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SETTLEMENTS AROUND TUNNELS IN SOIL: THREE CASE HISTORIES

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16. Abstract Three case studies of field observations of ground surface and subsurface settlements associated with shield-driven soil tunnels are reported. Two of the cases are twin, 20-ft (6-m) diameter, single-track tunnels for the Washington, D. C. Metro System. Section F2a, F Route, is a steel segment lined tunnel in interbedded sands and gravels and clays typical of downtown Washington. Section Gl, with an expanded rib and lagging lining, is in a transition from these deposits to a hard, fissured clay. The third case is a 9-ft (3-m) diameter sewer tunnel with an expanded rib and lagging lining driven in dewatered, dense sands at Rockford, Ill.

Detailed measurements of subsurface settlements at points 3 to 6 ft (1 to 2 m) above the tunnel crowns are used to determine sources and magnitudes of lost ground. Where the tunnel face was controlled to prevent large losses, ground losses due to overcutting and plowing of the shield were about one-half of the total estimated ground loss; incomplete filling of the tail void was the next biggest source of loss.

Ground surface settlement data, including widths, slopes, and volumes of the surface settlement troughs, are reported for several cross-sections on each tunnel and for points along the tunnel centerlines. The volume of surface settlement was less than the volume of ground loss because the disturbance of tunneling caused a net volume expansion in the dense granular materials. The relationship between ground loss and surface settlement volume, as shown by sand bin model test data, is also reported. A procedure for estimating the ground loss and surface settlement in advance of tunneling is suggested.

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PREFACE AND ACKNOWLEDGEMENTS

This report summarizes one year of field observations of surface and subsurface settlements around tunnels in soil conducted by the Department of Civil Engineering, University of Illinois for the Office of the Secretary, and the Urban Mass Transportation Administration, U. S. Department of Transportation, under Contracts DOT-OS-70024 and DOT UT-80039. The sponsors technical representatives were Mr. Russell K. McFarland and Mr. Gilbert Butler.

Field observations from three soil tunnel projects are summarized in the report. Field data for two of the case histories was collected by University of Illinois personnel. Data for the third case history was supplied by the Sanitary District of Rockford, Illinois. The main body of the report summarizes the case history data in light of previous field observations and model studies conducted by the University. The case histories are reported in detail in the Appendices.

The collection of this case history and field data has involved the generous support and assistance of many individuals and groups. The Rockford tunnel data was obtained through the courtesy of Mr. Kenneth D. Miller, Director of Engineering of the Sanitary District of Rockford. The field observations were made under the direction of Mr. Boyd Schleicher, Project Engineer, and his assistance in collecting and correlating the data is gratefully acknowledged. The assistance of the geotechnical consultants for the Sanitary District, Drs. A. J. Hendron, Jr., R. E. Heuer, H. W. Parker, and G. Fernandez-Delgado, is greatly appreciated.

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Field observations for the Metro projects were directed by H. H. MacPherson with the assistance of L. Lorig, J. W. Critchfield, M. D. Boscardin, J. G. Ulinski, C. P. Rodriguez, B. Johnson, and P. A. Lenzini. The model tests were done by S. W. Hong and A. P. Vonderohe.

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CHAPTER 1

INTRODUCTION

In urban areas, ground movements that develop around tunnels in soil during construction can cause significant building and utility damage. The prediction and control of ground movements is therefore a major concern of the engineers and agencies involved in tunnel construction. A previous report (Cording, et al., 1976) for this contract summarized relationships for:

1. comparing the volume of ground lost during tunneling with the tunneling procedures, shield behavior and configuration, and ground conditions.
2. relating the volume of the surface settlement trough to the volume of ground lost during tunneling and to volume changes in the soil mass.
3. evaluating the distribution of lateral and vertical ground movements at the surface.

A fourth category, the relation between ground movements and building damage has been studied in another phase of the contract (O'Rourke, Cording, and Boscardin, 1976; Boscardin, Cording, and O'Rourke, 1977). In this report, additional field observations of tunnels carried out or reviewed by University personnel in the past year, have been summarized. The observations have provided additional information on the relationships in the first three categories.

This report presents the results of three case histories of field observations of settlements around tunnels in soil. Two of the case histories are from the Washington, D.C. Metro System, Section F2a - F Route and Section G1. The Section F2a - F Route tunnels were mined through the Pleistocene terrace deposits that are typical of the Washington D. C. area. Case histories of the Section A2 and Section F2a - L Route tunnels, which were mined through the same deposits, were reported previously (Cording, et al., 1976). The data reported from Section G1 is for a portion of the tunnels located at the transition from the Pleistocene deposits to a hard, fissured Cretaceous clay. The third case history is a small sewer tunnel, driven in dewatered, medium dense sands at Rockford, Illinois.

Table 1.1 summarizes the soil and construction conditions for the three new cases, numbers 3b (F2a - F Route), 18 (Rockford), and 19 (G1). Two previously reported case histories from Washington, D.C., numbers 1 (A2) and 3a (F2a - L Route), are also included for comparison since they have similar soil conditions, and in Case 1, different construction details. Details of the new case histories are presented in the appendices. The depth of the tunnels, z, listed in the table is the depth from the ground surface to the tunnel axis. The diameter of the tunnels, 2R, is the nominal excavated diameter.

TABLE 1.1

SOIL AND CONSTRUCTION CONDITIONS

Case	Soil type	Construction method and initial lining
1 Washington, D.C. Metro Section A2, twin, single-track tunnels, 2R = 21 ft (6.4 m), z/2R = 2.3 (Hansmire, 1975; Cording, et al., 1976)	Medium - dense silty sand and gravel interbedded with sandy, silty clays	Shield with digger arm. Steel ribs and timber lagging expanded after shove. Partial dewatering with deep wells spaced 400 ft (120 m) on center.
3a Washington, D.C. Metro Section F2a, L'Enfant - Pentagon (L) Route, twin, single-track tunnels, 2R = 18 ft (5.4 m), z/2R = 3.7 to 4.1 (Cording, et al., 1976)	Dense sand and gravel, very dense clayey sand, overlain by silty sand and gravel interbedded with sandy, silty clays	Articulated shield with digger arm. Steel segments erected within tailskin and grouted prior to shove. Partial dewatering with deep wells spaced 160 ft (50 m) on center.
3b Washington, D.C. Metro Section F2a, Branch (F) Route, twin, single-track tunnels, 2R = 18 ft (5.4 m), z/2R = 2.0 (Appendix A)	Medium - dense silty sand and gravel interbedded with sandy, silty clays	Identical to L Route (Case 3a)
18 Rockford, Ill., ESLLIRP, Contract 1A, sewer tunnel, 2R = 9.3 ft (2.8 m), z/2R = 3.4 to 4.5 (Appendix C)	Medium - dense sands with some gravel	Rotating-wheel tunnel mole. Steel ribs with timber lagging expanded after shove. Dewatered with deep wells spaced 200 ft (60 m) average on center.
19 Washington, D.C. Metro Section G1, second tunnel, twin, single-track tunnels, 2R = 21 ft (6.4 m), z/2R = 2.1 (Appendix B)	Transition from sandy, silty clays and medium - dense, silty sand and gravel to hard, fissured clay	Articulated shield with digger arm (same as Case 3). Steel ribs and timber lagging expanded after shove. Dewatered with deep wells on irregular spacing.

CHAPTER 2

VOLUME LOSS AROUND TUNNELS

2.1 GENERAL CONCEPTS

Settlement and horizontal displacement of the ground surface resulting from tunneling can be related to the volume of ground lost during the tunneling operation. Lost ground is the amount of excavation in excess of the volume of the tunnel. Major sources of lost ground during the tunneling operation are 1) loss of material in the tunnel face, 2) loss over the tunnel shield, 3) loss on erection of the tunnel lining (at the tail of the shield), and 4) loss with time as the heading advances and the lining deflects. Lost ground can be evaluated by observing the displacements of the soil immediately surrounding the tunnel. The simplest and most frequently used observation is the measurement of the settlement at a point in the soil 3 to 5 ft (1 to 2 m) above the tunnel crown. The displacement, or deep settlement, of this point can be correlated with the advance of the heading and the various tunneling operations.

Figure 2.1 is a typical correlation of deep settlement with heading (or shield) position. The deep settlement provides an immediate indication of major sources of lost ground. Ground surface settlements do not provide this detailed picture. Three-dimensional spreading of the ground loss to the surface and volume changes in the soil above the tunnel results in the surface settlement lagging behind the heading. The presence of pavement, buried utilities, or other structures may distort or bridge over settlement, thus further obscuring the relation between surface settlement, ground loss, and tunneling operations.

Ground loss due to loss through the face of the tunnel is reflected in soil displacements occurring ahead of the tunnel face (see Fig. 2.1). Face loss may be due to elastic or plastic deformation of soil into the tunnel face or runs or flows of soil in the tunnel face.

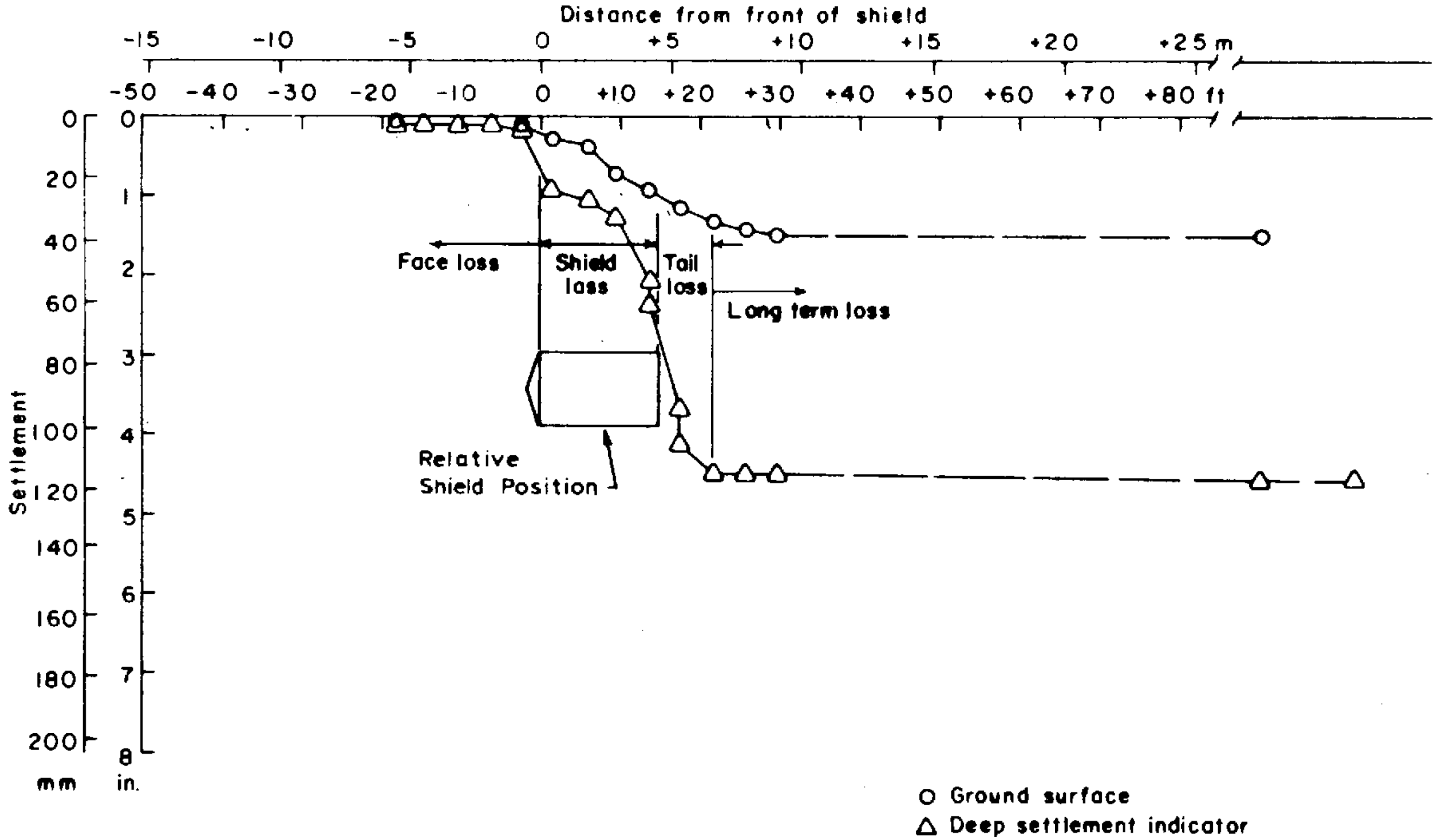


FIGURE 2.1 EXAMPLE OF DEEP SETTLEMENT MEASUREMENTS OVER A TUNNEL

Ground loss over the shield occurs between the leading edge and the tail. This loss may be due to overexcavation by projections on the outside of the shield (overcutter or poling plates for example), or it may be caused by plowing of the shield. The latter occurs when the shield is maneuvered through the ground at an angle to the tunnel axis. Plowing produces an elliptical rather than circular excavation with a cross-sectional area greater than the circular cross-section of the shield. The void left by overexcavation is filled by the collapse or squeezing-in of the surrounding soil. The volume of soil filling in the void is lost ground.

Ground loss during the erection of the tunnel lining, or tail loss, occurs when the lining emerges from the tail of the shield, and the annular void between the outside of the lining and the excavated soil surface is not completely filled with grout, or the lining is not expanded tight against the excavated soil surface. The remaining void is filled by collapse of the soil against the lining and ground is lost.

Long term ground loss occurs when any remaining unfilled voids in the soil left by the tunneling operations collapse and as the tunnel lining deflects with the application of the soil load. Additional ground loss may result from compression or volume decrease of loosened soil around the tunnel or from consolidation of the soil around the tunnel. The division between tail loss and long term loss in this report is taken at a point 2 to 3 shove lengths behind the tail of the shield. Figure 2.1 illustrates the assumed boundaries for the various categories of ground loss.

Table 2.1 summarizes deep settlements observed over the tunnels in the case histories presented in this report. The volume of the ground lost per unit length of tunnel can be approximated from the deep settlement measurements by the following empirical formula (Cording, et al., 1976).

$$V_L = 2\,\delta_v\,(R + y) \qquad \text{Eq. 2.1}$$

V_L = volume of ground lost per unit length of tunnel

δ_v = settlement at a point located directly over the tunnel at a distance, y, above the crown

R = radius of the tunnel

y = distance from crown to settlement point, $0 < y < 1/2\ R$

TABLE 2.1
LOST GROUND AROUND SINGLE TUNNELS

Case 1	Diameter 2R, ft 2	Depth to tunnel axis z, ft 3	z/2R 4	Lost ground Before face 5	Over shield 6	At tail 7	Lining deflection and time dependent movements 8	Total 9	Comments 10
1 Washington, D.C. Metro Section A2, first tunnel (Hansmire, 1975)	21	48	2.3	δ_V = 0.5 in. V_L = 0.7 ft^3/ft $\%V_L$ = 0.2%	9.8 in. 19.6 ft^3/ft 5.6%	1.6 in. 3.1 ft^3/ft 0.9%	1.7 in. 3.2 ft^3/ft 0.9%	13.6 in. 26.5 ft^3/ft 7.6%	Settlement point 1.5 ft above tunnel crown. Expanded lining.
Section A2, second tunnel				δ_V = 0.6 in. V_L = 0.7 ft^3/ft $\%V_L$ = 0.2%	2.3 in. 4.6 ft^3/ft 1.4%	0.9 in. 1.9 ft^3/ft 0.6%	1.1 in. 2.3 ft^3/ft 0.7%	4.9 in. 9.9 ft^3/ft 2.9%	Tunnels 36 ft apart cc. Different shield used on second tunnel.
3a Washington, D.C. Metro Section F2a, L Route, first and second tunnels at cross-sections (Cording, et al, 1976)	18	66 - 72	3.7 - 4.1	δ_V = 0.1 - 0.3 in. V_L = 0.2 - 0.7 ft^3/ft $\%V_L$ = 0.1 - 0.3%	0.2 - 1.2 in. 0.5 - 3.0 ft^3/ft 0.2 - 1.2%	0.1 - 0.2 in. 0.3 - 0.5 ft^3/ft 0.1 - 0.2%	0.1 - 0.4 in. 0.2 - 1.0 ft^3/ft 0.1 - 0.4%	0.7 - 2.1 in. 1.8 - 5.1 ft^3/ft 0.6 - 2.0%	Grouted lining. Construction procedure nearly identical to F Route. Tunnels 65 to 75 ft apart cc.
L Route - range of values along both tunnels		54 - 74	3.0 - 4.1	—	—	—	—	0.7 - 4.1 in. 1.8 - 8.8 ft^3/ft 0.6 - 3.4%	
3b Washington, D.C. Metro Section F2a, F Route, first tunnel at test section	18	36	2.0	δ_V = 0.2 in. V_L = 0.4 ft^3/ft $\%V_L$ = 0.1%	2.0 in. 4.3 ft^3/ft 1.7%	0.7 in. 1.5 ft^3/ft 0.6%	0.4 in. 0.8 ft^3/ft 0.3%	3.2 in. 7.0 ft^3/ft 2.7%	Settlement point 4 ft above tunnel crown. Grouted lining.
F Route, second tunnel at test section				δ_V = 0.4 in. V_L = 0.9 ft^3/ft $\%V_L$ = 0.4%	1.7 in. 3.7 ft^3/ft 1.5%	0.5 in. 1.0 ft^3/ft 0.4%	0.5 in. 1.1 ft^3/ft 0.4%	3.1 in. 6.7 ft^3/ft 2.7%	Tunnels 28 ft apart cc.
F Route - range of values along both tunnels		34 - 47	1.9 - 2.6	—	—	—	—	1.1 - 5.1 in. 2.6 - 12.0 ft^3/ft 1.0 - 4.7%	
18 Rockford sewer tunnel	9.3	32 - 42	3.4 - 4.5	δ_V = 0.1 - 0.4 in. V_L = 0.2 - 0.5 ft^3/ft $\%V_L$ = 0.3 - 0.7%	2.0 - 4.3 in. 2.8 - 5.5 ft^3/ft 4.1 - 8.0%	1.1 - 2.6 in. 1.6 - 3.4 ft^3/ft 2.3 - 5.0%	0.1 in. 0.1 ft^3/ft 0.2%	3.6 - 7.3 in. 5.0 - 9.4 ft^3/ft 7.3 - 13.7%	Wheel excavator type mole, expanded lining. Settlement points 3 - 4 ft above tunnel crown.
19 Washington, D.C. Metro Section G1, second tunnel	20.8	44	2.1	δ_V = 0.3 - 0.4 in. V_L = 0.8 - 1.0 ft^3/ft $\%V_L$ = 0.2 - 0.3%	0.9 - 0.7 in. 2.4 - 1.7 ft^3/ft 0.7 - 0.5%	0.4 - 0.3 in. 1.1 - 0.8 ft^3/ft 0.3 - 0.2%	0.6 - 0.3 in. 1.5 - 0.9 ft^3/ft 0.5 - 0.3%	2.2 - 1.7 in. 5.8 - 4.4 ft^3/ft 1.7 - 1.3%	Expanded lining. Settlement point 5 ft above tunnel crown.

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 ft^3/ft = 0.0929 m^3/m

The radius, R, is the nominal radius of the tunnel shield. The volume loss may also be expressed as a percentage (% V_L) of the nominal tunnel volume, πR^2, per unit of length. The volumes reported in Table 2.1 were estimated using this procedure.

2.2 RANGE OF OBSERVED VOLUME LOSSES

The data reported in Table 2.1 was developed from detailed observations at specific test sections located along the tunnels. For Cases 3a and 3b (Section F2a) additional data on total deep settlements was available from a number of points along the tunnels. This data has been used to indicate the range of deep displacement and loss that may occur.

On Section F2a, total ground losses were estimated to range from as low as 1 percent to as high as 5 percent of the nominal tunnel volume. This excludes two situations where large runs developed in the face. Average losses were about 1 percent on the L Route tunnels and 2 percent on the F Route. Higher than average losses occurred at locations where 1) the face was mostly in sand and gravel and small runs or raveling was more prevalent, 2) where the shield plowed more than normal or 3) the tail void was not completely grouted. The major difference between the L Route (Case 3a) and F Route (Case 3b) tunnels on Section F2a was that the L Route tunnels were much deeper and were more widely separated than the L Route tunnels. Otherwise the shields and construction procedures were nearly identical.

The estimated ground losses for the F2a tunnels were about the same order of magnitude as those previously observed on Section A2 (Case 1) of the Washington Metro. The A2 tunnels were mined in soils similar to those encountered on Section F2a (interlayered dense sands and gravels and stiff clays) and were at about the same depth as the F2a tunnels. The A2 tunnel shields were different however, and the tunnels were lined with expanded ribs and lagging rather than the grouted steel segments used on Section F2a. The first A2 tunnel had a shield with large projecting poling plates and difficult steering characteristics. These features resulted in excessive amounts of overexcavation. The lining expansion was also inadequate. The combination of excessive overexcavation and inadequate lining expansion led to large ground losses on the first A2 tunnel. These problems were corrected on the second tunnel, and the ground loss was reduced from 7.6 to 2.9 percent of the tunnel volume.

The ground loss data from Section G1 (Case 19) of the Washington Metro is from a limited section of tunnel. The range of ground loss, 1.3 to 1.7 percent of the tunnel volume, was similar to that of Section A2 (Case 1) and Section F2a (Case 3). The ground loss data for Section G1 was from a section of the second tunnel located at a transition from dense sand and gravel and stiff to medium clay to a hard plastic clay. The G1 tunnel shield was very similar to those used on Section F2a. The G1 tunnel was lined with expanded ribs and lagging.

The Rockford sewer tunnel (Case 18) had relatively large ground losses ranging from 7.3 to 13.7 percent of the tunnel volume. The tunnel was mined through dense sands that had been dewatered. The sands tended to run. Running sands are difficult to control in a tunneling operation, and any void formed by shield plowing or the tailskin of the shield will tend to fill in quickly. In raveling or firm ground, the same void may stand open long enough to allow grouting or expansion of the tunnel lining to fill the void and prevent ground loss.

2.3 SOURCES OF GROUND LOSS

The percentage of the estimated total ground loss attributed to each source is listed in Table 2.2. The face loss averaged about 10 percent of the total ground loss. (Excluding several large runs that occurred in Cases 3 and 18). Shield losses (overcutters and plowing) formed the largest source of loss, averaging about 50 percent of the total loss. Tail loss and long term loss averaged 23 and 18 percent of the total loss, respectively.

There is some difficulty in dividing up the observed deep settlement according to source. In firm ground, a void that formed during one operation, such as the tailskin void, may remain open for some time. When the

TABLE 2.2

SOURCES OF LOST GROUND AS A PERCENTAGE
OF TOTAL GROUND LOSS

Source / Case	Percent of total ground loss volume			
	Face	Shield	Tail	Long Term
1. - 1st tunnel	2.4	74.0	11.8	12.2
2nd tunnel	6.5	46.7 *	19.6	22.8
3.a - 1st tunnel	4.8 to 13.7	47.6 to 58.8	9.8 to 27.8	11.1 to 38.1
2nd tunnel	14.3	23.8	14.3	47.6
3.b - 1st tunnel	5.7	61.4	21.4	11.4
2nd tunnel	13.4	55.2	14.9	16.4
18.	3.3 to 16.7	29.2 to 58.5	32.0 to 50.0	1.1 to 4.2
19. - 2nd tunnel	13.8 to 22.7	38.6 to 41.4	18.2 to 19.0	20.5 to 25.9

See Table 2.1 for volumes of loss in each category and total volume of loss.

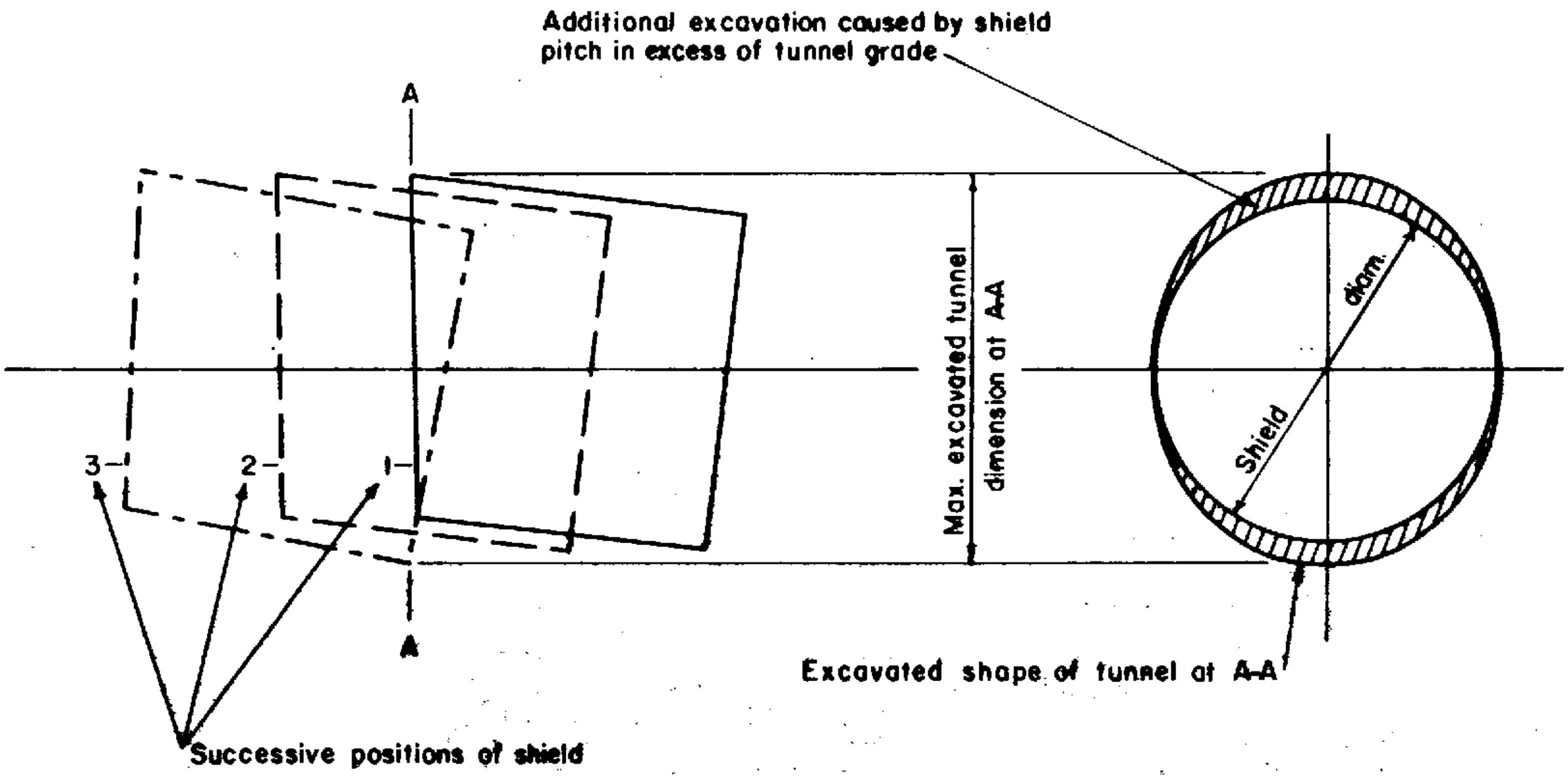
* Different shield on 2nd tunnel.

void finally collapses, the resulting settlement is recorded as part of the long term loss. Thus, some of the estimated tail loss may include some ground loss that actually is from voids formed over the shield. Likewise, some tail voids may be included in the estimated long term ground loss.

2.4 SHIELD LOSS ESTIMATES

As previously indicated, the ground loss that develops during the passage of the shield under a deep settlement point can come from two sources: 1) projections on the outside of the shield and 2) plowing of the shield. Case 1, Section A2, illustrates these effects. Poling plates projecting from the shield used for the first tunnel caused large ground losses (Table 2.1). Also, the large length/diameter ratio of the shield made steering difficult and resulted in excessive plowing. These problems were corrected on the second shield, and shield losses were reduced from 5.6 to 1.4 percent of the nominal tunnel volume.

The losses from projections and plowing can be computed theoretically. Ground loss per unit length of tunnel caused by projections is equal to the difference between the cross-sectional area of the shield including the projections and the circular area computed using the nominal outside diameter of the shield at the tail. The loss from plowing is more difficult to compute, since it is caused by both the shield attitude and the shield position relative to the tunnel axis. The shield attitude has both vertical (pitch) and horizontal (yaw) components. In addition, the shield may be above or below, and right or left of the tunnel axis. If surveys of the shield position and attitude are available for each shove, the vertical and horizontal location of the leading and trailing edges of the shield can be computed for each shove. The locations of the shield edges define the limits of excavation, and from this the shape and volume of the excavation can be computed. The volume of the shield is subtracted from the excavation volume to determine the volume of overexcavation due to plowing. Figure 2.2 illustrates this procedure.



Similar pattern of overexcavation created by shield yaw (horizontal angle)

FIGURE 2.2 OVEREXCAVATION DUE TO SHIELD PLOWING

In Table 2.3 theoretical shield volumes computed in this manner are listed along with shield loss estimated from the deep settlements using Eq. 2.1. For the articulated shields used on Cases 3a and 3b (Section F2a) and Case 19 (Section G1) the plowing computation has been adjusted to include the effect of the angle between the front and middle shield sections as the shields maneuvered through the curves at the test section locations. The effective shield length has also been reduced on the assumption that the trailing tail section of the articulated shields tracked along freely and did not contribute to any plowing. Detailed data was not available for Case 18 (Rockford sewer), and the pitch was estimated at 1 percent in excess of the tunnel grade. Since the losses from both the overcutters and plowing start at the leading edge of the shield, the observed losses could not be separated into two components.

The shield losses estimated from the observed deep settlements (line 5) were generally less than the theoretical losses (line 4), except for one section in Case 18. The observed losses may be underestimated for several reasons. Equation 2.1, which was used to convert the measured deep settlements to volumes of ground loss, may underestimate the losses. In firm ground, some of the void created by the shield may remain open until after the tail of the shield has passed by the deep settlement point. Since this portion of the void does not produce settlement of the deep point during the passage of the shield, its volume is not included in the observed shield ground loss estimated from the deep settlement.

The theoretical losses due to plowing may be overestimated also. A shield moving through the ground at an angle to the tunnel axis plows out ground in two regions: 1) above the springline at the leading edge and 2) below the springline at the tail (see Fig. 2.2). The ground lost at the leading edge is excavated in through the face. At the tail, however, the soil in the invert is confined and cannot be easily pushed out of the way. Some of this soil may be shoved or squeezed into the tailskin void, but much of the soil beneath the shield may simply be compressed, especially if the shield pitch is small. A portion of the loss in soil volume due to compression may be regained elastically as soon as the shield tail passes. Thus, the net observed loss may be less than the theoretical loss. Hansmire (1975) discusses this problem in conjunction with the detailed ground loss observations made on Section A2 (Case 1).

TABLE 2.3

GROUND LOSS OVER SHIELD

	Case					
	3a		3b		18	19
	First tunnel	Second tunnel	First tunnel	Second tunnel		Second tunnel
Shield diameter, ft	18	All four shields identical except for overcutters.			9.3	20.8
Shield length, ft	19.6 (13.4)				15	21.2 (14.4)
Length/diameter	1.1 (0.8)				1.6	1.02 (0.7)
Hood length/total length	0.2 (0.3)				0	0.25 (0.37)
1. Volume of overcutters and other projections, ft^3/ft	2.7	2.7	4.2	4.2	1.8	2.7
2. Average pitch or yaw in excess of tunnel grade or line, %	0.3 - 0.4% pitch (b) 0.3 - 0.4% yaw	1.2% pitch 1.3% yaw	0% pitch 0.3% yaw	0.8% pitch 1.0% yaw	1% pitch (estimated)	0.1 - 0.4% pitch 0.1 - 0.2% yaw
3. Volume lost due to pitching and yawing, ft^3/ft	1.5 - 2.8 (c)	2.5	3.8	0.4	1.1	0.4 - 1.3
4. Total shield loss, estimated from 1 and 3 above, ft^3/ft	4.2 - 5.5	5.2	8.0	4.6	2.9	3.1 - 4.0
5. Observed shield loss, ft^3/ft	0.9 - 3.0	0.5	4.3	3.7	2.8 - 5.5	2.4 - 1.7

Notes:

a. (Bracketed) values in Cases 3a and 3b are computed assuming that the articulation of the shield reduced the effective length from 19.6 to 13.4 ft.

b. Pitch and yaw of main body of articulated shields for Cases 3a and 3b. The effective pitch and yaw causing overexcavation depends on the attitude of the front section of the shield relative to the main body.

c. Computed from actual pitch and yaw, including the effect of front shield section articulation.

Conversion factors: 1.0 ft = 0.3038 m, 1.0 ft^3/ft = 0.0929 m^3/m

CHAPTER 3

GROUND SURFACE SETTLEMENT

Table 3.1 summarizes the volumes of ground surface settlement per unit of tunnel length, V_S, maximum surface settlement, δ_{max}, settlement trough widths, w, i, and β, and slopes observed at cross-sections located along the tunnels. These values are for single tunnels or for the first tunnel driven of a pair of tunnels. The various parameters reported are defined in Fig. 3.1. The volume of the surface settlement trough was determined from the cross-sectional area of the trough. The half-width of the trough, w, was estimated from the volume of the trough as:

$$w = \frac{V_S}{\delta_{max}} \qquad \text{Eq. 3.1}$$

If the shape of the trough approximates a Gaussian distribution as shown by the solid line on Fig. 3.1 (Schmidt, 1969), the parameter i can be determined from a plot of log δ_s versus the square of the distance from the centerline. Average and maximum slopes of the trough are also reported in Table 3.1. The longitudinal slope is parallel to the tunnel axis.

In Fig. 3.2, the trough width, w or i, is plotted in dimensionless form against depth of the tunnel. The boundaries suggested by Peck (1969) for settlement troughs developing in various soil conditions are shown on the plot. The points for Cases 1 and 3b fall close together. These tunnels were at similar depths ($z/2R = 2.0$ to 2.3) and in similar soils (interlayered sands and clays). Case 3a, however, had a much wider trough even though the construction methods were similar to Case 3b and the soil conditions were the same interlayered sands and clays. Case 3a was much deeper than Case 1 or 3b. The greater depth caused the settlement to be spread out over a wider area. A thick layer of stiff clay was also present over the crown of the tunnels at the observed cross-sections in Case 3a.

TABLE 3.1

VOLUMES AND DISPLACEMENTS FOR SINGLE TUNNELS

Case	Diameter 2R, ft	Depth to tunnel axis z, ft	z/2R	Vertical displacement δ_{max}, in. surface	Vertical displacement δ_{max}, in. deep	Volume of movement ft^3/ft surface V_S	Volume of movement ft^3/ft tunnel V_L	Trough width ft i/R	Trough width ft w, ft	Slope of surface settlement trough cross-section ave.	Slope of surface settlement trough cross-section max.	Slope of surface settlement trough longitudinal section
1	2	3	4	5	6	7	8	9	10	11	12	13
1 Washington, D.C. Metro Section A2, first tunnel (Hansmire, 1975)	21	48	2.3	6.0	13.6	18.3 (5.3%)	26.5 (7.6%)	1.3 - 1.7 (β = 28° - 36°)	35 - 45	1:75 - 1:180	1:50 - 1:140	1:50 - 1:200
3a Washington, D.C. Metro Section F2a, L Route, first tunnel at cross-sections (Cording, et al., 1976)	18	66 - 72	3.7 - 4.1	0.1 - 0.4	0.7 - 2.1	2.1 - 4.0 (0.8 - 1.6%)	1.8 - 5.1 (0.6 - 2.0%)	3.1 - 3.6 (β = 39° - 45°)	69 - 82	1:2500	—	1:1800 - 1:2500
Range of values along L Route first tunnel		54 - 73	3.0 - 4.1	0.1 - 1.1	0.7 - 4.1	0.7 - 5.8** (0.3 - 2.3%)	1.8 - 8.1 (0.6 - 3.4%)	—	—	—	—	—
3b Washington, D.C. Metro Section F2a, F Route, first tunnel at cross-sections (Appendix A)	18	36	2.0	1.8	3.2	4.4 (1.7%)	7.0 (2.7%)	1.3 (β = 30°)	30	1:200	1:180	1:380 - 1:1000
Range of values along F Route first tunnel		34 - 47	1.9 - 2.6	0.1 - 1.8	1.1 - 5.1	0.9 - 5.0* (0.4 - 2.0%)	2.6 - 12.0 (1.0 - 4.7%)	—	—	—	—	—
18 Rockford sewer tunnel, at cross-sections (Appendix C)	9.3	32 - 45	3.4 - 4.8	0.6 - 1.8	3.6 - 7.3	1.0 - 4.3 (1.5 - 6.3%)	5.0 - 9.4 (7.3 - 13.7%)	1 - 3 (β = 11° - 36°)	11 - 29	1:130 - 1:470	—	1:375 - 1:650

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 ft^3/ft = 0.0929 m^3/m

* Estimated assuming $\beta = 30^0$, $w = z \tan\beta + R$, and $V_S = \delta_{max} w$

** Estimated assuming $\beta = 39^0$

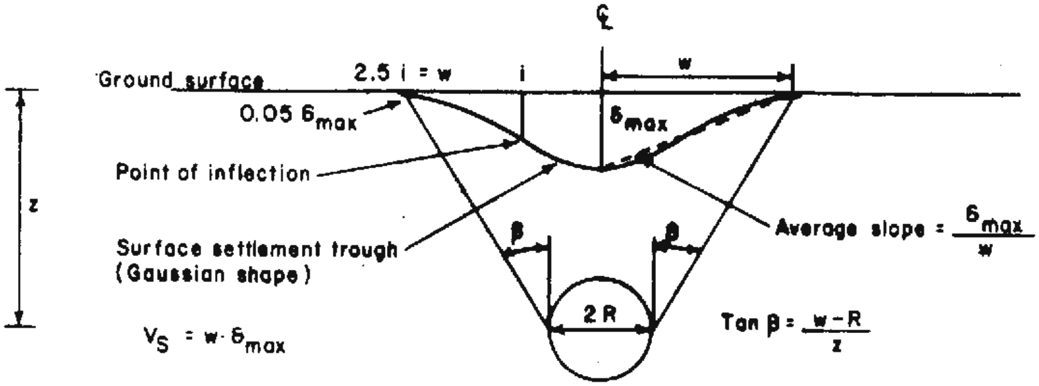


FIGURE 3.1 GROUND SURFACE SETTLEMENT TROUGH GEOMETRY

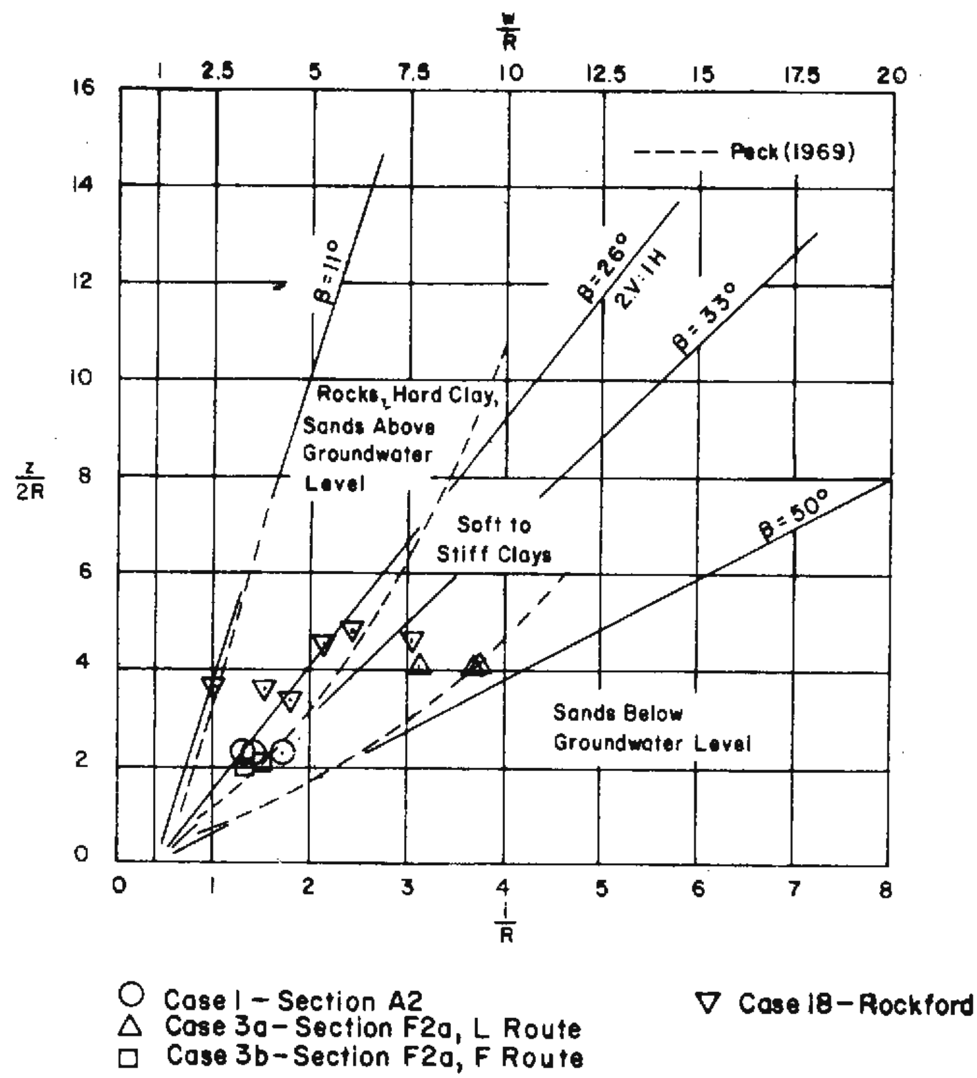


FIGURE 3.2 WIDTH OF SETTLEMENT TROUGH

This probably also helped to spread out the settlement. Finally, the volume of lost ground was slightly less in Case 3a than in the other two cases. This resulted in smaller, more elastic, soil movements and these smaller movements produced a wider settlement trough than in cases where there are larger movements. In general, large movements tend to produce a narrow, deep, central trough.

