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DEVELOPMENT OF A DESIGN TECHNOLOGY FOR GROUND SUPPORT FOR TUNNELS IN SOIL

Volume III: Observed Behavior of an Earth Pressure Balance Shield in San Francisco Bay Mud

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Final Report

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16. Abstract

One of the most successful and economic of the new generation soil tunneling machines is the earth pressure balance (EPB) shield. In 1981 this technology was utilized for the first time in the United States to construct a 12-foot diameter tunnel in San Francisco. The tunnel alignment is under a busy city street in a soft soil deposit with a high groundwater table. An extensive instrumentation program was undertaken to monitor the behavior. The results showed the EPB shield to have successfully controlled ground movements with a unique face support mechanism. No dewatering was required, and the shield was able to cut through 90 wooden piles during the tunneling. The EPB technology would appear to have considerable potential for future use in the United States.

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METRIC CONVERSION FACTORS

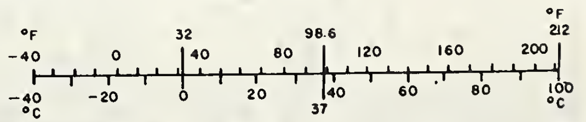
Approximate Conversions to Metric Measures

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

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

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



PREFACE

This report is the third in a series dealing with ground control for tunnels constructed by shield techniques in soil. The first two were devoted towards the development of analytical tools which can be used to study soil tunneling problems. Volume I concerns time-related behavior of clays caused by dissipation of excess pore pressures set up by the tunneling process. Volume II presents the results of a true three-dimensional analysis of shield tunneling processes. This volume contains the results of a field monitoring program designed to define the ground support mechanisms exerted by an earth pressure balance shield.

The field research described herein was primarily sponsored by the U.S. Department of Transportation Urban Mass Transportation Administration (UMTA) and the Transportation Systems Center (TSC) of Cambridge, Massachusetts. Mr. Philip A. Mattson of TSC served as contract monitor and provided invaluable assistance in procedural matters. Mr. Gilbert L. Butler of UMTA helped in the development of the original ideas for the project. Funding for the instrumentation was made available by the Environmental Protection Agency through the San Franciso Clean Water Project under the executive directorship of Mr. Donald J. Birrer. Mr. Birrer provided key support in the initial negotiations to undertake the instrumentation project. His continued interest in the effort aided the authors immeasurably.

The research could not have been carried out without the assistance and cooperation provided by a number of other individuals and organizations. Mr. Harry Chin of Deleuw-Greeley-Hyman Contract Managers helped the authors in numerous ways and made project data available at all times. Mr. John R. Theissen and Ms. Cynthia L. Shaw of Dames and Moore Consulting Engineers gave technical assistance and cooperated in data exchanges whenever requested. The contractor, Ohbayashi-OAC, generously allowed ready site access and opened their project files for documentation efforts. Speical thanks are due to Mr. Shigeo Kurasawa, chief project engineer of Ohbayashi in Tokyo, and Mr. Kaname Tonada, general manager of the San Franciso office of Ohbayashi. Other Ohbayashi personnel who helped include Russel G. Clough, Benjamin Etling, William C. O'Conner and Graham Wozencroft. Finally, the personnel of the Slope Indicator Company of Seattle, Washington, are acknowledged for providing help with the readout equipment and backup equipment as needed.

A special note of thanks is due a dedicated group of Stanford University students who aided in the field monitoring work on their own time. These include Messrs. Carlton L. Ho, Tarik Hadj Hamou, Jean Benoit, and Nader Shafi-Rad. To all the individuals and organizations listed herein the authors express their appreciation.

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EXECUTIVE SUMMARY

In response to rising costs associated with ground water control and requirements for increased control of ground movements, new soil tunneling procedures have been developed over the last decade. The primary innovations have been in the form of advanced shields such as the slurry, earth pressure balance (EPB) and water pressure balance shields. These machines allow soil excavation to be undertaken in difficult ground conditions without the use of compressed air in the tunnel. The Japanese in particular have pioneered the improvement of these shields to the point that they are economically competitive with alternative techniques. However, in spite of the potential for increased usage of the advanced shields, little detail is known about the actual ground support mechanisms they can provide, and under what conditions the best advantages can be achieved.

To examine the major issues concerning advanced shield tunneling, a two-pronged research effort was authorized by the U.S. Department of Transportation in 1979. One phase involved developing analytical tools which can be used to simulate tunneling considering the three dimensional loading effects at the face of a conventional or advanced shield. The second phase was directed towards a field monitoring effort for an advanced shield project which was designed to shed light on the actual behavior of these machines under operating conditions. This report, Volume III in a series, is directed toward the latter effort.

Volumes I and II present the results of the analytical simulation studies.

The instrumentation project was directed towards the first use of advanced shield technology in the United States. This involved a 3000 ft. (909 m) long tunnel built for the San Francisco Clean Water Project. A 12.14 ft. (3.7 m) diameter EPB shield was used by the contractor Ohbayashi-OAC to drive the tunnel. This shield operates by excavating the soil with a rotating cutterhead at the face. The excavated soil is passed into a spoils retaining area which is bounded on the rear by a steel bulkhead. The soil is removed from this area by a screw auger. If the soil comes through the cutterhead at the same rate as the screw auger removes it, a perfect earth balance is maintained.

The tunnel site is located on the northeastern end of the San Francisco peninsula in a busy and highly developed area near the waterfront and San Francisco Bay. Topography along the alignment is essentially flat and the ground water table is located about 15 ft. (3.0 m) under the ground surface. Soil conditions consist of an approximately 20 ft. (6.1 m) layer of rubble fill in a loose to medium density condition overlying a stratum of soft, interbedded silts, clays and sands with an average thickness of 30 ft. (9.1 m). These soils are underlain by a medium dense sandy clay. The tunnel lies entirely within the Bay Mud soil layer with an average crown depth of 25 ft. (7.6 m) and under a water head of 10 ft. (3 m). Located sporadically along the alignment were clusters of old abandoned wooden piles; special carbide cutter teeth were provided on the EPB shield cutterhead to cut through the piles.

The monitoring effort was primarily directed towards measuring ground movements during and after the EPB shield passage. Four lines of instrumentation were placed along the tunnel alignment to allow measurement of lateral and vertical subsurface movement. Survey control was also maintained at 150 locations along the alignment to measure surface settlements.

The observed behavior at each of the four lines showed consistent trends, although relative magnitudes of certain events were larger in some locations than others. One of the most unique aspects of the response occurred as the shield approached the instrument lines. Normally, in the case of a conventional shield, the soil is displaced towards the face of the tunneling machine as it approaches. This action leads to settlements at the ground surface. However, with the EPB shield, the soil was observed at all instrument lines to heave away from the approaching shield. To a degree, this was an intentionally induced effect designed to produce initial outward movements in order to partially counter the subsequent inward displacements caused by passage of the tail void created by the shield advance. The heaving phenomenon is produced by operating the EPB shield so that less soil is removed from the spoils retaining area than tries to enter the shield via the cutterhead.

There were two somewhat unexpected aspects to the effects of the EPB shield at the N-2 project. First, the initial outward movements were largely lateral, and primarily confined to the soft Bay Mud stratum surrounding the shield. Apparently, the overlying rubble fill and an overconsolidated crust on the top of the Bay Mud were stiff enough to cause the heaving displacements to be concentrated in the softer Bay Muds to the sides of the tunneling machine. Little vertical lifting of the soil was observed during shield approach; no more than 0.25 in. (0.6 cm) rise at the ground surface was measured. Second, there was a wide variation in the magnitudes of the heaves when comparing results at Instrumentation Lines 1 and 2 and 3 and 4. Maximum lateral heaves were 0.5 in. (1.3 cm) at Lines 1 and 2, but reached over 3 in. (7.6 cm) and 2 in. (5.1 cm) at Lines 3 and 4 respectively. The smaller displacements at Lines 1 and 2 were apparently typical of the shield performance in normal ground conditions. The behavior at Lines 3 and 4 can be traced to the presence of wooden piles on the tunnel alignment at these locations. As the EPB shield cut through the piles, wood fragments partially clogged the screw auger, causing a slow down in rate of soil removed from the spoils area. Reflecting this situation, the earth pressure cell, which was set on the inside of the retaining bulkhead, registered unusually high values at Lines 3 and 4. Where piles were not encountered these pressures were considerably lower.

After the initial heaves, the soil at all lines of instrumentation responded to the passage of the tail void at the rear of the shield by moving towards the shield. These movements were observed in both the rubble fill and Bay Mud soils, and there was a similarity in the magnitudes of movements at all lines, regardless of differences in initial heaving effects. Some limited differences in behavior were produced by differences in shield pitch at the instrument lines. The similarity of tail void movements at the four instrument lines was apparently caused by the relative lack of impact of the initial heaves on the soils overlying the shield as noted earlier. The basic ground response to the tail void for the EPB shield was not greatly different than that usually reported for conventional shields. The tail void effect is therefore the "equalizer" between shield types and serves as the principal agent causing surface settlements. Surface settlements over the centerline of the tunnel along the entire alignment ranged from 0.2 to 3.0 in. (0.5 to 7.6 cm), with a median value of 1.3 in. (3.3 cm). The largest values corresponded to areas where a prolonged work stoppage occurred, at which time the face support mechanism of the EPB shield was negated. Other than these cases, the settlements were below those which would be expected for a conventional shield operating in similar conditions. The improvement in settlement control for the EPB shield relative to a conventional shield can apparently be attributed to the ability of the EPB shield to control movements at the face.

In addition to the generally good ground movement control, the ground water table was essentially undisturbed by the EPB shield process. No changes in water levels were observed during the construction in observation wells set alongside the tunnel alignment. Very local effects did occur just as the shield approached the instrument lines in the form of rapid water level rises in the central inclinometer casings. This interesting phenomenon apparently reflected a rapid development of excess pore pressure just in front the shield due to the heaving movements induced at the face. However, the influence of this effect did not extend to any significant distance beyond the shield.

The performance of the EPB shield at the N-2 project was largely successful in terms of ground movement and ground water control as well as rate of progress. The instrumentation data help explain the reasons for the observed behavior, and should assist in evaluation of future advanced shield projects. Further research into maximizing the beneficial effects of the initial heaving which occurs as the shield approaches should lead to optimizing the performance of this machine.

Chapter 1

INTRODUCTION

Over the past 10 to 20 years, soil tunneling projects have been plagued by rapidly rising costs related to control of ground water, and increased demands that tunneling induced ground movements be minimized. In response, shield manufacturers have worked to develop equipment which would help mitigate these problems. This effort has led to the production of the advanced shields such as the slurry, earth pressure balance, and water pressure balance machines. These machines, developed primarily in Japan and Europe, provide immediate support to the soil being excavated at the tunnel face, and prevent or inhibit ground water flow into the working area of the tunnel (see Abe, et al., 1979, Bartlett, et al., 1973, Matsushita, H., 1979, Miki, et al., 1979, and Clough, 1980). The popularity of the advanced shields has grown rapidly, especially in Japan where over one hundred such projects have been completed in a wide range of difficult soil conditions (Murayama, 1979). Recently, the first such machine was used in the United States to construct a soil tunnel in San Francisco.

In spite of the increased usage of advanced shields, little detail is known about the actual ground support mechanisms they can provide. Nor is it clear exactly what degree of ground control can be achieved by using them instead of conventional techniques. Thus, the designer is faced with an ambiguous situation in regard to when and where the advanced shields should be used, and how their advantages can best be obtained. The research described in this report is devoted towards resolving some of the unknowns surrounding this promising new technology.

There are a number of techniques which can be used to study advanced shields including model tests, analytical tools, and field instrumentation programs. In this report, the latter is used. Observed behavior is presented for the recently completed earth balance shield project undertaken for the San Francisco Clean Water Program. This work represents the field documentation phase of a larger research program which involves supplementary analytical studies (Johnston and Clough, 1982 and Kasali and Clough, 1982). The different phases of the study are complementary in that to develop analytical techniques to model the shield response, the field behavior is needed to establish behavior baselines. At the same time, to fully explore the details of the shield support process and to allow extrapolation of the field data, the analytical procedures are needed.

The tunnel which is the subject for this report is located along the northeastern edge of the San Francisco Peninsula in the vicinity of the San Francisco Waterfront (See Figure 1.1). Prior to 1860, this area was occupied by the waters of San Francisco Bay. Over the period 1860 to 1900 it was filled with soil, refuse, and debris to form a level ground surface some 10 ft. (3 m) above the water surface. Underlying the fill are the soft sediments of the Bay. During the development of the area, many structures and wharves were founded on wooden piles

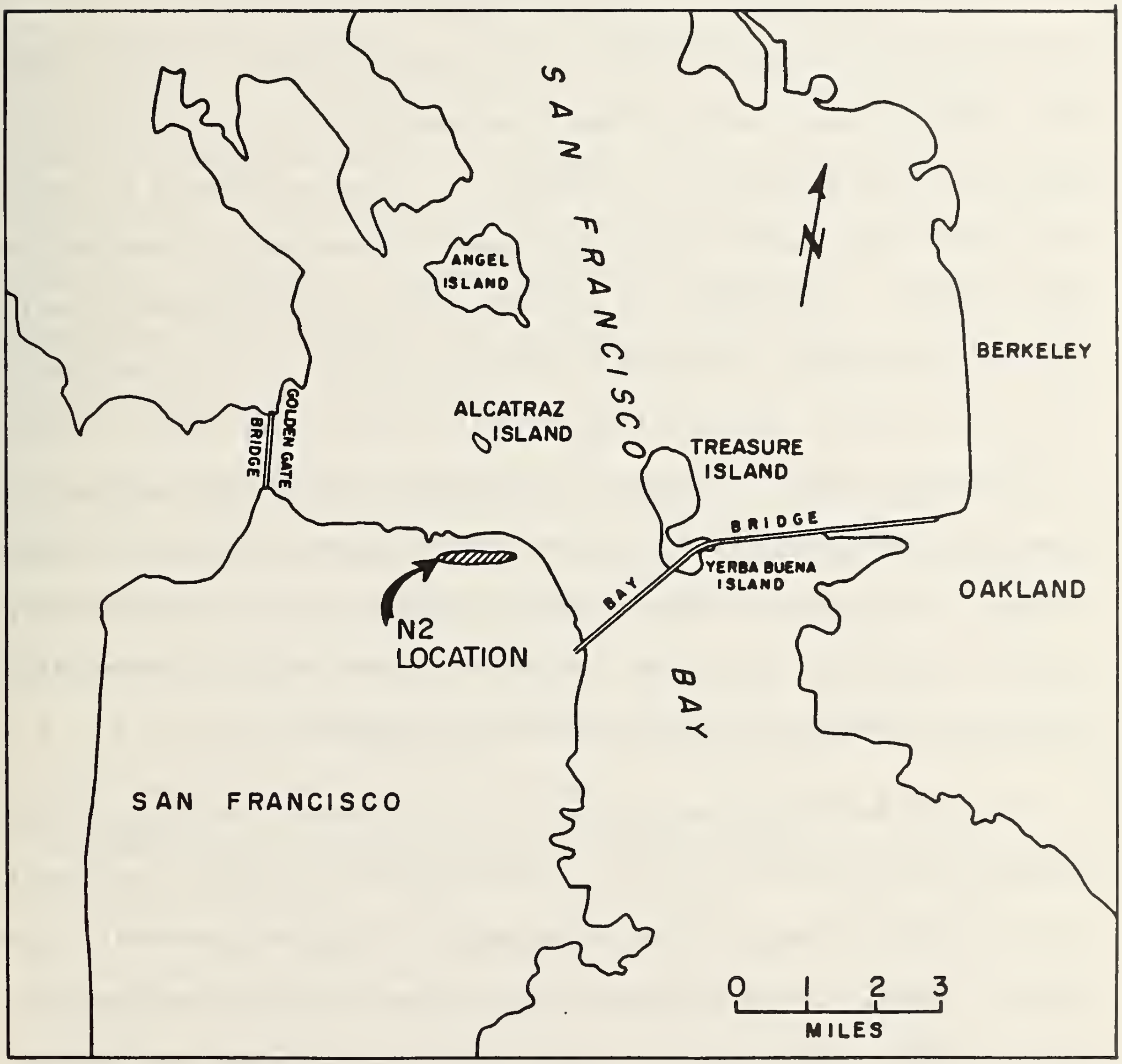


Figure 1.1: Site Location Plan

driven into hard strata below the sediments. Most of the early structures burned down, collapsed in the earthquake of 1906, or were razed for new buildings; the piles, however, remained in place. Today this area is a busy hub of tourist and commercial activity.

The tunnel, 12.14 ft. (3.7 m) in outside diameter, is about 3000 ft. (909 m) long, and traverses in an essentially east-west direction under the right-of-way of a city street. It was constructed as a part of a large storm water storage and transfer scheme and is known as the N-2 contract. The tunnel lies entirely within the soft sediments, with the crown just below the bottom of the overlying fill. Cover is from 20 to 30 ft. (6 to 9.1 m) and the water table is from 10 to 20 ft. (3 to 6 m) above the crown. The project presented a number of challenges in view of the soft, saturated soil along the alignment, the busy overlying streets, the commercial and residential structures abutting the street, many of which are on shallow foundations, and the old wooden piles penetrating through the proposed tunnel cross-section.

In 1979 the company Ohbayashi-OAC, won the bid for the project with a price of 12.7 million dollars, and proposed to complete the tunnel using an earth pressure balance (EPB) shield, a machine previously used only in Japan. Tunneling was underway by early 1981 and was completed in June, 1981. The project was considered to be a success by the owner and the contractor (see ENR, 1981).

For the research program, the instrumentation consisted mainly of three lines of monitoring devices located about a block apart, in the middle third of the route. The positions of the instrumentation lines were chosen to correspond to areas with somewhat differing soil conditions, and in several cases locations where wooden piles would be encountered. At each line lateral and vertical soil movements were measured during and after shield passage. Also, a limited amount of information was obtained on liner response via extensometer points and strain gages. In addition to the three instrument lines, the City of San Francisco and their consultants, DeLeuw-Greeley-Hyman Contract Managers and Dames and Moore Consulting Engineers, maintained survey control, monitored a fourth line of instrumentation identical to that used by the authors, and measured ground water levels during construction.

The instrumentation program generated a considerable volume of information on the response of the ground to the EPB shield process. Details as to how the measurements were made are provided in Chapter 4. Results and interpretations of the performance are described in Chapter 5. A collection of basic data sets on ground movements for different shield positions is included in Appendix A. Appendices B and C describe the tunnel liner instrumentation along with the available measured data.

It is felt that the instrumentation data provided herein will be of considerable value in optimizing the beneficial effects of the EPB shield for ground support in the future. It has already played a strong role in the ongoing computer simulation development effort.

Chapter 2

TUNNEL ALIGNMENT AND SUBSURFACE CONDITIONS

2.1 INTRODUCTION

The subsurface conditions at the N-2 project site were defined primarily through design studies performed by Dames and Moore (1977). Subsequently, additional holes were logged and sampled during the installation of the instrumentation. The samples were returned to the Stanford University geotechnical laboratory and tested.

A plan view of the alignment of the tunnel is given in Figure 2.1. It follows North Point Street, running from the Embarcadero on the east, to an intersection with Columbus Avenue on the west. The alignment roughly parallels the northern waterfront of San Francisco, and is located about two blocks to the south of San Francisco Bay.

2.2 HISTORICAL DEVELOPMENT

The impetus for the development of the San Francisco waterfront was the discovery of gold in 1848. Prior to this time, San Francisco was a small town mainly centered about what is today the Financial District on the eastern side. In 1848, the area around the present location of North Point Street was under water and had not yet been reclaimed from San Francisco Bay. With the need for additional and deeper water port facilities, fills were placed along the northern and eastern fringes of the San Francisco Peninsula beginning in 1850 or so. Also, wharves were

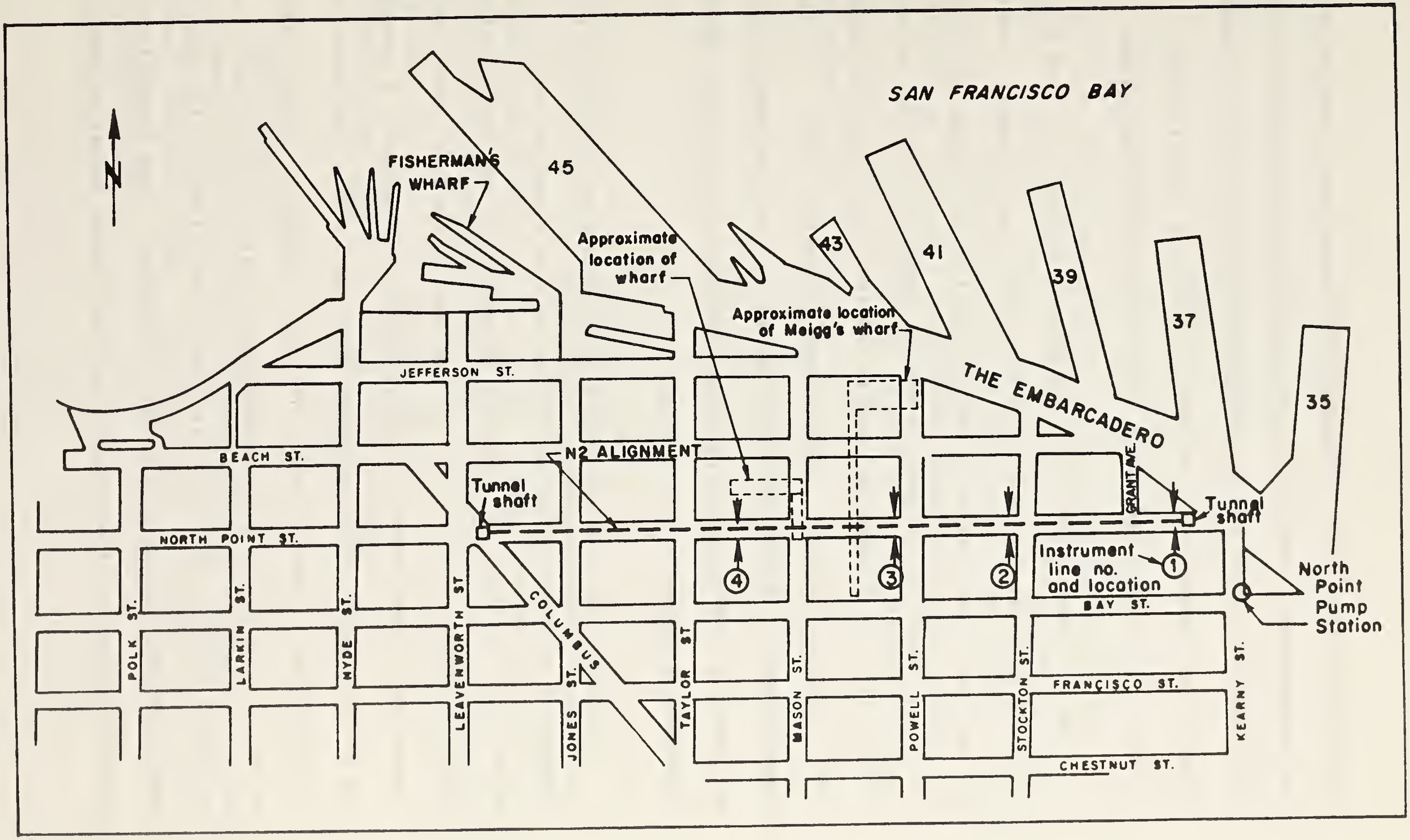


Figure 2.1: N-2 Tunnel Alignment

built out into the water on wooden pile foundations. Details concerning the sequences of this development are related by Dow (1973), Ohmstead and Ohmstead (1977) and Dames and Moore (1977). Fill placement was essentially complete in the area of the N-2 project by the 1890's, although some rubble was dumped there after the 1906 earthquake.

The fill was placed by random dumping procedures, and almost any available material was used. This included rock fragments, dune sand, Bay sediment, and rubbish. Because the fill was dumped through water, it is generally in a loose to medium density condition as has been demonstrated in a number of studies of the area around the waterfront (see Clough and Chameau, 1981, Dames and Moore, 1977, and Youd and Hoose, 1978). Along North Point Street the fill thicknesses vary from as little as five feet (1.5 m) to as much as 30 ft. (9.1 m); the average thickness is around 20 ft. (6.1 m).

Two major pile supported wharf structures were constructed prior to completion of the fill placement which have an impact on the N-2 tunnel. Meiggs Wharf was built in 1852-1853 between Powell and Mason Streets and extended 1600 ft. (485 m) into the Bay (see Figure 2.1). This structure crossed North Point Street at approximately right angles. A second pier was built in 1877, and it occupied about one third of the block bounded by North Point, Beach, Mason and Taylor Streets. Wooden piles were driven for both of the wharves through the soft sediments underlying the Bay and into denser soils beneath them. All of the piles remained in place after destruction of the wharf structures. Less substantial piles were also placed in other areas along North Point Street as markers for the water lots used to define commercial development prior to fill placement. All of the piles along the tunnel alignment presented obstacles to the EPB machine for the N-2 project.

2.3 PRESENT CONDITIONS

With the placement of fill, the topography along the tunnel alignment is generally level. Along North Point Street, the surface slopes uniformly downwards from Columbus Avenue at an elevation of +15 ft. (4.5 m) to +2 ft. (0.6 m) at Taylor Street. From this point to the eastern portal of the tunnel the ground elevation remains constant at about elevation +2 ft. (0.6 m).

As noted, the tunnel alignment stays entirely within the boundaries of North Point Street and does not pass directly under any buildings. However, it does pass under four sewers, three of which are founded on wooden piles. Also, it should be noted that the development alongside North Point Street is intense, with continuous rows of commercial and apartment structures on either side. Along the eastern end of the alignment the structures have as many as five stories, and are largely pile supported. On the western end, the structures are one to two stories and are founded on shallow support systems.

2.4 SOIL CONDITIONS

Figure 2.2 gives the soil profile along the alignment as developed by Dames and Moore (1977). There are three soil units of importance to the tunnel project:

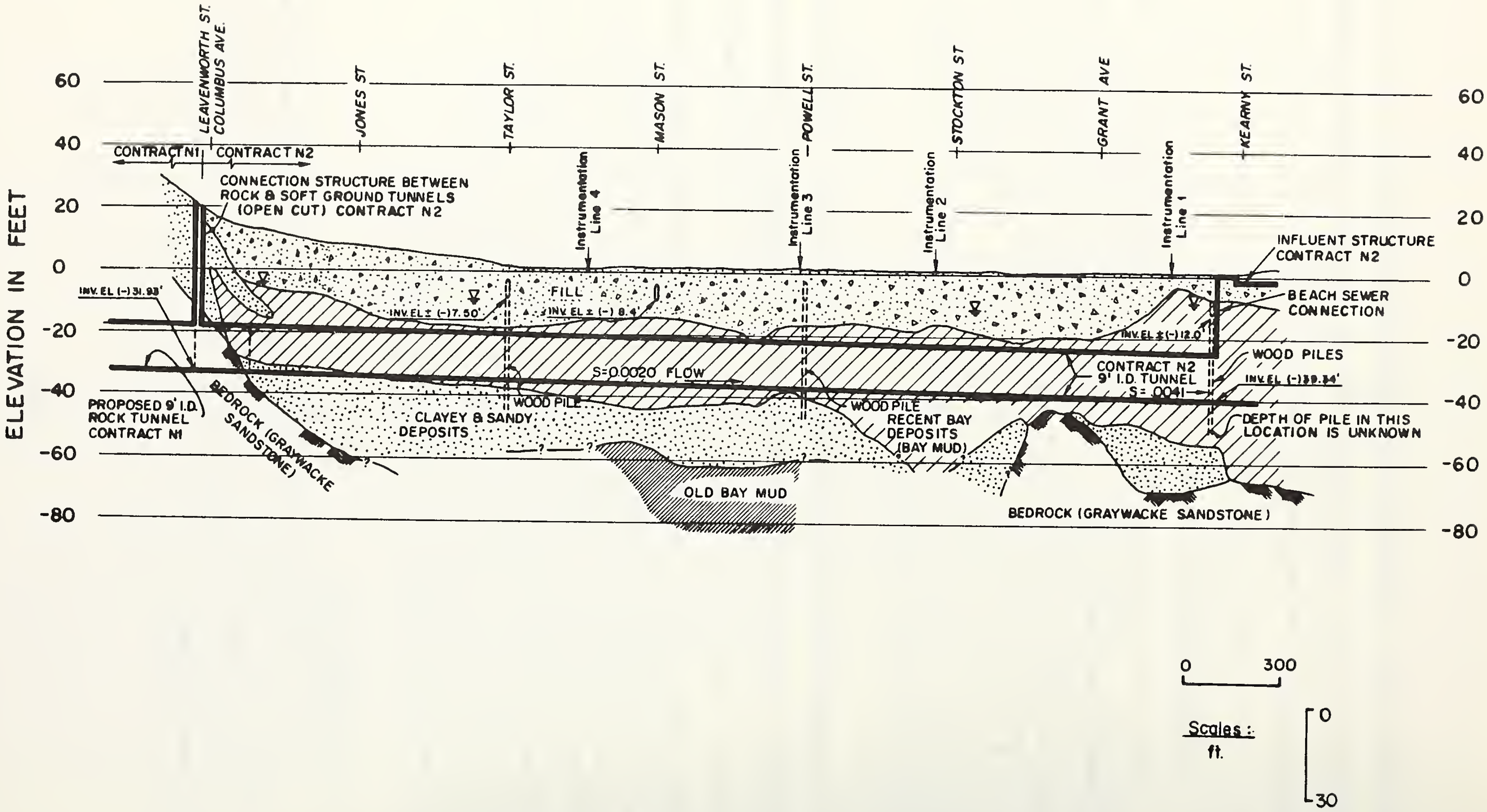


Figure 2.2: Generalized Subsurface Profile

1. The upper rubble fill, which was previously described, with an average thickness of 20 ft. (6.1 m);

2. A deposit of recent soft sediments underlying the fill, called locally Recent Bay Mud, which typically is 30 ft. (9.1 m) thick, except in the area of Stockton Street where the thickness reaches over 45 ft. (13.6 m); and,

3. a layer of colluvial and residual sandy clay, which is found below the recent sediments.

The thickness of this last layer is not completely defined since the borings did not in all cases penetrate it. Located underneath the sandy clay is either a stiff overconsolidated clay, known locally as Old Bay Mud, or bedrock.

The tunnel passes entirely within the Recent Bay Mud, except near the western terminus, where the invert of the tunnel encounters the lower sandy clay (see Figure 2.2). Ground cover above the crown varies from 20 to 30 ft. (6 to 9.1 m) and this mainly consists of the dumped rubble fill.

Also shown in Figure 2.2 are the locations of the four instrumentation lines for the project. Of primary interest are Lines 2, 3 and 4, those monitored specifically for this report. Line 1 was observed by Dames and Moore in order to assess construction techniques at the early stages of the work. At each of the lines, five holes were drilled across the tunnel alignment to install instrumentation, and their logs were used to further define soil profiles at each line. The profiles for lines 2, 3, and 4 are given in Figures 2.3, 2.4, and 2.5, respectively, and show that the soil strata thicknesses within a distance of 20 ft. (6.1 m) on either side of the tunnel centerline do not change significantly.

The logs in Figures 2.3, 2.4, and 2.5 also illustrate that the Recent Bay Mud can vary in composition from a clay to a sand. This is not unusual in the recent sediments deposited along the fringes of San Francisco Bay, although the term Recent Bay Mud is often taken as connoting a clayey soil exclusively. In this particular case, the Recent Bay Mud is primarily a silt or lean clay which in some cases contains sand interbeds. Along most of the tunnel alignment at the depth of the tunnel the Recent Bay Mud contains from 30 to 60% soil sizes passing the No. 200 sieve.

Quantitative data obtained from laboratory tests on samples of the fill and the Recent Bay Mud are given in Figure 2.6. The undrained shear strength, and Atterberg limits presented in this diagram are those for Recent Bay Mud soil samples which were cohesive in nature. The undrained shear strengths were measured in unconsolidated-undrained triaxial compression tests. These data show that the shear strength of the cohesive Recent Bay Mud is around 500 psf (24.3 kN/m^2) just below the fill, and increases slightly with depth. Studies of Bay Mud in other near waterfront areas suggest the undrained shear strength normally increases at a rate of 10 to 13 psf/ft. (0.5 - 0.63 $kN/m^2/m$) (See Tait and Taylor, 1974 and Duncan and Buchignani, 1973). This rate

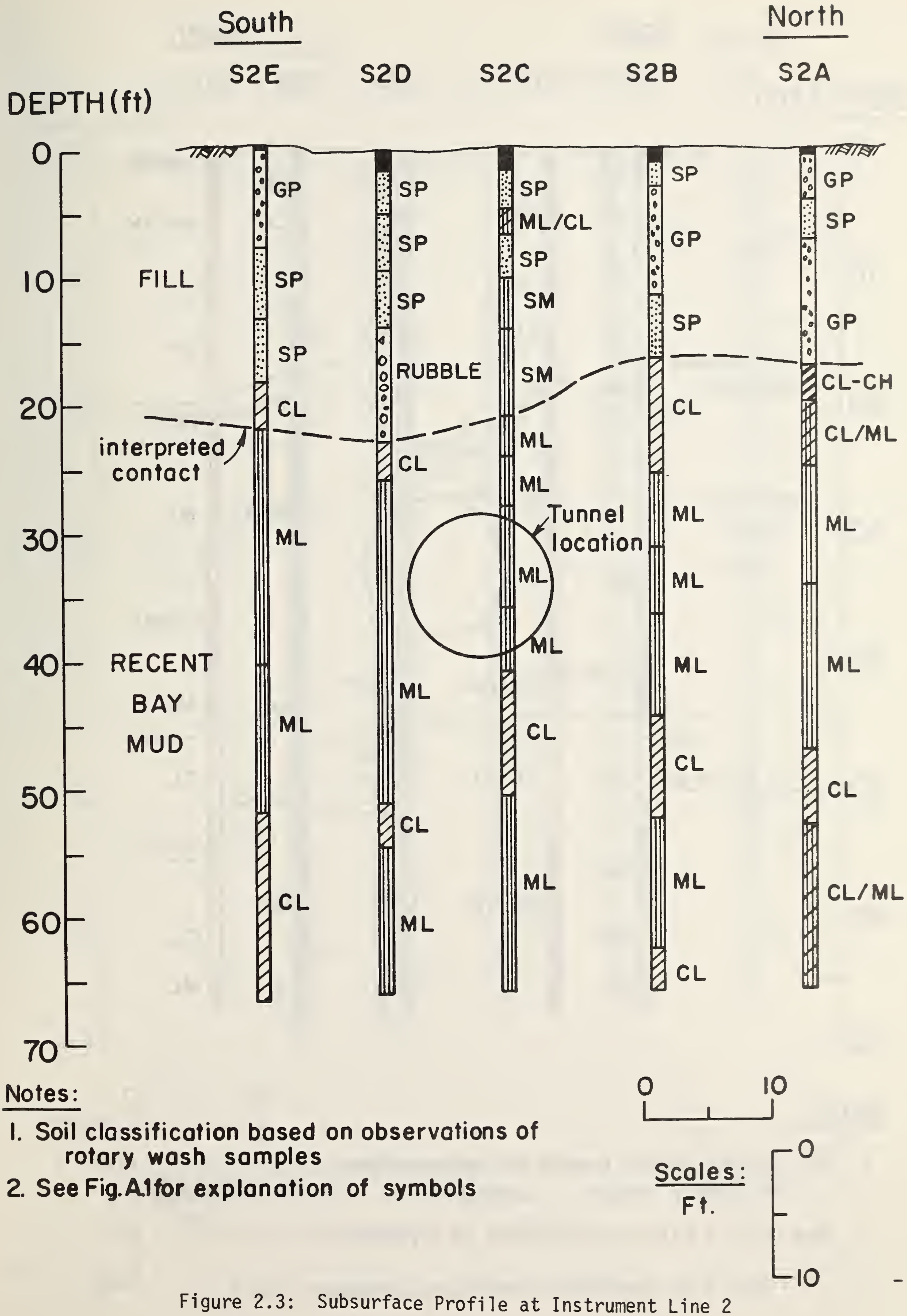
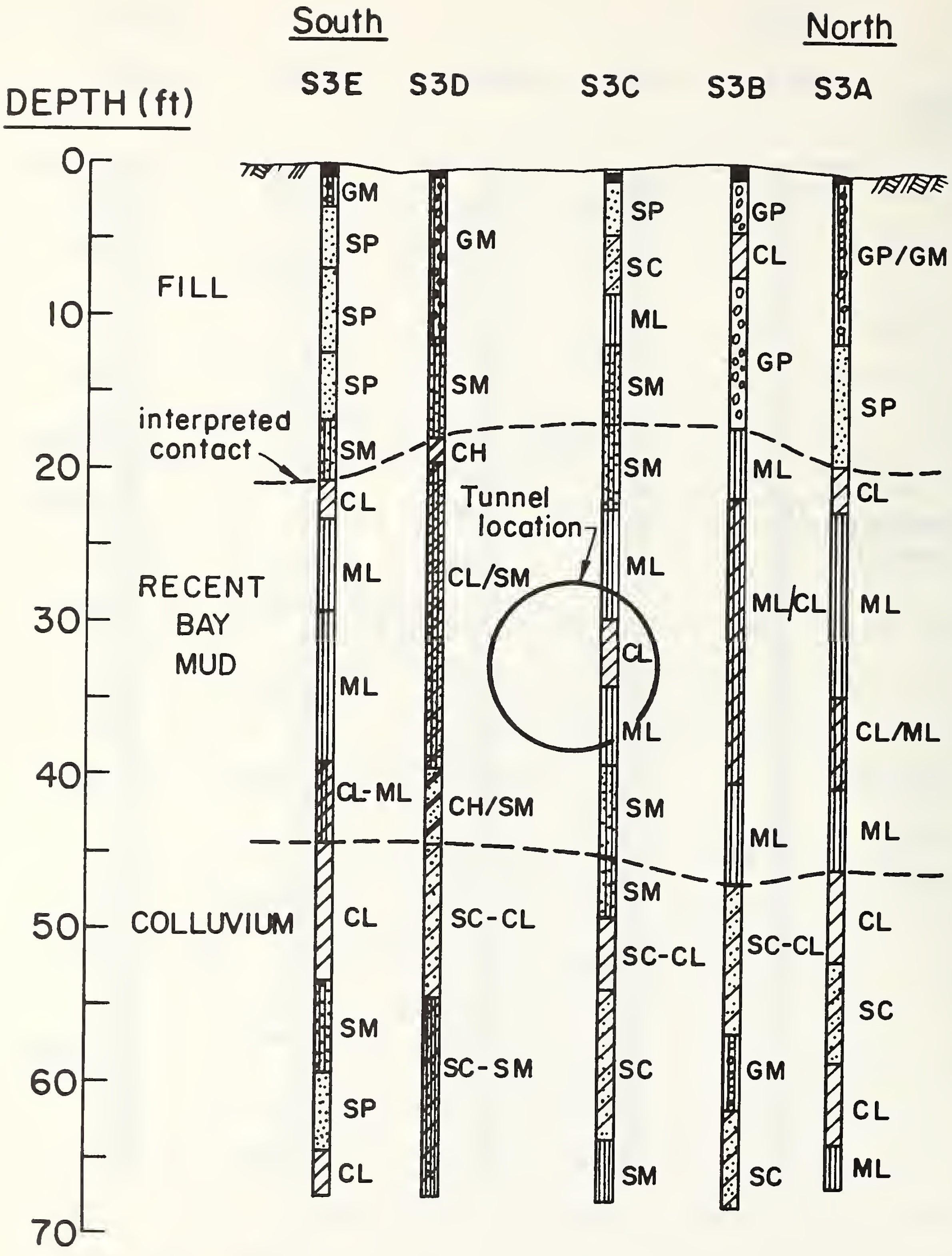


Figure 2.3: Subsurface Profile at Instrument Line 2

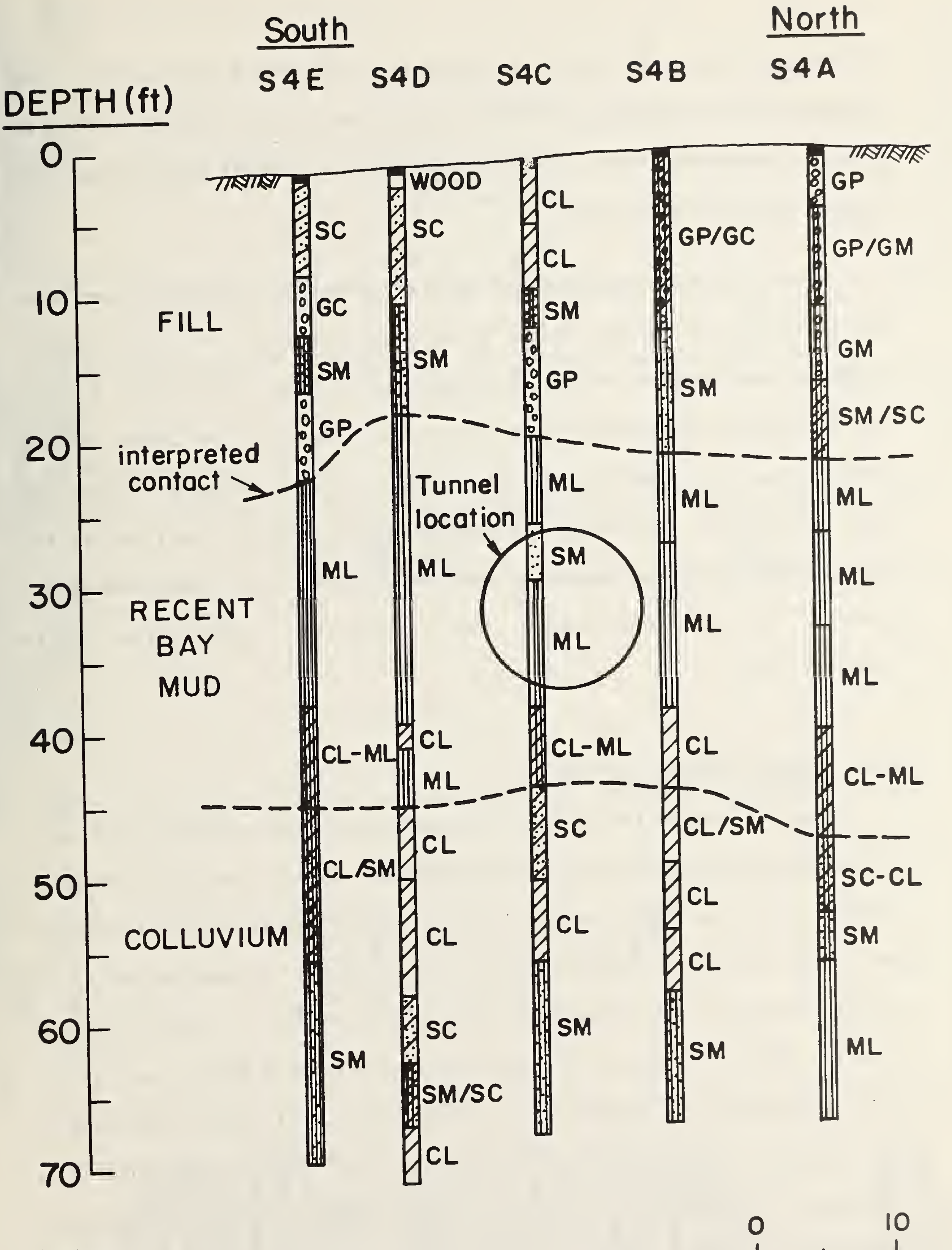


Notes:

1 Soil classification based on observations of rotary wash samples

2. See Fig. A.1 for explanation of symbols

Figure 2.4: Subsurface Profile at Instrument Line 3



Notes:

1. Soil classification based on observations of rotary wash samples
2. See Fig. A.1 for explanation of symbols

Figure 2.5: Subsurface Profile at Instrument Line 4

of increase would generally fit the data in Figure 2.6 as well. The measured unit weights, Atterberg limits and water contents are also largely consistent with results reported for Recent Bay Mud in the downtown San Francisco area.

While no consolidation tests were performed on the Recent Bay Muds, its similarity to nearby deposits would suggest that it is essentially normally consolidated except near the top of the layer where a small depth could be influenced by desiccation. Based on the water content and Atterberg limit data, this would appear to be the case. Near the upper portions of the Recent Bay Mud the water content is well below the liquid limit, while at depth it essentially equals it. These trends are typical of overconsolidated and normally consolidated soils respectively.

2.5 TUNNEL OVERLOAD FACTOR

The overload factor is a simple index relating overburden pressure at the tunnel springline, σ_z to the undrained shear strength, c, at that point. It correlates reasonably well with the degree of yielding which can occur in the soil around the tunnel during excavation and with general levels of ground movements which develop (e.g., see Clough and Schmidt, 1977). Using unit weights of 120 pcf (18.8 kN/m^3) and 105 pcf (16.4 kN/m^3) for the rubble fill and Bay Mud respectively, and assuming the undrained cohesion to be 650 psf (32 k/m^2) at the springline as indicated in Figure 2.6, the overload factor for the N-2 tunnel calculates as 5.8. Values of this magnitude are indicative of the development of significant plastic yielding zones in the soil when

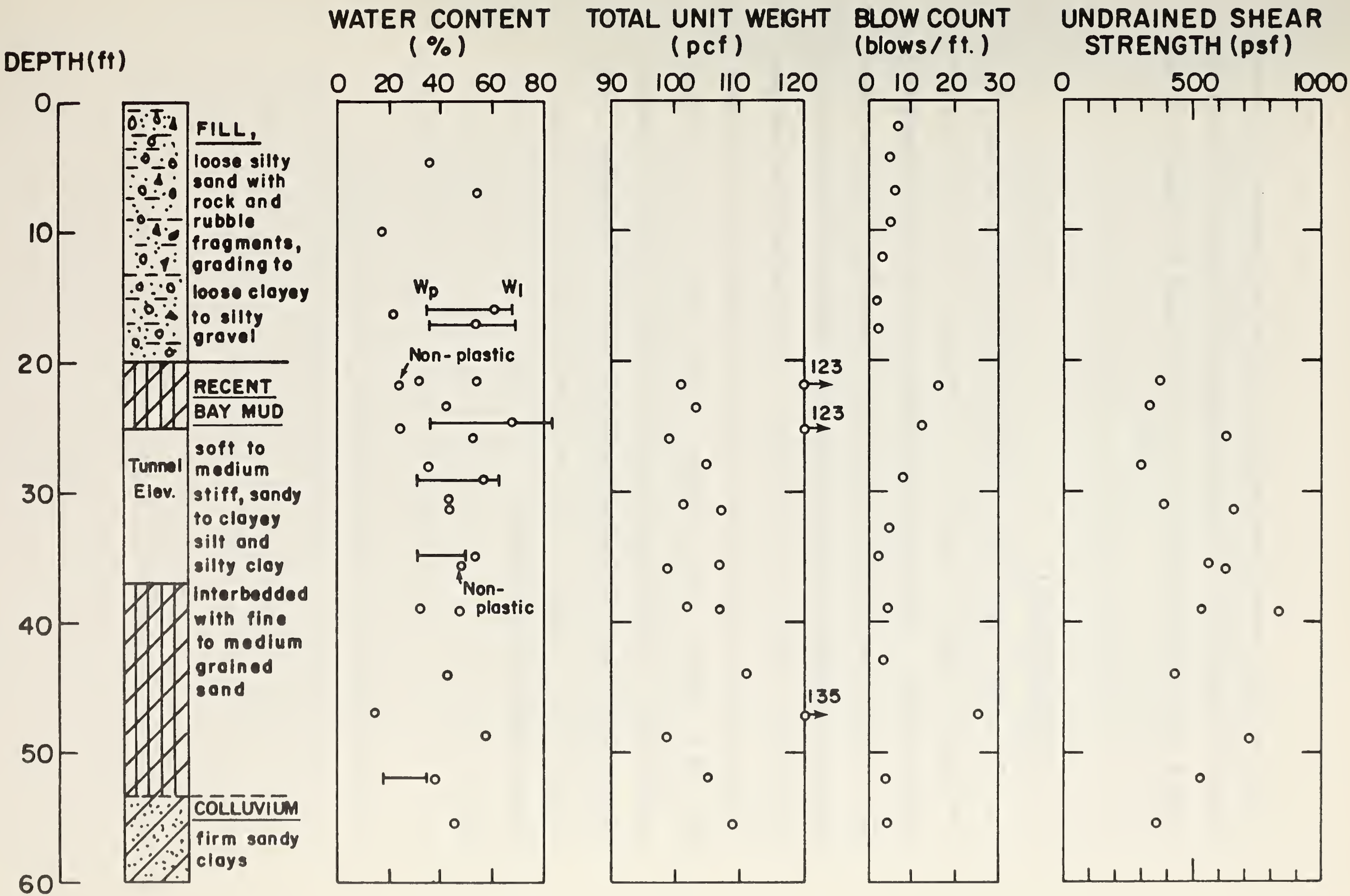


Figure 2.6: Summary of Soil Data at Instrument Lines 2, 3, and 4

tunneling with a conventional shield. Cording and Hansmire (1975) show data for conventional shields which suggest that upwards of 30% of the ground loss at overload factors of 5 to 6 is caused by movement into the open tunnel face.

2.6 GROUND WATER TABLE

The ground water table measured in the fill at the site is consistently located at a depth of approximately 10 ft. (3.3 m) below ground surface. This roughly corresponds with the level of the nearby San Francisco Bay, although no significant movements of the ground water table were observed to occur when the tides changed the Bay levels. The crown of the tunnel is on average 10 ft. (3.3 m) below the ground water table, leading to a hydrostatic pressure at the tunnel springline of 6.9 psi (48.3 kN/m^2). There was some initial concern about control of ground water flow into the earth pressure balance shield, and a special outlet control was included on the shield as will be discussed in the next chapter. However, this was later removed as no problems with ground water were encountered.

Chapter 3

TUNNELING TECHNIQUE, LINER SUPPORT AND PROGRESS

3.1 INTRODUCTION

Because of the relatively shallow depth of the N-2 sewer conduit, it was originally to be built by cut and cover methods. However, in considering the implications of such construction on the tourist traffic in the area, the City of San Francisco decided to require tunnel construction. In the initial phases of project planning, the tunnel was to have been 20 ft (6.1 m) in diameter, and the profile would have been partially in Bay Mud and partially in fill. Under these conditions, only slurry or earth pressure balance shield techniques were to be allowed for bidding, since conventional compressed air technique would be subject to air loss and potential ground collapse. Subsequently, the tunnel was reduced in size to a 9 ft. (2.7 m) inside diameter, and its profile located entirely in Bay Mud. Given this configuration, the City of San Francisco opened the bidding options to either the advanced shields or conventional shields with compressed air.

The low bid on the N-2 contract was 12.7 million dollars as submitted by the company Ohbayashi-OAC, a joint venture of Ohbayashi-Gumi Inc. of Tokyo and its Los Angeles based subsidiary, Ohbayashi American Corporation (OAC). They proposed to construct the tunnel using an EPB shield with an outside diameter of 12.1 ft. (3.7 m). The extra diameter allowance over nine ft. was needed for working space and the permanent cast-in-place concrete liner. For temporary support, a system of precast concrete segments was initially proposed, but later withdrawn in favor of conventional bolted steel segments.

3.2 THE TUNNEL MACHINE

The EPB shield was first introduced in Japan in the mid 1970's as a cheaper alternative to the slurry shield where water control during tunneling is not a critical item. Since that time, its popularity has increased and it has been used on a large number of projects (Murayama, 1979). Details concerning operation of an EPB shield are given in the publications by Kitamura, et al., (1981) and Clough (1980, 1981). In its basic form, the EPB shield is designed to operate in porous soil above the water table or relatively impervious soils (clays or silts) below the water table.

3.2.1 Basic Characteristics of N-2 EPB Shield

Front and rear view photographs of the EPB shield for the N-2 project, as assembled at the Mitsubishi manufacturing plant in Japan are shown in Figure 3.1 and 3.2. The front view shows the "blind" rotating cutterhead at the leading end of the shield. There are three sets of two slots in the cutterhead face which allow the excavated soil to enter into the shield. Between each set of slots are two rows of cutter teeth, pointed in opposite directions, which excavate the soil as the head is rotated. Only one set of teeth are in use during a given direction of rotation. The paired rows of teeth are set in opposite directions to allow the shield head to be rotated in either a clockwise

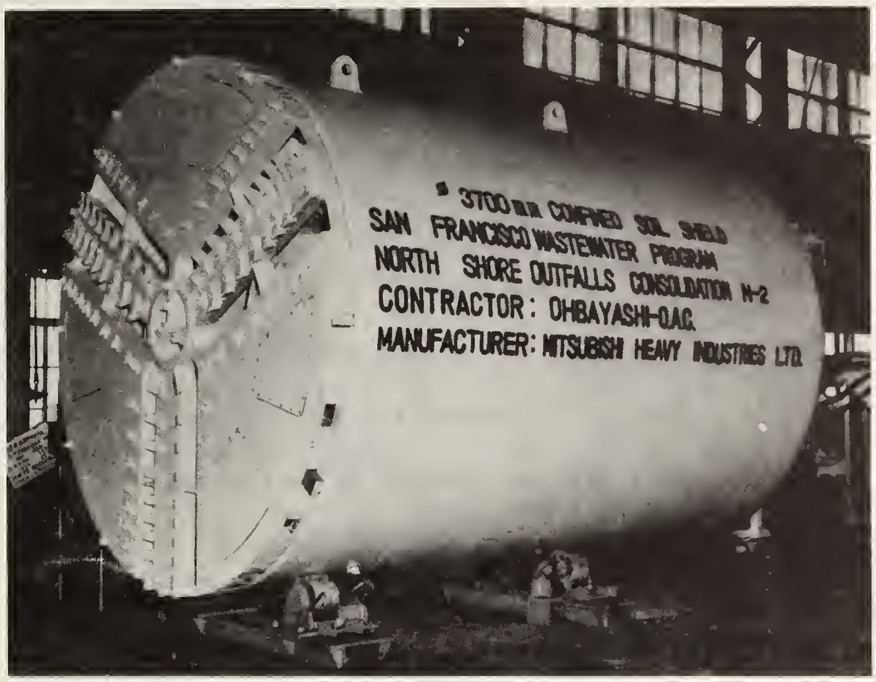


Figure 3.1: Front View of Earth Pressure Balance Shield

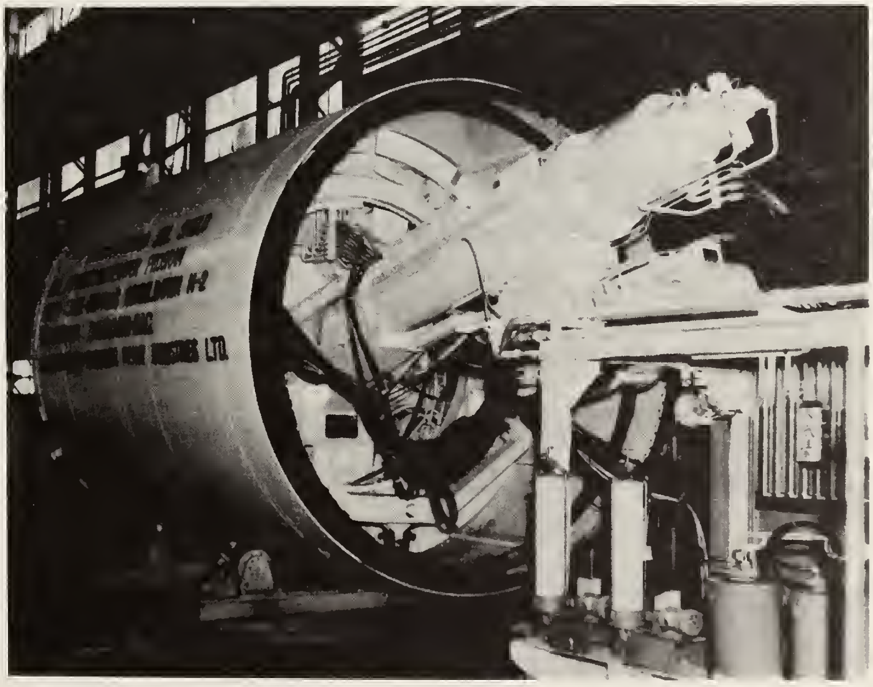


Figure 3.2: Rear View of Earth Pressure Balance Shield

or counterclockwise direction, to avoid torquing the shield consistently in one sense during the advance.

