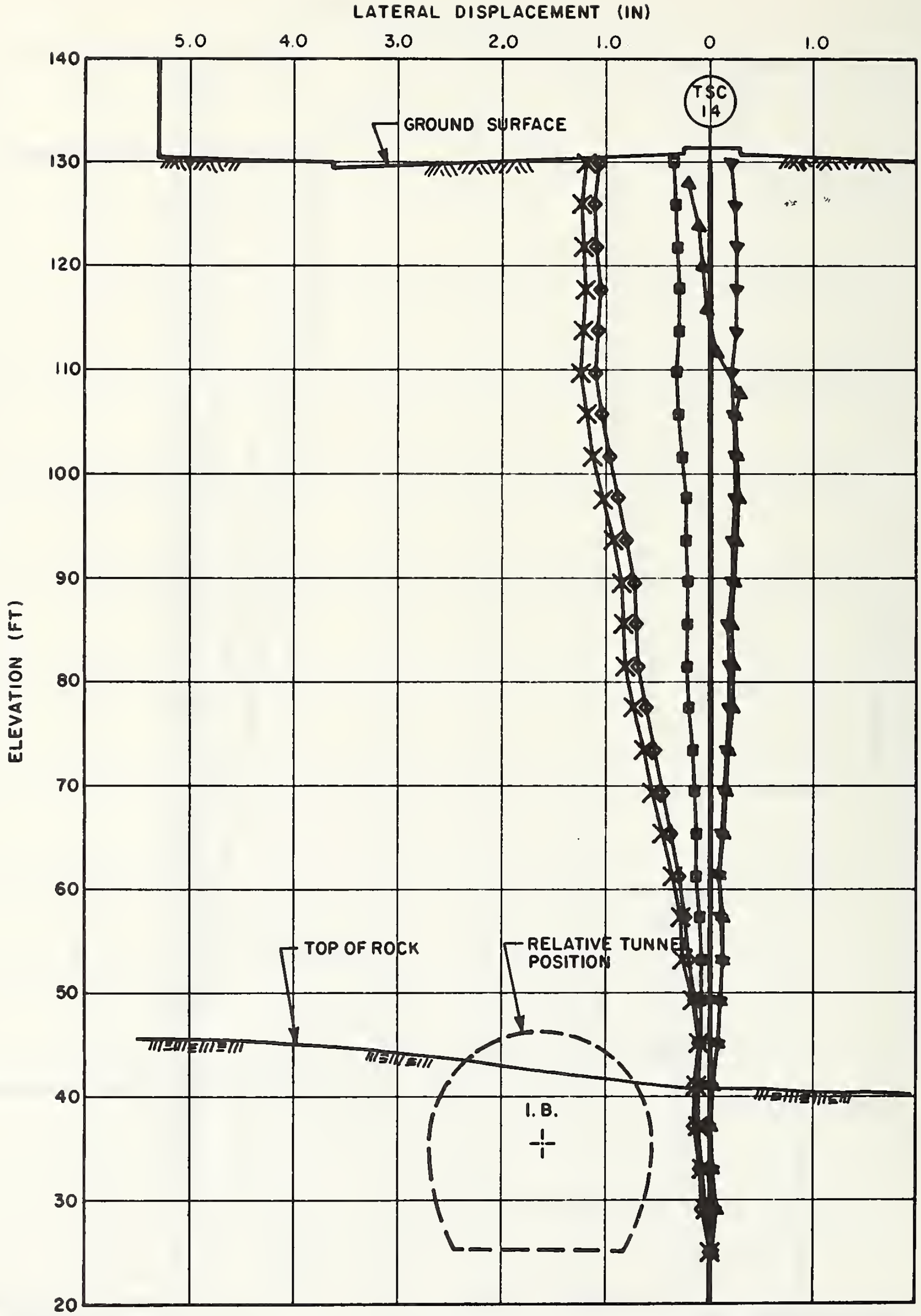


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 14



SYM	HOLE	DATE	TIME	AZI
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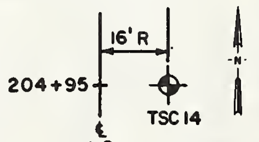
◇	14	12/03/81	11:00	270.
X	14	12/08/81	12:45	270.

KEY PLAN

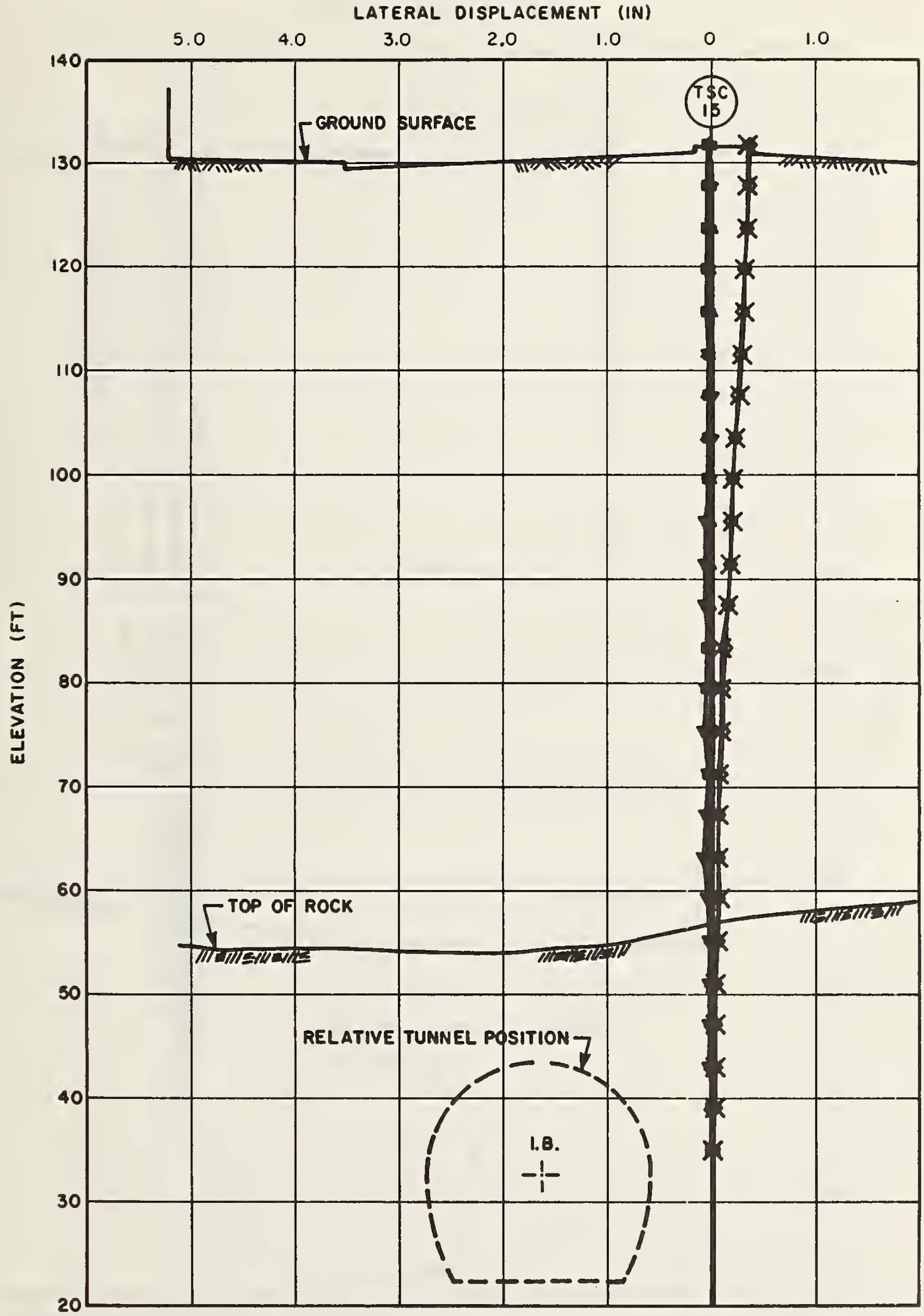


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 14



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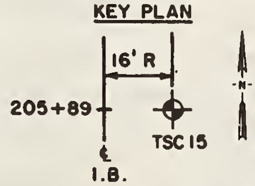
◇	15	11/13/80	10:30	270.
X	15	12/22/80	15:15	270.