Cording and Hansmire (1975) observed that when large settlements, δ_{max}/z greater than 0.5 percent, occur, the settlement trough may have two parts: a narrow, deep, inner trough and a wide, shallow, outer trough. This was observed in Case 1 and also appeared to be present at the cross-sections for Case 3b. For Case 3b, δ_{max}/z was equal to 0.4 to 0.5 percent. Figures A.12 and A.13 show the Case 3b troughs.

CHAPTER 4

COMPARISON OF VOLUME OF SURFACE SETTLEMENT TO VOLUME OF GROUND LOSS, SINGLE TUNNEL CASE

4.1 INTRODUCTION

Cording, et al. (1976) compared volumes of ground lost and surface settlement for single tunnels in different ground conditions. The settlement volume, V_S, was related to ground loss, V_L, by:

$$V_S = V_L - V_E + V_C \qquad \text{Eq. 4.1}$$

V_E represents a volume gain resulting from expansion of the soil above the tunnel in response to the disturbance and displacements caused by tunneling and loss of ground into the tunnel. This volume expansion is most noticeable in dense sands. Tunnels in dense sands typically have significantly less surface settlement volume than volume of lost ground, due to an increase in soil volume caused by expansion of the disturbed material. However, surface settlement volume and ground loss volume are approximately equal for most tunnels in clay where consolidation settlements are not a problem. For clays, volume expansion due to tunneling disturbance would be negligible. In loose sands, volume decreases, rather than increases, might occur.

V_C represents a volume decrease due to compression of the soil along the sides of the tunnel as a result of redistribution of the soil stress with time as the tunnel is excavated and advanced. The magnitude of V_C will depend upon the properties of the soil around the tunnel and the magnitude of the stress changes. In many cases, V_C will be small; usually much less than any expansion that may occur.

In the next two sections, volume losses and settlement volumes are compared for a series of tests on model tunnels in a sand bin and for Cases 1, 3, and 18.

4.2 MODEL TESTS

A series of tests were run with model tunnels in a sand bin. The apparatus and testing procedures were described in detail in Cording, et al. (1976).

Figure 4.1 is a simplified sketch of the apparatus. The tunnel lining was represented by a steel pipe (diameter 2r) that extended from the plate glass front wall of the bin to the rear wall. A circular metal sleeve (diameter 2R) was placed over the pipe to represent a tunnel shield. The sleeve extended the full length of the inner pipe and could be pulled out through the rear of the bin to expose the inner pipe (tunnel lining). The model thus simulated the formation of the tail void at the tail end of a tunnel shield as the shield moves forward and exposes the tunnel lining. The potential volume of ground loss was equal to the volume of the tail void,

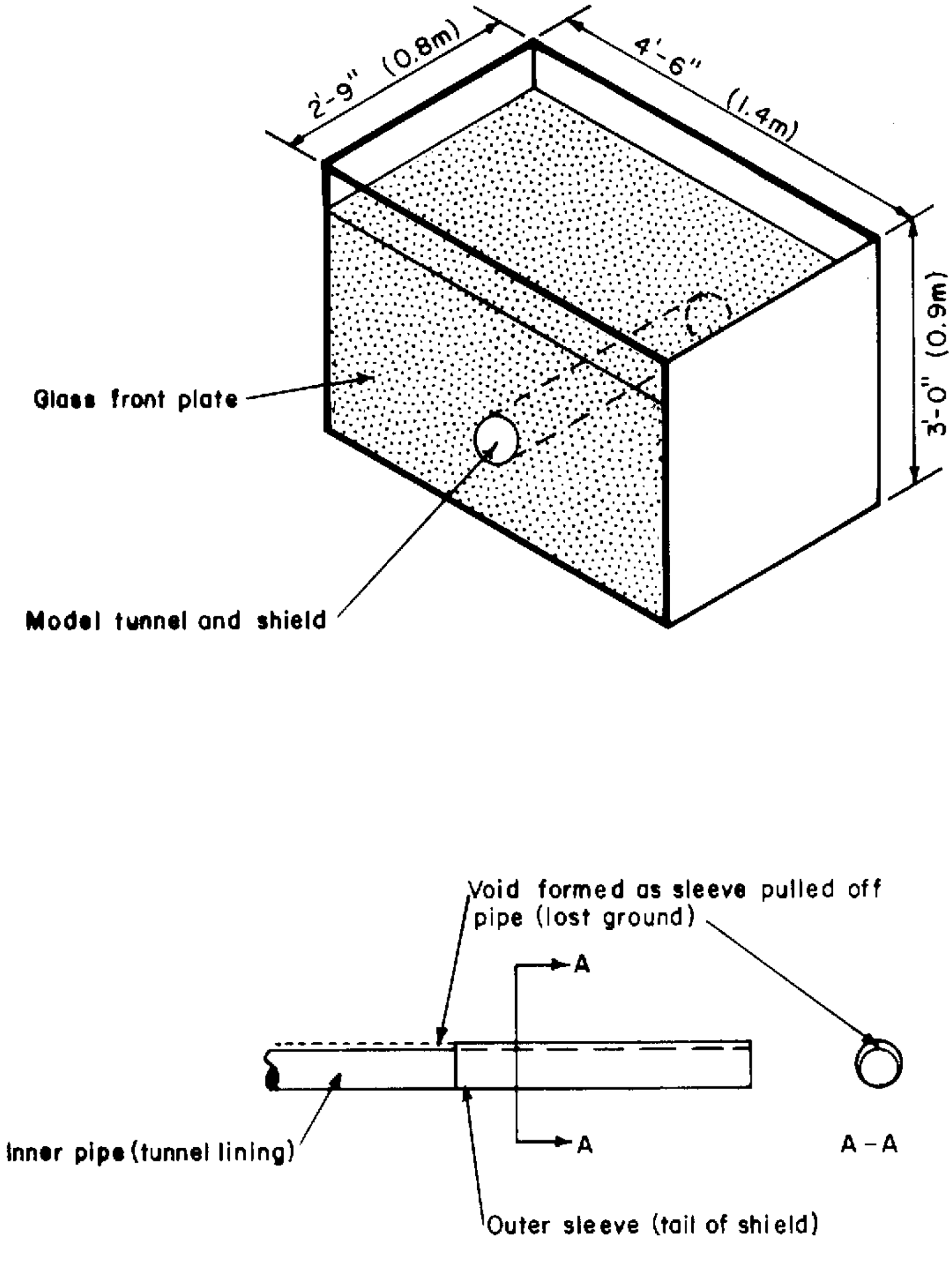
$$V_L = \pi (R^2 - r^2) \qquad \text{Eq. 4.2}$$

The volume of ground loss was varied by varying the relative sizes of the sleeve and inner pipe. The tunnel depth, z, was varied by using different heights of sand cover over the model tunnel. The sleeve was pulled off the pipe in stages and the settlements of the sand surface and several points buried at various depths above the tunnel were measured at each stage.

Mason sand was used in the tests reported here. The material was a uniform, medium sand with a trace of fine and coarse sand. Figure 4.2 shows the grain size curve for the Mason sand. The effective size, D_{10}, of the sand was 0.0075 in. (0.19 mm), and the material had a uniformity coefficient, C_u, of 2.00. The maximum unit weight of the sand was 112.3 pcf (17.6 kN/m^3); minimum unit weight was 90.0 pcf (14.1 kN/m^3). The sand particles were angular to sub-angular in shape.

The sand was placed in the bin by a controlled raining procedure where the height of fall and rate of deposition were kept constant. By varying the height of fall, the density of the sand could be varied in a narrow range. The relative densities, D_r, achieved in the model tests ranged from 62 to 76 percent.

Figure 4.2 also shows the grain size ranges for the glacial valley train deposits at the site of Case 18. These deposits consisted of a medium dense (N_{ave} = 25 blows/ft) sand and gravel, which was dewatered by deep pumping.



DETAIL OF MODEL TUNNEL

FIGURE 4.1 DETAILS OF MODEL TUNNEL AND SAND BIN

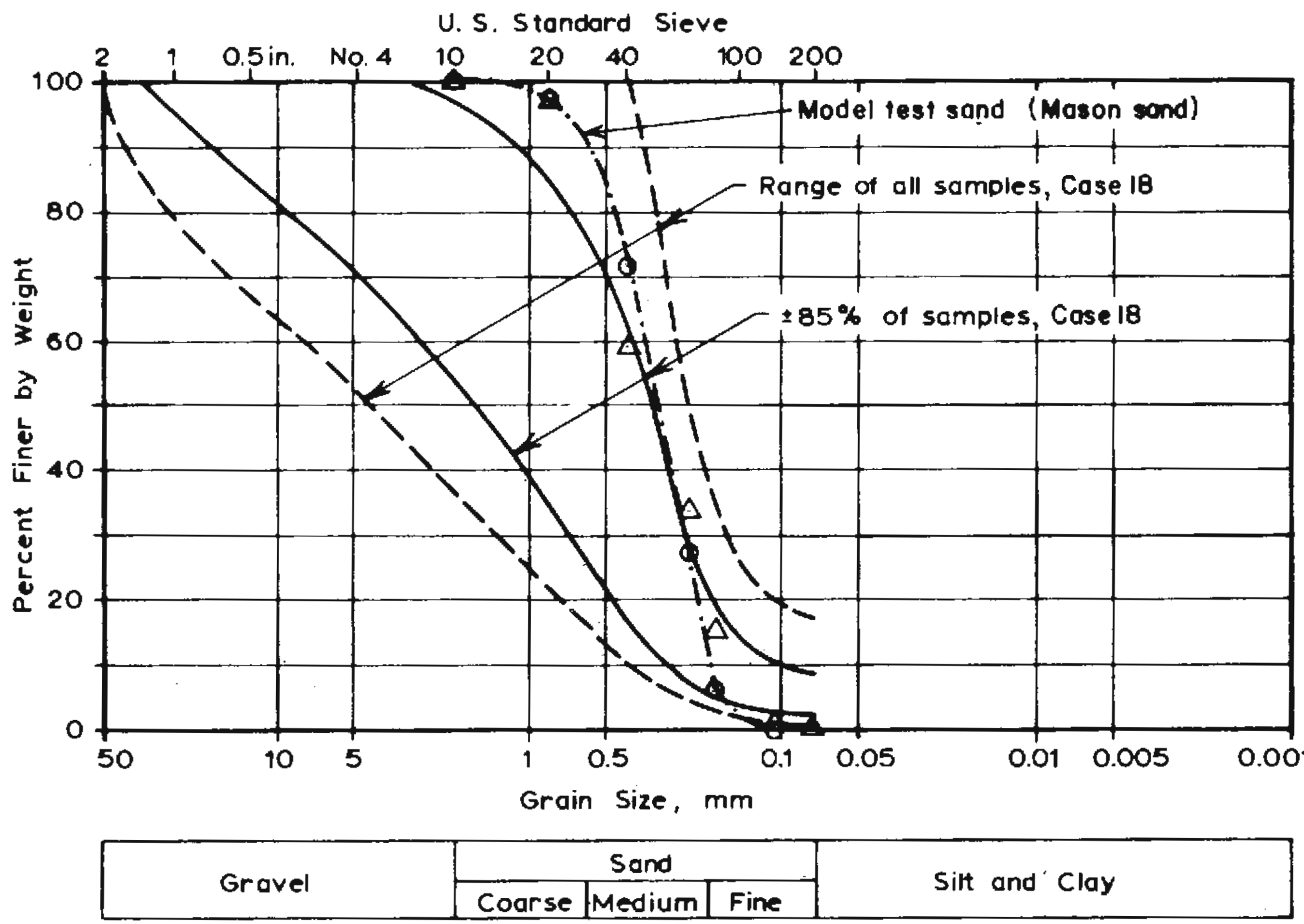


FIGURE 4.2 GRAIN SIZE DISTRIBUTION FOR MODEL TEST SAND

The ground losses, V_L, and surface settlement volume, V_S, as a percentage of tunnel volume for six tests are summarized in Table 4.1. Tests 7 through 10 used a shield (sleeve) with a diameter, 2R, of 5.94 in. (150.9 mm) and a tunnel (inner pipe) with a diameter, 2r, of 5.63 in. (143.0 mm). Tests 11 and 12 used values of 2R = 5.88 in. (149.4 mm) and 2r = 5.67 in. (144.0 mm).

TABLE 4.1

MODEL TEST VOLUMES

Test No.	z/2R	D_r %	V_L %	V_S %
7	1.60	68	9.0	3.3
8	1.64	72	9.8	2.8
9	2.93	62	9.8	3.5
10	2.57	70	9.8	2.0
11	2.23	74	7.0	1.4
12	2.93	76	7.0	1.4

When the shield was pulled off the tunnel in Tests 7 through 10, the sand did not run in under the tunnel, and a small void was left under the tunnel invert. This void was visible through the glass front of the sand bin. The volume of this void was estimated and subtracted from the volume of the annular gap between the shield and tunnel to determine the ground loss. No void was observed in Tests 11 and 12, and the annular gap volume was not corrected for any possible void.

The volume of surface settlement was determined from surface settlement measurements made with dial indicators resting on top of the sand. Both ground loss and settlement volume are reported as a percentage of the shield volume, πR^2.

In Fig. 4.3, the ground loss, % V_L, has been plotted against settlement volume, % V_S. The volume of surface settlement was considerably less than the volume of lost ground in the model tests. This indicates that the sand above the model tunnels expanded as it displaced downward towards the tunnel when the shield was removed. The test results indicate that both relative depth and relative density affected the amount of expansion that occurred.

Tests 8, 9, and 10 had the same ground loss, 9.8 percent. Tests 9 and 10 also had approximately the same relative depth, but Test 10 had a higher relative density than Test 9 (70 percent versus 62 percent). Test 10 had less surface settlement volume than Test 9. This indicates that, as might be expected, more volume expansion and hence a greater reduction in surface settlement occurred in the denser sand. A similar pattern is evident if Tests 7 and 8 or 9 and 12 are compared. In these cases, the variations in ground loss also affect the settlement volume.

Tests 8 and 10 had similar relative densities, but Test 8 was much shallower than Test 10 (z/2R = 1.6 versus 2.6). Since both their ground loss and relative densities were similar, the two tests should have had a similar volume expansion in the sand above the tunnels. However, since Test 8 was shallower than Test 10, there was less sand above the tunnel. Since there was a smaller volume of material available for expansion, the total volume increase was less for Test 8 than Test 10. Because Test 8 had less volume expansion to offset the ground loss, it had more surface settlement volume than Test 10.

Tests 11 and 12 should have shown a similar effect since the relative depth was the only difference between the two tests. However, both had identical settlement volumes. This may be because the difference in depth is not as significant as it was in Tests 8 and 10, and that a considerable difference in depth is required to result in major differences in total volume expansion.

Field data (Cording and Hansmire, 1975) indicate that at lower values of ground loss the volume expansion may tend to be a higher percentage of the ground loss. Thus at small values of ground loss very little surface settlement occurs. For larger volumes of ground loss, shear planes develop

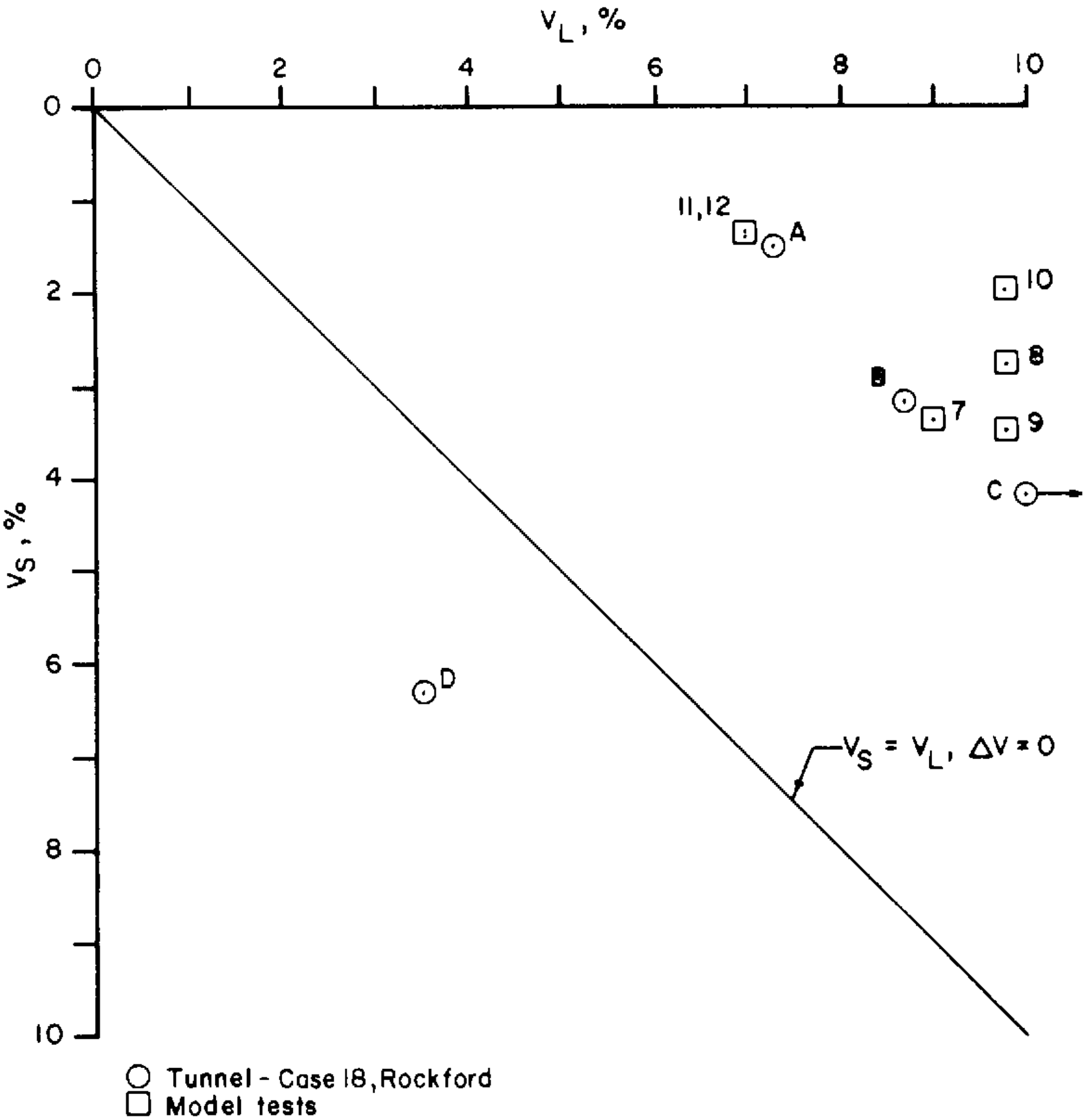


FIGURE 4.3 COMPARISON OF VOLUME OF LOST GROUND AND VOLUME OF SURFACE SETTLEMENT, SINGLE TUNNEL, MODEL TESTS AND CASE 18

upwards from the sides of the tunnel and approacn the ground surface, and volume expansion reaches a maximum. Further displacements do not produce additional volume increase, but only increased settlement. The model tests have involved relatively large and constant ground losses, and this effect of magnitude of ground loss on the volume of expansion is not apparent in the test data.

Data from Case 18, Rockford, has also been plotted on Fig. 4.3. The Rockford tunnel was at a much greater relative depth ($z/2R = 3.4$ to 4.5) than the model tunnels. The data for Case 18 is listed in Table 3.1 and in Appendix C. The Rockford tunnel had net expansion volumes of 63 to 79 percent of the volume of ground loss. The one point, D, that falls below the 45^{0} line indicates a volume decrease, but it is unlikely that the Rockford sand and gravel was loose enough for this to occur. The V_L estimate is probably in error due to a faulty deep settlement indicator. (See Appendix C).

Points A and B have the same relative depth ($z/2R = 3.5$), and presumably nearly the same relative density existed at the two measurement locations. Although the ground loss was larger at B, the net amount of volume expansion was less, 63 percent versus 79 percent at A. This fits the assumption stated earlier that percentage of volume expansion would decline as ground loss increased. Point C had a volume expansion of 69 percent of the ground loss, and the ground loss was one and a half times that at point B. However, the tunnel was also much deeper at C, and the increased depth would allow room for additional volume expansion.

4.3 FIELD OBSERVATIONS: CASES 1 AND 3

Figure 4.4 is a plot of percent surface settlement volume versus percent of ground loss volume for the first tunnels of Case 1, Section A2 and Case 3, Section F2a of the Washington Metro. Both projects are in the Pleistocene terrace soil sequence of Washington, D. C., which consists of alternating layers of dense sands and gravels and stiff clays.

The dashed line is the relationship proposed by Hansmire (1975) based on data from Case 1 (Points A, B, and C). As previously discussed, relatively more volume expansion occurs at small magnitudes of ground loss

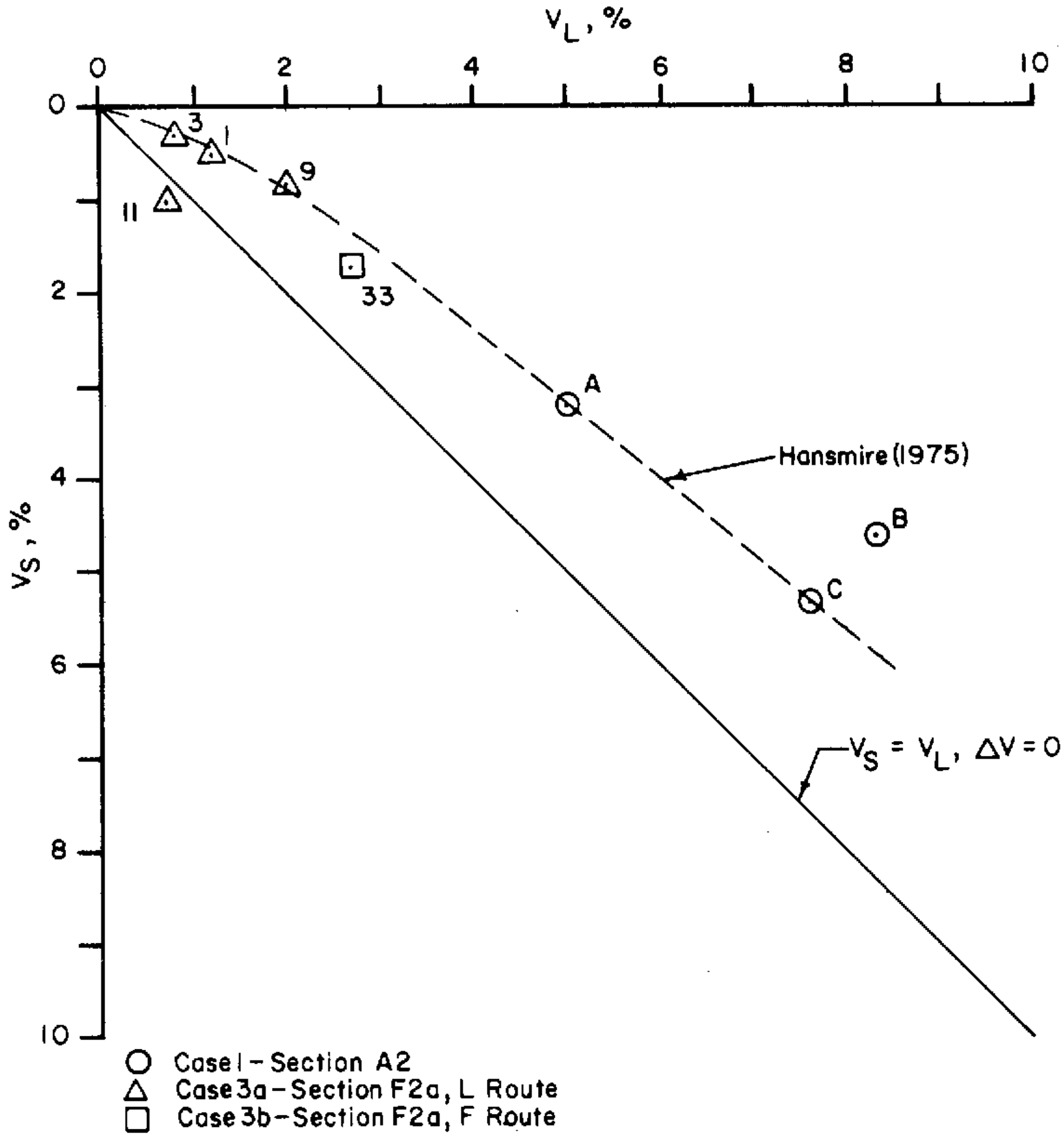


FIGURE 4.4 COMPARISON OF VOLUME OF LOST GROUND AND VOLUME OF SURFACE SETTLEMENT, SINGLE TUNNEL, CASES 1 AND 3, WASHINGTON, D.C.

than at larger values. As ground loss increases, a point is reached beyond which no further increase in volume expansion occurs, and additional ground loss results in an equal volume of surface settlement.

The data from Case 3 appears to fit this relationship. There is probably an effect of relative depth present as was seen in the model tests. The tunnel in Case 3a, represented by points 1, 3, 9, and 11, was at relative depths, z/2R, of 3.7 to 4.1. The relative depth for Case 3b, point 33, and Case 1 was 2.0 to 2.3. The greater relative depth of Case 3a probably resulted in additional volume expansion that reduced surface settlement volume, and consequently the points 1, 3, and 9 plot slightly higher than they would if the tunnel had been shallower. Point 11 shows a volume decrease, possibly due to errors in estimating V_S from the relative small magnitude of settlement or from an underestimate of V_L caused by the deep settlement indicator not being on the tunnel centerline.

The net volumes of expansion observed in Case 1 and 3a were about 30 to 45 percent of the ground loss. In Case 3b, the net expansion volume was about 58 to 63 percent.

CHAPTER 5

SUMMARY

5.1 CONCLUSIONS FROM FIELD OBSERVATIONS

This report and a previous one (Cording, et al., 1976) have presented the results of detailed observations of ground movements that have occurred during construction of five soil tunnel segments of the Washington, D. C. Metro. The tunnels were typical twin, 20-ft (6-m) diameter, shield-driven tubes. They were mined in hard, fissured clays or interlayered, dense sands and gravels and stiff clays at relative depths, z/2R, of 1.9 to 4.1. The results of the observations have shown the following:

1. Ground losses occurring during driving of a tunnel can be separated into two categories: 1) generalized losses that typically occur throughout the length of the tunnel drive and are due to the normal construction procedures employed in the typical ground conditions for the tunnel drive, and 2) large, localized losses (sometimes catastrophic losses) that occur at specific locations, usually due to some serious lapse or change in normal construction procedures or an abrupt change in ground or water conditions. For the five Metro tunnel segments studied, the general losses for a single tunnel have ranged from 1 to 5 percent of the nominal tunnel volume. For a small sewer tunnel (2R = 9.3 ft, 2.8 m; z/2R = 3.4 to 4.5) driven in difficult running sands at Rockford, Illinois much higher ground losses of 7.5 to 13 percent were observed.

2. Detailed field observations of deep settlements occurring immediately over the crown of a tunnel can be used to identify sources of ground loss and to estimate the magnitude of the ground loss. Ground loss can be categorized according to the source as 1) face loss, 2) shield loss (overcutter and plowing), 3) tail loss, and 4) long term loss. For the

five Metro tunnels, the largest sources of ground loss were the shield overcutters and plowing (above 50 percent of the total ground loss), with the tail losses forming the next largest portion (23 percent) of the total ground loss. Face losses were generally small for the Metro tunnels, but the potential for large, localized losses was present when the faces were in sands and gravels. Long term losses were small (10 to 30 percent of the total losses) in the stiff clays and dense granular Metro soils.

Large face losses occurred initially for the Rockford tunnel, but these were controlled as experience was gained in operating the excavator wheel, and face loss was reduced to 5 to 10 percent of the total ground loss. Shield losses were about 45 to 50 percent of the total ground loss, and tail loss formed about 30 to 50 percent of the total losses. Long term loss formed less than 5 percent of the total ground loss.

3. The approximate magnitude of the ground losses can be estimated by computing potential volumes of overexcavation created by overcutters on the shield, plowing of the shield, and failure to fill the tail void at the rear of the shield. For the Metro tunnels, the estimated losses compared reasonably well with the observed losses.

4. Average maximum ground surface settlements were about 1 to 2-1/2 in. (25 to 64 mm) for the tunnels studied. The settlement trough in the ground surface had a shape similar to a Gaussian distribution curve in most cases. Typical relationships between the width of the settlement trough and depth for a single tunnel for various soil conditions are shown in Fig. 3.2.

5. The field observations showed a direct relationship between the volume of ground surface settlement occurring and the volume of ground lost during tunneling. The surface settlement volume is the sum of the volume of ground lost, plus any decrease in volume of the soil around

the tunnel due to compression or consolidation, less any volume increase or expansion of the soil around the tunnel due to the disturbance of tunneling. (See Eq. 4.1). For the dense granular soils typical of many of the Metro tunnels, the volume expansion of the disturbed soil is large and offsets a portion of the ground loss so that the net surface settlement volume is much less than the ground lost during tunneling. There appears to be a limit to the amount of volume expansion that can occur in the soil disturbed by tunneling. Once this limit is reached, further increases in ground loss result in equal volumes of surface settlement. Deeper tunnels or tunnels in denser soils should have a greater potential volume expansion and hence less surface settlement volume than shallower tunnels or tunnels in less dense soils with equal ground losses.

6. In view of the direct relationship between surface settlement volume and ground loss, it is important to control and limit ground losses during tunneling in order to limit settlements. Desirable construction practices to reduce ground loss include:

a. adequate control and support of the tunnel face by advance dewatering, use of compressed air, advance grouting of the soils, full face breasting and breasting shelves, or other means,
b. limiting the projections on the exterior of the shield to reduce overexcavation,
c. providing a maneuverable shield with a small length-diameter ratio to reduce plowing of the shield,
d. using as thin a tailskin as practical on the shield, and
e. promptly and completely grouting the tail void while the lining is within the tailskin or as the lining emerges from the tailskin, or promptly expanding the lining into full contact with the soil as the lining emerges from the tail.

5.2 PROCEDURE FOR ESTIMATING GROUND LOSS AND SETTLEMENT

The following procedures might be used to estimate potential amounts of ground loss and related surface settlement for single tunnels or the first tunnel of a pair similar to those described in this report. It is assumed that positive steps will be taken to control and support the face and promptly fill the tail void. The procedure has the following steps:

1. Estimate expected ground losses, V_L, (face, shield, tail, etc.)
2. Estimate settlement volume, V_S using V_S vs V_L correlations or simply assume $V_S = V_L$.
3. Estimate settlement trough width, w, using Fig. 3.2.
4. Calculate surface settlement over tunnel centerline using the estimated V_S and w (Eq. 3.1).

This procedure provides an estimate of the magnitude and extent of the average surface settlement likely to occur for a tunnel driven with good workmanship and technique.

In addition, the designer should estimate the potential for large, localized ground losses due to runs or flowing in the face. Occurrences of large, localized ground losses may be more common than normally supposed. In soft soils, large ground losses and settlements due to squeezing-in of the soil at the face of the tunnel and consolidation of the soils around the tunnel must be considered.

Estimates of settlements developed in this manner will be very useful to the tunnel designer. They can form the basis for decisions about the need for and extent of underpinning required to protect buildings and utilities. The estimating procedures focus the designer's attention on the source of settlement, the ground loss around the tunnel during driving. In developing the estimates of ground loss, the designer must consider his assumptions about the support of the tunnel face, the configuration of the shield, and the method of filling the tail void, and he must consider to what extent the actual construction procedures will follow those envisioned in the design and specifications.

The settlement estimates also provide the construction manager and designer with a guide for assessing the actual tunneling work. In this aspect field observations should be used to 1) verify the estimates of ground loss and settlement volume and extent, and 2) suggest methods to reduce ground loss and settlement if necessary. Field observations of ground loss and surface settlement should be made at the start of tunneling, at locations where there are significant changes in ground conditions, tunnel geometry, or tunneling procedures, and as the tunnel approaches critical structures or buildings. Careful control of the tunneling process, aided by such field observation, will help insure that the cost savings in underpinning and other features assumed in the design stage are actually achieved in the field.

The next few sections discuss the various aspects of the procedures for estimating ground loss and settlement.

5.2.1 GROUND LOSS ESTIMATES

The volume of ground loss can be estimated by summing up the individual sources of loss: face loss, shield loss, tail loss, and long term loss.

1. face loss - Typical ranges of face loss for granular soils where the face is controlled are listed in Table 2.1 and in Cording, et al. (1976). Since face losses are erratic, these values can be considered as lower limits. The upper limit would be a run in the face and collapse of the surface.

2. shield loss - An upper bound on shield loss volume can be obtained by computing the volume of projections on the exterior of the shield and adding an allowance for overexcavation caused by plowing of the shield. Plowing volumes can be estimated by the following equation (Cording, et al., 1976):

$$V_{L,plow} = \pi R \frac{L}{2} \text{ (excess pitch or yaw)} \qquad \text{Eq. 5.1}$$

R = radius of shield

L = length of shield

An excess pitch of 0.5 to 1.0 percent in excess of the tunnel grade appears to be a reasonable estimate, unless the shield is unusually difficult to maneuver.

3. tail loss - The maximum tail loss is approximately equal to the difference between the outside diameter of the shield and the outside diameter of the ungrouted or unexpanded tunnel lining. The minimum loss would be equivalent to the thickness of the tailskin for the case where the tail void is grouted while the lining is still within the tail of the shield. For an expanded rib and lagging lining, a minimum loss would be equal to the difference between the volume of the expanded rib circle and the expanded lagging circle, although some additional, irretrievable losses may occur immediately behind the tail before the lining can be fully expanded into place.

4. long term loss - For the case histories presented herein, soil conditions were such that long term, time dependent losses were small. The values in Table 2.1 or those given in Cording, et al. (1976) can be used as a guide in estimating these losses. If lenses or layers of soft, compressible clays or other materials are present in the soil profile, the disturbance of the tunneling or dewatering can cause the compressible materials to consolidate, and this can result in a delayed loss and large long term settlements. Soft soil may also squeeze in through gaps between the lagging of rib and lagging linings and result in long term ground loss.

5.2.2 SURFACE SETTLEMENT ESTIMATES

Although some relationships between V_S and V_L have been presented herein for specific soil and construction conditions, additional research and field observations are needed to obtain a better understanding of the effect of relative density, tunnel depth, magnitude of ground loss, layering of soils, and other factors. For single tunnels or the first tunnel of a pair, assuming $V_S = V_L$ and ignoring possible volume expansion in granular soils will provide an upper bound on the volume of surface settlement. This may be overly conservative for deep tunnels in dense granular soils. The magnitude of surface settlement can then be estimated from the settlement volume using a settlement trough width estimated from information such as that shown in Fig. 3.2 or contained in Peck (1969) or Cording, et al. (1976).

5.2.3 VERIFYING ESTIMATES WITH FIELD OBSERVATIONS

The estimates of ground loss made using these procedures can be verified during construction by using cased, deep settlement points located on the tunnel centerline to measure subsurface settlements at a point 2 to 5 ft (0.6 to 1.5 m) above the tunnel crown. (See Cording, et al., 1975, for descriptions and details on settlement measuring devices and procedures for making such observations.) Careful and frequent observations of the subsurface settlement during the passage of the tunnel shield and correlation of these observations with passage of the shield and details of the construction can indicate the sources of ground loss and the magnitude of the loss associated with each source. Complete documentation of the construction procedures and details as the shield passes under the settlement point is especially important. This data is needed to determine if the settlements and ground losses are typical for the procedures employed, or if some lapse in normal practice occurred as the shield passed. If construction procedures are modified in an attempt to reduce or control particular sources of large losses, subsurface settlement observations can be used to see how successful the modified procedures are in reducing the losses.