Immediately adjacent to each of the slots is located a set of narrow carbide teeth. These were especially designed to rip and cut wooden piles in place along the tunnel alignment from the old pier structure, water lots, and sewers. This scheme was developed especially for the N-2 project and is not a common feature for an EPB shield. Also added in this case were a limited number of small square steel blocks set along the outer edge of the face designed to help break up the wooden piles.

The front view of the shield also shows two other features of the EPB cutterhead. First, between the slots and teeth there are bolted trapezoidal shaped plates set flush into the face. These can be opened in case of emergency to remove blockages or replace cutter teeth. Second, along the periphery of the cutterhead there are several small openings with recessed solid steel bars, termed overcutters, which can be extended several inches into the surrounding soil. One such bar can be seen extended in the lower portion of the cutterhead in Figure 3.1. These are used only in the event of a slight widening of the tunnel opening is needed to aid in steering the shield. They were never needed during the course of the N-2 project.

In the rear view of the shield, the screw auger prominently projects outward. The function of this key element in the EPB operation can be better visualized in Figure 3.3, a section showing the main EPB shield components. The screw auger serves to remove the soil excavated

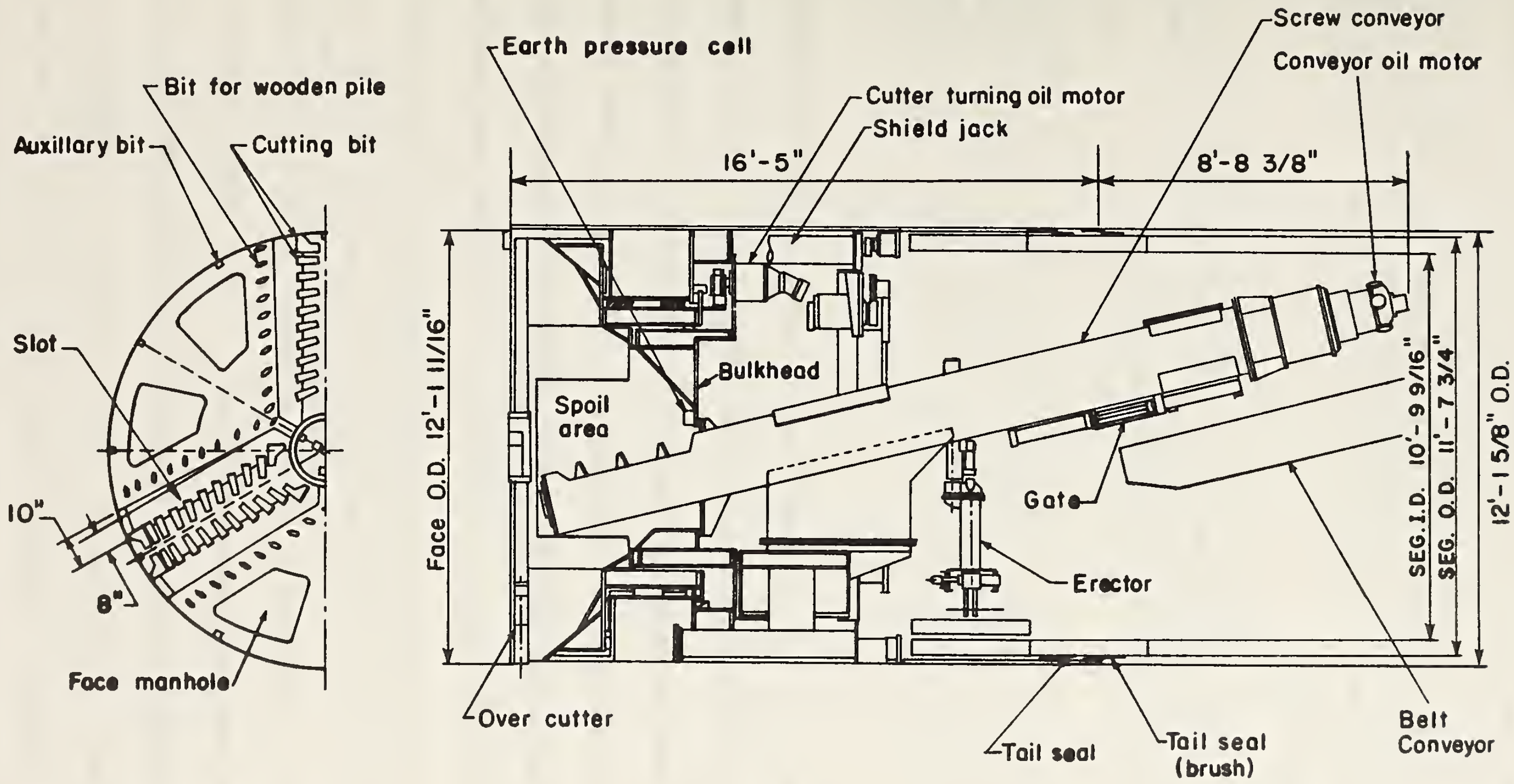


Figure 3.3: Earth Pressure Balance Shield Section

by the cutter head to a muck conveyor belt system. There are two open sections of the screw auger:

1. in the area just behind the cutterhead and in front of the containment bulkhead - at this opening the soil is picked up by the screw auger; and,

2. at the rear of the screw auger where the soil is passed out onto the conveyor belt to fall into a waiting muck car.

The rate at which the screw auger is turned determines the rate at which soil is removed from the spoil containment area ahead of the inner bulkhead.

The relative rates of soil removed by the screw auger and volume of soil occupied by shield advance play a significant role in the ultimate response of the ground to the EPB tunneling process. To maintain a perfect mass balance in the spoil containment area, the volume of soil removed by the screw auger should exactly equal the volume of soil occupied by the forward movement of the shield. In this case, the muck containment area is always kept full, a pre-requisite to ground control with the EPB shield. Should the screw auger remove soil faster than the volume of soil which is taken up by shield advance, a void will develop in the stockpile in the muck containment area. This could allow soil ahead or above the shield to ravel, run, or flow into the shield and lead to large surface settlements or even a collapse. Because it is essential that the operator know the stockpile area is full, an earth pressure cell is placed on the front side of the containment bulkhead (see Figure 3.3). This is precalibrated so that the operator knows what pressure level corresponds to a full stockpile. Should a lower pressure be recorded, then the cutterhead rotation can be speeded up to bring soil in faster, or the screw auger can be slowed down to remove soil slower.

In the actual operation of the EPB shield, it is not uncommon to remove slightly less soil with the screw auger than is occupied by the shield advance. This causes the shield to heave the soil in front of the shield. The soil which cannot enter the cutterhead is squeezed aside during the advance of the machine. The purpose of this is apparently to provide an initial soil movement away from the shield, in order to partially counter the inward movement which is generated by passage of the tail void (see Kitamura et al., 1981). As shown by Johnson and Clough (1982), it also precompresses the soil and reduces initial shear stress levels, thereby reducing potential development of excess pore pressures and subsequent consolidation. Future research is to be directed to this issue to assess the amounts of movement useful to optimize this effect.

Figures 3.2 and 3.3 also show a number of other key features of the EPB shield for the N-2 project:

1. The shove jacks, located just inside the tail of the shield, which are used to propel the shield by pushing off the in-place liner

2. The gate on the rear of the screw auger which can be closed to restrain soil movement out of the auger should flowing or running soil be encountered. In the photograph of Figure 3.2 an additional unit is shown attached to the opening of the screw auger. This is a rotary hopper, designed as a second level control mechanism on soil moving through the screw auger. During actual operation of the N-2 shield, this was removed early on since it was found not to be needed.

3. The tail seals, located in the tail of the shield, which are compressed between the tail and the in-place liner segments. These seals serve to prevent water flow into the shield, and are vital to the tunneling operations. They occupied a space of 1.26 in. (3.2 cm) in the operating condition between the tail of the shield and the liner. For the N-2 shield, the seals were made of a protective rubber flap which partially covered a multistrand fiber brush. Many other types of seals are used in Japan, consisting of wire brushes or layered rubber and steel systems (see Clough, 1980).

Dimensions of the N-2 EPB shield are given in Figure 3.3. The outside diameter is 12.14 ft. (3.7 m), while the length is 16.4 ft. (4.97 m). Slot openings on the face of the shield flared from 10 in. to 8 in. (0.25 to 0.2 m) from the periphery to the center of the cutterhead. The skin of the shield is 1.25 in. (3.2 cm) thick.

3.2.2 Operation of the Shield

To provide power to the shield and to control its many functions, a train followed behind it. This unit was 170 ft. long (51.5 m) and consisted of the motors and pumps needed to operate the cutterhead along with a small semi-enclosed booth for the operator. The booth trailed the rear of the shield by about 10 ft. (3 m). Inside the booth, the operator could observe dial gages which registered information as to cutterhead torque and rate of rotation, screw auger torque and rate of rotation, pressure in each shove jack, earth pressure inside the spoils storage area etc. Based on the available information, the operator was able to modify procedures so as to control shield alignment and the earth balance process.

It should be noted that the control of the EPB shield for the N-2 project was entirely a manual process and dependent upon operator skill. In Japan, an alternative, semi-automated technique is used by some companies (see Clough, 1980). In this case, the rates of rotation of the cutterhead and screw auger can be controlled by a computer system which monitors key parameters and feeds information back to the shield in order to correct response as needed.

3.3 TEMPORARY LINER AND TAIL VOID GROUTING SYSTEM

The temporary liner system installed during the tunneling process consisted of bolted steel segments, with six segments per ring. Details and dimensions of the liner are given in Figure 3.4. It has an internal diameter of 10.8 ft. (3.3 m) and an outside diameter of 11.65 ft. (3.55 m). The liner segments are 3.3 ft. (1 m) long.

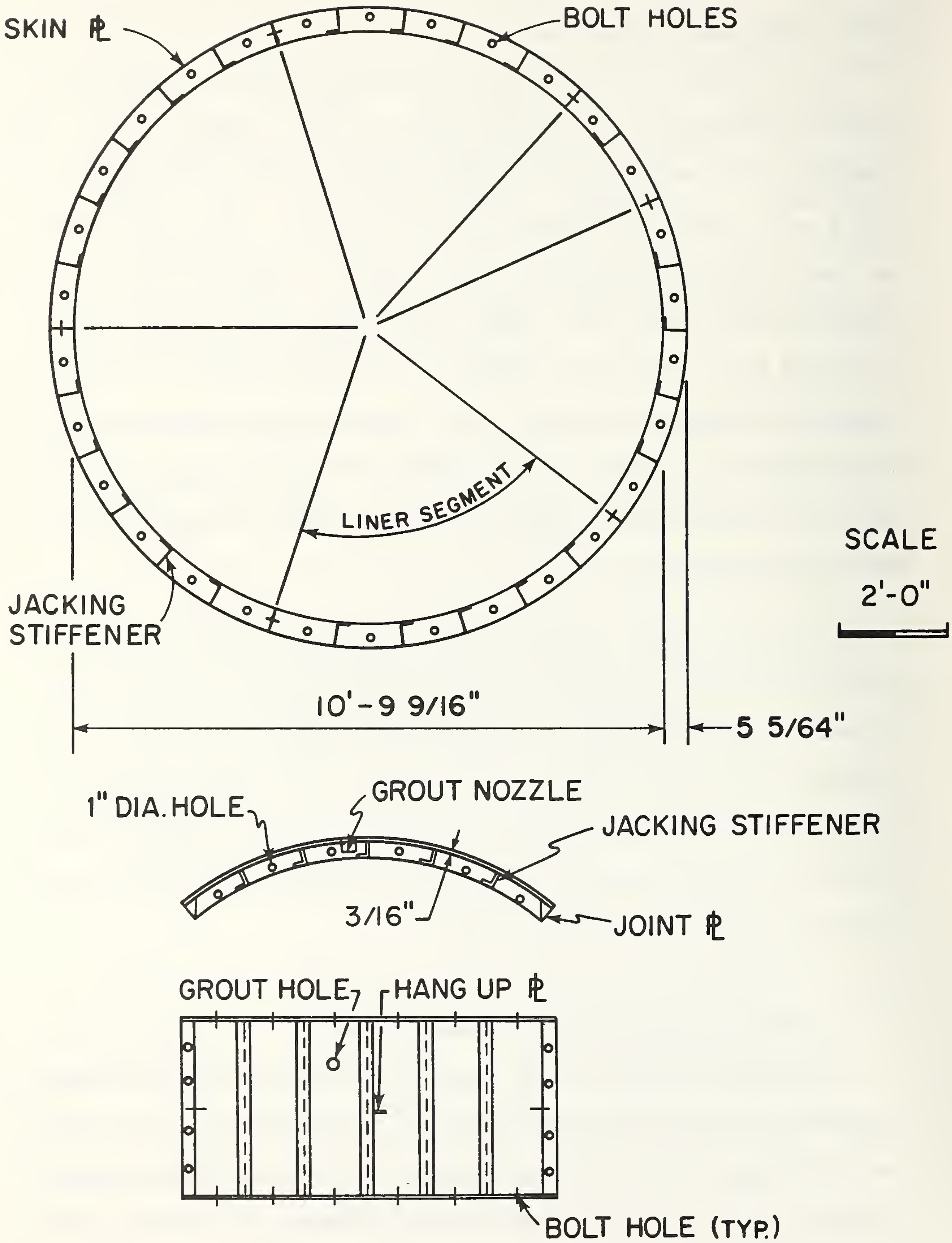


Figure 3.4: Liner Ring and Liner Segment

As is conventional practice, the liner was installed in the tail of the shield with the assistance of a power erector. The gap, or tail void, created behind the liner with the advance of the shield was about 3 in (7.6 cm). This was grouted through grout holes in the steel segments some four rings behind the most recently placed ring. The grout consisted of sand-bentonite-cement mortar and was mixed at the entrance portal to the tunnel. Grouting pressures fluctuated between 20 and 60 psi (140 to 420 kN/m^2).

3.4 TUNNELING PROCESS

The basic tunnel work was completed ahead of schedule and was considered an all-around success (see ENR, 1981). Working on three 8 hour shifts and a five day week, the EPB shield averaged 30 ft. (9.1 m) per day and reached as high as 100 ft. (30.3 m) per day in one case. The Bay Mud soil along the profile turned out to be a good tunneling medium for the EPB process, since it was not so cohesive as to be sticky, but it was cohesive enough to be impervious and prevent uncontrolled water flow through the screw auger. Movement of the soil in the spoil area, down the sides of the bulkhead and into the screw auger was enhanced by the spraying of water through nozzles set on the bulkhead. As the Bay Mud passed onto the conveyor belt it was generally in coherent, soft chunks.

The wooden piles along the alignment were readily cut to pieces by the carbide teeth on the cutterhead face. The contractor estimated that 90 piles were encountered. A photograph of a piece of pile after it passed through the shield is provided in Figure 3.5. This is one of the

Figure 3.5: Remnants of Wooden Pile Excavated Through EPB Shield

larger chunks to emerge; many smaller pieces were found in the muck cars. The piles had three effects on the operations:

1. the tunneling rate was slowed to insure that the piles were cut cleanly; and,

2. the bulkhead earth pressures were observed to increase, possibly due to clogging of the screw auger by pile fragments; and

3. in one case, the cutterhead slots were jammed with remnants of the piles, requiring the shield to stop and to be emptied of soil so that the slots could be cleared.

In addition to its success in handling the soil and piles along the tunnel alignment, the fact that the tunnel was under ground water table by 20 ft. (6 m) also presented no problems. No significant quantities of ground water were observed to pass through the screw auger and there was no evidence in any of the open wells set along the alignment that the water table in the fill dropped. If anything, it appears that water was forced away from the shield during the advance. Water was observed to rise rapidly in, and in some cases to flow out of, the inclinometer tubes located directly on the centerline of the tunnel as the shield approached. This appeared to be due to the formation of high excess pore pressures as the soil was compressed in front of the shield. purposes.

3.5 SUMMARY

The 12.14 ft. (3.7 m) O.D. EPB shield used on the N-2 project was in most ways very similar to that which has been applied to Japanese tunnel projects. The basic principle of operations involves bringing excavated soil into the shield through slots in a rotating cutterhead, and storing it in a retaining area located between the cutterhead and a bulkhead set about 4 ft. (1.2 m) behind the cutterhead. The soil is removed via a screw auger which takes it through the bulkhead and deposits it onto a conveyor belt. Ideally, the cutterhead brings soil in at the same rate that the screw auger removes it so that the retaining area remains full at all times.

Control of the soil entering and leaving the retaining area is crucial to the operation of the EPB shield. For the N-2 project, this was done manually by an operator who sat in a semi-enclosed booth which was a part of the trailing power train. On some projects in Japan the control process is partly automated through a computer monitoring system.

The average progress rate for the shield advance was 30 ft. (9.1 m) per day with a high of 100 ft. (30 m) per day achieved on one stretch. The shield also successfully cut through 90 wooden piles which had been left in place from old structures built in the late 1800's. Special carbide teeth were added to the cutterhead for this purpose.

Tunneling through the Bay Mud soils proved to be easier than originally anticipated. These materials had just enough cohesion to act as an impervious barrier to water flow through the cutterhead, but lacked the cohesion to be sticky and create problems with muck removal. No problems with ground water were encountered even though the tunnel crown was 20 ft. (6 m) below the ground water table. From the operations point of view, the EPB tunneling effort was considered a success.

Chapter 4

GROUND MOVEMENT INSTRUMENTATION AND MONITORING PLAN

4.1 INTRODUCTION

The responsibilities for the instrumentation at the site were subdivided between four parties: (1) Ohbayashi-OAC Inc.; (2) The City of San Francisco; (3) Dames and Moore and Deleuw-Greeley-Hyman; and (4) Stanford University. The contractor, Ohbayashi-OAC Inc., was required by the specifications for the N-2 project to provide, install, and maintain the subsurface and tunnel instrumentation. The City of San Francisco and their consultants made arrangements for surface surveying, and monitored ground water levels and miscellaneous subsurface and tunnel instrumentation. The Stanford University effort was responsible for intensive and long term measurements of three lines of instrumentation designed to define soil behavior during shield passage as well as after shield passage.

Locations of the instrumentation lines are shown in Figure 2.1; Line 1 lies between Kearny St. and Grant Ave. only 200 ft. (61 m) from the entrance portal. This line was monitored by Dames and Moore and the data were provided to the authors. However, because it was not possible to read this line during actual shield passage, only the long term results are of interest herein. Considerably more emphasis will be placed on Lines 2, 3 and 4 which were monitored throughout the tunneling process. Lines 2, 3, and 4 were positioned about 925, 1360 and 2050 ft.

(280, 412 and 620 m) respectively from the entrance portal, with each location selected with a slightly different purpose in mind. At the position of Line 2, the depth of the Bay Mud is the deepest anywhere along the tunnel alignment, and it was felt that it was far enough along from the portal that the construction crews would be well out on the learning curve and typical behavior would be observed. Line 3 was located so as to be in an area where the initial encounters with the wooden piles would occur. Finally, Line 4 was placed in an area where all the adjacent structures are on shallow foundations and the shield has completed cutting most of the wooden piles. Specific soil cross-sections for Lines 2, 3, and 4 are given in Figures 2.3, 2.4, and 2.5, respectively.

The typical arrangement of ground movement instruments at each line consisted of five 60 ft. (18.2 m) long inclinometer casings equipped with telescoping plastic couplings at 10 ft. (3 m) depth intervals (Figure 4.1). The inclinometer casings were designed to allow lateral and vertical soil movements to be measured in front of and adjacent to the tunnel. It was originally planned that the inclinometer casings would be set on a line perpendicular to the tunnel axis, with the center casing set directly on the center line of the tunnel and the others equally distributed about the center casing at 10 ft. (3 m) spacing. This exact arrangement was never achieved. First, the 10 ft. (3 m) spacings often put the intended casing location directly over a utility, a sidewalk curb, or some other obstruction. Thus, the positions were often shifted one to two feet (0.3 - 0.6 m). Second, after installing the casings in the ground, the alignment of the tunnel in the vicinity

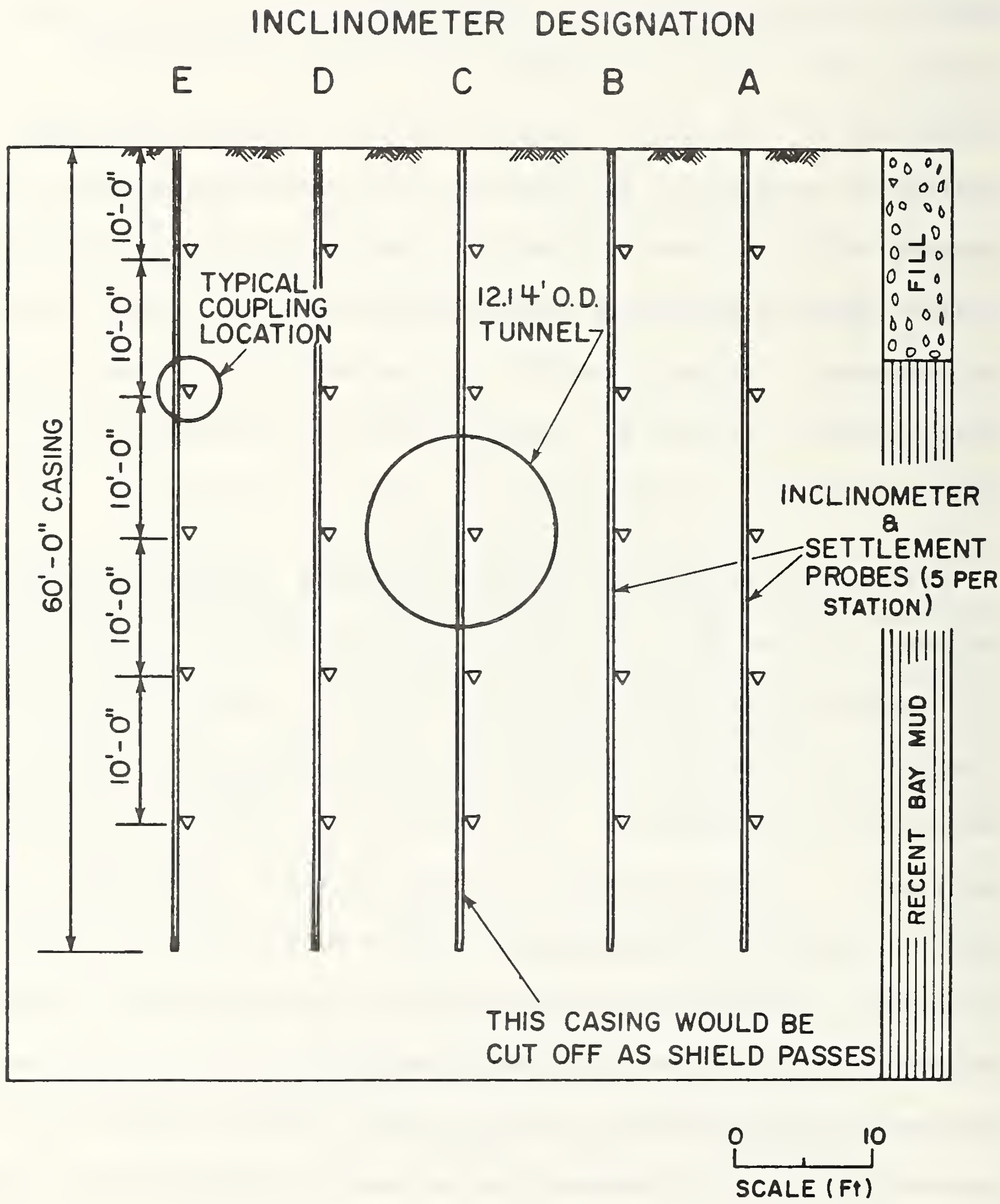


Figure 4.1: Typical Instrumentation Station

of Lines 2 and 3 was shifted slightly leading to a nonsymmetrical distribution of the casings about the centerline. Exact locations of the casings relative to the tunnel center line were established by survey measurements. These will be presented in Chapter V prior to discussion of the movements at each line.

Surface measurements of settlements were generally provided by the City of San Francisco. However, because these data could not be obtained at the unusual hours and with the frequency often necessary, an independent survey was obtained at Line 4 by the Stanford team.

4.2 MONITORING SUBSURFACE MOVEMENTS

Twenty, aproximately 60-ft. deep inclinometers were installed in 6 in. (15 cm) diameter boreholes at the 4 instrumentation lines. Each inclinometer consisted of six, 10 ft. (3 m) long sections of 2.75 in. (7.8 cm) O.D. x 2.32 in. (5.9 cm) I.D. ABS plastic casing with 4 lengthwise grooves spaced at 90 degrees on the inside circumference. Sections were connected with 2.87 in. (7.3 cm) O.D. x 2.55 in. (6.5 cm) I.D. ABS plastic telescoping coupling as shown in Figure 4.2. The couplings were used to measure settlement at every 10 ft. (3 m) interval along the inclinometer casings. Both casings and couplings were manufactured by the Slope Indicator Company (SINCO). The couplings separated individual sections of casing by a maximum of 6 in. and allowed each section to move independently. After an inclinometer casing was assembled and placed in a borehole, the annular space between soil and casing and cover was filled with a cement-bentonite grout. A steel casing and cover was placed flush with street level to protect each installation.

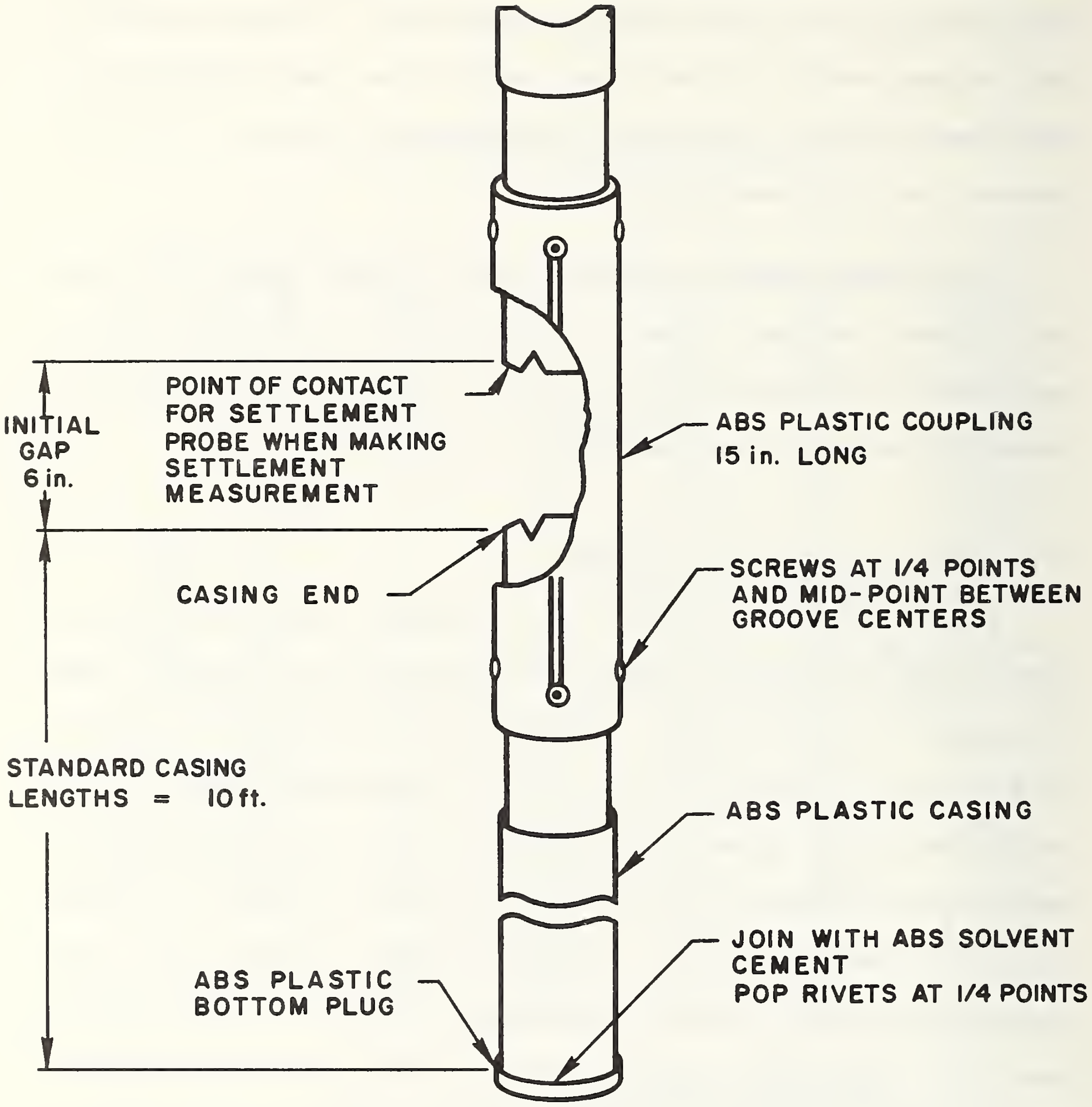


Figure 4.2: Inclinometer Casing with Telescoping Coupling

4.2.1 <u>Lateral Movements</u>

The inclinometers and couplings were used to obtain lateral and vertical movements at depth. Lateral movements were measured by lowering a SINCO Digitilt Model 50325 Sensor (Figure 4.3) to the bottom of the casing and reading a SINCO Digitilt Model 50308 Mag-tape Indicator at 2 ft. (0.6 m) interval as the sensor was pulled back to the surface. The sensor measures inclination of the casing from vertical. The grooves in the casing control the orientation of the sensor. The mag-tape indicator automatically records data on an integral cassette recorder. All readings were referenced to initial readings taken prior to construction. Displacements at depth were obtained by summing the angle changes from bottom of casing to that depth. Vertical control of tops of the inclinometer casings was maintained by periodic surveys by Stanford personnel.

The inclinometer sensor contains two servo-accelerometers mounted with the sensitive axes 90 degrees apart. One accelerometer is aligned with the sensor's wheels and the movements measured by that accelerometer is designated herein as the A axis of displacement. Displacements measured in the other direction are designated as B axis displacements. The A-component readings are used for maximum accuracy in the directions of primary interest. For the four inclinometers located beyond the tunnel periphery (A, B, D, E in Figure 4.1), movements were obtained by aligning the sensor's wheels, and thus the A axis, perpendicular to the tunnel alignment.

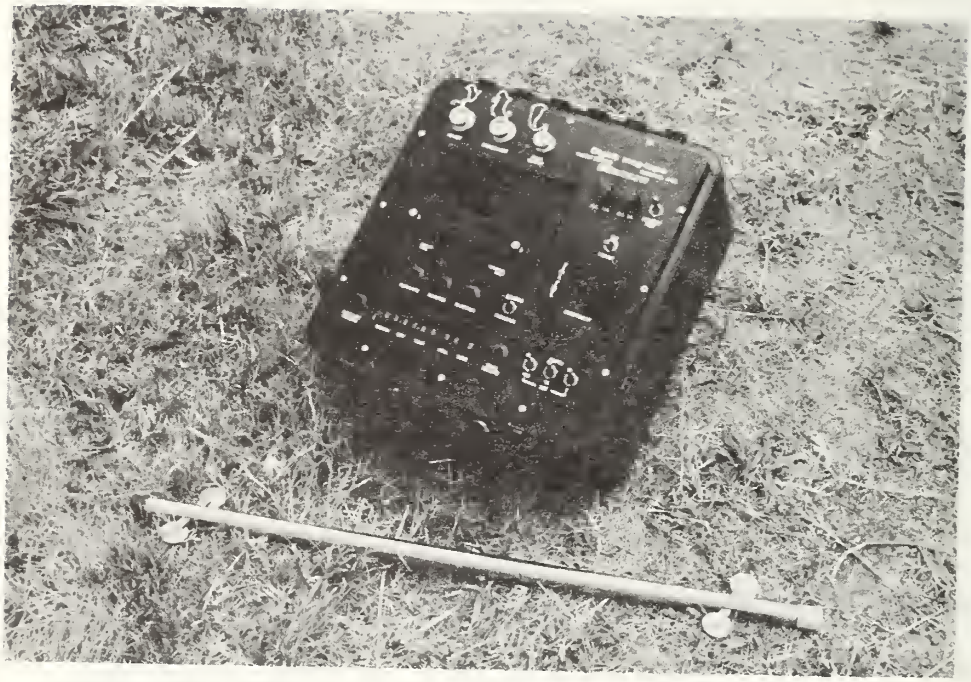


Figure 4.3: SINCO Digitilt Sensor

Maximum accuracy was insured to measure what were expected to be the maximum lateral movements of the soil in response to tunneling operations. Inclinometer C of each line was an exception to this rule since it was located in a position so as to be cut off by the advance of the shield. Its primary purpose was to measure movements parallel to the tunnel axis, and in this case the wheels of the sensor were placed in the grooves oriented along the tunnel alignment. Thus maximum accuracy was insured for measuring soil response directly in front of the advancing shield as it approached each line.

Initial measurements were performed twice at each inclinometer to insure accurate baseline data. These data also indicate that the repeatability of measurements was within ± 0.015 in. (0.3 mm).

4.2.2 Vertical Movements

Vertical movements at depth were measured using a USBR type, SINCO Settlement Probe, Model 50810 and a 100 ft. (30 m) long surveyor's chain. Figure 4.4 shows the probe with extended pawls which are used to hook on the bottom edge of each 10 ft. long piece of casing. The spring-loaded wheels shown on the figure are used to orient the probe along the grooves in the casing. In this manner, the depth to bottom of each section of casing is read from the surveyor's chain. Upon reaching the last joint, the probe is allowed to telescope together. This latches the pawls in a closed position permitting the probe to be withdrawn. Elevations of bottoms of individual casings were established by surveying the top of the casing at regular intervals. Theoretically this system yields movements defined with an accuracy of ± 0.01 ft.

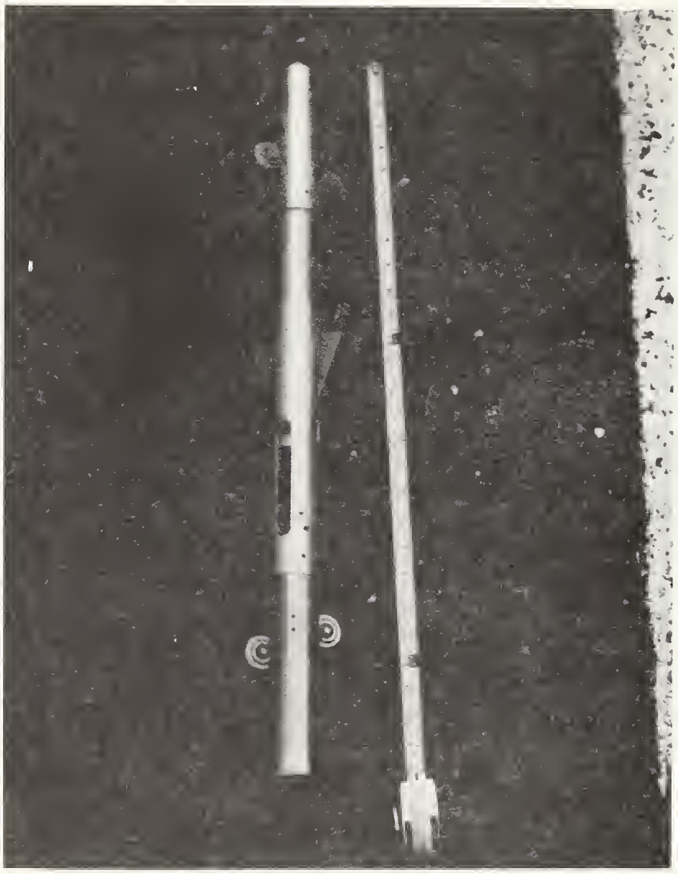


Figure 4.4: SINCO Settlement Probe

(0.003 m). However, when field variables are considered the actual accuracy is around ± 0.03 ft. (0.01 m).

4.3 READING SCHEDULE

After initial inclinometer and settlement readings were obtained prior to construction, data was collected continuously for approximately 24 hours as the tunneling operations approached and passed by each of Lines 2 through 4. These 24 hours correspond to the period when the shield face was approximately 15 ft. (4.6 m) in front to 40 ft. (12.2 m) past each instrumentation line. Special attention was given to the middle three inclinometers during the intensive reading period since they are most directly impacted by the tunneling process. After the shield cut through the middle inclinometer, no further data could be obtained. As the operations reached the latter instrumentation lines, a complete set of data (inclinometer, settlement probe and top of casing survey) was obtained from the already bypassed lines. Long term measurements were also made. Generally, data for each line was obtained during the 24 hour period as the shield passed and one, two and four weeks after the shield passed. Long term data is being presently collected and analyzed; results will be presented in a later report.

Stanford personnel also established 7 surface settlement points at Instrument Line 4 to measure the width of the settlement trough at this location. Reference points were PK nails established on the street surface. This settlement data was obtained on the same schedule as the above data. The City of San Francisco surveyed the street surface at approximately 20 ft. (6 m) intervals along the tunnel alignment. At each interval, reference points were established along the centerline of the tunnel alignment and generally 10 ft. (3 m) to both sides of the centerline. Reference points were PK nails established on the street surface. Data was obtained prior to construction and five to six months after tunneling operations were completed.

4.4 INSTRUMENT PERFORMANCE

The ground movement instruments typically performed very well. The inclinometers, in particular, yielded results which were very consistent. While the USBR-type settlement probe generally yielded consistent results when the measured movements were relatively large, they were not capable of defining small levels of displacement because of accuracy limitations. Since vertical movements for this project were the largest after shield passage, the settlement probe data could be most reasonably applied only to the latter stages of behavior. The inclinometer data however served to define effects prior to and during shield passage .

Chapter 5

OBSERVED GROUND MOVEMENTS DURING AND AFTER TUNNELING

5.1 INTRODUCTION

Active tunneling with the EPB shield began in January, 1981, and was completed by June, 1981. Monitoring of the instruments commenced in February, prior to the passage of the shield through Line 2. Readings for all of the lines through June, 1981, are covered in this report. The monitoring effort is continuing into 1982 to keep track of time-dependent trends; this information will be made available in a subsequent report.

Key results related to ground movements and shield behavior are presented in this chapter. For purposes of documentation and possible later study, a file of significant inclinometer data is compiled in Appendix A.

5.2 SHIELD PERFORMANCE IN VICINITY OF INSTRUMENTATION LINES 2, 3, & 4

Before examining ground movements at the individual instrumentation lines, it is useful to consider the shield performance characteristics as it passed each one. The parameters of concern are the orientation of the shield and the measured earth pressure inside the bulkhead. The orientation of the shield is important since it influences the amount, as well as the distribution of ground movements. If the shield is moving exactly horizontally, then ground movements should arise only as a result of (1) displacements at the face of the shield; and, (2) displacements into the tail void created by the thickness of the shield, the space occupied by the tail seals, and the space allowed for construction tolerances to erect the liner. The theoretical tail void gap for the N-2 shield in a perfectly horizontal orientation is 3.0 in. (7.6 cm). However, should the shield be pitched upwards, it will plow through the soil, creating an additional space in the tail void area which will lead to extra soil movement.

This situation is depicted in Figure 5.1; on an upward pitch the shield "plows" through the soil, creating an extra dimension for the void from the springline to the crown. Below the shield, the soil is compressed by the tail of the shield, which drags on the bottom. This type of behavior is common to either a conventional or an advanced shield and creates larger surface settlements (see Hansmire and Cording, 1975) than if the shield moves horizontally. It is often necessary to operate a shield at a slightly upward pitch to overcome the natural diving tendency caused by the shield weight.

The magnitude of the pressure measured by the earth pressure cell attached to the shield bulkhead is not significant in itself. This parameter is not the actual pressure at the face of the shield, but rather is an index as to whether the spoil retaining area ahead of the bulkhead is full or not. It also can indicate if a shoving action being exerted on the soil by the shield if the pressures are unusually high. In such case the soil is not being removed quickly enough by the screw auger relative to the rate at which it is being brought in by the cutterhead.

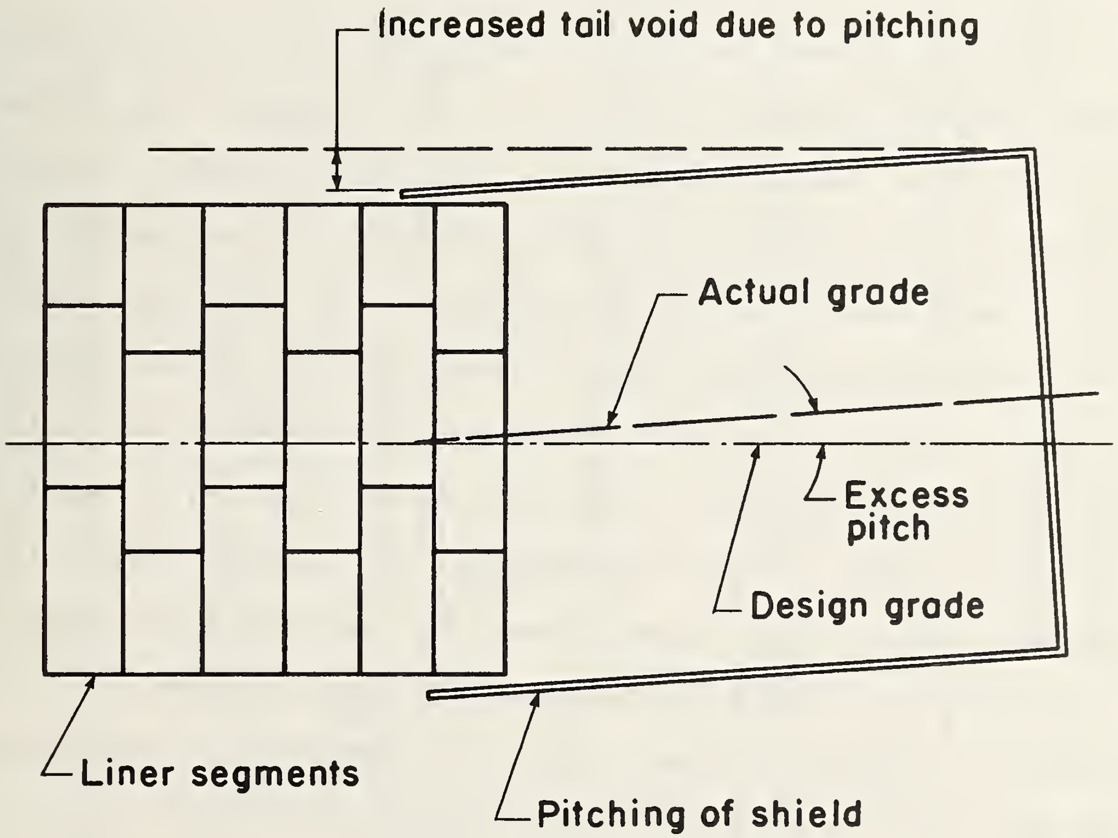


Figure 5.1: Pitch of Shield

Both the pitch and the bulkhead earth pressure are useful to determine if the shield operations could account for any of the observed differences in performance between Lines 2, 3, and 4. Should the pitch or earth pressure be different, then the magnitude and distribution of ground movements should not be the same even if all other conditions are the same.

Figure 5.2 presents the pitch of the shield and the bulkhead earth pressure in the vicinity of Lines 2, 3, and 4, plotted against the position of the face of the shield. The pitch shows the greatest variation in the area around Line 2. As the shield approached this line, it was pitched as much as 3% above grade. Just before passing the line, the pitch was reduced to 1% and this orientation was held until the face was 10 ft. (3 m) past, whereupon it was reduced to about 0.4%. In crossing Lines 3 and 4 the pitch was more consistent with an average value of only 0.15%. Hansmire and Cording (1975) provide a formula for estimating the tail void volume produced by the pitch of the shield. For pitches of 3, 1 and 0.15% the extra tail void volumes are 4.7, 1.6 and 0.2 $ft.^3/ft$ (0.44, 0.15 and 0.02 m^3/m), respectively. Considering that the theoretical on-line tail void for the EPB shield is 9.2 $ft.^3/ft.$ (0.85 m^3/m) the pitch can be seen to be a potentially important parameter. Because the pitch at Line 2 is larger than at Lines 3 and 4, some differences in behavior can be expected just due to this factor.

The bulkhead earth pressure is presented in Figure 5.2 in nondimensionalized form, obtained by dividing the actual pressure by the maximum observed pressure during the tunneling process. Generally the

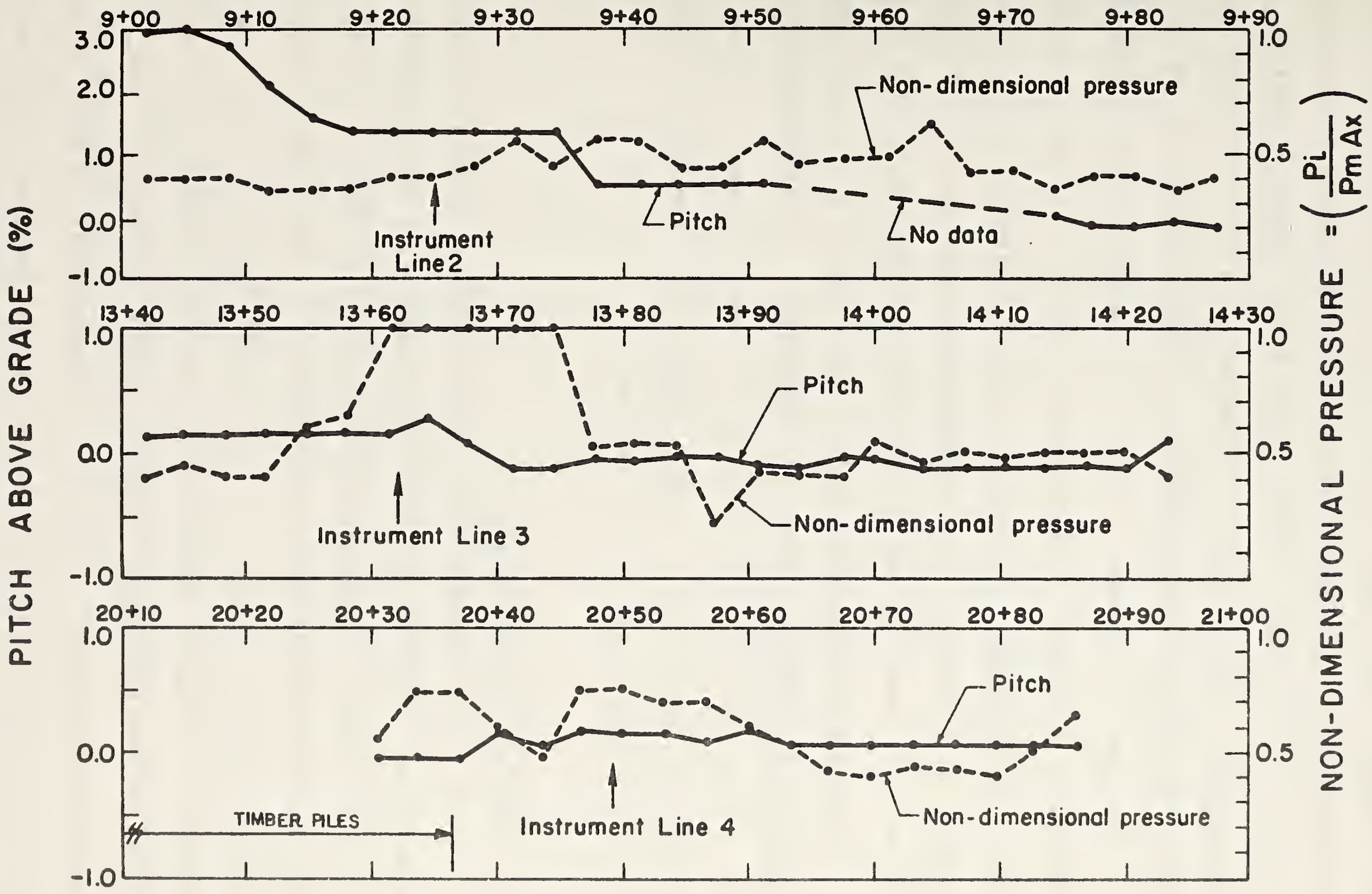


Figure 5.2: Pitch and Bulkhead Pressure vs. Position of Shield Face

nondimensionalized pressure fluctuates around a value of 0.5 for Line 2. Approaching Line 3 it was as low as 0.4, but about 10 ft. (3 m) in front of the line, when some of the wooden piles were encountered, the pressure began increasing until it reached a value of 1.0 just before Line 3. This value was maintained for a shield movement of 10 ft. (3 m), and then it dropped to about 0.5 again. There were also fluctuations of pressure around Line 4 where the shield was still encountering the piles. At Line 4 it reached a value of 0.75. Although not shown in Figure 5.2, the bulkhead pressures outside of the areas with the piles generally ranged from 0.4 to 0.5, as was the case at Line 2. The high bulkhead pressures where the piles were being cut suggest that the pile fragments clogged the screw auger, and prevented the soil from moving freely through the spoils retaining area. Since the pressures at Lines 3 and 4 were higher than average, a somewhat specialized behavior may be expected there.

5.3 GROUND MOVEMENTS AT LINE 2

During and following the EPB shield passage, vertical and lateral movements were induced in the ground and measured by the instrumentation at Line 2. However, because the monitoring system could measure lateral movements more accurately than vertical, the lateral movements demonstrated the effects of the shield tunneling most clearly, especially in regard to the small magnitude changes which occurred during different stages of shield advance. Thus, in this, and subsequent discussion, the lateral movements will be used to define detailed trends of behavior during shield passage. The vertical movements will primarily be introduced for final stage response where the movements were large enough to be well beyond the range of accuracy of the measurement technique.

Distances of the inclinometer positions from the tunnel centerline are indicated in Table 5.1.

TABLE 5.1

Position of Inclinometers at Line 2

Inclinometer	Distance from Tunnel* Ft.	Distance from Tunnel Periphery at Springline Ft.
2A	+ 23	17
2B	+ 12	6
2C	+ 2	-
2D	- 8	2
2E	- 18	12

* + = Right of centerline looking in direction of tunneling

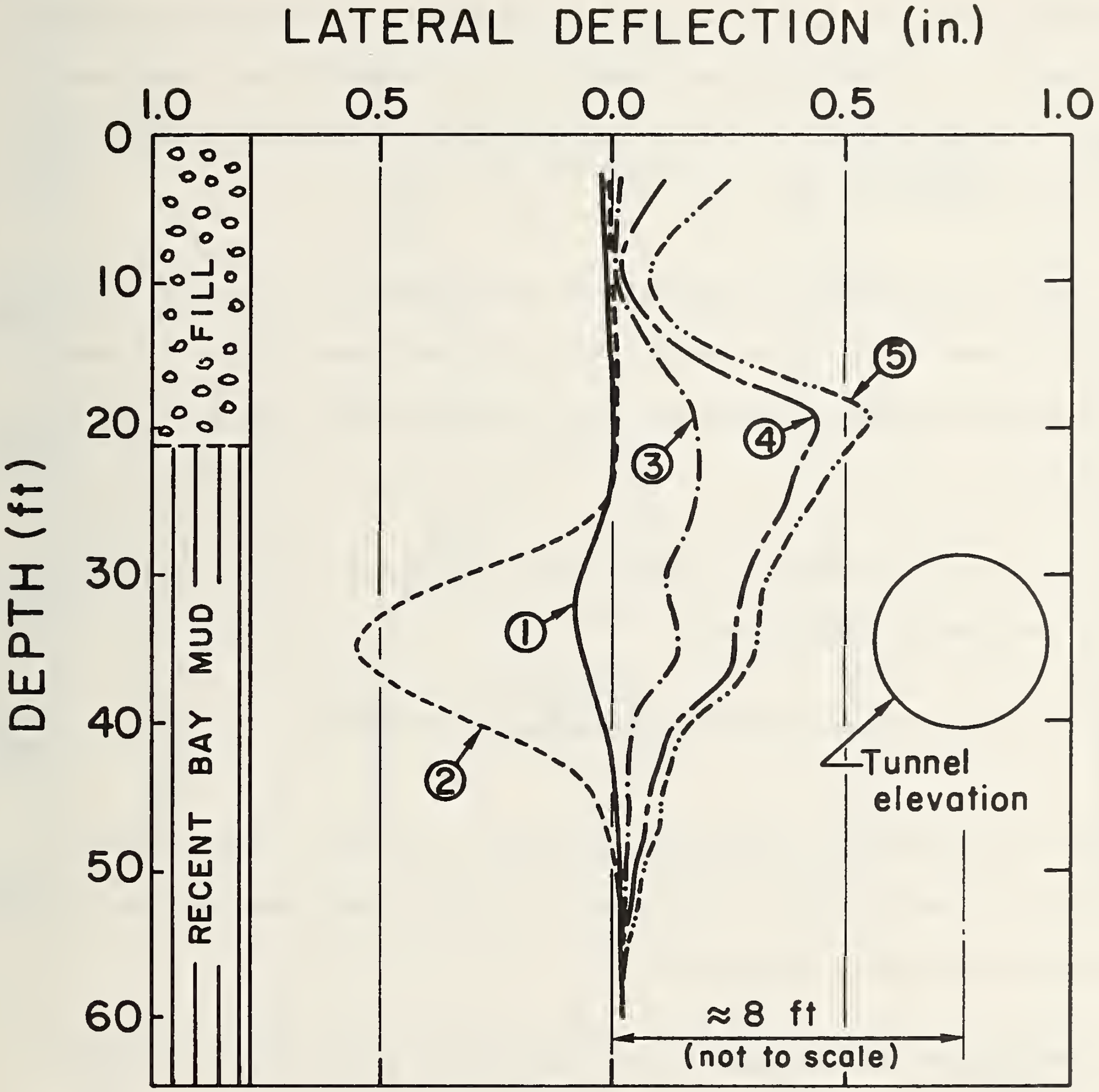
- = Left of centerline looking in direction of tunneling

The nonsymmetry apparent in the positions was produced primarily by the shift made in the tunnel alignment after the instruments were in the ground. Inclinometer 2D ended up only 2 ft. (0.6 m) from the tunnel periphery at the springline, and it yielded the largest measured movements. The next closest to the tunnel was Inclinometer 2B which was 6 ft. (1.8 m) from the tunnel at the springline.

5.3.1 Lateral Movements Perpendicular To The Tunnel Axis

The observed lateral displacements for Inclinometer 2D perpendicular to the tunnel axis for a series of different positions of the shield are given in Figure 5.3. The first measurements were taken when the shield face was 4 ft. (1.2 m) from Line 2. At this time, there was a small, but measurable, lateral displacement away from the shield in the Bay Mud centered about the elevation of the tunnel springline. When the face had advanced three ft. (0.9 m) past Line 2, the heave effect was more pronounced, with the maximum displacement reaching 0.6 in. (1.5 cm) at the level of the springline. During the development of these movements in the Bay Mud essentially no displacement occurred in the overlying fill.

The next measurements shown for Inclinometer 2D in Figure 5.3 were taken with the shield face 22 ft. (6.7 m) past Line 2. This means that the tail of the 16 ft. (4.8 m) long shield had also just passed, and the impact of the tail void was being felt. The inclinometer shows a reversal of the previous trends of movement away from the shield. These new inward displacements occurred well before any grouting could be applied through the liner so as to fill the tail void. They were sufficient to lead to a net position of the inclinometer towards the tunnel, and occurred in both the Bay Mud around the tunnel as well as the rubble fill above the crown. Maximum net movements at this stage reached about 0.2 in. (0.5 cm). Inclinometer readings taken at 5 days and 30 days after shield passsage showed continuing movements towards the tunnel. Maximum values of net inward lateral movements of 0.5 in. (1.3 cm) were recorded in the lower portions of the rubble fill. The



① Shield face 4 ft from line ; Tail void 20 ft from line
② Shield face 3 ft past line ; Tail void 13 ft from line
③ Shield face 22 ft past line ; Tail void 6 ft past line
④ 5 days after shield passage
⑤ 30 days after shield passage

Figure 5.3: Lateral Deflections (A Axis) of Inclinometer S2D Before and After Shield Passage

maximum inward increment of movement after the initial heaving process amounted to 0.8 in. (2.0 cm), and this occurred near the springline of the tunnel. The time-dependent nature of the response may have been caused by consolidation effects in the Bay Muds or a delayed response to the stress changes in the fill.