KEY PLAN

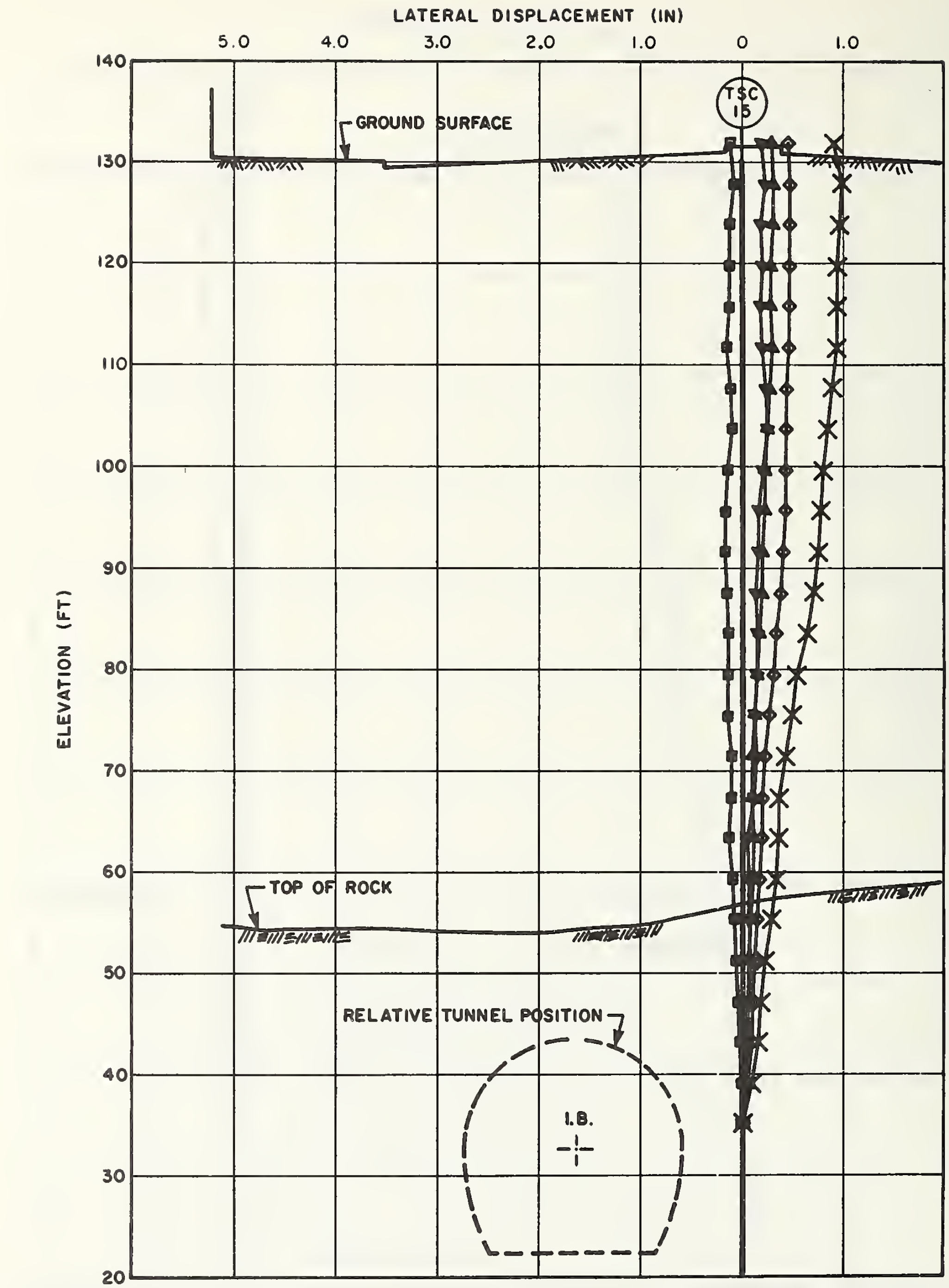


PROJECT RED LINE TEST SECTION
CAMBRIDGE, MASS

CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 15



SYM	HOLE	DATE	TIME	AZI
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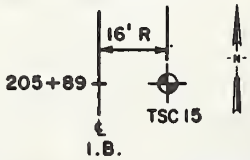
□	15	10/19/81	15:00	270.
▽	15	10/22/81	11:00	270.
◇	15	10/26/81	10:30	270.
X	15	12/08/81	14:00	270.

KEY PLAN



PROJECT RED LINE TEST SECTION

CAMBRIDGE, MASS

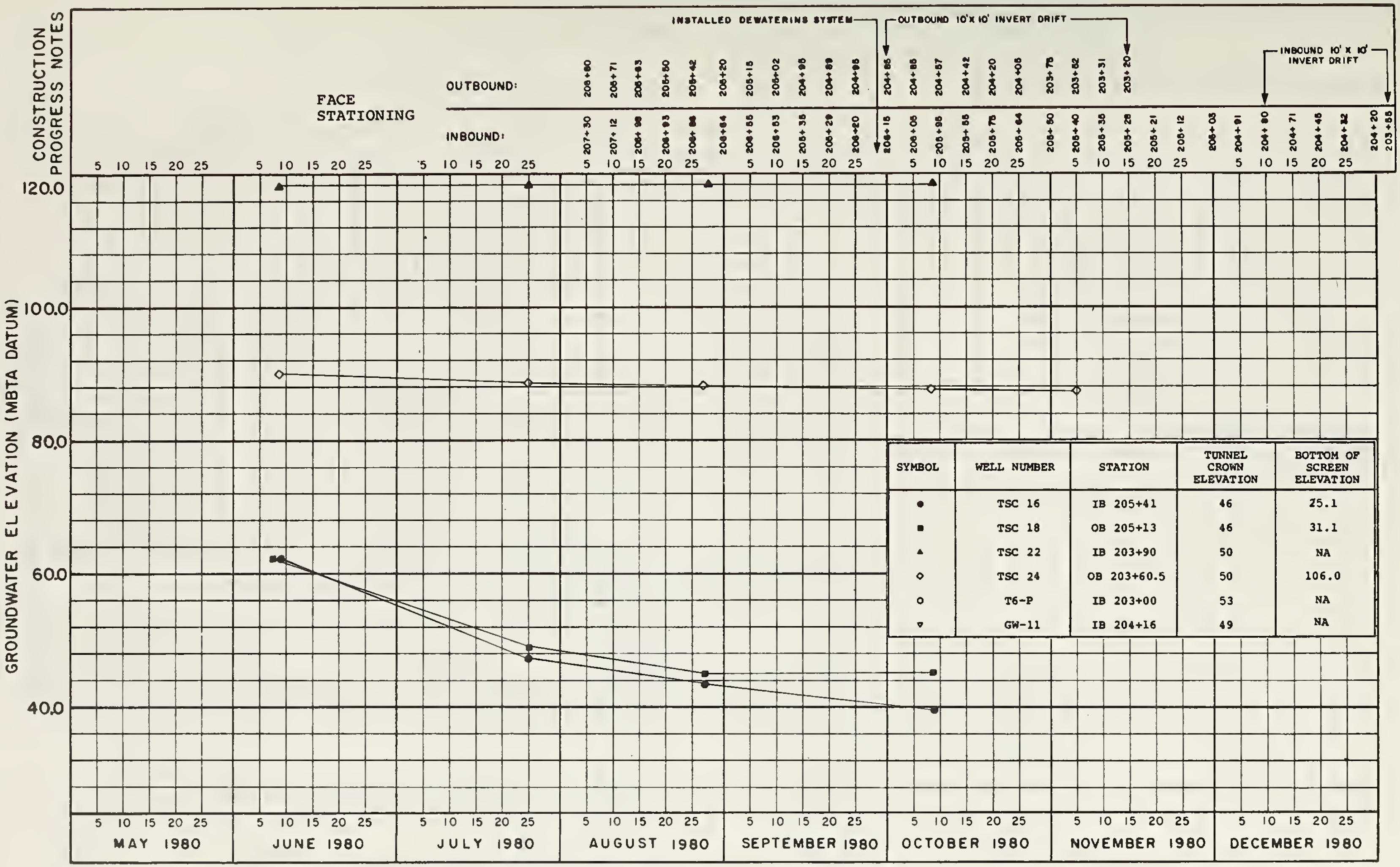
CLIENT DOT/TSC

INCLINOMETER OBSERVATIONS

CASING NO. TSC 15

APPENDIX G

GROUNDWATER OBSERVATIONS



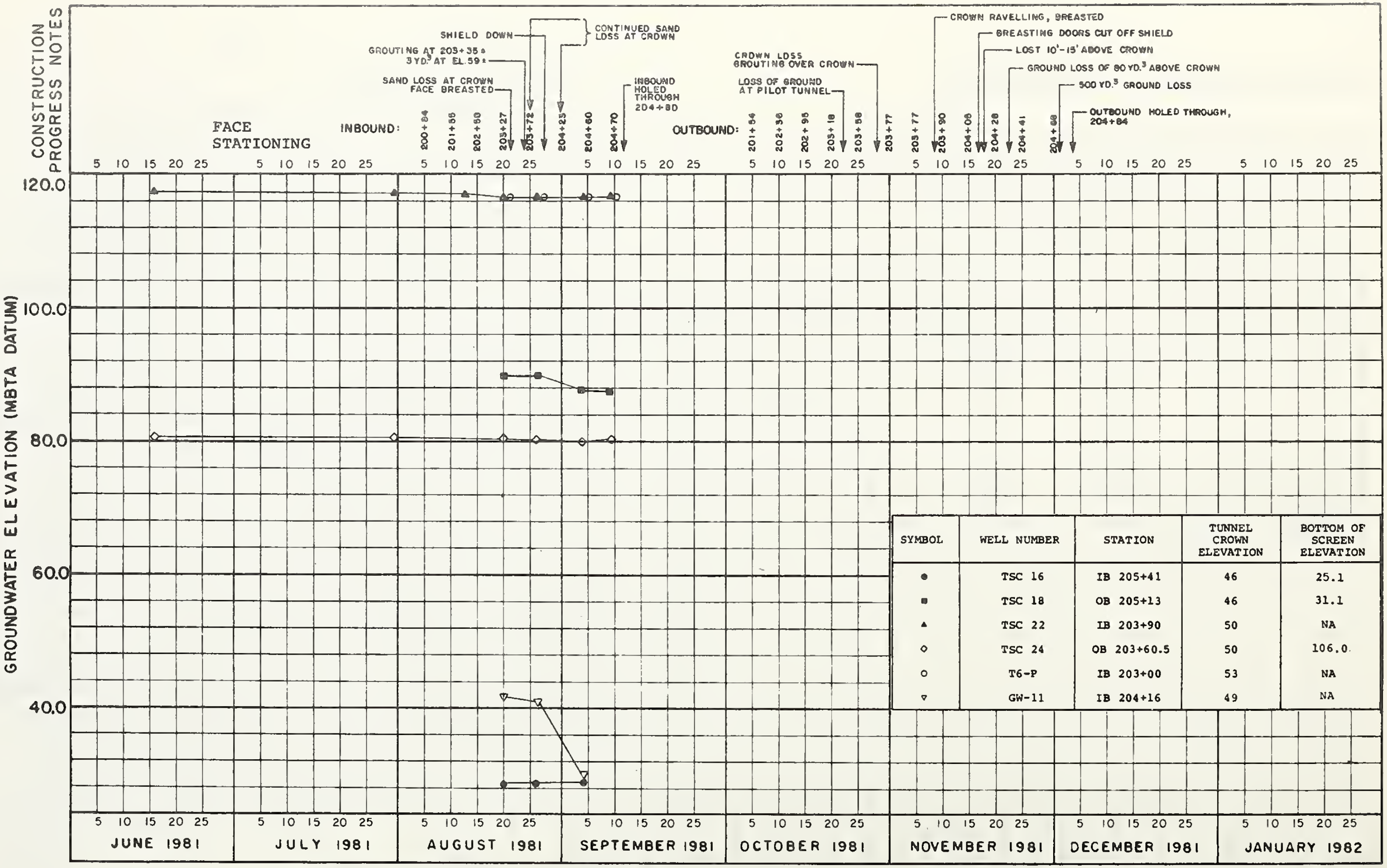
SYMBOL	WELL NUMBER	STATION	TUNNEL CROWN ELEVATION	BOTTOM OF SCREEN ELEVATION
●	TSC 16	IB 205+41	46	25.1
■	TSC 18	OB 205+13	46	31.1
▲	TSC 22	IB 203+90	50	NA
◇	TSC 24	OB 203+60.5	50	106.0
○	T6-P	IB 203+00	53	NA
▽	GW-11	IB 204+16	49	NA

PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

GROUNDWATER OBSERVATIONS



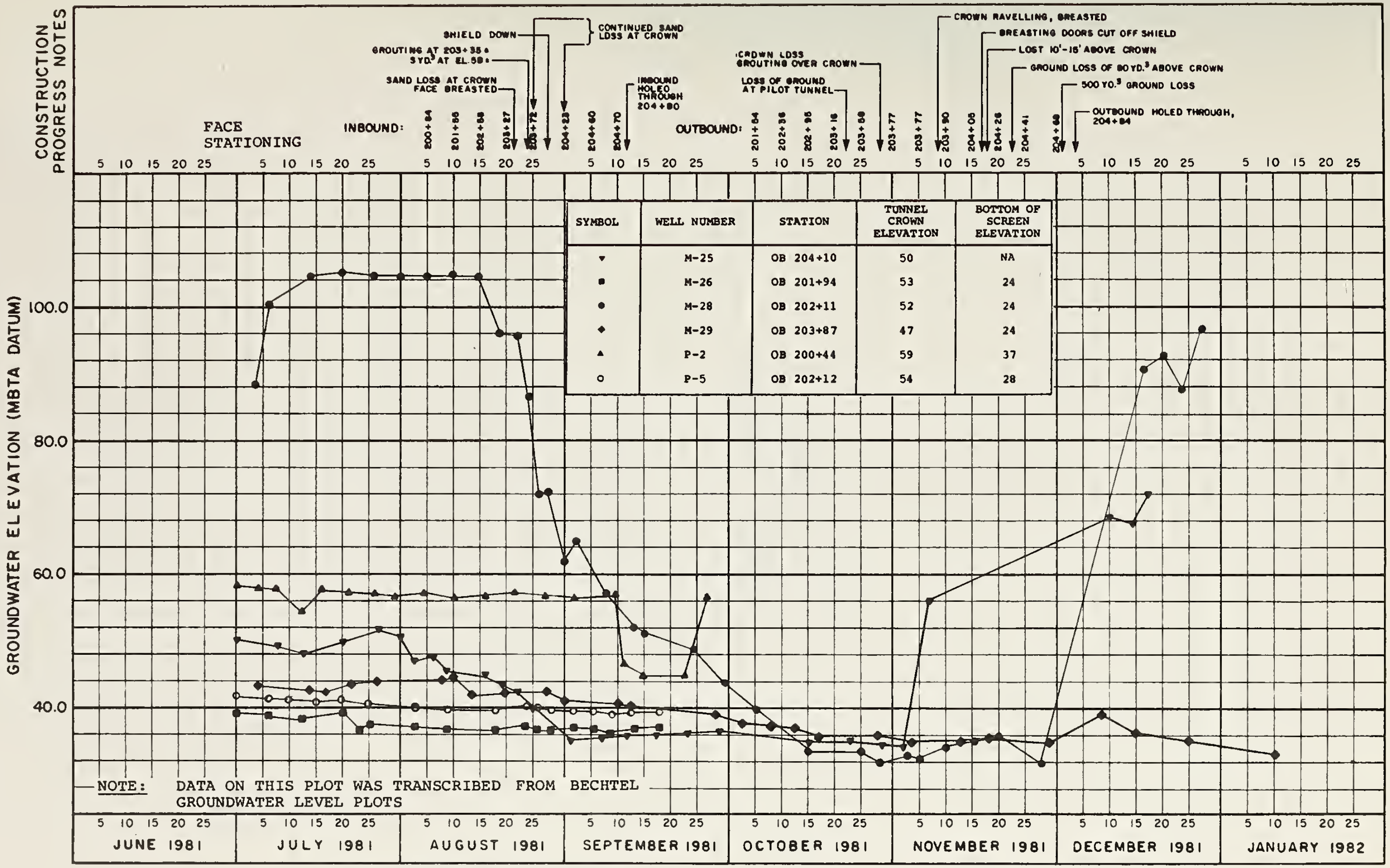
SYMBOL	WELL NUMBER	STATION	TUNNEL CROWN ELEVATION	BOTTOM OF SCREEN ELEVATION
●	TSC 16	IB 205+41	46	25.1
■	TSC 18	OB 205+13	46	31.1
▲	TSC 22	IB 203+90	50	NA
◇	TSC 24	OB 203+60.5	50	106.0
○	T6-P	IB 203+00	53	NA
▽	GW-11	IB 204+16	49	NA

PROJECT RED LINE EXTENSION TEST SECTION

CAMBRIDGE, MASSACHUSETTS

CLIENT DOT/TSC

GROUNDWATER OBSERVATIONS



SYMBOL	WELL NUMBER	STATION	TUNNEL CROWN ELEVATION	BOTTOM OF SCREEN ELEVATION
▼	M-25	OB 204+10	50	NA
■	M-26	OB 201+94	53	24
●	M-28	OB 202+11	52	24
◆	M-29	OB 203+87	47	24
▲	P-2	OB 200+44	59	37
○	P-5	OB 202+12	54	28

PROJECT RED LINE EXTENSION TEST SECTION
CAMBRIDGE, MASSACHUSETTS
CLIENT DOT/TSC

GROUNDWATER OBSERVATIONS

APPENDIX H

GEOLOGIC MAPPING

EXPLANATION OF MAPPING SYMBOLS

ROCK TYPE SYMBOLS WITH ◯ INDICATES OBSERVATION MADE AT TUNNEL HEADING. THOSE WITHOUT ARE FROM EXPLORATORY BORING LOGS.