The volume of ground surface settlement also should be determined at the location of deep settlement points so that the relationship between surface settlement volume and ground loss can be verified. To do this, the settlement of the ground surface should be measured at intervals along a cross-section at right angles to the tunnel at the location of a deep settlement point. Driven metal stakes or other types of surface points should be spaced along the cross-section at intervals of 5 to 10 ft (1.5 to 3 m), and the points should extend to a distance of at least 20 ft (6 m) beyond the estimated edge of the settlement trough. The surface points should be anchored securely below pavement or other structures that might bridge over or otherwise mask the true magnitude and pattern of settlement. The elevations of all the points, surface and subsurface, must be referenced to a stable benchmark, and high survey accuracy should be maintained. (See Cording, et al., 1975 for further discussion of soil tunnel settlement observation procedures.) The relationship between ground loss and surface settlement can then be determined by careful and frequent observations of the surface and subsurface settlements as the tunnel face passes the observation section.

APPENDIX A

CASE 3b - WASHINGTON, D.C. METRO SECTION F2a, F ROUTE TUNNELS

A.1 PROJECT DESCRIPTION

Section F2a (Contract 1F0021) of the Washington, D.C. Metro System is located in Southwest Washington along 7th Street, SW and Maine Avenue, SW. Figure A.1 shows the location of Section F2a in relation to other earth tunnel sections of the Metro System. Section F2a consists of four single-track earth tunnels with a total length of 8,815 ft (2687 m). Two of the tunnels serve the L'Enfant - Pentagon Route (L Route) and two serve the Branch Route (F Route). Figure A.2 shows the alignment of the tunnels and the locations of the settlement observation cross-sections. Table A.1 lists the lengths of the individual tunnel drives and their dates of construction. The settlement observations made on the L Route tunnels (section 9, 10, and 11) were reported in Cording, et al. (1976); the F Route tunnel observations on sections 33-38 and 33-38A are reported here.

The F2a tunnels are identical single-track, circular-section, shield-driven earth tunnels lined with a segmental steel lining with an outside diameter of 17 ft - 6 in. (5.33 m). Construction procedures and the shields used to drive the tunnels were nearly identical for all four tunnels. The L Route tunnels were much deeper, more widely separated and were driven in denser, stiffer soils than the F Route tunnels. The average depth to tunnel springline, z, for the L Route tunnels is 70 ft (21 m), while the average depth for the F Route tunnels is about 40 ft (12 m). The average distance between centerlines of the L Route tunnels is 70 ft (21 m), while the F Route tunnels are separated about 30 ft (9 m) centerline to centerline, or about one tunnel diameter edge to edge. The inbound L Route tunnel goes beneath both F Route tunnels. (See Fig. A.2). The vertical clearance between the crown of the inbound L tunnel and the invert of the F tunnels is 7 ft (2 m) at the inbound F tunnel and 13 ft (4 m) at the outbound F tunnel. All four tunnels were driven from south to north, in the direction of decreasing stationing.

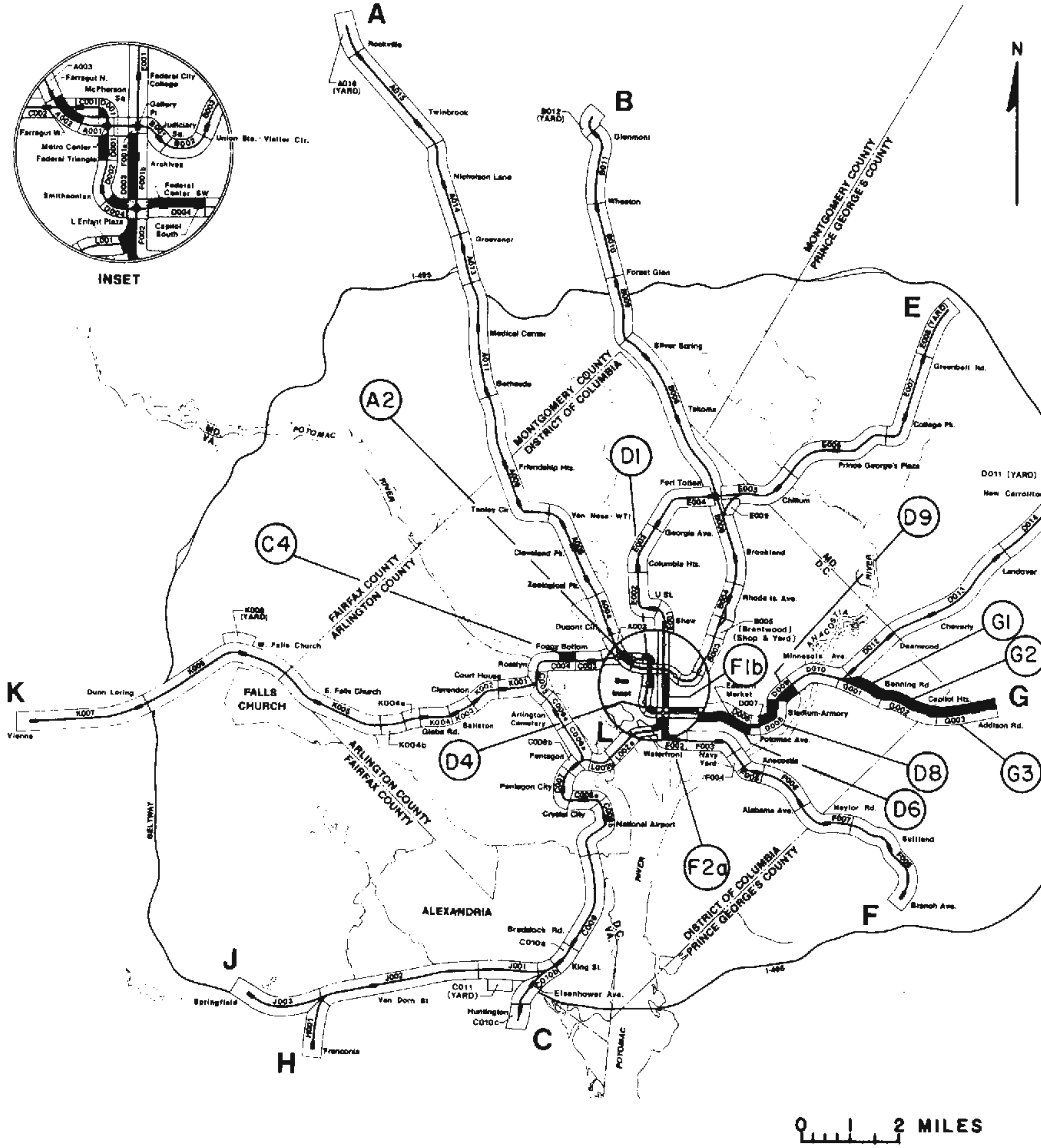


FIGURE A.1 LOCATION MAP OF WASHINGTON, D.C. METRO SYSTEM EARTH TUNNELS

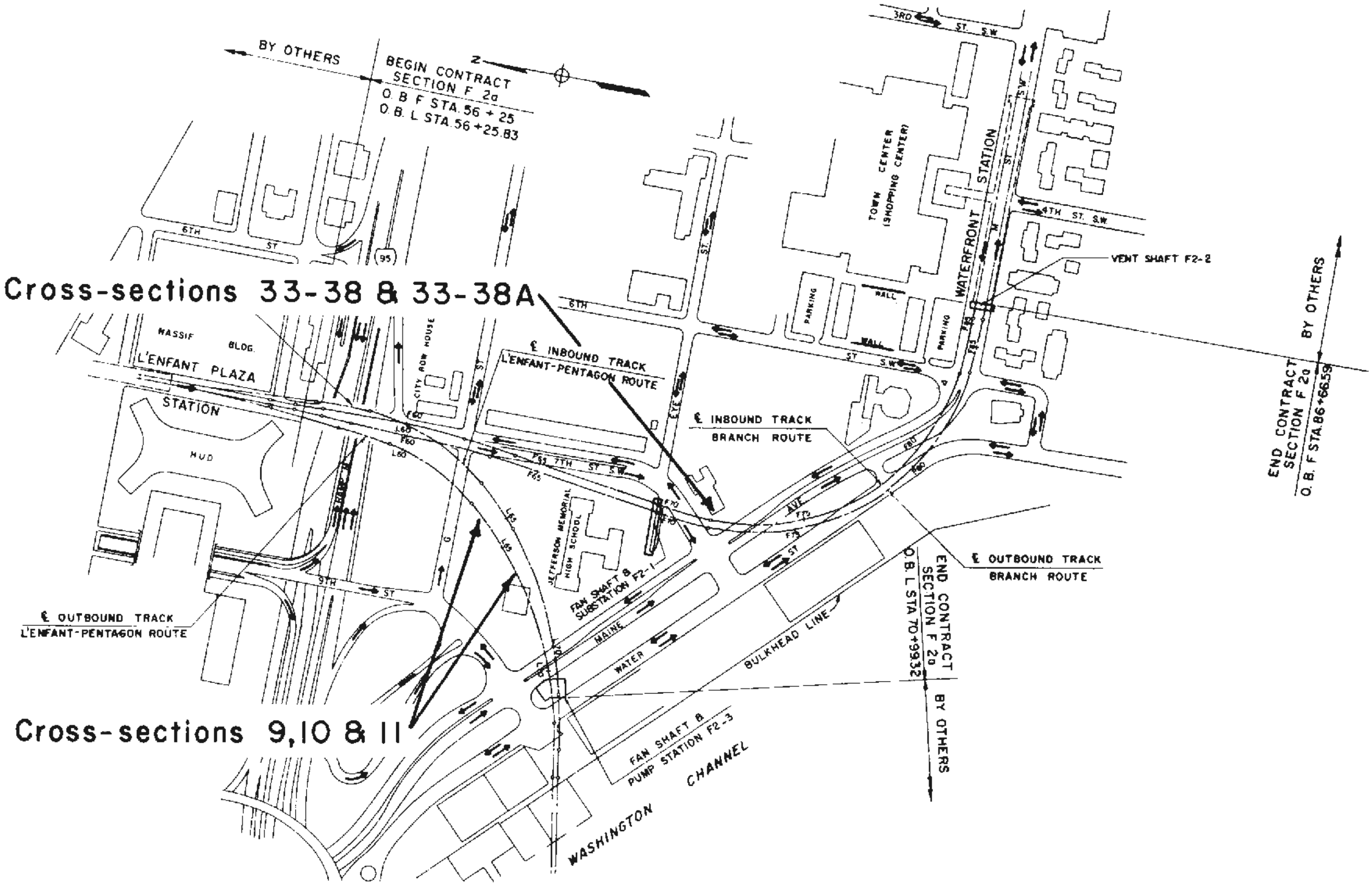


FIGURE A.2 SITE PLAN - METRO SECTION F2a

TABLE A.1

TUNNEL STATISTICS - SECTION F2a

	L Route		F Route	
	Inbound	Outbound	Inbound	Outbound
Length of tunnel drive	1514 ft (462 m)	1421 ft (433 m)	2916 ft (889 m)	2964 ft (903 m)
Depth from ground surface to tunnel springline, Z	52 to 73 ft (15.9 to 22.3 m)		37 to 48 ft (11.3 to 14.6 m)	
Centerline to centerline separation of tunnels	30 to 90 ft (9.1 to 27.4 m)		27 to 45 ft (8.2 to 13.7 m)	
Date tunneling started	30 May 1975	15 Sept. 1975	15 Jan. 1976	30 Mar. 1976
Date tunneling completed	20 Nov. 1975	22 Dec. 1975	18 May 1976	9 Aug. 1976

A.2 GEOLOGY AND SOIL PROPERTIES

The geologic profile of Metro Section F2a consists of a series of flat-lying, Pleistocene, river terrace deposits overlying Cretaceous sediments. The Pleistocene deposits are covered by 5 to 10 ft (1.5 to 3.0 m) of fill. Recent alluvial deposits are present locally in old stream channels. The F Route tunnels were driven in sand and gravel, silty sand, and silty clay entirely within the Pleistocene deposits. The L Route tunnels were driven at the interface between the lowermost Pleistocene sand and gravel layer and the upper Cretaceous silty sands and hard fissured clays. (See Cording, et al., 1976, for details of L Route soil conditions). Figure A.3 is a geologic profile along the F Route tunnels. Only the major deposits are indicated. The Pleistocene deposits are designated by the letter T; Cretaceous units are designated by the letter P. Table A.2 contains a brief description of the main soil units and their properties. Detailed descriptions of the soils and boring logs are presented in Mueser, Rutledge, Wentworth and Johnston (1970a, 1970b, and 1973.) The following discussions are based on the data in those reports and construction logs of tunnel face geology.

The Pleistocene terrace deposits typically consist of sequences of coarse, basal sand and gravel overlain by a silty sand that in turn is capped by a silty clay. The layering within these sequences is complex, and the deposits are interfingered and lenticular. Along the F Route, two of these sequences of units are present: a lower sequence of T5 sand and gravel, T4 silty sand, and T1 (lower) silty clay and an overlying sequence of T3 sand and gravel, T2 silty sand, and T1 (upper) silty clay.

The groundwater table was located in the T3 sands and gravels at about El. - 2 ft, 0 to 13 ft (0 to 4 m) above the tunnel crowns. The soils along the tunnel routes were dewatered in advance of tunnel construction by deep well pumping. The deep wells consisted of 30-in. (0.76 m) diameter drilled holes with 9-in. (0.23-m) diameter slotted well screens and a graded filter sand backpacking. Submersible electric pumps were used, and some wells were equipped with vacuum systems. The wells extended to below the tunnel inverts and were spaced at an average distance of 160 ft (49 m) apart on the L Route and 400 ft (122 m) apart along the F Route. Pumping rates ranged from 12 to 60 gal/min (0.05 to 0.23 m^3/min). A deep well was located on the south-

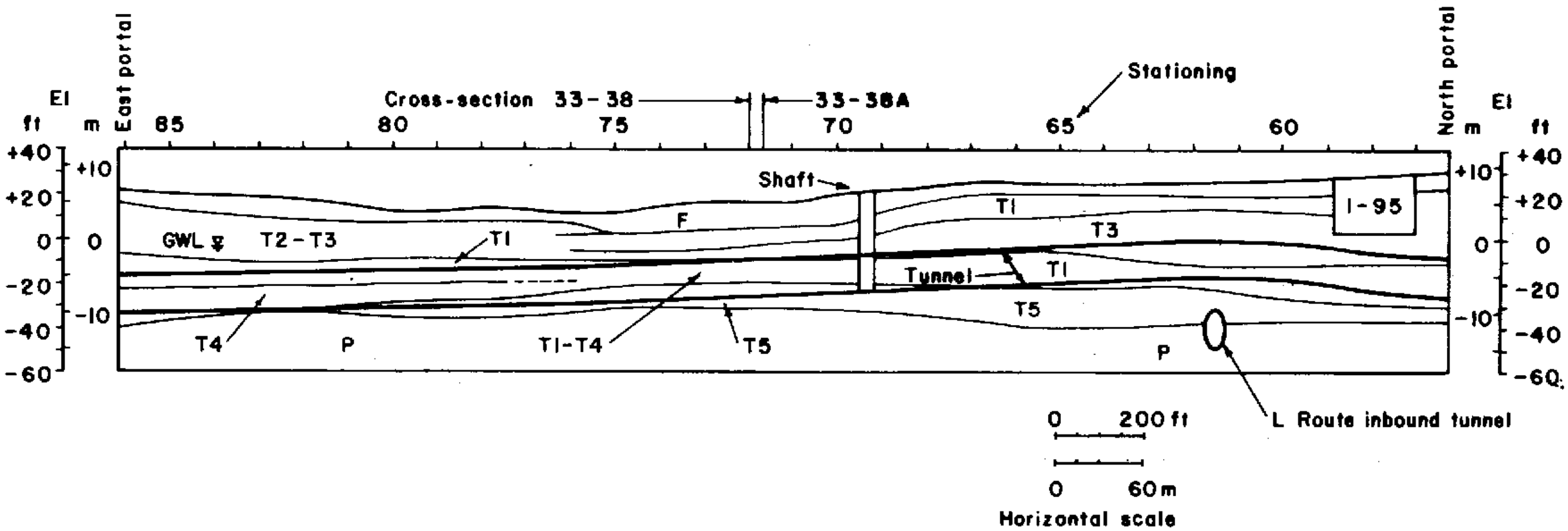


FIGURE A.3 GEOLOGIC PROFILE - METRO SECTION F2a, F ROUTE TUNNELS

TABLE A.2

SOIL PROPERTIES - F ROUTE TUNNELS

Soil Stratum	Description	Unified Soil Classification primary	Unified Soil Classification secondary	Standard Penetration Resistance, N	Natural water content, %	w_L, Liquid Limit, %	I_p, Plasticity Index, %
F	Fill, generally of inorganic soil obtained from nearby natural materials.	ML, SM	SC, CL	--	--	--	--
T1(E)	Stiff to medium light brown or gray or mottled brown-gray silty clay or clayey silt with lenses of brown silty fine sand.	CL, CH, OL	Lenses of SM or SC	10-30	28	45	24
T2	Medium compact to compact brown and orange-brown silty or clayey fine to medium sand with traces of small gravel.	SM, SP	SC, SW	15-40			
T3	Compact to very compact brown and red-brown fine to coarse sand with some silt and gravel, or sand and gravel with a trace of silt and numerous boulders.	SM, SP	GP, GM	40-60 (100 + in gravels)	--	--	--
T4	Medium compact to compact gray and gray-brown fine to medium sand with some silt and small gravel containing lenses of dark gray clay.	SM, SP	SC, SW	20-40	--	--	--
T5	Compact to very compact gray and gray-brown fine to coarse sand with some gravel and some to trace silt, or sand and gravel with numerous boulders.	SM, SP	GP, GM	30-100 +	--	--	--
P1	Hard mottled red-brown and gray or light gray and tan plastic clay with occasional pockets of fine sand.	CH	CL	40-50	22	60	37
P2	Compact to very compact light gray or tan silty or clayey fine to medium sand with pockets of silty clay and trace of small gravel, occasional lignite fragments.	SC, SM	SP	50-100	--	--	--
P3	Hard gray-green or gray-blue silty or sandy clay and sandy silt or silty or clayey fine sand with occasional small gravel.	CL, SC	CH, SM	50-100	--	--	--

Mueser, Rutledge, Wentworth and Johnston (1970 a) east corner of Maine Avenue and 7th Street, opposite the F Route settlement cross-sections.

The dewatering along the F Route was generally effective in reducing and controlling groundwater inflows into the tunnel headings. Slight seepage flows of 1 to 5 gal/min (0.004 to 0.02 m^3/min) were encountered at contacts between sands and gravels and clays or silts, and occasional heavier inflows were encountered in sandier lenses. The following sections describe the various soil strata encountered in the tunnel faces.

STRATUM T1 - Stiff to medium, gray, silty clay or clayey silt. T1 clay is present in an upper layer between El. + 20 to + 10 ft from the northern end of the project to El. 0 to - 5 ft near station 68 + 00. (See Fig. A.3) The upper layer finally disappears near station 78 + 00. The lower T1 layer is located between El. -10 to -30 ft at the north end of the project and gradually rises to between El. -5 to -20 ft near station 66 + 00. Between stations 66 + 00$\pm$ to 75 + 00 $\pm$, the clay is intermingled with T4 silty sand in a series of 1 to 3- ft (0.3 to 1-m) thick alternating layers of silty clay and silty sand. Beyond station 75 + 00, the T1 clay again appears in a distinct layer between El. -10 to -15ft. The lower T1 clay was present in the tunnel face over the entire F Route tunnel length, usually in the upper half to third of the face.

Torvane measurements of shear strength on medium plasticity lower T1 clay in the tunnel face gave values of 0.6 to 0.9 tsf (57.5 to 88.2 kPa) Pocket penetrometer measurements on clay in the tunnel face near the settlement cross-sections (station 72 + 00) gave unconfined compression values of 1.5 to 4 tsf (143.6 to 383.0 kPa). The clay contains organic material in the form of chunks of wood and roots and has occasional pockets of sand. Seepage of groundwater was negligible, except at sandier lenses.

At the settlement cross-section monitored by the University, (Sections 33-38, Fig. A.2) T1 clay in 1 to 2-ft (0.3 to 0.6-m) thick bands alternated with layers of silty sand (T4) in the upper half of the tunnel face. Figure A.4 shows the tunnel face geology at the cross-sections. Slight seepage flows of 1 to 2 gpm (.004 to .008 m^3/m) occured on top of the clay layers. The sand tended to wash and ravel out of the face and undermine the clay layers. The clay layers would then break off in chunks 2 to 2-1/2 ft

TABLE A.3

TUNNELMAN'S GROUND CLASSIFICATION

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow raveling - - - - Fast raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive - running - - - - Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (± 30°-35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of mont morillonite.

HEUER (1974)

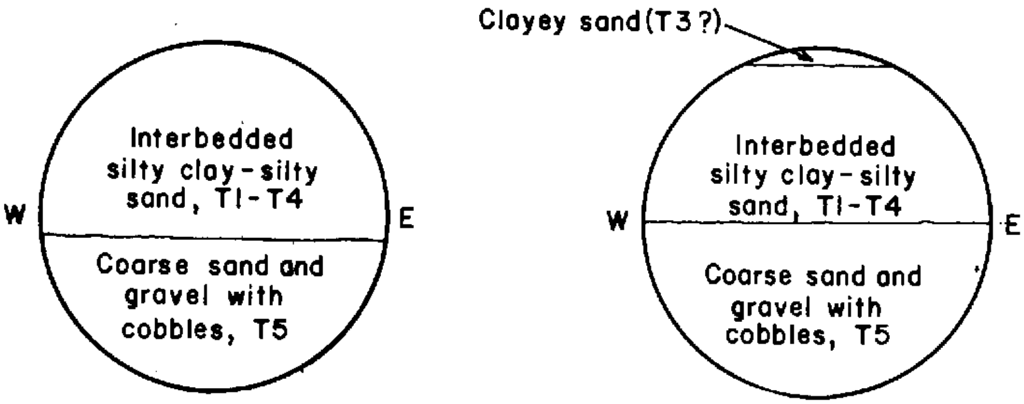
thick, 2 ft wide and 5 to 6 ft long (0.7 m X 0.7 m X 2 m). Breasting was partial to full (all doors closed) between shoves and partial during the shove. The face in this layered material would be slow to fast raveling depending on the amount of seepage and undermining.

From station 72 + 10 IB and station 70 + 50 OB to the middle shaft, an orange-tan, clayey sand to sandy clay appeared intermittently in the very crown of the tunnel. Groundwater seepage from this material was concentrated in pockets. This material may have been either a weathered T1-T4 surface or the base of the overlying T3 sand and gravel. In either case, it indicated that the contact between the T1-T4 silty clay - silty sand and the T3 sand and gravel was very close to the tunnel crown in the settlement cross-section area.

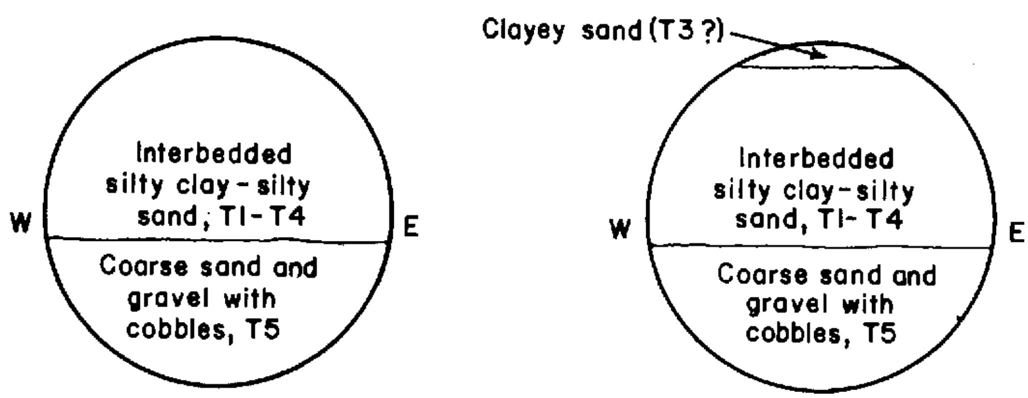
STRATUM T4 - Compact to very compact, grey, micaceous, silty sand or light tan fine sand with traces of organic matter and some lenses of dark grey clay. T4 silty sand is present between El. -20 to - 30 ft from the start of the tunnels at station 86 + 00 to station 75 + 00 where it becomes interlayered with T1 clay. The T4 silty sands are absent except for occasional lenses north of the middle shaft.

Face conditions in the T4 silty sands would be classified as slow to fast raveling. Occasional lenses of more pervious material produced locally heavy seepage and some minor running and flowing conditions. This situtation was usually controlled with the breasting doors. The material would occasionally ravel out to the edge of shield at the crown behind the breasting doors.

At the settlement cross-sections, the T4 silty sands were interlayered with the T1 silty clay. A 1 to 2-ft (0.3 to 0.6-m) thick layer of laminated silty sand-sand and gravel was usually present at the T4 - T5 sand and gravel contact at mid-height in the face. A 2 to 3-ft (0.6 to 1-m) thick layer of T1 silty clay lay on top of this, and then silty sand - silty clay layers alternated to the crown. The sand near the crown appeared to be less silty. The silty sand tended to ravel out from beneath the clay, undermining the clay and eventually allowing large chunks of clay to break off. Concentrated seepage along the top of some of the clay layers accelerated the process. The breasting doors were used to control the face when it was sandy and raveling or washing occurred. The sand layer at the crown tended to ravel out to a vertical face at the edge of the shield behind the breasting doors.



Section 33-38



Section 33-38A

Outbound | Inbound

FIGURE A.4 TUNNEL FACE GEOLOGY AT F ROUTE SETTLEMENT CROSS-SECTIONS

STRATUM T5 - Compact to very compact, rust brown, coarse sand and gravel with cobbles and occasional boulders. The T5 sand and gravel is present in the lower half of the tunnel or just below the invert for the middle one-third to one-half of the tunnel lengths. On the L Route, the T5 sand and gravel was present in the upper half of the tunnel face and presented a serious problem with running ground that forced the contractor to install a breasting shelf in the shields to help control the face. In the F Route tunnels, however, the T5 sand and gravel presented few problems even though at the settlement cross-section the T5 was present up to a point just below the springline. (See Fig. A.4.) The large muck pan in the shield allowed the muck to be piled in the shield invert to breast the T5. The T5 sand and gravel would be classified as fast raveling, but in the absence of heavy seepage flows and because it was present only in the lower, less critical part of the face, the T5 sand and gravel was not a problem.

STRATUM T3 - Compact to very compact, rust brown, fine to coarse sand and gravel with cobbles and occasional boulders. The T3 sand and gravel is very similar to the T5 except that it usually is more heavily iron stained. The T3 sand and gravel was just above the tunnel crown from about station 74 + 00 to station 65 + 00, where it gradually moved down into the tunnel face to occupy the upper half of the face. The T3 remained in the face at or above springline for the remainder of the tunnel lengths. When present in the upper portion of the tunnel face, the T3 sand and gravel, like its counterpart, the T5, is a running material that can be very difficult to control. Concentrated seepage along an underlying impervious boundary, such as the T1 clay, and local pockets of water saturated sand can result in flowing as well as running conditions in the face.

The upper half of the inbound heading of the F Route was in T3 sand and gravel as it crossed over the previously constructed L Route inbound tunnel between stations 60 + 66 IB-F to 58 + 95 IB-F. The crown of the F Route tunnel was 28 ft (8.5 m) below the street at this point, and the clearance between the invert of the F Route Tunnel and the crown of the L Route tunnel was only 7 ft (2 m). When the L Route tunnel was driven through this area earlier, it encountered heavy groundwater inflows at the T5-P1 clay interface. The L Route heading was stopped temporarily at station 60 + 85 while an additional dewatering well was installed at station 60 + 08 in the middle of 7th street. When mining resumed, the L Route heading still encountered continuing problems of running and flowing conditions that were difficult to control with the breast doors, breasting shelf, and hand breasting below the shelf.

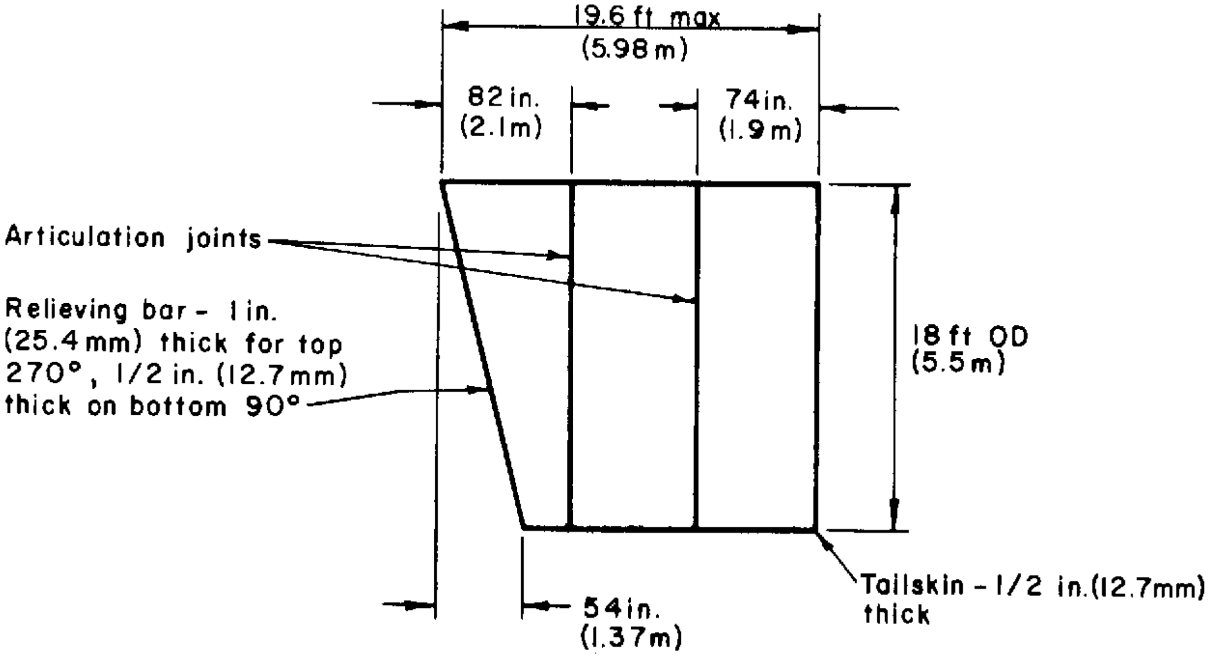
As the F Route inbound heading advanced beyond station 60 + 35 IB-F, and over the L Route tunnel, it encountered severe face control problems. T3 sand and gravel was present from the springline upwards, while T1 silty clay was present below the springline. Heavy seepage inflows were present at the T1 - T3 interface and about 4 to 5 ft (1.2 to 1.5 m) below the crown where there was a contact between a silty T3 lense under a sandier T3 layer. Water also was seeping in around the tail of the shield. Several runs were noted in the sands in the arch. The heading inspector noted that about 5 extra cars of muck were removed over the course of the ten shoves made on the shift. This is about 24 to 30 yd^3 (18 to 23 m^3) of loss or roughly one ring length (2.5 ft, 0.8 m) of excavation. At the start of the next shift, a void, 4 ft wide by 3 ft high (1.2 m by 1 m) and of unknown length, was discovered over the cutting edge of the shield at station 60 + 08. It was filled with 2 yd^3 (1.5 m^3) of fly ash grout.

In the next 2 days the face was advanced to station 60 + 00. Despite shoving with the breasting doors closed and the face full of muck, runs and loss of ground continued. An additional 10 to 12 yd^3 (8 to 9 m^3) of fly ash and neat cement grout was placed through the face and holes cut through the shield. The heading was shut down at station 60 + 00 after a void appeared under the middle of 7th Street at station 60 + 02 over the west edge of the tunnel. The void was filled from the surface with 4-1/2 to 5 yd^3 (3.4 to 3.8 m^3) of pea gravel. A second void was discovered 2 days later over the east edge of the tunnel. It was filled with an additional 5 yd^3 (3.8 m^3) of pea gravel. The two voids were connected and were limited to an old utility trench that ran across 7th Street at a right angle to the tunnel. Over the following month, a visible settlement trough developed in 7th Street over the F Route tunnel between station 59 + 95 and station 60 + 40 $\pm$.

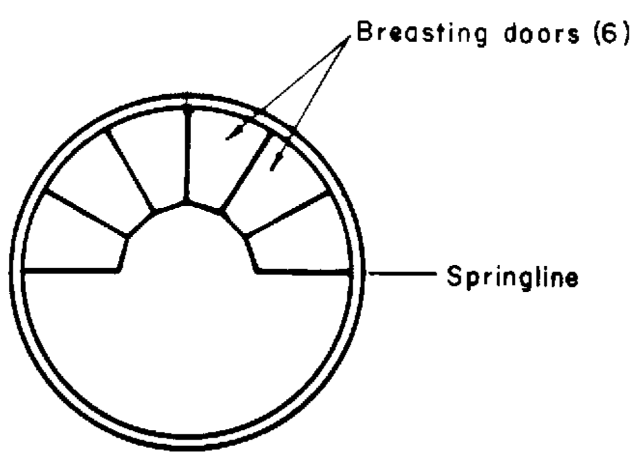
The contractor used chemical grouting to stabilize the T3 sand and gravel so that mining could resume. The heading was stopped about 100 ft (30 m) south of the existing chemically grouted zone that was placed to protect the 7th Street bridge over I 95. (See Cording, et al., 1976, for descriptions of this grouting.) Two holes were drilled from the surface about 6 ft (2 m) ahead of the face, and then a single row of holes, spaced 8 ft (2.4 m) apart along the tunnel centerline, was drilled up to the previously grouted zone. The holes were drilled through the T3 sand and gravel into the T1 clay. A total of 11,900 gals (45 m^3) of sodium silicate grout was pumped into 13 holes. The following day mining was resumed in the inbound tunnel. The face was now firm and stable, and could be excavated nearly vertically and controlled by closing the breasting doors. Seepage was greatly reduced. The heading continued without further problems.

A.3 CONSTRUCTION DETAILS

The four F2a tunnels were excavated with nearly identical Robbins articulated shields equipped with hydraulic breasting doors and a hydraulic digger arm. The shields have an outside diameter of 18 ft (5.5 m), an overall length of 19 ft 7-1/2 in. (5.98 m) at the crown, and a hood 4.5 ft (1.37m) long. (See Fig. A.5.) The shields are constructed in 3 segments with articulation joints between the segments. The articulation joints can provide up to 3 in. (76 mm) of longitudinal movement between each segment. Thus, the shield can be curved for easier steering through horizontal or vertical curves. The front articulation section is connected to the middle section by 20 hydraulic articulation jacks. The opposite ends of these jack cylinders contain the thrust jacks. The articulation jacks are used to control the attitude of the front section relative to the middle section. When increasing grade, for example, the top jacks would be retracted while the bottom ones are extended.



Side view



Front view

FIGURE A.5 F ROUTE TUNNEL SHIELD DIMENSIONS

The tail section is attached to the middle section by 20 hydraulic cylinders. The attitude of the tail section relative to the middle section is not controlled, but is allowed to follow freely behind the rest of the shield.

The major difference between the F Route and L Route tunnel shields was the size of the relieving bar or overcutter on the lead edge of the shield. The F Route shields used a 1-in. (25.4-mm) thick steel relieving bar over the top 270^{0} of the shield, and a 1/2-in. (12.7-mm) thick bar for the bottom 90^{0}. The L Route shields used a 3/8-in. (9.5-mm) bar over the full circumference of the lead edge.

The face of the shields was equipped with 6 hydraulically operated breasting doors (flaps) that extend around the upper half of the face from springline to springline. The breasting doors were hinged to the shield circumference. The lower half of the face could be supported if necessary by piling muck against the face and by using the digger arm. The use of the breasting doors depended upon the face conditions and the judgement of the shield operator. The shields were usually shoved with the doors open or partially open. At the end of the shove, all or at least the center two doors were closed to support the face. On the L Route, runs in the T5 sand and gravel when it was present in the upper half of the face could not be controlled with the breasting doors, and the springline doors were removed and replaced with a 4-ft (1.2 m) wide shelf extending across the full face width. Although the F Route tunnels encountered problems in the T3 sand and gravel, breasting shelves were not used.

The tunnel linings for the F Route were the same 17 ft-6 in. (5.33 m) outside diameter, bolted, segmental steel linings as used for the L Route. The lining rings consisted of 6 segments plus a key, and the rings were 30 in. (0.76 m) wide. Inside diameter of the lining was 16 ft - 6 in. (5.03 m).

The lining rings were erected in the tail of the shield. The annular void between the outside of the lining and the inside of the shield tailskin was then filled with a sand-fly ash-cement grout before the next shove and before the ring emerged from the tail of the shield. This procedure leaves only the 1/2-in. (12.7 mm) thickness of the shield tailskin ungrouted and greatly reduces the amount of lost ground at the tail of the shield. A grout seal, consisting of a plastic foam packer, was inserted into the gap around the lead edge of the ring and held in place during the grouting by the shield jack ring. Grout was injected at low pressures through one or two grout holes in the crown of the lining.

The grouting procedure appeared to be effective. Inspections of the grout fill in the vicinity of the settlement cross-sections generally found the void completely filled. In instances where the grouting was incomplete or grout was lost when the jack ring was retracted, the ring was usually regrouted one or two shoves later. If the crown of the tunnel was in sand, this regrouting may not have been quick enough to avoid loss of ground into the tail void. In the clayey soils, the voids usually stayed open for some time. For example, a void on the right upper quarter of Ring 570 was still open when Ring 572 was erected. The grouting of Ring 572, however, forced grout into and filled the void on Ring 570.

A second stage grouting with a sand-cement grout was done on a third shift. Second stage grout takes were usually small.

Figure A.6 is a mining progress chart for the F Route tunnels. Two 8-hr shifts were worked, although the second shift often worked 10 to 12 hrs. The inbound tunnel started on 15 January 1976 and holed thru on 18 May 1976. Average progress was 39 ft (11.9 M) per day or about 16 rings. The best day's progress was 77.5 ft (23.6 m) for two 8-hr shifts. The outbound heading was started on 30 March 1976 and completed on August 1976. Average progress was 41 ft (12.5 m) per day. Best day's progress was 105 ft (32.0 m).