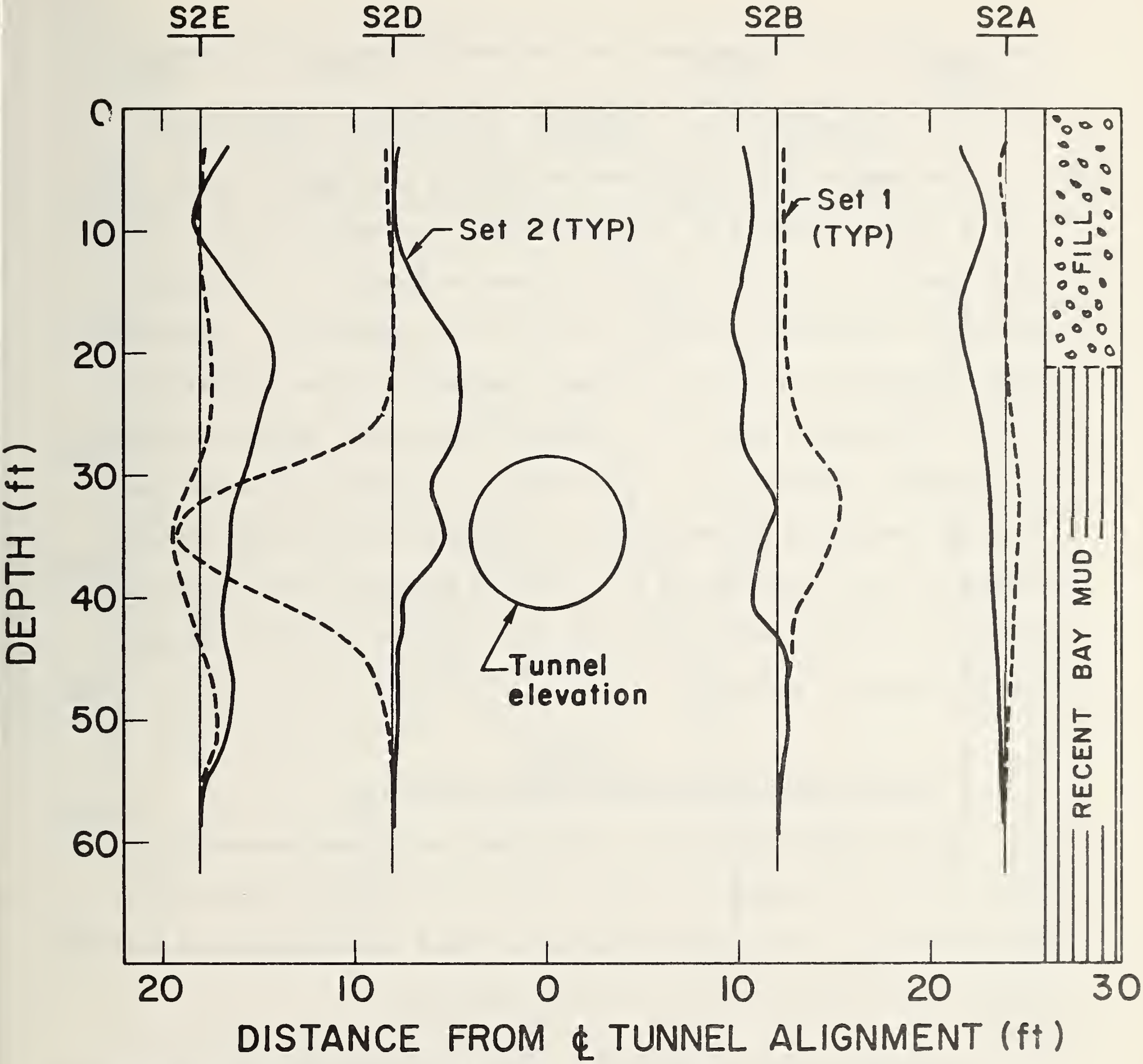
Before discussing the behavior at Inclinometer 2D further, it is useful to examine that measured by the other inclinometers. In Figure 5.4 the observed lateral movements for Inclinometers 2A, 2B, 2D, and 2E are given for:

1. a condition where the face of the shield but not the tail void has passed Line 2; and,

2. a condition where the tail void has passed Line 2.

Not all the readings were taken at exactly the same shield location because it required some 20 minutes to read an inclinometer, during which time the shield was moving.

The general trends observed for Inclinometers 2A, 2B, and 2E are similar to those described for 2D except that the magnitudes of the displacements are not as large. The differences in the displacement magnitudes generally correspond with the distance of the inclinometer from the tunnel, i.e., the farther away the location, the smaller the movements. All the inclinometers displayed an initial movement away from the tunnel as the shield face passed by, with this effect most pronounced at the springline elevation and confined primarily to the Bay



Distance of shield face past line 2 at time of measurement		
	Set 1	Set 2
S2A	9 ft	28 ft
S2B	1 ft	19 ft
S2D	3 ft	22 ft
S2E	7 ft	38 ft

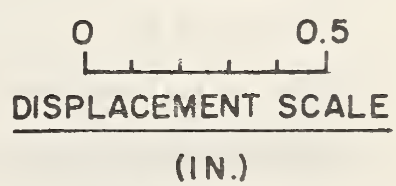


Figure 5.4: Lateral Deflections (A Axis) at Line 2 After Shield Passage and Tail Void Passage

Mud. After tail void passage, the net movements were inward and this effect involved both the Bay Mud and the rubble fill.

Final displacements, measured some 30 days after shield passage of Line 2 for Inclinometers 2A, 2B, 2D, and 2E, are shown in Figure 5.5. Again there is a strong similarity in behavior trends. In all cases, the inward movements started by the tail void passage are increased relative to those in Figure 5.4, and a maximum net value consistently occurs at a depth of about 20 ft. (6.1 m), just above the crown and at the bottom of the rubble fill. This effect is probably due to the fact that the EPB shield was traveling on a slightly upward pitch when it passed by Line 2 (see Figure 5.2). Such an alignment would led to the opening of a larger than normal tail void, especially in the area from the springline to the crown.

5.3.2 <u>Lateral Movements Parallel to the Tunnel Axis</u>

Lateral movements parallel to the tunnel axis were measured using the B-Axis of Inclinometers 2A, 2B, 2D, and 2E and the A Axis of the Inclinometer 2C. As noted in Chapter 4, the A axis measurements are typically more accurate than the B axis measurements.

The movements monitored by the A axis of Inclinometer 2C were very small, never exceeding 0.1 in. (0.3 cm) even with the shield face as close as 3.5 ft. (1.1 m) from the inclinometer casing (see Figure 5.6). In the initial response to the shield approach, the inclinometer moved slightly towards the shield, but the last readings taken, when the shield was 3.5 ft. (1.1 m) from the casing, show that a slight bulge occurred away from the shield.

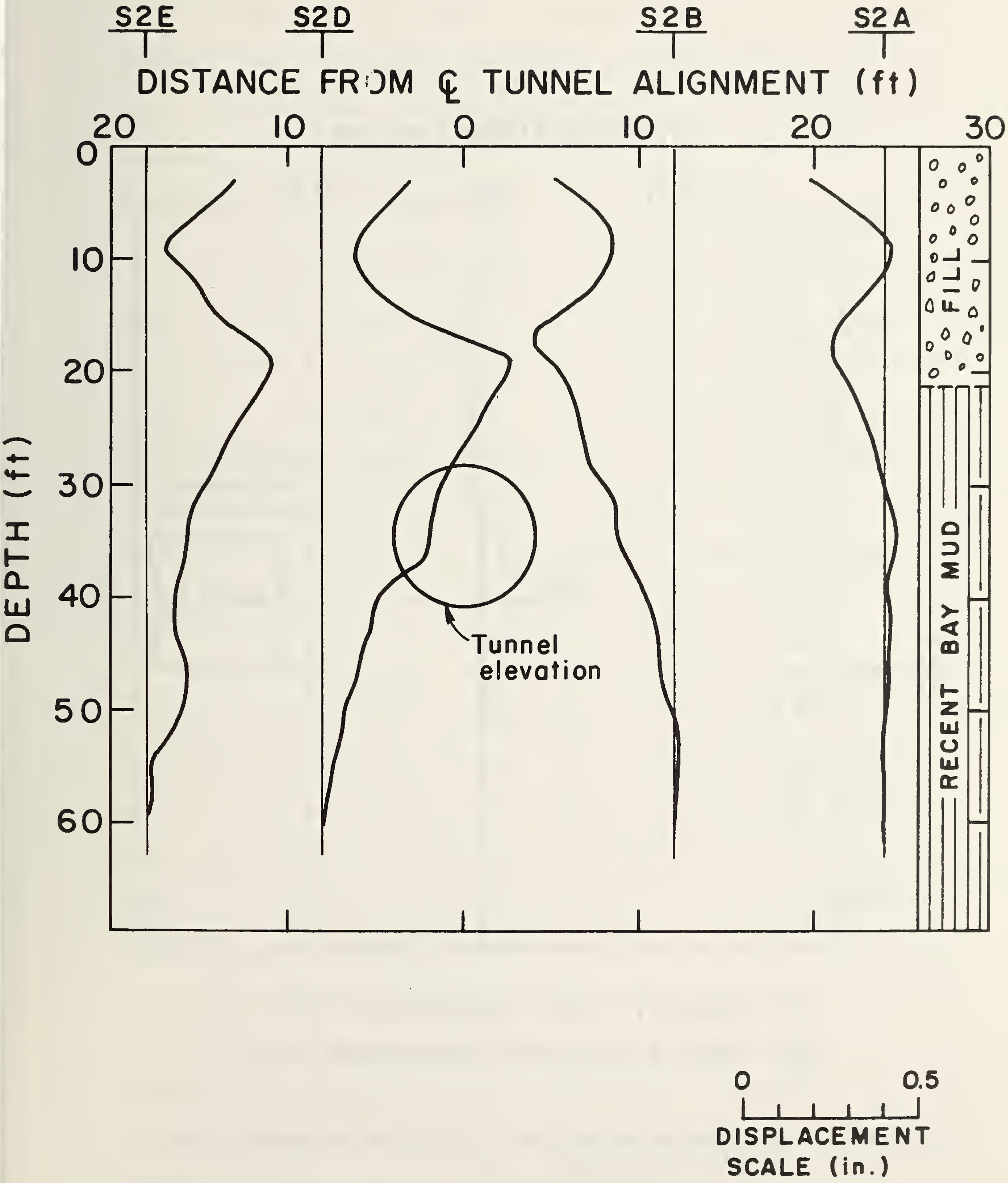


Figure 5.5: Lateral Deflections (A Axis) at Line 2 30 Days After Shield Passage

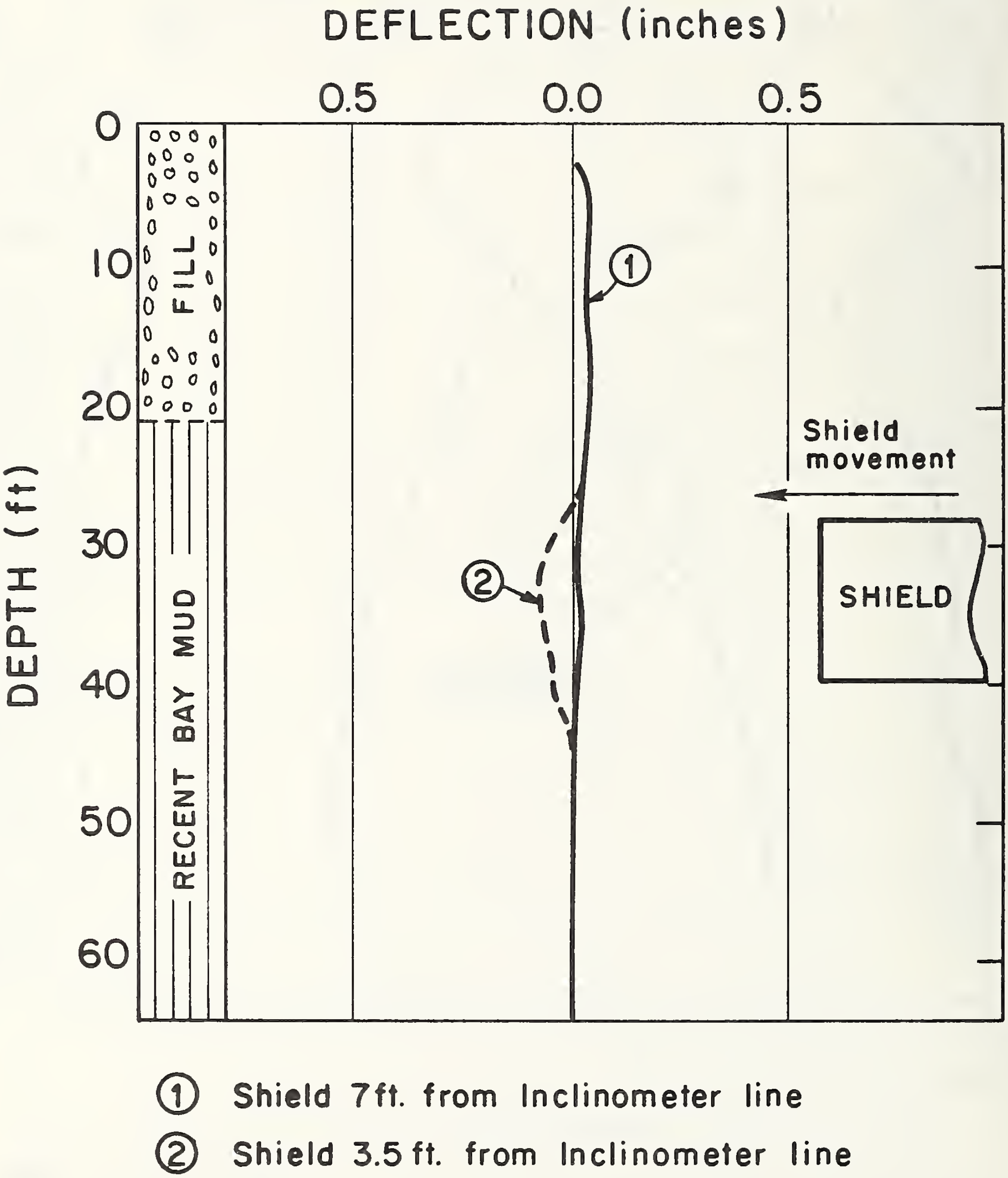
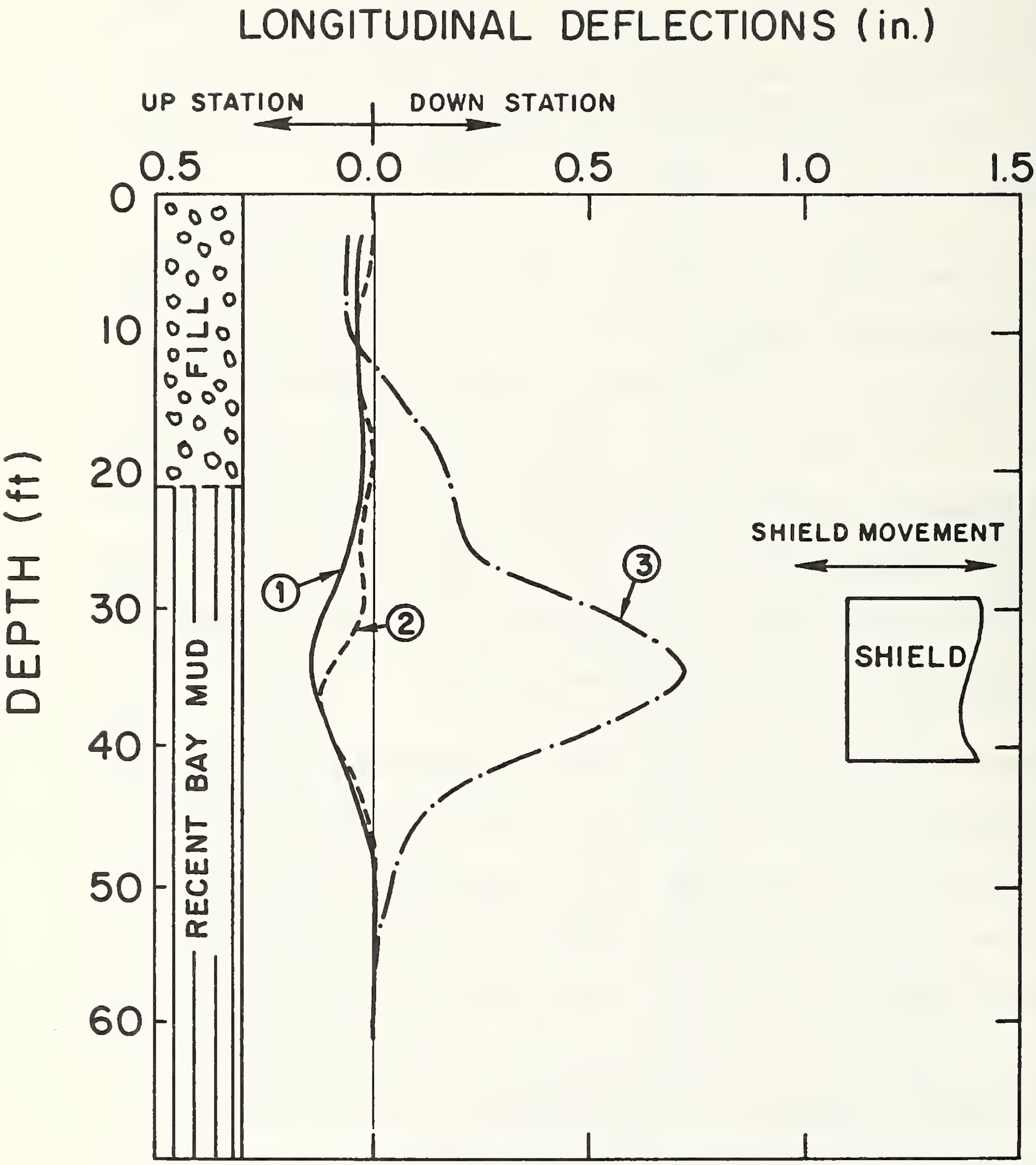


Figure 5.6: Longitudinal Deflections (A Axis) of Inclinometer S2C as Shield Approaches

The displacements at this time were completely confined to the Bay Mud, as also shown by the other inclinometers perpendicular to the tunnel axis. Interestingly, accompanying the movement away from the shield, the water level in Inclinometer 2C was noted to rise rapidly as the shield moved very near the casing.

Small longitudinal movements away from the shield as it approached Line 2 were also measured by the B-Axis of Inclinometer 2D. These are given in Figure 5.7 along with a set of readings taken when the shield was 22 ft. (6.7 m) past Line 2. The last data set show that the soil reversed the early direction of movement and ended up displacing towards the entrance portal by as much as 0.7 in. (1.8 cm) at about the level of the springline of the tunnel. This behavior presumably reflects the inward movements of the soil towards the tail void.

B-Axis readings from the other inclinometers demonstrated a response consistent with that observed at 2D, although the movements were generally smaller since they are not located as close to the shield as 2D. The fact that the B-Axis readings from the inclinometers were consistent suggests that this axis was accurate enough to define the general soil response parallel to the shield. A picture of the combined lateral movements measured at all inclinometers at the elevation of the tunnel springline for a series of shield positions is given in Figure 5.8. The results show that the Bay Mud heaves longitudinally and laterally away from the face of the shield as it approaches. With the arrival of the tail void, the soil begins to displace both longitudinally and laterally towards the shield.



① Shield face 4 ft from line; Tail void 20 ft from line

② Shield face 3 ft past line; Tail void 13 ft from line

③ Shield face 22 ft past line; Tail void 6 ft past line

Figure 5.7: Longitudinal Deflections (B Axis) of Inclinometer S2D Before and After Shield Passage

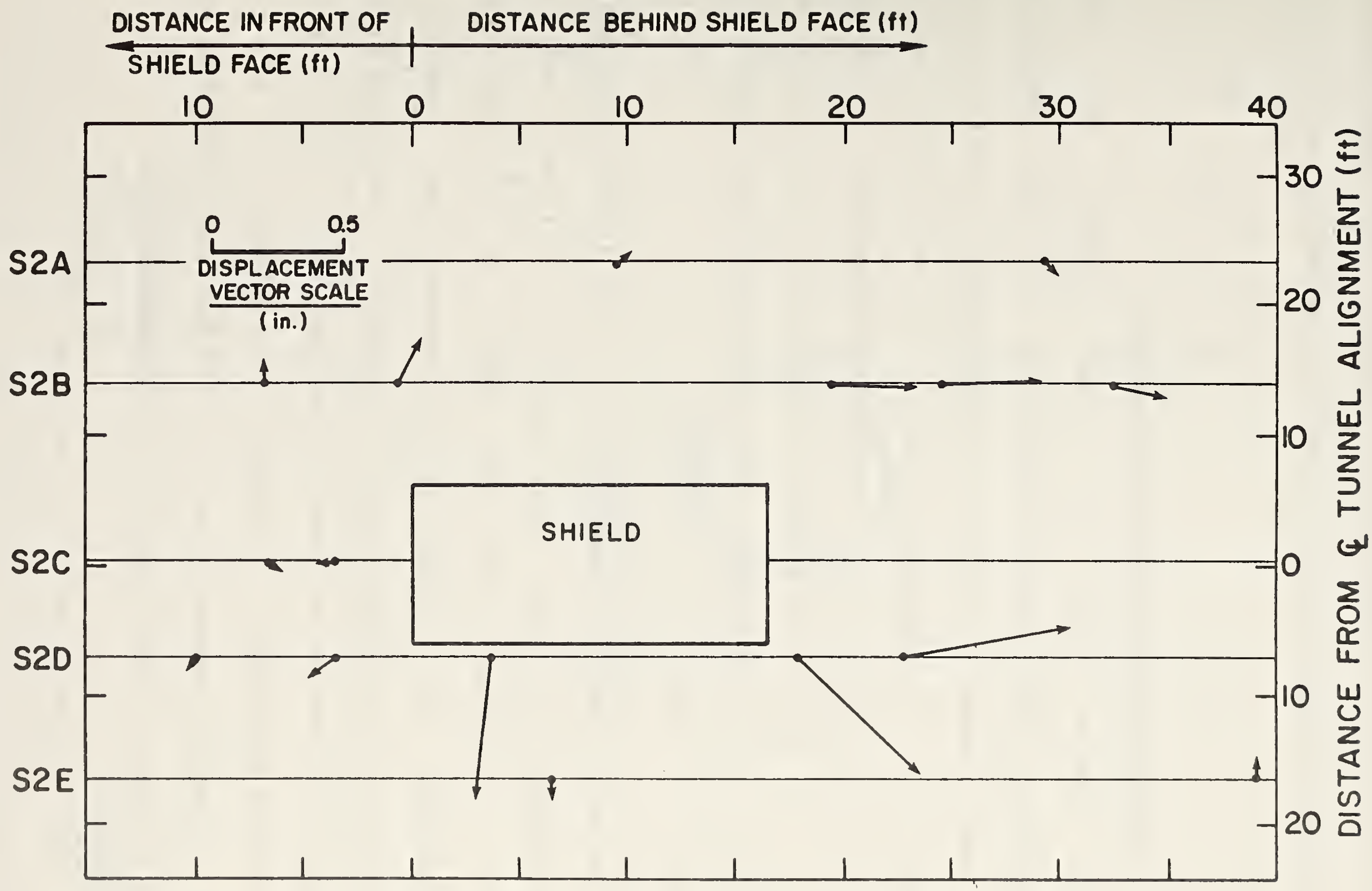


Figure 5.8: Horizontal Displacement Vectors at Springline, Line 2

The final net movements are inwards towards the tunnel and away from the direction of the shield movement. The tail void effect eventually dominated the response and overrode the initial small outward heaves at Line 2.

5.3.3 Combined Lateral and Vertical Movements

The combined lateral and vertical movements in a plane perpendicular to the tunnel axis measured at Line 2 at a date 30 days after shield passage in the form of a displacement vector plot are given in Figure 5.9. The vectors are drawn for the 20 subsurface locations where both lateral and vertical movements were measured in Inclinometers 2A, 2B, 2D, and 2E. Each vector is defined by an initial point at a condition of zero displacement, and a final point using the measured movements 30 days after shield passage.

The general pattern of movements is for points above the tunnel to move downwards and inwards toward the crown, while those below move upwards and inwards towards the invert. As with the lateral displacements, the largest vertical movements were measured at Inclinometer 2D, the nearest one to the tunnel. The maximum measured settlement occurred in the rubble fill and was 1.5 in. (3.8 cm), a value considerably larger than the largest lateral movement of 0.5 in. (1.3 cm). In the inclinometers other than 2D, the lateral movements tended to be as large or larger than the vertical movements.

Three surface survey points were monitored by the City of San Francisco at Line 2, one on the centerline of the tunnel and one 10 ft.

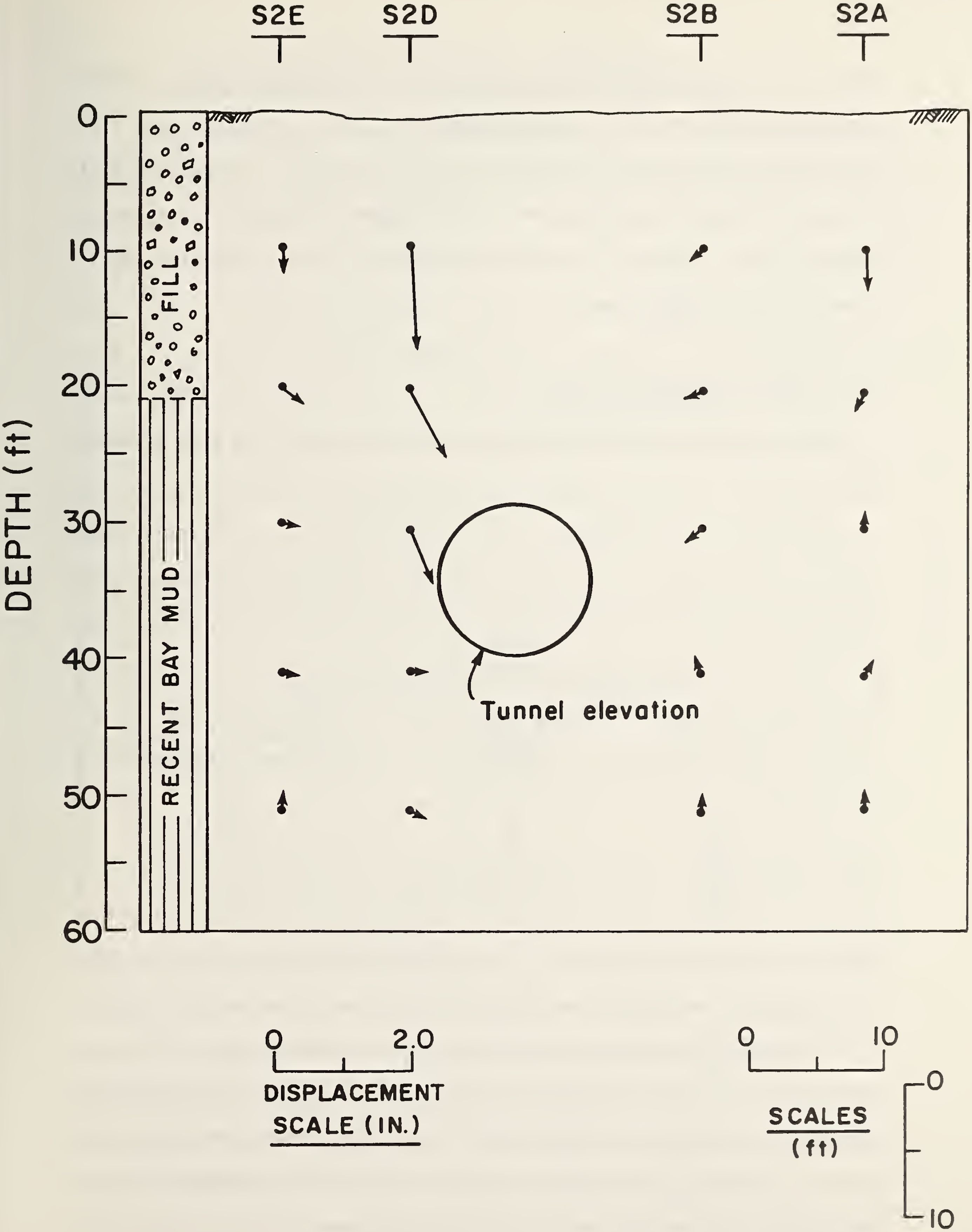


Figure 5.9: Transverse Displacement Vectors 30 Days After Shield Passage, Line 2

(3 m) on either side of the centerline. Five months after shield passage the settlement at the centerline was 1.8 in. (4.6 cm) while on either side the settlement reached 1.7 in. (4.3 cm). These levels of vertical movement are consistent with those observed in the upper portions of the rubble fill at the location of Inclinometer 2D (8 ft. from the tunnel centerline).

5.4 GROUND MOVEMENTS AT LINE 3

The positions of the inclinometer casings at Line 3 are given in Table 5.2.

TABLE 5.2

Positions of Inclinometers at Line 3

Inclinometer	Distance from Tunnel* Ft.	Distance from Tunnel Periphery at Springline Ft.
3A	18	12
3B	11	5
3C	2	-
3D	- 9	3
3E	-17	11

* + = Right of centerline looking in direction of tunneling

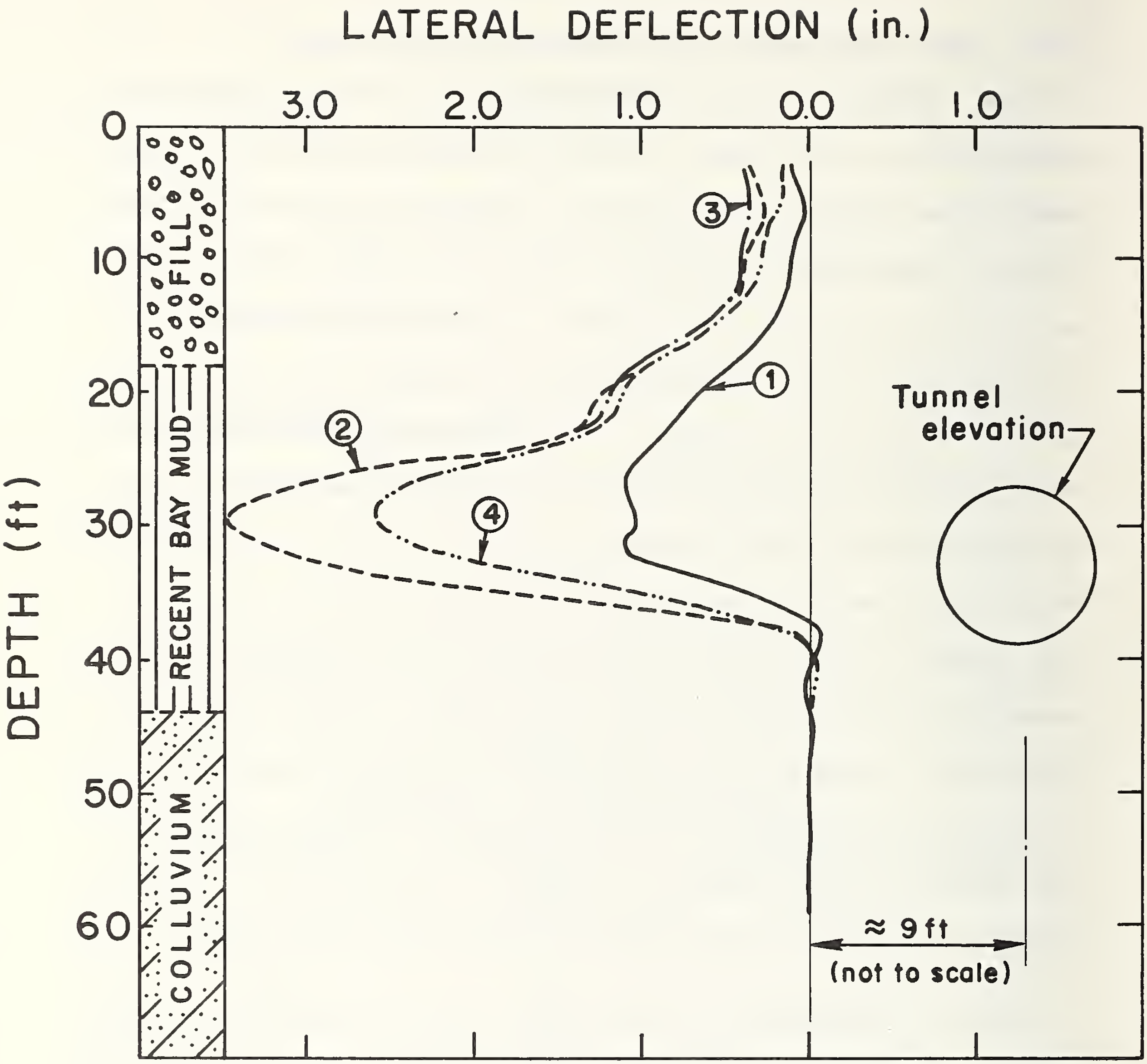
- = Left of centerline looking in direction of tunneling

As at Line 2, the inclinometer locations are not distributed exactly symmetrically around the tunnel centerline. Inclinometer 3D is as close as 3 ft. (0.9 m) to the tunnel periphery while on the opposite side of the tunnel, Inclinometer 3B is 5 ft. (1.5 m) away.

5.4.1 Lateral Movements Perpendicular to the Tunnel Axis

The observed lateral displacements for Inclinometer 3D perpendicular to the tunnel axis for a series of different shield positions are given in Figure 5.10. It is obvious at the earliest stages of measurement that the soft Bay Mud is being strongly forced away from the tunnel at Line 3. When the shield face is still 3 ft. (0.9 m) from the line, the outward lateral movement at the elevation of the springline is 1 in. (2.5 cm). The displacements reach a maximum of 3.5 in. (8.9 cm) before the tail of the shield passed Line 3. While the most prominent lateral movements are in the Bay Mud, they extend in smaller magnitudes into the rubble fill, and essentially to the surface. Upon passage of the tail of the shield, the incremental lateral movements are into the tail void. As at Line 2, these displacements occurred well before any tail void grouting could be accomplished. At Line 3, the inward movements were however, small relative to the heaves which preceded them, leaving Inclinometer 3D displaced away from the tunnel by as much as 2.6 in. (6.6 cm) 30 days after shield passage.

Lateral displacements for Inclinometers 3A, 3B, 3D and 3E are shown in Figure 5.11 for situations where first the shield face, and then the tail of the shield, are just past Line 3. It is apparent that the large lateral heaves observed at Inclinometer 3D were not an exception. Maximum outward movements just as the shield face passed reached 3.0 in. (7.6 cm) at Inclinometer 3B (5 ft. from tunnel) and 1.2 in. (3 cm) at Inclinometer 3A (12 ft. from tunnel). After the inward movements caused by the tail void closure, all of the inclinometers still show significant net outward displacements from the tunnel. Note that the



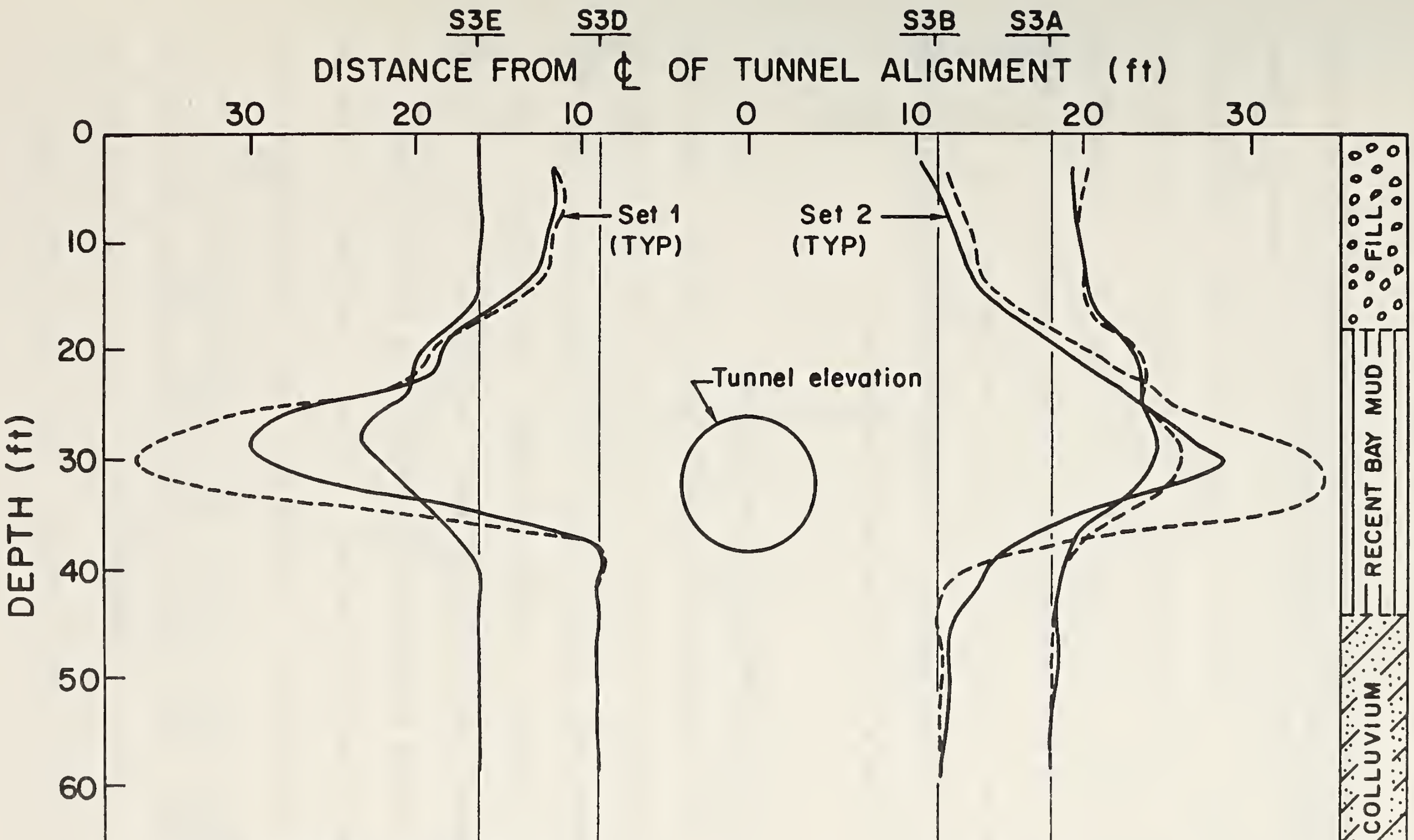
① Shield face 3 ft from line; Tail void 19 ft from line

② Shield face 15 ft past line; Tail void 1 ft from line

③ Shield face 32 ft past line; Tail void 16 ft past line

④ 30 days after shield passage

Figure 5.10: Lateral Deflections (A Axis) of Inclinometer S3D Before and After Shield Passage



	Distance of shield face past line 3 at time of measurement	
	Set 1	Set 2
S3A	8.7 ft	34.6 ft
S3B	12 ft	21.4 ft
S3D	15.3 ft	31.3 ft
S3E	NO DATA	36.2 ft

Figure 5.11: Lateral Deflections (A Axis) at Line 3 After Shield Passage and Tail Void Passage

immediate inward movement effects in Figure 5.11 are confined to the Bay Mud soils and little, if any, movement has occurred in the rubble fill.

The inward movements begun by the tail void passage increased over the next 30 days. The 30 day inclinometer positions, shown in Figure 5.12, illustrate that, in spite of the additional inward displacements, the net positions in the Bay Muds are still sharply away from the tunnel. In the rubble fill there are some small net displacements toward the tunnel at this stage near the ground surface. In this respect the behavior at Line 3 is similar to that at Line 2. A subsequent section of this report will contrast the two behaviors and explain reasons for the similarities and differences.

5.4.2 Lateral Movements Parallel to Tunnel Axis

The large outward movements at Line 3 away from the tunnel that were exhibited perpendicular to the axis were also reflected parallel to the axis. Readings made for the center Inclinometer, 3C, when the shield was 10 ft. (3 m) away from the instrument line show up to 1.5 in. (3.8 cm) of movement away from the face (see Figure 5.13). These displacements are almost exclusively confined to the Bay Mud layer, and concentrate around the shield location. By way of contrast, it may be remembered that Inclinometer 2C showed very small displacements even when the shield was 3.5 ft. (1.1 m) from the shield face. The water level in Inclinometer 3C was also more strongly influenced than that at Inclinometer 2C. When the shield was still 10 ft. (3 m) from the 3C casing the water actually flowed over the top for a period of about ten minutes.

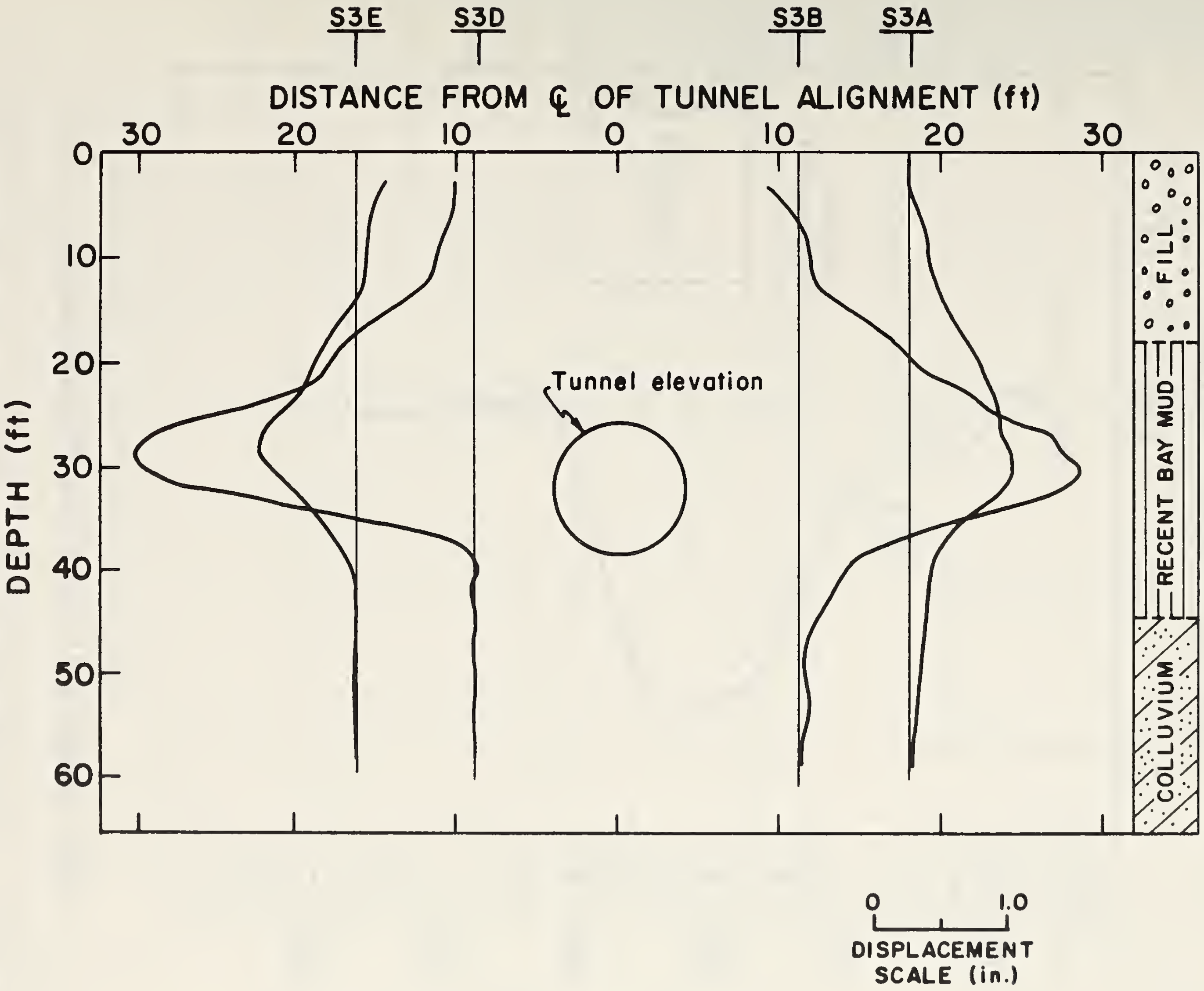
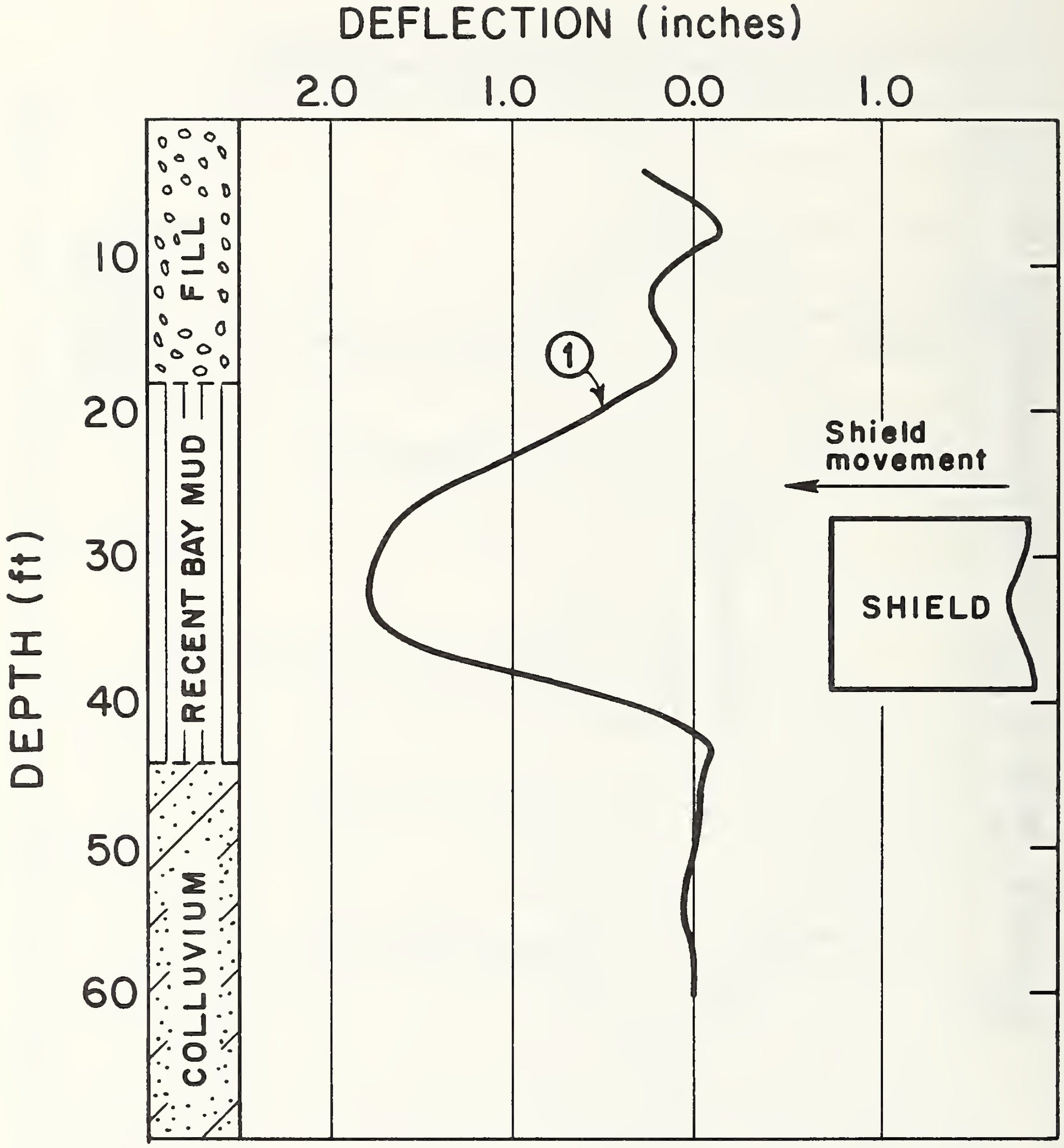


Figure 5.12: Lateral Deflections (A Axis) at Line 3 30 Days After Shield Passage



① Shield 11 ft from Inclinometer Line

Figure 5.13: Longitudinal Deflections (A Axis) of Inclinometer S3C as Shield Approaches

B-Axis measurements on the other inclinometers confirm the large longitudinal movements observed at Inclinometer 3C. Figure 5.14 shows longitudinal movements recorded at Inclinometer 3D for various shield face locations. With the face 3 ft. (0.9 m) from Line 3, the longitudinal displacements reach values as high as 2.0 in. (5.0 cm) in the Bay Muds. Subsequent readings, reflecting the effects of the tail void, show a reversal in the outward movement trend. Even so, the net displacement is in the direction of shield movement, the influence of the tail void notwithstanding.

A view of the combined lateral and longitudinal movements at the springline elevation for a series of shield positions is provided in Figure 5.15. As the shield advances, the soil is initially moved outward and away from the tunnel axis. Subsequently, as the tail void approaches the line, the soil moves inward and against the direction of the shield advance. However, the net displacement pattern still reflects a heaved configuration and the fact that the soil remained pushed beyond its initial position in the vicinity of the tunnel. This pattern of displacement is markedly different than that observed for Line 2 (Figure 5.8), and reflects the difference in shield operation at the two lines.

5.4.3 Combined Lateral and Vertical Movements

Displacement vectors for vertical and lateral movements in the plane perpendicular to the tunnel axis 30 days after shield passage are shown in Figure 5.16. Interestingly, in spite of the large lateral movements away from the tunnel, all of the vertical displacements above

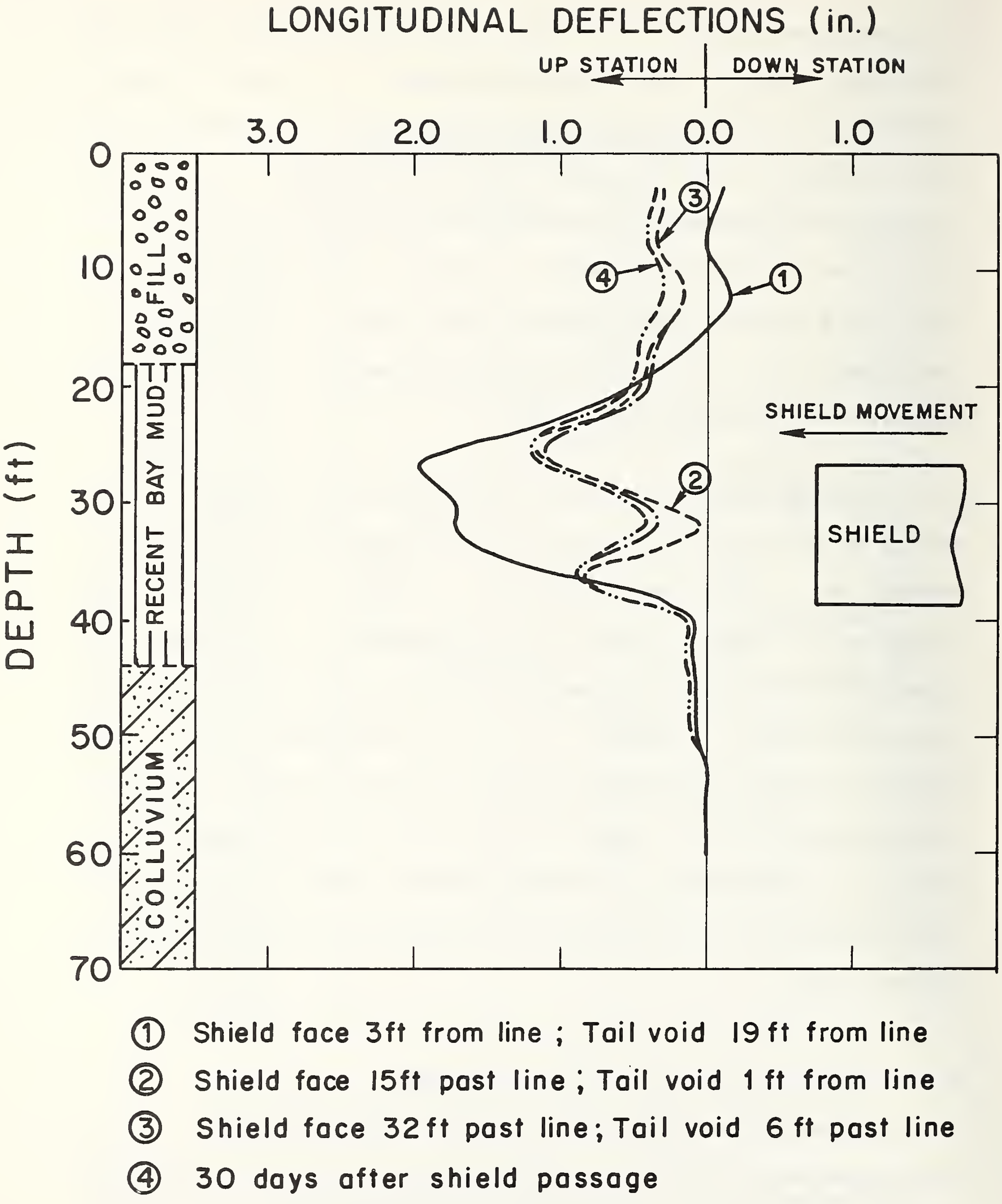


Figure 5.14: Longitudinal Deflections (B • Axis) of Inclinometer S3D Before and After Shield Passage

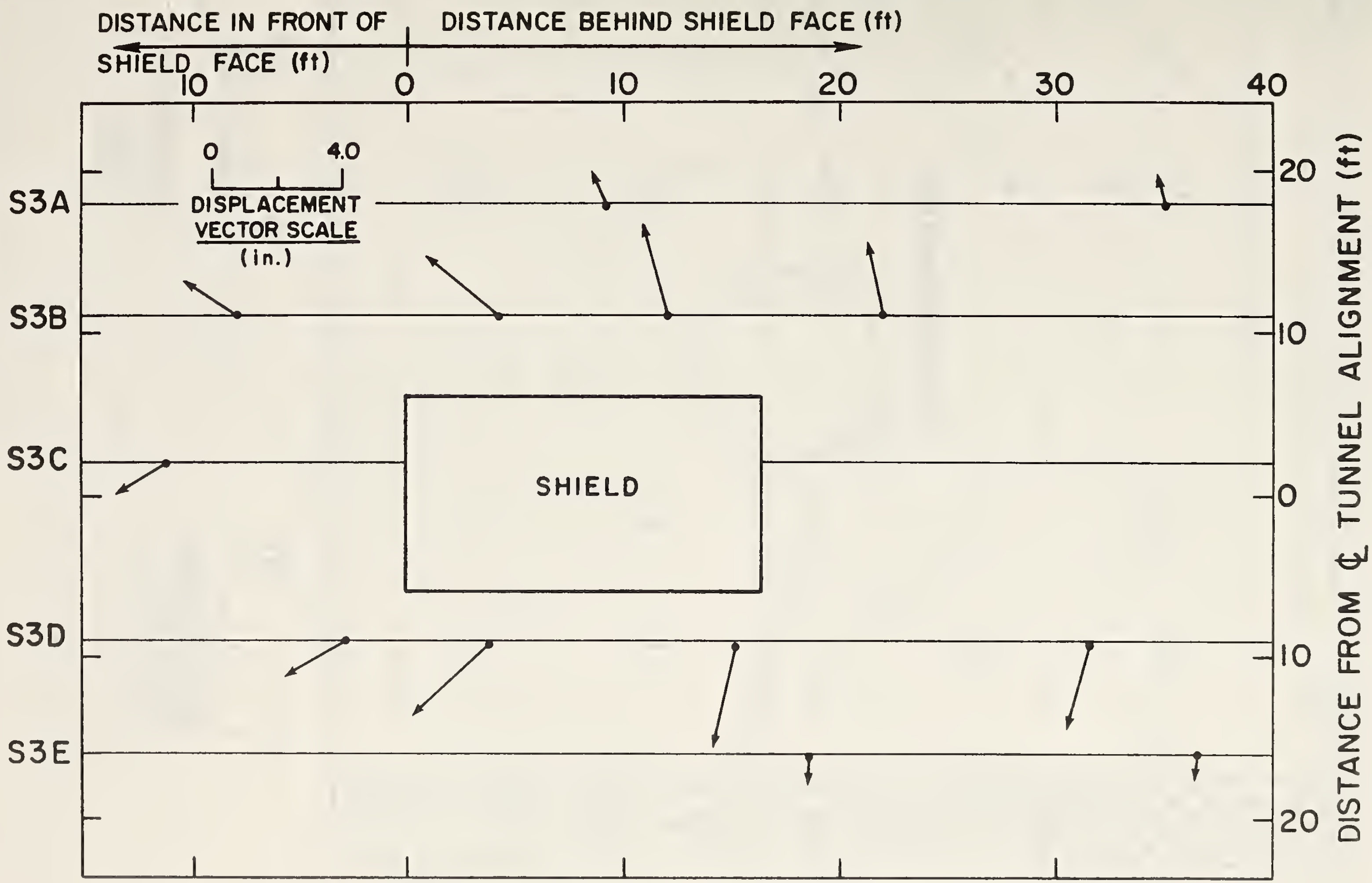


Figure 5.15: Horizontal Displacement Vectors at Springline, Line 3

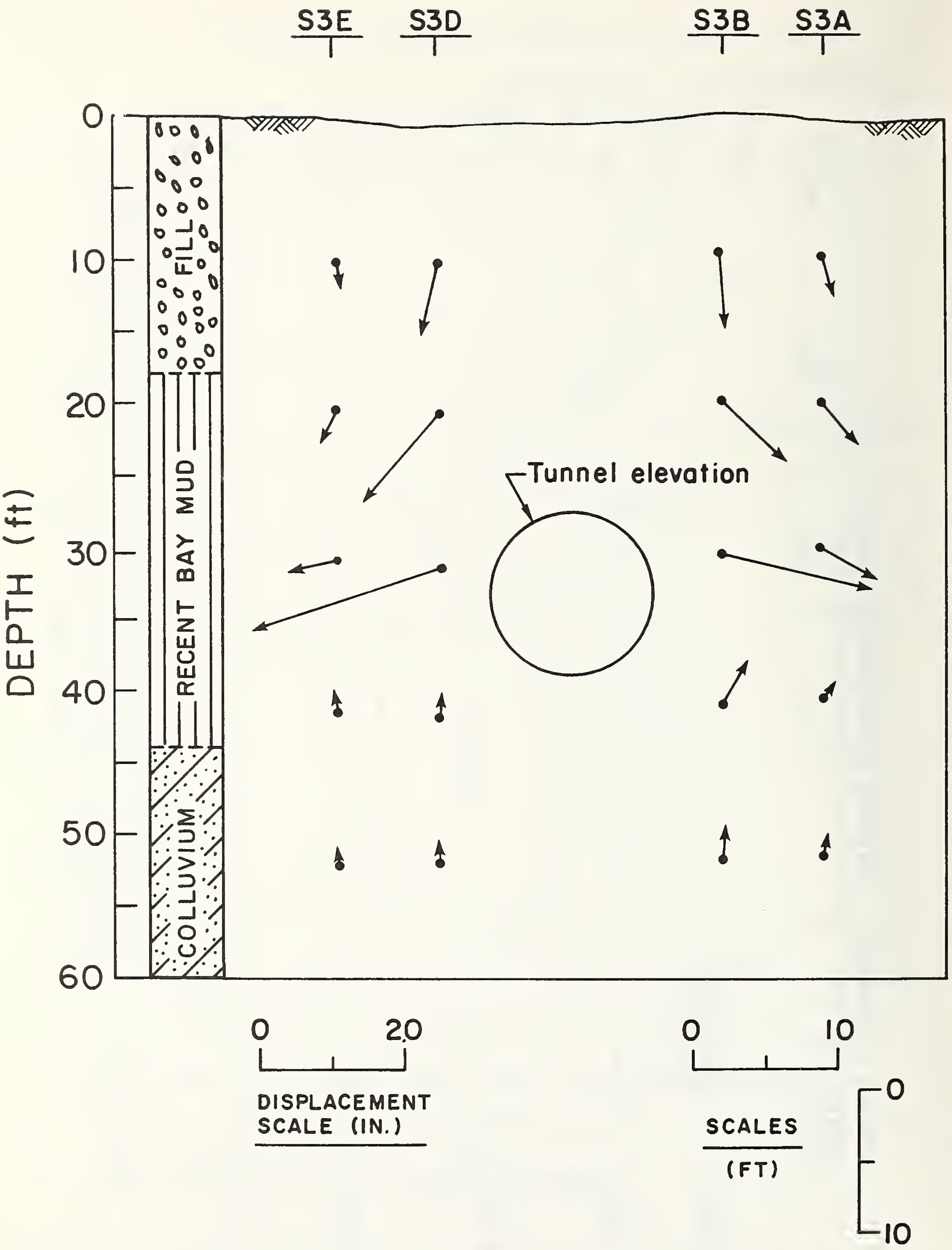


Figure 5.16: Transverse Displacement Vectors 30 Days After Shield Passage, Line 3

the springline are down toward the tunnel. Maximum settlements reached values of 1.3 in. (3.3 cm) and these occurred at the points closest to the tunnel crown. Typically below the tunnel, there is a small net heave indicated at all points.