A_D = DECOMPOSED ARGILLITE, AT OR NEAR TOP OF ROCK

$\boxed{A}$ = ARGILLITE

A_A = ALTERED ARGILLITE

SS_A = ARGILLACEOUS SANDSTONE

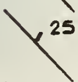
$\boxed{D}$ = DIABASE

D_D = DECOMPOSED DIABASE

 = SEEPAGE OBSERVED AT TUNNEL HEADING

$\bar{R}$ = TOP OF ROCK ELEVATIONS ARE TAKEN FROM EXPLORATORY BORINGS MADE DURING PHASE I

STRIKE & DIP OF JOINTS



= STRIKE & DIP OF BEDDING

= STRIKE & DIP OF IGNEOUS CONTACT

= STRIKE & DIP OF SHEARED ZONE

= STRIKE & DIP OF FAULT PLANE

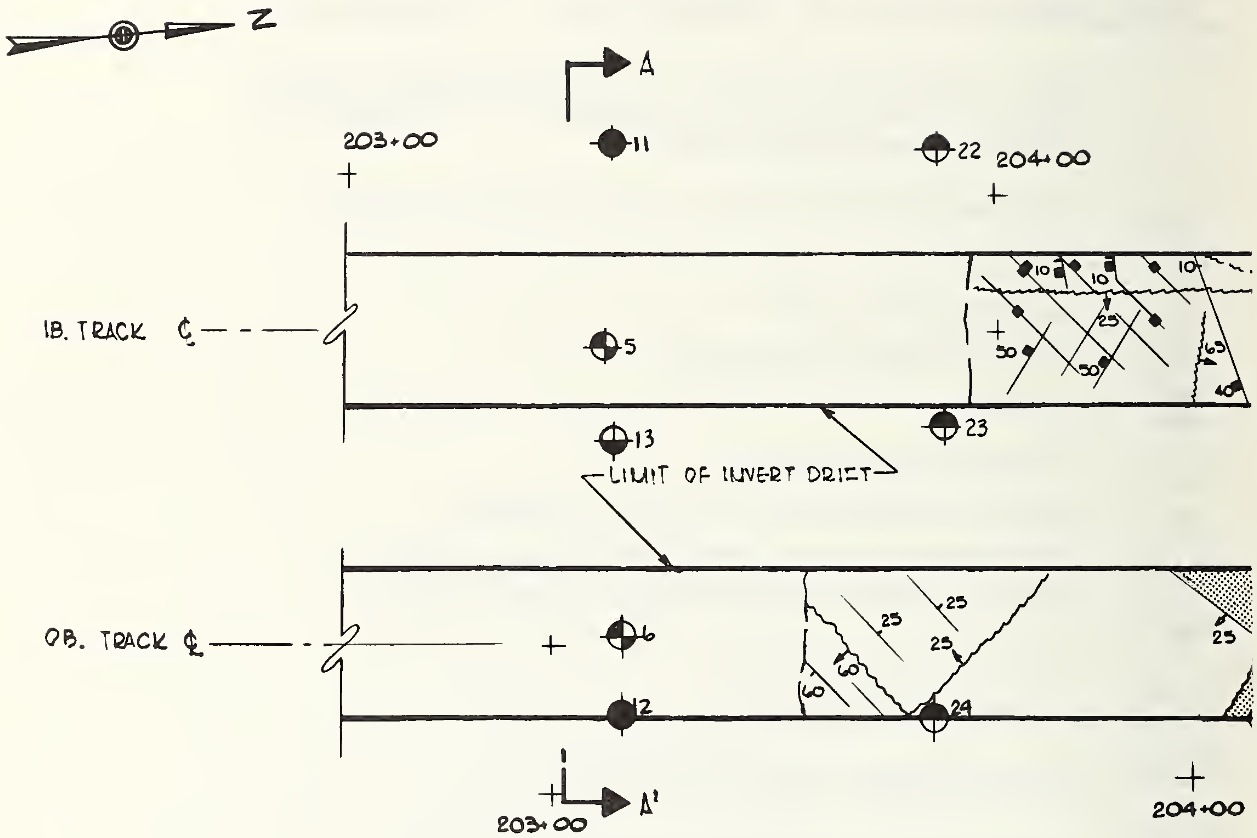
TEST BORING SYMBOLS

 = 6–INCH DIAMETER TEST BORINGS WITH DETAILED AND ADVANCED SAMPLING METHODS

 = 6–INCH DIAMETER TEST BORINGS WITH STANDARD SAMPLING METHODS

 = 4–INCH DIAMETER UNSAMPLED TEST BORINGS

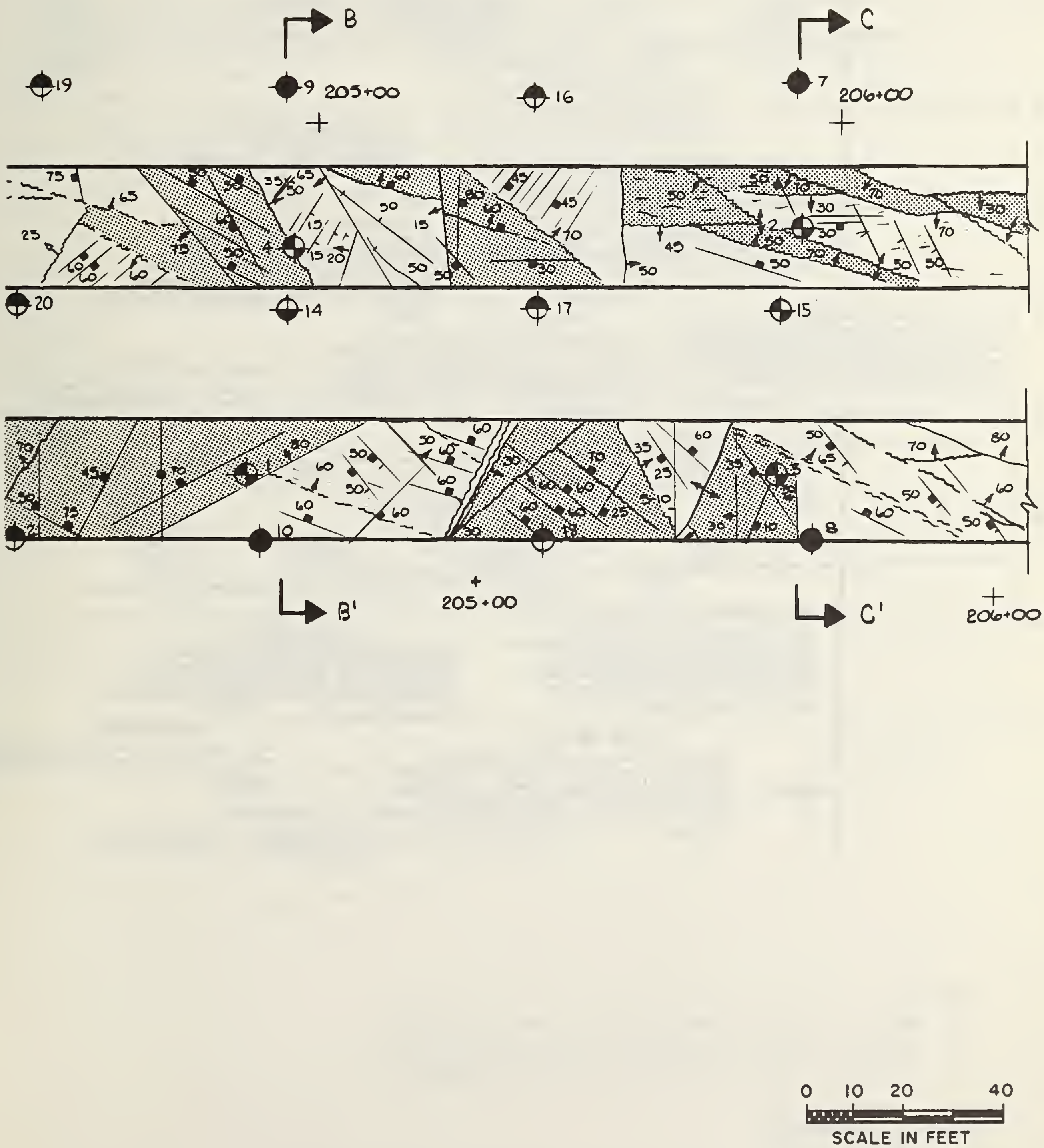
 = 5–INCH DIAMETER SHALLOW UNSAMPLED TEST BORINGS

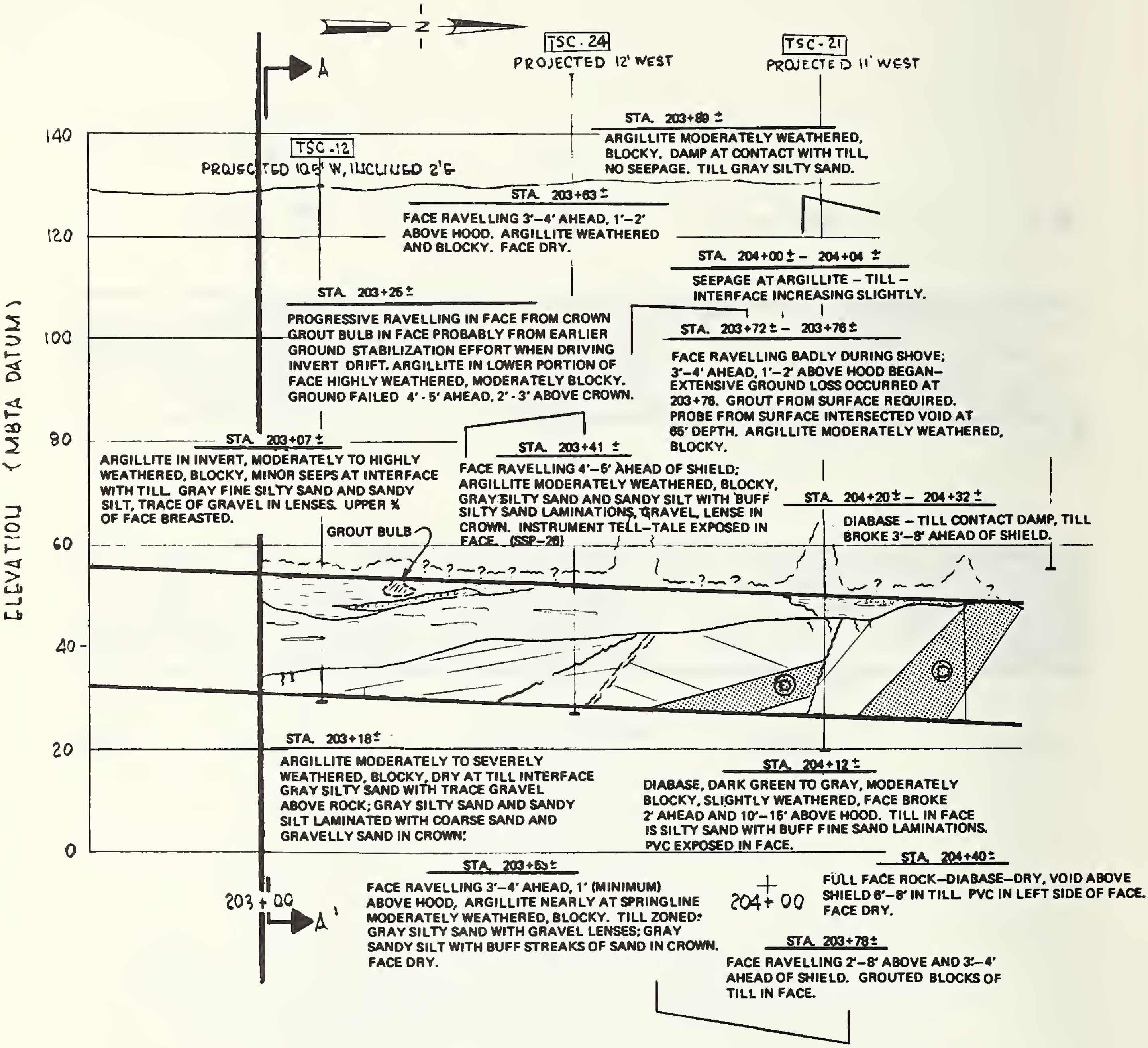


NOTES:

1. Refer to H-3 for explanation of mapping symbols.
2. Refer to H-6 through H-14 for profiles, construction summaries, and cross sections.