A.4 OBSERVATION AND MONITORING PROVISIONS

The contractor was required by Metro specifications to monitor ground surface settlements at 20-ft (6.1 m) intervals along the tunnel centerlines. The contractor was also required to install and monitor deep settlement indicators. These consisted of a 1-in. (25 mm) diameter steel pipe inside of a 2-in. (51-mm) diameter steel casing. The casing is set in a bore-hole, and the inner pipe is driven about 3 ft (11 m) below the bottom of the casing to a point about 4 to 6 ft (1.2 to 2 m) above the crown of the tunnel. The settlement of the inner pipe was monitored as an indication of the settlement occurring immediately above the tunnel. Deep settlement indicators were installed

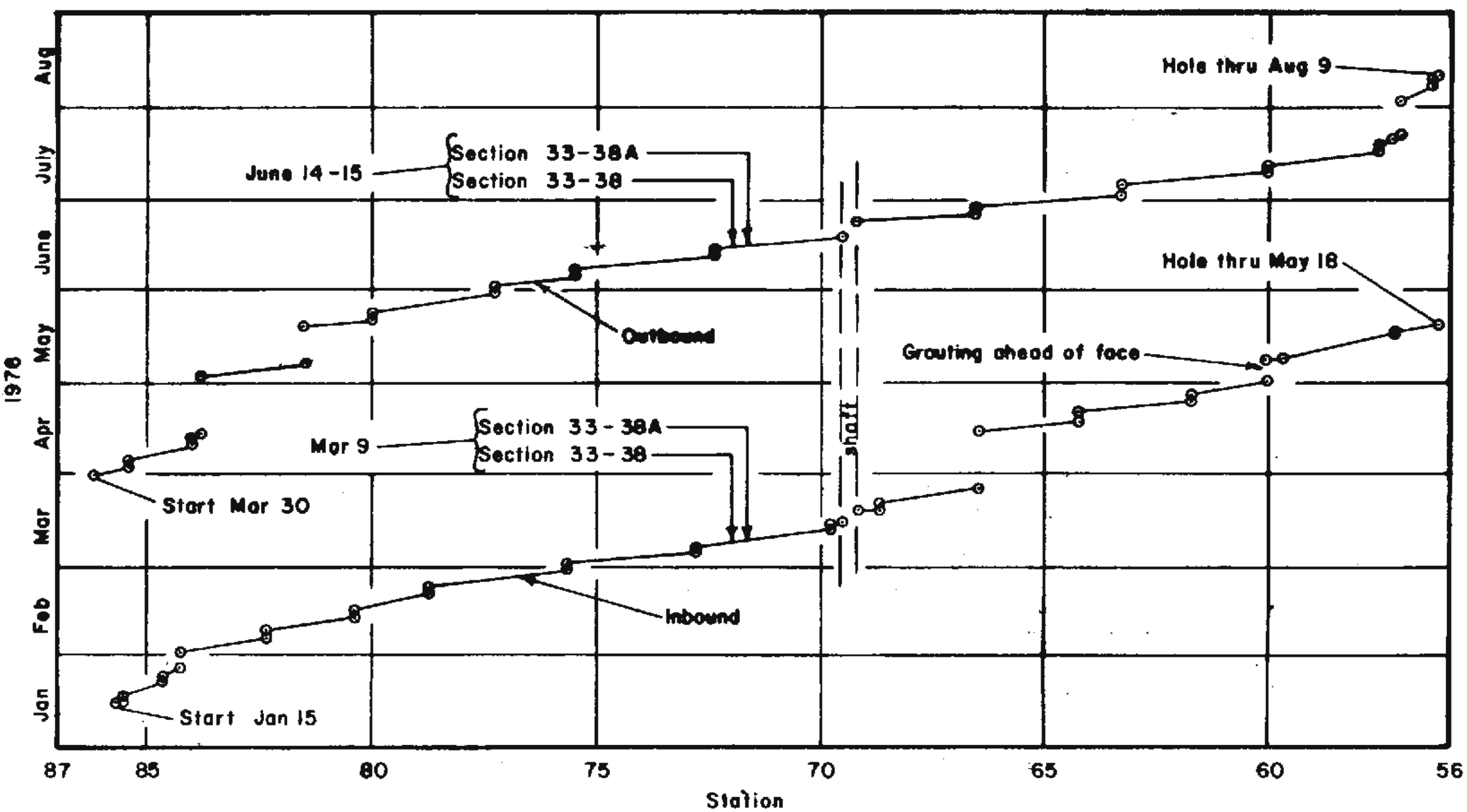


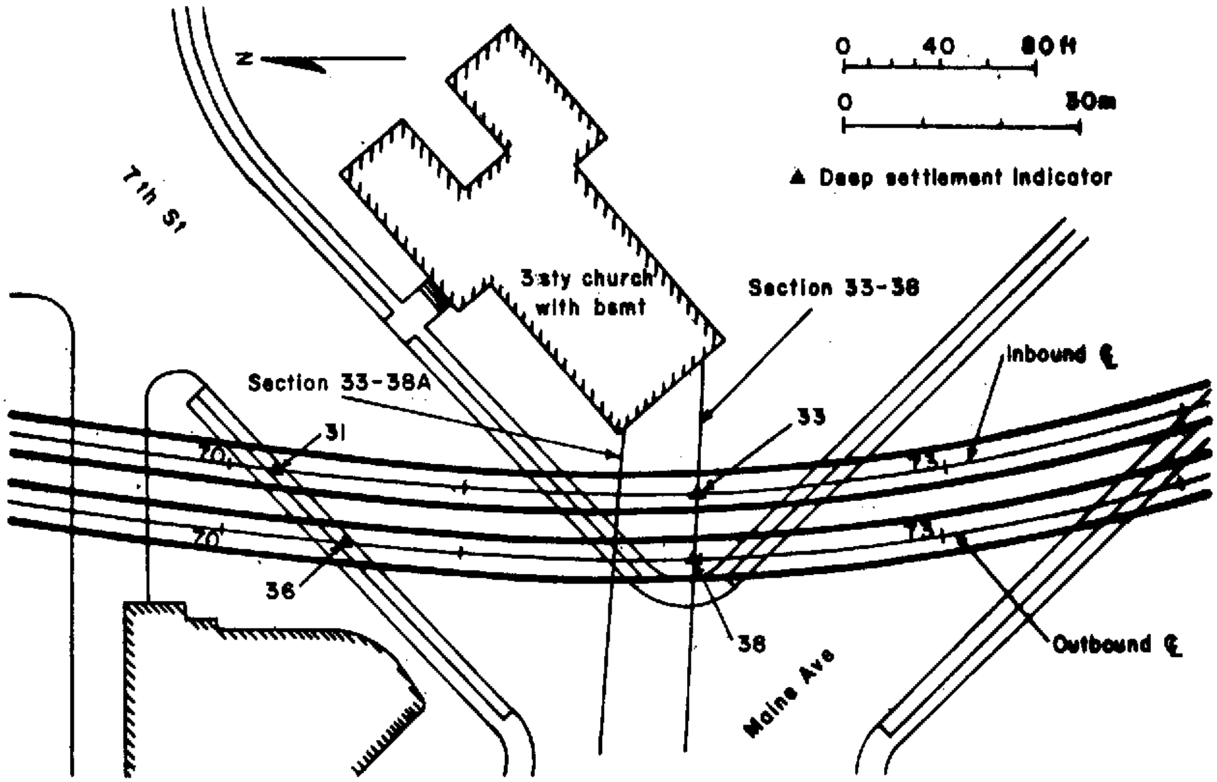
FIGURE A.6 MINING PROGRESS CHART - F ROUTE TUNNELS on the centerline of both tunnels at 100 to 200-ft (30 to 61-m) intervals. The elevations of the deep settlement indicators and ground surface settlement points were monitored daily by the contractor during tunnel construction.

On the northeast corner of the intersection of 7th Street and Maine Avenue, two deep settlement indicators, No. 33 on the inbound tunnel and No. 38 on the outbound tunnel, were located opposite each other at station 71 + 97 IB-F. At this point the tunnels were at minimum separation, 28 ft (8.5 m) centerline to centerline, and at a depth of 35 ft (10.7 m) from ground surface to the tunnel axis (springline). The tunnels are on a minimum radius (800 ft or 244 m) curve and on a vertical curve with an average grade of +0.9 percent in the direction of the tunnel drives (south to north).

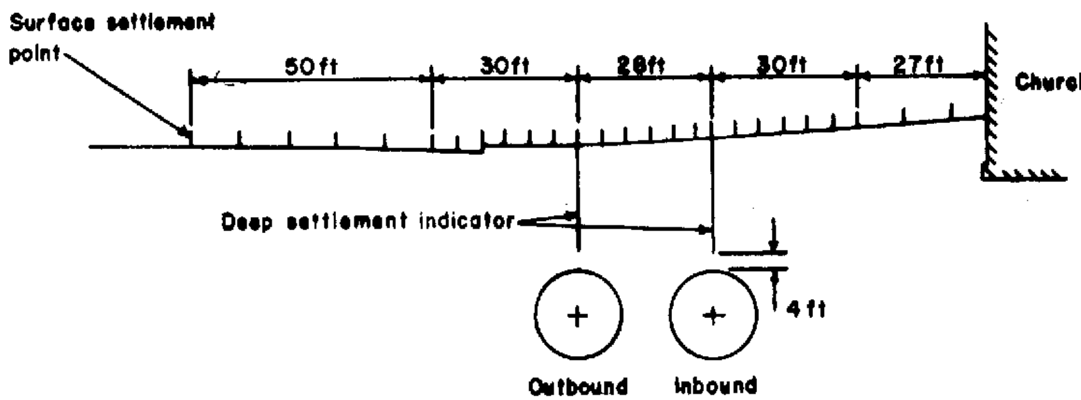
The University of Illinois installed a series of surface settlement points along a cross-section (No. 33-38) perpendicular to the tunnel centerlines and centered on the deep settlement indicators. Figure A.7 shows the cross-section. A similar cross-section (No. 33-38 A) was placed at station 71 + 61 IB-F. This is the point of closest approach of the inbound tunnel to the church shown on the plan. The horizontal distance from the west corner of the church to the east edge of the tunnel was 19.5 ft (5.9 m). The distance from the church to the east edge of the tunnel at cross-section 33-38 was 48 ft (14.6 m). Table A.4 lists the stations of the cross-sections and deep settlement indicators and other dimensions.

The eastern portion of the cross-section is on a lawn that slopes down from the church. Settlement points in the lawn consisted of 3-ft (1-m) lengths of number 8 reinforcing bar driven flush with the ground. For the western portions of the cross-sections, which lie on sidewalks or street pavements, the settlement points were marked by cross-cuts in the sidewalk or curbs and masonry nails in the asphalt street surface.

In addition to the cross-section points, settlement points were located at 5-ft (1.5-m) intervals along the tunnel centerlines from stations 72 + 20 to 71 + 61. Leveling points were also established on the church. All leveling was done with a Zeiss Ni-2 automatic level. Elevations were read to the nearest 0.001 ft (0.3 mm), and three-wire leveling procedures were used to tie the cross-sections to benchmarks several blocks away.



a. Plan



b. Section 33 - 38

FIGURE A.7 DETAILS OF F ROUTE SETTLEMENT CROSS-SECTIONS

TABLE A.4

LOCATIONS OF OBSERVED CROSS-SECTIONS
AND DEEP SETTLEMENT INDICATORS

Cross-section	33 - 38		33 - 38A
Deep settlement indicator	33	38	None
Centerline station			
inbound	71 + 97		71 + 61
outbound		71 + 98	71 + 65
Distance between tunnel centerlines		28 ft (8.5 m)	
Depth to tunnel springline, z			
inbound	36 ft (11 m)		38 ft (11.6 m)
outbound		35 ft (10.7 m)	34 ft (10.4 m)
Distance above tunnel crown to tip of deep settlement indicator, y	4 ft (1.2 m)	4 ft (1.2 m)	

A.5 OBSERVATIONS AND INTERPRETATIONS

A.5.1 LOSS OF GROUND

The ground lost around the tunnels during construction is reflected in the settlements of the deep indicators located over the tunnel crowns. Figures A.8 and A.9 show the correlation between deep settlements and the position of the tunnel shields, at deep indicators 33 and 38, respectively. The settlement of the ground surface at the deep indicator locations is also included on the plots. Very little deep settlement occurred ahead of the tunnel face. The majority of the deep settlement occurred as the shields passed under the deep indicators, with slightly less settlement occurring after the tails of the shields had passed. The amount of deep settlement and the associated volume of ground loss are summarized in Table A.5. The volume of lost ground was estimated from the deep settlements using Eq. 2.1

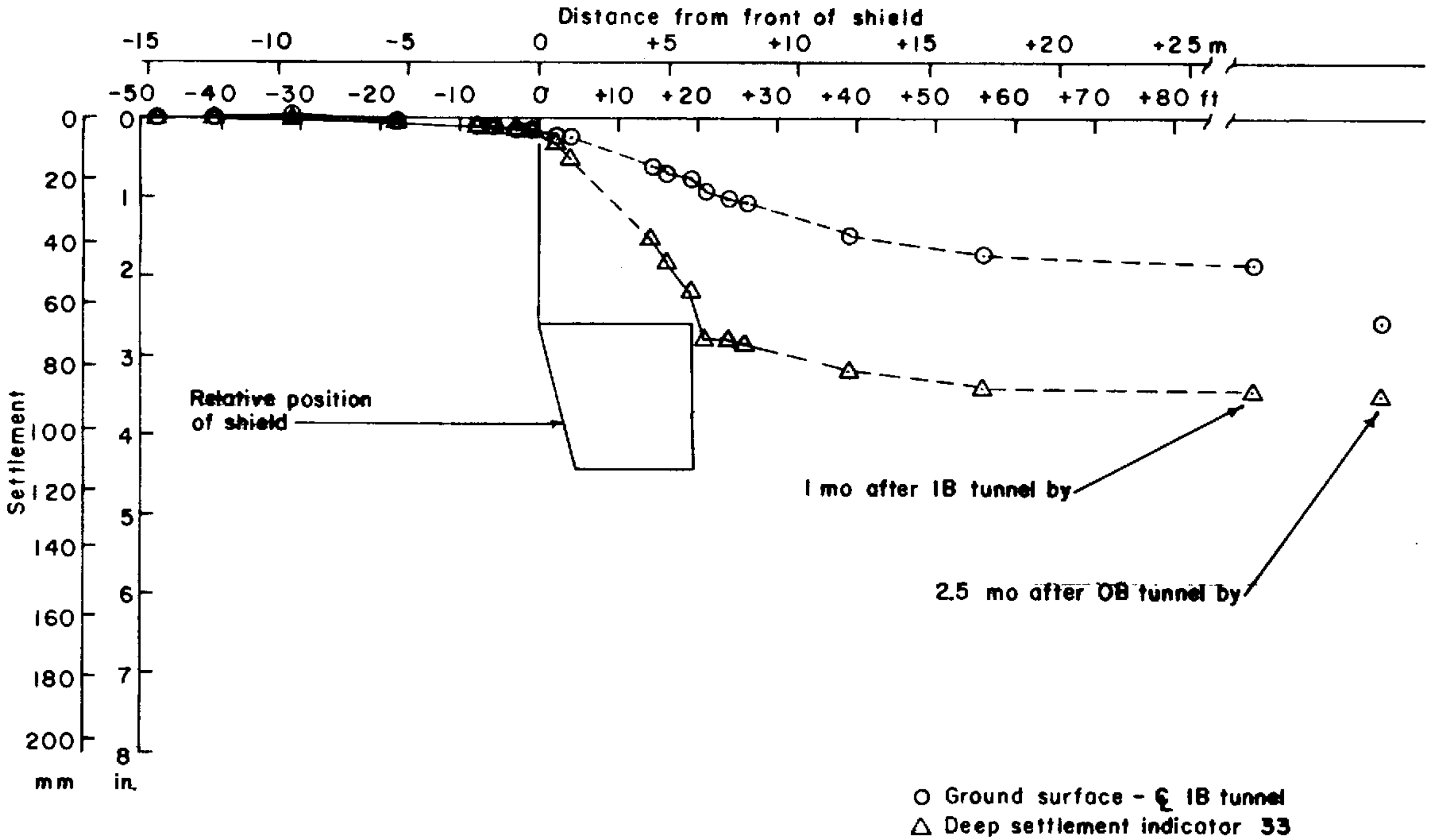


FIGURE A.8 GROUND SURFACE AND DEEP SETTLEMENTS, DEEP SETTLEMENT INDICATOR 33, FIRST (INBOUND) TUNNEL

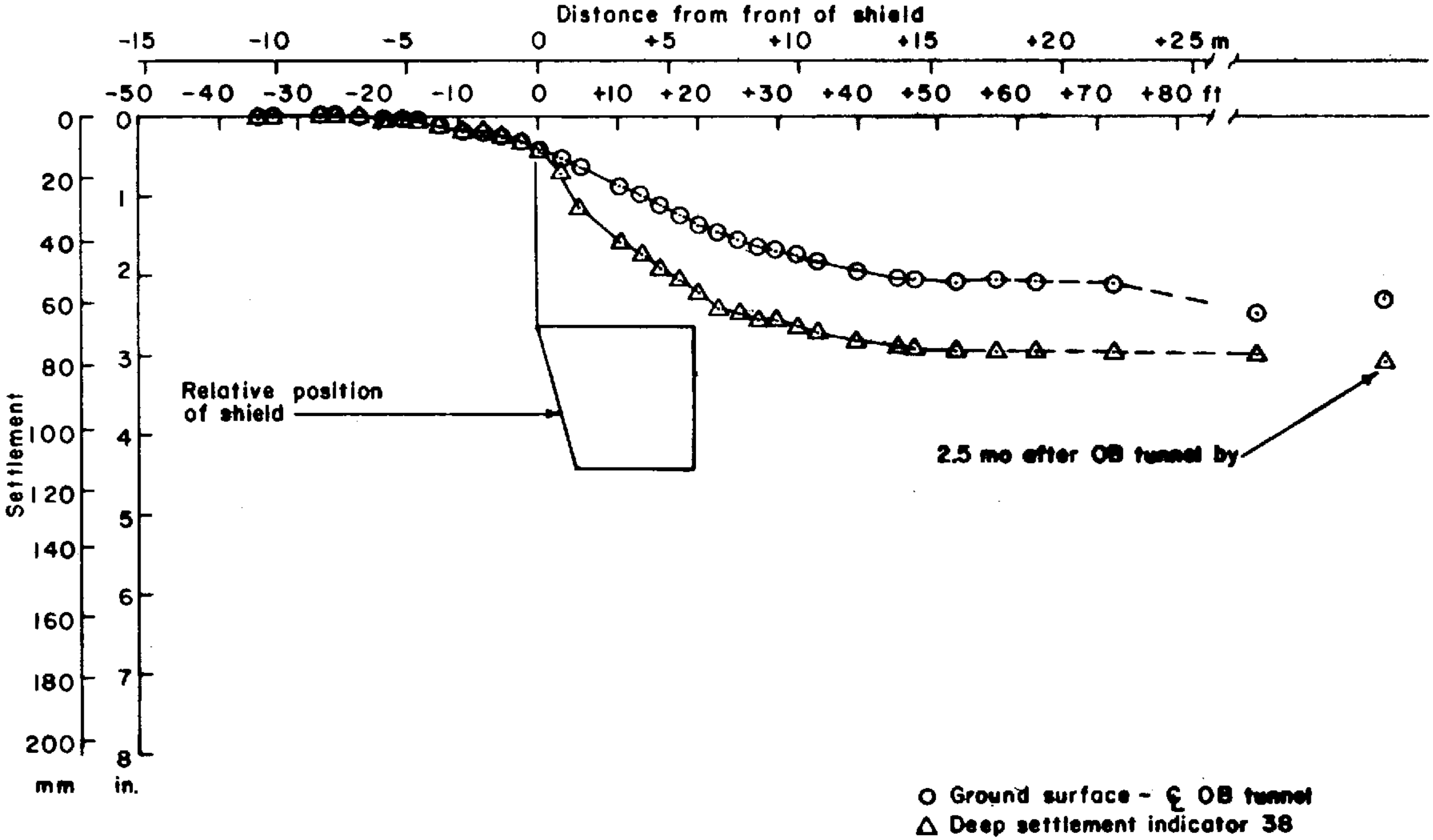


FIGURE A.9 GROUND SURFACE AND DEEP SETTLEMENTS, DEEP SETTLEMENT INDICATOR 38, SECOND (OUTBOUND) TUNNEL

TABLE A.5

LOST GROUND AROUND F ROUTE TUNNELS

Case 1	z/2R 2	Lost ground				
		Before face 3	Over shield 4	At tail 5	Lining deflection and time dependent movements 6	Total 7
3b F Route, first (Inbound) tunnel at cross-section 33 - 38	2.0	δ_v = 0.2 in. V_L = 0.4 ft^3/ft $\%V_L$ = 0.1%	2.0 in. 4.3 ft^3/ft 1.7%	0.7 in. 1.5 ft^3/ft 0.6%	0.4 in. 0.8 ft^3/ft 0.3%	3.2 in. 7.0 ft^3/ft 2.7%
Second (Outbound) tunnel at cross-section 33 - 38		δ_v = 0.4 in. V_L = 0.9 ft^3/ft $\%V_L$ = 0.4%	1.7 in. 3.7 ft^3/ft 1.5%	0.5 in. 1.0 ft^3/ft 0.4%	0.5 in. 1.1 ft^3/ft 0.4%	3.1 in. 6.7 ft^3/ft 2.7%
F Route, range of values along both tunnels	1.9 - 2.6	—	—	—	—	δ_v = 1.1 - 5.1 in. V_L = 2.6 - 12.0 ft^3/ft $\%V_L$ = 1.0 - 4.7%
3a L Route, values at cross-sections	3.7 - 4.1	δ_v = 0.1 - 0.3 in. V_L = 0.2 - 0.7 ft^3/ft $\%V_L$ = 0.1 - 0.3%	0.2 - 1.2 in. 0.5 - 3.0 ft^3/ft 0.2 - 1.2%	0.1 - 0.2 in. 0.3 - 0.5 ft^3/ft 0.1 - 0.2%	0.1 - 0.4 in. 0.2 - 1.0 ft^3/ft 0.1 - 0.4%	0.7 - 2.1 in. 1.8 - 5.1 ft^3/ft 0.6 - 2.0%
L Route, range of values along both tunnels	3.0 - 4.1	—	—	—	—	δ_v = 0.7 - 4.1 in. V_L = 1.8 - 8.8 ft^3/ft $\%V_L$ = 0.6 - 3.4%

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft^3/ft = 0.0929 m^3/m

The volume of lost ground observed at deep indicators 33 and 38 was 2.7 percent of the total tunnel volume for both tunnels. The ground losses observed at the other deep indicators along the F Route varied from 1.0 to 4.7 percent. The average ground loss over the inbound tunnel was 1.8 percent; the average loss over the outbound tunnel was 2.1 percent. The ground loss varies erratically, as it is strongly influenced by minor variations in soil conditions and construction operations. Typicals values of lost ground for the L Route tunnels are also included in Table A.6. The L Route losses were of the same magnitude as the F Route losses. The average L Route ground loss for both tunnels was 1.4 percent versus 2.1 percent for both F Route tunnels. Since both routes were in similar soil conditions and used identical construction techniques, the volumes of lost ground should be similar.

The detailed observations at deep indicators 33 and 38 indicate that the majority of the ground loss occurred as the shields passed under the indicators. This ground loss was caused by the overcutter bars on the noses of the shields and by plowing and yawing of the shields. Another major loss of ground occurs behind the tails of the shields due to incomplete grouting of the tail void and deflection of the tunnel lining.

The ground lost ahead of the shield through raveling or squeezing of the soil into the tunnel face was very small, 0.1 to 0.4 percent. The tunnel faces at the test section were in stiff, silty clay and clayey sands (see Fig. A.4) that were slow raveling and easily controlled by the shield breasting doors. The stability ratio for the tunnel faces was:

$$\frac{\gamma z}{s_u} = \frac{130 \text{ pcf } 36 \text{ ft}}{1800 \text{ psf}} = 2.6 \qquad \text{Eq. A.1}$$

where γ = unit weight of soil

z = depth to tunnel axis

s_u = undrained shear strength of soil.

This value was well below the values of 5 to 6 at which large ground losses may occur due to squeezing in of the tunnel face.

The ground loss after the tail of the shield passes under the deep indicator can be divided into two parts: an immediate loss occuring from the time the tail passes to the time when the tail of the shield is about 3 shove lengths, 7.5 ft (2.3 m), beyond the indicator, and a long term loss. The immediate loss is assumed to be caused by incomplete grouting of the tail void. The long term losses are assumed to be caused by deflection of the tunnel lining under gradual application of the soil load, compression or consolidation of the tail void grout, gradual closure of any ungrouted voids, and compression or consolidation of the soil around the tunnel.

Observations of the tail void grouting indicated that the void was adequately grouted and that the ground loss would be limited to the thickness of the shield tailskin, a volume of 2.4 ft^3/ft (0.2 m^3/m).

As the tunnel lining is gradually loaded by the overburden, the lining rings deform to an elliptical shape with a decrease (shortening) of the vertical diameter and an increase (lengthening) of the horizontal diameter. Measurements of the diameters of the inbound tunnel lining rings at cross-section 33-38 showed an average vertical diameter distortion of - 0.4 percent (shortening) of the theoretical circular diameter. The average horizontal diameter distortion was + 0.6 percent (lengthening). The ring deflected down at the crown and out at the springline. The resulting elliptical ring shape had a volume 0.2 percent (0.5 ft^3/ft, 0.05 m^3/m) greater than the volume of a perfectly circular ring. If the theoretical tail void volume of 2.4 ft^3/ft (0.22 m^3/m) is reduced by 0.5 ft^3/ft (0.05 m^3/m), the theoretical volume loss is now 1.9 ft^3/ft (0.18 m^3/m). This compares with the observed loss (tail plus long term) of 2.3 ft^3/ft (0.21 m^3/m).

Overexcavation by the shields accounted for more than half of the total observed ground loss. The overcutter bar on the front edge of the shield cuts out a excess volume of 4.2 ft^3/ft (0.39 m^3/m). Plowing of the shield as it is maneuvered through the soil produces an elliptical hole larger than the circular cross-section of the shield. Computation of the size of the openings plowed out by the shields was complicated by the articulation feature of the shields. The front section of the shield could be bent at a slight angle to the main body of the shield, while the free floating tail section could track along in the path of the main shield body. The laser alignment data for the shield was used to compute the position of the main body of the shield after each shove. The position of the front portion of the shield relative to the main body was determined from the setting of the articulation jacks. From this, the positions of the front edge of the shield and the front and rear articulation joints after each shove could be determined relative to the theoretical line and grade of the tunnel. The path followed by these points on the shield from shove to shove was used to estimate the shape and size of the tunnel opening plowed out by the shield. In this procedure, the tail section of the shield was ignored under the assumption that it tracked behind the main shield body and thus did not cause any additional plowing. This assumption reduces the effective length of the shield from 19 ft 7-1/2 in. (5.98 m) to 13 ft 5-1/2 in. (4.10 m). The cross-sectional area of the excavated tunnel opening was then compared to the circular shield cross-sectional area to determine the volume of overexcavation per foot of tunnel due to plowing.

Table A.6 summarizes the theoretical volumes of ground lost over the shields from the overcutters and from plowing. The theoretical volumes range from 4.6 to 8.0 ft^3/ft(0.43 to 0.74 m^3/m), while the actual losses occurring over the shields, estimated from the deep settlements, were 3.7 to 4.3 ft^3/ft (0.34 to 0.40 m^3/m).

A.5.2 GROUND SURFACE SETTLEMENTS

Final ground surface settlements that occurred along the centerlines of the F Route tunnels are plotted on Figs. A. 10 and A. 11. The settlements are those resulting from the passage of a single tunnel only and do not include any additional settlement due to mining of an adjacent tunnel. Average centerline surface settlements along the F Route tunnels were about 1.5 to 2.0 in. (40 to 50 mm). Since the pattern of ground surface settlement over a soft ground tunnel is bowl-shaped with the deepest part usually centered over the tunnel, the centerline settlements represent the maximum ground surface settlements.

Figures A. 12 and A. 13 show the ground surface settlements observed along cross-section 33-38 and 33-38 A. Two settlement troughs are shown for each cross-section; one showing the settlement after the first tunnel (inbound) was mined, and one showing the final combined settlement after both tunnels were completed. The maximum settlement after mining of the first tunnel was 1.8 in. (46 mm) at section 33-38, and 2.2 in. (56 mm) at section 33-38 A. The surface settlement troughs were roughly symmetrical about the inbound tunnel centerline. The eastern half of the trough at section 33-38 A is interupted by the presence of the church at the edge of the trough.