The vectors in Figure 5.16 illustrate a consistent symmetrical pattern with settlements above the tunnel following the bowl type shape associated with conventional tunneling. Significantly, the vectors of movement for the points in the rubble fill layer show largely vertical displacement. This is one facet of behavior consistent with Line 2 response. In both cases, the rubble fill was little affected by the initial lateral spreading observed so clearly in the soft Bay Mud. Reasons for these phenomena are discussed in a subsequent section of this chapter.

Surface settlements measured five months after shield passage at Line 3 by the City of San Francisco at the centerline were 1.6 in. (4 cm) while 10 ft. (3 m) on either side the values ranged from 1.3 to 1.4 in. (3.3 to 3.6 cm). These results are very consistent with those measured by the settlement couplings on Inclinometers B and D (those closest to the survey point locations) in the rubble fill.

5.5 GROUND MOVEMENTS AT LINE 4

Inclinometer hole locations at Line 4 are given in Table 5.3; casing 4B ended up to be the closest of all inclinometers to the tunnel with only a 1.5 ft. (0.5 m) space between the casing and the tunnel periphery after tunnel construction.

TABLE 5.3

Positions of Inclinometers at Line 4

Inclinometer	Distance from Tunnel* Ft.	Distance from Tunnel Periphery at Springline Ft.
4A	18	12
4B	7.5	1.5
4C	- 2	-
4D	-11	5
4E	-18	12

* + = Right of centerline looking in direction of tunneling

- = Left of centerline looking in direction of tunneling

5.5.1 Lateral Movements Perpendicular To The Tunnel Axis

Measured lateral movements perpendicular to the tunnel axis for Inclinometer 4B for a series of shield positions are given in Figure 5.17. As in the case of both Lines 2 and 3, the inclinometer shows a heaving effect occurring in the Bay Mud soils as the shield approaches, and as the face passes by the instrumentation line. The maximum heave reaches 2.0 in. (5.1 cm) just after the face advanced by the inclinometer, and the movements at this stage are almost exclusively confined to the Bay Mud. This behavior is consistent with that observed at Lines 2 and 3, but the magnitude of the movements falls between the two. Interestingly, the magnitude of the nondimensional bulkhead earth pressure at Line 4 was 0.8, a value between the 0.4 at Line 2 and the 1.0 of Line 3.

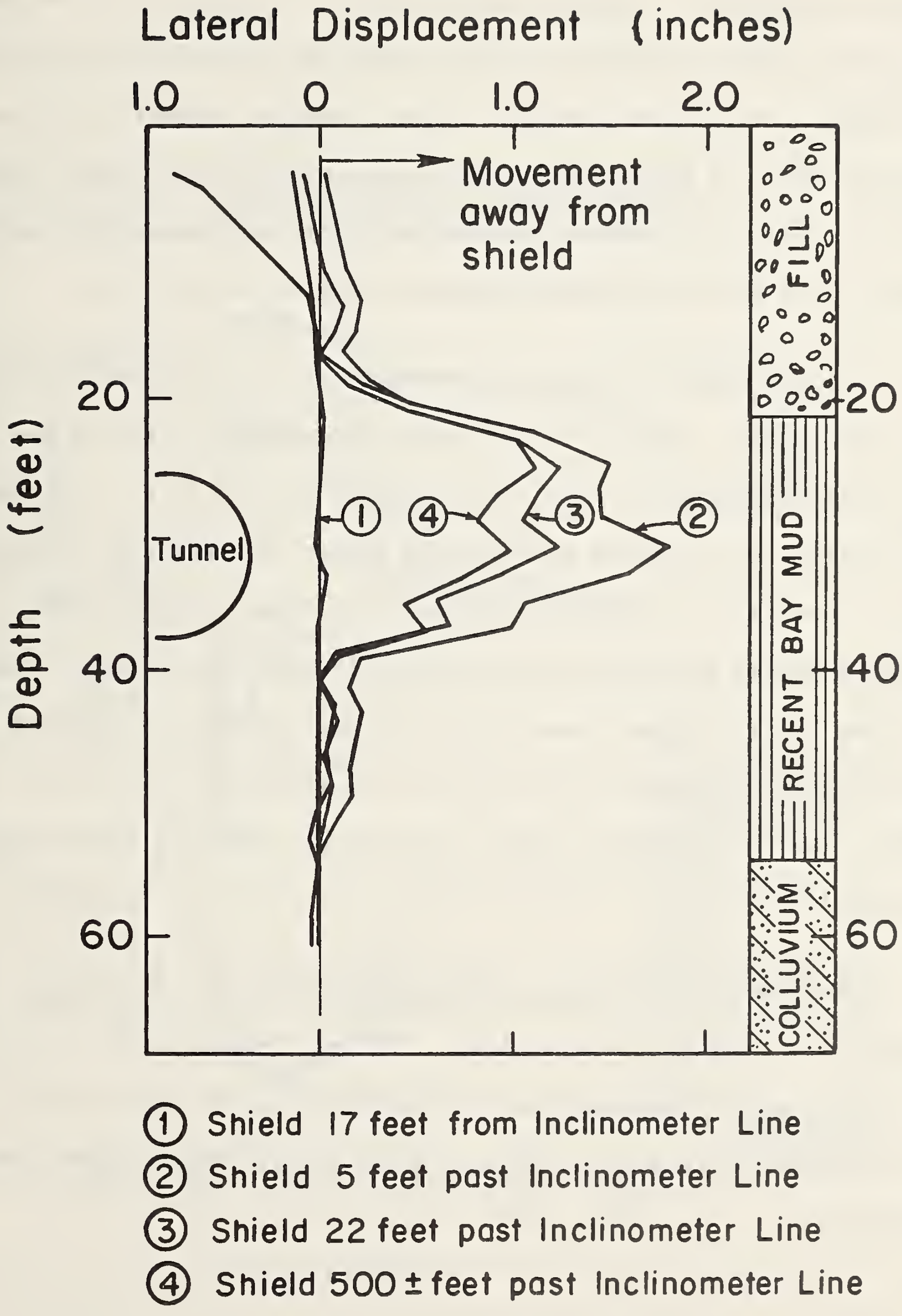
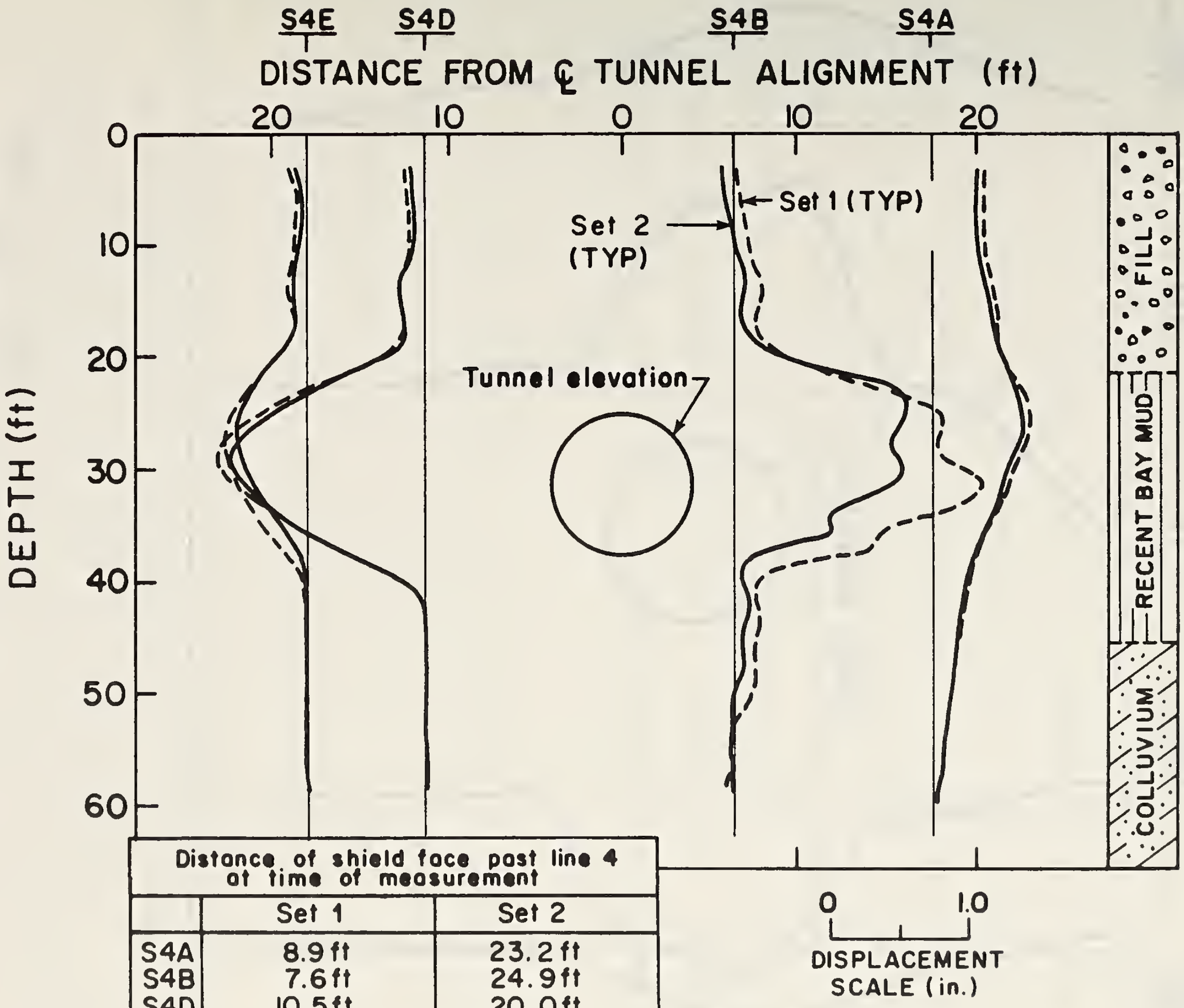


Figure 5.17: Lateral Deflections (A Axis) of Inclinometer S4B Before and After Shield Passage

After tail void passage, the incremental lateral displacements were towards the shield. However, as at Line 3, the initial heaves were so large that the net position is still away from the shield in the Bay Mud well after the shield passage. This response generally follows the patterns observed for Line 3 and is consistent with the fact that the pitch of the shield in the vicinity of Line 4 was essentially zero (see Figure 5.2), and the bulkhead pressures were relatively high.

Plots of measured lateral displacements for Inclinometers 4A, 4B, 4D, and 4E for cases where the shield face and the tail of the shield have moved just beyond Line 4 are given in Figure 5.18. The data for the casings on different sides of the shield are reasonably symmetric, with movements diminishing with distance from the centerline. The measured heaves are largely confined to the Bay Mud stratum, beginning with the initial outward heaves during face passage and followed by the small inward movements after the tail void effect is first felt. In all cases, the inclinometers remain in positions that are heaved away from the shield.

The final inclinometer positions recorded 15 days after shield passage are given in Figure 5.19. While there is some tendency for increased inward movement near the surface in the rubble fill, the inclinometer positions in the Bay Mud are still heaved outward away from the shield.



	Distance of shield face past line 4 at time of measurement	
	Set 1	Set 2
S4A	8.9 ft	23.2 ft
S4B	7.6 ft	24.9 ft
S4D	10.5 ft	20.0 ft
S4E	13.8 ft	26.5 ft

Figure 5.18: Lateral Deflections (A Axis) at Line 4 After Shield Passage and Tail Void Passage

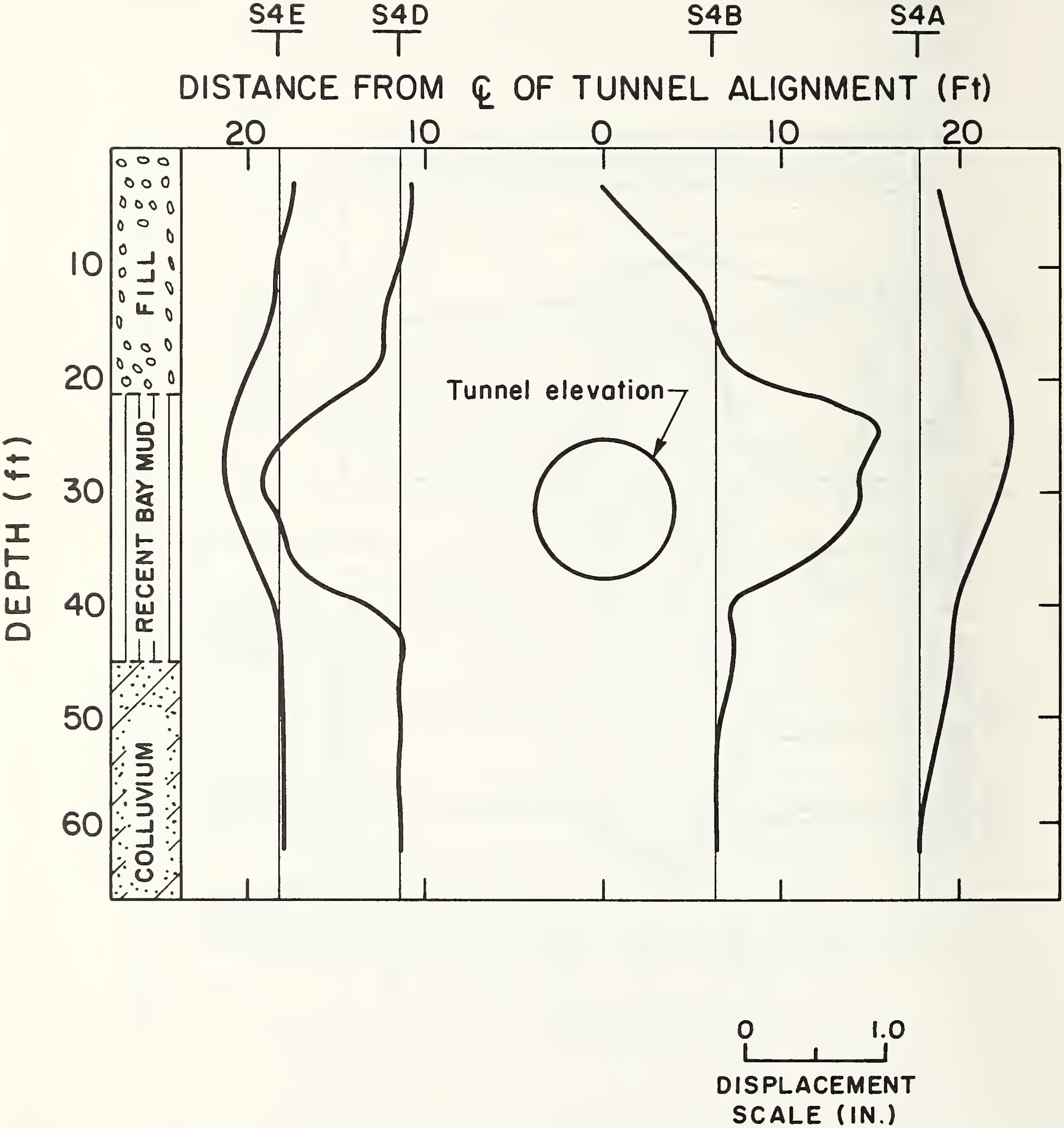


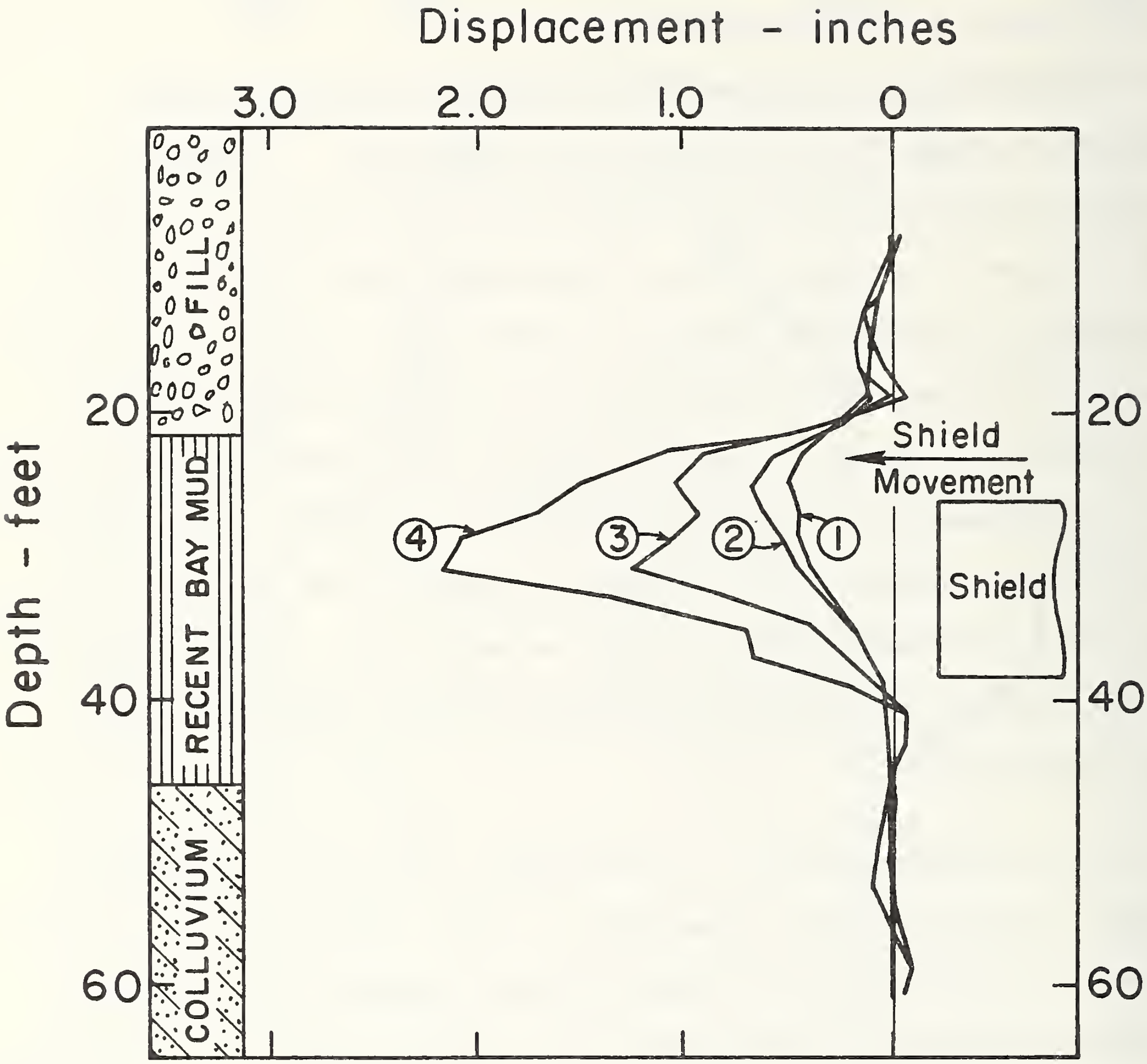
Figure 5.19: Lateral Deflections (A Axis) at Line 4 15 Days After Shield Passage

5.5.2 Lateral Movements Parallel To Tunnel Axis

For Line 4 an attempt was made to monitor closely the lateral movements in front of the shield using Inclinometer 4C. Four sets of readings were made when the shield face was 18,, 11, 8, and 4 ft. (5.5, 3.3, 2.4 and 1.2 m) away from the inclinometer (Figure 5.20). The development of the heaving pattern is clearly shown, and as at other lines, it is concentrated in the Bay Mud. The central bulge occurs not at the center of the face, but the vicinity of the upper third of the face. Displacements grow rapidly as the shield approaches, and reach about 2.1 in. (5.3 cm) at its closest position to the casing. As observed in Inclinometers S2C and S3C, the water level in Inclinometer S4C was noted to rise rapidly to the ground surface as the shield moved very near the casing.

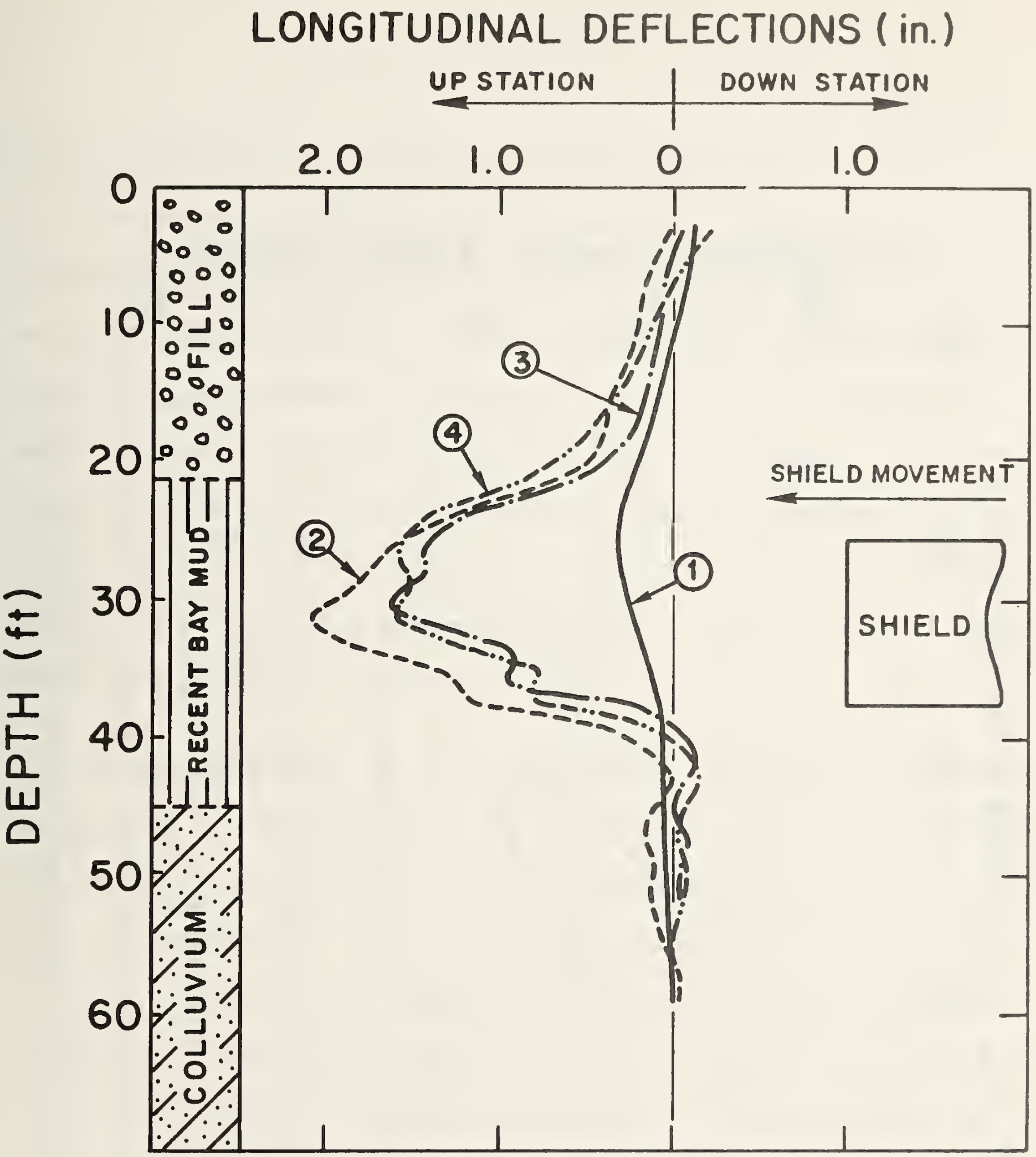
The B-Axis measurement, for Inclinometer 4B show a consistent pattern relative to those measured in 4C (see Figure 5.21). All displacements are in the direction of shield movement until the tail void passes the line, at which time a small reversal occurs.

Figure 5.22 shows a plan view of displacement vectors for combined lateral movements parallel and perpendicular to the tunnel axis at the springline elevation for a number of shield positions. The pattern depicted is quite similar to that shown for Line 3 in Figure 5.15. The soil is pushed ahead of, and away from, the shield as it advances. After shield passage, the vectors reflect small movements into the tail void, but the net positions are away from the tunnel.



① Shield 18 feet from Inclinometer Line
② Shield 11 feet from Inclinometer Line
③ Shield 8 feet from Inclinometer Line
④ Shield 4 feet from Inclinometer Line

Figure 5.20: Longitudinal Deflections (A Axis) of Inclinometer S4C Prior to Shield Passage



① Shield face 14 ft from line; Tail void 30 ft from line
② Shield race 7 ft past line; Tail void 9 ft from line
③ Shield face 25 ft past line; Tail void 9 ft past line
④ 8 days after shield passage

Figure 5.21: Longitudinal Deflections (B Axis) of Inclinometer S4B Before and After Shield Passage

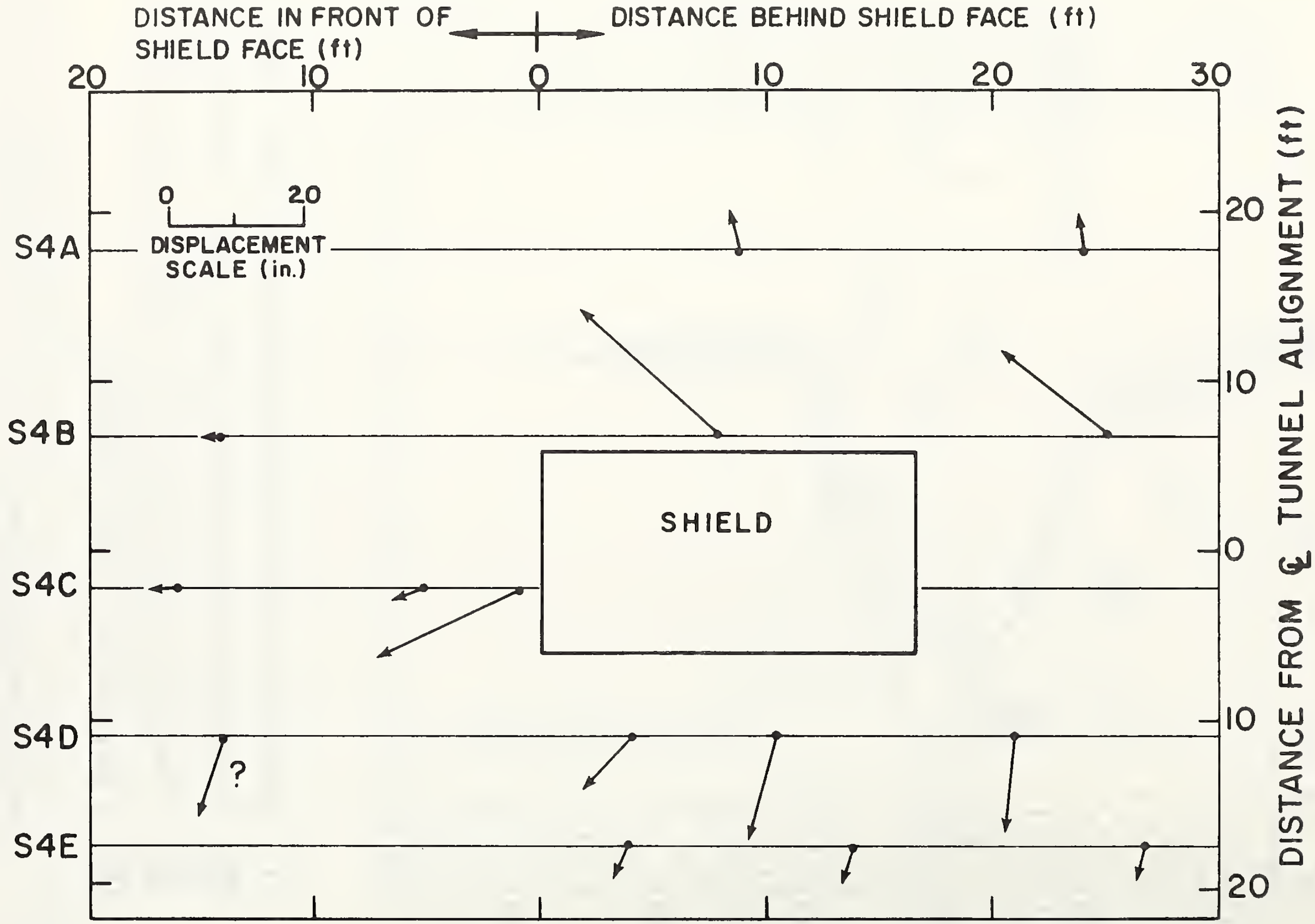


Figure 5.22: Horizontal Displacement Vectors at Springline, Line 4

5.5.3 Combined Lateral and Vertical Movements

Displacement vectors showing combined lateral and vertical movements in the plane perpendicular to the tunnel axis for a stage 15 days after tunnel passage are given in Figure 5.23. Vertical movement measurements demonstrated reasonable trends except at Inclinometer 4A, where the settlement couplings apparently did not function. Other than at Inclinometer 4A, the displacement vector pattern in Figure 5.23 has a close resemblance to that observed at Line 3 (Figure 5.16). Maximum settlements are indicated in the rubble fill, and they reach values of about three quarters of an inch (1.9 cm).

Surface settlements at Line 4 were closely monitored at a series of points along a line perpendicular to the tunnel axis. Values measured at times of 8 hours, 8 days and 40 days after passage of the shield are given in Figure 5.24. There is a steady increase in the settlements with time, with all of the respective profiles following a bowl pattern centered over the centerline of the tunnel. The bowl pattern has the shape and dimensions common to tunnels in soil constructed by conventional shields (Peck, 1969 and Schmidt, 1969). The maximum settlement reaches a value of 1.2 in. (3.0 cm) at a time of 40 days after shield passage.

The variation of the vertical surface movement above the centerline of the tunnel with time for a period of 150 days is shown in Figure 5.25. Closely spaced readings taken as the shield approached Line 4 show that the ground surface underwent a slight heave of 0.25 in. (0.63 cm) initially. Subsequently, the ground began to settle and this trend

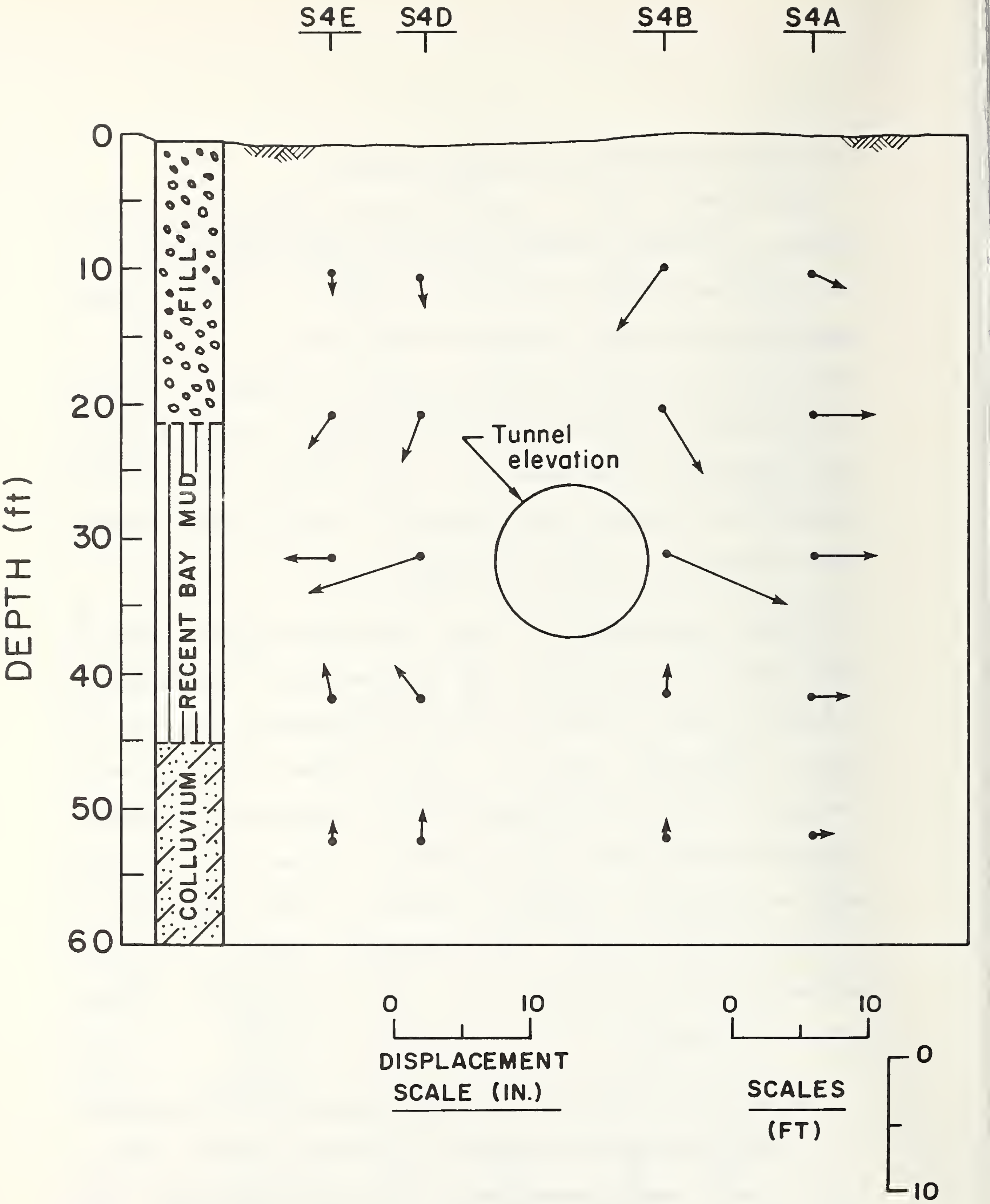


Figure 5.23: Transverse Displacement Vectors 15 Days After Shield Passage, Line 4

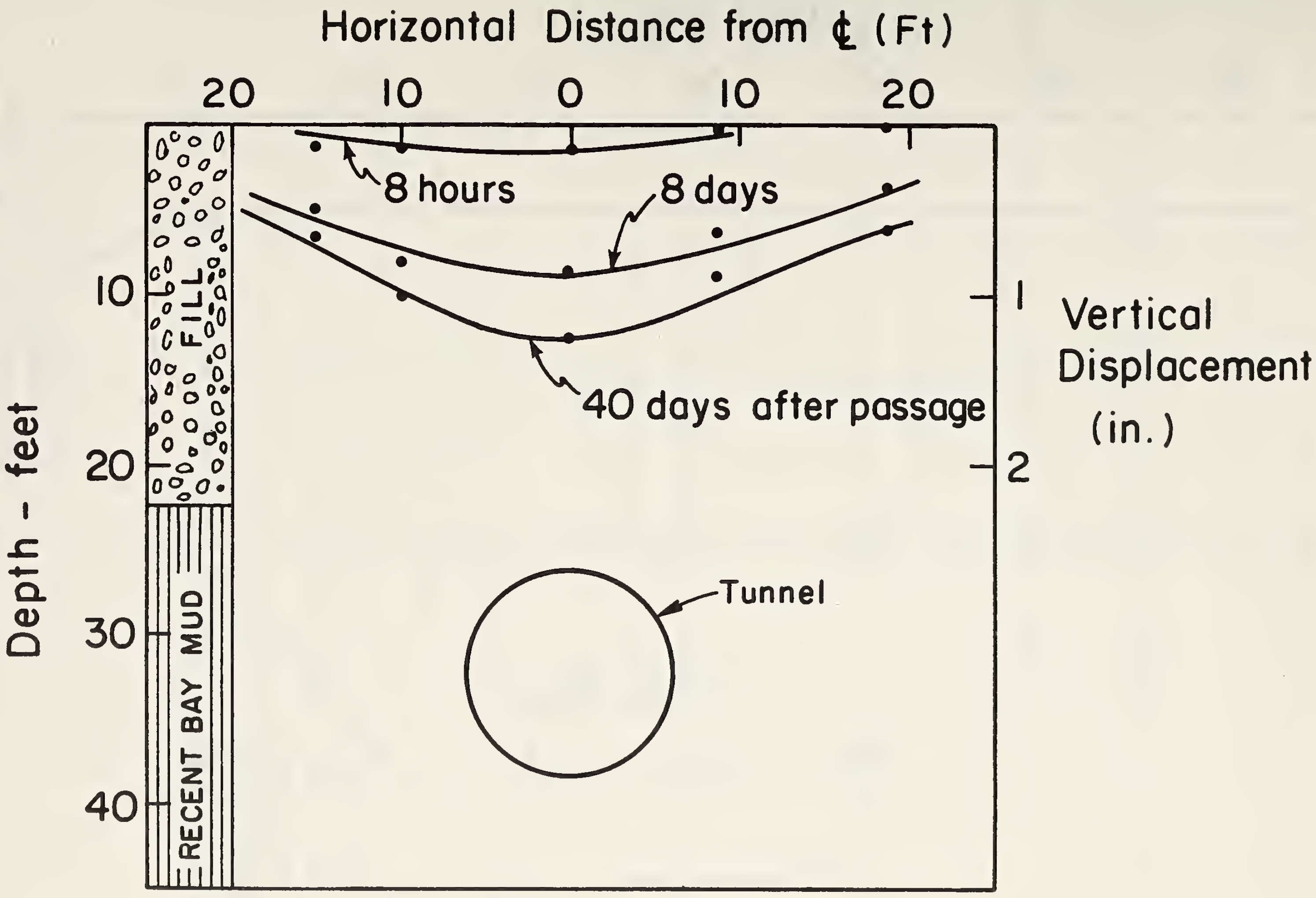


Figure 5.24: Surface Settlement Profile at Line 4

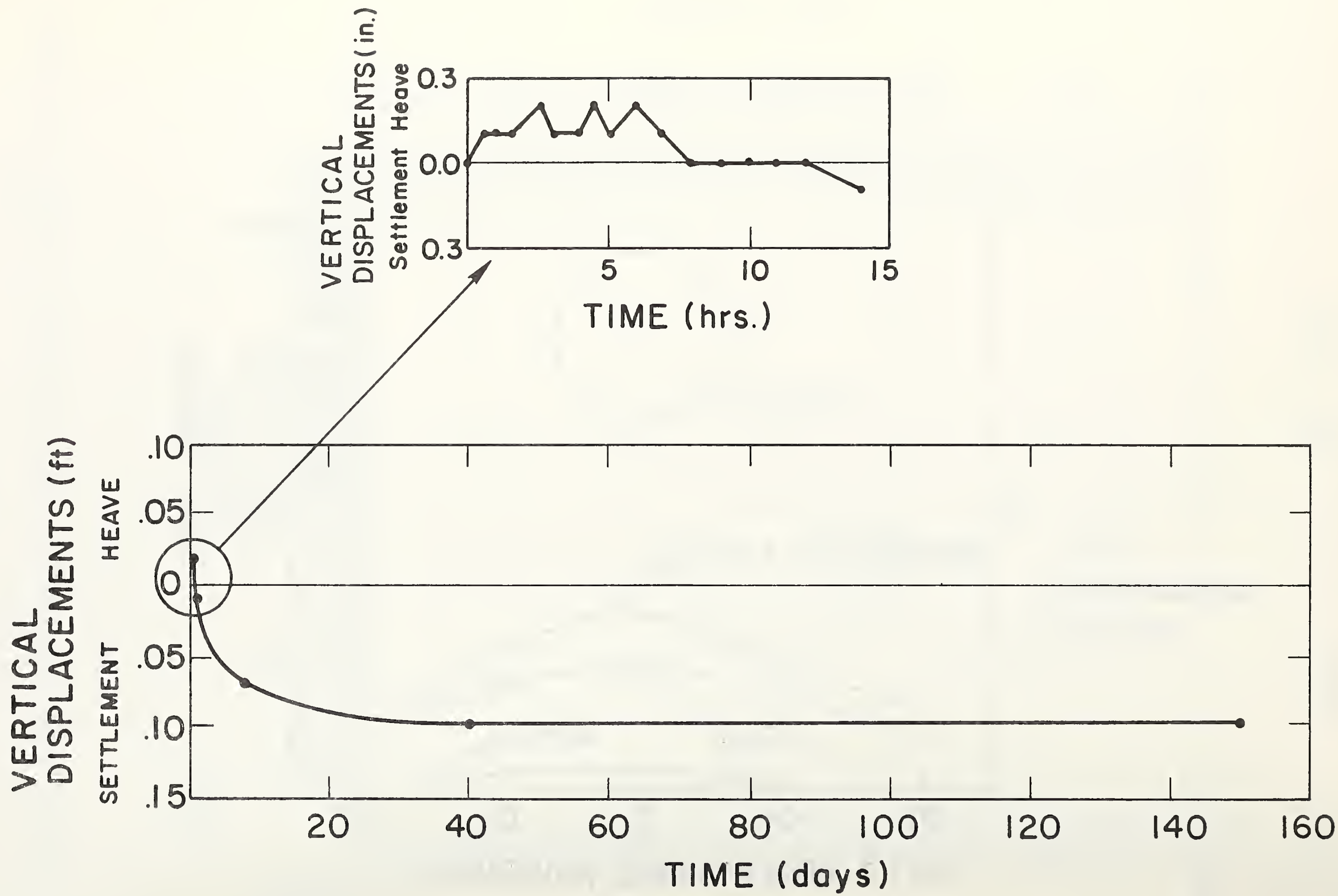


Figure 5.25: Surface Settlement versus Time, Centerline of Line 4

continued for 40 days. The reading taken at 150 days showed essentially no increase beyond that recorded at 40 days.

5.6 GROUND MOVEMENTS OUTSIDE OF STANFORD INSTRUMENTATION LINES

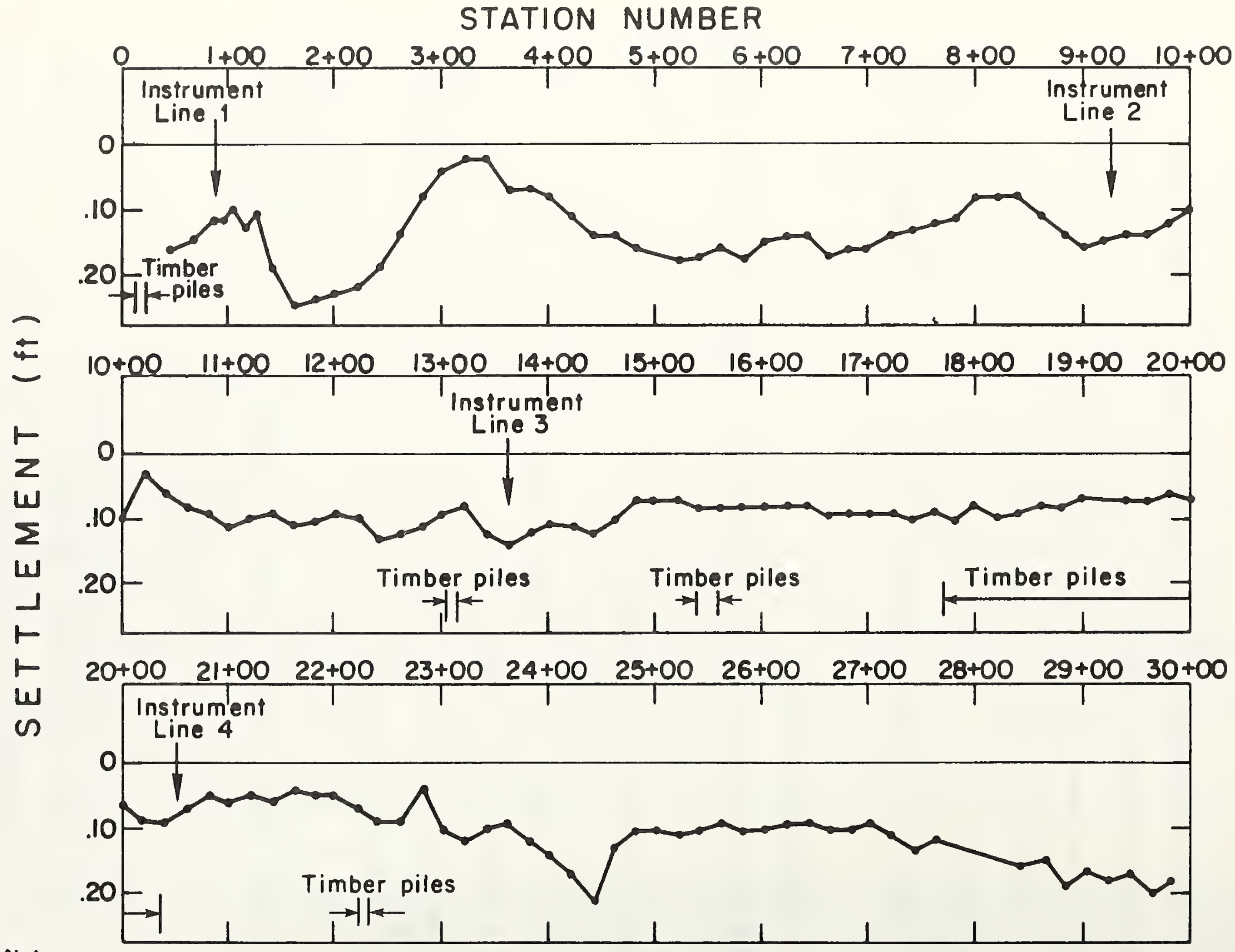
As noted earlier, surface settlements along the entire alignment, and lateral movements at selected locations other than Lines 2, 3, and 4, were monitored by the City of San Francisco and its consultants. The surface survey in particular provides useful data since it provides an overall view of the ground movements induced by the EPB shield process.

5.6.1 Surface Settlements

Surface settlements measured along the centerline of the tunnel at a date some five months after the active excavation work was completed are shown in Figure 5.26. There are several noticeable trends:

1. In the beginning of the work between Stations 0 and 5+00, the settlements are quite variable, probably reflecting the fact that the tunnel crews were on the early part of the learning curve. The largest settlement in this region reached 0.25 ft. (3 in. or 1.6 cm) and occurs at Sta 1+60; this represents the effect of a 15 day work stoppage which was required to get the EPB power train into the tunnel and connected to the shield.

2. Between Stations 5+00 and 23+00 the settlements did not vary substantially, and did not exceed 0.15 ft. (1.8 in. or 4.6 cm). The instrumentation lines for this project (Lines 2, 3, and 4) fall in this area, and would appear to reflect the typical behavior.



Note:
1. All settlements measured 14 and 15 October 1981
2. Timberpile locations obtained from N2 contract Drawing No. 41,409

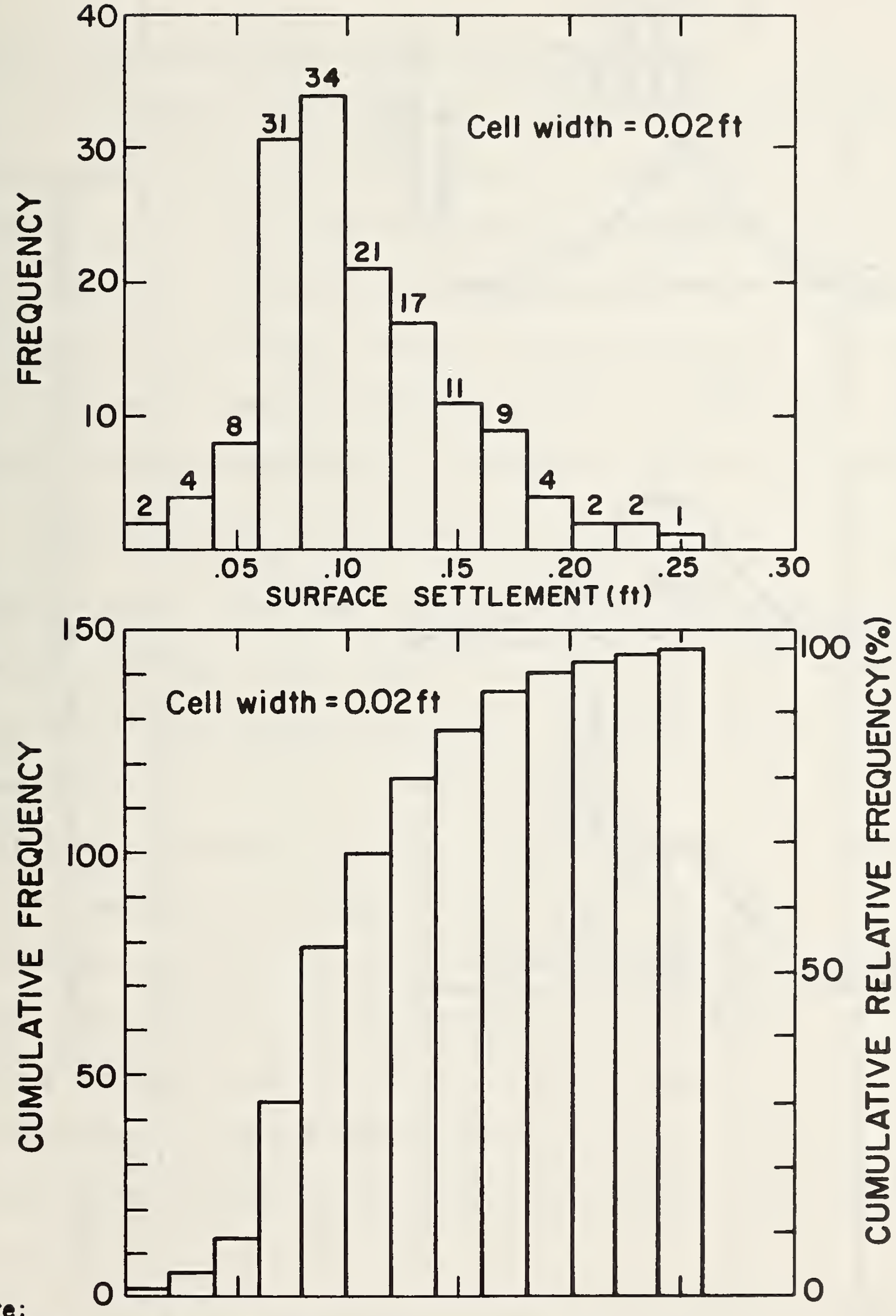
Figure 5.26: Surface Settlement Profile Along Centerline of Tunnel Alignment

3. After Station 23+00 there is one area with a relatively sharp increase in settlements (Sta 24+30) and a general trend towards increasing settlement near the end of the job. The settlements around Sta 24+30, which reach as high as 0.22 ft. (2.6 in. or 6.7 cm), were caused by a second major work stoppage. This was required to clean the slots in the cutterhead which had become clogged with wooden debris from the piles encountered along the alignment. At this location the spoils retaining area was emptied of soil to gain access to the cutterhead. The trends towards increasing settlements near the end of the job cannot be attributed to any one factor, although in this area there were some labor problems (a brief strike occurred) and the Bay Mud soils were sandier than previously encountered. At Sta 28+00 sandy clays were intersected by the tunnel invert.

The sharp increases in settlements which accompanied the two major work stoppages provide an interesting object lesson in the workings of the EPB shield. To be fully effective, the spoils retaining area behind the cutterhead of the shield needs to be full of soil, and the shield should be advancing on a regular basis in order to prevent soil movement towards the shield due to any form of stress relief effect. At the site of the second work stoppage, the spoils retaining area was emptied, thus temporarily removing pressure at the face and in the area of the cutterhead slots. When the first work stoppage occurred, the spoil was left in the retaining area and the shove jacks were held in position, but stress relief apparently still occurred. This was possibly caused by a small backwards movement of the shield or creep or consolidation of the soil inside the spoils chamber and immediately outside the face. It is apparent that the relatively continuous positive outward pressure exerted at the face during normal operation serves to help control ground movements, as witnessed by the performance between Stations 5+00 and 23+00. The short, two day weekend breaks in operations did not noticeably increase settlements.

A statistical view of the surface settlement measurements is provided by Figure 5.27, a histogram plot showing the frequency with which certain levels of settlement occurred. The most commonly recorded value is 0.09 ft. (1.1 in. or 2.7 cm), while the maximum value is 0.25 ft. (3 in. or 7.6 cm) which occurred only once, and can be attributed to the effects of a work storage.

The distribution of settlements about the centerline is depicted in Figure 5.28. Minimum, maximum and mean values are shown for the three points monitored by surveying PK nails in the pavement at the centerline and 10 ft. (3 m) on either side of the centerline of the tunnel. Also for reference, the results recorded along Line 4 are included. As expected, the settlements on either side of the centerline are typically less than that at the centerline, however, the difference between the settlements is small. The settlement profile at Line 4 shows a more classical bowl shape than the other results, but this may be due to the fact that the measuring points extended further from the centerline than elsewhere providing an opportunity for better definition.