PLAN

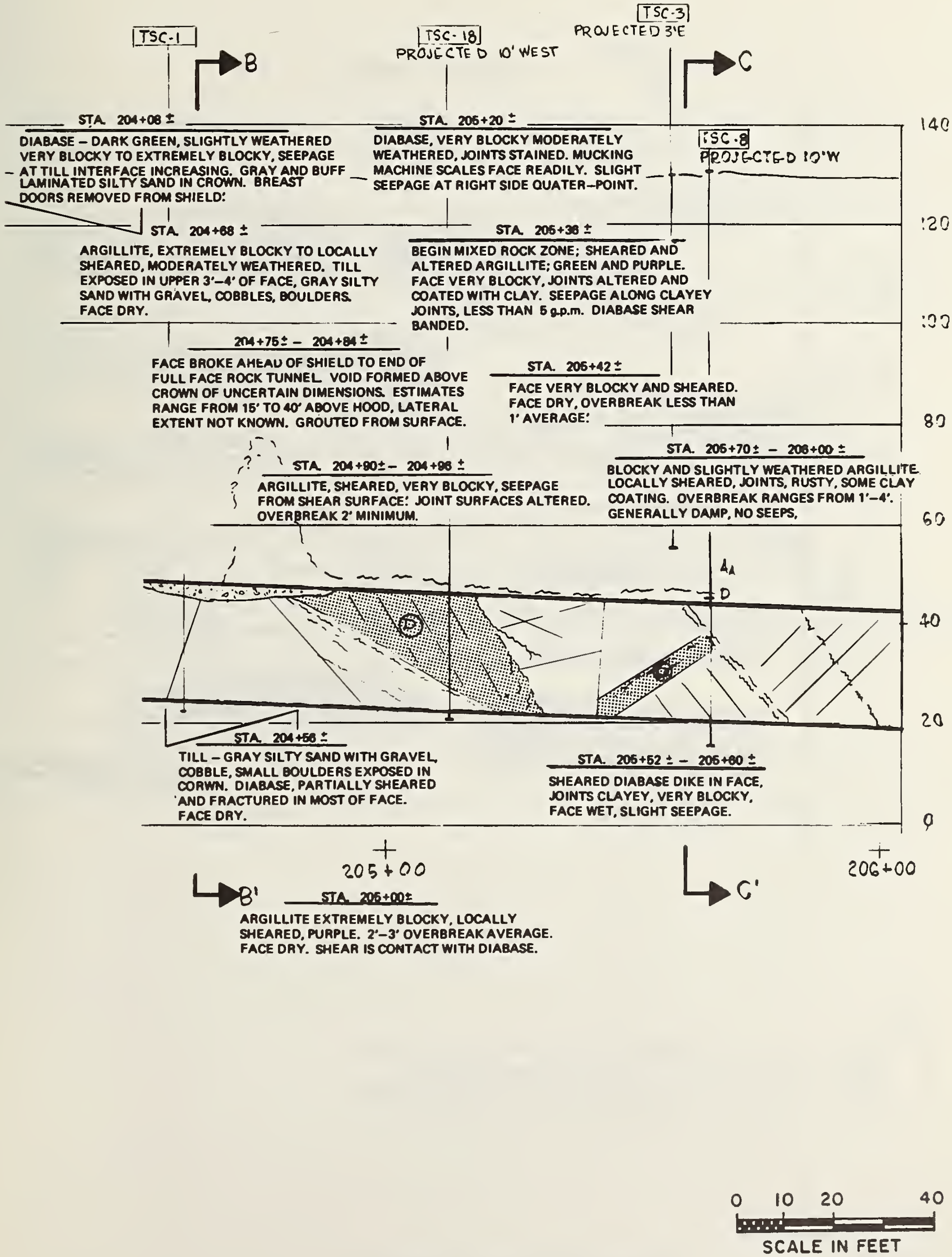

B
C
205+00
206+00
B'
C'
205+00
206+00
0 10 20 40
SCALE IN FEET

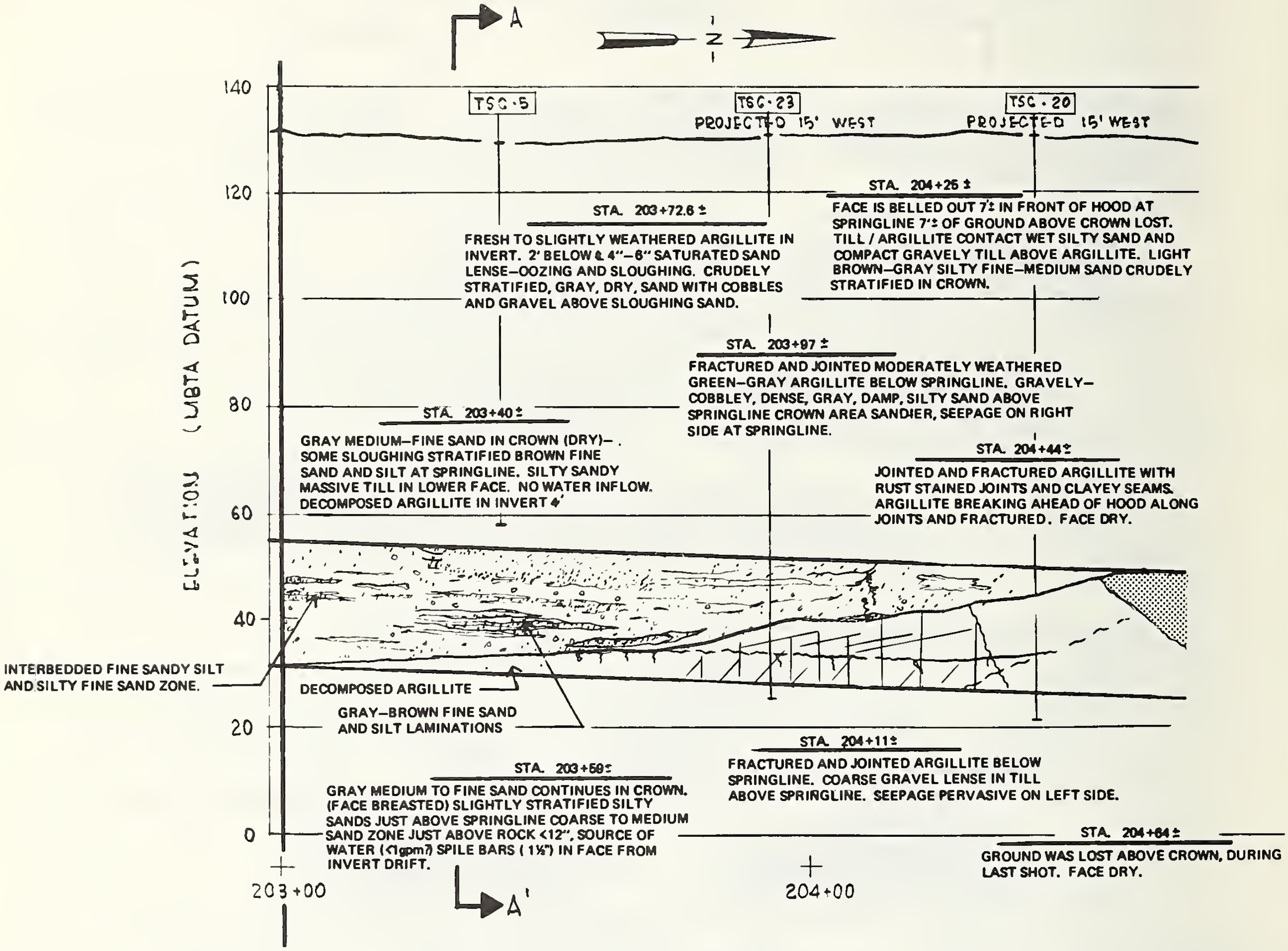


NOTES:

1. Refer to H-3 for explanation of geologic symbols.
2. Refer to H-12, H-13, H-14 for cross-sections.
3. Refer to H-4, H-5 for plan view of geologic maps.
4. Refer to H-10, H-11 for construction summaries.