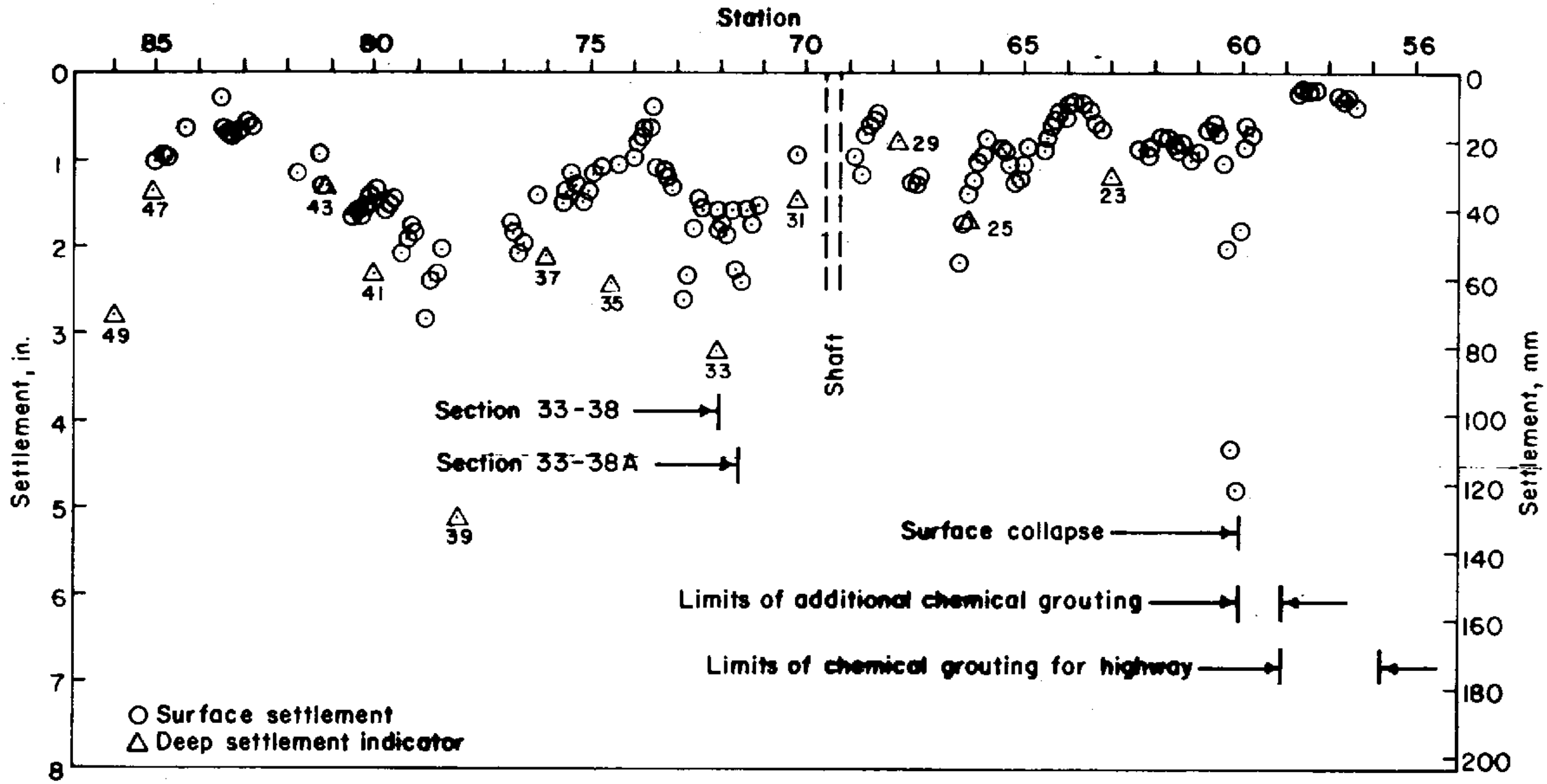


FIGURE A.10 GROUND SURFACE SETTLEMENT ALONG CENTERLINE OF FIRST (INBOUND) TUNNEL

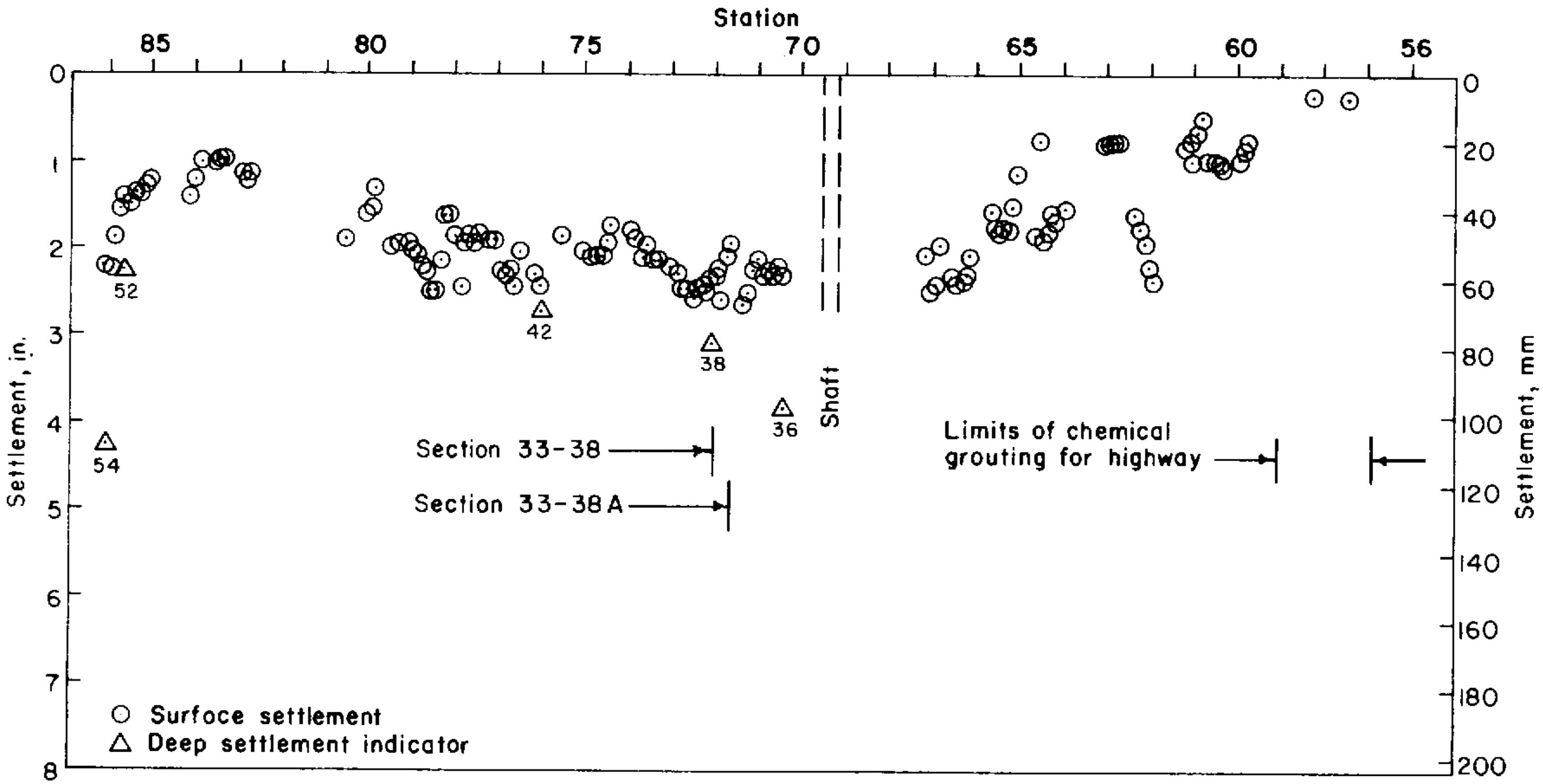


FIGURE A.11 GROUND SURFACE SETTLEMENT ALONG CENTERLINE OF SECOND (OUTBOUND) TUNNEL

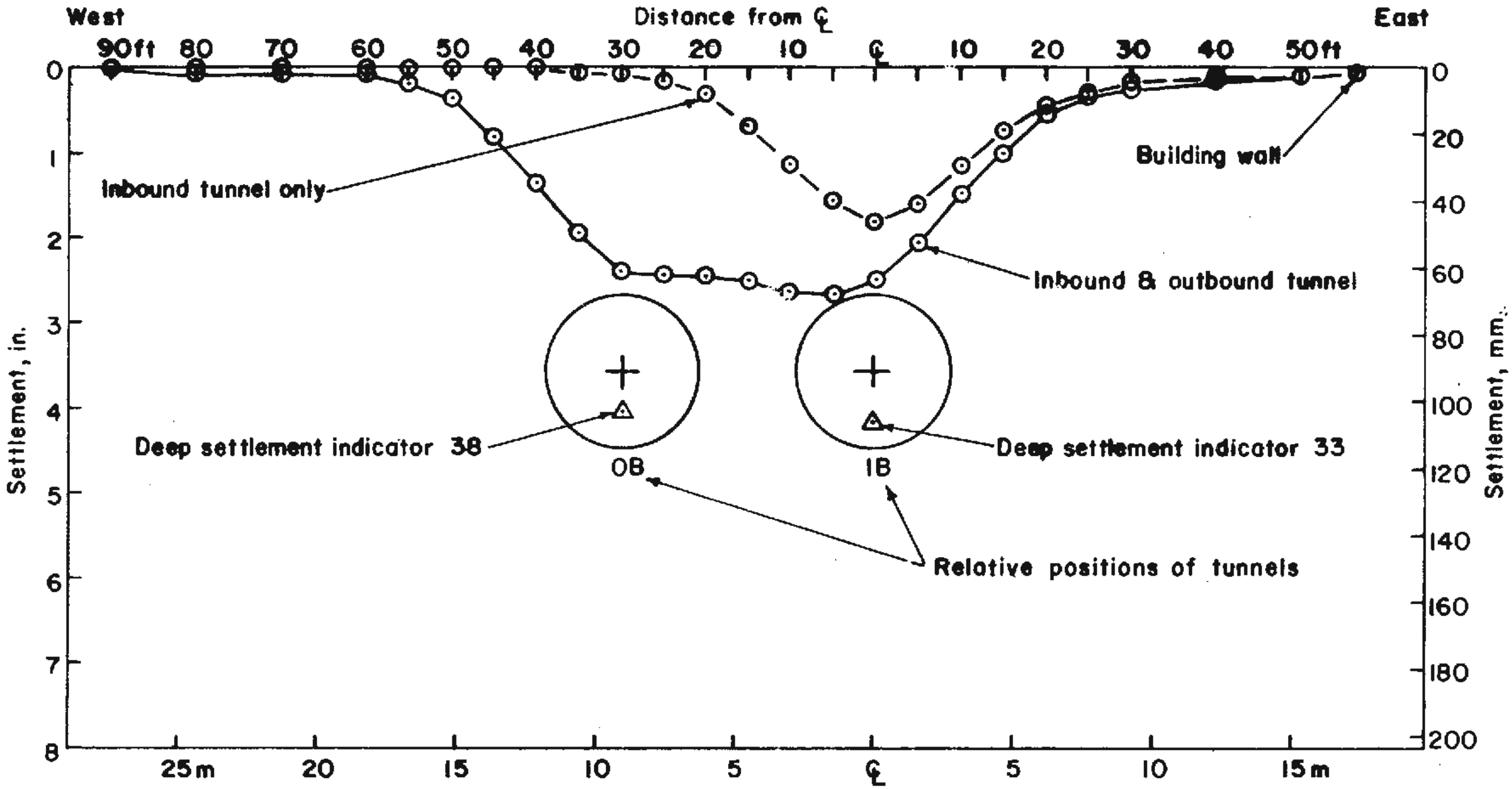


FIGURE A.12 GROUND SURFACE SETTLEMENT, CROSS-SECTION 33-38

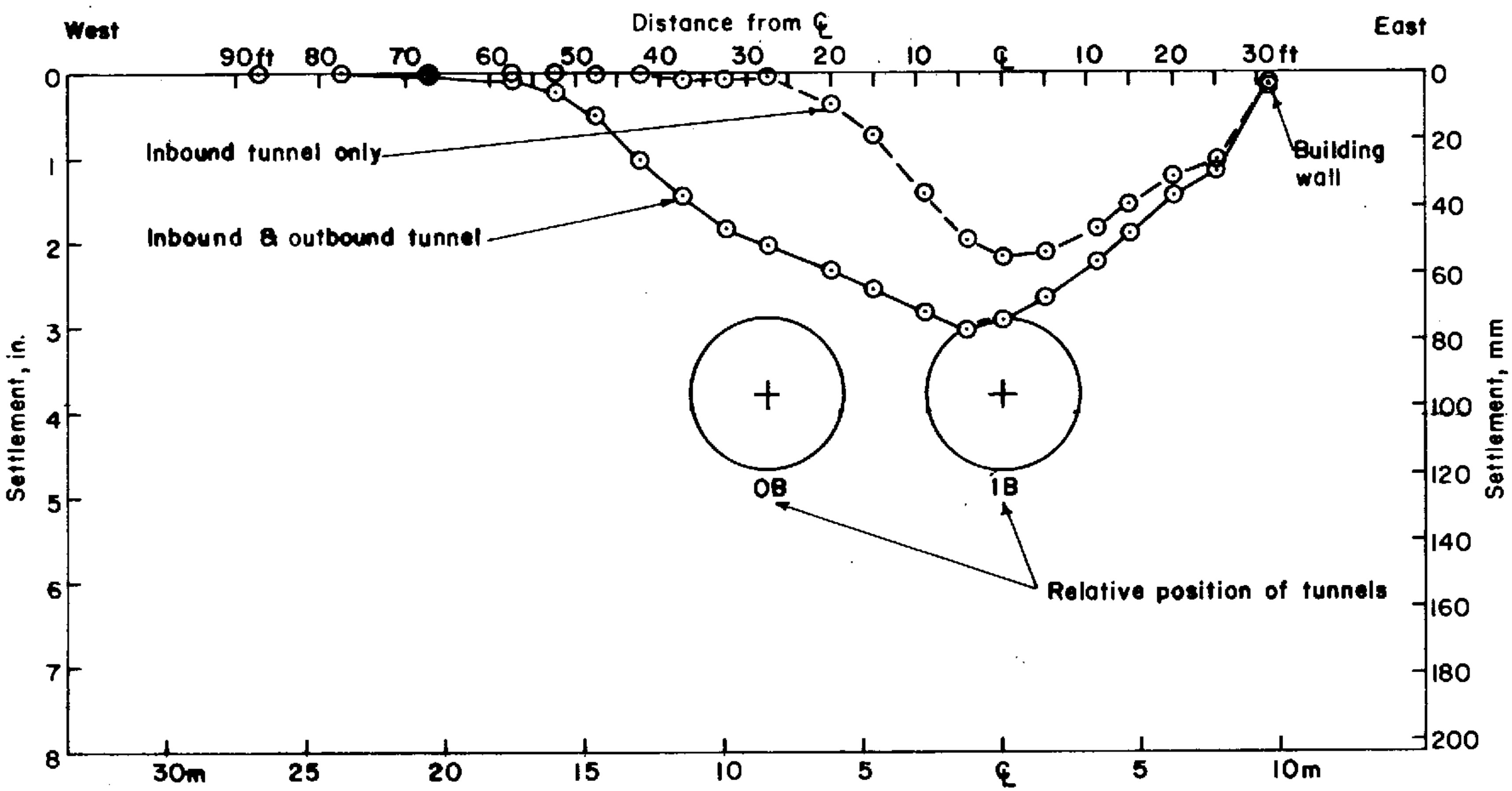


FIGURE A.13 GROUND SURFACE SETTLEMENT, CROSS-SECTION 33-38A

TABLE A.6

COMPARISON OF GROUND LOST OVER SHIELD, F ROUTE

	Volume due to overcutter ft^3/ft (m^3/m)	Volume due to plowing ft^3/ft (m^3/m)	Total ft^3/ft (m^3/m)
Inbound tunnel:			
theoretical	4.2 (0.39)	3.8 (0.35)	8.0 (0.74)
actual (DI 33)			4.3 (0.40)
Outbound tunnel:			
theoretical	4.2 (0.39)	0.4 (0.04)	4.6 (0.43)
actual (DI 38)			3.7 (0.34)

The half-width, w, of the first tunnel settlement trough was 25 to 30 ft (7.6 to 9.1 m) and the β angle was 24° to 30°. Table A.7 lists the geometric parameters, including slopes, of the trough. The volume of the surface settlement trough at the two cross-sections for the first tunnel ranged from 1.7 to 2.4 percent of the nominal tunnel volume.

The combined settlement trough after both tunnels were mined was asymmetrical with the deepest part located between the two tunnels. Settlement at the centerline of the outbound tunnel due to the mining of that tunnel was 2.3 in. (58 mm) at cross-section 33-38 and 2.0 in. (50 mm) at section 33-38 A. This is about the same as for the first tunnel. Maximum settlements for the combined troughs were 2.7 to 3.0 in. (69 to 76 mm) and were located at a point 5 ft (1.5 m) west of the inbound centerline. The additional volume of surface settlement due to mining of the second tunnel was 2.7 to 2.9 percent; the total volume of the combined trough was 4.6 to 5.1 percent.

The volume of surface settlement caused by the second tunnel is greater than that caused by the first tunnel primarily because of the disturbance of the ground by the mining of the first tunnel and the resulting interference between the surface settlement troughs of the two tunnels. The interference between the two settlement troughs results in additional settlement volume beyond that obtained by simply adding two single-tunnel troughs.

TABLE A.7

VOLUMES AND DISPLACEMENTS - F ROUTE TUNNELS

Case	z/2R	Vertical displacement δ_{max}, in. surface	deep	Volume of movement ft^3/ft surface V_S	tunnel V_L	Trough width i/R	w, ft	Slope of surface settlement trough cross-section	longitudinal section
1	2	3	4	5	6	7	8	9	10
Cross-section 33 - 38, first (Inbound) tunnel	2.0	1.8	3.2	4.4 1.7%	7.0 2.7%	1.3 (β = 30°)	30	1:200	1:380 - 1:1000
Second (Outbound) tunnel		2.3	3.1	7.4 2.9%	6.7 2.7%	1.1 (β = 24°)	25	1:130	—
Cross-section 33 - 38A, first (Inbound) tunnel		2.2	—	6.1 2.4%	—	1.4 (β = 32°)	33	1:180	—
Second (Outbound) tunnel		2.0	—	6.9 2.7%	—	1.3 (β = 29°)	30	1:180	—

For second tunnels, $V_S' = 5.0\ ft^3/ft \pm$ if interference volume is subtracted from total volume in col. 5.

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 ft^3/ft = 0.0929 m^3/m

The western half of the combined settlement trough beyond the outbound tunnel centerline is least affected by the trough interference. As a result, the shape of the western edge of the combined settlement trough is very similar to that of the first tunnel trough. In fact, the half-width and β angle are nearly identical (see Table A.7). If the second tunnel had been mined first, its settlement trough might have been approximately the same as that obtained by simply doubling the western edge of the combined trough. The volume of the doubled trough would be about 5.0 ft^3/ft (0.47 m^3/m). The actual volume of settlement caused by driving of the second tunnel was 6.9 to 7.4 ft^3/ft (0.64 to 0.69 m^3/m). (See Table A.8.) Based on these assumptions and observed volumes, the additional volume of surface settlement due to interference between the two settlement troughs would be 1.9 to 2.4 ft^3/ft (0.18 to 0.22 m^3/m). The increase in surface settlement volume due to interference is 38 to 48 percent more than the single-tunnel settlement volume.

TABLE A.8
SETTLEMENT TROUGH INTERFERENCE VOLUME
F ROUTE TUNNELS

	Cross-section	
	33 - 38	33 - 38A
Settlement trough volume,		
1. due to first tunnel, V_{S1}	4.4 ft^3/ft	6.1 ft^3/ft
2. due to second tunnel, V_{S2}	7.4	6.9
3. total, $V_{S1} + V_{S2}$	11.8	13.0
Volume of second tunnel trough if tunnel driven as single tunnel, V_{S2}'	5.0	5.0
Volume of surface trough interference,		
$\Delta V_S = V_{S2} - V_{S2}'$	2.4	1.9
$\frac{\Delta V_S}{V_{S2}'} \times 100$	48%	38%

1.0 ft^3/ft = 0.0929 m^3/m

The effect of mining a second tunnel adjacent to the first was also evident in the deep indicator settlements. When the first tunnel was driven, deep indicator 38, located at the future second tunnel centerline and 28 ft (8.5 m) away from the first tunnel centerline, settled 0.1 in. (2.5 mm). When the second tunnel was driven, deep indicator 33, located over the first tunnel centerline, settled 0.3 in. (7.6 mm). Measurements of the first tunnel lining diameters after mining of the second tunnel showed that the vertical diameter had decreased by 0.2 in. (-0.1 percent of circular diameter) while the horizontal diameter had increased by 0.2 in. (+0.1 percent).

A.5.3 COMPARISON OF V_S AND V_L

The volumes of the surface settlement trough, V_S, and ground loss, V_L, observed for the F Route tunnels at cross-section 33-38 are summarized in Table A.7. These values are plotted as $\%V_S$ versus $\%V_L$ in Fig. A.14. Deep settlements were also measured at a number of other locations along the tunnels. The ground losses at these locations were estimated from the deep settlements using Eq. 2.1. The losses ranged from 1.0 to 4.7 percent of the nominal tunnel volume. Measurements of surface settlement along cross-sections were not made at these other deep settlement indicators, but surface settlements at the tunnel centerline were measured. Assuming that the surface settlement troughs at these locations will have a shape similar to those measured at cross-sections 33-38 and 33-38 A, i.e. $\beta = 30^o$, the volume of surface settlement can be roughly estimated by assuming:

$$V_S = w\,\delta_{max} = \delta_{max}\,(z \tan\beta + R) \qquad \text{Eq. A.2}$$

where V_S = volume of surface settlement, ft^3/ft of tunnel length

w = half-width of settlement trough

δ_{max} = maximum surface settlement at centerline of tunnel, ft

z = depth to tunnel axis, ft

β = angle of trough width

R = radius of tunnel, ft

Settlement volumes estimated in this manner ranged from 0.4 to 2.0 percent of the nominal tunnel volume. These values have been plotted in Fig. A.14 along

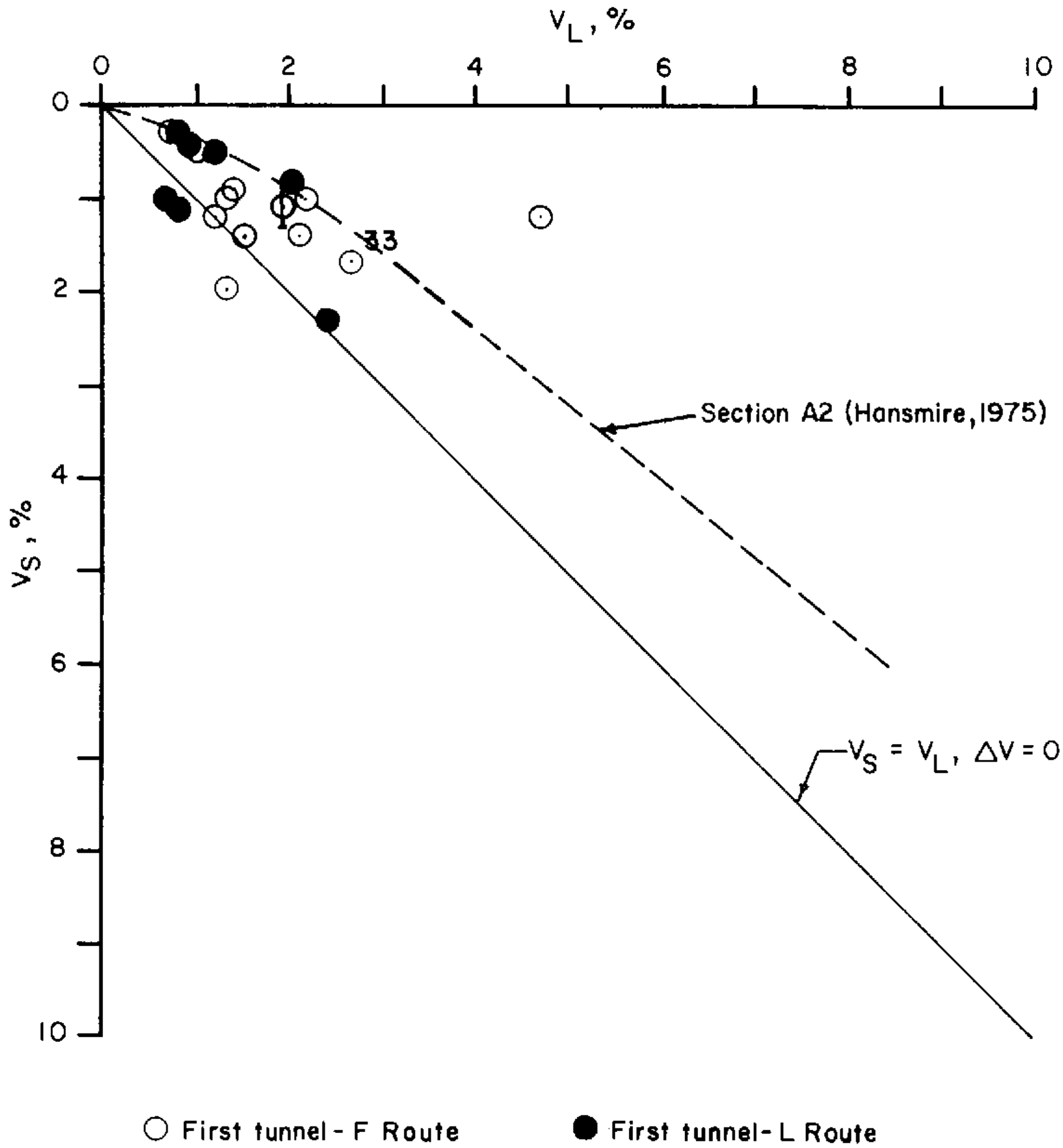


FIGURE A.14 COMPARISON OF VOLUME OF LOST GROUND AND VOLUME OF SURFACE SETTLEMENT, FIRST TUNNELS, F AND L ROUTES with data from the L Route tunnels and Section A2 of the Metro. The plotted values are for the passage of the first tunnel of the pair of tunnels.

The values of % V_S and % V_L in Fig. A.14 generally plot above the line % V_S = % V_L. This indicates that for these tunnels and soil conditions a net volume expansion has occurred as a result of the tunneling disturbance in the soils around and over the tunnels, and consequently this net volume expansion has offset part of the ground loss during tunneling and reduced the volume of surface settlement. The points that plot below the % V_S = % V_L line are from locations where the surface settlement data was not very detailed, and thus there may be some error in the estimate of the surface settlement volume. A net volume decrease, as implied by points below the % V_S = % V_L line, would not occur in these dense soils.

The data from Section F2a tunnels agrees with the relationship developed from the Section A2 data. The volume of ground loss was less on Section F2a than A2 because of better construction procedures. As a result, the surface settlement volume was also less in Section F2a.

APPENDIX B

CASE 19 - WASHINGTON, D.C. METRO
SECTION G1 TUNNELS

B.1 PROJECT DESCRIPTION

Metro Section G1 consists of two, circular, single-track, earth tunnels running east along Benning Road, NE, from the Anacostia Freeway to East Capitol Street in Northeast Washington, D.C. Figure A.1 shows the location of Section G1 in relation to other earth tunnel sections of the Metro System. On Section G1, a total of 8,600 ft (2,620 m) of 21-ft (6.4-m) diameter, single-track tunnel was driven from four headings. The tunnels were driven with articulated shields equipped with breasting flaps and digger arms. The shields were nearly identical to the ones used on Metro Section F2a, and both sections were built by the same contractor. The tunnels were initially lined with steel ribs and wood lagging, expanded in place. A permanent lining of cast-in-place concrete was placed later. Two headings were driven east and two were driven west from a construction shaft located between the Anacostia Freeway and Minnesota Avenue, NE.

This appendix reports the results of ground surface and subsurface settlement observations made at three cross-sections located over the second east tunnel heading (Inbound-East or IB-E) between stations 307+90 and 308+70 on the northwest corner of Minnesota Avenue and Benning Road, NE, about 250 ft (76 m) east of the construction shaft. Several buildings located on the northern edge of the tunnel were instrumented for measurements of settlement, tilt, and strain. The results of these building studies are reported by Boscardin, Cording, and O'Rourke (1977).

B.2 GEOLOGY AND SOIL PROPERTIES

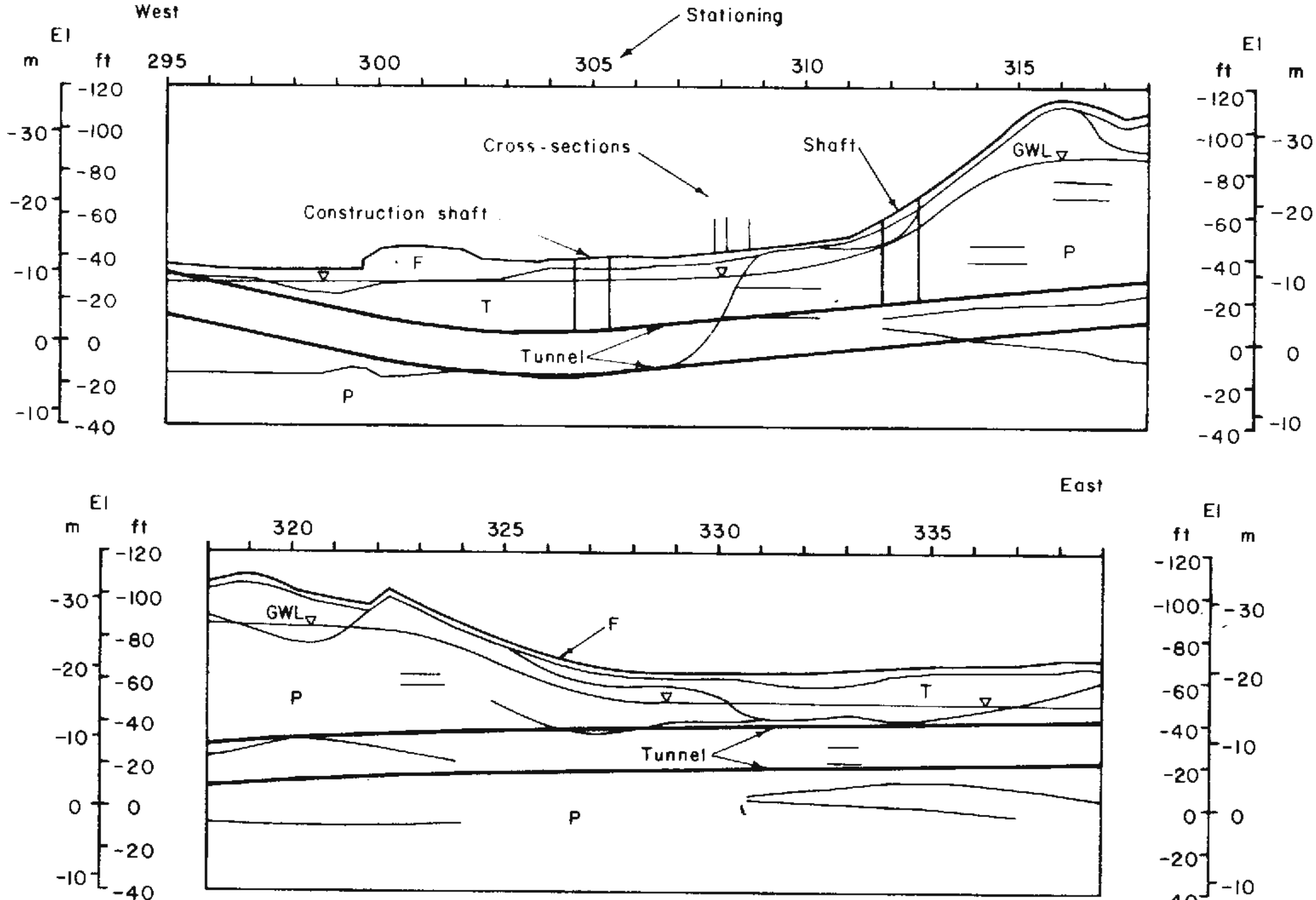
Section G1 runs from the flood plain of the Anacostia River east into the uplands adjoining the river valley. About thirty percent of the tunnel is in interlayered Pleistocene stream deposits of silty sand

and silty clays in the valley bottom. Seventy percent of the tunnel is in stiff, fissured clays and very dense, clayey sands of the Cretaceous age Potomac Formation that forms the uplands. Figure B.1 is a general profile of Section G1 showing these major divisions. The settlement cross-sections were located at the transition from the Pleistocene valley deposits to the Cretaceous deposits. Figure B.2 shows the details of the soil profile at the cross-sections. These profiles were developed from information in Mueser, Rutledge, Wentworth, and Johnston (1973 b, 1974 and 1975) and from observations in the tunnel heading. The materials are very similar to those encountered on other Metro projects, and the same general strata designations, T1, T2, T3, T4, T5, P1, P2, etc. are used here. A description of the soil strata and a summary of their properties are presented in Table B.1.

In the Inbound-East heading, the transition from a full face in Pleistocene materials to a full face in Cretaceous materials occurred over a distance of about 180 ft (55 m). The T1 clay encountered in the upper part of the tunnel face is slightly over consolidated and has a shear strength of 0.8 to 1.2 ksf (38.3 to 57.5 kN/m^2). This clay is softer and has a higher organic content than the T1 clay on Section F2a. The stability ratio for the T1 clay in the tunnel face at the settlement cross-sections was:

$$\frac{\gamma z}{s_u} = \frac{130 \text{ pcf } 40 \text{ ft}}{0.8 \text{ to } 1.2 \text{ ksf}} = 7.2 \text{ to } 4.8$$

This value is high enough that large plastic deformations might occur in the tunnel face and result in large volumes of ground lost through the tunnel face. However, at the settlement cross-sections, the clay occupied only the top few feet of the face and was supported with the breasting flaps on the shields. The rest of the face was in the very stiff to hard Cretaceous clay. As a result, there was no excessive deformation or loss of ground through the tunnel face at the cross-sections. The remolded T1 clay did extrude around the edges of the breast flaps during shoves of the shield.



F = Fill
T = Pleistocene
P = Cretaceous

0 200 ft
0 60 m
Horizontal scale

FIGURE B.1 GEOLOGIC PROFILE - METRO SECTION G1

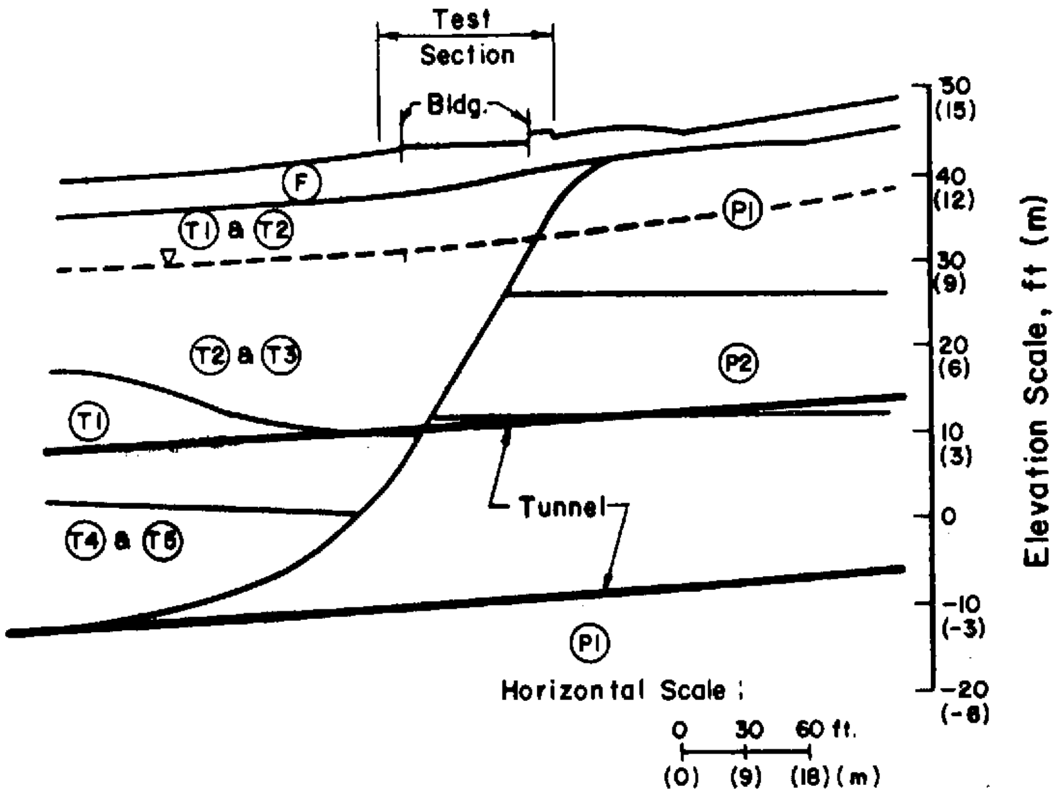


FIGURE B.2 GEOLOGIC PROFILE AT SECTION G1 CROSS-SECTIONS

TABLE B.1

SOIL PROPERTIES - SECTION G1

Soil Stratum	Description	Unified Soil Classification primary	Unified Soil Classification secondary	Standard Penetration Resistance, N	Natural water content, %	w_L, Liquid Limit, %	I_p, Plasticity Index, %
F	Fill, generally of inorganic soil obtained from nearby natural materials.	ML, SM	SC, CL	--	--	--	--
T1	Stiff to medium light brown or gray or mottled brown-gray silty clay with lenses of brown silty fine sand.	CL, CH, OL	Lenses of SM or SC	10-30	28	45	24
T2	Medium compact to compact brown and orange-brown silty or clayey fine to medium sand with traces of small gravel.	SM, SP	SC, SW	15-40			
T3	Compact to very compact brown and red-brown fine to coarse sand with some silt and gravel, or sand and gravel	SM, SP	GP, GM	40-60 (100 + in gravels)	--	--	--
T4	Medium compact to compact gray and gray-brown fine to medium sand with some silt and small gravel containing lenses of dark gray clay.	SM, SP	SC, SW	15-50	--	--	--
T5	Compact to very compact gray and gray-brown fine to coarse sand with some gravel and some to trace silt.	SM, SP	GP, GM	30-100 +	--	--	--
P1	Hard mottled red-brown and gray or light gray and tan plastic clay with occasional pockets of fine sand.	CH	CL	40-50	22	60	37
P2	Compact to very compact light gray or tan silty or clayey fine to medium sand with pockets of silty clay and trace of small gravel, occasional lignite fragments.	SC, SM	SP	30-60)	--	--	--
P3	Hard gray-green or gray-blue silty or sandy clay and sandy silt or silty or clayey fine sand with occasional small gravel.	CL, SC	CH, SM	50-100	--	--	--

Mueser, Rutledge, Wentworth and Johnston (1973 b)

The P1 plastic clay encountered at the transition from the Pleistocene deposits to the Cretaceous hillside deposits appeared to be weathered and less plastic and more brittle than the P1 clay encountered several hundred feet further in under the hill. The clay was very stiff to hard with an unconfined compressive strength greater than 4.5 tsf (430.9 kN/m^2) as determined by penetrometer tests on the tunnel face. The clay was fissured and some slickensides were observed. The clay along the fissures appeared brown and weathered. Overall the clay at the transition had a brownish cast as opposed to the more typical red-gray mottling. Thin seams of white-gray clayey sand were scattered throughout the clay.

Throughout the test section length, the tunnel face was generally excavated to a vertical plane. The face was firm with very little raveling in the P1 clay. The breasting flaps were used when the T1 clay was present at the crown.

The groundwater table was located about 20 ft (6.1 m) above the crown of the tunnel. A deep dewatering well was located at station 308+50 between the two tunnels and adjacent to the cross-sections. The inbound tunnel heading was dry except for slight seepage (1 gpm (4 liters per min) or less) along a silt-sand seam at the contact between the T1 clay and P1 clay. The seepage increased slightly during the shoves. Seepage from this contact also flowed in through the lagging of the tunnel lining at the tail of the shield. Very heavy flows that washed in some silt and sand occurred temporarily when the lining ribs were expanded. These flows diminished and dried up 6 to 10 ft (1.8 to 3.0 m) behind the tail of the shield.

B.3 CONSTRUCTION DETAILS

The G1 tunnels were excavated using 20 ft - 10 in. (6.35 m) diameter Robbins, articulated shields equipped with hydraulic digger arms and breast flaps. Except for their size, the shields were nearly identical to those used on Section F2a. Figure B.3 shows the details of the shield. The shield was equipped with a 1/2-in. (13-mm) thick overcutter or reliever bar

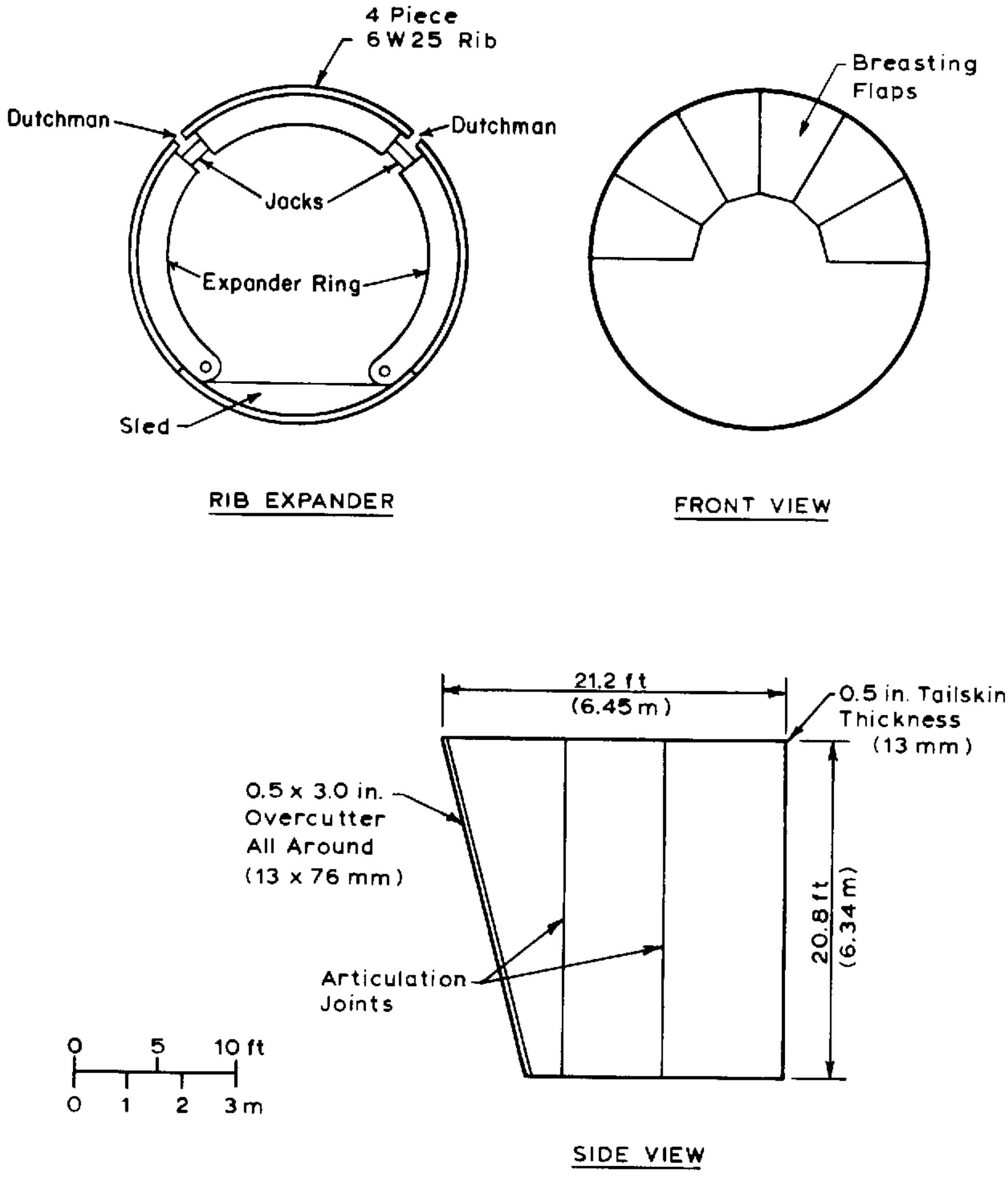


FIGURE B.3 SECTION G1 TUNNEL SHIELD DIMENSIONS at the leading edge and had a 1/2-in. (13-mm) thick tailskin. The tailskin was not stepped as on the F2a shields. The shield had three sections with a 3-in. (76 mm) articulation joint between sections. The front and middle sections were connected with a series of hydraulic jacks that could be used to control the attitude of the front section relative to the middle section. The tail section was attached to the middle section so that it could freely track behind the main shield body. The free floating tail section had the effect of reducing the overall length of the shield from 21.2 ft (6.46 m) to 14.4 ft (4.39 m) (Length/Diameter from 1.0 to 0.7). This makes it more manuverable, and can help reduce the volume of ground lost through plowing of the shield through the ground.

The tunnels were lined initially with circular steel ribs and wood lagging that were erected within the tail of the shield and then expanded into tight contact with the soil after they emerged from the tail of the shield. The ribs were 6-in. WF 25 lb (152 mm WF 11.34 kg) sections of A36 steel. The ribs were in four pieces with two bolted joints and two joints that were connected by short, welded-in sections of rib (dutchmen) after the rib was expanded. The ribs were on 3 ft-9 in. (1.14 m) centers with 5 in. by 8 in. (127 mm by 203 mm) wood lagging placed between the rib flanges.

The ribs were erected in the tail of the shield with the rib joints located as shown in Fig. B.3. The joints in which the dutchmen were later inserted were located on the upper right and left quarter points rather than on the springline. When a new rib was erected, the previous rib, still unexpanded, was about 8 in. (203 mm) inside the tail of the shield. On the next shove of the shield, this rib would emerge from the tail. At the end of the shove, the trailing sled of the shield and expander ring would be moved forward, and the expander ring would be positioned under the unexpanded rib. The expander ring consisted of a heavy steel ring beam that was hinged to the trailing sled as shown in Fig. B.3. The expander ring was cut at the upper quarter points, opposite the dutchmen joints in the ribs, and two hydraulic jacks were installed there. When these jacks were operated, the circumference of the ring expanded, shoving the tunnel rib into contact with the surrounding soil. Dutchmen of appropriate length (usually 4 to 6 in., 101 to 152 mm) were inserted into the open rib joints and welded in place. The expander ring was relaxed slightly but left in place until after the next shove, when it was moved forward to the next rib.

An unexpanded tunnel rib had an outside diameter of 20 ft-9 in. (6.32 m). The outside diameter of the shield, including overcutter, is 20 ft-11 in. (6.38 m). To expand the tunnel rib to this diameter in order to bring the ribs into full contact with the soil and to fill the tailskin void would require expanding the rib to accept a total length of dutchmen (right side plus left side) of 8.5 in. (216 mm). Usually the combined length of dutchmen inserted was 10 in. (254 mm), so the minimum amount of rib expansion was usually met.

Although the expanded tunnel ribs were tightly in contact with the soil, the lagging between the ribs was not necessarily in contact with the soil. The lagging rests against the inner flange of the rib, leaving a gap of 1 in. (25 mm) or more between the surface defined by the outside of the tunnel ribs and the outside of the lagging. (See Fig. B.9). Observations in the Inbound-East tunnel indicated that there was usually soil in contact with the lagging all around the tunnel circumference. Thus this gap was either filled by expansion of the ribs or by collapse of surrounding soil into the gap. The volume loss associated with this gap is discussed more in Section B.5.1.

The lining ribs are not expanded as soon as they emerge from the tail of the shield, but at the end of a shove. This means that they are expanded at a point 3 ft (1 m) behind the tail of the shield. Because of this slight delay in expanding the rib, some of the tail void is filled by collapse of the surrounding soil into the void. This would be especially true in granular soils or a soft clay. Some of this loss is offset by the subsequent rib expansion pushing material back, but there is still some ground loss. A few observations in the Inbound-East tunnel where the tunnel crown was in the T1 clay or the sandy material at the T1-P1 contact indicated that a substantial part of the tail void over the crown was filled with soil by the end of the shove. In the stiff P1 clay, the clay did not immediately fill the tail void, and a 1-in. (25 mm) gap or more could be observed between the clay surface and the lagging over the crown. The effect of delaying the expansion of the lining was not so great in this case where the soil could stand unsupported for some time.

B.4 OBSERVATION AND MONITORING PROVISIONS

The section of the Inbound-East tunnel where settlements were observed was located in the contractor's yard area adjacent to several two-story brick buildings. Figure B.4 shows the general plan of the settlement observation points. Figure B.5 is a section through the tunnels showing the relative positions of the tunnels and adjacent buildings. The tunnels at this location are 42 ft (12.8 m) apart, center to center, and 44 ft (13.4 m) deep, ground surface to tunnel axis.