Note:

1. Settlements measured 14 and 15 October 1981

Figure 5.27: Distribution of Surface Settlement Data

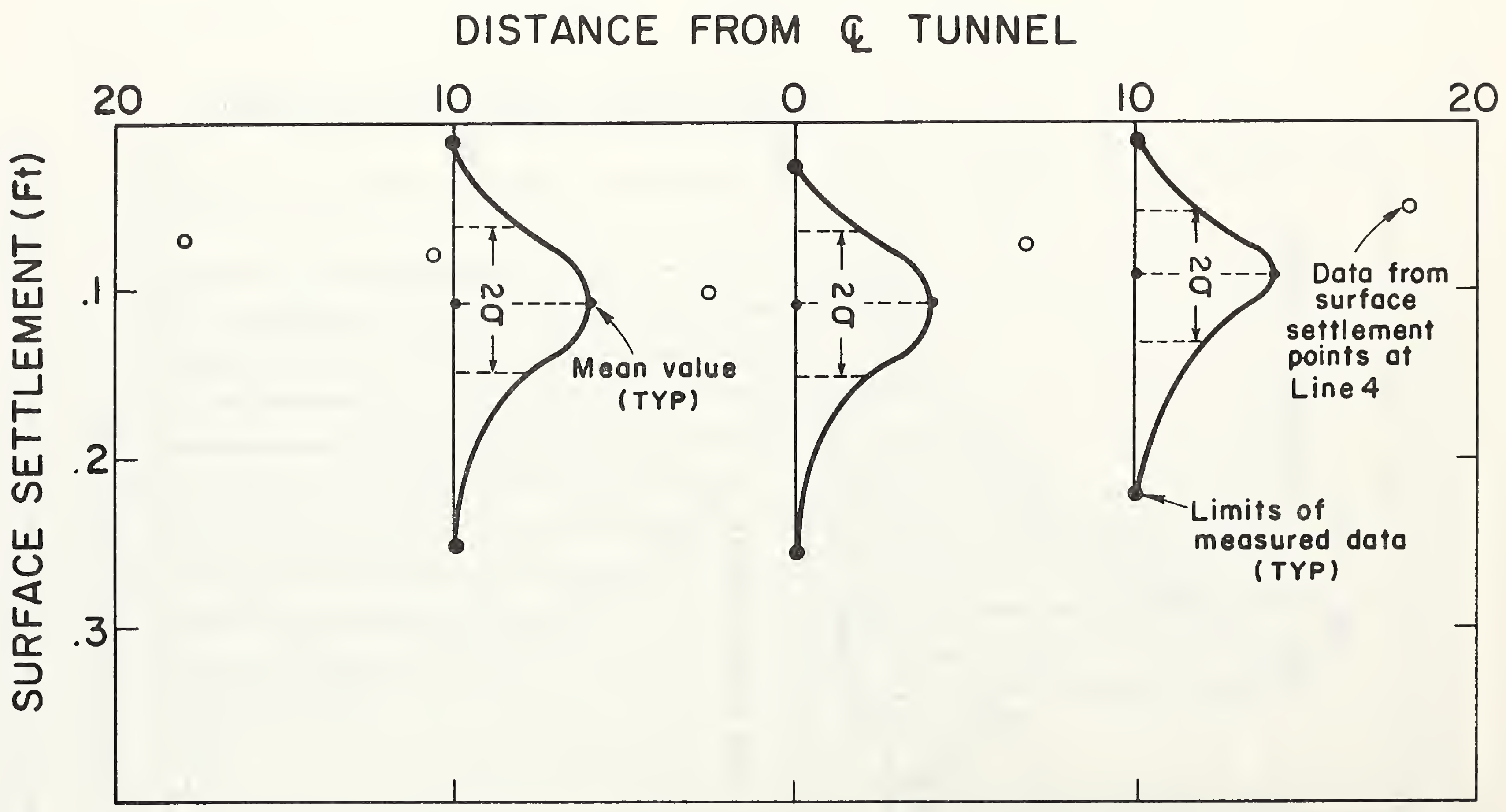


Figure 5.28: Distribution of Surface Settlement Data Along Tunnel Alignment

5.6.2 Ground Movements at Instrumentation Line 1

This instrumentation line is positioned at Sta 0+90, just beyond the entrance portal. It was monitored by Dames and Moore on a periodic basis to check on the early performance of the shield. At this stage, the shield was in the tunnel without the power train trailing behind. The effects of the shield advance at this station were measured using the inclinometers and the settlement casings as at the other instrument lines. However, the settlement casings gave inconsistent results.

Figure 5.29 gives the measured lateral movements at 40 days after shield passage. The net positions of the inclinometers is inward towards the tunnel, and there is a strong similarity between these results and those observed at Line 2. Maximum inward movements reach about 0.5 in. (1.3 cm) at a depth of around 25 ft. (7.6 m), a position just above the crown of the tunnel. As in the case of Lines 2, 3, and 4, the inward lateral movement trends after shield passage were preceded by heaves of the soil as the shield face approached. Maximum values heave are not known, but the similarity of the results with those of Line 2 suggests a similar heaving effect as measured there. Maximum lateral heaves at Line 2 reached values of 0.5 in. (1.3 cm). Notably, the bulkhead pressure at Line 1 was essentially the same as that at Line 2.

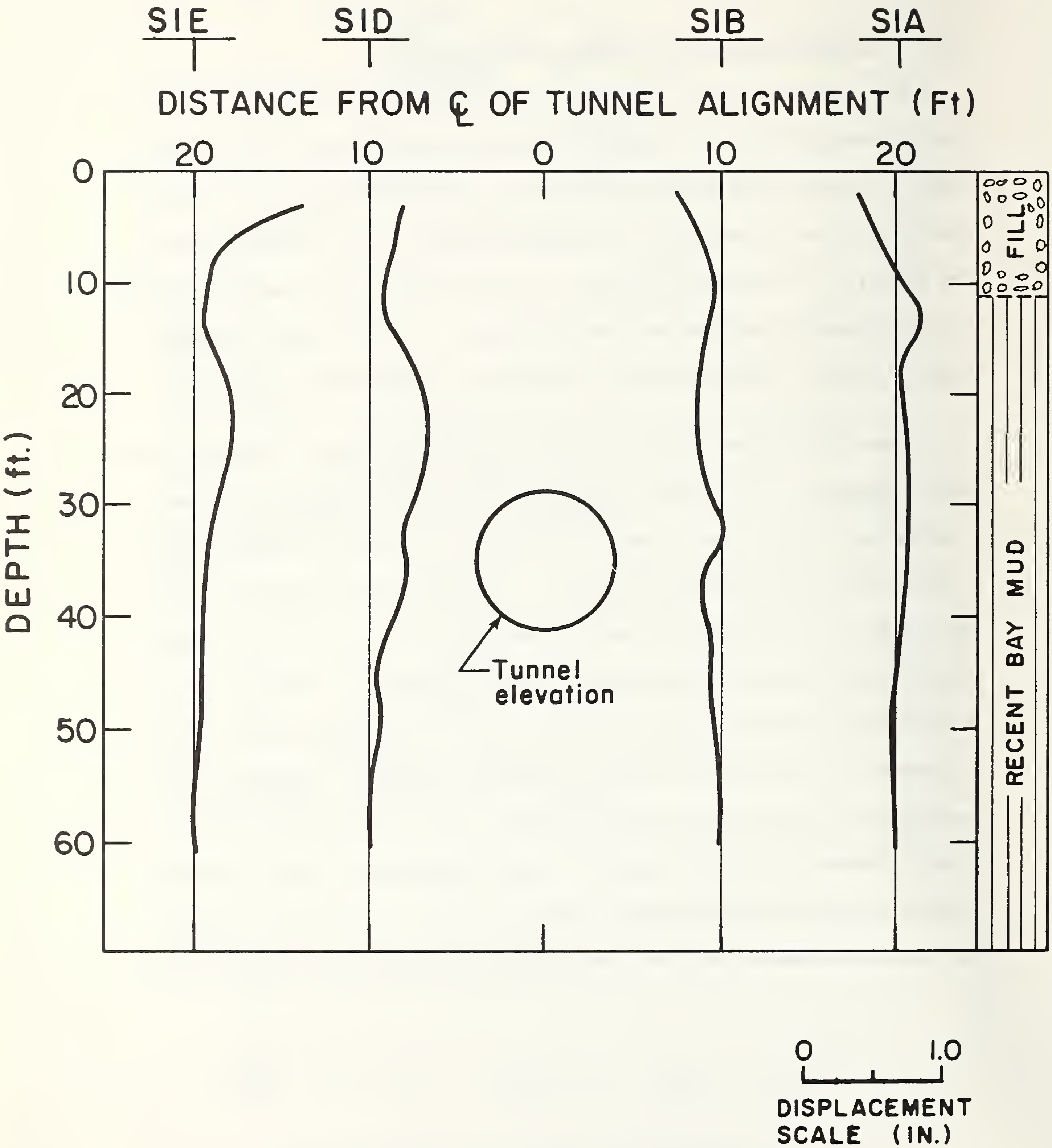


Figure 5.29: Lateral Deflections (A Axis) at Line 1 40 Days After Shield Passage

5.7 COMPARISONS OF MOVEMENTS BETWEEN INSTRUMENTATION LINES

5.7.1 Lateral Movements

Plots showing the measured lateral movements for all four instrumentation lines at times well after shield passage are given in Figure 5.30 and 5.31 for the inclinometers on the left and right sides of the tunnel respectively (looking in the direction of shield advance). Figure 5.32 compares the lateral movements measured for the closest inclinometers on both the left and right sides of the tunnel. In these plots it is assumed that each set of Inclinometers A - D at each line are located at the same position relative to the tunnel to facilitate comparison. In fact, it should be remembered that the positions of a given inclinometer set, e.g., A, may vary from line to line by several feet.

There is an interesting contrast in the behaviors in Figures 5.30 through 5.32: Lines 1 and 2 show net inward movements while Lines 3 and 4 show net outward movements. Note that Line 3 yielded by far the largest net outward displacements. The differences in the lateral movement response at the lines appear to correlate well with the measured earth pressures inside the EPB shield bulkhead (Figure 5.2). Where the pressures were high, as at Lines 3 and 4, initial outward heaves occurred which were large enough to exceed the subsequent inward movements due to the tail void. On the other hand, where the pressures were low, the initial outward heaves were small relative to the inward

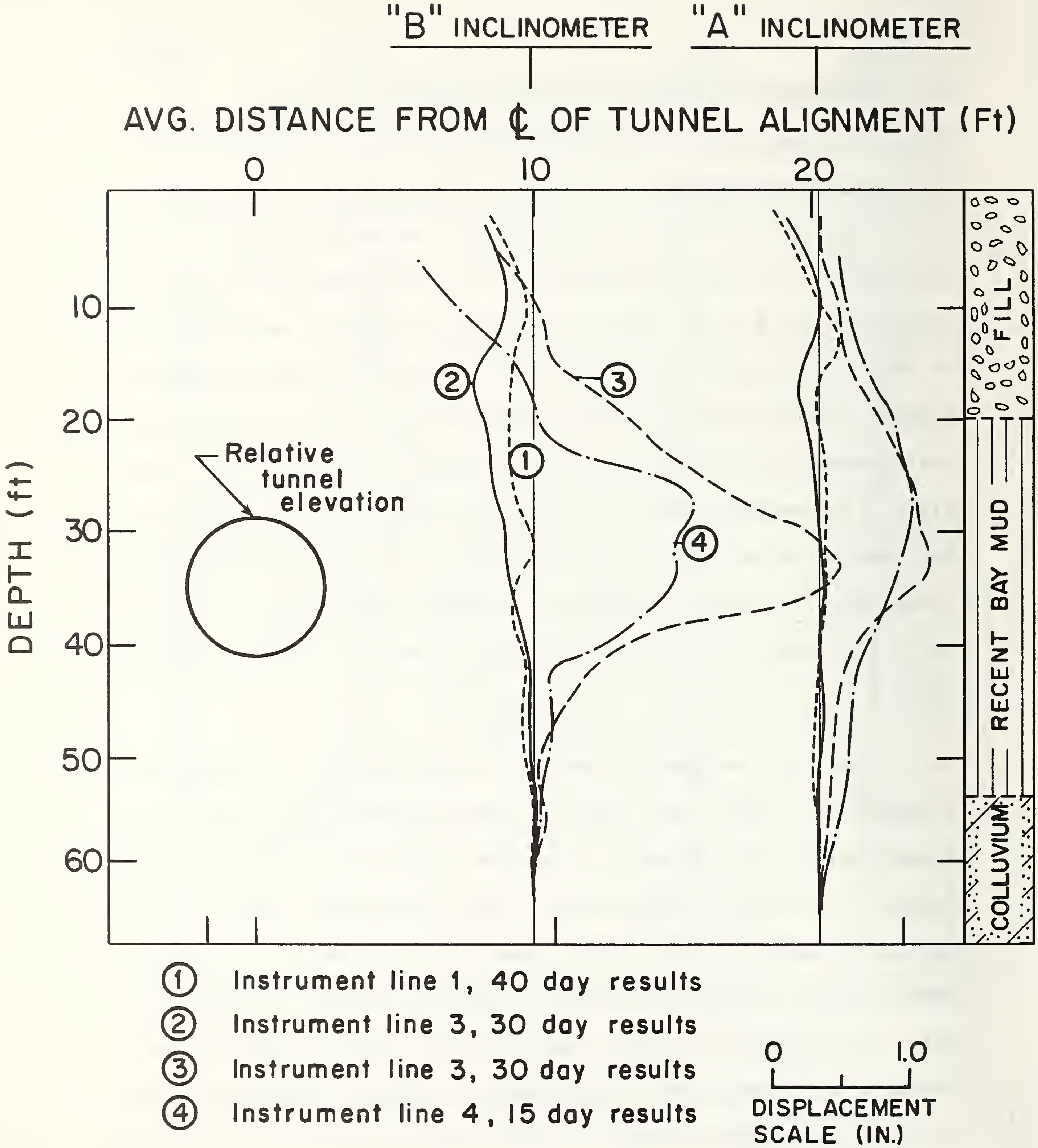


Figure 5.30: Lateral Deflections (A Axis) for "A" and "B" Inclinometers, Lines 1 through 4

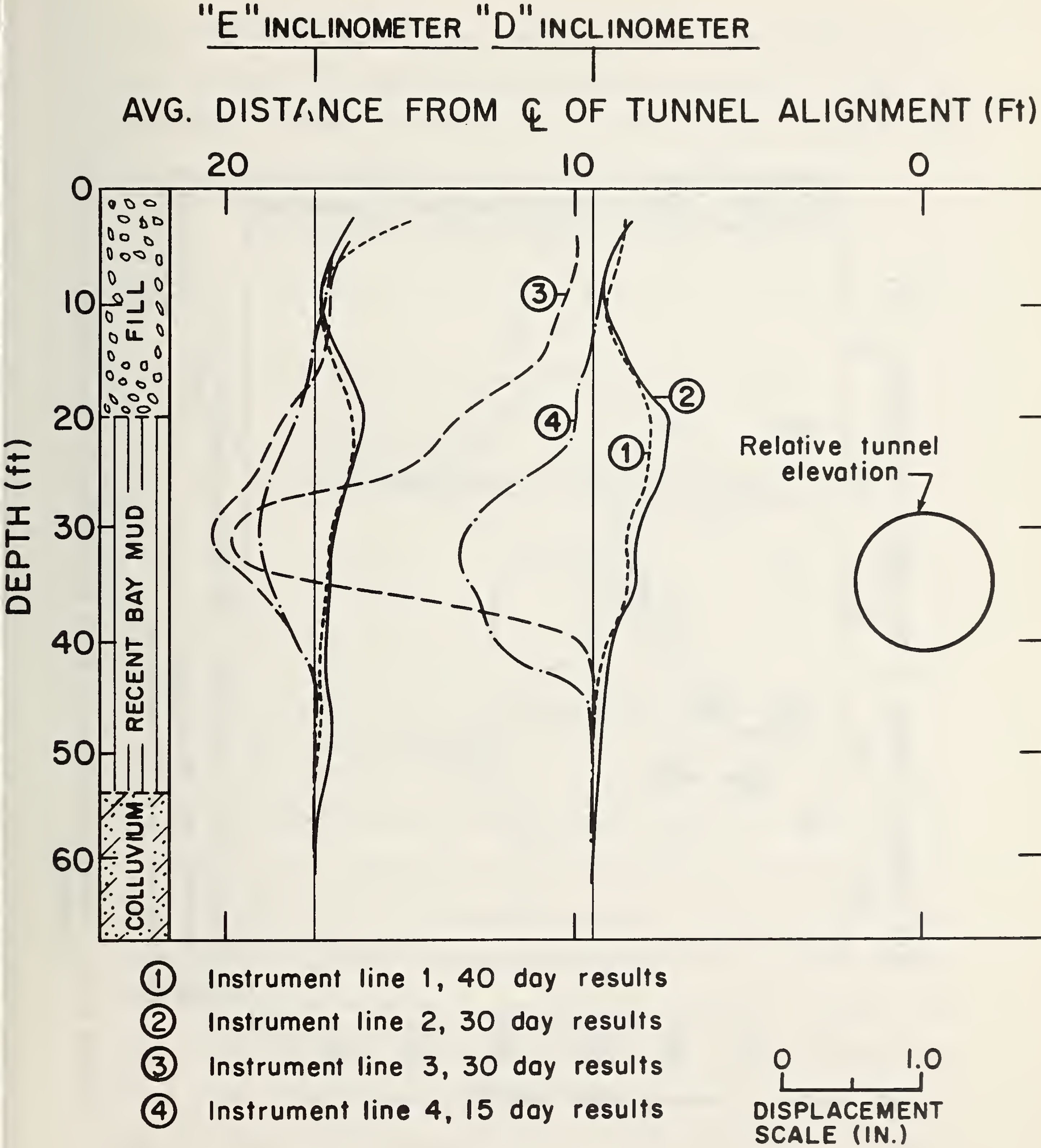


Figure 5.31: Lateral Deflections (A Axis) for "D" and "E" Inclinometers, Lines 1 through 4

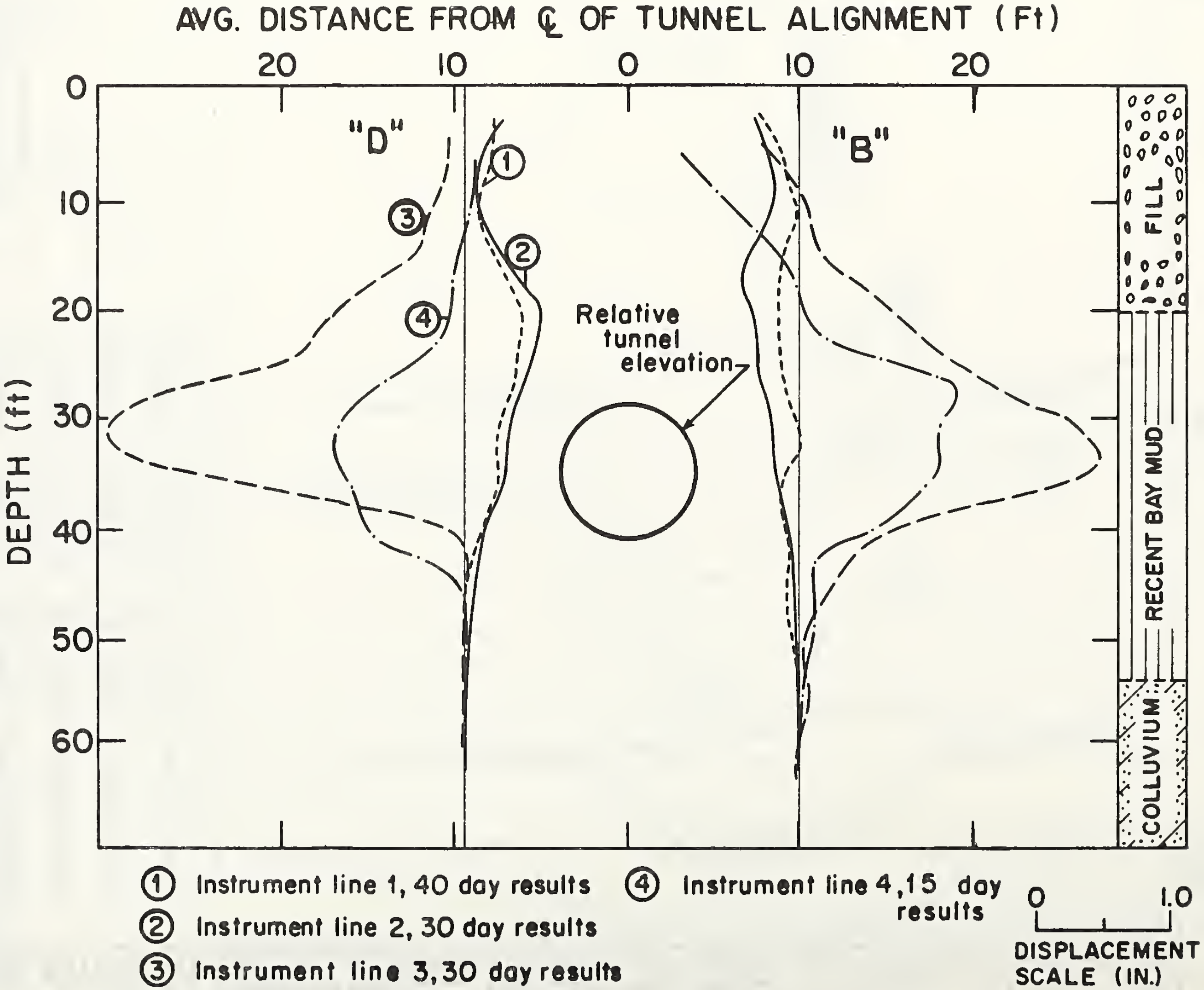


Figure 5.32: Lateral Deflections (A Axis) for "B" and "D" Inclinometers, Lines 1 through 4

movements due to the tail void. Table 5.4 shows the correlation between maximum lateral displacements and bulkhead earth pressure. The data are consistent, demonstrating that the higher the bulkhead pressure, the larger the initial heaves.

TABLE 5.4

Bulkhead Earth Pressure And Key Lateral Movements

Line	NonDimensional Bulkhead Earth Pressure*	Max. Lat. Heave During Shield Passage in.	Max.Net Lat. Move After Shield Passage in.
1	0.4	NA	- 0.4 (inward)
2	0.4	0.6	- 0.5 (inward)
3	1.0	3.2	+ 2.3 (heave)
4	0.8	2.0	+ 1.2 (heave)

* - Actual bulkhead pressure / maximum observed during tunneling

The initial heaves which occurred to some degree at all lines apparently reflect the fact that the EPB shield was being operated so as to remove a soil volume from the spoil retaining area somewhat smaller than the volume which was attempting to pass into the shield via the cutterhead. In general, this was the intent of the contractor in order to minimize ground movements. As noted earlier however, the fact that higher than average bulkhead pressures occurred in the vicinities of Lines 3 and 4 is apparently related to the process of cutting through the old wooden piles and possibly some clogging of the screw auger by wood fragments.

While the final positions of the inclinometers, as indicated in Figures 5.30, 5.31, and 5.32 show significant differences in the net movements at the instrumentation lines it is significant that there are similarities in the results as well. In every case, some heaving effects were measured in the soil as the shield approached, and this was subsequently always followed by inward movements due to passage of the tail void. The heaving displacements were confined almost exclusively to the soft Bay Mud except in the case of Line 3 where the initial lateral movements were so large as to extend into the lower portion of the overlying rubble fill. The subsequent inward movements however, occurred in both the Bay Mud and the rubble fill. This may be shown by examining the inward lateral movements which occurred after the position of maximum heave was recorded. In Figure 5.33 this displacement is plotted using the result measured by the inclinometers closest to the tunnel for Lines 2, 3, and 4. A general inward tilting effect is observed in all cases in the upper 10 ft. (3 m) of the rubble fill. At a depth of 30 to 36 ft. (9.1 to 10.9 m), in the area from the springline to the crown, the results show the largest inward deflections, with maximum values of 0.8, 0.9 and 1.2 in. (2.0, 2.3, and 3.1 cm) respectively for Lines 2, 3 and 4. Inward movements diminish rapidly below this in the results for Lines 3 and 4 and somewhat more slowly for Line 2.

It is apparent that in spite of the differences in initial heaves, the resulting maximum inward movements were similar and all occurred at about the same depth. This leads to the conclusion that the major effect of the tail void was apparently similar at each of the lines,

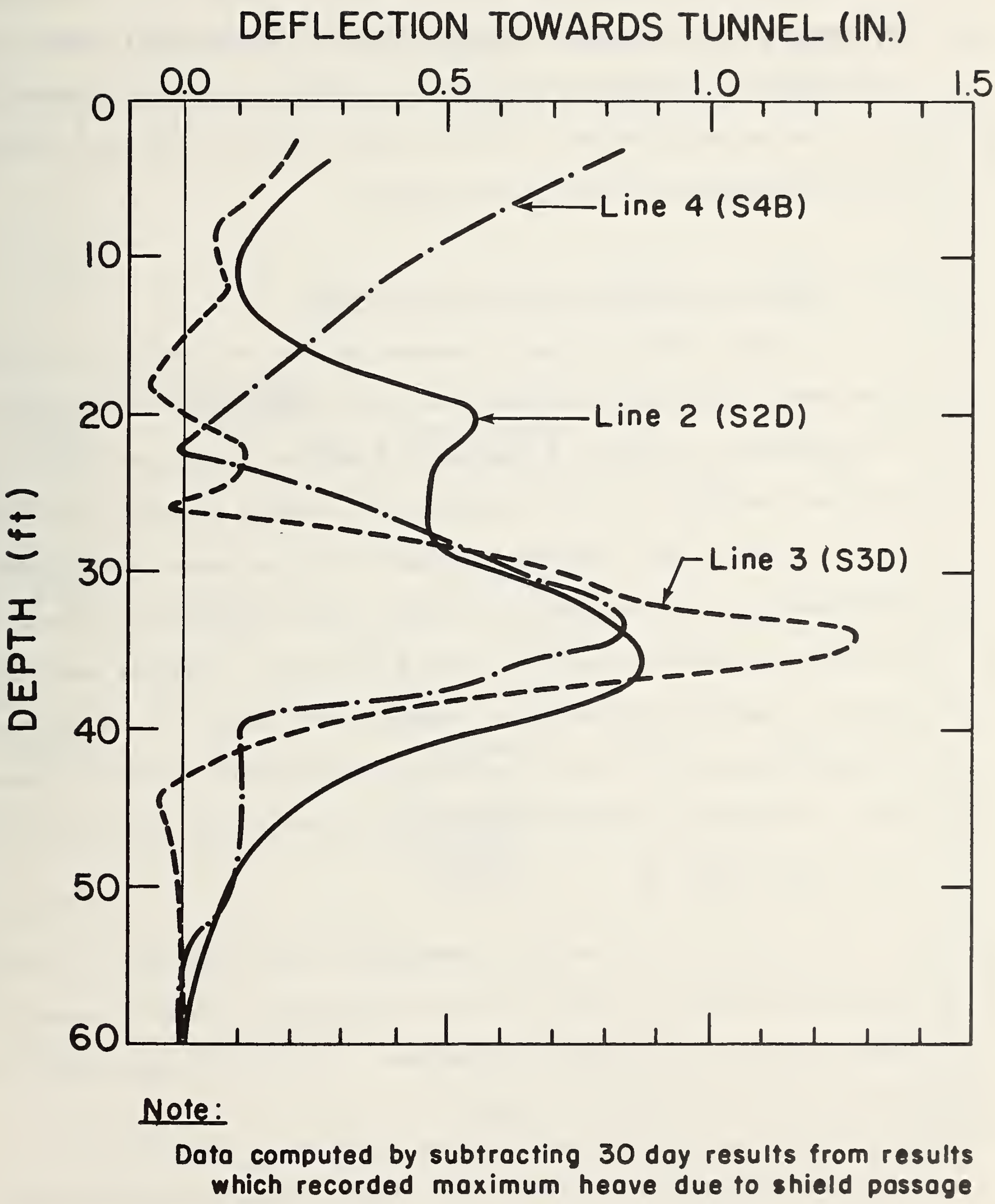


Figure 5.33: Maximum Lateral Movements Toward Tunnel at Lines 2,3, and 4

although it had a broader impact with depth at Line 2 (see Figure 5.33). The special aspects of the response at Line 2 are most likely due to the excess pitch at this position (Figure 5.2). The large inward movements below the shield could be in part, due to the fact that the Bay Mud soils were deeper at Line 2 than elsewhere.

5.7.2 Combined Lateral And Vertical Movements

To compare the displacement response at the instrumentation lines, displacement vectors for vertical and lateral movements at Lines 2 and 3 are superimposed in Figure 5.34. Line 4 results (see Figure 5.23) are not shown here since they fall between the extremes defined by Lines 2 and 3. The vector comparison shows the previously well defined differences in lateral components, but it also illustrates that there is a reasonable similarity in the vertical movements. This is consistent with the fact that the measured surface settlements outside of work stoppage areas are of similar magnitude all along the alignment (Figure 5.26), regardless of differences in shield performance characteristics or soil conditions.

The similarity of vertical movements at the instrument lines is somewhat surprising in view of the differences in lateral movements. However, there is a plausible explanation. In order to develop it, the following points should be recalled:

1. In spite of the sometimes large lateral heaves which occurred in the Bay Mud around the tunnel during shield passage, little, if any, lateral or vertical displacement was observed in the rubble fill.

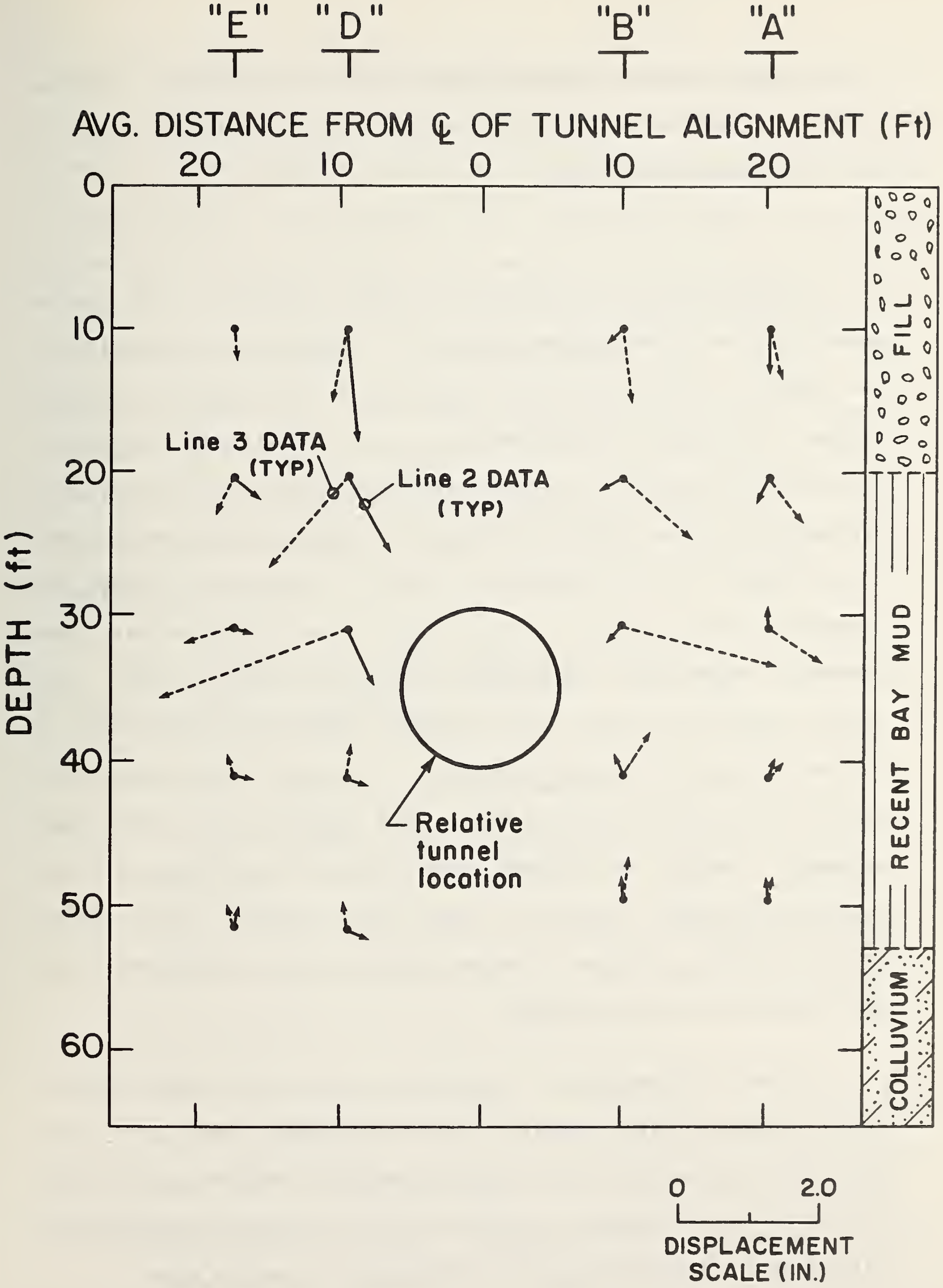


Figure 5.34: Transverse Displacement Vectors 30 Days After Shield Passage, Lines 2 and 3

2. Inward movements, which followed the tail void passage, occurred in both the rubble fill and the Bay Mud and were close in magnitude at all lines.

The results at all of the lines suggest that most of the heaving effect occurred in the Bay Mud directly in front of, and to the side of, the shield. This apparently reflects the fact that the Bay Mud alongside the shield is weak and has a very low stiffness relative to the overlying granular fill and the overconsolidated crust at the top of the Bay Mud. Rather than displace the fill upwards towards the surface, it was easier for the Bay Mud in front of the shield to push the neighboring Bay Mud aside. As a result, the heaving effects were basically lateral and concentrated around the shield. Also, this suggests they were probably magnified over what they would have been if the overlying soil had been less stiff. It would appear that if the entire soil profile were composed of soft Bay Mud, that the outward movements from the shield would have been distributed over a wider volume than actually occurred. Larger heave movements at the surface would likely have occurred, and as a result smaller heaves would have been observed around the shield.

In the final analysis, the heaving of the soil during initial shield passage resulted largely in lateral movements, which were mainly confined to the Bay Mud. Following this phase of behavior, the soil was induced to move in response to the tail void, and possibly consolidation effects in the Bay Mud soils due to dissipation of excess pore pressures. The instrumentation data suggest that the Bay Mud moved into the tail void before it could be grouted. Subsequently, because the rubble fill is basically noncohesive, it apparently raveled or collapsed into the space left by the Bay Mud. Because the rubble fill had not displaced significantly during the heaving process, the net effect which showed up was only downward and inward displacements in this material, regardless of the degree of the initial heaving in the Bay Mud. And, since the downward and inward displacements were a function of the tail void, they were similar so long as the tail void was similar. As noted earlier, this seems to have been the case at Lines 2, 3, and 4, except for some small differences produced by variations in shield pitch which ranged from 0 to 1%.

This scenario of behavior depicted by the measured response has several significant implications:

1. The N-2 site conditions were somewhat unique because of the relationship between the soft Bay Mud and the stiffer overlying rubble fill. In a uniform clay soil, it could be expected that larger heaves would be observed at the ground surface during shield advance which might serve to help reduce the net ground settlements after tail void effects are felt.

2. Control of movements into the tail void remains a key issue for the EPB shield as it is in the conventional shield since this effect has a significant impact on the net surface settlement. This has been confirmed recently in three-dimensional finite element studies performed for research related to this effort (Kasali and Clough, 1982).

The final trend which should be mentioned in regard to the movements after shield passage concerns their time-dependent nature. As shown clearly in Figures 5.24 and 5.25 for Line 4, movements occurred for up to 150 days after shield passage (and may still be accumulating). To a certain degree this is true at all of the lines as can be seen in comparing the measurements taken at 5 and 30 days after shield passage. These time-dependent trends can be attributed partly to consolidation effects in the Bay Mud soils around the tunnel, and partly to gradual restructuring of the rubble fill in the loosened zone above the crown.

5.7.3 Volume Changes

Volume changes in the soil around a tunnel occur as a result of the generation of shearing and compressions stresses induced by the tunneling process. The volume changes can be calculated based on the vertical and lateral movement measurements as per procedures described by Hansmire and Cording (1975). For the N-2 project, these techniques can only approximately be applied since vertical movements were measured at ten foot intervals. The results of such a calculation for Line 3 are shown in Figure 5.35 for the movements observed 30 days after shield passage. In the Bay Mud soils around the crown and springline areas, a significant compression or volume reduction is indicated. The rubble fill as shown to have expanded in the section above the crown at the tunnel centerline, but to have contracted outside of this.

The volume reduction in the Bay Mud soils is most likely due to consolidation under the compressive effects of the outward soil movements generated during initial shield passage at Line 3. The

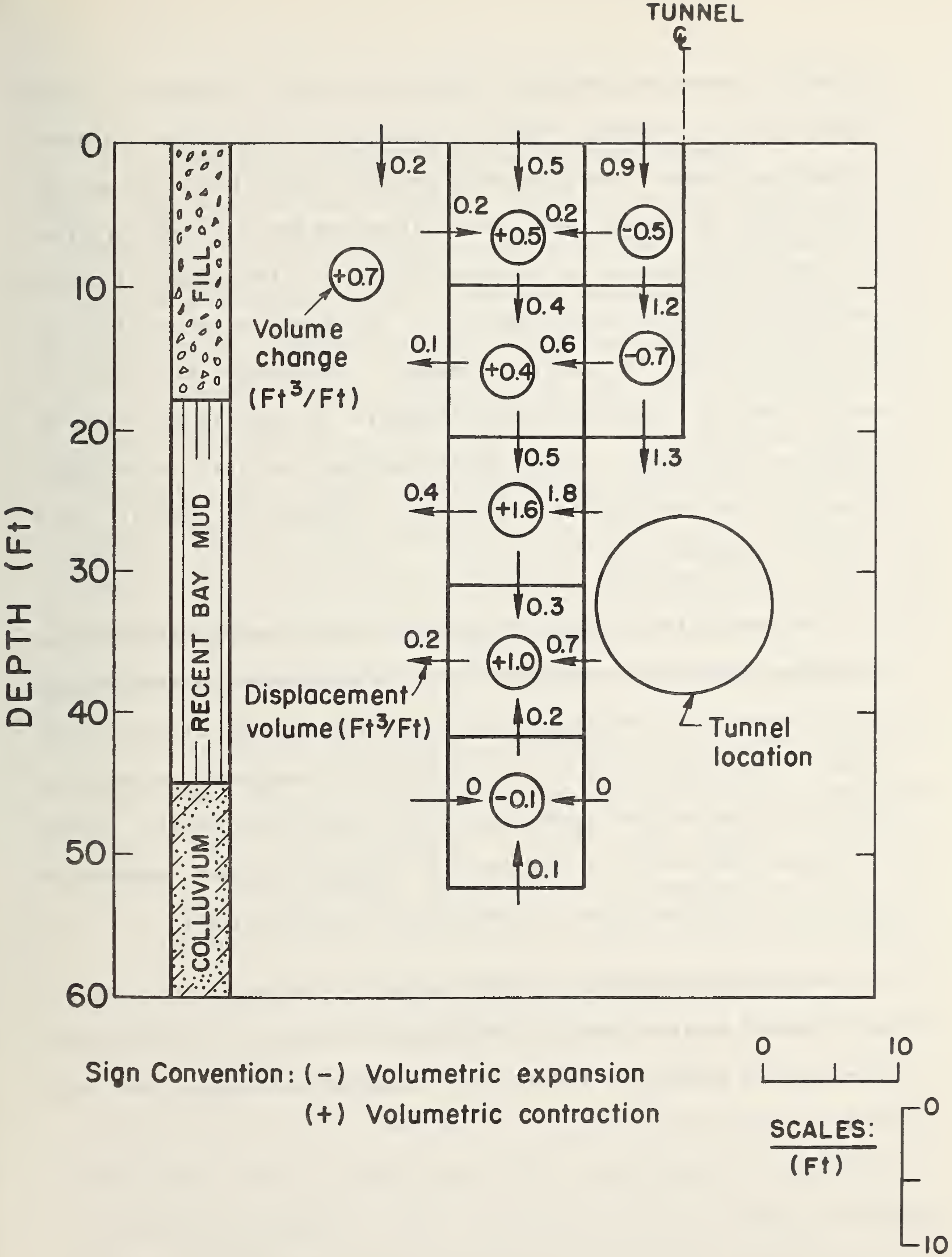


Figure 5.35: Displacement Volumes and Volume Changes 30 Days After Shield Passage, Line 3

volumetric expansion indicated immediately above the tunnel in the rubble fill is presumably caused by raveling of the fill as it moved downward and inward toward the space created by the tail void and the consolidation of the Bay Mud. This effect has been observed for other soil tunnels by Hansmire and Cording (1975). In this case, it may be somewhat surprising that the rubble fill can expand in volume given its presumably initial low density. However, cone penetration tests in nearby sites have shown the relative density of sands in the fills to range from 30 to 65%. Given these densities, and the low confining pressures which exist above the tunnel, expansion of the fill is a plausible phenomenon.

The contraction of the rubble fill indicated outside of the centerline area can be explained in terms of compression effects induced by the formation of the ground arch around the loosened zone above the crown. This area of the fill is subjected to compressive stresses as the soil tries to pick up the load which was formerly carried by the soil above the crown. A similar type response has been reported by Hansmire and Cording (1975) for conventional shield projects.

Volume change calculations using measured displacements at Lines 2 and 4 yielded similar trends to those shown for Line 3, although they were not as consistent because the measured settlements were more erratic.

5.7.4 Comparison of Movements at N-2 To Other Projects

The most obvious difference between the EPB shield and the conventional shield is at the face of the machine. The EPB shield has a rotating cutterhead; the soil at the face is supported by the portion of the cutterhead in contact with the ground, and the soil to soil contact at the cutterhead slots (assuming that the muck retaining area is properly full of soil at all times). If a perfect earth balance is maintained, there should be no movements towards or away from the EPB shield. However, in the case of a conventional shield, movements into the open face are inevitable. Cording and Hansmire (1975) report that upwards of 30% of the ground loss for conventional shields at overload factors of 5 to 6 may be due to displacements into the shield face.

In the case of the N-2 project, the field data clearly demonstrate that the movements around the face of the EPB shield in most cases were actually away from the machine. Model test results, analytical prediction and field data show that for a conventional shield the initial movements during shield passage are towards the shield (Casarin and Mair, 1981, Hansmire and Cording, 1975 and Ghaboussi and Ranken, 1977). The initial heaving behavior for an EPB shield is apparently not unique to the N-2 project. Kitamura, et al., (1981) presented surface movement data above the centerline of an EPB shield project in Osaka, Japan which clearly demonstrated initial upward ground displacement as the shield approached. Their observations were for both silt and clay soils. These data, plus those at the N-2 project, show that the EPB shield provides full face support, and can be used to even force the soil away from the face during the advance, a behavior in marked contrast to that for conventional shields. Assuming tail void effects to be equal, this suggests that the EPB shield should generate smaller settlements and ground loss than a conventional shield because of the difference in behavior at the face of the shields.

Using the survey measurements made for the tunnel alignment by the City of San Francisco and those obtained at Line 4 by the Stanford personnel, the ground losses at the surface were calculated at some 150 locations. The volume loss per foot was computed using an equation proposed by Schmidt (1969) and Peck (1969) which assumes the surface settlement profile to be shaped like the classical error function. The volume loss per foot, expressed as a percentage of the undeformed tunnel volume per foot, is plotted against overload factor in Figure 5.36. Also shown are data for tunnel projects in clay assembled by Schmidt (1969) and Clough and Schmidt (1977), as well as theoretical baselines established from elastic theory for closure into an opening in homogeneous clay.

The results for the N-2 data are subdivided into those representing typical behavior and those related to the two prolonged work stoppages. The ground losses for typical behavior range from 0.6 to 5% while those for work stoppage areas are from 5 to 7%. Median values for both cases are 3% and 6% respectively. The larger settlements in locations where prolonged work stoppages occurred have been previously explained as due to movements towards the face of the machine, movements similar to those

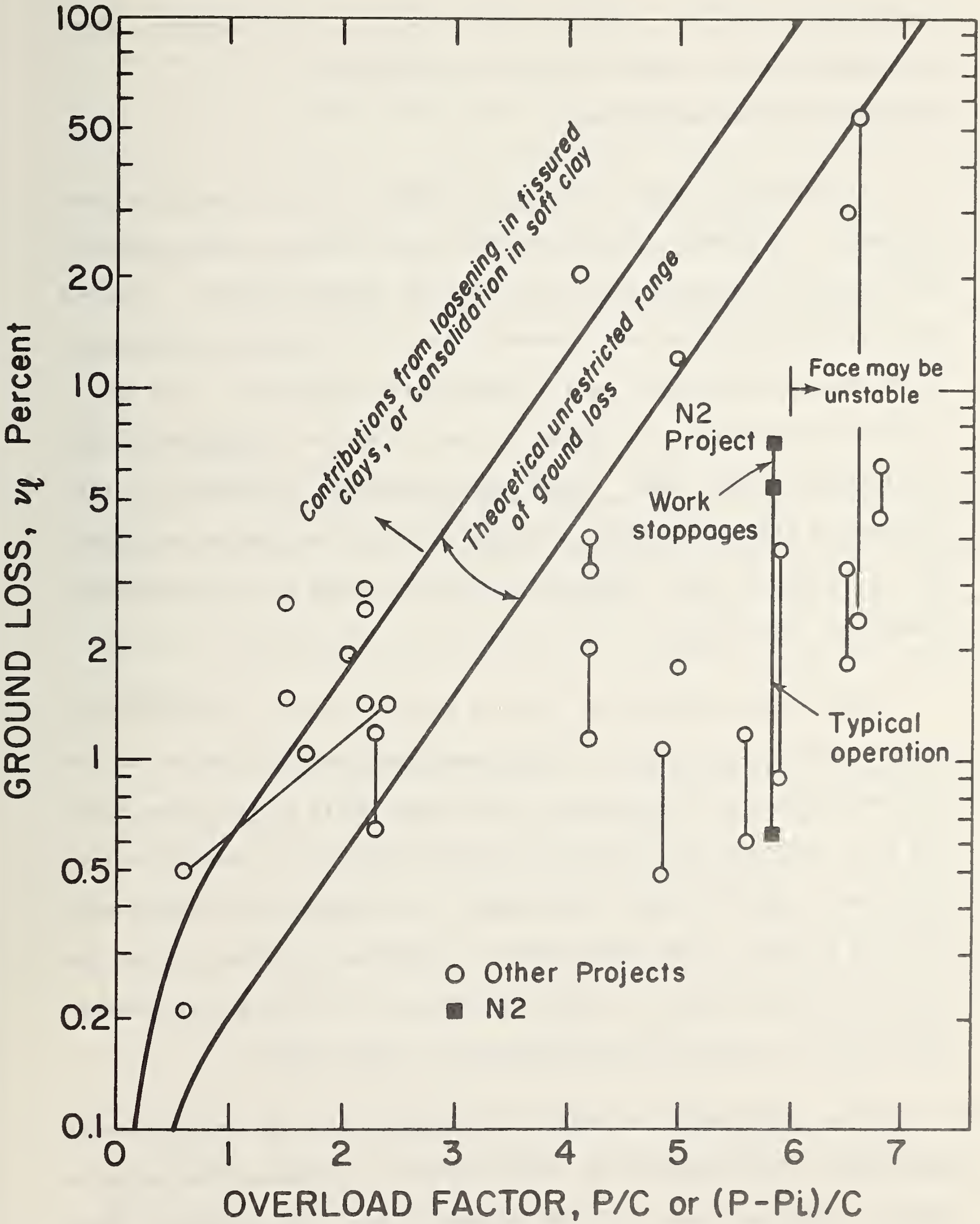


Figure 5.36: Overload Factor versus Ground Loss (adapted from Clough and Schmidt, 1977)

which might occur for a conventional shield. It is apparent these displacements play a significant role in increasing the ground lost at the surface.

The comparative data from other projects shown on Figure 5.36 are scattered. In general, the ground losses representing the mean value of the typical N-2 behavior are on the low side of these results. The N-2 data from the prolonged work stoppage areas are more akin to the average of the conventional shield data. Perhaps most importantly, there are no isolated large ground loss points for the N-2 project, as exists for the conventional shield data. These cases represent a collapse or near collapse of these tunnel face, an event which did not develop on the N-2 job in spite of the sometimes difficult nature of the subsurface conditions at the site.

The general agreement of the N-2 results for the prolonged work stoppage areas with those of the conventional shields may be due to similar ground support conditions, but it also could be fortuitous since the soil conditions at the site of the N-2 project are somewhat unique given the overlying rubble fill layer. It remains to perform more detailed studies using finite element techniques to better define the relative roles of the Bay Mud and rubble fill to the performance. Results of this type will be presented in a future report.

It is also useful to compare the shape of the surface settlement profiles to those observed for other projects. Assuming the available surface settlement data fit the commonly used error function shape (Schmidt, 1969), distances from the centerline were calculated to the inflection point of the settlement profile. For all of the N-2 data, the average value is 13.5 ft. (4.1 m). Based on a compilation of data of this type for conventional shields by Cording and Hansmire (1975), a typical value of 13 ft. (3.9 m) is obtained. Thus, it appears that the settlement profile for the EPB shield is very similar to that for a conventional shield. This is not surprising in view of the fact that the principal contribution to the settlements for both machines is the tail void closure.

5.8 SUMMARY

In the preceding sections of this chapter the details of the observed ground movements for the N-2 project are presented and compared to those for other soil tunnels. The results suggest that the primary difference between a conventional shield and the EPB shield is in the face support mechanism.

The EPB shield on the N-2 project was operated so as to generate a small heaving effect in the soil as it advanced. At the locations of Instrument Lines 1 and 2, the shield caused outward movements as high as 0.5 in. (1.3 cm) as the face of the shield passed. These displacements were largely lateral in nature, and were almost exclusively confined to the soft Bay Mud soils around the shield. Little effect of the advance was observed in the rubble fill overlying the tunnel. It appears as if the rubble fill had enough strength to resist the tendency for deformation, and caused a concentration of the heaving effects in the Bay Mud soils to the sides of the shield.

More pronounced heaving phenomena were observed at Lines 3 and 4 than Lines 1 and 2, with maximum outward movements of 3.5 in. and 2 in. (8.9 and 5.1 cm) being measured respectively. Again, the heaves were largely lateral and confined to the soft Bay Mud soils, with little effect observed insofar as the rubble fill or the ground surface was concerned. The reason for the large heaves is apparently related to the effects on the EPB shield of the old wooden piles, which were encountered at or very near Lines 3 and 4. During shield advances in these areas, unusually high bulkhead earth pressures were observed, values 1.5 to 2.0 times those near Lines 1 and 2. It is possible the pile fragments entering the spoils retaining area caused clogging of the screw auger, and prevented passage of the normal volume of soil. With more soil trying to enter the spoils retaining area than could get out, the bulkhead pressures would rise and more heaving of the soil outside of the shield would occur.

After shield passage, immediate inward movements of the soil around the shield occurred at all lines, in response to the presence of the tail void. The inclinometers actually reflected the effects of the tail void before the void itself reached the instrument line. The first response involved a movement of the inclinometer in the direction opposed to shield movement as the soil moved towards the void at the rear of the shield. When the tail was more aligned with the instrument line, the inclinometers showed a displacement more directly into the centerline of the tunnel.

The vertical and lateral movements associated with the tail void were in many ways similar at all lines. First, the maximum amount of lateral displacement inward to the tunnel was similar, ranging from 0.8 to 1.2 in. (2 to 3.1 cm). Second, the maximum settlement which occurred in the rubble fill above the tunnel crown, were of comparable magnitudes and distributions. Third, the movements due to the tail void occurred in both Bay Mud and rubble fill. Finally, after the immediate response, the inward movements continued with time, although at a diminishing rate. Perhaps the most interesting of all of these trends is that the tail void movements were all similar regardless of the differences observed in initial heaves which occurred as the face of the shield passed. This behavior is attributed to the fact that the initial heaves were largely concentrated in the soft Bay Muds as lateral displacements, which had only a small impact on the rubble fill.

Ground surface settlements measured at over 150 locations along the alignment ranged from 0.2 in. to 3.0 in. (0.5 to 7.6 cm), with a median value of 1.3 in. (3.3 cm). The largest settlement values generally corresponded with areas where a major work stoppage occurred. The sharp increases in settlements at these locations apparently were induced by a partial loss of the face support pressure which resulted in the EPB shield performing more like a conventional shield. Comparing ground loss volumes computed from surface settlement measurements ranged from 1 to 5% of the tunnel volume during the typical advancing shield case, but from 5 to 7% during work stoppage areas. Given that the overload factor for the N-2 tunnel is close to 6, the typical behavior shows lower losses than for most conventional shield projects, while those for the work stoppage areas are akin to the median expected for conventional shields. In general the ground losses are indicative of a good ground control. The face support provided by the EPB shield exerted a very positive influence on the performance.

Chapter 6

SUMMARY AND CONCLUSIONS

Control of ground movements and ground water during tunneling in soil are two of the major issues concerning construction in an urban area. Over the past 5 to 10 years new tunneling equipment has been developed to help minimize any detrimental effects related to these issues. One of the most promising of the new machines is the earth pressure balance (EPB) shield; it was used for the first time in the United States in 1981 to construct a tunnel for the San Francisco Clean Water Program. In order to develop a better understanding of the ground support mechanisms this shield can actually provide, the San Francisco tunnel, termed the N-2 contract, was instrumented to define the nature of the ground movements which occurred during and after shield passage. This report presents the data obtained and an analysis of the observed behavior.

The east-west tunnel alignment stays entirely within the boundaries of North Point Street, which passes through a heavily developed area in the heart of the Fisherman's Wharf section of San Francisco. The topography along the 3000 ft. (909 m) long, 12.1 ft. (3.7 m) diameter, tunnel is generally level; ground surface elevations range from +15 ft. (4.5 m) at the western portal to +2 ft. (0.6 m) at the eastern portal.

The subsurface conditions along the alignment consist of an upper layer of rubble fill of about 25 ft. (7.6 m) thickness underlain by generally 30 ft. (9.1 m) of soft sediments known locally as Recent Bay Mud. At this site the Bay Mud is largely silt and silty clay. Underlying the Recent Bay Mud is a stratum of sandy clay. Ground water is encountered at a depth of approximately 10 ft. (3.3 m) below ground surface. Wooden piles from abandoned wharf structures are located along the tunnel alignment and presented obstacles to the EPB machine.

The tunnel passes entirely within the Recent Bay Mud, except near the western terminus, where the invert encounters the lower sandy clay stratum. Ground cover above the crown varies from 20 to 30 ft. (6 to 9.1 m) and consists mainly of the rubble fill. The ground water level is typically 10 to 20 ft. (3 to 6 m) above the tunnel crown.

The EPB tunneling machine operates by bringing excavated soil into the shield through slots in its rotating cutterhead and storing it in a retaining area located between the cutterhead and a bulkhead, about 4 ft. (1.2 m) behind the cutterhead. The soil is removed via a screw auger which takes it through the bulkhead and deposits it onto a conveyor belt. Ideally, the cutterhead brings soil in at the same rate that the screw auger removes it such that the retaining area remains full at all times thus maintaining an earth balance. Control of this operation is crucial to the proper performance of the shield. In the N-2 project the control was exercised by an operator working in a semi-enclosed booth which was a part of the power train trailing the EPB shield. From the shield monitoring devices the operator is able to detect key parameters concerning the equipment functions. One of these items is the earth pressure detected by an earth pressure cell set onto the inner bulkhead. This cell gives an indication as to whether the spoils retaining area is full or not. If the cell reads too low a value, the operator knows the spoils area may contain a void, and that he needs to speed up the cutterhead or slow down the screw auger. If the cell reads too high a value, the operator can anticipate a problem whereby the muck is trying to come in faster than it is being withdrawn by the screw auger.

The average progress rate for the shield advance was 30 ft. (9.1 m) per day with a maximum rate of 100 ft. (30 m) per day. The shield successfully cut through an estimated 90 wooden piles which had been left in place from nineteenth century structures. Special carbide teeth had been added to the cutterhead for this purpose.

Four instrumentation lines were established along the tunnel alignment. Lines 2, 3 and 4 were monitored throughout the tunneling process. Readings at Line 1 were observed less frequently. Each line of instrumentation consisted of five 60 ft. (18.2 m) long inclinometer casings equipped with telescoping settlement couplings at 10 ft. (3 m) depth intervals. The casings were set at approximately 10 ft. (3 m) intervals perpendicular to the tunnel alignment with the middle inclinometer approximately on the centerline of the tunnel. This array was designed to allow lateral and vertical soil movements to be measured in front of and adjacent to the tunnel. Surface settlements were monitored by survey measurements made at about 150 locations along the tunnel alignment. In addition to the ground movement instrumentation, a limited number of strain gages and extensometer points were set onto the tunnel liner at the instrumentation lines to observe tunnel liner response.