OUTBOUND TUNNEL PROFILE

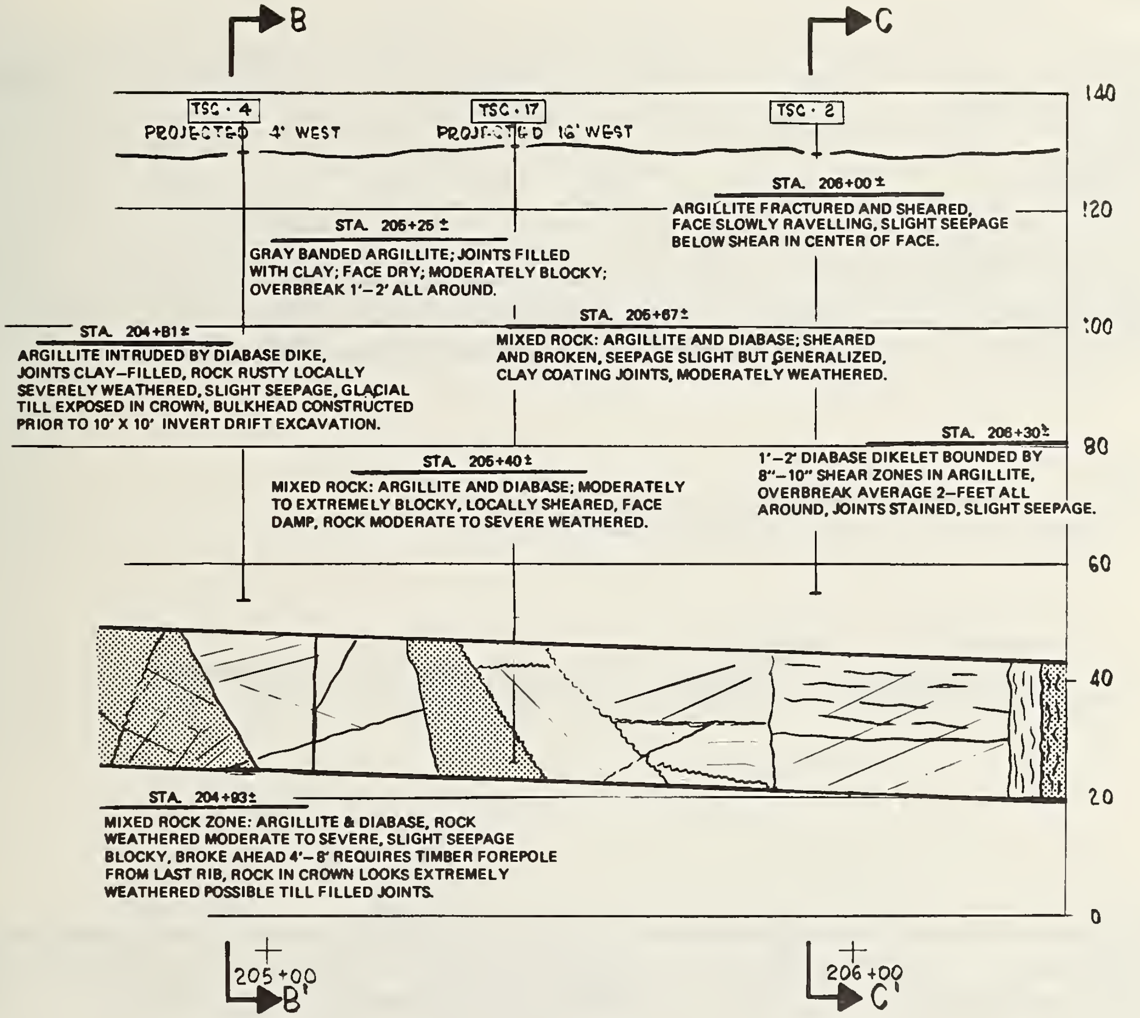

TSC-1
B
TSC-18
PROJECTED 10' WEST
TSC-3
PROJECTED 3'E
C
STA. 204+08 ±
DIABASE – DARK GREEN, SLIGHTLY WEATHERED VERY BLOCKY TO EXTREMELY BLOCKY, SEEPAGE AT TILL INTERFACE INCREASING. GRAY AND BUFF LAMINATED SILTY SAND IN CROWN. BREAST DOORS REMOVED FROM SHIELD.
STA. 205+20 ±
DIABASE, VERY BLOCKY MODERATELY WEATHERED, JOINTS STAINED. MUCKING MACHINE SCALES FACE READILY. SLIGHT SEEPAGE AT RIGHT SIDE QUATER–POINT.
TSC-8
PROJECTED 10'W
140
STA. 204+68 ±
ARGILLITE, EXTREMELY BLOCKY TO LOCALLY SHEARED, MODERATELY WEATHERED. TILL EXPOSED IN UPPER 3'–4' OF FACE, GRAY SILTY SAND WITH GRAVEL, COBBLES, BOULDERS. FACE DRY.
STA. 205+36 ±
BEGIN MIXED ROCK ZONE; SHEARED AND ALTERED ARGILLITE; GREEN AND PURPLE. FACE VERY BLOCKY, JOINTS ALTERED AND COATED WITH CLAY. SEEPAGE ALONG CLAYEY JOINTS, LESS THAN 5 g.p.m. DIABASE SHEAR BANDED.
120
100
204+75± – 204+84 ±
FACE BROKE AHEAD OF SHIELD TO END OF FULL FACE ROCK TUNNEL. VOID FORMED ABOVE CROWN OF UNCERTAIN DIMENSIONS. ESTIMATES RANGE FROM 15' TO 40' ABOVE HOOD, LATERAL EXTENT NOT KNOWN. GROUTED FROM SURFACE.
STA. 205+42 ±
FACE VERY BLOCKY AND SHEARED. FACE DRY, OVERBREAK LESS THAN 1' AVERAGE.
80
STA. 204+90± – 204+96 ±
ARGILLITE, SHEARED, VERY BLOCKY, SEEPAGE FROM SHEAR SURFACE. JOINT SURFACES ALTERED. OVERBREAK 2' MINIMUM.
STA. 205+70± – 206+00 ±
BLOCKY AND SLIGHTLY WEATHERED ARGILLITE. LOCALLY SHEARED, JOINTS, RUSTY, SOME CLAY COATING. OVERBREAK RANGES FROM 1'–4'. GENERALLY DAMP, NO SEEPS.
60
A
D
D
40
20
STA. 204+56 ±
TILL – GRAY SILTY SAND WITH GRAVEL, COBBLE, SMALL BOULDERS EXPOSED IN CORWN. DIABASE, PARTIALLY SHEARED AND FRACTURED IN MOST OF FACE. FACE DRY.
STA. 205+52 ± – 205+60 ±
SHEARED DIABASE DIKE IN FACE, JOINTS CLAYEY, VERY BLOCKY, FACE WET, SLIGHT SEEPAGE.
0
205+00
206+00
B'
C'
STA. 205+00±
ARGILLITE EXTREMELY BLOCKY, LOCALLY SHEARED, PURPLE. 2'–3' OVERBREAK AVERAGE. FACE DRY. SHEAR IS CONTACT WITH DIABASE.
0 10 20 40
SCALE IN FEET

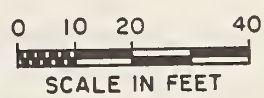


NOTES:

1. Refer to H-3 for explanation of geologic symbols.
2. Refer to H-12, H-13, H-14 for cross-sections.
3. Refer to H-4, H-5 for plan view of geologic maps.
4. Refer to H-10, H-11 for construction summaries.

INBOUND TUNNEL PROFILE


B
C
TSC - 4
PROJECTED 4' WEST
TSC - 17
PROJECTED 16' WEST
TSC - 2
140
120
100
80
60
40
20
0
STA. 206+00±
ARGILLITE FRACTURED AND SHEARED, FACE SLOWLY RAVELLING, SLIGHT SEEPAGE BELOW SHEAR IN CENTER OF FACE.
STA. 205+25 ±
GRAY BANDED ARGILLITE; JOINTS FILLED WITH CLAY; FACE DRY; MODERATELY BLOCKY; OVERBREAK 1'–2' ALL AROUND.
STA. 204+81±
ARGILLITE INTRUDED BY DIABASE DIKE, JOINTS CLAY-FILLED, ROCK RUSTY LOCALLY SEVERELY WEATHERED, SLIGHT SEEPAGE, GLACIAL TILL EXPOSED IN CROWN, BULKHEAD CONSTRUCTED PRIOR TO 10' X 10' INVERT DRIFT EXCAVATION.
STA. 205+67±
MIXED ROCK: ARGILLITE AND DIABASE; SHEARED AND BROKEN, SEEPAGE SLIGHT BUT GENERALIZED, CLAY COATING JOINTS, MODERATELY WEATHERED.
STA. 206+30±
1'–2' DIABASE DIKELET BOUNDED BY 8"–10" SHEAR ZONES IN ARGILLITE, OVERBREAK AVERAGE 2-FEET ALL AROUND, JOINTS STAINED, SLIGHT SEEPAGE.
STA. 205+40±
MIXED ROCK: ARGILLITE AND DIABASE; MODERATELY TO EXTREMELY BLOCKY, LOCALLY SHEARED, FACE DAMP, ROCK MODERATE TO SEVERE WEATHERED.
STA. 204+93±
MIXED ROCK ZONE: ARGILLITE & DIABASE, ROCK WEATHERED MODERATE TO SEVERE, SLIGHT SEEPAGE BLOCKY, BROKE AHEAD 4'–8' REQUIRES TIMBER FOREPOLE FROM LAST RIB, ROCK IN CROWN LOOKS EXTREMELY WEATHERED POSSIBLE TILL FILLED JOINTS.
205+00
B'
206+00
C'


0
10
20
40
SCALE IN FEET

203 +00 204 +00

LITHOLOGY	TILL | ARGILLITE | DIABASE
TUNNELING CONDITION	MIXED FACE TUNNEL
WEATHERING	M S M-S M M M-S S M M Sl-S Sl F-M
OVERBREAK	1'-2' 2'-5' 1'-3' 1'-4' 2'->10' 3' 7-8'
DISCONTINUITY SPACING	<1' | 1'-3' | <1' | 1'-3'
WATER INFLOW (GPM)	0-5 DRY 2-3' 1'-3' 0
TEMPORARY SUPPORT	W 8x35 CIRCULAR RIBS: 4-FOOT o/c
TUNNELING METHOD	SHEILD-DRIVEN PRECEDED BY INVERT PILOT DRIFT

NOTES:

1. Refer to H-4, H-5 for plan view of geologic maps.
2. Refer to H-6, H-7 for outbound tunnel profile.

OUTBOUND TUNNEL CONSTRUCTION SUMMARY

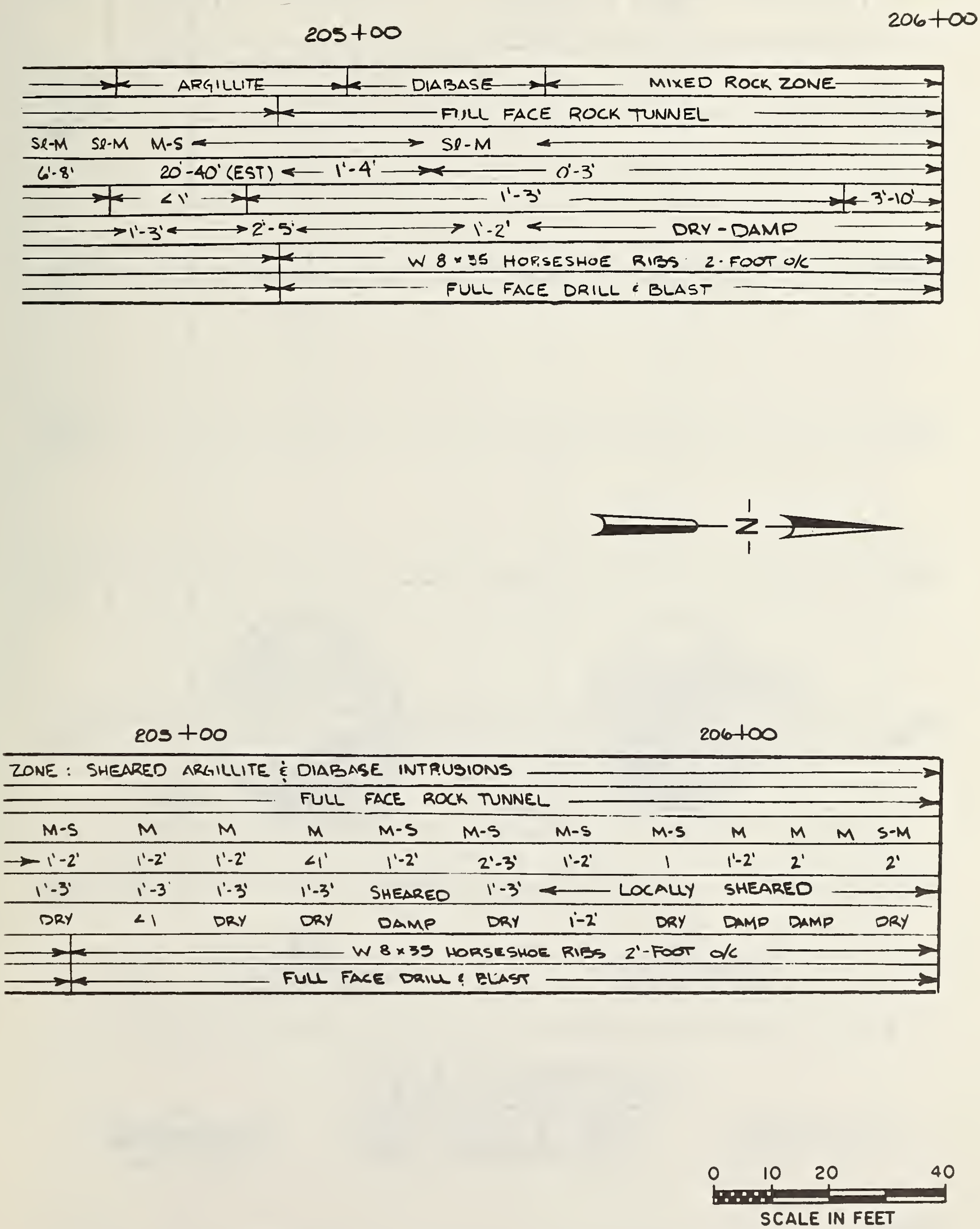
203+00 204+00

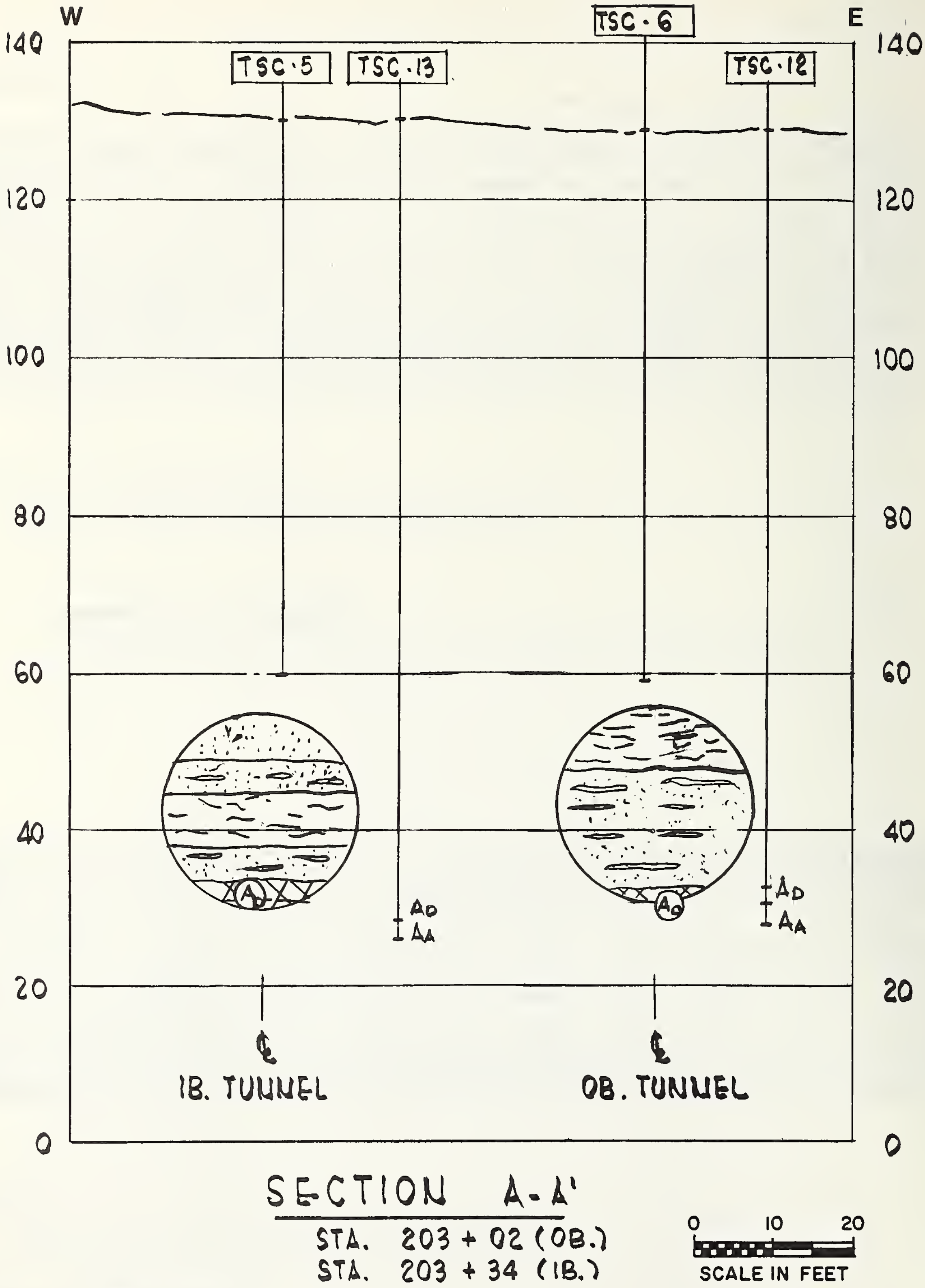
LITHOLOGY	TILL | MIXED ROCK
TUNNELING CONDITION	MIXED FACE TUNNEL
WEATHERING	S S M Sl-M Sl-M Sl M M
OVERBREAK	NO DATA
DISCONTINUITY SPACING	<1' <1' <1' <1' <1' 1'-3' 1'-3' 1'-3'
WATER INFLOW (GPM)	5-10' | <1' <1' <1' <1' DRY DRY
TEMPORARY SUPPORT	W 8x35 CIRCULAR RIBS: 4'-FOOT o/c
TUNNELING METHOD	SHIELD DRIVEN PRECEDED BY INVERT PILOT DRIFT

NOTES:

1. Refer to H-4, H-5 for plan view of geologic maps.
2. Refer to H-8, H-9 for inbound tunnel profile.

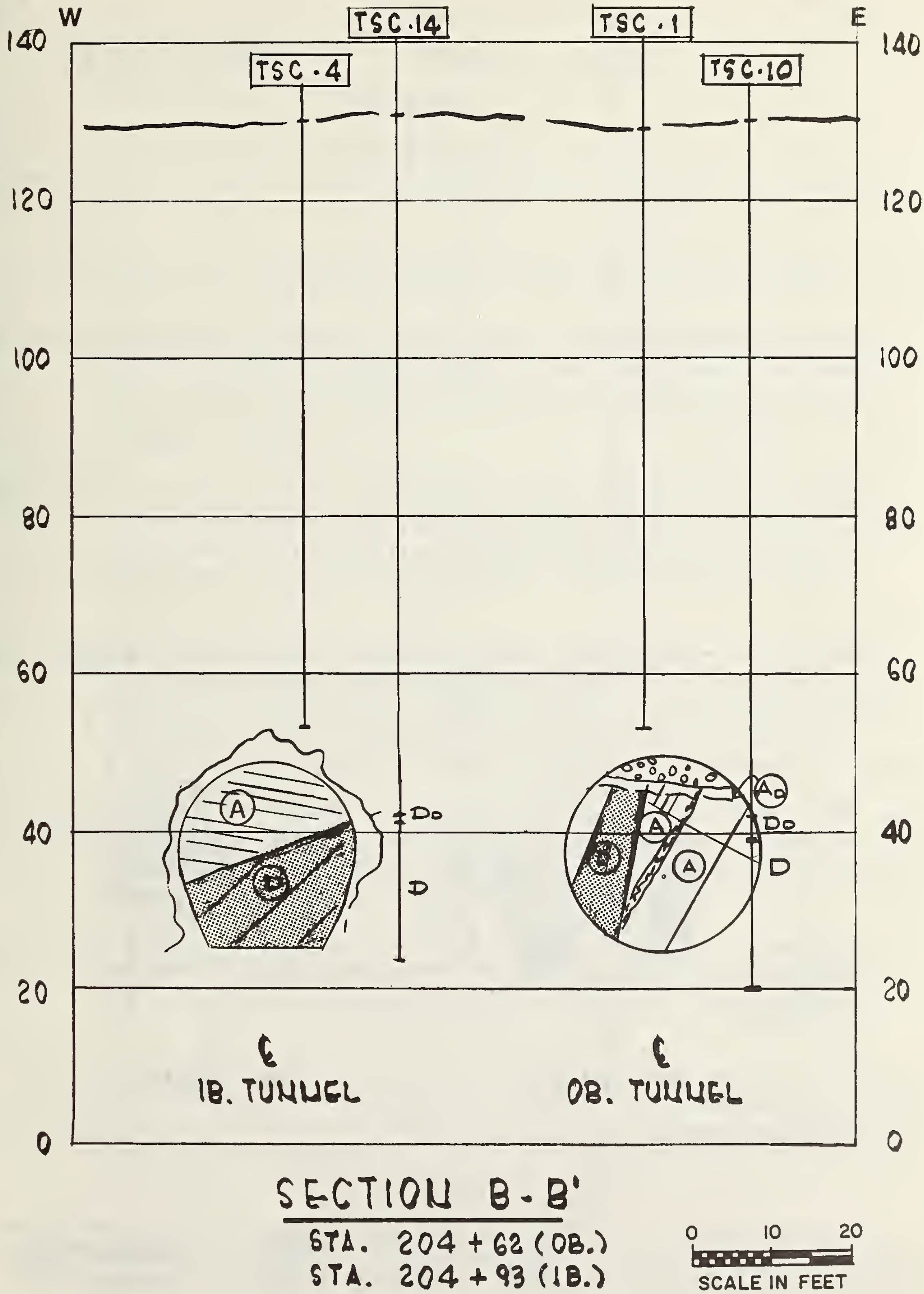
INBOUND TUNNEL CONSTRUCTION SUMMARY


205+00
206+00
ARGILLITE
DIABASE
MIXED ROCK ZONE
FULL FACE ROCK TUNNEL
Sl-M Sl-M M-S
Sl-M
6'-8'
20'-40' (EST)
1'-4'
0'-3'
<1'
1'-3'
3'-10'
1'-3'
2'-5'
1'-2'
DRY - DAMP
W 8 x 35 HORSESHOE RIBS 2-FOOT o/c
FULL FACE DRILL & BLAST
N
205+00
206+00
ZONE : SHEARED ARGILLITE & DIABASE INTRUSIONS
FULL FACE ROCK TUNNEL
M-S M M M M-S M-S M-S M-S M M M S-M
1'-2' 1'-2' 1'-2' <1' 1'-2' 2'-3' 1'-2' 1 1'-2' 2' 2'
1'-3' 1'-3' 1'-3' 1'-3' SHEARED 1'-3'
LOCALLY SHEARED
DRY <1 DRY DRY DAMP DRY 1'-2' DRY DAMP DAMP DRY
W 8 x 35 HORSESHOE RIBS 2'-FOOT o/c
FULL FACE DRILL & BLAST
0 10 20 40
SCALE IN FEET