A series of three cross-sections were established perpendicular to the inbound tunnel centerline at stations 307+90, 308+15, and 308+70. Ground surface settlement points were located along these sections and along the tunnel centerline at 5-ft (1.5 m) intervals. The lengths of the cross-sections south of the tunnel were limited by the contractor's plant and the traffic lanes of the street.

The surface settlement points on cross-section 307+90 and at points adjacent to the deep settlement indicators consisted of 3-ft (1-m) lengths of steel reinforcing bars driven into the ground. The remainder of the surface points were masonry nails driven into asphalt pavement or chisel cuts on concrete pavement.

Four deep settlement indicators previously installed by the contractor as part of the Metro tunnel contract were incorporated into the settlement observations. The locations of these deep settlement indicators are shown on Fig. B.4. Two of the indicators were located over the inbound tunnel, and two were over the previously driven outbound tunnel. The stations and depths of the indicators are given in Table B.2. The indicators consisted of a 1-in. (25 mm) diameter steel pipe encased in a

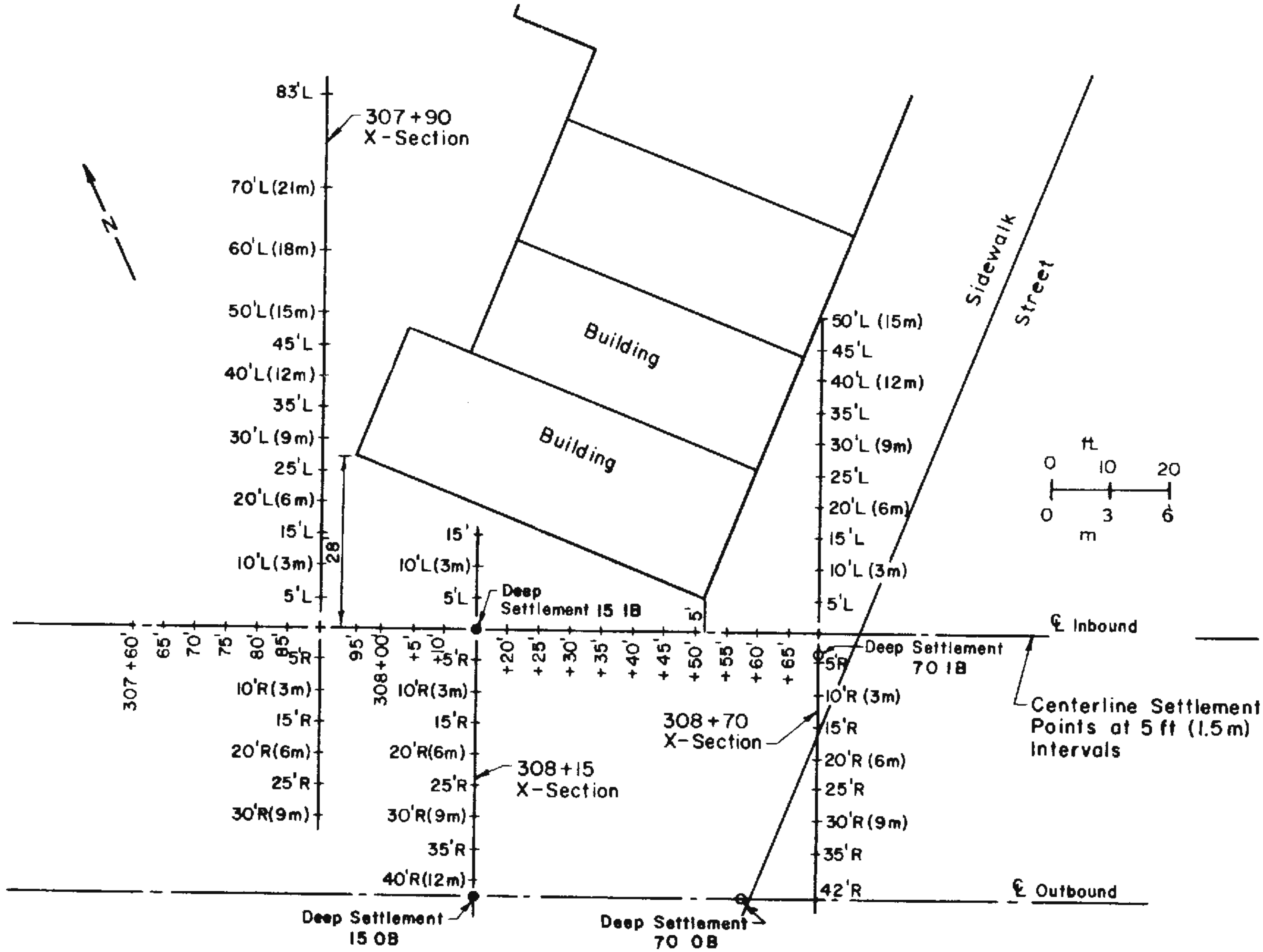


FIGURE B.4 DETAILS OF SECTION G1 SETTLEMENT CROSS-SECTIONS

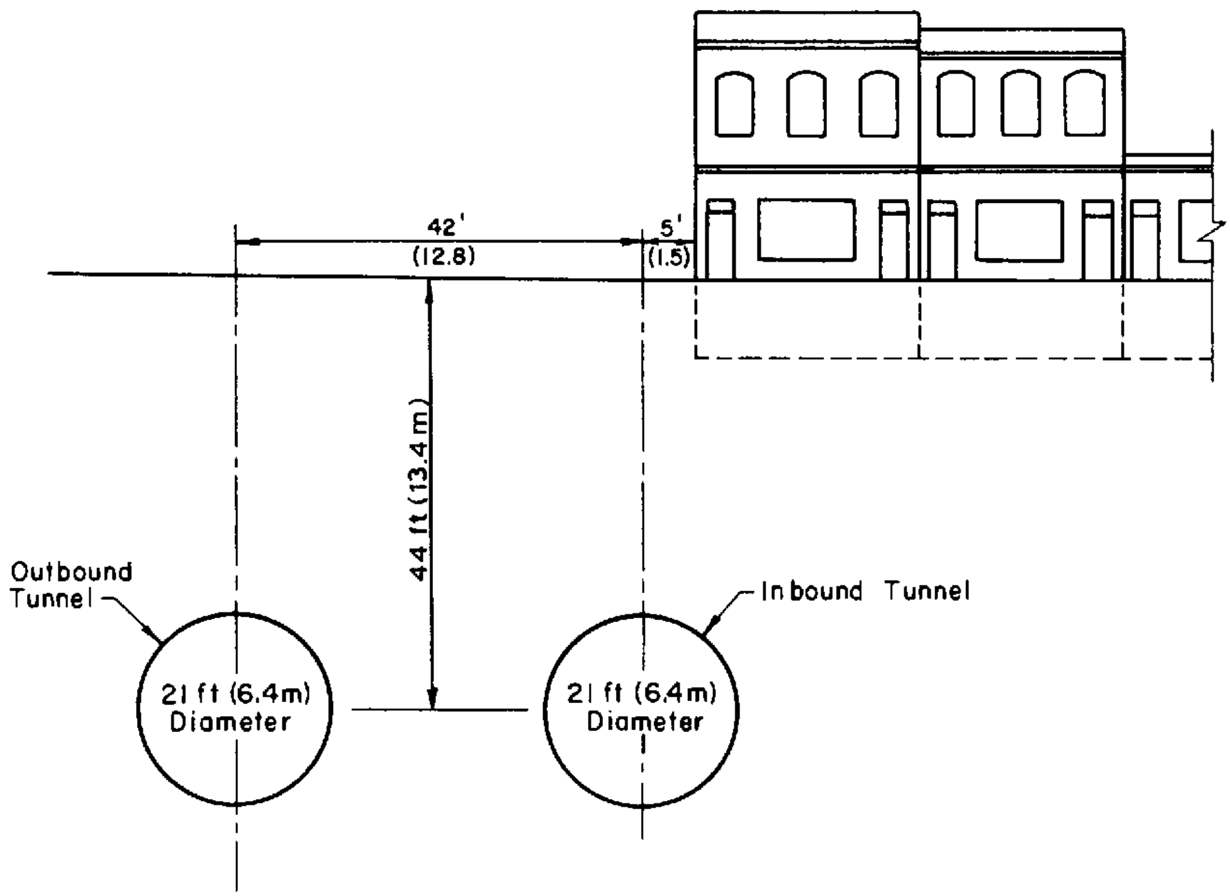


FIGURE B.5 CROSS-SECTION AT SETTLEMENT OBSERVATION LOCATIONS

2 1/2-in. (84 mm) diameter steel pipe. The tip of the 1-in. pipe was set 5 ft (1.5 m) above the crowns of the tunnels. Settlements of the inner pipe thus reflect settlements of the ground 5 ft (1.5 m) above the tunnel. A surface settlement point, consisting of a 3-ft (1-m) length of reinforcing bar was set next to each deep indicator.

TABLE B.2

LOCATIONS OF OBSERVED CROSS-SECTIONS AND DEEP SETTLEMENT INDICATORS

Cross-section	90	15	70
Deep settlement indicator	None	15 IB 15 OB	70 IB (1) 70 OB (2)
Centerline station	307+90 IB 307+83 OB	308+15 IB 308+07 OB	308+70 IB 308+62 OB
Distance between tunnel centerlines		42 ft (12.8 m)	
Depth to tunnel springline, Z			
inbound	43 ft (13.1 m)	44 ft (13.4 m)	44 ft (13.4 m)
outbound	43 ft (13.1 m)	42 ft (12.8 m)	43 ft (13.1 m)
Distance above tunnel crown to tip of deep settlement indicator, y		5 ft (1.5 m)	

Deep settlement indicators located on centerline at cross-section, except
(1) 5 ft (1.5 m) right of IB centerline
(2) 308+52 OB

The surface and subsurface settlement points were observed regularly as the inbound tunnel heading was driven through the area. A Zeiss Ni-2 automatic level was used with a Philadelphia leveling rod. Two temporary benchmarks were established on buildings near the sections. These benchmarks were in turn tied to another benchmark 500 ft (150 m) away. Closure differences on leveling circuits around the cross-sections were on the order of ± 0.001 ft (± 0.3 mm).

The Outbound-East heading was driven through the future cross-section locations in October, 1976. The cross-section points were installed in late January, 1977, and the Inbound-East heading was driven through the area in early February, 1977. Final measurements were made two weeks later. Both headings were driven from west to east in the direction of increasing centerline station. The inbound heading was worked on three 8-hr shifts. The rate of advance through the cross-section area was about 90 ft per day (27 m per day).

B.5 OBSERVATIONS AND INTERPRETATIONS

B.5.1 LOSS OF GROUND

Detailed observations were not made on the first tunnel, the Outbound-East tunnel. Contractor's data indicated that the deep settlement above the first tunnel crown was 2.4 in. (61 mm) at the deep settlement indicator located at station 308+07 OB (No. 15 OB) and 2.0 in. (51 mm) at station 308+62 OB (No. 70 OB). No ground surface settlements were measured immediately adjacent to these deep settlement indicators, but data from other nearby points showed an average surface settlement of 1.0 in. (25 mm) along the outbound tunnel centerline.

Detailed measurements of deep settlement and adjacent ground surface settlements were made for the second tunnel, the Inbound-East, and the results for the deep indicators at stations 308+15 IB and 308+70 IB are plotted versus position of the tunnel on Figs. B.6 and B.7 respectively. The deep settlements over the second tunnel were about the same magnitude as those over the first tunnel. Total deep settlement observed at 308+15 IB was 2.2 in. (56 mm) and 1.7 in. (43 mm) at 308+70 IB. The deep indicator at 308+70 IB (No. 70 IB) was located about 5 ft (1.5 m) right of the centerline.

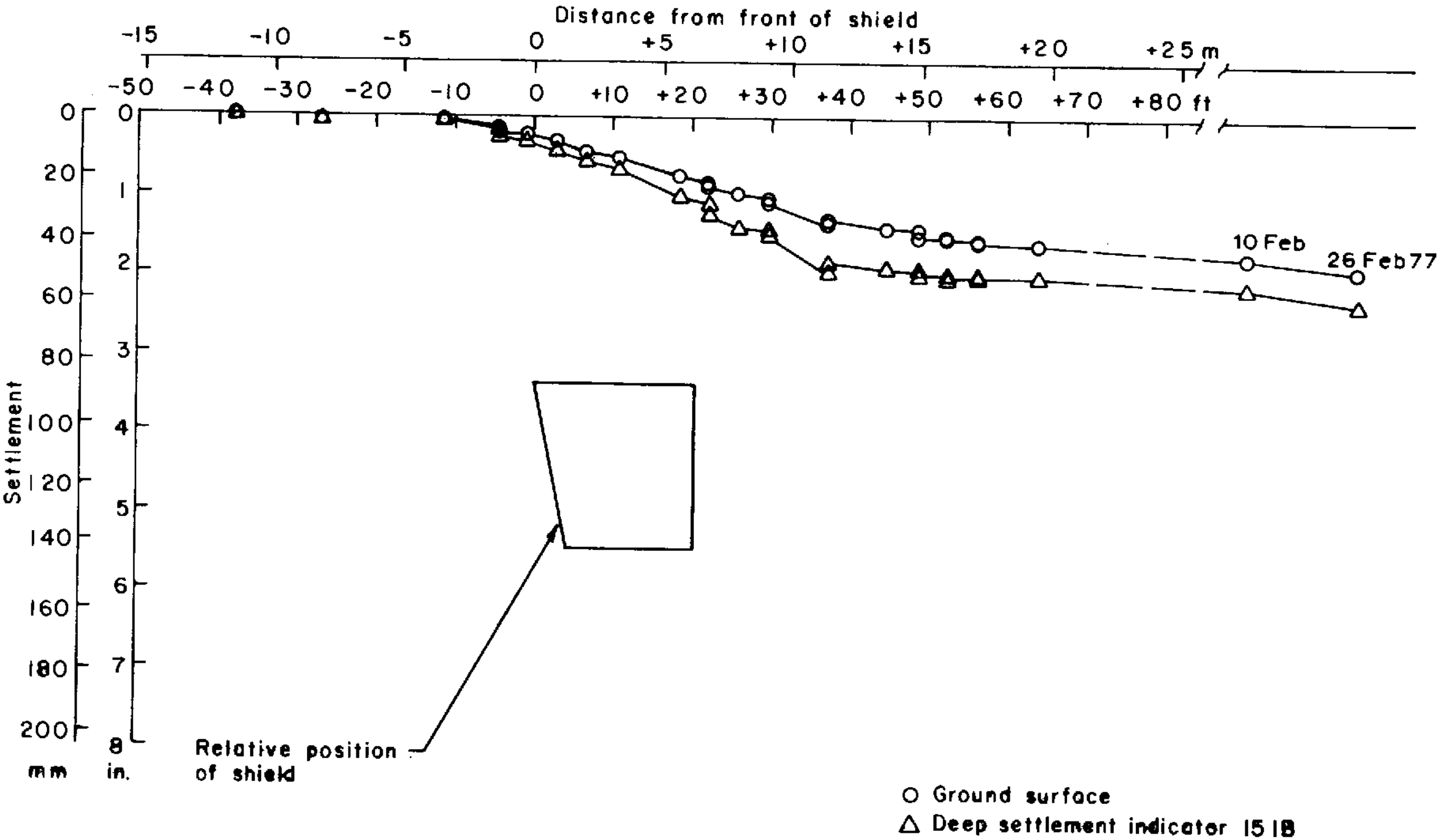


FIGURE B.6 GROUND SURFACE AND DEEP SETTLEMENTS, CROSS-SECTION 308+15, SECOND (INBOUND) TUNNEL

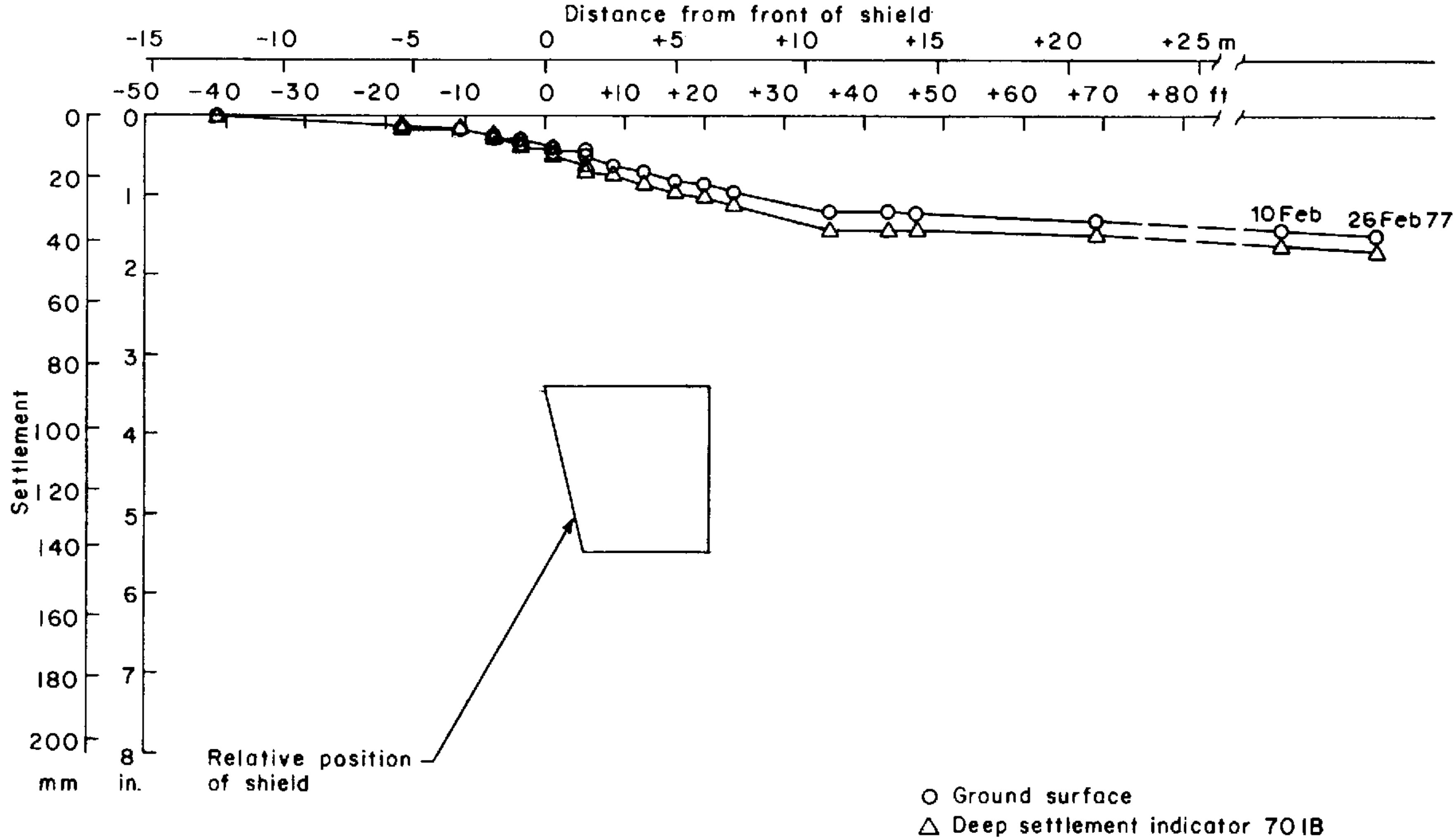


FIGURE B.7 GROUND SURFACE AND DEEP DETTLEMENTS, CROSS-SECTION 308+70, SECOND (INBOUND) TUNNEL

The adjacent ground surface settlements over the second tunnel were 2.4 in. (61 mm) at 308+15 IB and 1.6 in. (41 mm) at 308+70 IB. These are larger than observed over the first tunnel. Part of this increase however is due to interaction between the first tunnel and second tunnel. This interaction usually produces additional surface settlement volume and settlement above the second tunnel. In general, the surface and subsurface settlements appear to be about the same for both tunnels. This would be expected since the tunnels were driven in identical conditions with identical construction procedures.

An interesting trend was evident in the ground surface settlements along the inbound tunnel centerline. See Fig. B.8. The final surface settlements steadily decreased as the tunnel went from the Pleistocene soils to the Cretaceous soils. This same decrease was evident in the deep settlements; those at 308+70 were less than those at 308+15.

At 308+15 IB the silty-sandy contact between the Tl clay and Pl clay was located just below the crown of the tunnel. The material at the contact tended to collapse readily into any voids left by the shield overcutter and tail. Seepage along the contact helped aggravate this. In contrast, at 308+70 IB the tunnel was entirely in the hard Pl clay. This material was strong enough to stand unsupported for some time, thus allowing the voids around the tunnel to stand open longer. The longer the void stands open, the better the chance that it will be supported by expansion of the tunnel lining, and if this occurs, the less the volume of ground loss. If the volume of ground lost around the tunnel is reduced by driving into stronger materials, the ground surface settlements should be reduced also. This is the trend shown by Fig. 8.

The volumes of ground lost around the second tunnel at various stages of the tunnel advance were estimated from the deep settlement data using Eq. 2.1 and are summarized in Table B.3. The first source of lost ground, excessive displacement of the ground into the tunnel face, was a very small portion of the total estimated volume loss. At both deep indicator locations the Pl clay formed most of the face, and very little displacement would occur in this material. The presence of Tl clay and

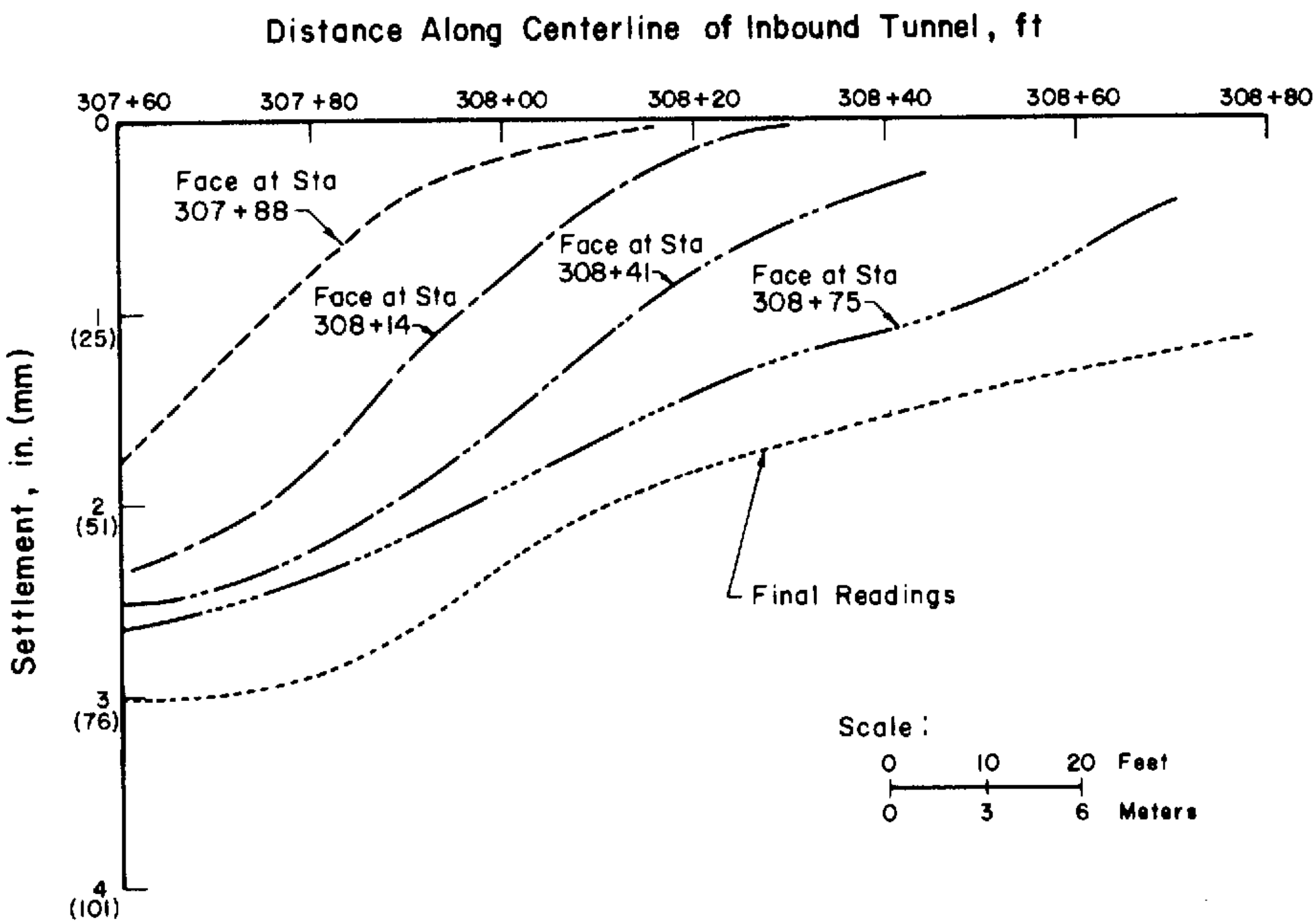


FIGURE B.8 GROUND SURFACE SETTLEMENTS ALONG CENTERLINE OF SECOND (INBOUND) TUNNEL

TABLE B.3

LOST GROUND AROUND SECTION G1 TUNNELS

Case 1	z/2R 2	Lost ground Before face 3	Over shield 4	At tail 5	Lining deflection and time dependent movements 6	Total 7
Second (Inbound) tunnel at cross-section 308 + 15	2.1	δ_v = 0.3 in. V_L = 0.8 ft^3/ft $\%V_L$ = 0.2%	0.9 in. 2.4 ft^3/ft 0.7%	0.4 in. 1.1 ft^3/ft 0.3%	0.6 in. 1.5 ft^3/ft 0.5%	2.2 in. 5.8 ft^3/ft 1.7%
Second (Inbound) tunnel at cross-section 308 + 70	2.1	δ_v = 0.4 in. V_L = 1.0 ft^3/ft $\%V_L$ = 0.3%	0.7 in. 1.7 ft^3/ft 0.5%	0.3 in. 0.8 ft^3/ft 0.2%	0.3 in. 0.9 ft^3/ft 0.3%	1.7 in. 4.4 ft^3/ft 1.3%

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft^3/ft = 0.0929 m^3/m

silty-sandy contact in the upper 2 ft (0.6 m) of the tunnel face at 308+15 did not affect the overall loss of ground through the face because the material was controlled by the breasting flaps.

The major portion of the estimated volume loss at both deep indicators, occurred over the shield due to overexcavation. The 1/2-in. (12-mm) overcutter on the nose of the shield contributed a volume loss of 2.7 ft^3/ft (0.25 m^3/m). Additional overexcavation comes from plowing of the shield. The theoretical ground losses due to plowing of the shield were computed from shield alignment data. The computed volume at 308+15 was 0.4 ft^3/ft (0.04 m^3/m) and 1.3 ft^3/ft (0.12 m^3/m) at 308+70. Total computed loss (overcutter plus plowing) over the shield would then be 3.1 ft^3/ft (0.29 m^3/m) and 4.0 ft^3/ft (0.37 m^3/m) respectively. These values are much greater than the volumes estimated from the observed deep displacements, 2.4 ft^3/ft (0.22 m^3/m) and 1.7 ft^3/ft (0.16 m^3/m). The differences in the computed and observed values are probably due to the error in estimating ground loss all around a tunnel from a single displacement measurement. The observed deep settlements however do show less displacement when the tunnel is in the stronger P1 clay.

The other major sources of lost ground are a) loss at the tail of the shield due to delay in expanding the lining and failure to expand the lining enough to completely fill the excavated cross-section, and b) long term, more time dependent volume losses due to deflection of the lining, consolidation and compression of the soil around the tunnels, etc. In this case the soils surrounding the tunnel including the T1 clay, were slightly to heavily overconsolidated. Any additional consolidation and settlement caused by stress redistribution around the tunnel and drainage of the groundwater was probably negligible, especially over the 2 to 3 week period of the settlement measurements. Similarly, compression of the soils due to stress redistribution was probably small. Deflection of the lining due to the soil loading however could be large, especially for a very flexible lining of ribs and lagging.

Another potential source of large, long term ground loss was voids remaining behind the lining after expansion. Most of the tail void, the gap between the outside of the unexpanded tunnel lining as it emerges from the tail of the shield and the outside of the tail of the shield, would be filled by immediate collapse of the surrounding soil and expansion of the tunnel lining. A few pockets might remain open for some distance (or time) behind the tail of the shield, especially in the P1 clay. Gradual collapse and closure of these few voids and deflection of the lining under loading thus would constitute most of the long term settlement and ground loss.

Since the process of transfer of soil load to the lining and collapse of voids around the lining begins right after the lining emerges and is expanded, the division between long term and immediate tail loss is somewhat arbitrary. For the values listed in Table B.3, long term deep settlements were considered to be those occurring after the tail of the shield had advanced more than 2 shove lengths (7.5 ft or 2.3 m) beyond the deep settlement indicator. The combined tail loss and long term loss was slightly less at station 308+70 IB than at 308+15 IB (0.5 percent vs 0.8 percent). The major factor affecting the magnitude of these losses was the effective size of the lining after expansion.

As a check on the effectiveness of the rib expansion, the length of dutchmen inserted and the horizontal and vertical rib diameters after expansion were measured at several ribs on either side of the deep indicators. Table B.4 summarizes this data.

The volume of the expanded lining was computed from this data in two ways. The lengths of the dutchmen were added to the circumference of the 4 rib segments to obtain an expanded circumference. A radius and volume per foot of length were computed for a circle with an equivalent circumference. A second procedure assumed that the measured diameters were the major and minor axes of an ellipse and computed the volume per foot of length of that ellipse. Both computed volumes were then corrected for the gap between the outside of the ribs and the outside of the lagging. As mentioned earlier, the lagging rested on the inner flanges of the ribs

TABLE B.4

DIFFERENCE BETWEEN VOLUME OF LINING EXPANSION
AND VOLUME OF TUNNEL - INBOUND EAST TUNNEL

Rib No.	Station	Post-expansion OD Horizontal	Post-expansion OD Vertical	Volume Difference	Dutchmen	Volume Difference
		ft	ft	ft^3/ft	in.	ft^3/ft
1	2	3	4	5	6	7
72	308+08.99	21.01	20.84	-3.11	9	-2.77
73	308+12.76	21.00	20.88	-2.62	10	-1.90
74	308+16.35	21.03	20.84	-2.79	10	-1.90
75	308+20.12	21.01	20.80	-3.77	9	-2.77
76	308+23.89	21.00	20.85	-3.11	10	-1.90
	Average:			-3.08		-2.25
86	308+61.65	20.98	20.91	-2.45	10	-1.90
87	308+65.40	20.98	20.88	-2.95	10	-1.90
88	308+69.15	20.99	20.92	-2.12	11	-1.03
89	308+72.90	20.98	20.93	-2.12	11	-1.03
90	308+76.85	21.01	20.94	-1.46	12	-0.15
	Average:			-2.22		

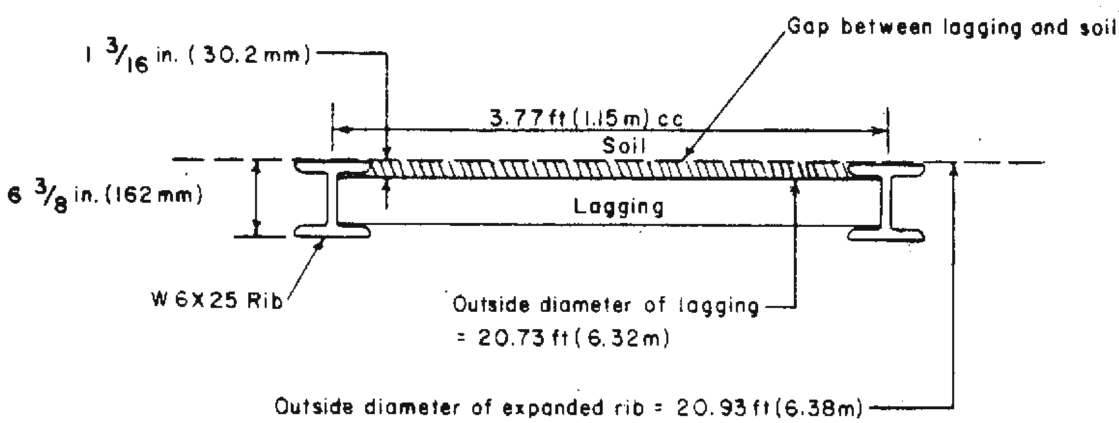
Col. (5) Volume difference = (horiz OD) (vert OD) $\pi/4$ - 6.12 ft^3/ft - 340.88 ft^3/ft.

Col. (7) Volume difference = πD^2 - 6.12 ft^3/ft - 340.88 ft^3/ft, D = [(dutchmen) + 65.02 ft] $\div \pi$.

Correction for lagging not being in contact with soil = 6.12 ft^3/ft.

Volume, 20 ft-10 in. OD shield = 340.88 ft^3/ft.

Conversions: 1 in. = 25.4 mm, 1 ft = 0.3048 m, 1 ft^3/ft = 0.0929 m^3/m.



Lagging, 5 in. X 8 in. (127 mm X 203 mm) nominal, 4 3/4 in. (121 mm) finished thickness

FIGURE B.9 DIMENSIONS OF GAP BETWEEN TUNNEL LINING LAGGING AND SOIL between the flanges. This left a gap or void of about 1-3/16 in. (30.2 mm) between the lagging and the soil, as shown in Fig. B.9. The volume of this void, 23.1 ft^3 (0.65 m^3) per lining ring or 6.1 ft^3/ft (0.57 m^3/m) was subtracted from the two computed lining volumes to obtain a corrected lining volume. The difference between the corrected lining volume and the volume of a 20 ft-10 in. (6.35 m) circle (tail of the shield) was assumed to indicate how effective the rib expansion was in preventing ground loss. The rib diameters were measured at a distance of several tunnel diameters behind the shield, and thus also include most of the deflection of the ribs due to soil loading.

Even though the ribs were expanded to a diameter equal to or slightly greater than the diameter of the shield (20.83 ft, 6.35 m), the volume of the expanded lining was still 1 to 3 ft^3/ft (0.09 to 0.28 m^3/m) less than the volume of the tunnel opening. The difference was due to the gap between the outside of the ribs and the outside of the lagging. To expand the lining so that the lagging would be in contact with the excavated surface (assuming a 20 ft-10 in. or 6.35 m diameter) would require an expanded rib diameter of slightly over 21 ft (6.4 m) or a dutchmen length of over 12 in. (305 mm). The lining expansion was slightly better at 308+70 IB as shown by the dutchman lengths, and hence the volume loss was less, as previously suggested by the deep settlements.

Table B.5 summarizes the volume losses estimated from the deep settlement data and those calculated from the overcutter dimensions, shield plowing and expanded lining dimensions. The estimated lining expansion losses include both the immediate loss and the long term loss listed in Table B.3. The estimated losses determined from the deep settlements using Eq. 2.1 are generally less than the calculated theoretical ground loss. For the lining expansion, the agreement between the two values is good.