The intensive reading of Lines 2, 3 and 4 during and following shield passage allowed the effects of the EPB shield on the ground to be closely tracked. The results show certain unique differences between Line 2 and Lines 3 and 4 behavior.

The inclinometer measurements perpendicular to the tunnel axis obtained at Line 2 indicate the Recent Bay Mud soils were laterally displaced up to 0.6 in. (1.5 cm) away from the tunnel (heaved) as the shield face passed the inclinometers. At the same time, no significant movement was observed in the overlying fill. When the tail of the shield, and thus the tail void, passed the line, the Bay Mud and the rubble fill displaced back towards the tunnel such that an immediate net inward movement up to 0.2 in. (0.5 cm) resulted. Data obtained 5 and 30 days after shield passage show continuing inward lateral movements up to 0.5 in. (1.3 cm) at the contact between the fill and Bay Mud. The general trends for all inclinometers are similar except the magnitudes of the displacements are not equal; the further the distance of the inclinometer from the tunnel, the smaller the recorded movements.

Examination of combined lateral and longitudinal displacements shows that the Bay Mud heaved longitudinally and laterally from the face of the shield as it approached. As the tail void passed, both the Bay Mud and fill begin to displace both longitudinally and laterally towards the tail void. The final movements are inwards toward the tunnel and away from the direction of the shield movement.

Long term, combined lateral and vertical movements in a plane perpendicular to the tunnel axis at Line 2 show that the general pattern is for points above the tunnel to move down as much as 1.5 in. (3.8 cm) and in towards the crown, while those below move up and in towards the invert.

Results from Line 1, although taken less frequently than at Line 2, were consistent with those at Line 2. It appears that for practical purposes behavior at these two lines were similar.

Inclinometer measurements perpendicular to the tunnel axis obtained at Lines 3 and 4 indicate the Bay Mud soils adjacent to the shield heaved from 2.0 in. (5.1 cm) to 3.5 in. (8.9 cm) at Lines 4 and 3 respectively as the shield face passed the inclinometers. When the tail void passed the lines, incremental lateral movements in the Bay Muds and the overlying fill, of 0.9 in. (2.3 cm) at Line 3 and 0.7 in. (1.8 cm) at Line 4, occurred toward the tail void. These displacements leave the inclinometers displaced away from the tunnel by as much as 2.6 in. (6.6 cm) and 1.3 in. (3.3 cm).

Examination of combined lateral and longitudinal displacements at both lines 3 and 4 shows the Bay Mud heaved longitudinally and laterally from the face of the shield as it approached. As the tail void passed, the soil moved inward and against the direction of the shield advance, but remained pushed beyond its initial position because of the large initial heaves.

Results of long term, combined lateral and vertical movements in a plane perpendicular to the tunnel axis at Lines 3 and 4 show that the general pattern of movements for points above the tunnel is to move down and away from the crown while those below show a small net heave. The maximum settlement was measured to be 1.3 in. (3.3 cm) above the crown of the tunnel in the Bay Mud at Line 3 an 0.8 in. (2.0 cm) above the crown of the tunnel in the fill at Line 4.

To evaluate the differences in observed response between the four lines of instrumentation, two key shield operation parameters, the bulkhead earth pressure and shield pitch were examined. The bulkhead earth pressures were particularly revealing. The highest values along the entire tunnel alignment were recorded in the vicinity of Line 3. Relatively high values were also observed at Line 4, while those at Lines 1 and 2 were consistent with the typical pressures along most of the alignment. Values for pitch of the shield showed the shield to be essentially level at Lines 1, 3 and 4, but to be pitched upward at an angle of one degree at Line 2.

The levels of the initial lateral heaves which occurred at the instrument lines correspond well with the relative magnitudes of the bulkhead earth pressures. The highest heaves were observed at Line 3 while the next highest occurred at Line 4. Heave values at Lines 1 and 2 were lower than at 3 or 4. The reason for the differences in bulkhead pressures and the resulting heaves apparently is related to conditions which developed during periods when the shield cut through the wooden piles. At both Lines 3 and 4 wooden piles were encountered, and apparently the pile fragments served to partially clog the screw auger. This leads to a situation where soil is trying to come into the spoils retaining area at a faster rate than it can be removed, resulting in a build-up of pressure against the bulkhead.

After the initial heaves, all of the lines show the soil to move towards the shield as the effect of the tail void is felt. These movements differed from the initial heave behavior not only in direction, but also in that both the rubble fill and the Bay Mud were involved, and vertical as well as lateral movements occurred. The degree of inward movement also tended to be similar at all lines, although at Line 2, they tended to be distributed over a broader area around the shield, probably reflecting the effect of the upwards pitch of the shield at Line 2.

The similarity of the post heave movements at all lines may seem somewhat surprising in view of the large differences in initial heaves. This phenomenon is thought to be caused by two factors: (1) The character of the soil profile; and, (2) the relatively uniform tail void volume at all lines. The soil profile is important because the soft Bay Mud was apparently more receptive to distorting under the pressures exerted at the shield face than the overlying, and apparently stiffer, rubble fill. This led to the heaving effects being concentrated in the Bay Muds and showing up as largely lateral movements. As a result, the rubble fill was relatively undisturbed until the tail void was filled by the Bay Mud surrounding the upper half of the shield. As this occurred, the noncohesive rubble fill then raveled into the area vacated by the

Bay Mud, resulting in the vertical and lateral movements in the rubble fill. These movements were largely uniform at the different lines since the tail void was similar in all cases.

To obtain an overall view of the ground movements induced by the EPB shield process, surface settlements measured 5 months after shield passage along the entire tunnel centerline were examined. There are several noticeable trends. Between Sta 0+00 and 5+00, settlements are quite variable. This can be attributed to the tunnel crews becoming familiar with operating techniques during this time. The maximum settlement of 3.0 in. (7.6 cm) was recorded at Sta 1+60; this represents the effect of the 15 day work stoppage required for connecting the EPB power train to the shield. Between Sta 5+00 and 23+00, settlements are generally uniform and averaged 1.2 in. (3.0 cm). This reach appears to reflect ground response to EPB tunneling under typical conditions. Note that the surface movements were uniform in spite of the fact that some 90 wooden piles were encountered by the shield at Stations 13+00 to 23+00. Between Sta 23+00 and 30+00, settlements gradually increased towards the end of the job. A sharp increase of 2.6 in. (6.6 cm) in the settlement data was noted at Sta 23+40. This peak was caused by a work stoppage where the spoils retaining area was emptied of soil, so that the slots in the cutterhead could be cleaned of wooden debris from the piles encountered along the route. Other larger settlements near the end of the job cannot be attributed to any one factor, although there were some labor problems during this reach. The measured pitch of the shield was higher than usual, and the tunnel invert encountered the sandy clay stratum near Sta 28+00.

The general magnitudes of ground losses observed at the ground surface for the areas where the EPB shield was progressing regularly were on the low side of those expected for conventional shields in conditions similar to the N-2 tunnel. This can be attributed to the fact that there are no ground losses generated with the EPB shield due to movements into the face of the shield. The larger ground loss areas which were observed when the shield was stopped for 15 days or where the spoils retaining area was emptied are consistent with average behavior of a conventional shield. This is, in part, explainable by the likelihood that movements towards the face of the EPB shield occurred under these conditions. The instrumentation on the tunnel liner at each line consisted of four pairs of vibrating wire strain gages and eight extensometer reference points. Strain gages were intended to monitor trends in loads acting on the liner. The extensometer reference points were set to allow measurement of liner diameter changes. The strain gage data were not numerous, nor consistent enough to allow actual behavior trends to be defined. Extensometer data indicated the liner gradually increased horizontal and decreased vertical diameters with time. Relative distortions of the liner are similar to those reported in the literature.

Ground water levels outside of the tunnel alignment were unaffected by the EPB shield process. The only ground water level changes noted were in areas where dewatering occurred due to shaft construction. Locally, high pore pressures were developed in the Bay Mud in front of the shield just before passage. At each of the instrument lines the water level in the inclinometer casing on the centerline of the tunnel rose rapidly as the shield came close. At Line 3 the water actually flowed out of the casing, probably reflecting the large heave effects at this location.

Conclusions based on the data obtained during the field instrumentation are as follows:

1. The Recent Bay Mud soils at the site, which were composed largely of silts and silty clays, proved to be an ideal tunneling medium for the EPB shield. They were cohesive enough to prevent water flow through the screw auger, but not so cohesive as to be sticky and clog the equipment.

2. Even though the EPB shield operated under about 10 ft. (3 m) to 20 ft. (6 m) of water head along the entire alignment, there were no problems with the ground water. No ground water level changes could be attributed to the shield other than very local rises due to excess pore pressures set up during an advance of the shield.

3. The ground movement instruments generally performed well. Lateral displacements were very consistent even where movements were small. The vertical movement sensors were much less sensitive than the lateral sensor system, but the final vertical displacements were accurate enough to define the trends of the ground response to tunneling operations.

4. As the shield approached each line of instrumentation, the first response was for movements away from the shield to occur. These increased until the face of the shield had passed, and they were

largely lateral and confined mainly to the soft Bay Mud soils around the shield. The degree of the lateral movements was directly proportional to the measured bulkhead earth pressure.

5. The passage of the tail of the shield led to inward movements at all lines. In contrast to the initial heaves, the inward movements had both lateral and vertical components, and were observed in the Bay Mud as well as the overlying rubble fill. The maximum levels of the movements were similar at all lines although the distribution varied with pitch of the shield.

6. Net lateral movements were a function of the relative magnitudes of the initial heave effects and the subsequent inward displacements due to the tail void. At Lines 3 and 4 the initial lateral movements were very large and the net lateral positions of the inclinometer casings were away from the tunnel in the Bay Mud soils. The opposite was true at Lines 1 and 2.

7. Little in the way of vertical heave was observed as the shield face approached the instrument lines apparently because the rubble fill layer overlying the Bay Mud was stiff enough to force the deformations to be restricted to the Bay Mud soil. The rubble fill underwent settlements with passage of the tail void however since it could ravel into the area above the subsiding upper surface of the Bay Mud.

8. Surface settlements measured along the centerline of the tunnel averaged 1.3 in. (3.3 cm). The settlements were uniform except

where the normal tunneling operations were disrupted; when this occurred, higher settlement values were measured. This effect appears to be caused by movements toward the shield face, as occurs in the conventional shield.

9. When the EPB shield was operating smoothly, the resulting surface settlements were on the low side of those reported for conventional shields.

The results of this instrumentation program show that the EPB shield at the N-2 project had a generally successful performance. Ground movements were held within limits which were typically smaller than those which would be expected with a conventional shield, and ground water loads were largely unaffected by the tunneling. In order to maximize the potential beneficial effects which can be attained with the EPB shield, there are a number of areas which deserve further research:

1. An investigation of the levels of heave which should be induced at the face of the shield to minimize detrimental ground movements in various types of soil.

2. An examination of the long term response of clay soils to effects of the EPB face support mechanism.

3. Additional field documentation of EPB and slurry shield projects to define more clearly typical trends of behavior.

Appendix A

INCLINOMETER READINGS

This appendix contains documentation on key inclinometer readings taken at Lines 2, 3 and 4 during and after shield passage. For reference, the following information on shield location is also provided:

TABLE A.1

Date and Time of Shield Passage

Line	Station	Date Shield Passed	Approximate Time of Passage
2	9+25	22 Apr 81	1450
3	13+60	30 Apr 81	1200
4	20+50	14 May 81	0500

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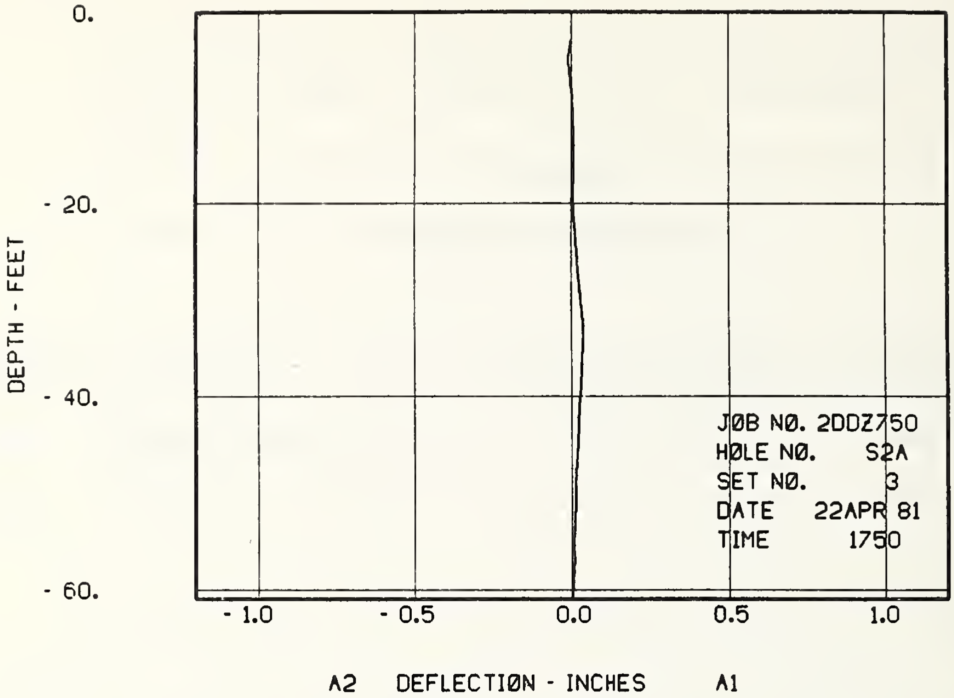

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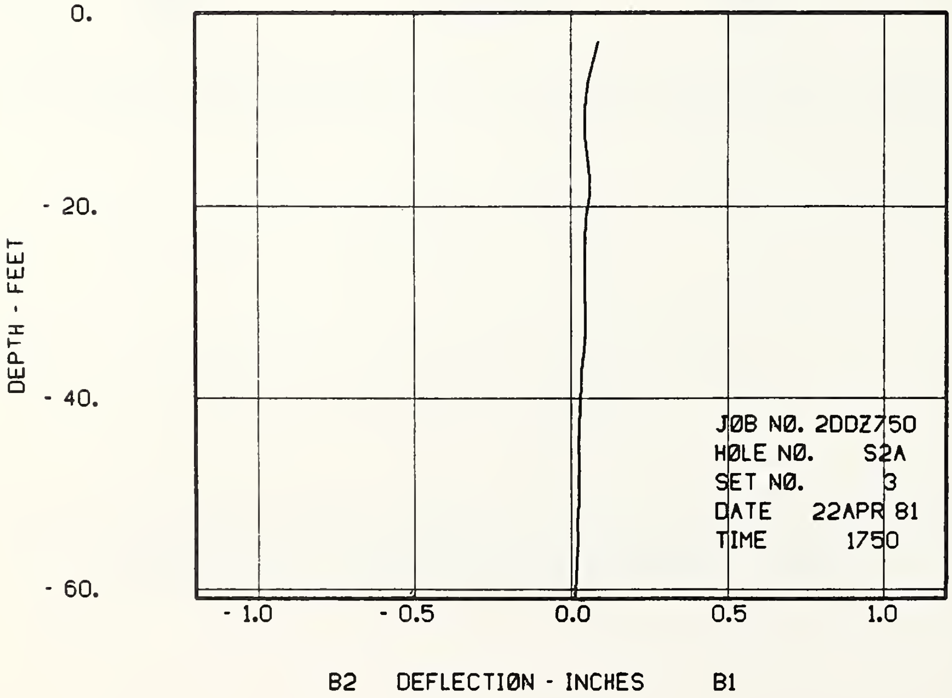

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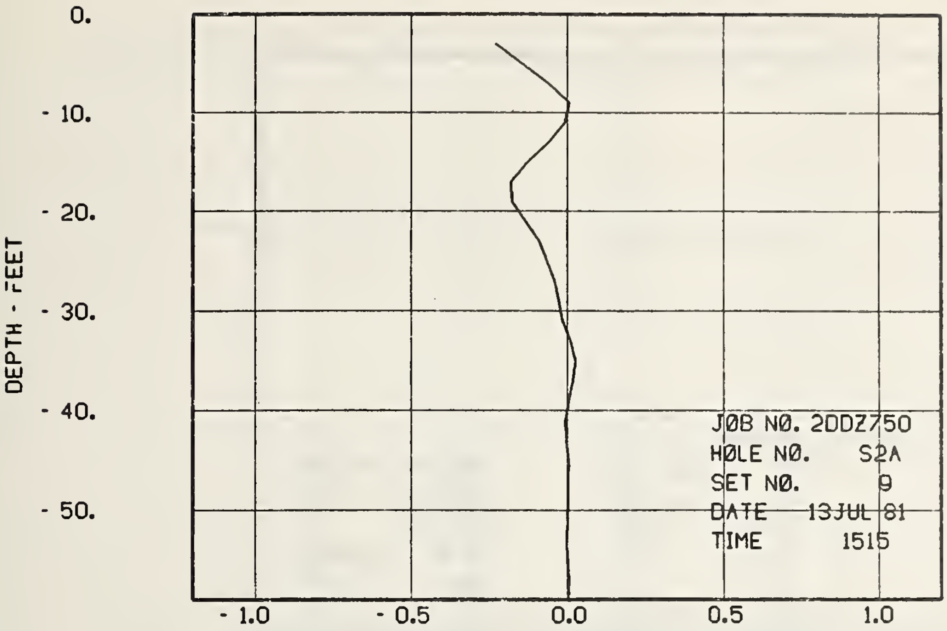

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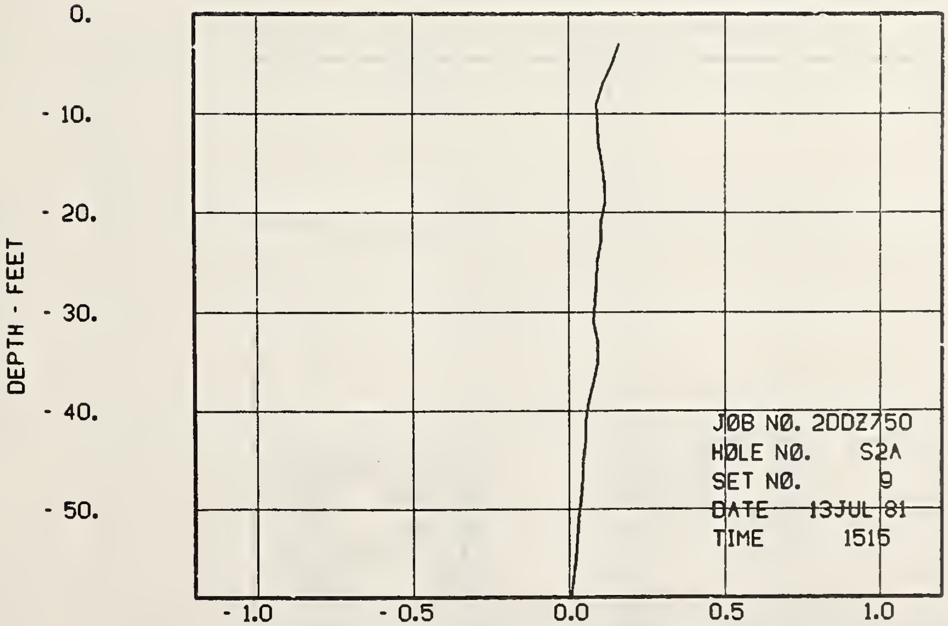

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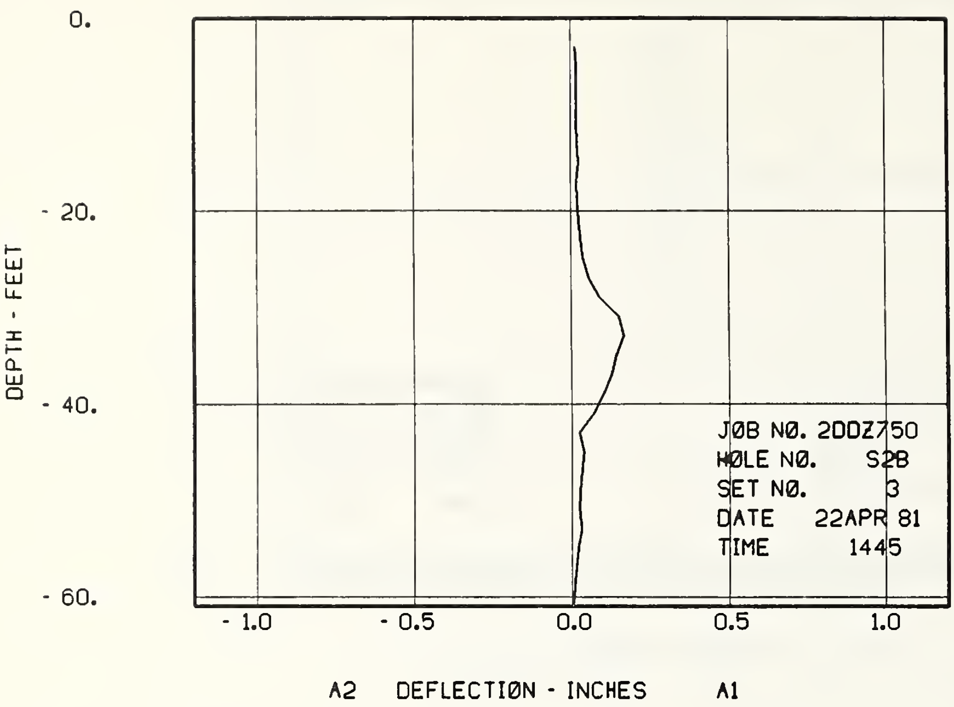

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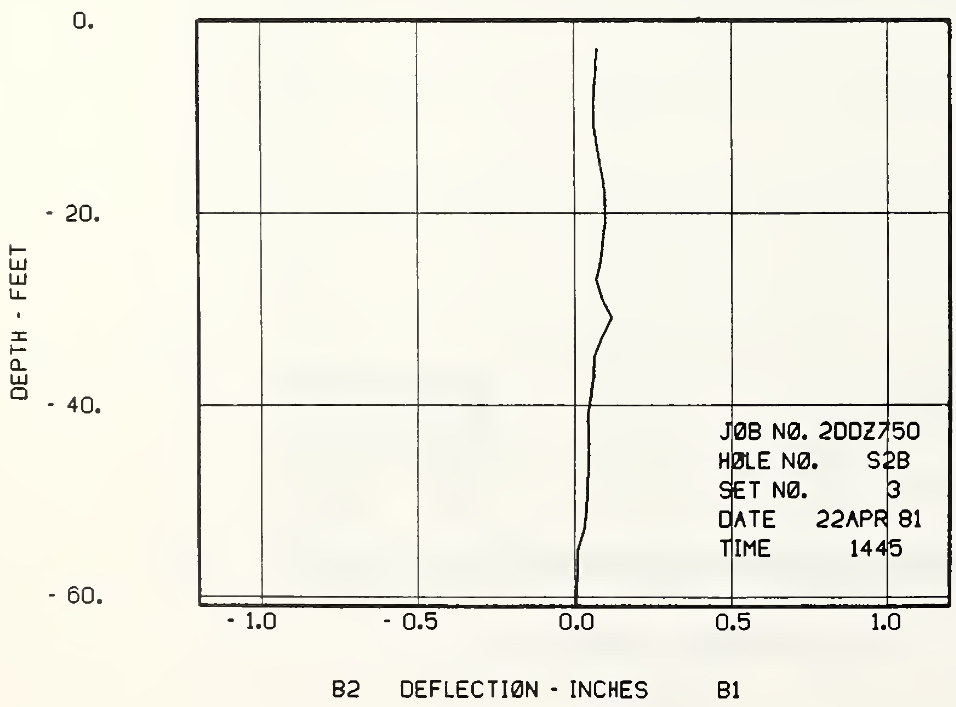

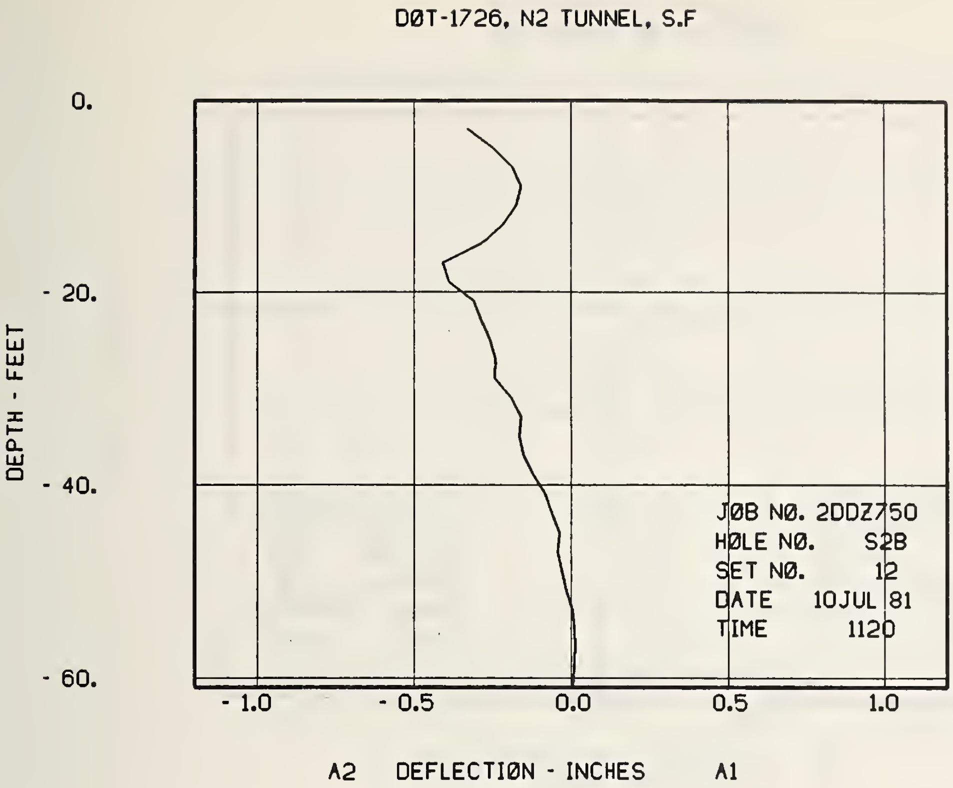
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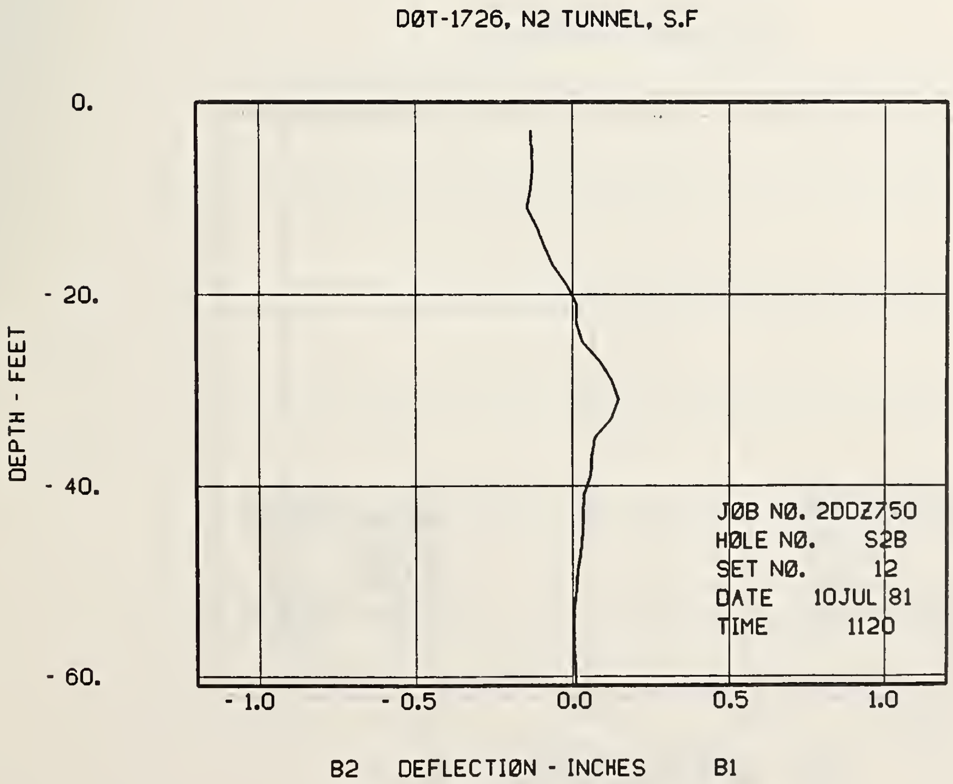
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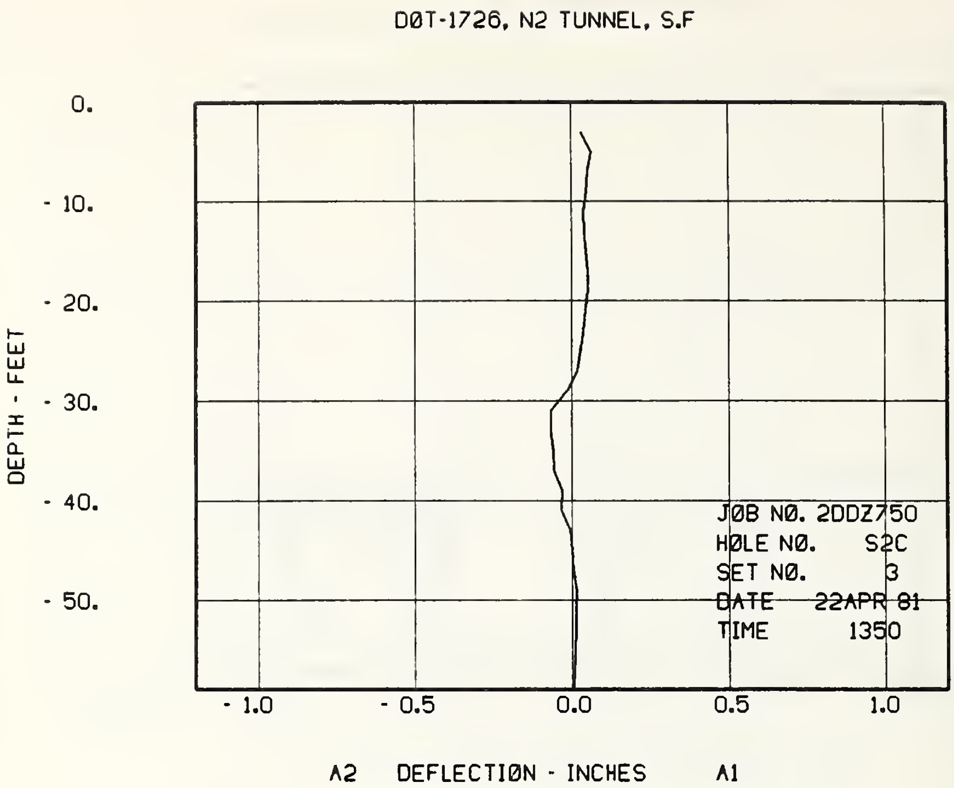
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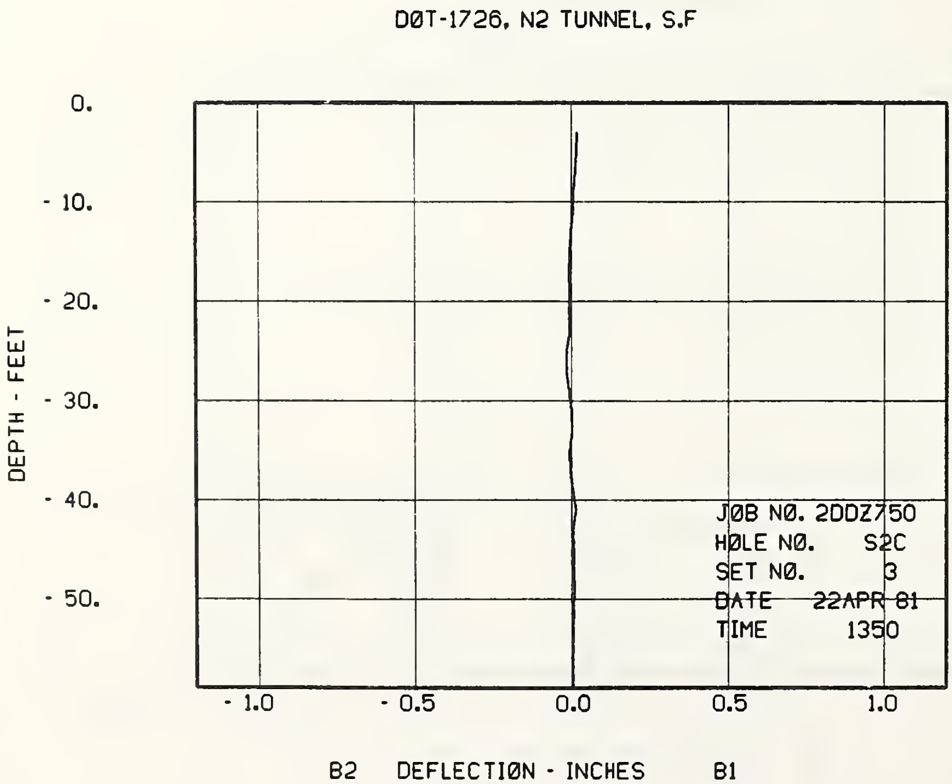
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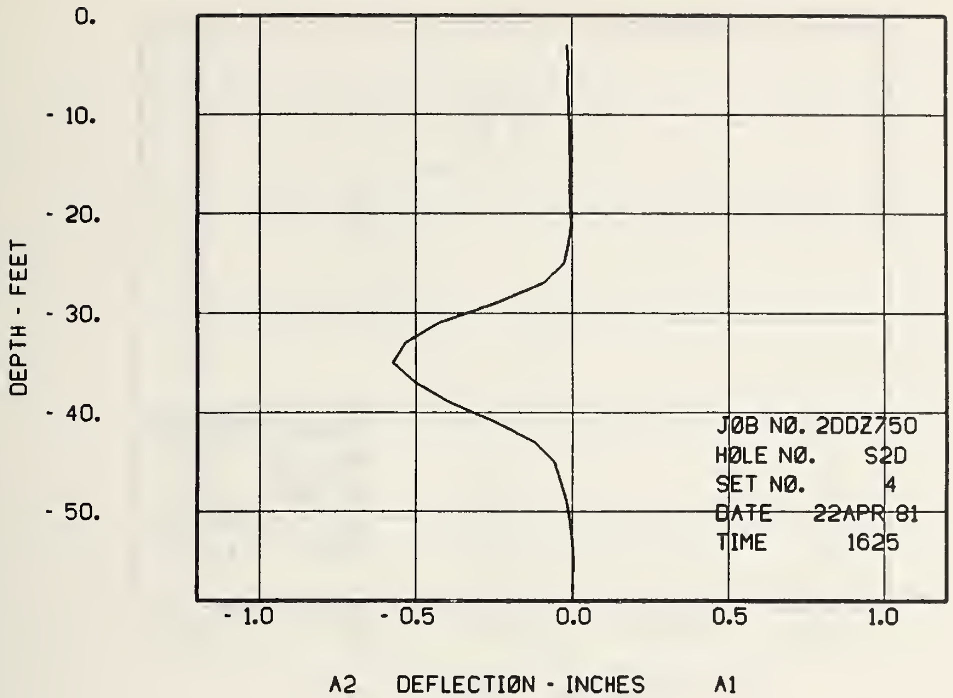

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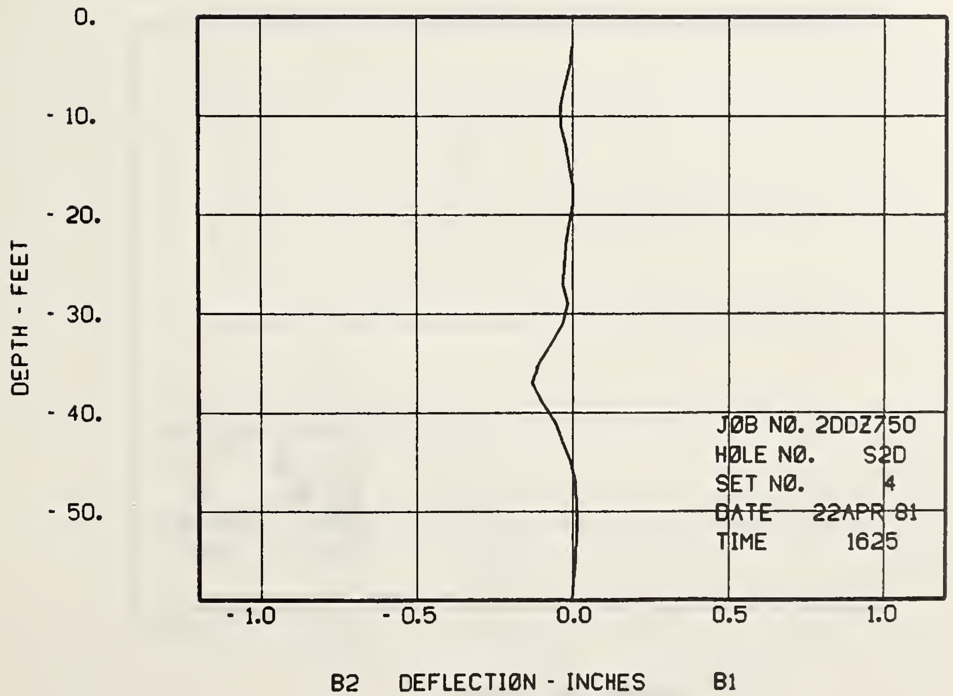

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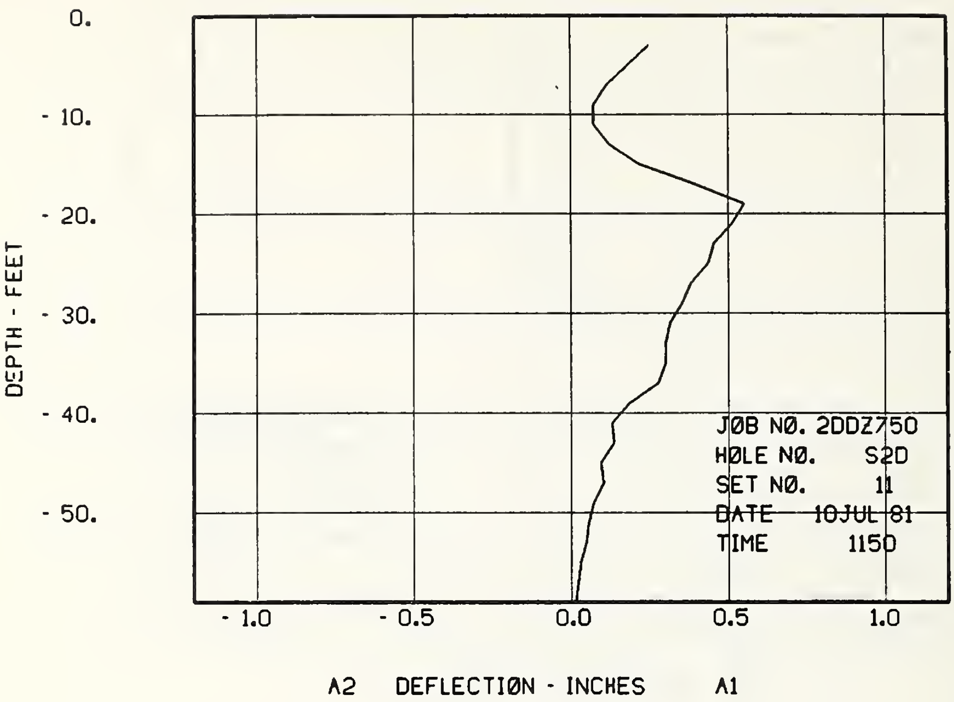

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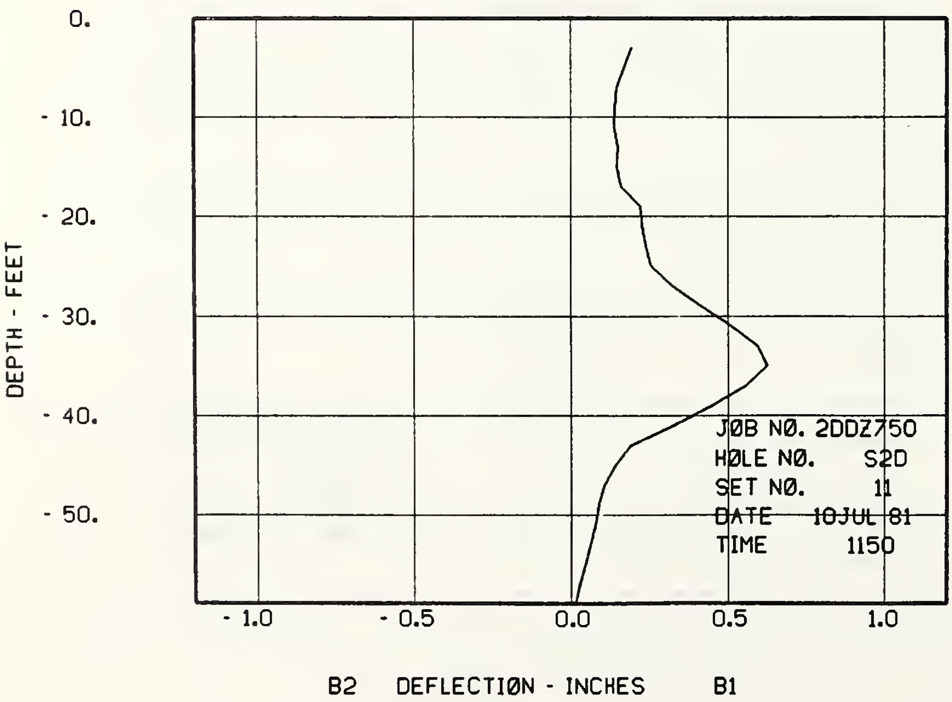

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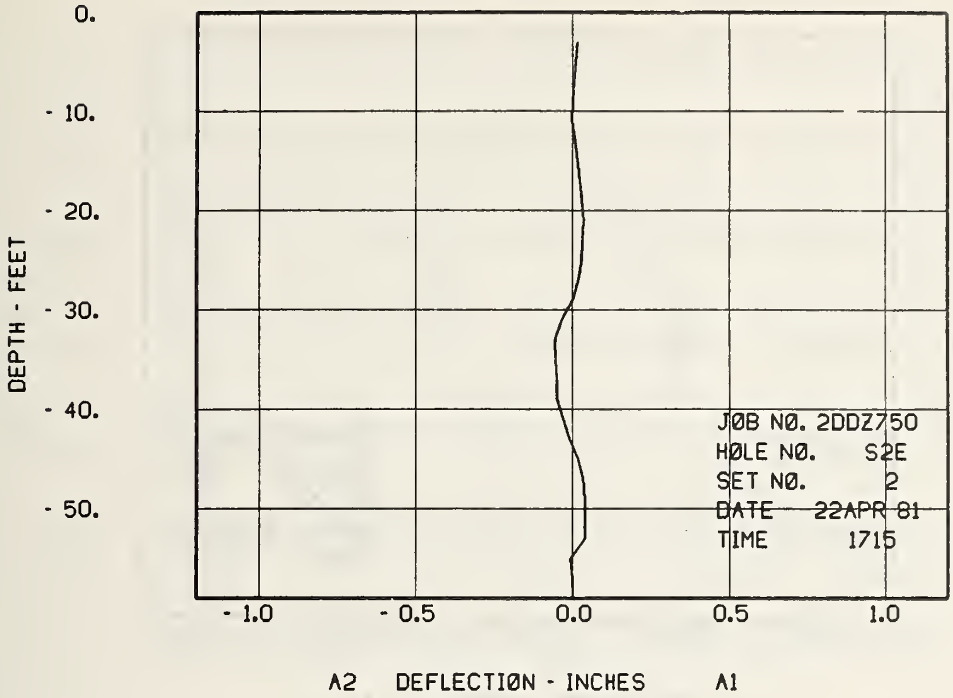

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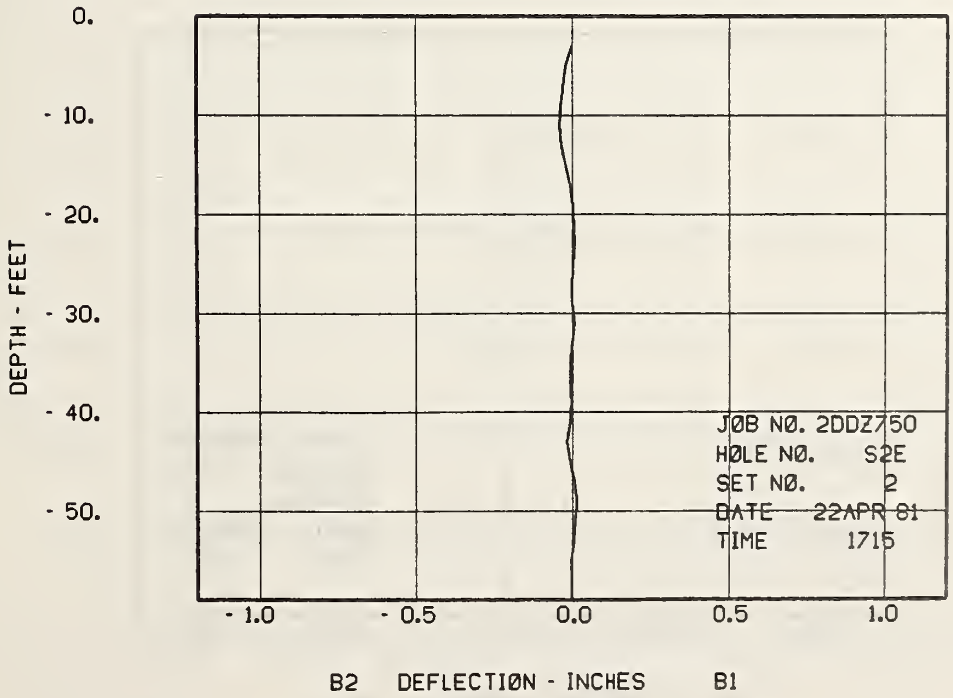

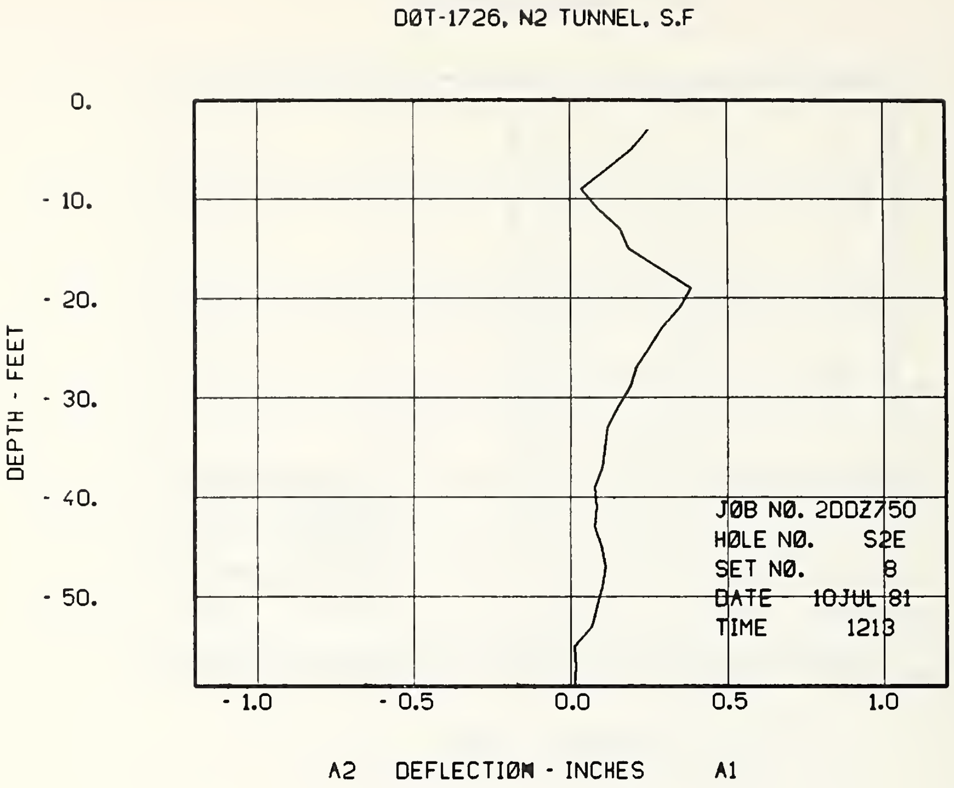
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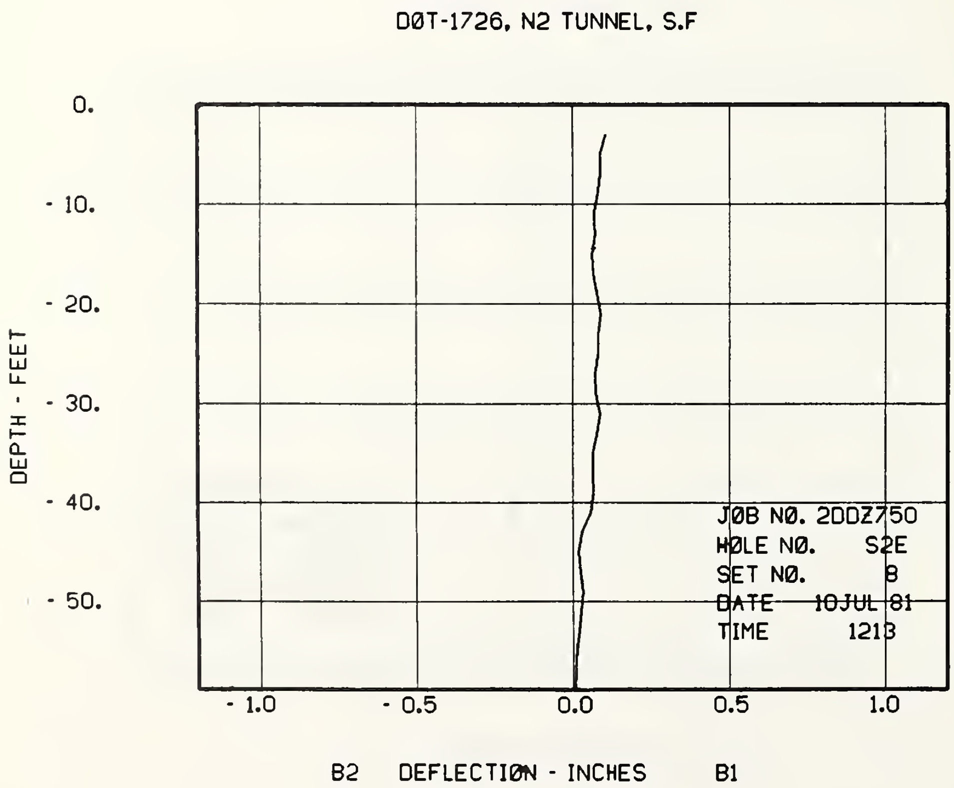
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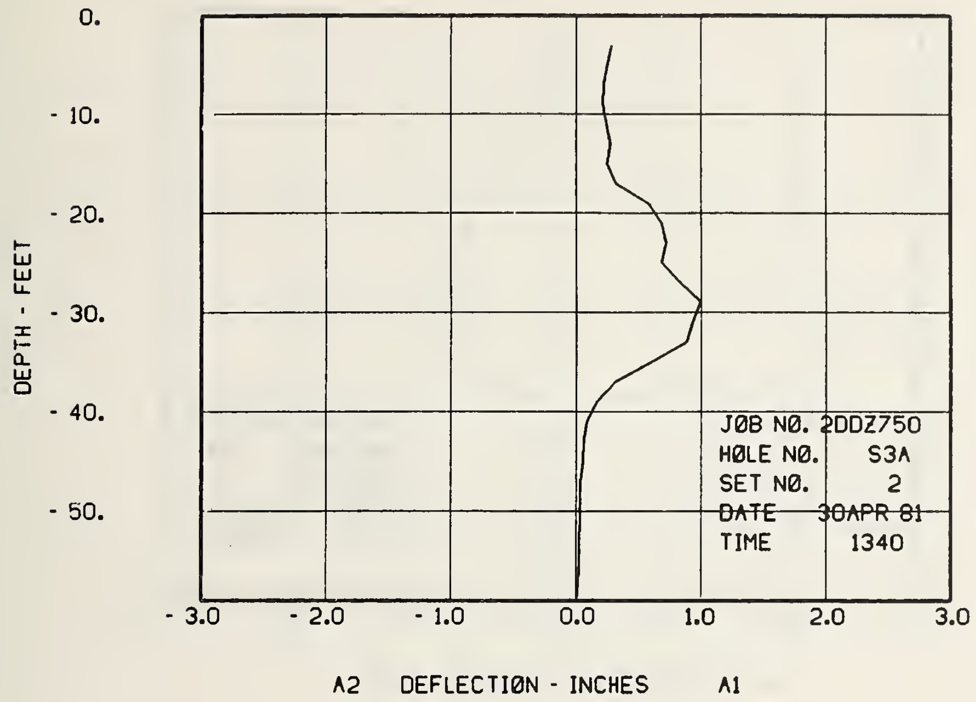

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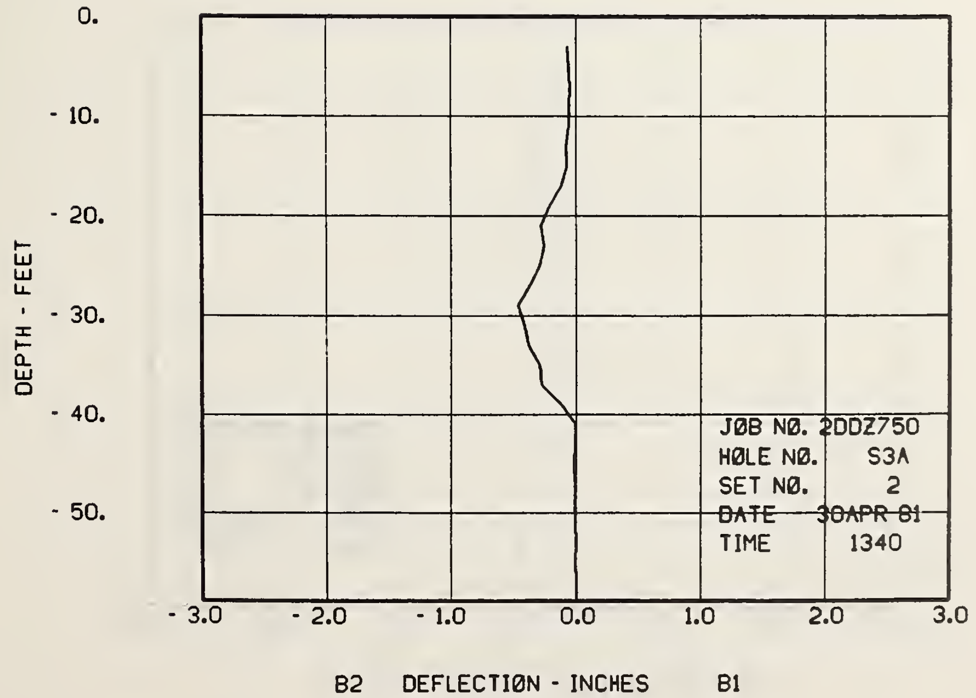

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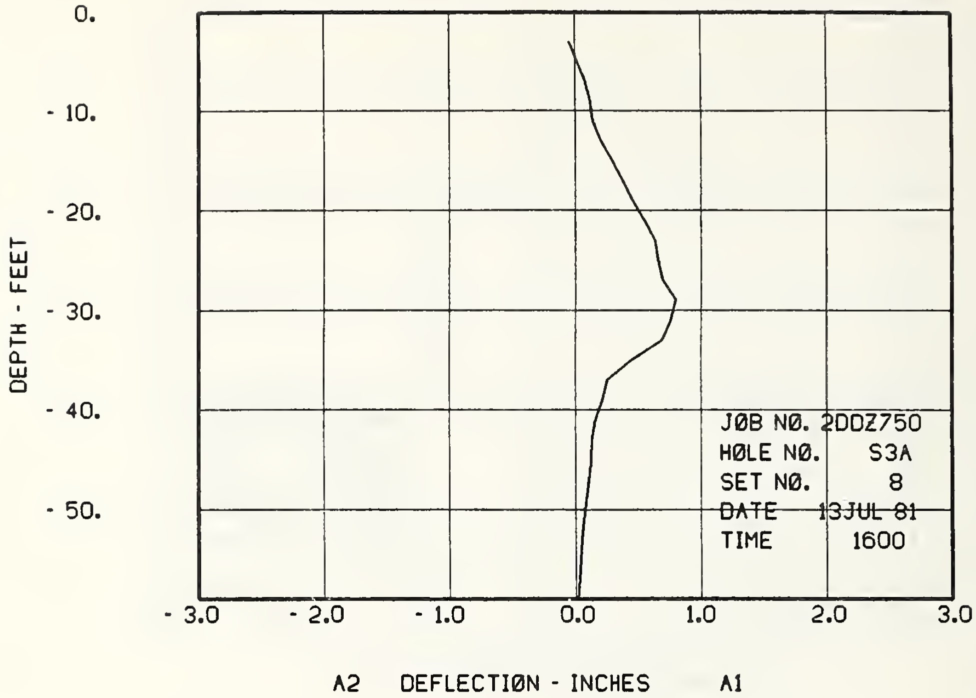