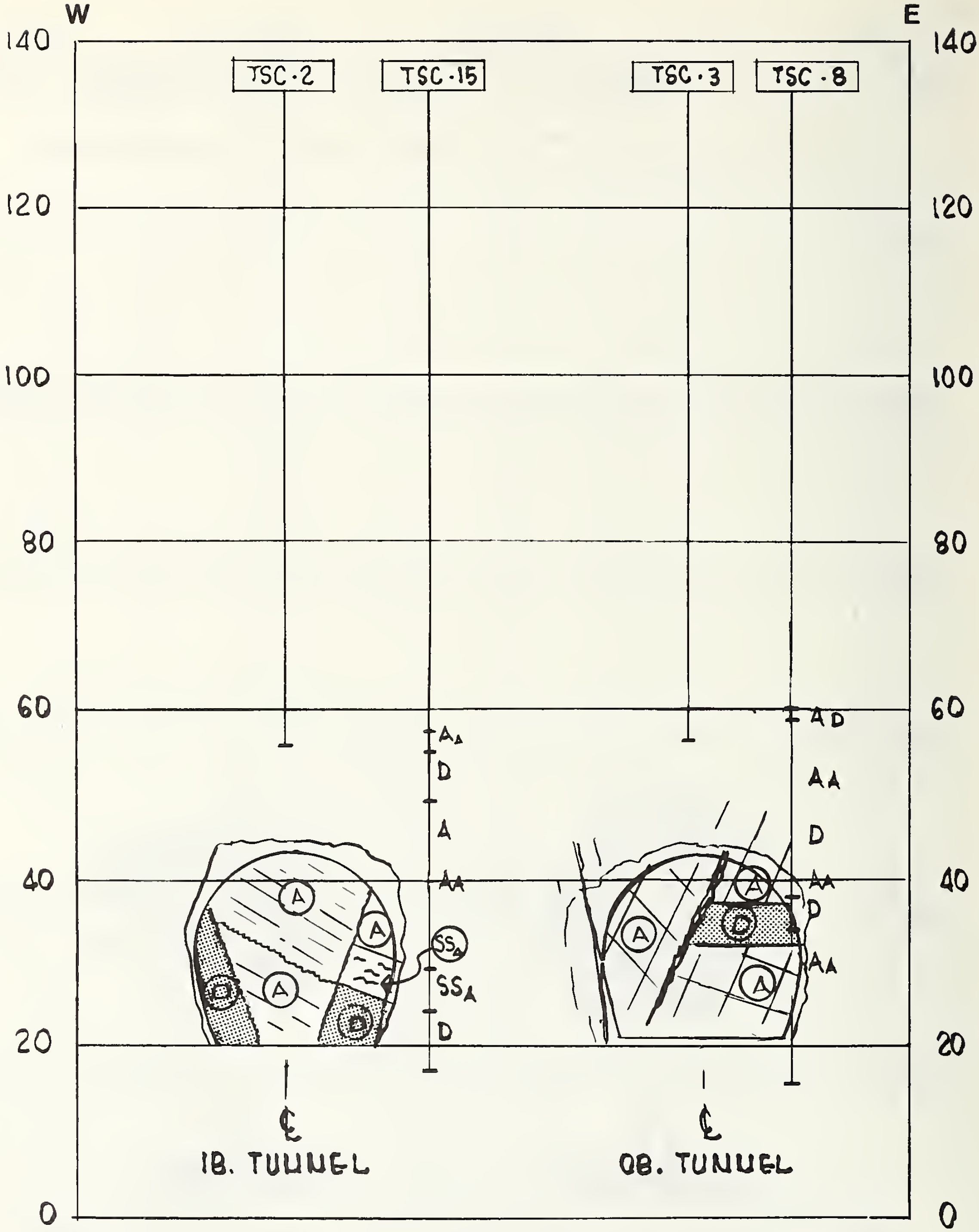
NOTES:

1. Located at approximate position of Section D of Stage I report.
2. Refer to H-4 through H-9 for location in plan and profile.
3. Refer to H-3 for explanation of mapping symbols.



NOTES:

1. Located at approximate position of Section G of Stage I report.
2. Refer to H-4 through H-9 for location in plan and profile.
3. Refer to H-3 for explanation of mapping symbols.



SECTION C-C'

STA. 205 +62 (OB.)
STA. 205 +93 (IB.)

NOTES:

1. Located at approximate position of Section I of Stage I report.
2. Refer to H-4 through H-9 for location in plan and profile.
3. Refer to H-3 for explanation of mapping symbols.

APPENDIX I

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APPENDIX J

GLOSSARY OF TUNNELING AND GEOLOGIC TERMS

<u>backpacking</u> - any granular material which is used to fill the empty space between lagging and ground surface.

<u>blocking</u> - wood blocks placed between the excavated surface of a tunnel and the bracing system (ribs).

<u>breasting</u> - boards placed against the face of a drive to prevent sloughing. The breasting itself is usually supported by one or several vertical soldiers, each held in place by a raker, or strut sloping back to the floor.

<u>bridge action time</u> - the time between firing the shot and the time the first installment of rock drops out of the roof without provocation.

<u>bulkhead</u> - a partition built in an underground structure or structural lining to prevent the passage of air, water, or mud.

<u>cover</u> - the perpendicular distance from a point in a tunnel to the ground surface.

<u>crown</u> - the highest point of an arched tunnel cross section. The term is also used to designate the arched roof above springline. Also called the "roof" or "back".

<u>dike</u> - tabular igneous intrusion that cuts across the planar structure of the surrounding rock.

<u>drift</u> - an approximately horizontal passageway or portion of a tunnel. In the latter sense, depending on its location in the final tunnel cross section, it may be classified as a "crown drift", "side drift", "invert drift", etc. A small tunnel driven ahead of the main tunnel or at any angle to the main tunnel.(See pilot drift).

<u>drypacking</u> - a stiff mortar mix which is used to fill a cavity or confined space.

<u>Dutchmen</u> - steel blocks inserted between rib segments to fill to gap created by expansion of the ribs against the tunnel walls.

<u>excelsior</u> - fine curled wood shavings, used in back-packing.

<u>face</u> - the advance end or wall of a tunnel, drift, or other excavation at which work is progressing.

forepole - a pointed board or steel rod driven ahead of timber or steel sets for temporary excavation support.

forepoling - driving forepoles ahead of the excavation, usually supported on the last set erected, and in an array which furnishes temporary overhead protection while installing the next set.

flow structure - the texture, of an igneous rock, characterized by a wavy or swirling pattern in which platy or prismatic minerals are oriented along planes of lamellar flowage in fine-grained and glassy igneous rocks.

fault - a surface or zone of rock fracture along which there has been displacement.

foliation - planar arrangement of textural or structural features in any type of rock.

gouge zone - a layer of fine, wet, clayey material occurring near and at either side of a fault.

grout - neat cement slurry or a mix of equal volumes of cement and sand which is poured into joints in masonry or injected into rocks. Also used to designate the process of injecting joint filling material into rocks.

heading - the wall of unexcavated rock at the advance end of a tunnel. Also used to designate any small tunnel and a small tunnel driven as a part of a larger tunnel.

initial or temporary ground or rock support - support required to provide stability of the tunnel opening, installed directly behind the face as the tunnel or shaft excavation progresses, and usually consisting of steel rib sets, shotcrete, or rock reinforcement, or a combination of these.

joints - a surface of actual or potential fracture in a rock, without displacement.

lagging - longitudinal supporting members such as boards or steel channels placed between bracing and the rock surface.

muck - broken rock or earth excavated from a tunnel or shaft.

invert - the lowest point on the cross section of an underground passage, or the lowest section of the lining consisting essentially of the floor paving.

overbreak - the quantity of rock that is actually excavated beyond the perimeter established as the desired tunnel outline.

pilot drift - a drift or tunnel driven to a small part of the dimensions of a large drift or tunnel. It is used to investigate the rock conditions in advance of the main tunnel excavation, or to permit installation of bracing before the principal mass of rock is removed.

portal - the entrance from the ground surface to a tunnel.

rib - an arched individual frame, usually of steel, used in tunnels to support the excavation. Also used to designate the side of a tunnel.

round - a group of holes fired at essentially the same time. The term is also used to denote a cycle of excavation consisting of drilling blast holes, loading, firing, and then mucking.

scaling - the removal of loose rock adhering to the solid face after a shot has been fired. A long scaling bar is used for this purpose.

schistosity - the foliation in schist or other coarse-grained, crystalline rock due to the parallel, planar arrangement of mineral grains of the platy, prismatic, or ellipsoidal types, usually mica.

shaft - a usually vertical linear excavation, but may be excavated at angles greater than about 30 degrees from the horizontal.

shear zone - a tabular zone of rock that has been crushed and brecciated by many parallel fractures due to shear strain.

shield - a movable steel framework or canopy which furnishes protection to workers. The earliest applications included working platforms for miners and supported the face.

sill - a tabular igneous intrusion that parallels the planar structure of the surrounding rock.

slaking - the crumbling and disintergration of earth materials upon exposure to air or moisture.

slickenside - a polished and smoothly straited surface that results from friction along a fault plane.

soft ground - term used to describe any tunneling in soil rather than in rock. The ground may be hard or soft in consistency; the word "soft" differentiating it only from "hard" rock.

spiles, spiling - pointed boards or steel sections driven ahead of the excavation (same as forepoles).

springline - the point where the curved portion of the roof meets the top of the wall. In a circular tunnel, the springlines are at opposite ends of the horizontal centerline.

stand-up time - see bridge action time.

stope - an inclined excavation driven from the main tunnel or drift in an upward direction. Excessive overbreak in the crown of a tunnel, if occurring in only a short distance, is sometimes referred to as a stope, as is a local roof fall.

tunnel - an elongate, narrow, essentially linear excavated underground opening with a length greatly exceeding its width or height. Usually horizontal, but may be driven at angles up to 30 degrees from the earth's surface.

APPENDIX K

REPORT OF NEW TECHNOLOGY

The work performed under this contract, while leading to no new technological inventions, has evaluated existing new and advanced methods of geotechnical instrumentation for rapid transit tunneling. Conclusions and recommendations regarding the various types of equipment and procedures are intended to expand and improve the level of geotechnical information obtained from instrumentation programs.

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