B.5.2 GROUND SURFACE SETTLEMENTS

The ground surface settlements along the centerline of the inbound tunnel were presented in Fig. B.8. Figures B.10, B.11, and B.12 show the ground surface settlements along the three cross-sections at right angles

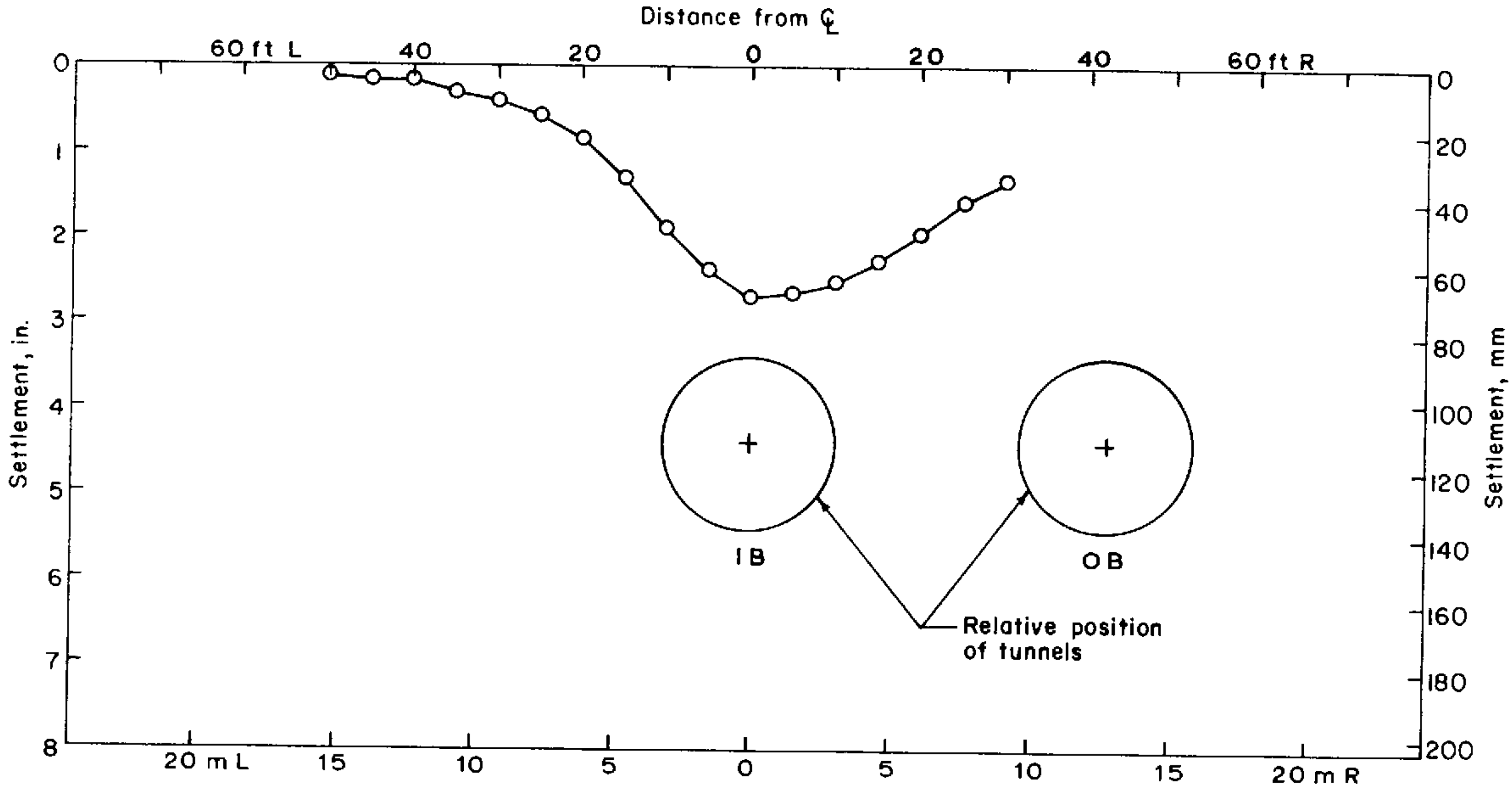


FIGURE B.10 GROUND SURFACE SETTLEMENTS, CROSS-SECTION 307+90

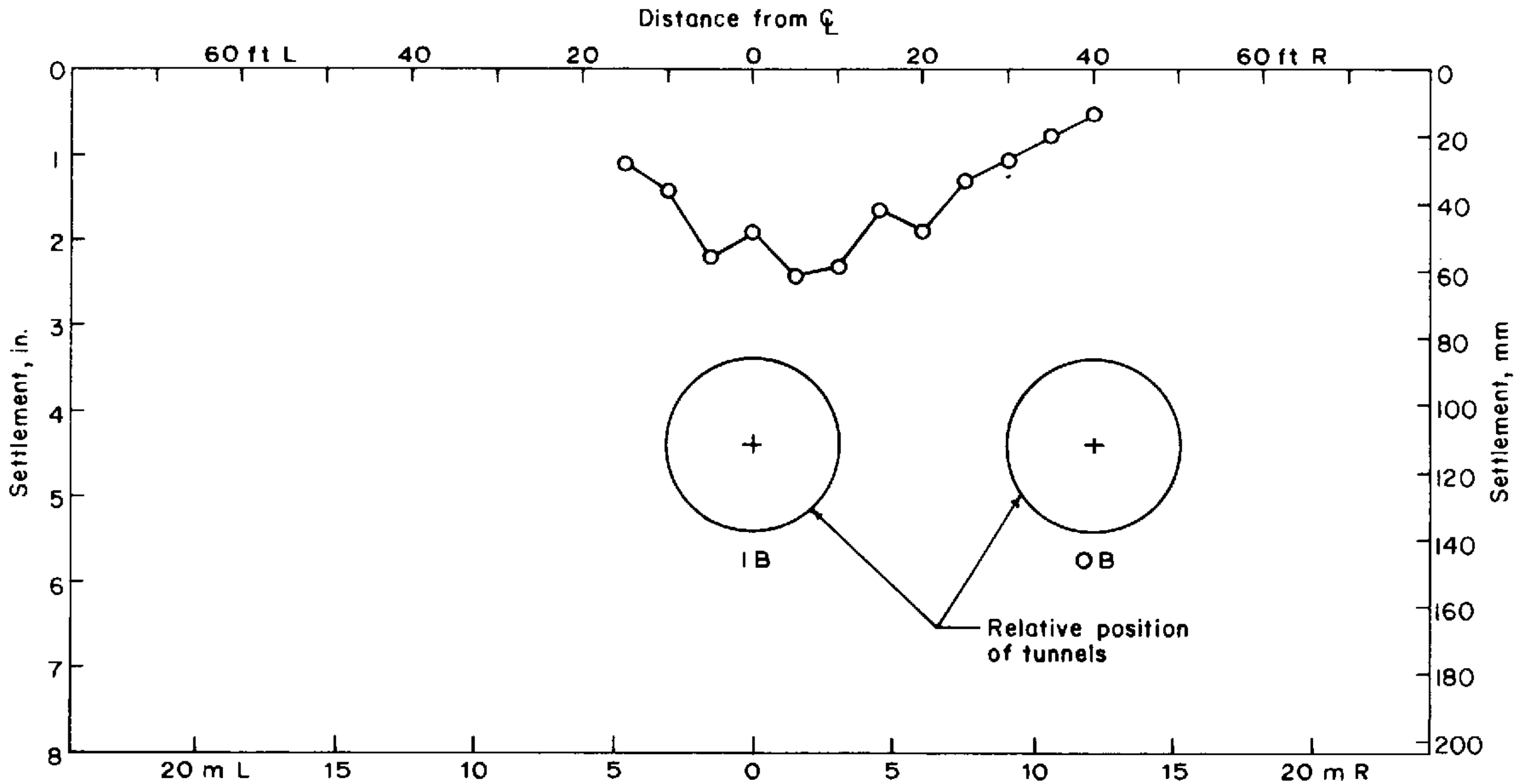


FIGURE B.11 GROUND SURFACE SETTLEMENTS, CROSS-SECTION 308+15

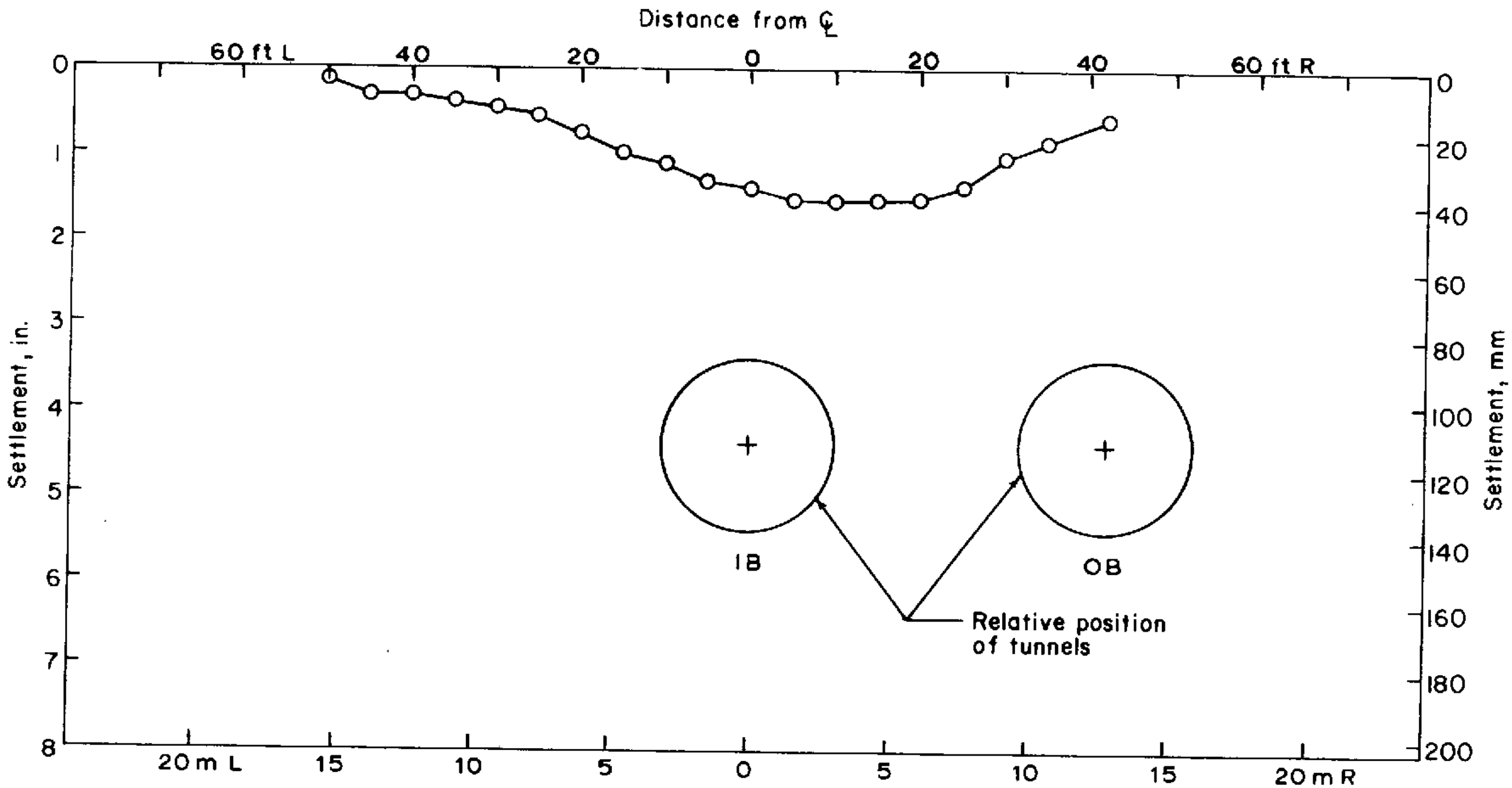


FIGURE B.12 GROUND SURFACE SETTLEMENTS, CROSS-SECTION 308+70

TABLE B.5

COMPARISON OF CALCULATED AND OBSERVED
VOLUMES OF LOST GROUND - INBOUND EAST TUNNEL

Source	Section 308+15		Section 308+70	
	Calculated V_L ft^3/ft	Observed V_L ft^3/ft	Calculated V_L ft^3/ft	Observed V_L ft^3/ft
1. Shield				
a. overcutter	2.7		2.7	
b. plowing	0.4		1.3	
c. total, a + b	3.1	2.4	4.0	1.7
2. Lining expansion	2.3-3.1	2.6	1.2-2.2	1.7
3. Total, 1 + 2	5.4-6.2	5.0	5.2-6.2	3.4

1 ft^3/ft = 0.0929 m^3/m

to the inbound centerline. All of these values are for the mining of the second tunnel (Inbound-East) only and do not include any settlement due to mining of the first tunnel. As noted before and shown in Fig. B.8, the surface settlements along the tunnel steadily decreased as the tunnel progressed into the stiffer Cretaceous soils. This decrease is also evident when the cross-section settlements at 307+90 are compared with those at 308+70 (Figs. B.10 and B.12).

Although the cross-sections do not extend completely across the surface settlement trough, the data does show the typical assymetrical shape of a second tunnel settlement trough. As a result, the maximum surface settlement of the combined first and second settlement troughs would be located between the tunnels.

Table B.6 summarizes the maximum settlements, settlement volumes, and geometric parameters for the second tunnel mining. The settlement

TABLE B.6

VOLUMES AND DISPLACEMENTS - SECTION G1 TUNNELS

Case	z/2R	Vertical displacement δ_{max}, in.		Volume of movement ft^3/ft		Trough width		Slope of surface settlement trough	
		surface	deep	surface V_S	tunnel V_L	1/R	w, ft	cross-section	longitudinal section
1	2	3	4	5	6	7	8	9	10
Cross-section 307+90, second (Inbound) tunnel	2.1	2.7	—	11± 3.2%	—	1.3 - 1.8 (β = 27° - 43°)	32 - 50	1:220	—
Cross-section 308+15, first (Outbound) tunnel		—	2.4	—	6.1 1.8%	—	—	—	—
Second (Inbound) tunnel		2.4	2.2	11.1 3.2%	5.8 1.7%	1.8 (β = 42°- 49°)	50 - 60	1:180 - 1:300	—
Cross-section 308+70, first (Outbound) tunnel		—	2.0	—	5.0 1.4%	—	—	—	—
Second (Inbound) tunnel		1.6	1.7	8.2 2.4%	4.4 1.3%	2.5 (β = 42°- 50°)	50 - 63	1:280 - 1:490	—

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft = 0.3048 m, 1.0 ft^3/ft = 0.0929 m^3/m

trough parameters are for the northern-half of the troughs. The northern-half of the trough was less affected by interaction of the first and second tunnel, and hence it is more representative of a single tunnel case. Where the cross-section was not complete, a suitable shape was sketched in to complete the trough and allow computation of the volume.

APPENDIX C

CASE 18 - ESLLIRP TUNNEL, ROCKFORD, ILLINOIS

C.1 PROJECT DESCRIPTION

The East Side Low Level Interceptor Relief Project (ESLLIRP), Contract 1A is located on the east bank of the Rock River between Harrison Street on the north and Brooke Road on the south, in Rockford, Illinois. Significant surface features along the construction centerline are shown on the site plan, Fig. C.1. The project consisted of 2,123 lineal ft (647 m) of 6 ft (1.8 m) diameter sewer, of which, 1628 ft (496 m) was constructed using a tunnel boring machine.

The tunnel was mined from north to south through medium dense to dense sand and gravel (glacial valley train outwash), at a depth to tunnel springline of 30 to 45 ft (9 to 14 m). These deposits are quite permeable and dewatering by deep wells was required prior to construction.

A Carl W. Decker, Inc. tunnel boring machine, equipped with a wheel type excavator, closed bulkhead and guillotine door, was used in tunnel construction. The temporary lining consisted of steel ribs and timber lagging. A 6-ft (1.8 m) inside diameter, reinforced concrete pipe was jacked into the tunnel from the shafts. The void between the pipe and the temporary lining was filled with lean concrete.

Construction of the tunnel began on 12 August 1976 and was completed on 12 October 1976. Figure C.2 shows a chart of mining progress. The maximum rate of advance was 101 ft/day (31 m/day) (one eight-hour shift per day). The average rate of advance was about 55 ft/day (17 m/day), not including days lost to breakdowns and other delays.

C.2 GEOLOGY AND SOIL PROPERTIES

Figure C.3 shows a generalized subsurface profile along the centerline of construction. The project is located in an area where the

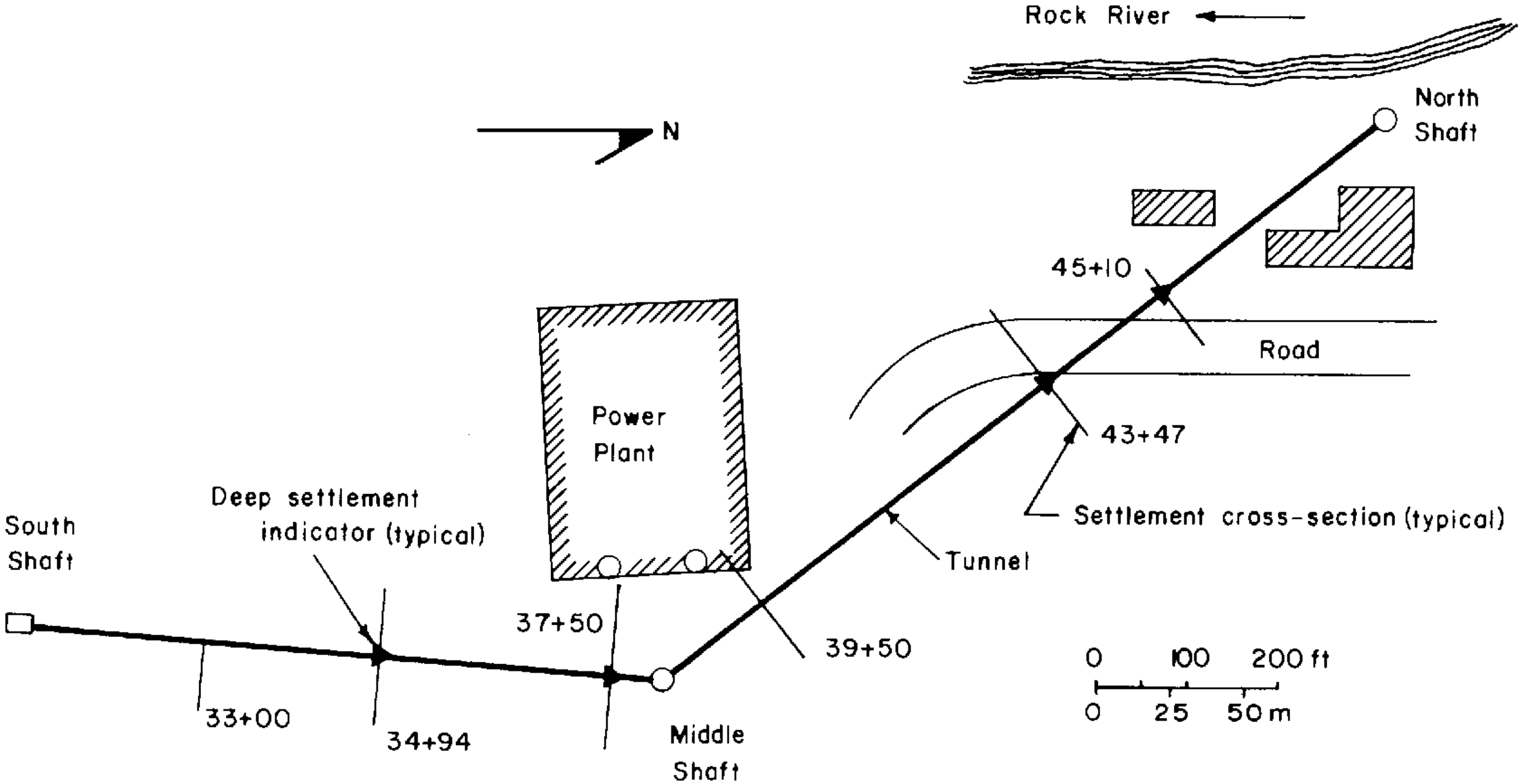


FIGURE C.1 SITE PLAN - ESLLIRP TUNNEL

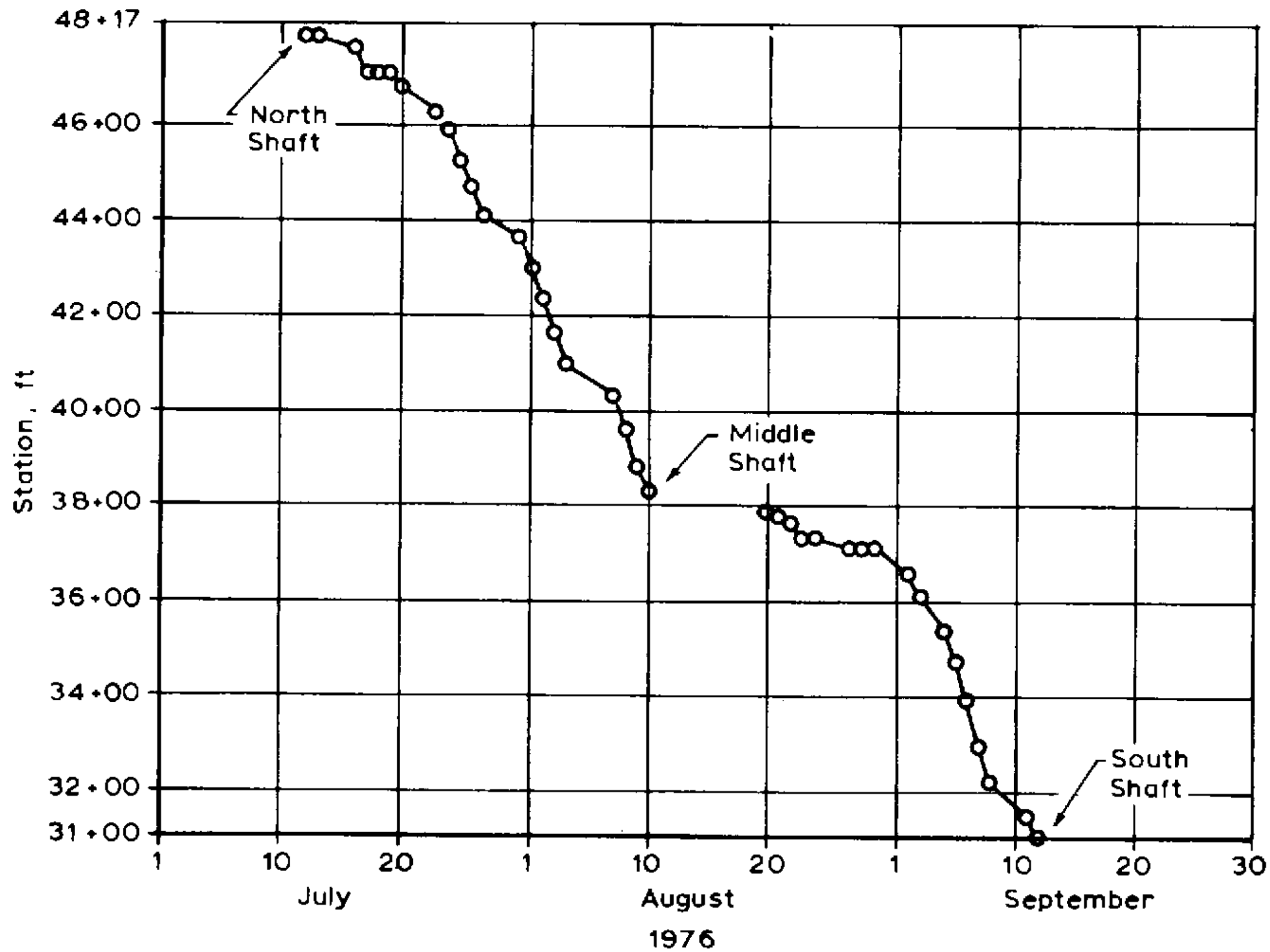


FIGURE C.2 MINING PROGRESS CHART

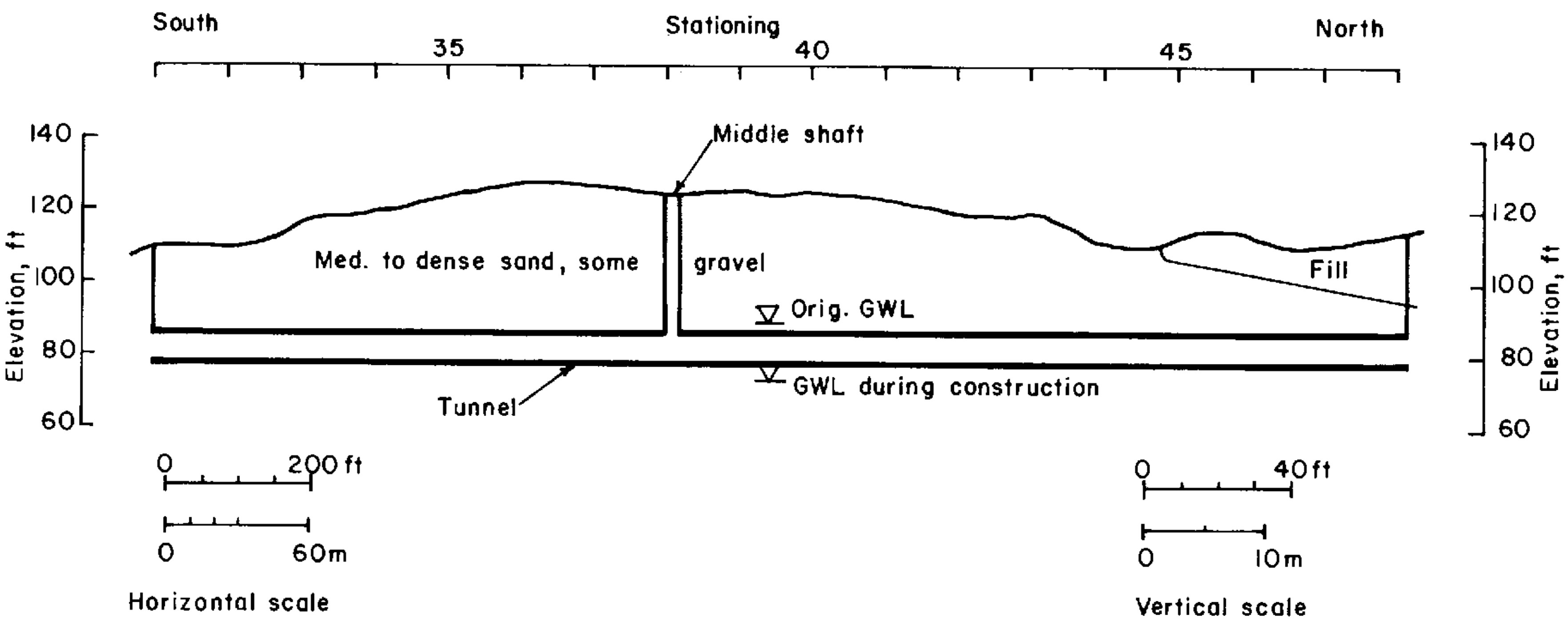


FIGURE C.3 GEOLOGIC PROFILE - ESLLIRP TUNNEL bedrock valley of the ancestral Rock River is now filled with as much as 200 to 300 ft (60 to 90 m) of younger glacial valley train deposits. These deposits were shaped by successive periods of erosion and deposition during the Pleistocene Epoch to form three distinguishable terraces along the Rock River. Most of the ESLLIRP Tunnel lies within an area that has been designated as the Upper Terrace. The Upper Terrace forms an intermediate level between the lower Terrace and the Rockford Terrace. The extreme northern end of the project is in a low-lying area where some recent alluvium may be present (Hendron, et al., 1975).

The terrace materials were deposited by moving water, probably in the form of braided streams, and consist primarily of clean sands and gravels, with some silt or clay present in thin lenses or as binder. These deposits tend to be irregularly stratified and lenticular, elongated in the north-south direction, sub-parallel to the valley axis. Gravel several inches in diameter commonly makes up a significant portion (perhaps 25 to 30 percent) of these materials. However, boulders larger than 6 to 12 in. (150 to 300 mm) are relatively rare. Figure C.4 summarizes the results of grain size tests performed on samples recovered from boreholes along the tunnel centerline.

The sand and gravel is generally in a medium dense to dense condition. Standard Penetration Test N-values ranged from 8 to 52 at tunnel grade, with an average N-value of about 25. Cemented zones, formed by iron and calcium precipitates, are often found near the groundwater level. These cemented zones can be several feet thick and may extend several tens to hundreds of feet horizontally.

The area near the north shaft has been a dump for refuse and rubble fill, including large, thick conrete fragments. The approximate depth of the fill zone is indicated on Fig. C.3. Some recent alluvium may be present beneath the fill and overlying the glacial valley train outwash deposits.

Groundwater levels along the tunnel are normally at the same level as the Rock River. Between June and January the Rock River level varies between El. 83 to 85 ft. A deep well pump test was conducted to provide

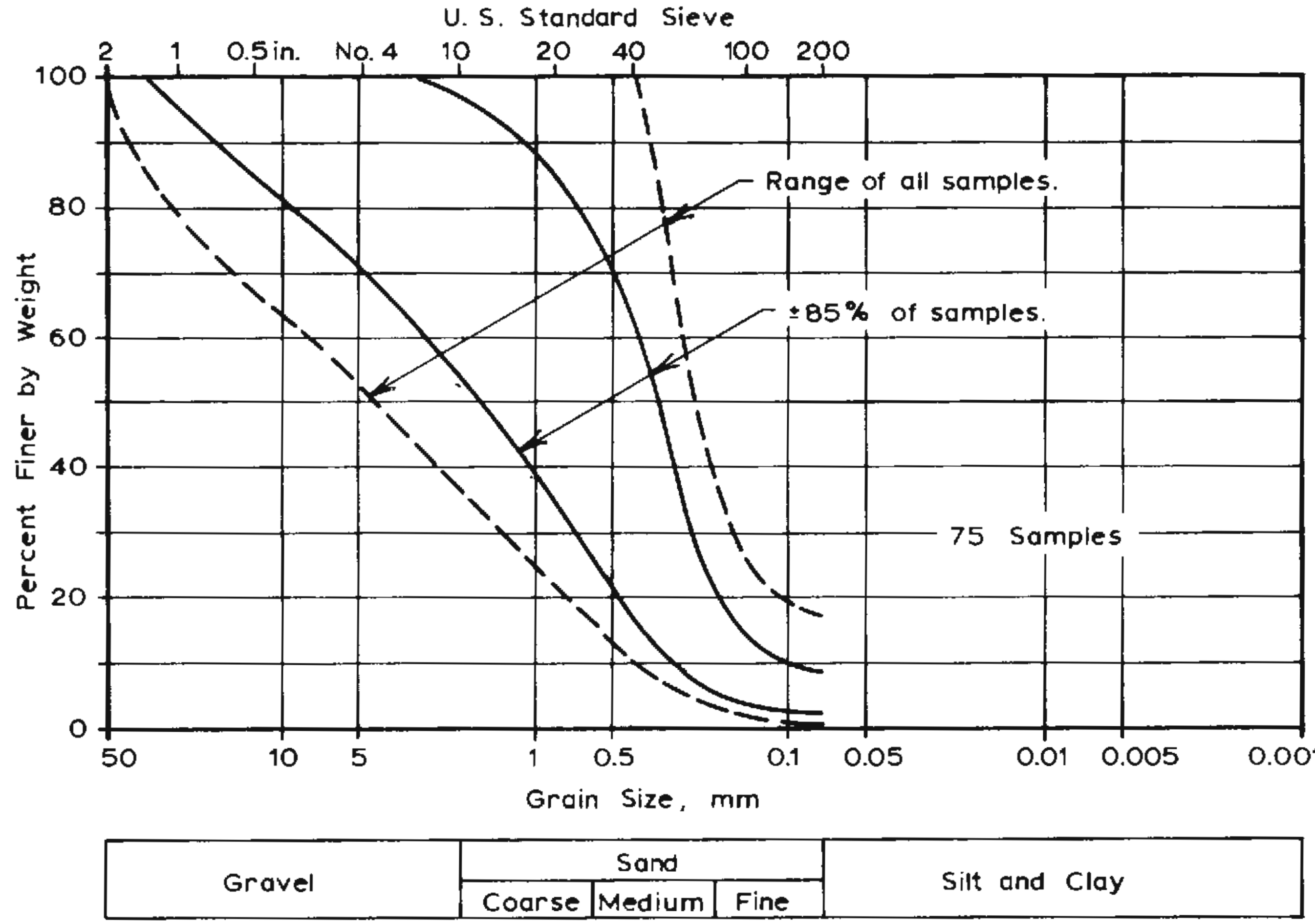


FIGURE C.4 GRAIN SIZE DISTRIBUTIONS - ESLLIRP TUNNEL data for design of a dewatering system. The pump test was located about 20 ft (6 m) west of the construction centerline, near Sta. 37+45.

The well was 3 ft (0.9 m) in diameter and extended 40 ft (12 m) below the static water level. The well delivered about 1600 gpm (100 l/sec) during two days of pumping. The transmissivity of the formation was determined to be about 300,000 gpd/ft (43 l/sec/m) with a storage coefficient of 0.01. The maximum drawdown in the well was 25 ft (7.6 m) and the specific capacity of the well was 64 gpm/ft (13 l/sec/m) of drawdown.

Well and piezometer installation began about 2 months prior to tunnel construction. Wells were installed in an 18 in. (457 mm) diameter rotary drilled hole. Each well included a 12-in. (305 mm) diameter well screen, 40 ft (12 m) long. A gravel filter pack (#6 to #10 sieve size material) was placed to about 15 ft (4.5 m) above the top of the screen. Submerged line shaft turbine pumps were used. Well tips were placed at about El. 40 (top of screen at about El. 80). A total of 13 wells were installed along the tunnel line at spacings of 140 to 300 ft (43 to 91 m), with a minimum clear distance of about 10 ft (3 m) between the well and tunnel springline.

The groundwater level was drawn down to about 3 ft (1 m) below the tunnel invert. Pumping rates for individual wells were 850 to 2300 gpm (55 to 145 l/sec). Total discharge for the system was 12,000 to 14,000 gpm (750 to 880 l/sec). Dewatering was successful and the tunnel was constructed in the dry.

The soil at tunnel grade can be described in terms of the tunnelman's ground classification system as running ground (see Table A.3). On one occasion, in the process of making a borehole to check the alignment of the tunnel, sand running in through the hole in the lining nearly blocked the tunnel. About 2.5 muck cars or $\pm$ 10 yd^3 ($\pm$ 7.5 m^3) of material came into the tunnel.

C.3 CONSTRUCTION DETAILS

The north and south shafts, both 24 ft (7.3 m) in diameter, were constructed within an open cut and then backfilled. The 24 ft (7.3 m) diameter middle shaft was constructed as a braced excavation, using liner plates. The ground around the middle shaft was stabilized by injection of chemical grout as the excavation progressed.

The Carl W. Decker, Inc. tunnel boring machine used on the ESLLIRP tunnel is shown on Fig. C.5. The shield is 9.33 ft (2.84 m) in diameter at the front, with a 0.25 to 0.375 in. (6 to 10 mm) thick "bead", built up by welding, at the cutting edge. The shield is 9.27 ft (2.82 m) in diameter at the tail and is 15 ft (4.6 m) in length. The machine is equipped with oscillating cutters and a closed bulkhead. Soil excavated by the cutters is loaded on a retractable conveyor by rotating baffle plates and admitted into the tunnel through a guillotine-type door. The alignment of the tunnel can be controlled by extending the steering flaps to react against the soil and by control of nine available shove jacks.

The temporary lining consisted of 3-piece 4-in. M 13 lb (102 mm M 5.9 kg) A-36 cold-formed steel ribs and 3 by 6 in. (75 by 150 mm) hardwood lagging. Ribs were spaced at 4-ft (1.2 m) centers, and expanded at the tail to a 9.31 ft (2.84 m) outside diameter. The rib expander mechanism is illustrated on Fig. C.5. Split pipe dutchmen were inserted at the springlines after expansion.

The permanent lining consisted of 6 ft (1.8 m) inside diameter, Class III reinforced concrete pipe sections (ASTM C-76) with rubber gasket joints (ASTM C-443). The pipe was jacked into place from the shafts, sliding on a cast-in-place concrete invert. The void between the temporary and permanent linings was filled with cement grout (lean concrete).

To start the tunnel, a concrete collar was formed against the inside of the shaft. Liner plate within the collar was then removed to expose steel sheeting driven just outside the shaft. The tunnel boring machine was then positioned with the cutting edge within the collar, and sand was poured through a pipe to fill the space between the shield

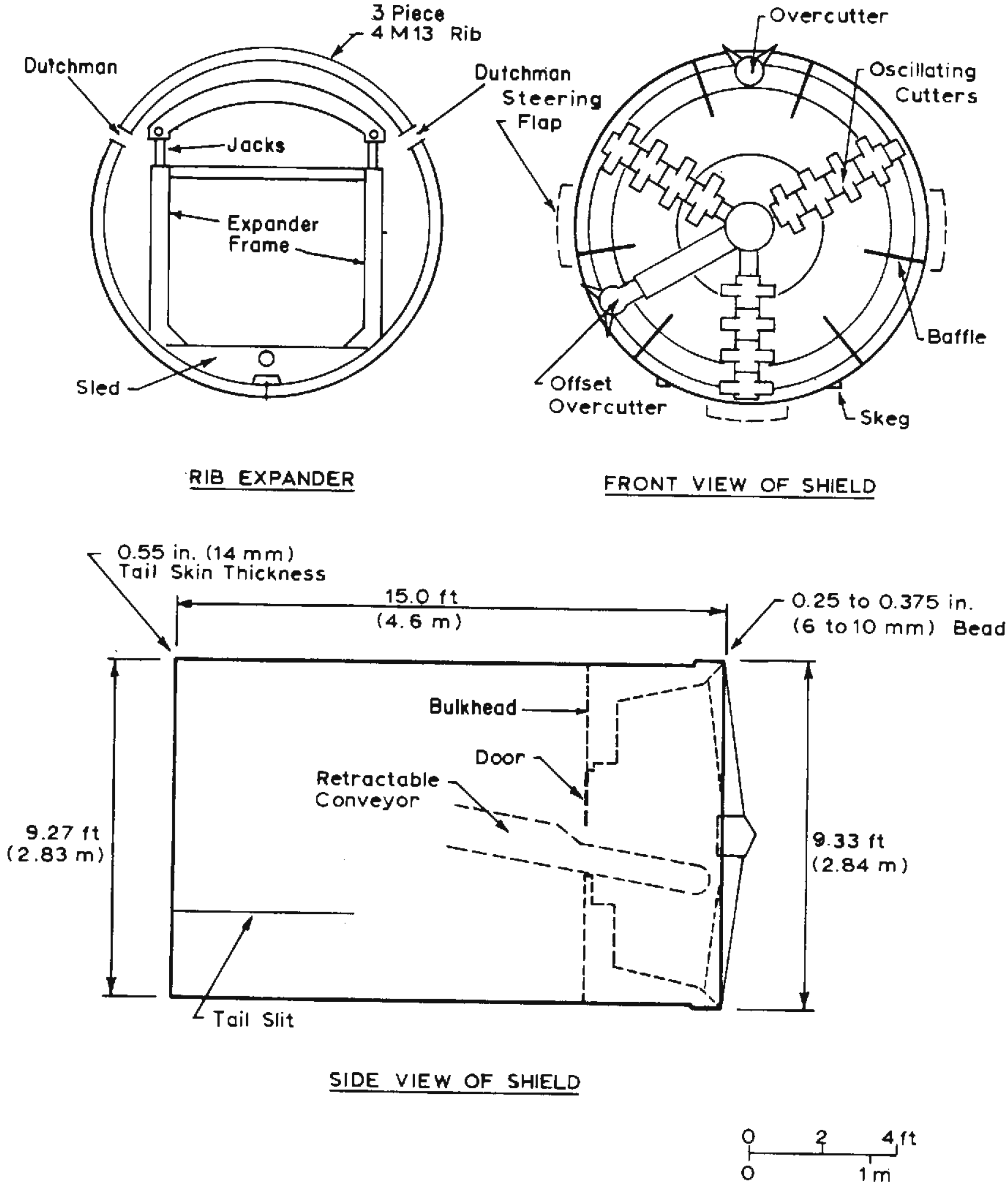


FIGURE C.5 ESLLIRP TUNNEL SHIELD DIMENSIONS bulkhead and the sheeting. Finally, the sheeting was pulled and the shield advanced from the shaft by pushing against a reaction frame. The rib expander and conveyor system were not assembled until the shield had advanced about 100 ft (30 m) from the north shaft and about 78 ft (24 m) from the middle shaft.

C.4 OBSERVATION AND MONITORING PROVISIONS

As shown by Fig. C.1, the tunnel passes close to an existing thermal power plant. This facility includes abandoned coal conveyor structures, gas, fuel oil and electric lines, fuel oil storage tanks and a switch yard. At one point the tunnel passes within about 75 ft (23 m) of the main smoke stacks (tunnel at about 45 ft (14 m) depth). It was estimated that some sensitive plant structures could tolerate no more than about 0.12 in. (3 mm) of settlement. For this reason, the specifications imposed limits on the amount of surface settlement resulting from tunnel excavation. Additionally, a settlement monitoring program was used to check compliance with the specifications and to provide information that would be helpful in changing tunnel excavation procedures to reduce any excessive settlements.

According to the specifications, ground surface settlements in the area of the power plant were not to exceed 1.5 in. (38 mm) within 10 ft (3 m) of the tunnel centerline, 0.5 in. (13 mm) within an area from 10 to 30 ft (3 to 9 m) from the centerline and 0.25 in. (6 mm) in an area 30 ft (9 m) outside the tunnel centerline, on either side of the tunnel. If the settlements exceeded these limits, tunnel excavation was to be halted and necessary changes made in excavation procedure to reduce surface settlement. Provisions were also included for temporary support of certain critical structures and utilities, monitoring of settlement and maintenance or restoration of the elevations of sensitive structures in the power plant area.

It should be noted that the power plant was taken out of service by the power company prior to tunnel construction. Therefore the need for limitations on surface settlement became somewhat less critical than considered in the design and specifications.

Surface settlement markers were established at 25 to 50 ft (7.5 to 15 m) intervals along the tunnel centerline. Six cross-sections, consisting of surface settlement markers in a line perpendicular to the tunnel centerline, were located as shown by Fig. C.1. Four of the cross-sections included deep settlement indicators. Table C.1 gives the location, depth to tunnel springline, and the distance between the tunnel crown and tip of the deep settlement indicator for each cross-section.