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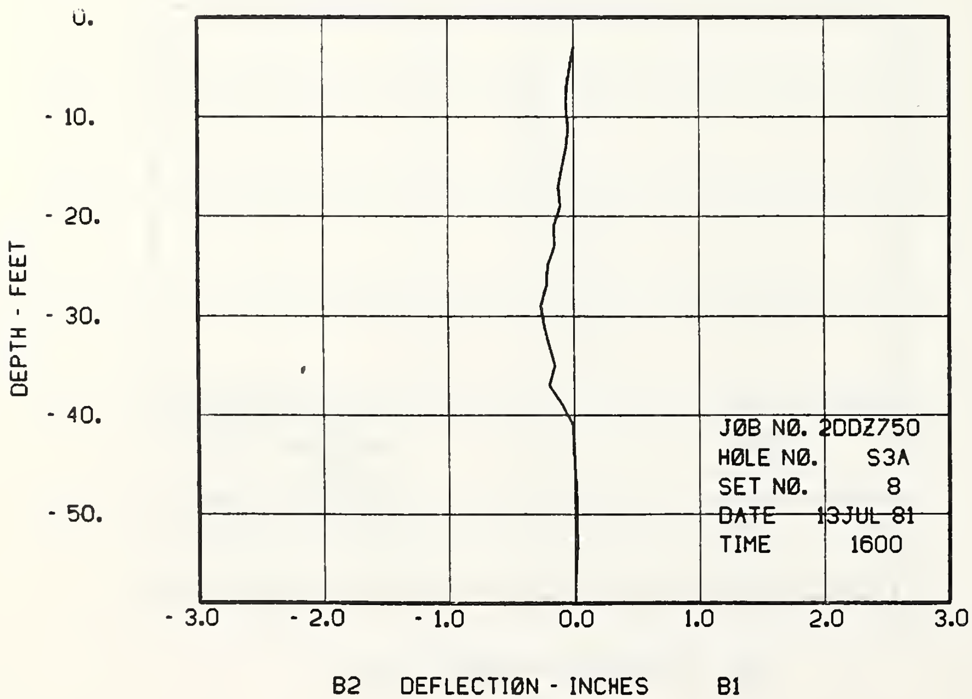

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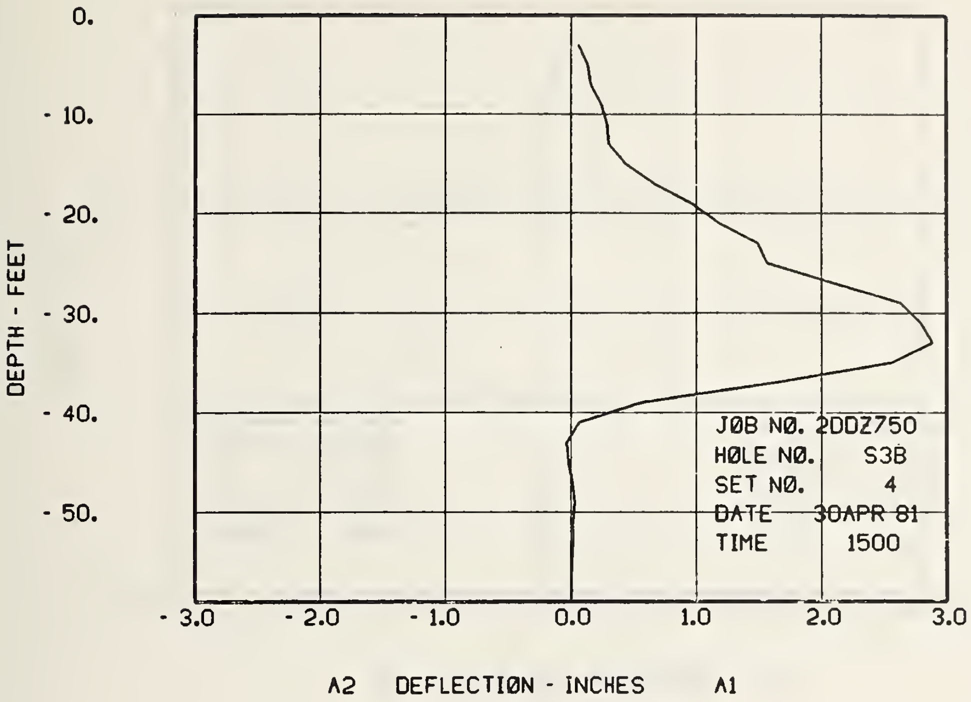

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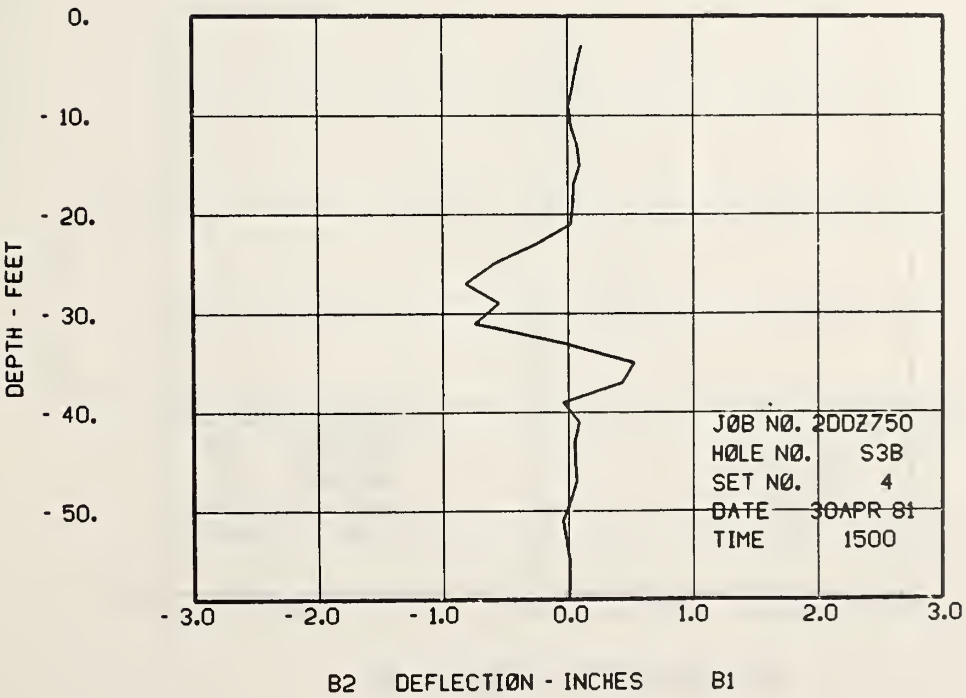

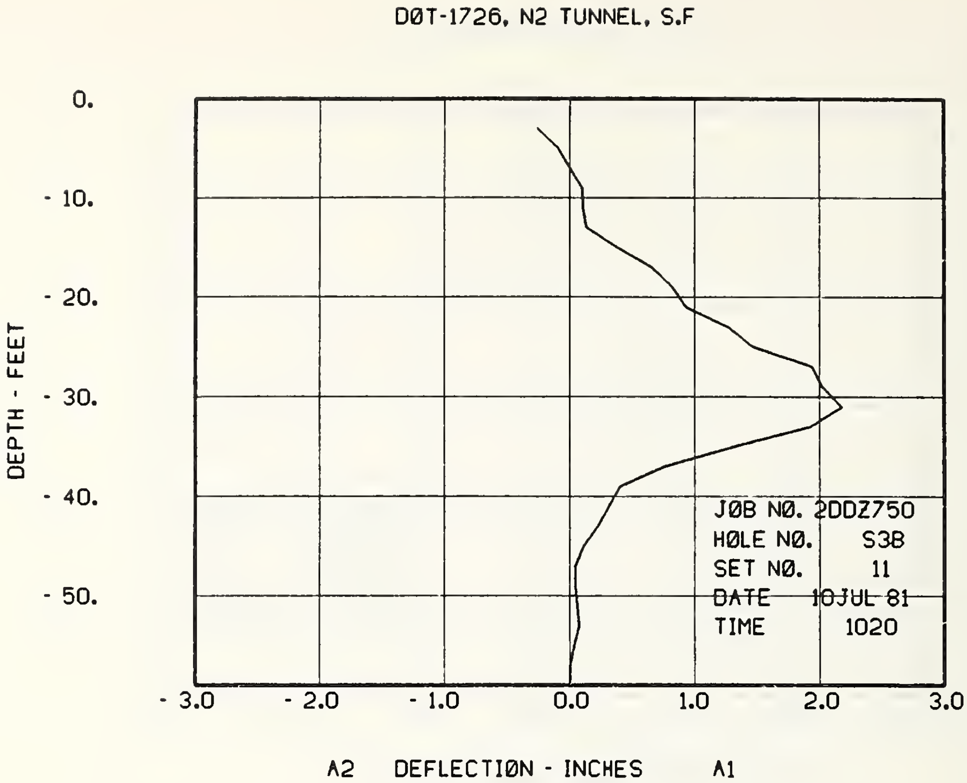
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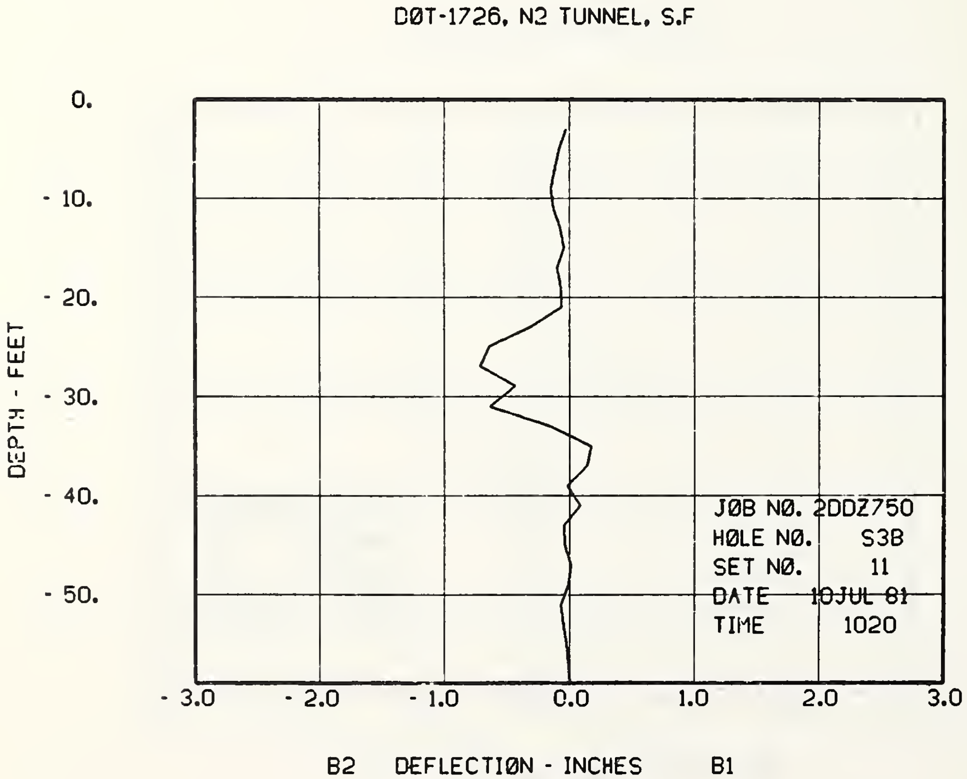
- 10.
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3B
SET NØ. 11
DATE 10JUL 81
TIME 1020
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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- 10.
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3B
SET NØ. 11
DATE 10JUL 81
TIME 1020
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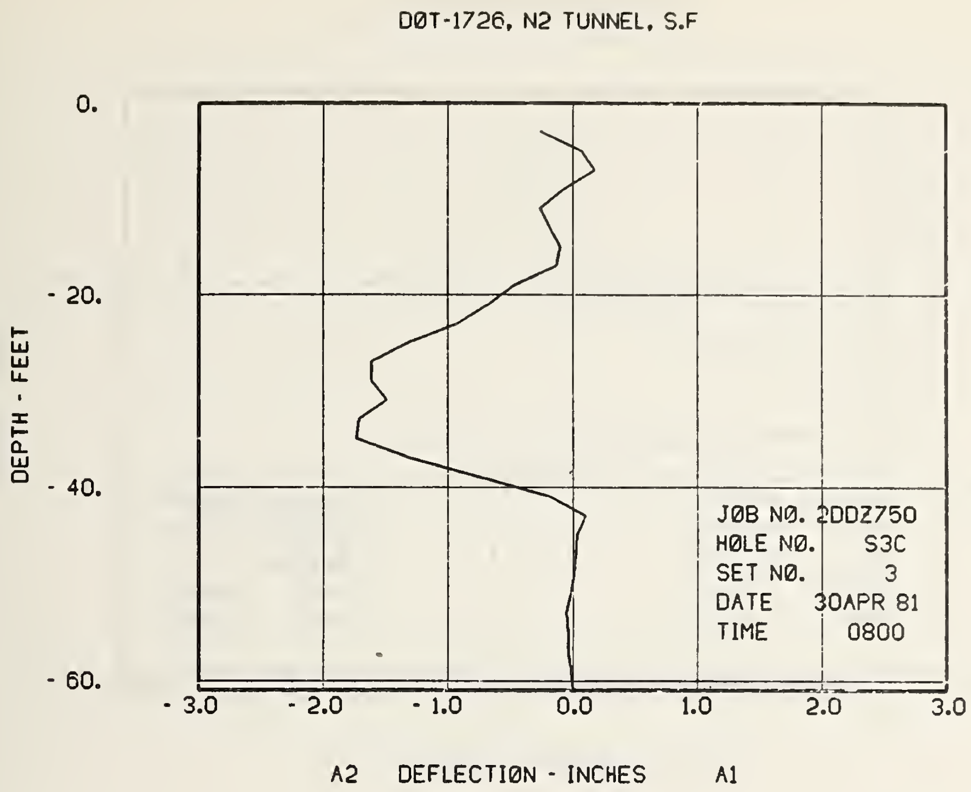
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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- 40.
- 60.
DEPTH - FEET
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HØLE NØ. S3C
SET NØ. 3
DATE 30APR 81
TIME 0800
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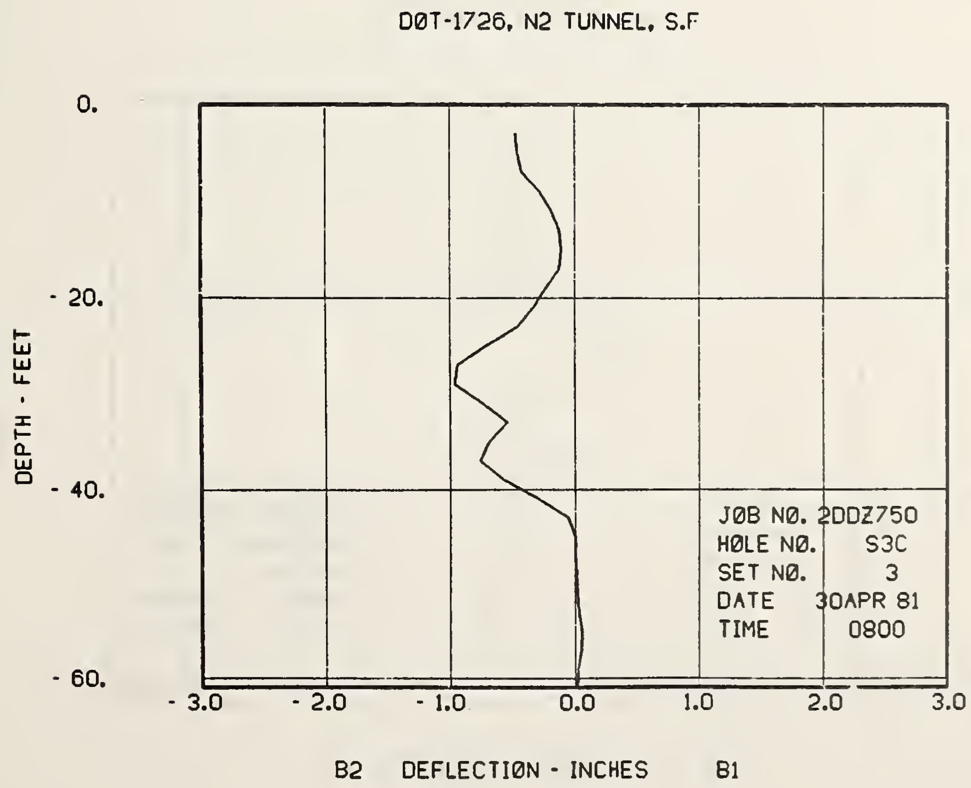
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3C
SET NØ. 3
DATE 30APR 81
TIME 0800
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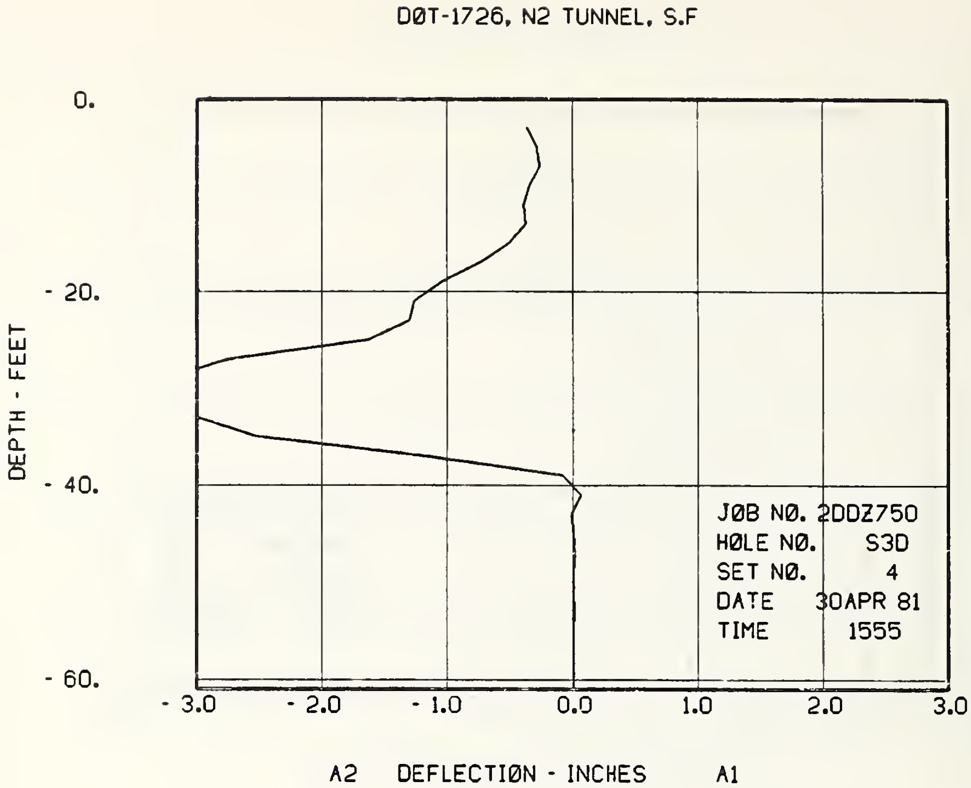
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3D
SET NØ. 4
DATE 30APR 81
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- 1.0
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2.0
3.0
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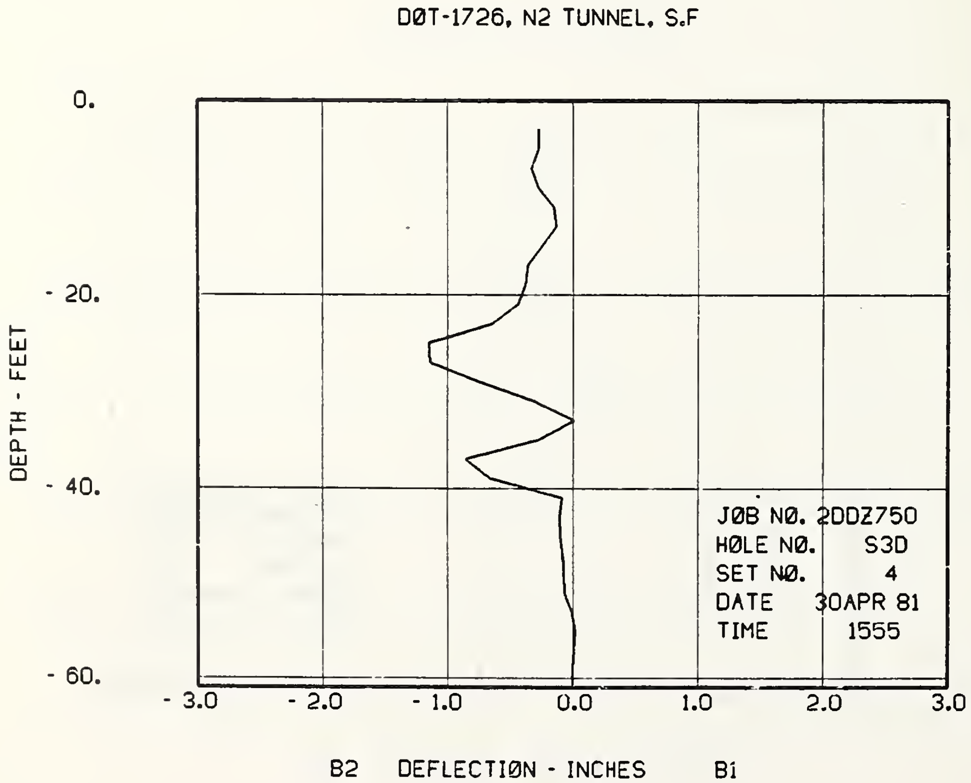
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- 20.
- 40.
- 60.
DEPTH - FEET
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HØLE NØ. S3D
SET NØ. 4
DATE 30APR 81
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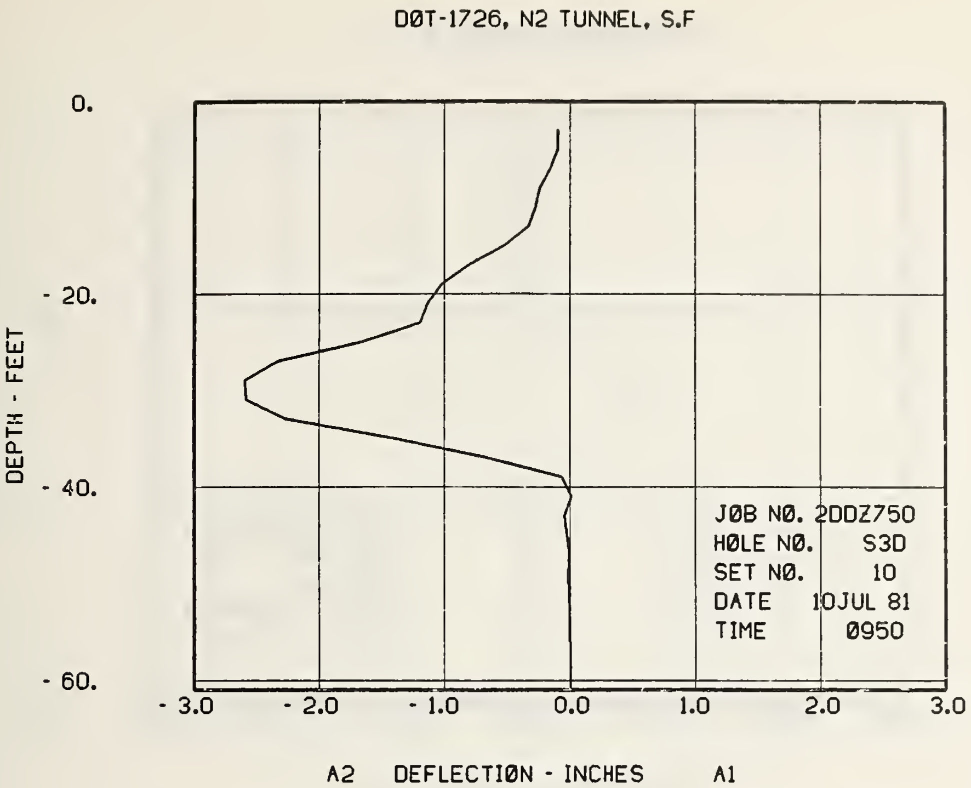
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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- 40.
- 60.
DEPTH - FEET
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HØLE NØ. S3D
SET NØ. 10
DATE 10JUL 81
TIME Ø950
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- 2.0
- 1.0
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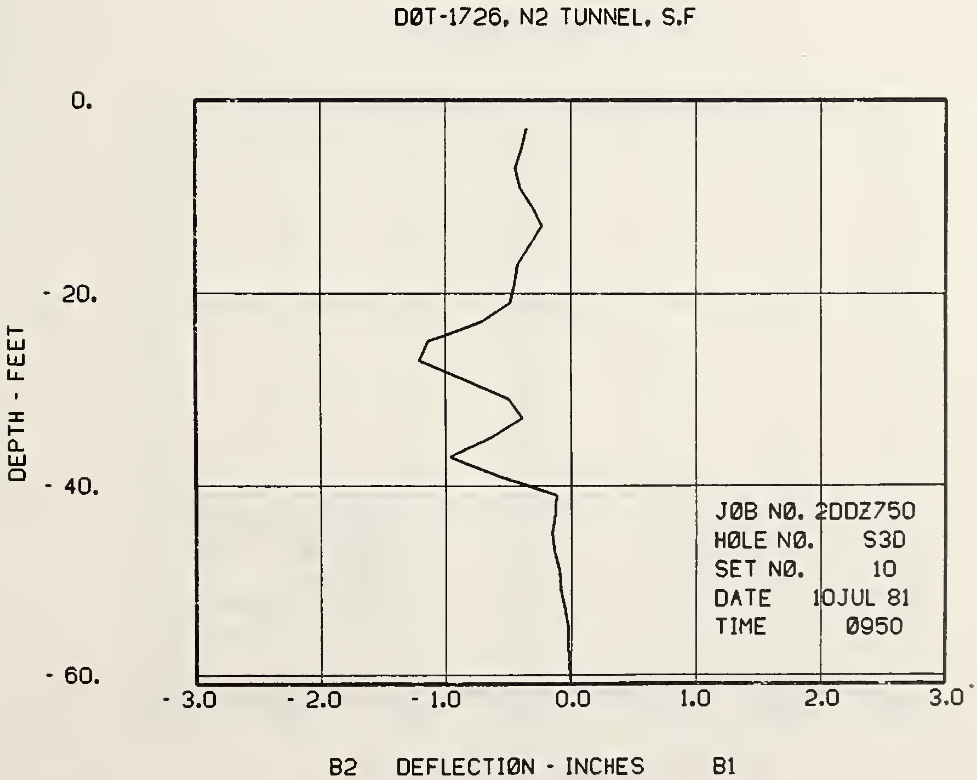
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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0.
- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3D
SET NØ. 10
DATE 10JUL 81
TIME Ø950
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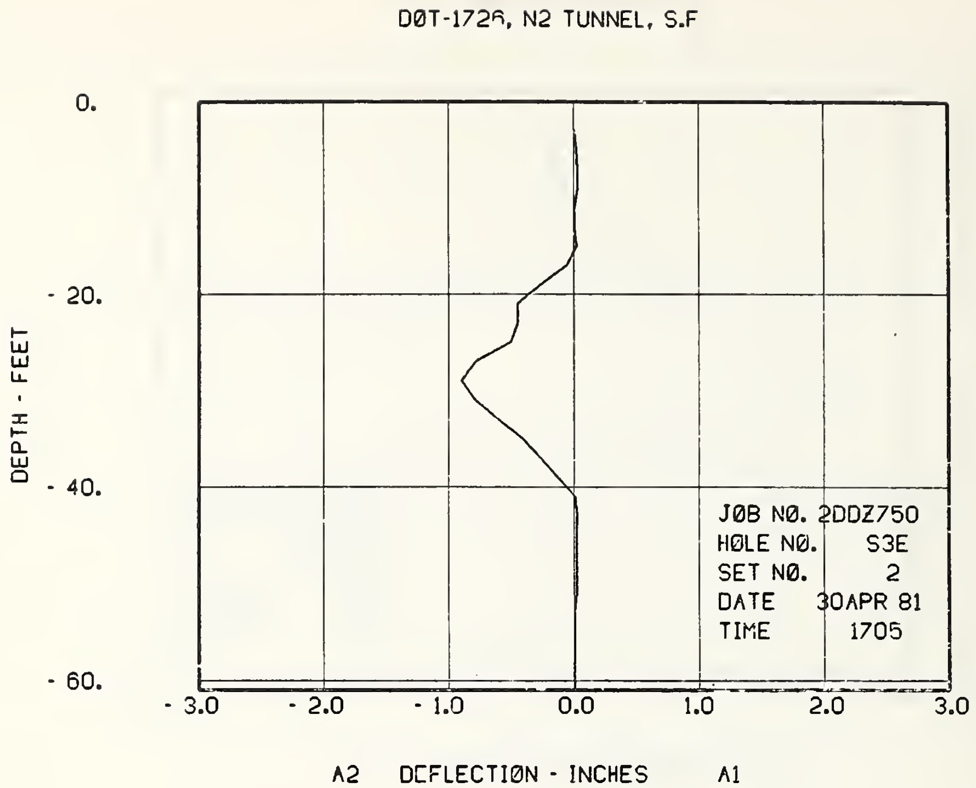
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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- 40.
- 60.
DEPTH - FEET
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HØLE NØ. S3E
SET NØ. 2
DATE 30APR 81
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- 2.0
- 1.0
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2.0
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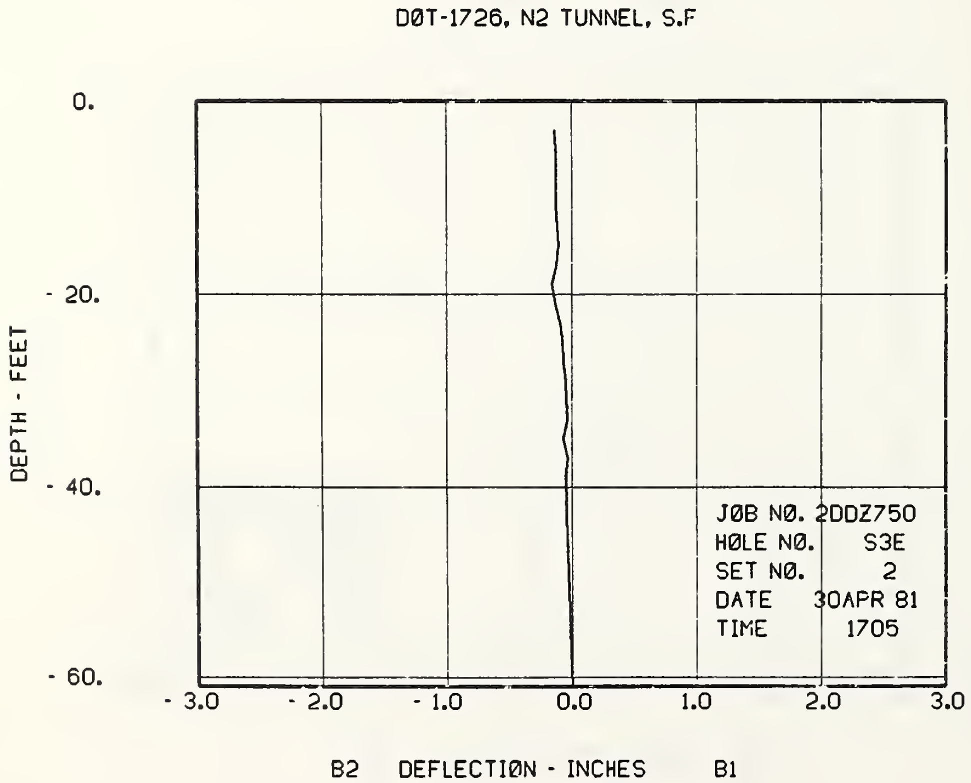
A2 DEFLECTIØN - INCHES A1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- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3E
SET NØ. 2
DATE 30APR 81
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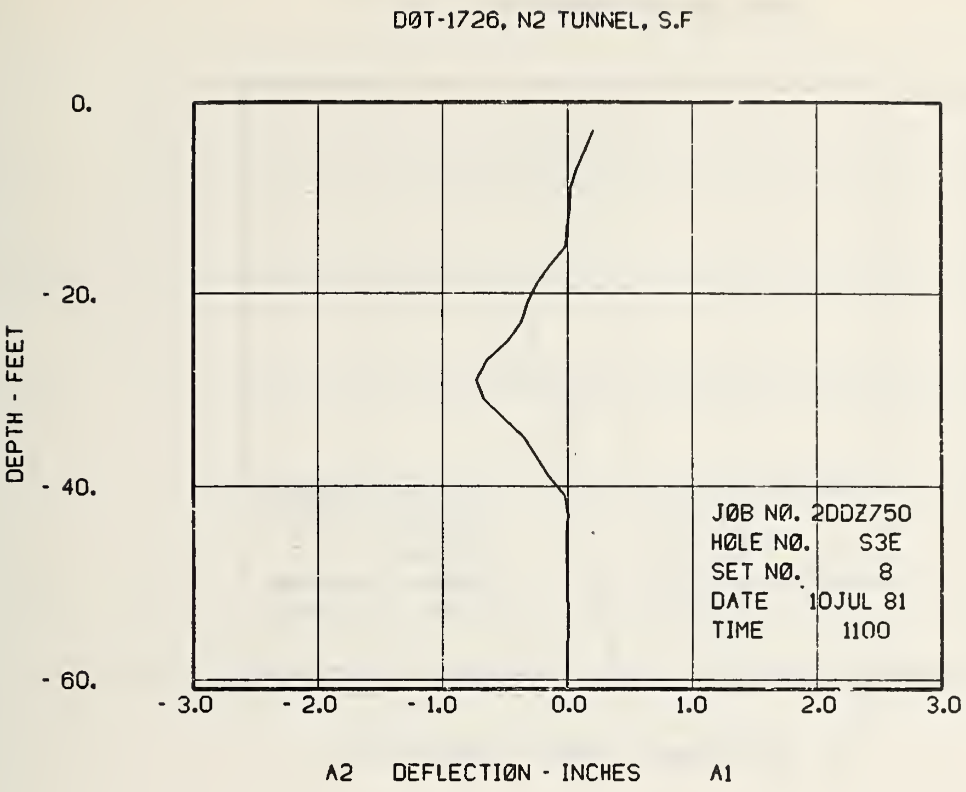
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3E
SET NØ. 8
DATE 10JUL 81
TIME 1100
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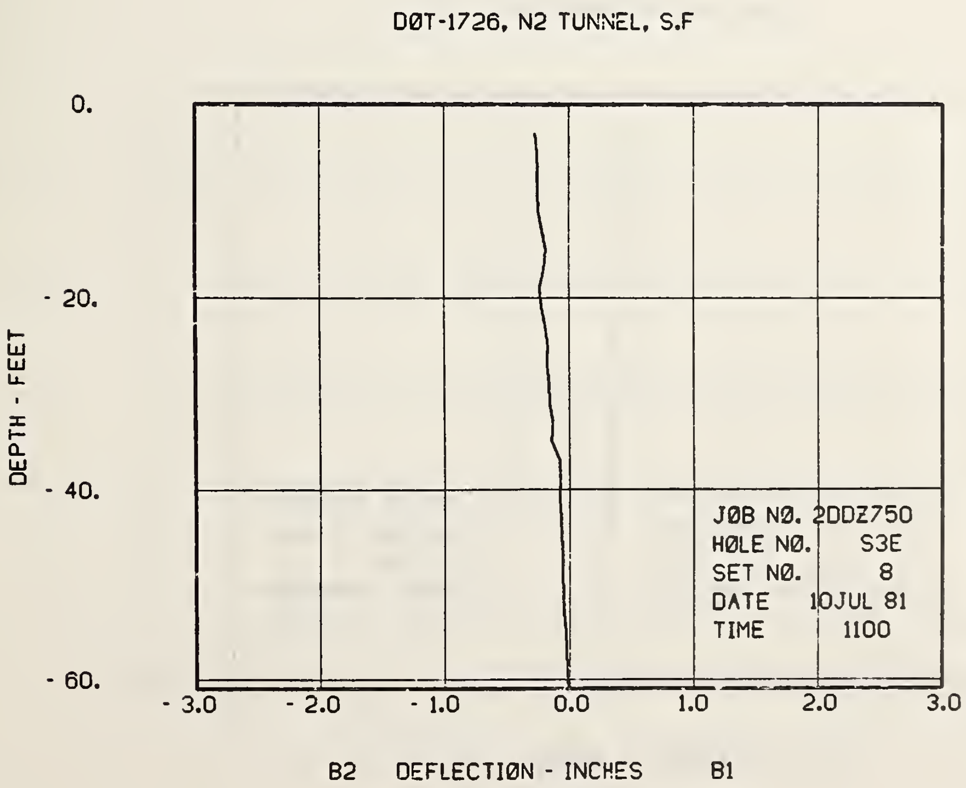
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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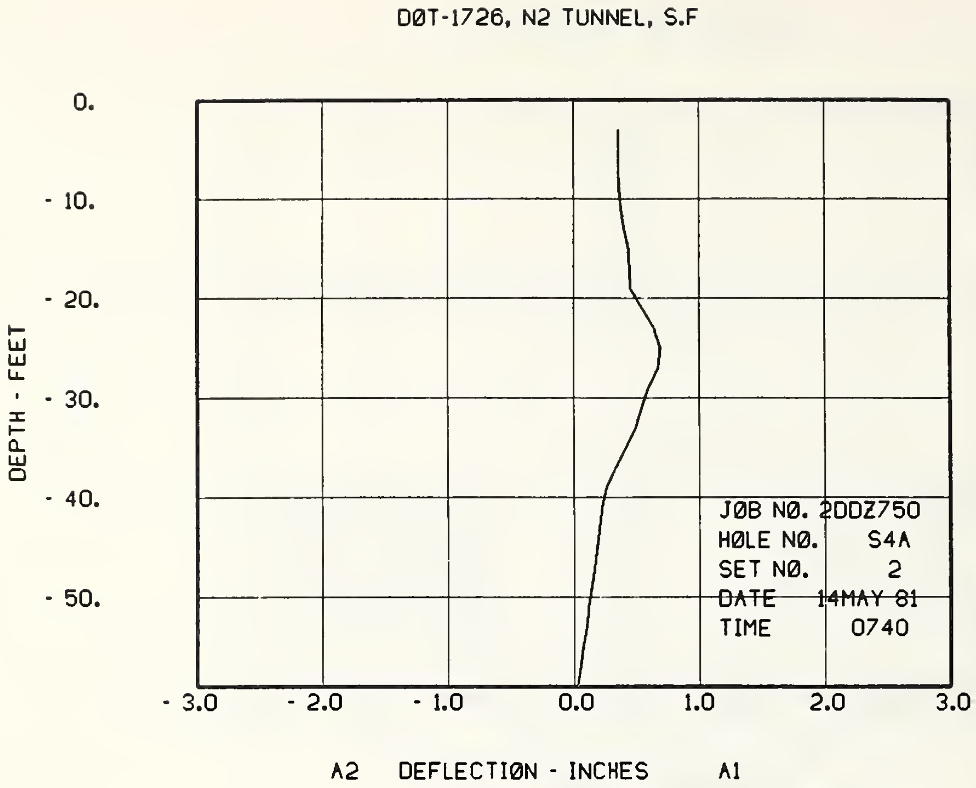
- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S3E
SET NØ. 8
DATE 10JUL 81
TIME 1100
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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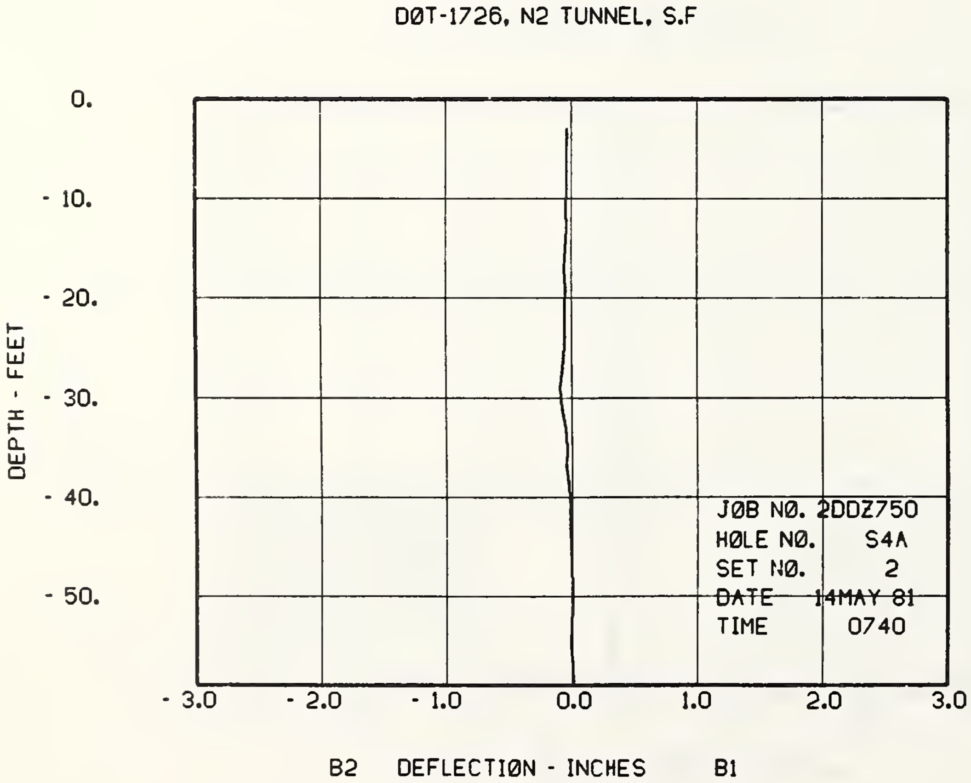
- 10.
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S4A
SET NØ. 2
DATE 14MAY 81
TIME 0740
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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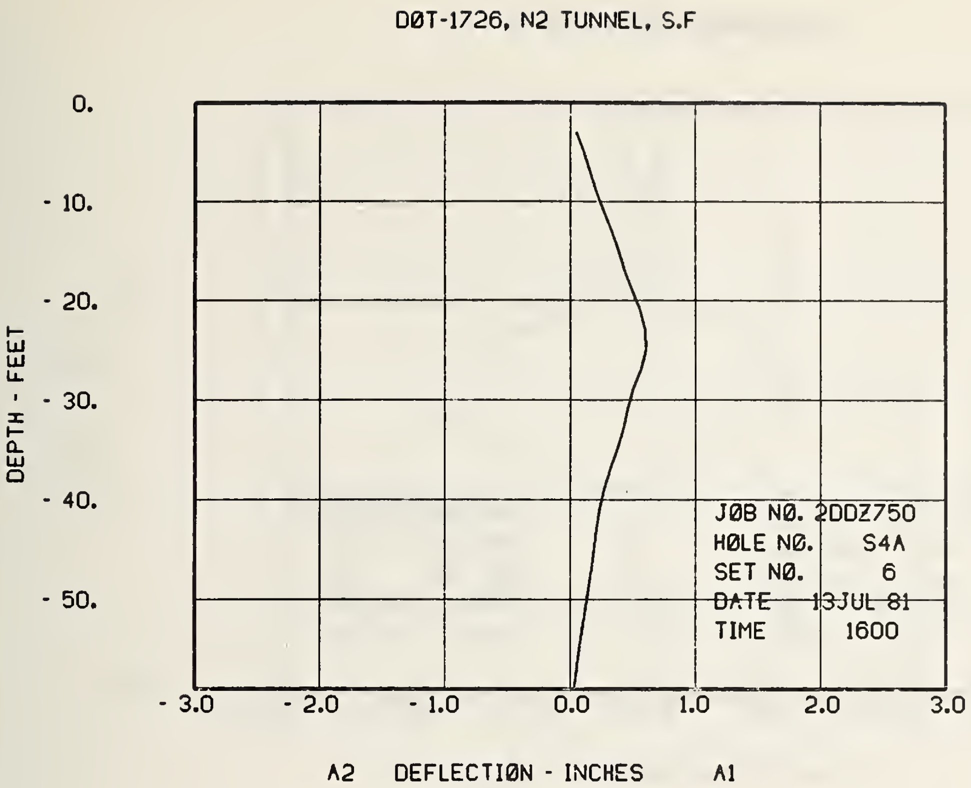
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S4A
SET NØ. 2
DATE 14MAY 81
TIME 0740
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1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DEPTH - FEET
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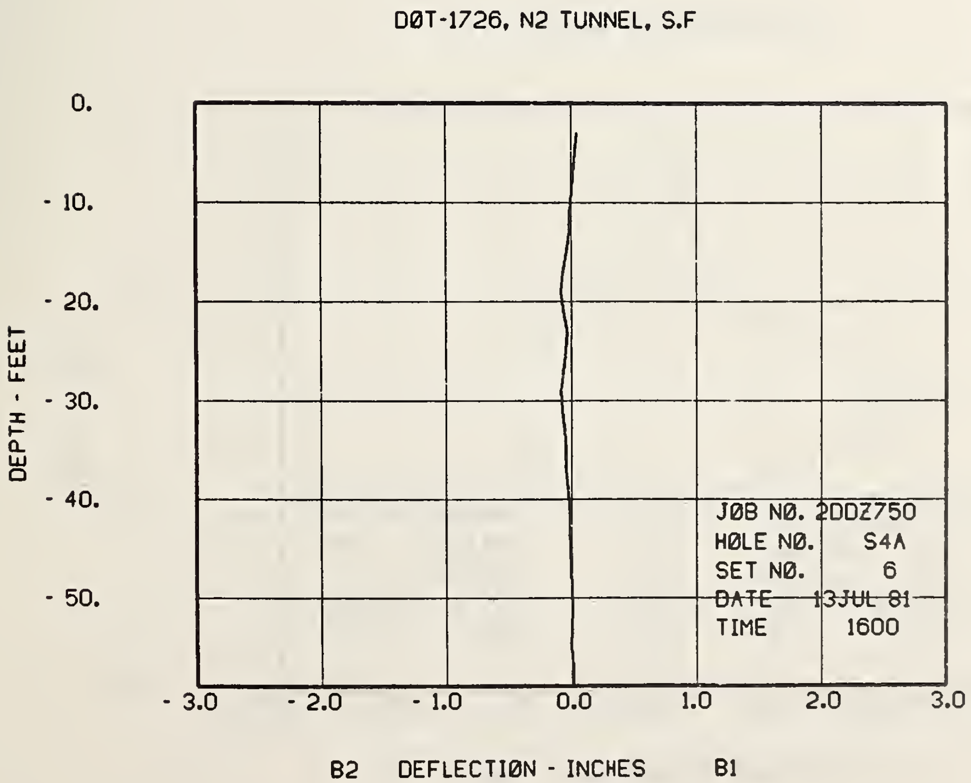
- 10.
- 20.
- 30.
- 40.
- 50.
JØB NØ. 2DDZ750
HØLE NØ. S4A
SET NØ. 6
DATE 13JUL 81
TIME 1600
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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- 20.
- 30.
- 40.
- 50.
JØB NØ. 2DDZ750
HØLE NØ. S4A
SET NØ. 6
DATE 13JUL 81
TIME 1600
- 3.0
- 2.0
- 1.0
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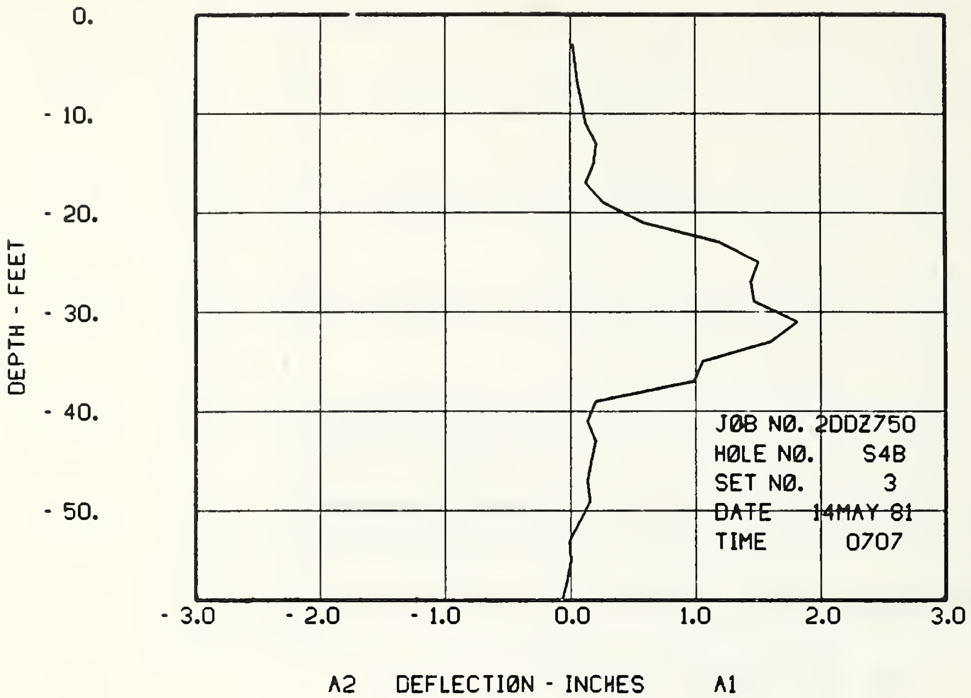
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1

DØT-1726, N2 TUNNEL, S.F



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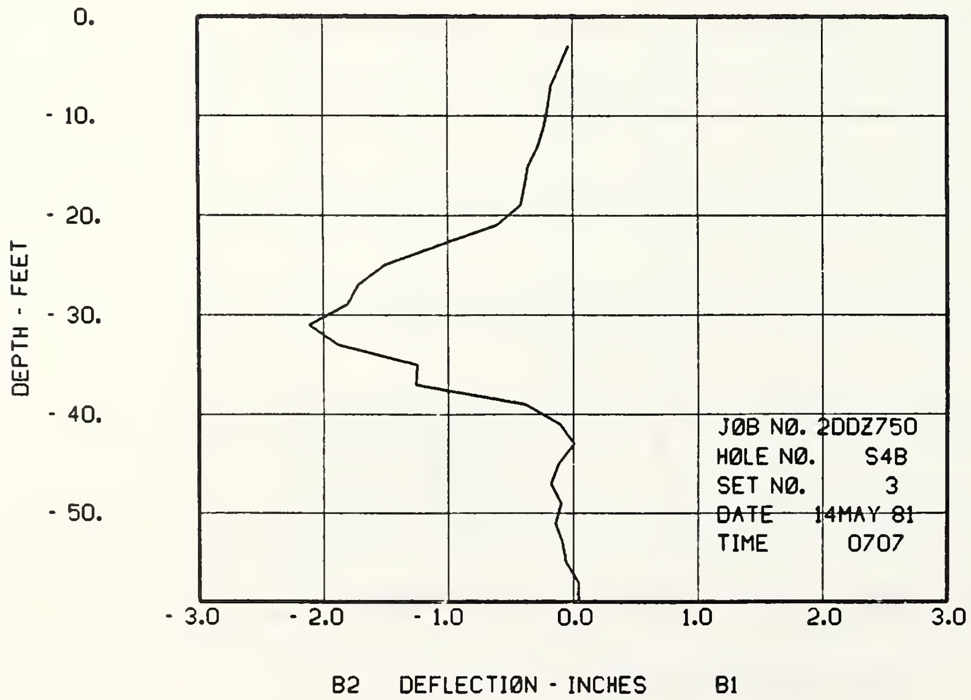

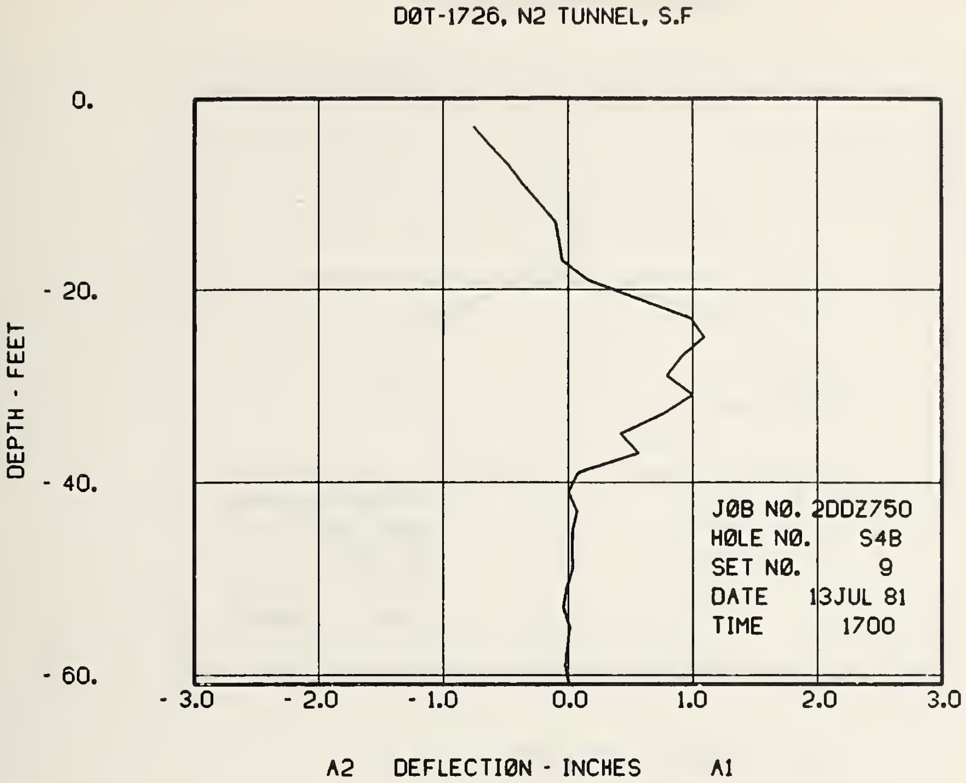
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- 40.
- 60.
DEPTH - FEET
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HØLE NØ. S4B
SET NØ. 9
DATE 13JUL 81
TIME 1700
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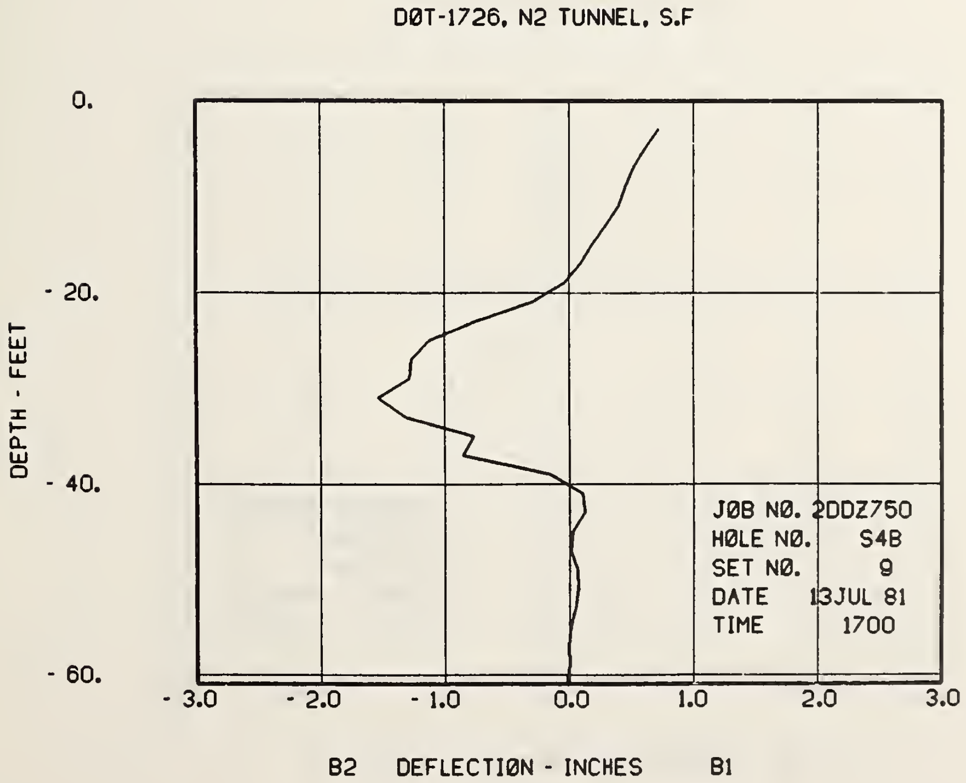
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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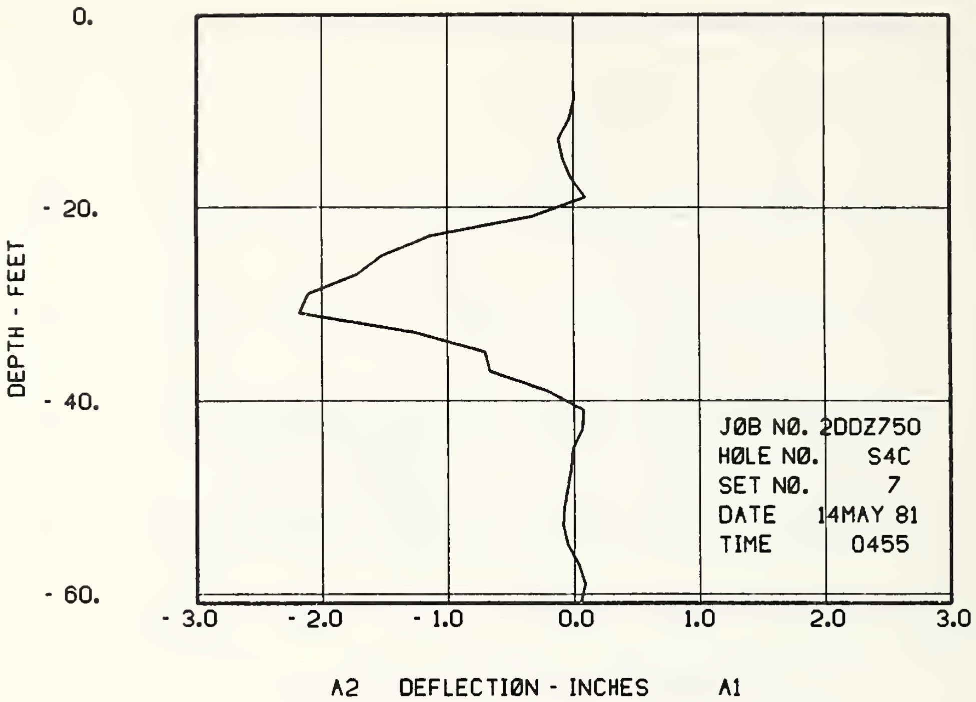
- 20.
- 40.
- 60.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S4B
SET NØ. 9
DATE 13JUL 81
TIME 1700
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1

DØT-1726, N2 TUNNEL, S.F



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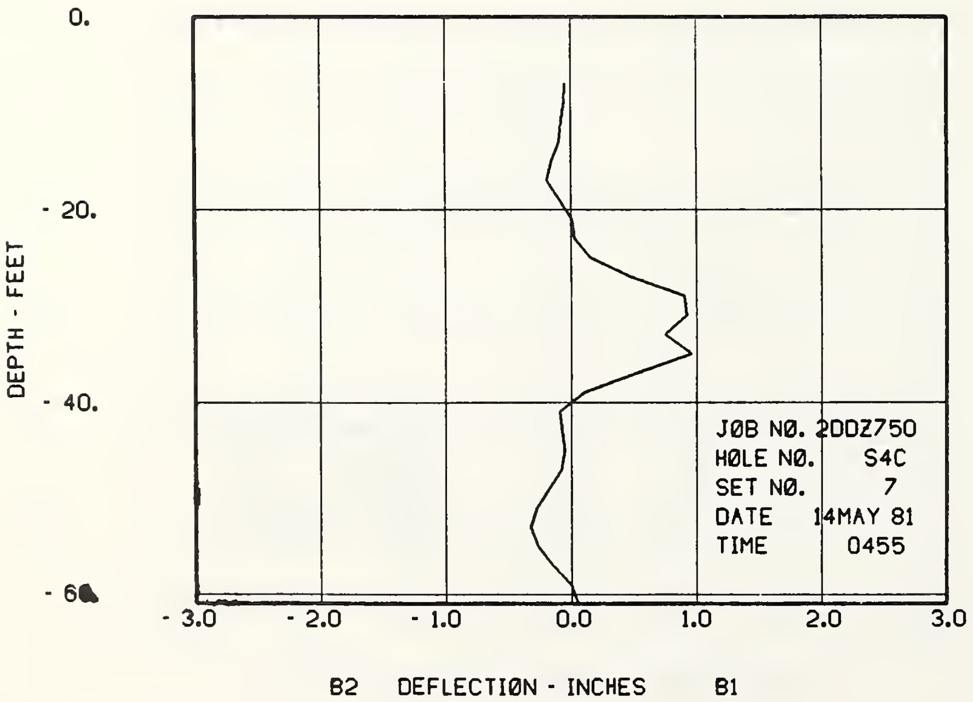

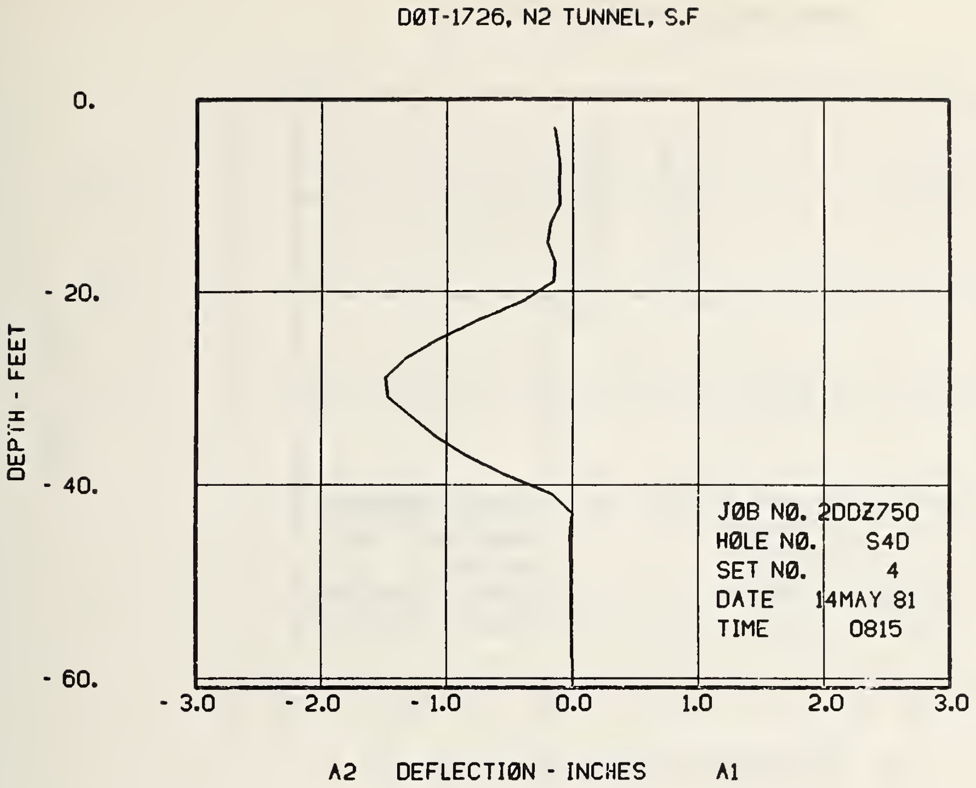
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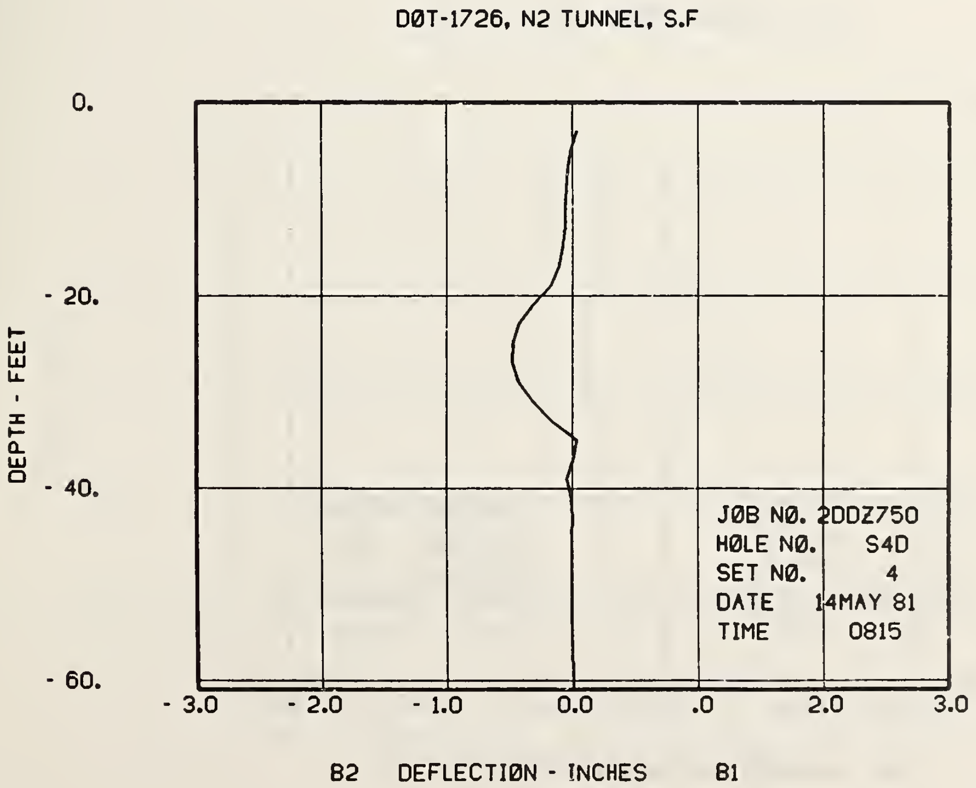
- 20.
- 40.
- 60.
DEPTH - FEET
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1
JØB NØ. 2DDZ750
HØLE NØ. S4D
SET NØ. 4
DATE 14MAY 81
TIME 0815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- 40.
- 60.
DEPTH - FEET
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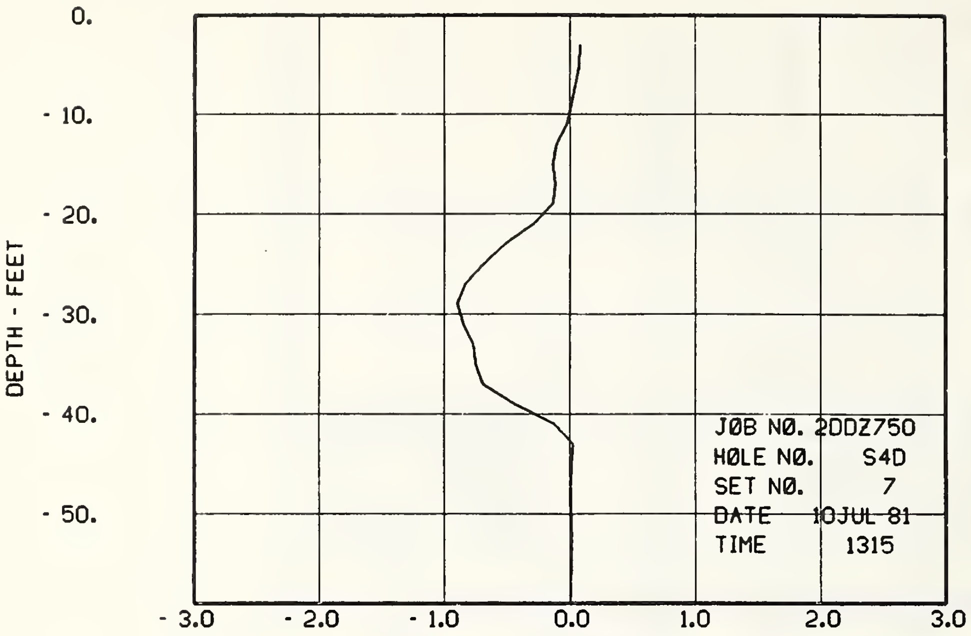
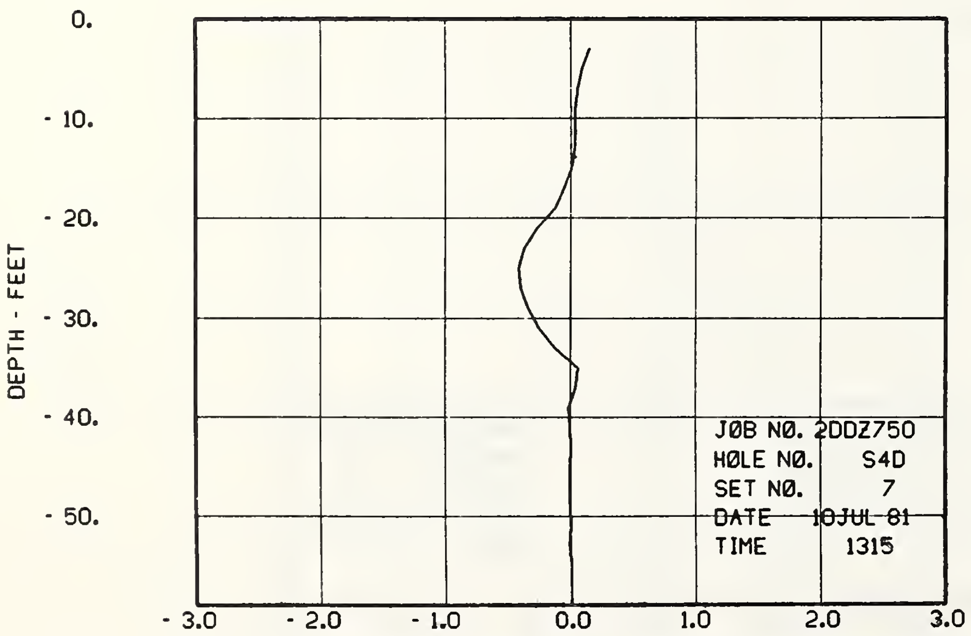
- 2.0
- 1.0
0.0
.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1
JØB NØ. 2DDZ750
HØLE NØ. S4D
SET NØ. 4
DATE 14MAY 81
TIME 0815

DØT-1726, N2 TUNNEL, S.F

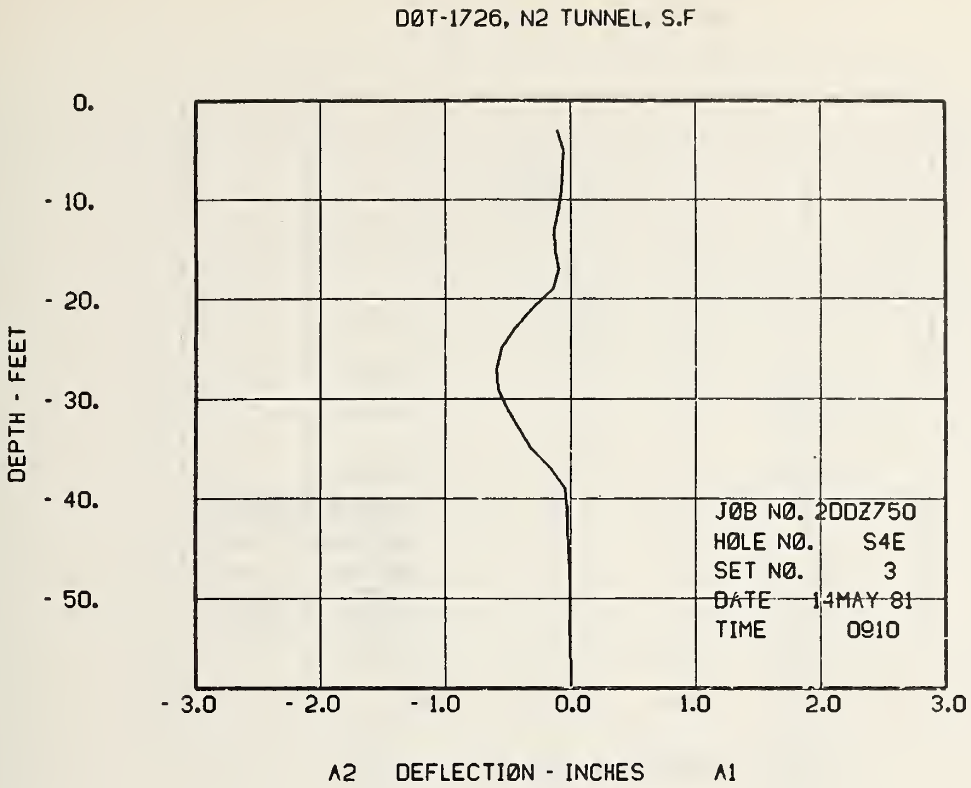


A2 DEFLECTIØN - INCHES A1

DØT-1726, N2 TUNNEL, S.F



B2 DEFLECTIØN - INCHES B1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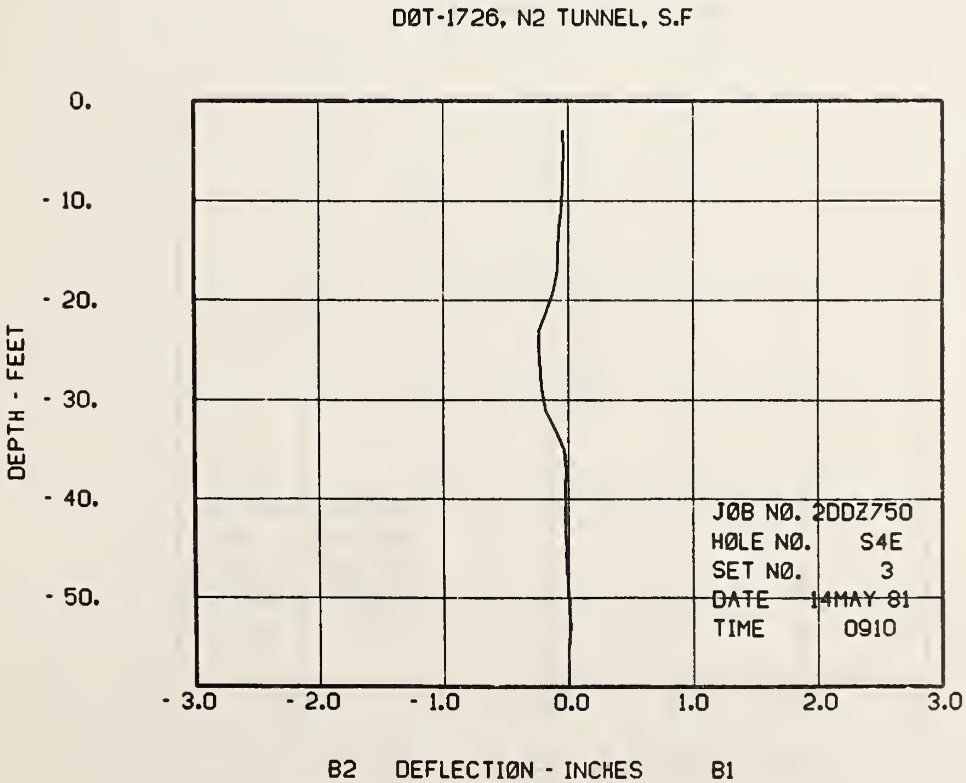
- 10.
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S4E
SET NØ. 3
DATE 14MAY 81
TIME 0910
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
A2 DEFLECTIØN - INCHES A1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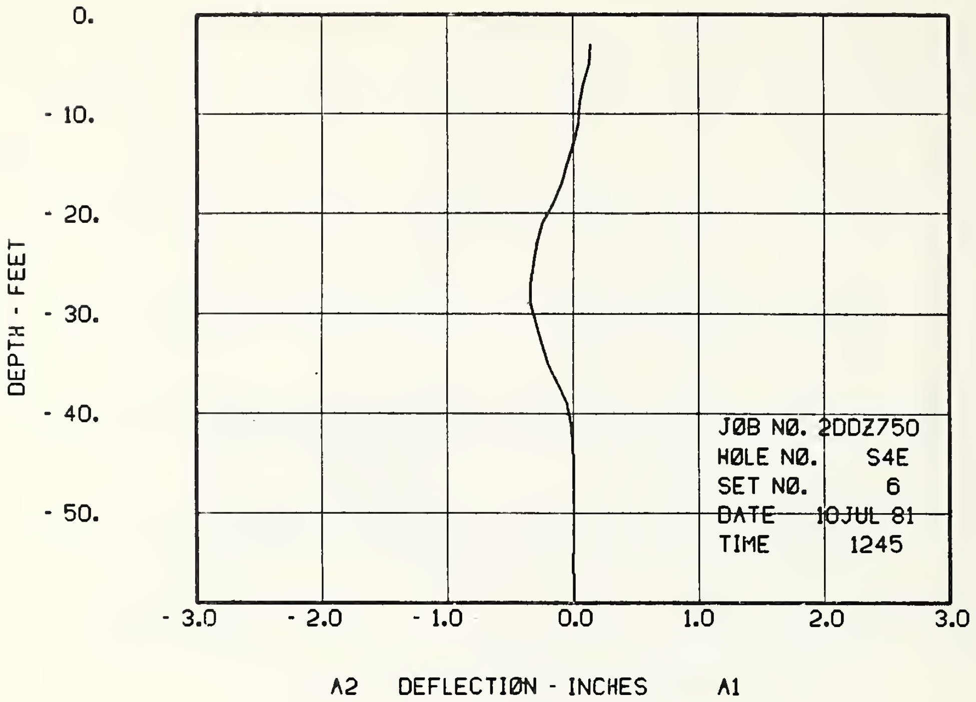
- 10.
- 20.
- 30.
- 40.
- 50.
DEPTH - FEET
JØB NØ. 2DDZ750
HØLE NØ. S4E
SET NØ. 3
DATE 14MAY 81
TIME 0910
- 3.0
- 2.0
- 1.0
0.0
1.0
2.0
3.0
B2 DEFLECTIØN - INCHES B1

DØT-1726, N2 TUNNEL, S.F



DØT-1726, N2 TUNNEL, S.F

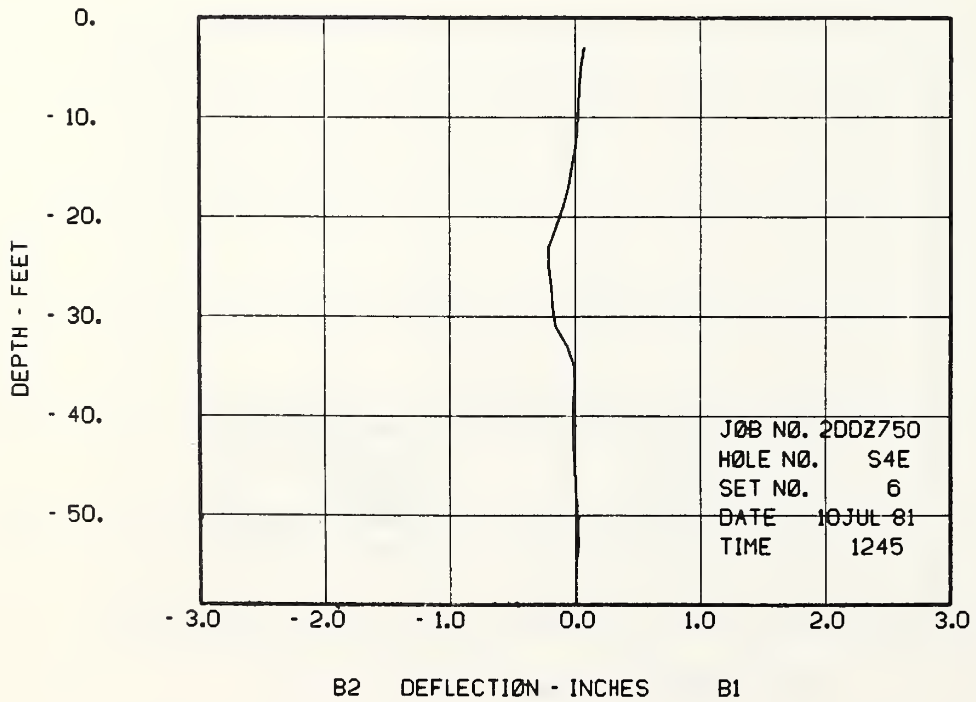


TABLE A.2

Correlation of Time and Key Shield Positions

Date	Time	Approximate Station of Shield Face
22 Apr 81	1350	9+22
" " "	1625	9+28
" " "	1715	9+34
" " "	1750	9+34
30 Apr 81	0800	13+49
" " "	1340	13+69
" " "	1500	13+72
" " "	1555	13+75
" " "	1705	13+81
14 May 81	0455	20+47
" " "	0707	20+54
" " "	0740	20+59
" " "	0815	20+61
" " "	0910	20+64

Appendix B

DIAMETRIC DEFORMATION MEASUREMENTS

Changes in diameter of the tunnel liner were measured using a SINCO model S18115 tape extensometer. Measurements were made from eight eyebolt reference points located equidistant around the liner. The accuracy of the measurements was ± 0.005 in (0.01 cm).

Extensometer readings were taken as soon after liner erection as possible. Due to the protrusion of the screw auger from the rear of the shield and the trailing power train, the initial readings could not be obtained until the drive train had passed the instrumented liner segment. This was approximately 180 ft, (55 m) of tunnel advancement and 7 days after the segment had been erected. Subsequent data were obtained on the same schedule as the ground movement instrumentation.

Approximately 3 months of tunnel liner measurements were obtained. After this period, concreting operations to install the permanent liner precluded additional readings. Whenever possible, measurements were obtained from each bolt to all other bolts except those adjacent to that bolt. These data provided sufficient redundancy to verify the accuracy of the diametric measurements.

Results of the measurements are summarized on Figures B.1 through B.4. Figure B.1 shows the relative shortening of the vertical diameter

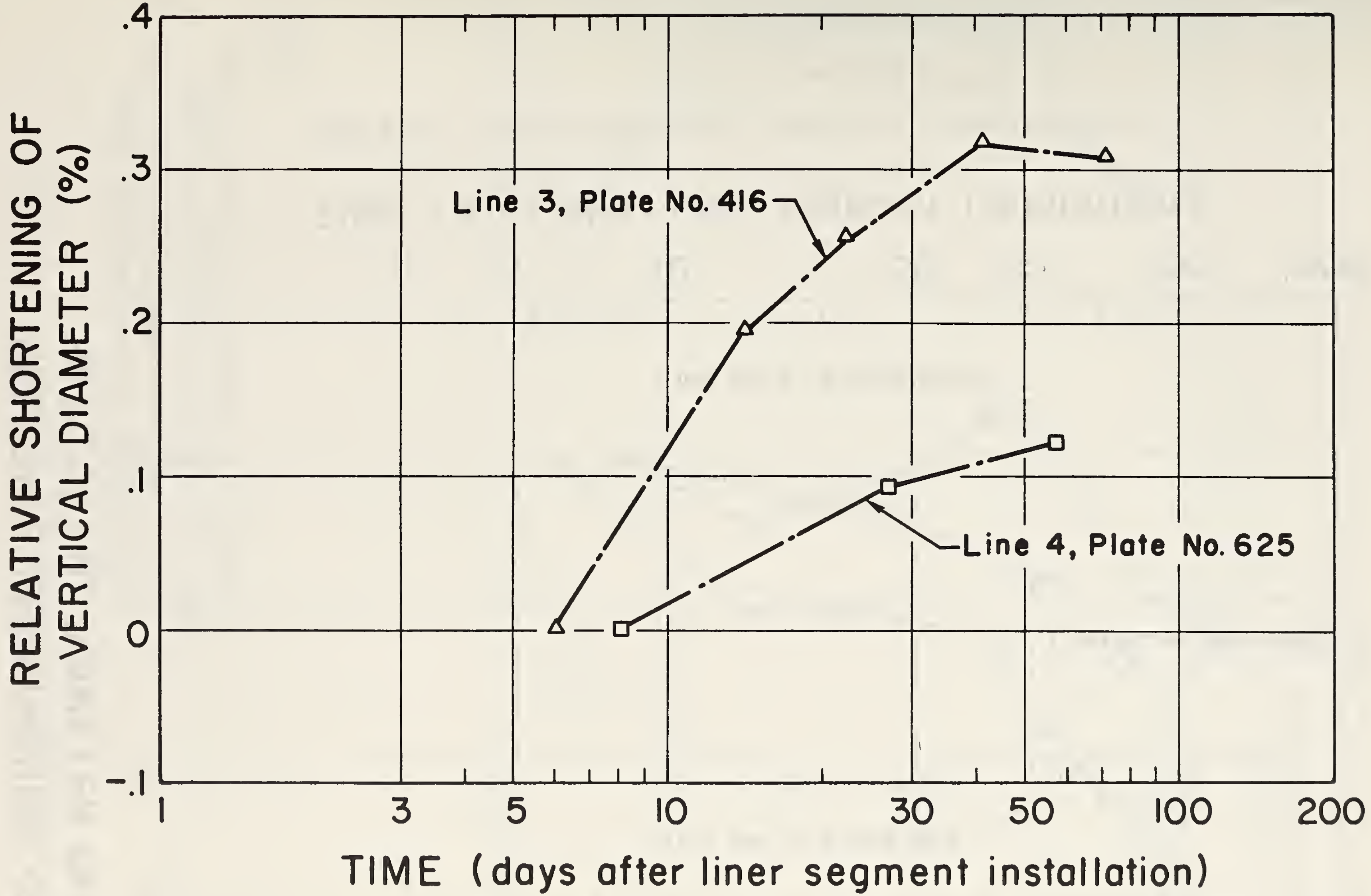


Figure B.1: Changes of Vertical Diameter of Liners with Time

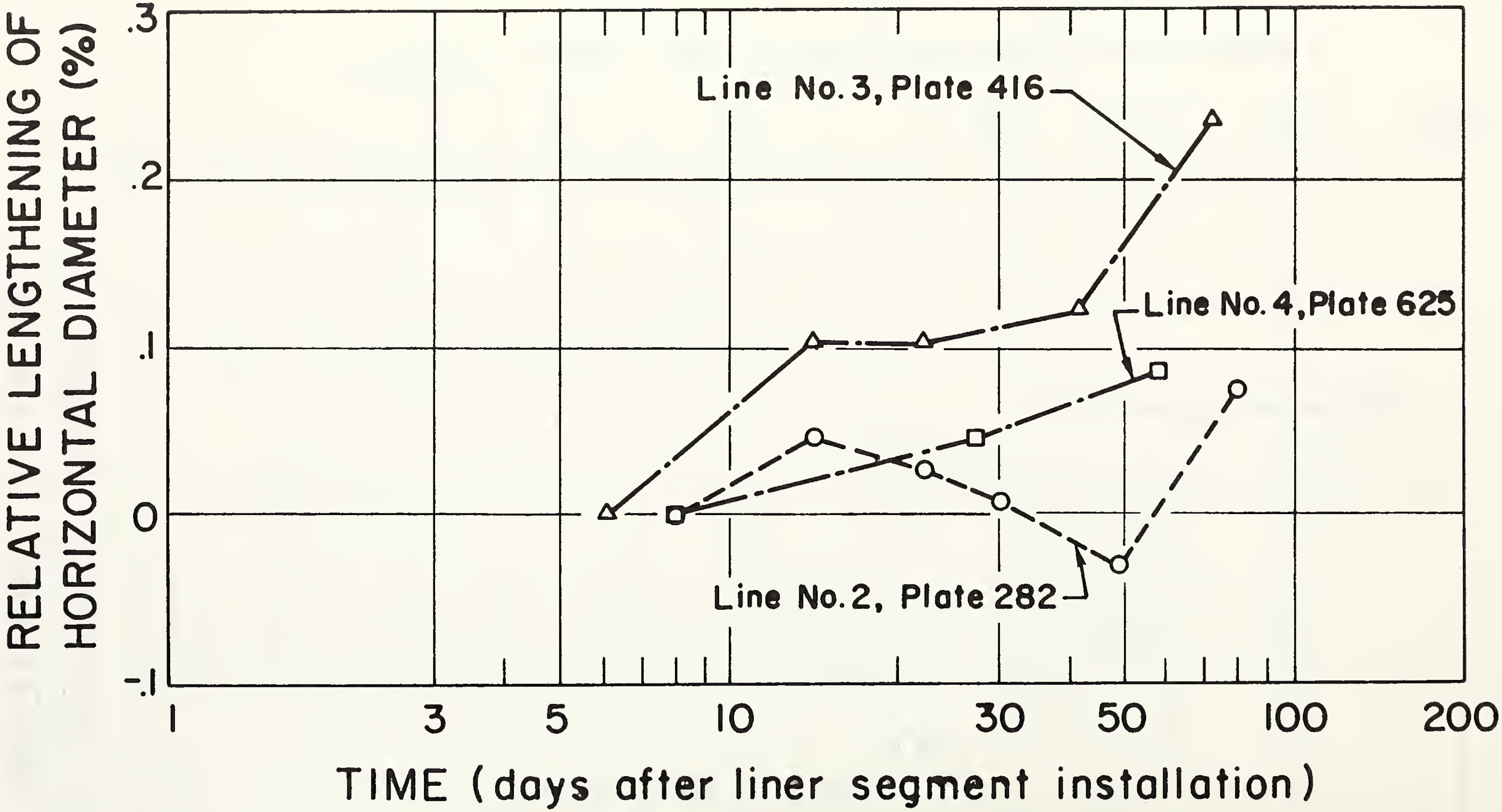
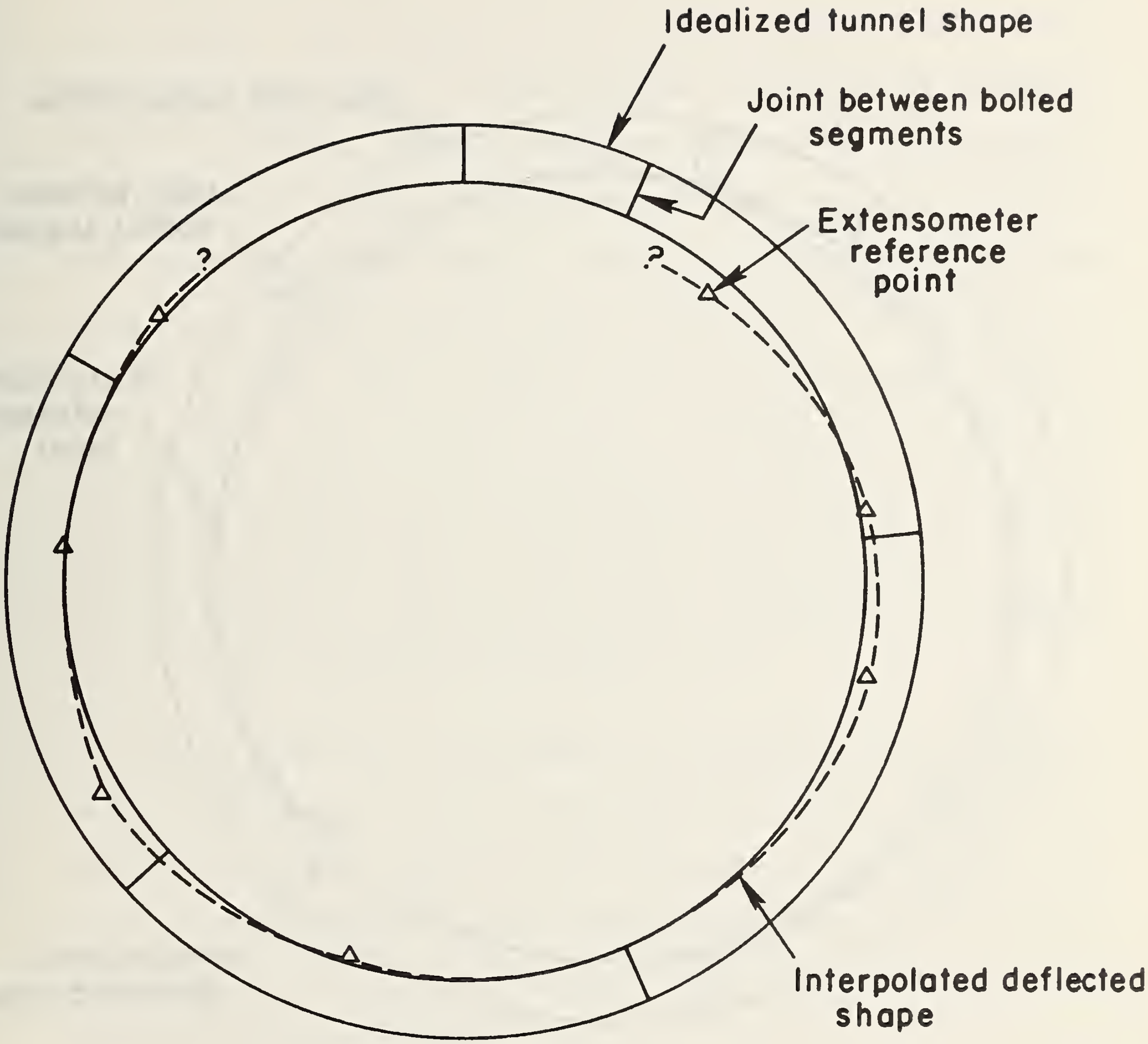


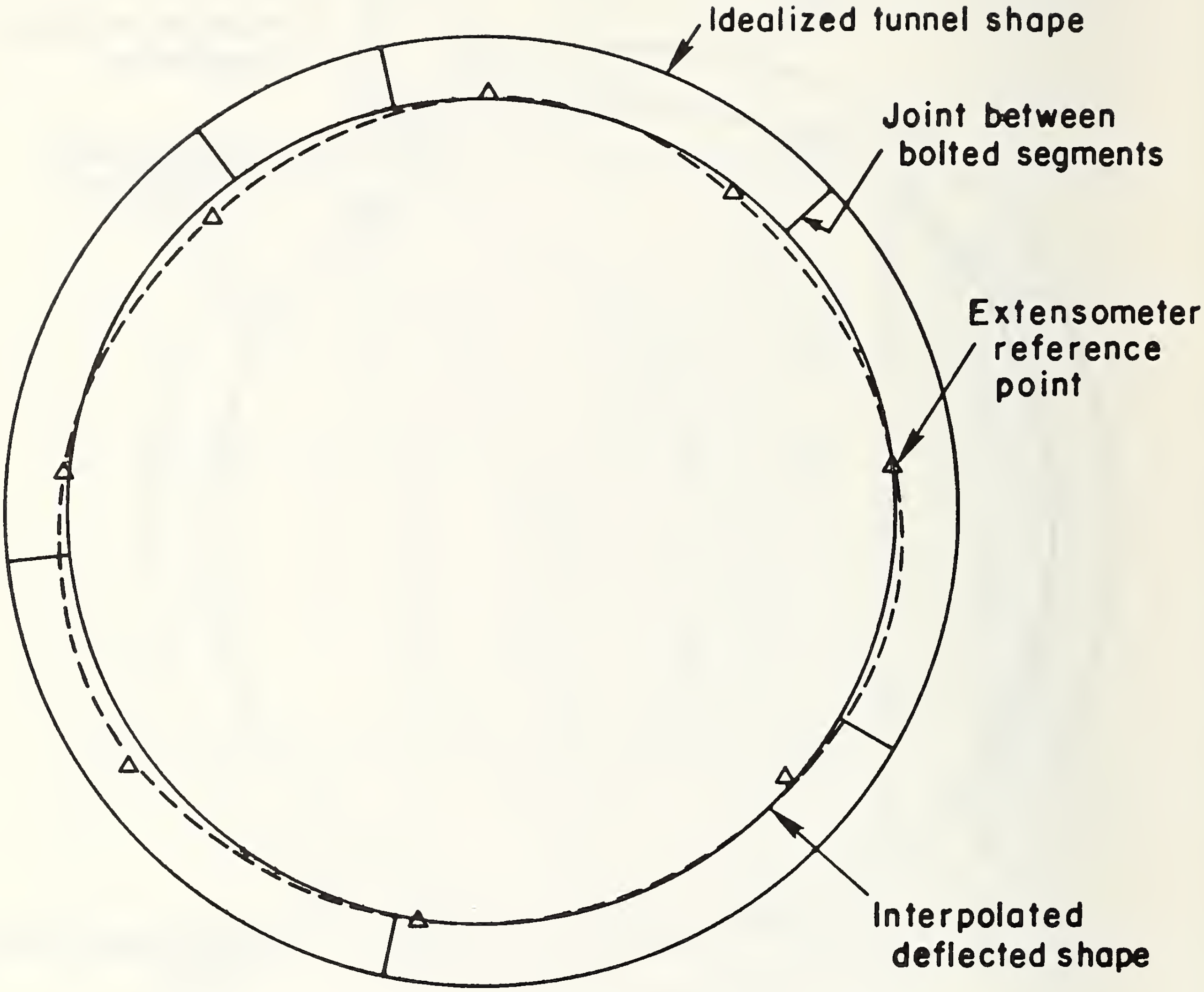
Figure B.2: Changes of Horizontal Diameter of Liners with Time



Notes:

1. Circular tunnel size shown to illustrate relative changes of deflected shape
2. Data for extensometer reference point at crown of tunnel were inconsistent

Figure B.3: Liner Deformation Pattern at Line 2



Note:

1. Circular tunnel size shown to illustrate relative changes of deflected shape

Figure B.4: Liner Deformation Pattern at Line 4 with time for Lines 3 and 4. The vertical diameter could not be measured at Line 2 due to faulty placement of one of the eyebolts. Figure B.2 shows the relative lengthening of the horizontal diameter with time for all lines. The zero points on these figures correspond to the initial extensometer readings. Results of redundant readings among the reference points provided a means to sketch the liner deformation patterns at Lines 2 and 4 (Figures B.3 and B.4, respectively). As shown on the figures, the horizontal diameter extended while the vertical diameter contracted as time progressed to form an oval-shaped deformation pattern. This is the typical behavior observed in past tunneling projects. Insufficient data was obtained due to equipment interference for this sketch to be made for Line 3.

Results of diametric measurements in Figures B.1 and B.2 indicate the relative distortion of the liner varied from 0.08 to 0.31% after 3 months. It should be noted that the deformations of the liner under its own weight and those during drive train passage could not be measured and thus are not included in the figures. Thus the reported distortions represent a lower bound value.

Deere, et al., (1969) tabulated data for diametric measurements for many projects and found the relative distortions to be all less than one percent. In soft clays, the distortions were of the order 0.3 to 0.7 percent. Case histories reported by Deere were for liners of varying stiffnesses; implying that the amount of distortion is generally not altered by large variations in the rigidity of the liner. Data from the N-2 diametric measurements fall within the lower limits of the conventional projects.

Appendix C

LINER STRAIN GAGE RESULTS

In a limited program to attempt to measure trends of earth and water loads developing on the steel segmental liner, eight strain gages were affixed to the liner at each instrument line, and monitored along with the ground movement devices. This effort was generally unsuccessful because: (1) the strain distribution in the liner plate system is too complex to be defined by only eight gages; and, (2) a number of gages were lost due to improper installation, damage from accidental impact during tunneling operations, or deterioration due to water exposure. In this appendix, those readings which could be interpreted are documented.

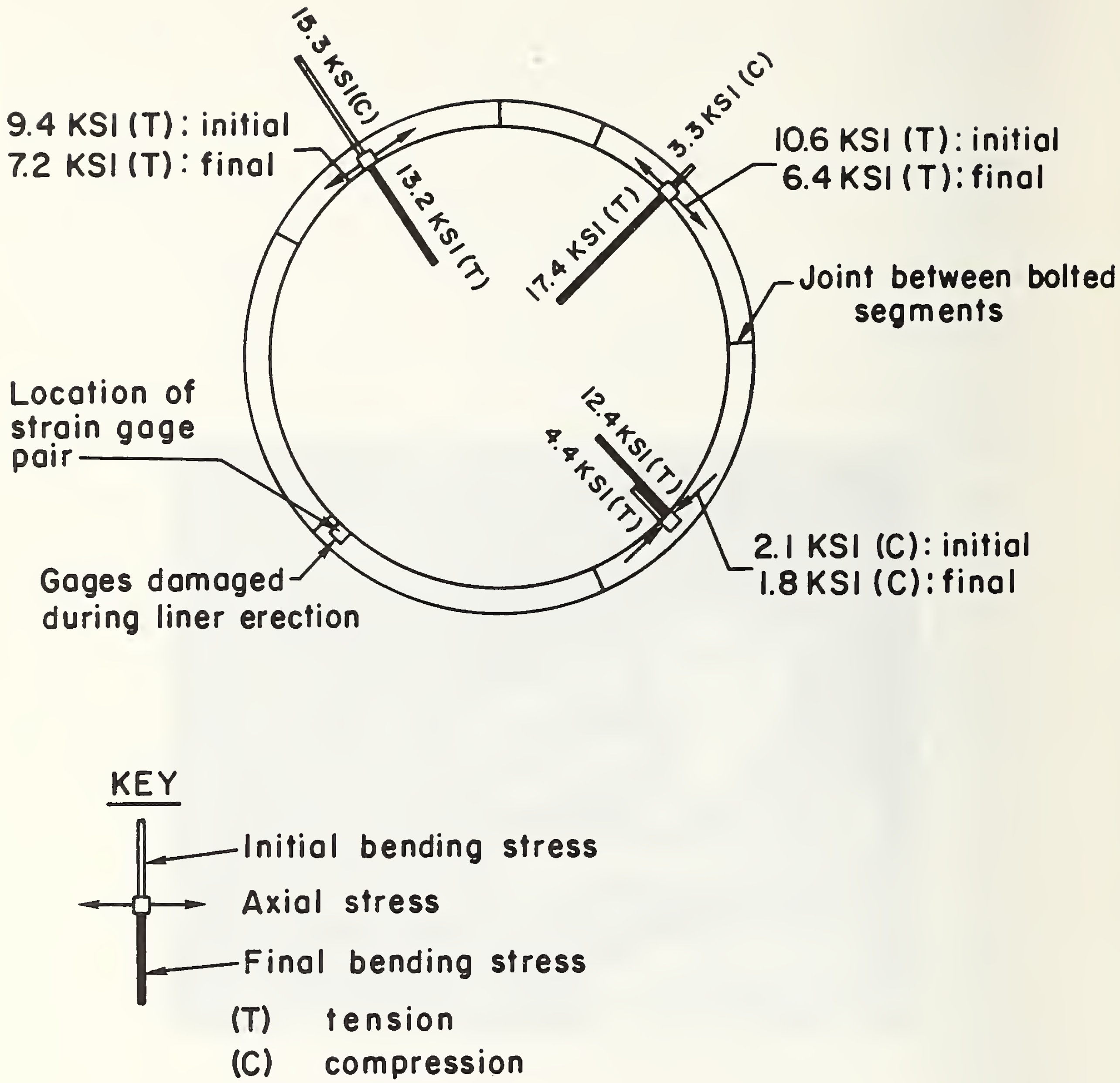
The strain gages that were used were of the weldable vibrating wire type (SINCO Model No. 52621). The locations for the gages was a problem since there is hardly any position on a ring of the segments which can be considered typical. As can be seen in Figure 3.4, there are a large number of stiffening plates and bolt holes, along with a number of grout nipples set around the plate. The stiffening plates and bolt holes can produce stress concentrations, and grouting through the nipples often caused local flexure of the segment, all of these effects undesirable from the point of view of strain measurement. It was decided to locate the gages in pairs on the segments of the butt plates between rings where no bolt holes would be encountered. This position can be seen in the photograph in Figure C.1. The pairs of gages were positioned at four equidistant locations around the liner with their axes of strain measurement perpendicular to the tunnel axis. Thus, the gages should ideally measure circumferential loads and resulting bending stresses on the liner, but not be able to detect longitudinal load effects such as would be induced by the shove jacks.

The gage itself consists of a wire mounted onto a plate which can be spot welded onto steel which has been cleaned of rust or contamination. The plate is 2.65 in. (6.7 cm) by 0.3 in. (0.8) and the wire is 0.07 in. (0.2 cm) in diameter. A SINCO Model S2622 pickup sensor was mounted permanently on top of the gage. Readings were obtained within the tunnel using a portable SINCO Model 52601 Strain Indicator. The strain gage data were interpreted in terms of bending and circumferential stresses in the tunnel liner. Complete data sets were obtained only for Lines 2 and 4. A readout gage malfunction during liner erection at Line 3 prevented all but long term data from being collected. Nineteen of the 24 gages installed at all of the lines remained operational throughout the monitoring program, a 80% survival rate.

Summaries of results of data from Lines 2 and 4 are presented on Figure C.2 and C.3, respectively. Bending stresses and circumferential (axial) stresses shown on these figures were computed using data obtained after liner assembly inside the shield as the zero stress conditions. Initial values refer to stress changes measured after the liner segment was first exposed to earth loading. Final values were obtained 11 weeks after initial earth loading.

Results shown on Figures C.2 and C.3 indicate that a consistent pattern was not observed for magnitude or sense of axial or bending stresses developed during earth loading. However, lower bending stresses are noted for the gage sets located closest to the joints between the bolted segments. This implies that the bolted connections provided for partial moment transmission between segments, thus reducing the overall liner stiffness from that assumed for a continuous liner. The seemingly erratic response of the gages can be attributed to differences in applied torque to the bolts, during assembly in the liner, stress concentrations in the liner itself, residual stresses in the liner caused by the jacking loads, and lack of redundancy in the gage arrangement.

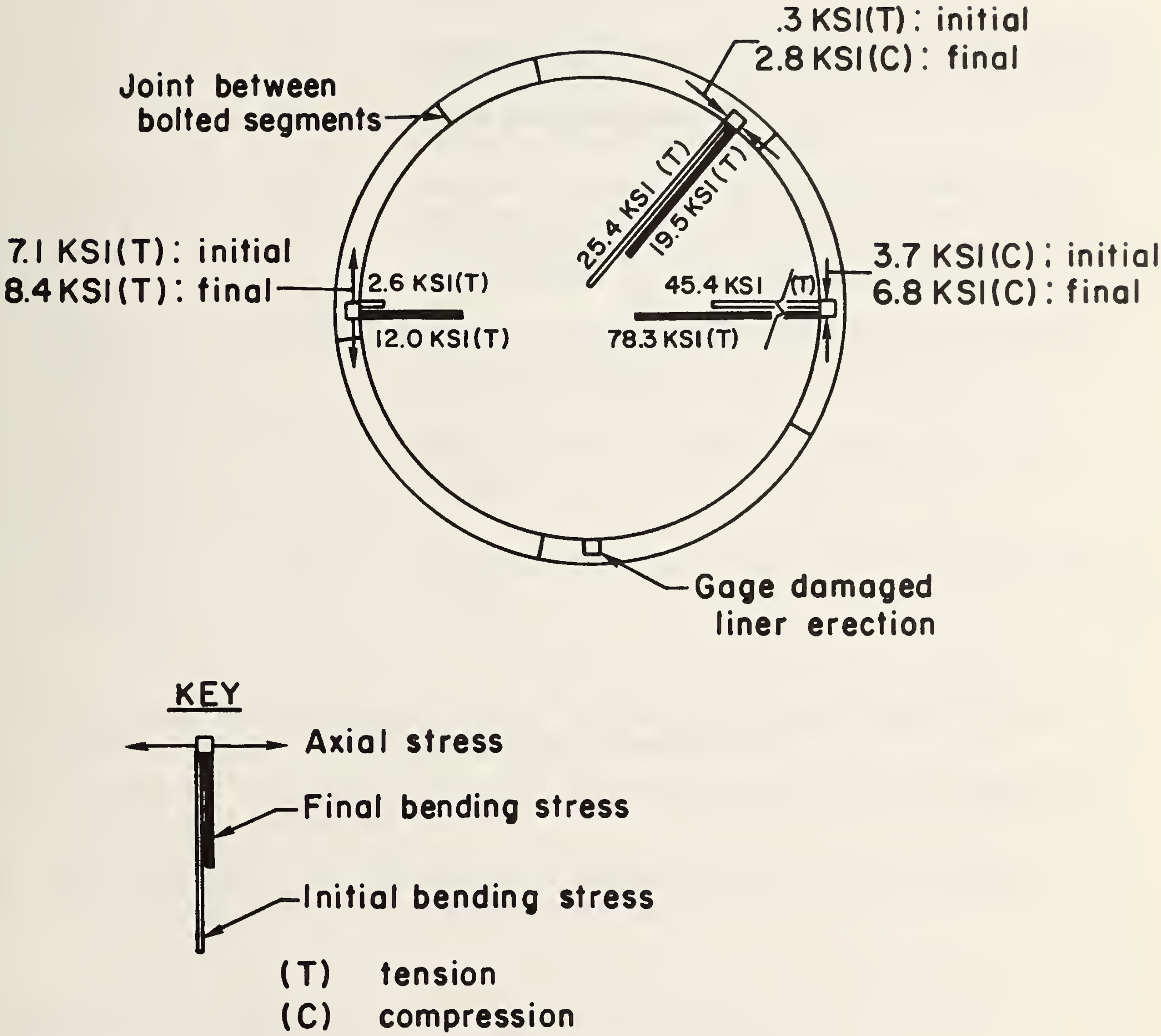
Figure C.1: Strain Gage Installation Detail



Notes

1. Stresses were computed using data obtained after liner erection inside the shield as the "zero" stress condition
2. Initial values refer to stress changes measured after the liner segment had first been exposed to soil loading
3. Final values were obtained 11 weeks after initial values

Figure C.2: Liner Stresses Due to Earth Loading at Line 2



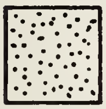
Notes

1. Stresses were computed using data obtained after liner erection inside the shield as the "zero" stress condition
2. Initial values refer to stress changes measured after the liner segment had first been exposed to soil loading
3. Final values were obtained 8 weeks after initial values

Figure C.3: Liner Stresses Due to Earth Loading at Line 4

Appendix D

KEY TO SYMBOLS USED ON SUBSURFACE PROFILES

SYMBOL	USCS DESIGNATION	DESCRIPTION
	CH	Inorganic plastic to very plastic CLAYS with liquid limit >50% and which plots above "A" line on plasticity chart
	CL	Inorganic slightly plastic silty CLAYS, and medium plastic to plastic CLAYS with liquid limit <50% and which plots above the "A" line and hatched zone on the plasticity chart
	ML	Inorganic non-plastic and slightly plastic SILTS and medium plastic clayey SILTS with liquid limit <50% and which plots below the "A" line and hatched zone on the plasticity chart
	CL/ML	Inorganic slightly plastic SILT and CLAY with the limit data plotting in the hatched zone on the plasticity chart
	SP	SANDS with less than 5% ML, CL, or CH fines
	GP	GRAVELS with less than 5% ML, CL or CH fines

1. Soil is classified according to the Unified Soil Classification System (USCS), ASTM D 2487-69 and D 2488-69
2. A combined classification, ie SC is graphically shown by combining th symbols for each soil.
3. Any combination of designations separated by a slash means alternating beds of the designated soil

PLASTICITY CHART:

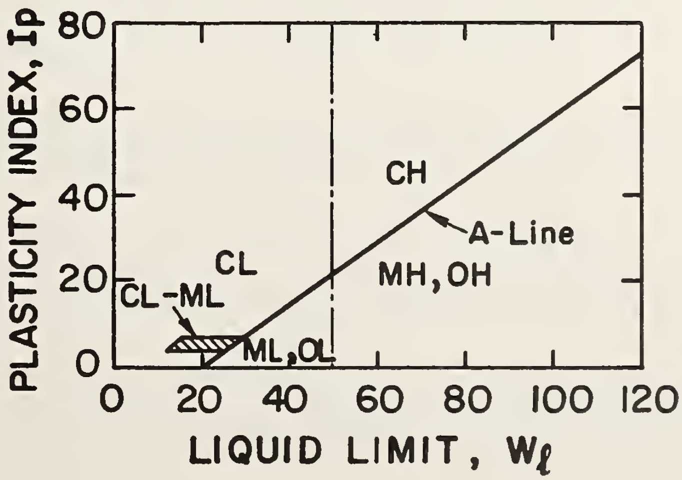


Figure D.1: Explanation of Symbols Used on Subsurface Profiles

Appendix E
REPORT OF NEW TECHNOLOGY

The work performed under this contract has lead to no new technological inventions. Conclusions and recommendations regarding various types of equipment and procedures, design parameters, and soil/structure interaction are intended to expand and improve the state-of-the-art of tunnel design and construction in soft ground.

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