TABLE C.1

LOCATIONS OF OBSERVED CROSS-SECTIONS
AND DEEP SETTLEMENT INDICATORS

Station of Cross-Section	33+00	34+94	37+50	39+50	43+47	45+10
Depth to Tunnel Springline, z	34 ft (10.4 m)	42 ft (12.8 m)	45 ft (13.7 m)	43 ft (13.1 m)	34 ft (10.4 m)	32 ft (9.7 m)
Distance above tunnel crown to tip of deep settlement indicator, y	-	3.0 ft (0.9 m)	2.9 ft (0.9 m)	-	3.2 ft (1.0 m)	3.6 ft (1.1 m)

Surface settlement markers consisted of wooden stakes or steel rods driven into the ground. Deep settlement markers consisted of a 1-in. (25 mm) steel pipe within a 2-in. (50 mm) steel casing. The casing was placed about 6 ft (2 m) above the tunnel crown, using a hollow stem auger. The steel pipe was then driven to within about 3 ft (1 m) of the tunnel crown.

Surface settlement markers along the centerline were monitored by level surveying on each work day. Surface and deep settlements were monitored several times each day as the tunnel approached and passed each cross-section.

C.5 OBSERVATIONS AND INTERPRETATIONS

C.5.1 LOSS OF GROUND

The volume of lost ground, V_L, was estimated using Eq. 2.1. The displacements of the deep settlement indicators were correlated with the position of the tunnel face and construction events to evaluate the amount of lost ground. Potential sources of lost ground include loss through the tunnel face, loss over the shield, loss at the tail, and long term losses.

Figures C.6 through C.9 are plots of deep settlement versus the position of the front of the tunnel shield for the four cross-sections where deep settlement markers were installed. The ground surface settlement at the tunnel centerline is also shown for comparison.

The plots of deep settlement versus shield position indicate that most of the settlement and ground loss occurred over the shield and at the tail. Table C.2 summarizes the deep settlements and estimated volumes of lost ground observed at the four deep settlement indicators.

The amount of deep settlement observed to occur before the face was quite small. This suggests that mining procedures, as the tunnel approached each of the four deep settlement points, were adequate to minimize lost ground caused by sand runs in the face. To control face losses it was essential to fully support the face at all times and to guard against overmining. In several instances repair work or hand digging in the face to clear obstructions allowed excess sand to run into the tunnel, ultimately resulting in surface depressions. In the next section, a correlation between overmining and surface settlement will be discussed.

The amount of lost ground which occurred over the shield must be attributable to two main sources: 1) the presence of the overcutter bar

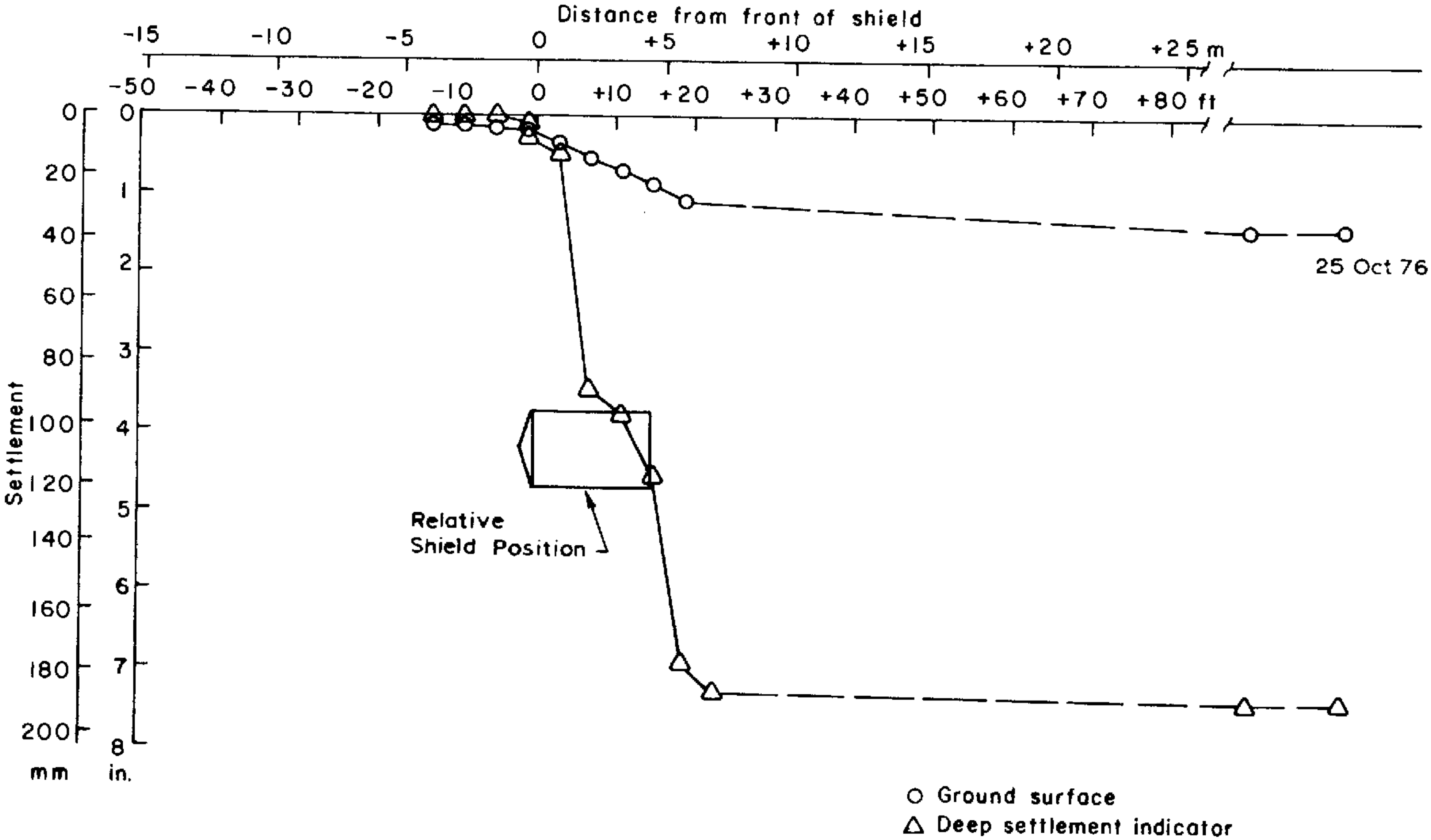


FIGURE C.6 GROUND SURFACE AND DEEP SETTLEMENTS, CROSS-SECTION 34+94

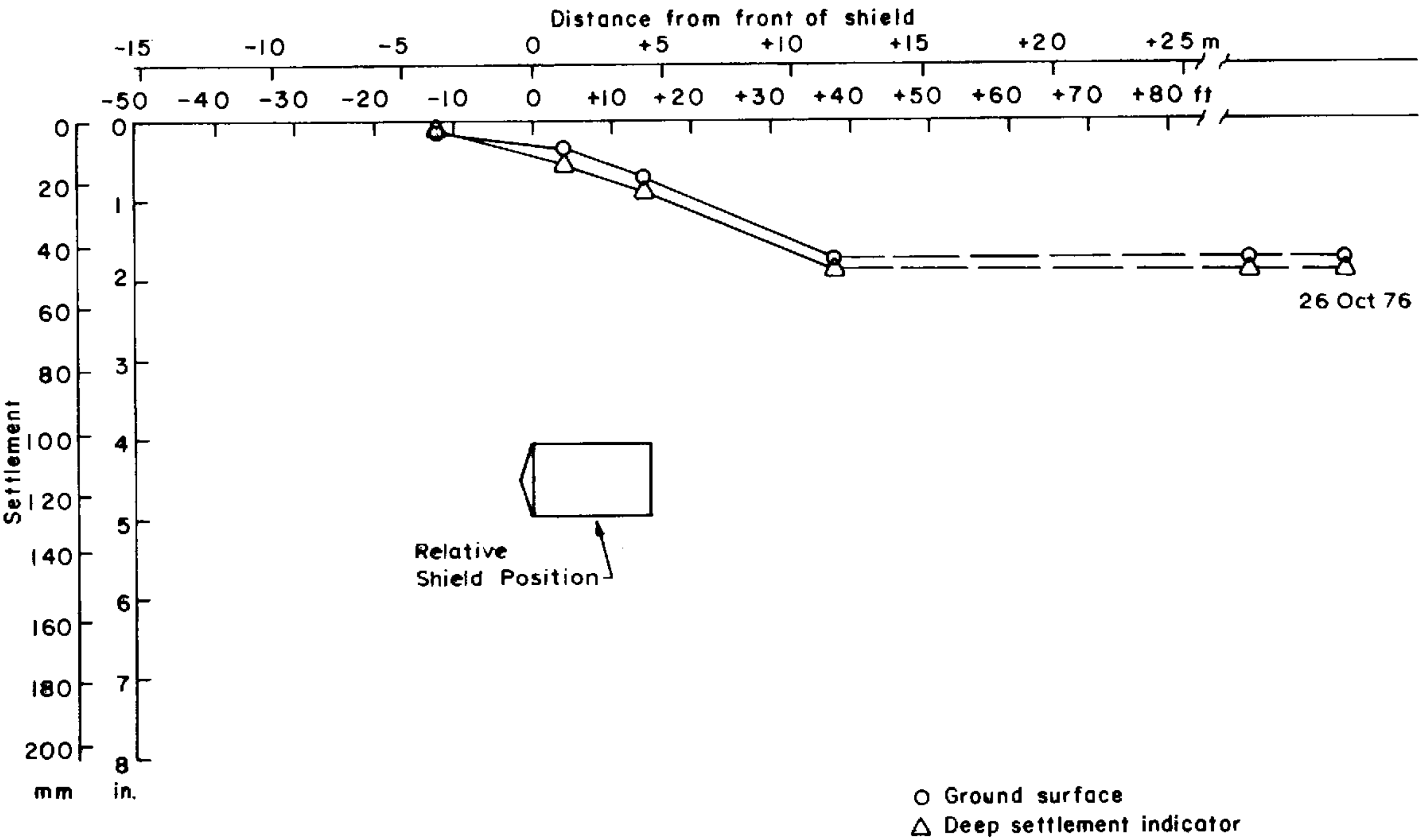


FIGURE C.7 GROUND SURFACE AND DEEP SETTLEMENTS, CROSS-SECTION 37+50

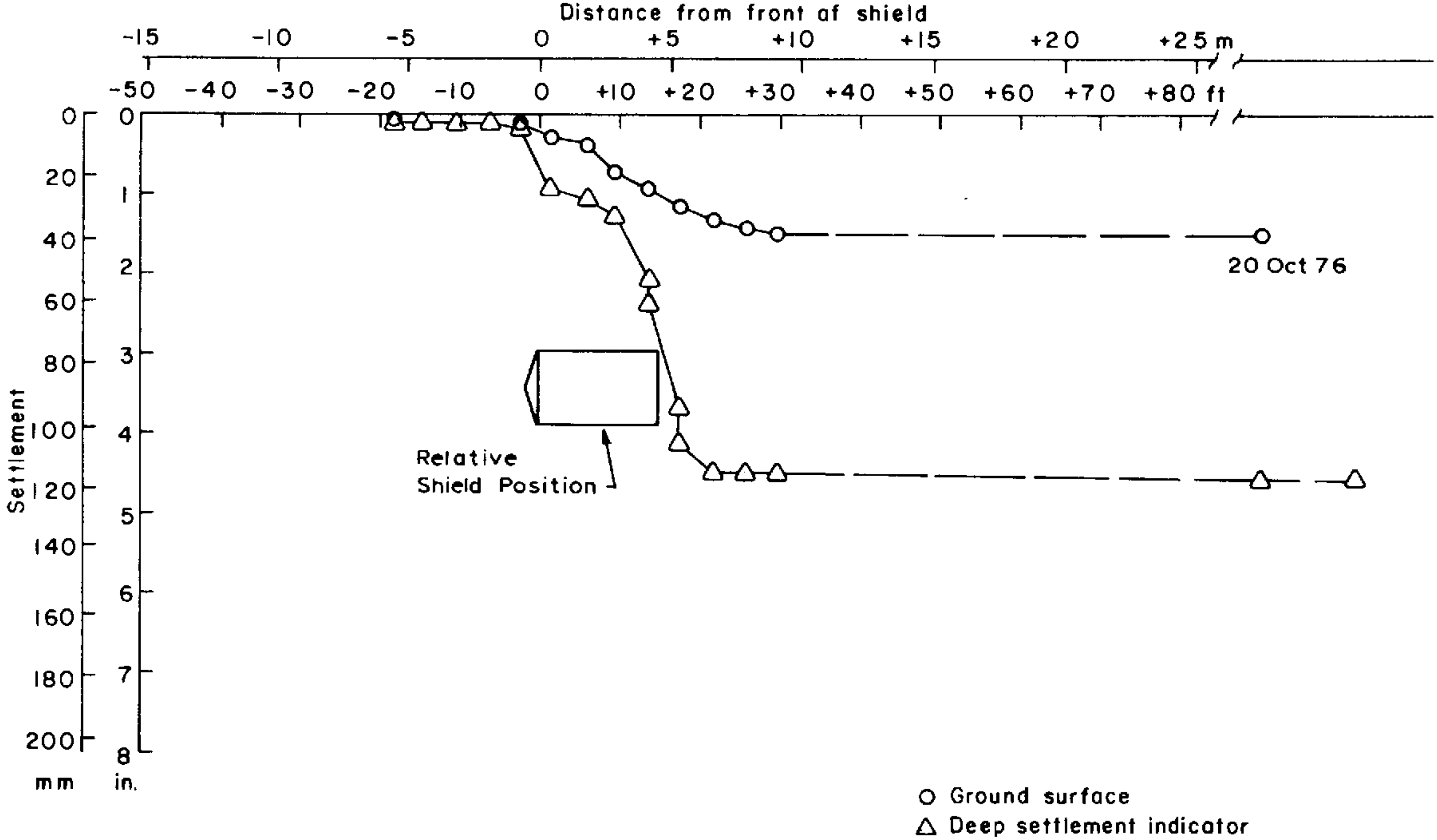


FIGURE C.8 GROUND SURFACE AND DEEP SETTLEMENTS, CROSS-SECTION 43+47

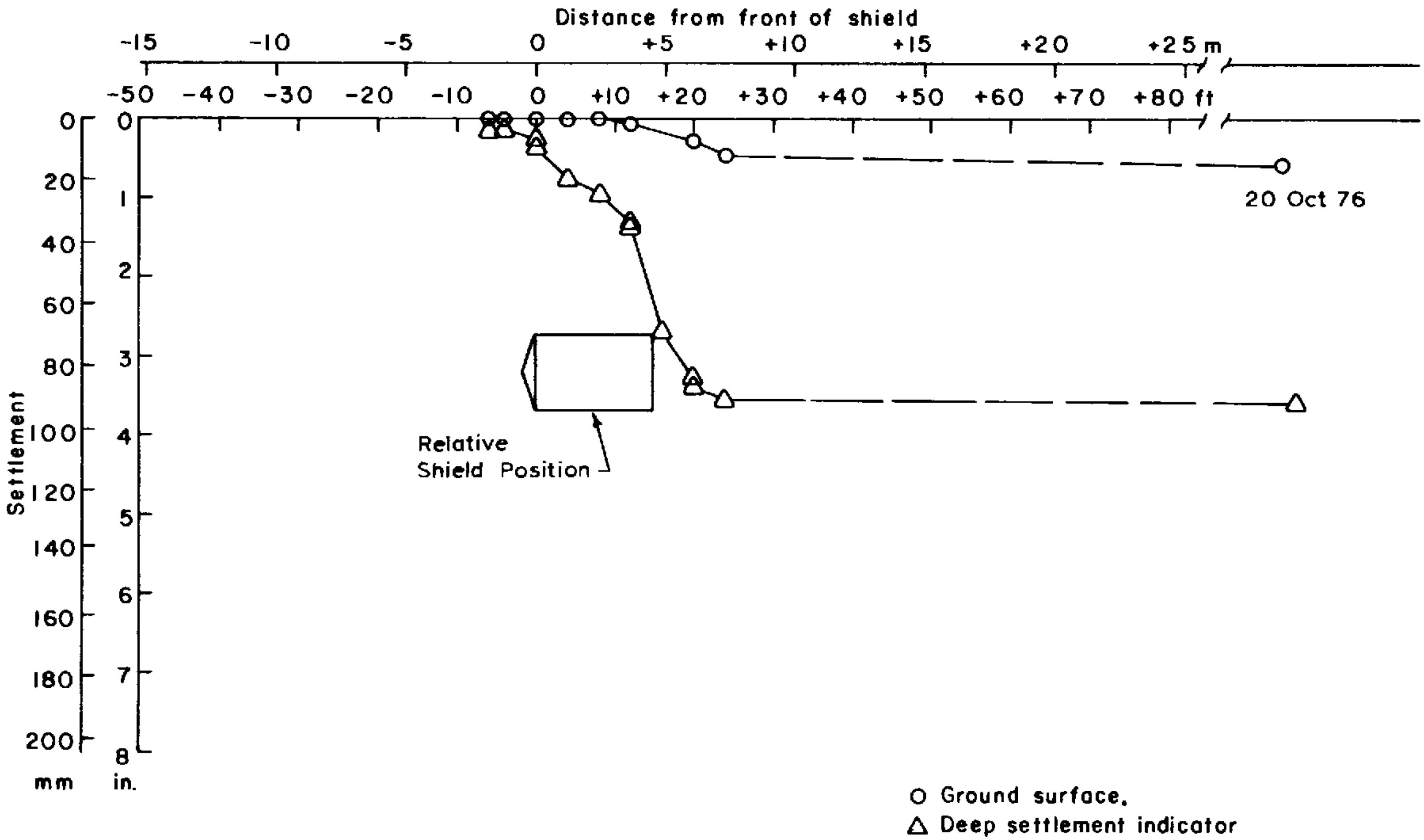


FIGURE C.9 GROUND SURFACE AND DEEP SETTLEMENTS, CROSS-SECTION 45+10

TABLE C.2

LOST GROUND AROUND ESLLIRP TUNNEL

Case 1	z/2R 2	Lost ground Before face 3	Over shield 4	At tail 5	Lining deflection and time dependent movements 6	Total 7
Cross-section 34+94	4.5	δ_v = 0.3 in.	4.3 in.	2.6 in.	0.1 in.	7.3 in.
		V_L = 0.4 ft^3/ft	5.5 ft^3/ft	3.4 ft^3/ft	0.1 ft^3/ft	9.4 ft^3/ft
		$\%V_L$ = 0.5%	8.0%	5.0%	0.2%	13.7%
Cross-section 37+50	4.8	δ_v = 0.4 in.	0.5 in.	0.9 in.	0.1 in.	1.9 in.
		V_L = 0.4 ft^3/ft	0.7 ft^3/ft	1.2 ft^3/ft	0.1 ft^3/ft	2.4 ft^3/ft
		$\%V_L$ = 0.7%	1.0%	1.7%	0.1%	3.5%
Cross-section 43+47	3.6	δ_v = 0.1 in.	2.2 in.	2.1 in.	0.1 in.	4.5 in.
		V_L = 0.2 ft^3/ft	2.9 ft^3/ft	2.8 ft^3/ft	0.1 ft^3/ft	6.0 ft^3/ft
		$\%V_L$ = 0.3%	4.3%	4.0%	0.1%	8.7%
Cross-section 45+10	3.4	δ_v = 0.4 in.	2.0 in.	1.1 in.	0.1 in.	3.6 in.
		V_L = 0.5 ft^3/ft	2.8 ft^3/ft	1.6 ft^3/ft	0.1 ft^3/ft	5.0 ft^3/ft
		$\%V_L$ = 0.7%	4.1%	2.3%	0.2%	7.3%

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft^3/ft = 0.0929 m^3/m at the cutting edge of the shield and 2) excess pitch or yaw of the shield. The overcutter bar and built-up welding bead at the cutting edge (see Fig. C.5) cause the shield to excavate a volume slightly larger than the tail diameter of the shield. It is likely that the resulting annular void, estimated to be 1.5 to 1.8 ft^3/ft (0.14 to 0.17 m^3/m) or 2.2 to 2.6 percent of the theoretical tunnel volume was immediately filled by sand. Additional overexcavation occurs when the shield plows or yaws during a shove. For each one percent of excess pitch above tunnel grade, the shield would overexcavate about 1.1 ft^3/ft (0.10 m^3/m), a volume of lost ground equal to about 1.6 % of the theoretical tunnel volume. Also, some of the lost ground attributed to losses over the shield may actually have occurred due to overexcavation at the face. As shown by Table C.2, the estimated volumes of lost ground over the shield are quite variable. There is insufficient data to obtain a better estimate of the actual quantity of lost ground or to account for this volume by more detailed examination of the above potential sources of ground loss.

At the tail, there is a potential for loss of ground equal to the difference in volume between the outside of the shield tail and the unexpanded temporary lining. Before expansion, the ribs have an outside diameter of about 9.0 ft (2.7 m). The potential volume loss then would be about 5.2 ft^3/ft (0.48 m^3/m) or about 8.4 percent of the theoretical tunnel volume. The observed volumes of ground lost at the tail, shown in Table C.2, are significantly less than the above amount. This suggests that the expansion of the ribs as they cleared the rear of the shield was effective in limiting ground loss at the tail.

Deep settlements observed after the shield passed each test section were very small. Therefore, long term losses due to soil volume change or lining deflection were apparently very small or negligible.

C.5.2 GROUND SURFACE SETTLEMENTS

Figure C.10 shows the final ground surface settlements observed along the tunnel centerline. Also, shown is a plot of volumes excavated per shove (in terms of muck cars per shove) made using observations

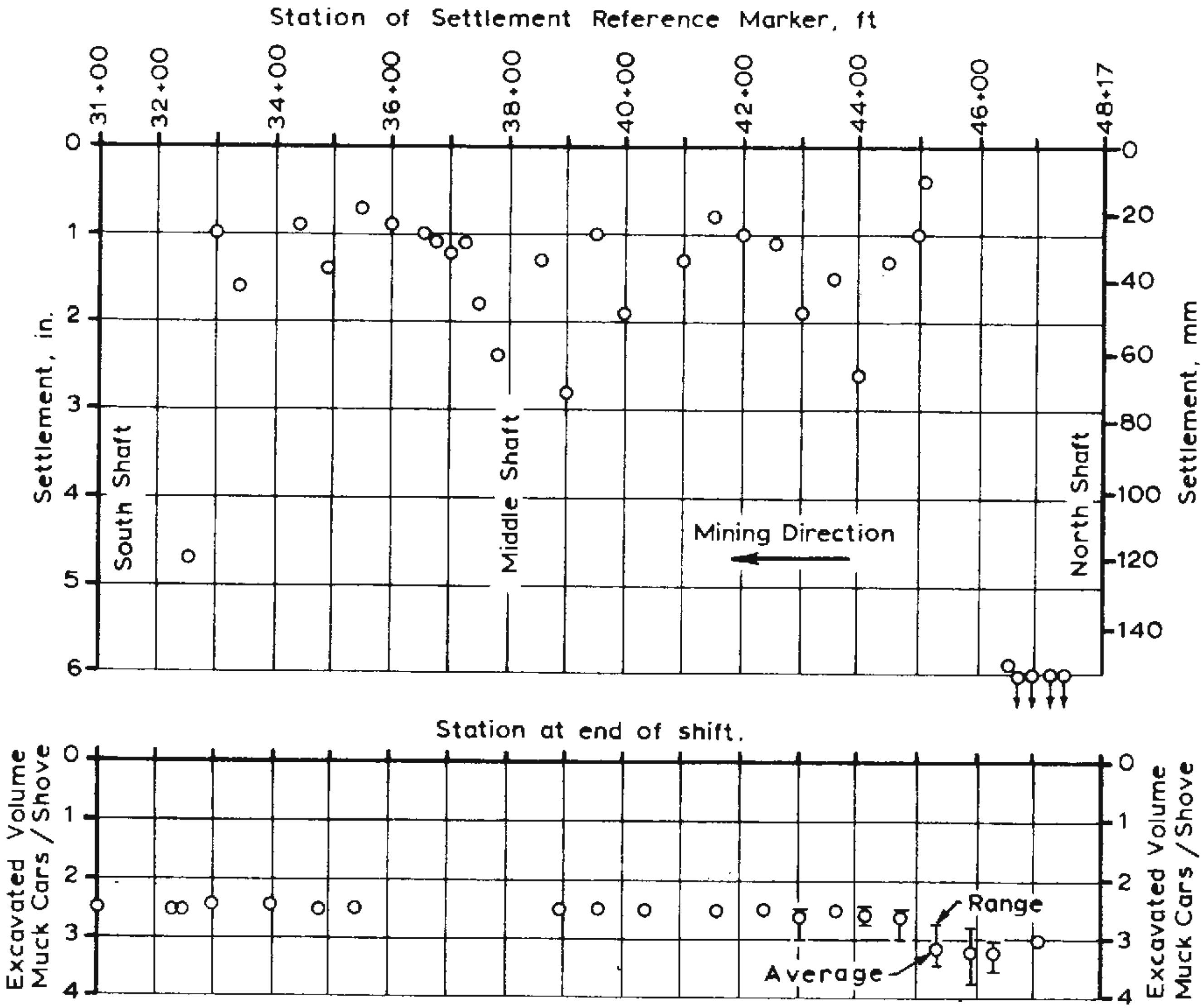


FIGURE C.10 GROUND SURFACE SETTLEMENTS ALONG CENTERLINE OF TUNNEL AND OBSERVED EXCAVATION VOLUMES noted in the contractor and Resident Engineer's Daily Reports. Each muck car holds about 4 yd^3 (3 m^3) of excavated material. It is interesting to compare the final surface settlements and the volume excavated per shove. Large surface settlements occurred at the north end of the project. These settlements resulted in part from the presence of relatively loose backfill around the shaft and the fact that the temporary lining was not expanded until the tunnel had been advanced far enough to install the rib expander and conveyor systems. However, the excavated volume data suggests that the large settlements may have resulted from overexcavation. It can be seen from the plot that as the tunnel was mined from north to south and experience was gained in operation of the tunnel boring machine, the amount of material excavated per shove decreased from 3 muck cars per shove and became nearly constant at a rate of about 2.5 muck cars or about 10 yd^3 (7.6 m^3) per shove. Correspondingly, ground surface settlements were greatly reduced compared to those which occurred at the north end, and with a few exceptions settlements were less than 1.5 to 2 in. (38 mm to 51 mm). The theoretical volume excavated for each 4 ft (1.2 m) advance is about 10.3 yd^3 (7.8 m^3). Therefore, it appears that when mining at the rate of 2.5 muck cars per shove, excessive settlements caused by overmining were avoided.

It may also be observed from the settlement data on Fig. C.10 that settlements near the middle shaft were much less than those near the north shaft. The smaller settlements at the middle shaft can be attributed to several factors: 1) The middle shaft was constructed as a hand-dug braced excavation rather than by backfilling around a bracing system constructed in an open cut, 2) The ground around the middle shaft was stabilized with grout and 3) At the stage of construction of the middle shaft the miners were more experienced.

Coming out of the middle shaft, the tunnel went off-line (too high). Lost ground resulting from maneuvering the shield and subsequent remining to lower the invert probably contributed to the surface settlements.

Figures C.11 through C.16 show the final surface settlements observed along each of the cross-sections. Table C.3 lists the calculated

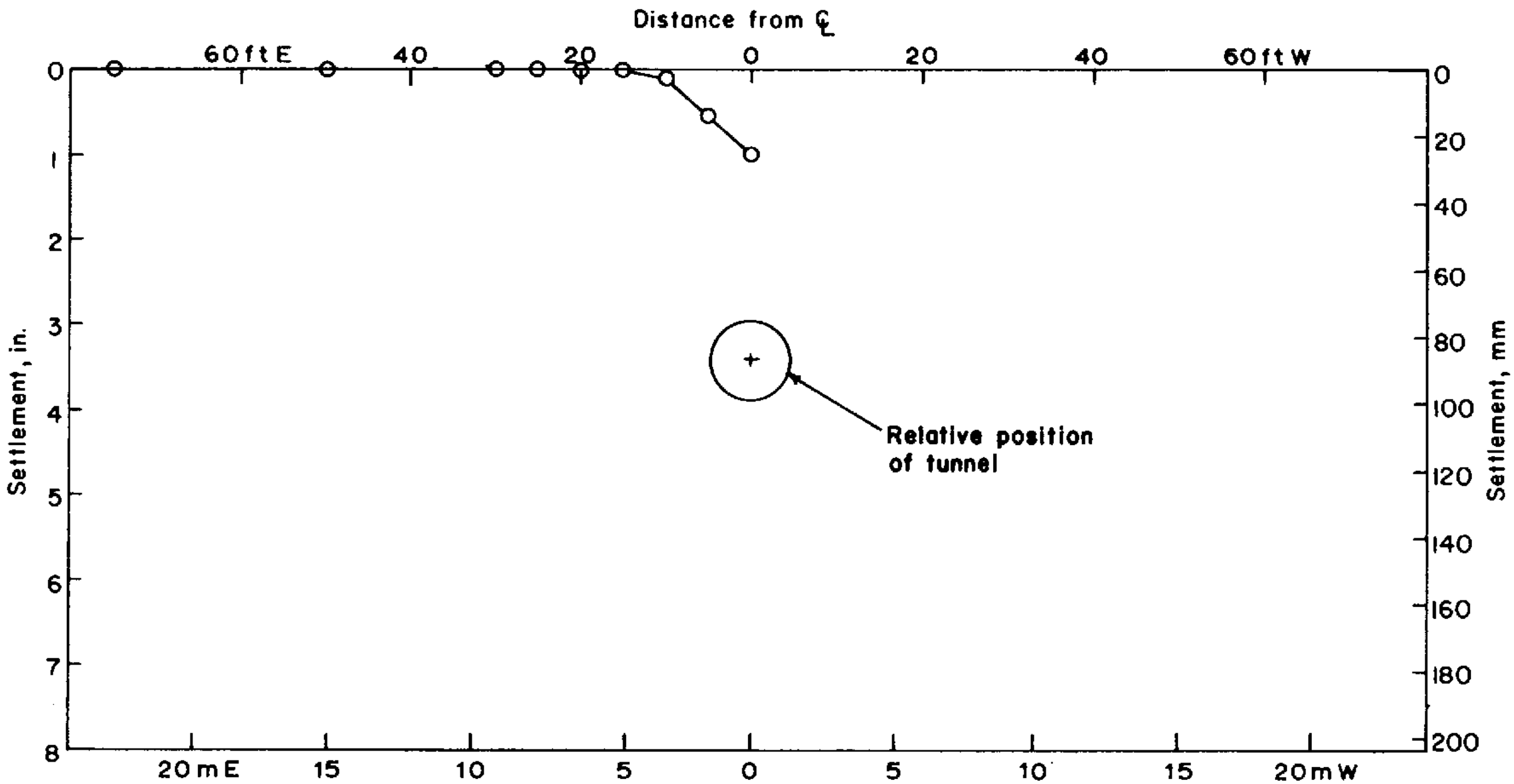


FIGURE C.11 GROUND SURFACE SETTLEMENT, CROSS-SECTION 33+00

C-22

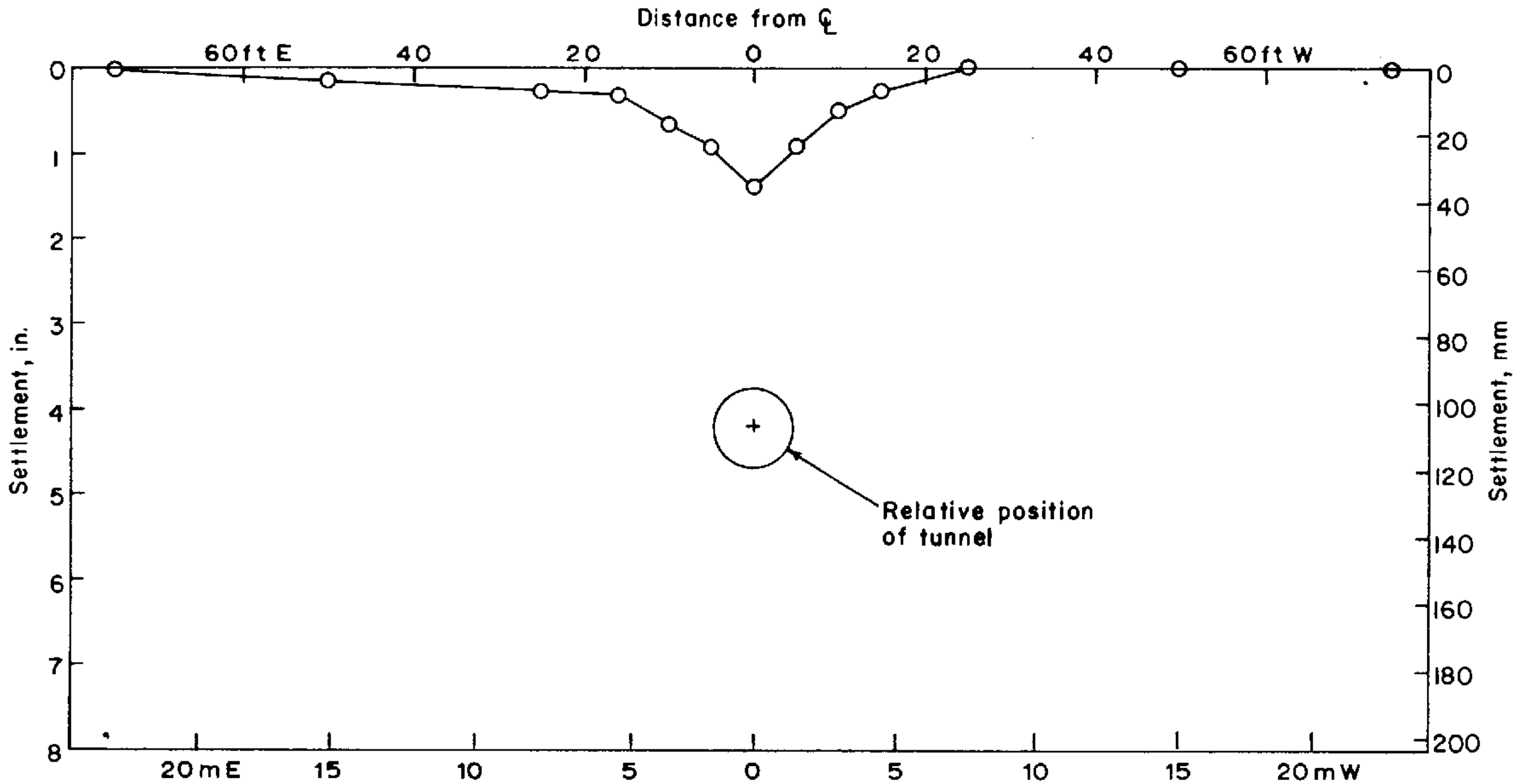


FIGURE C.12 GROUND SURFACE SETTLEMENT, CROSS-SECTION 34+94

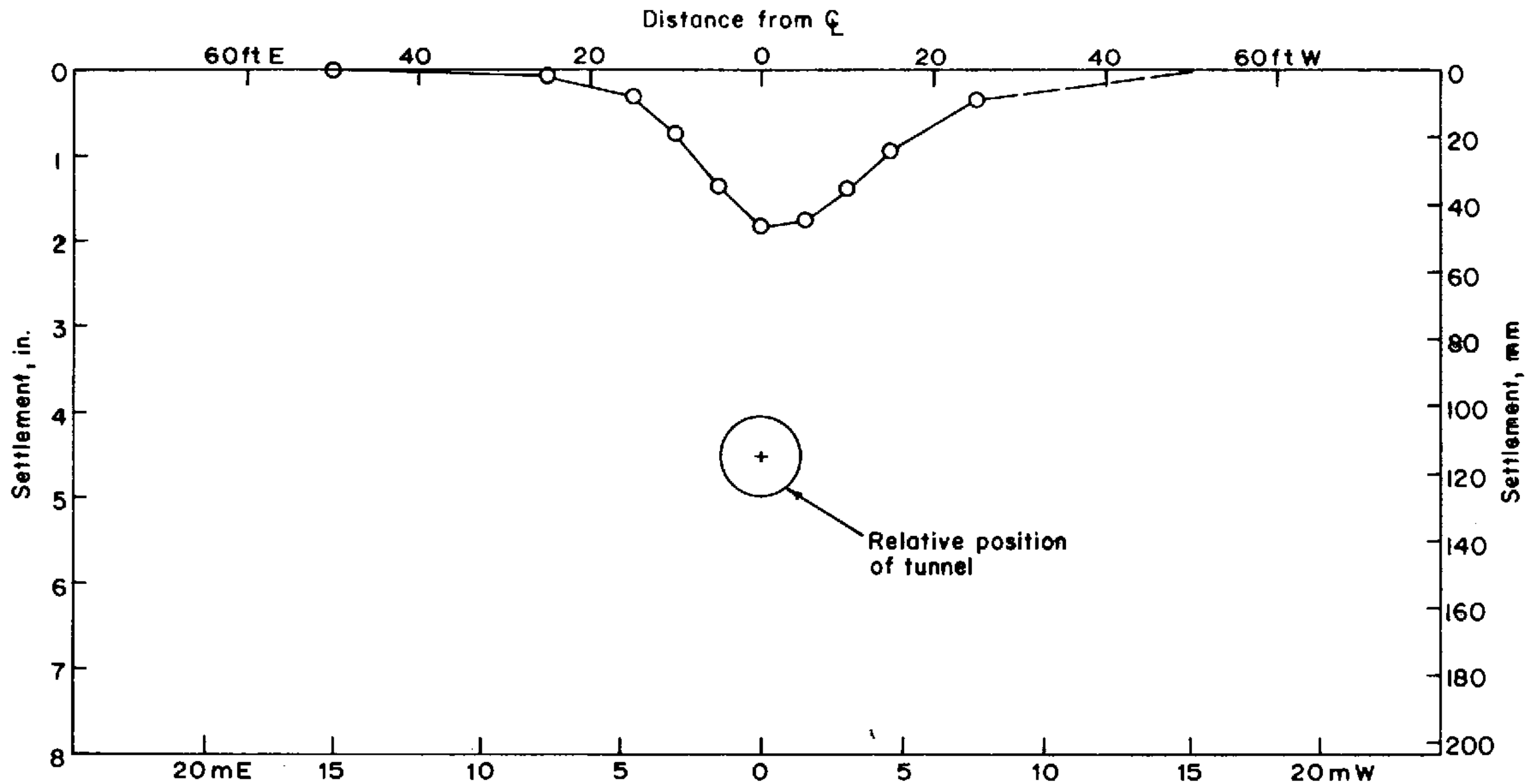


FIGURE C.13 GROUND SURFACE SETTLEMENT, CROSS-SECTION 37+50

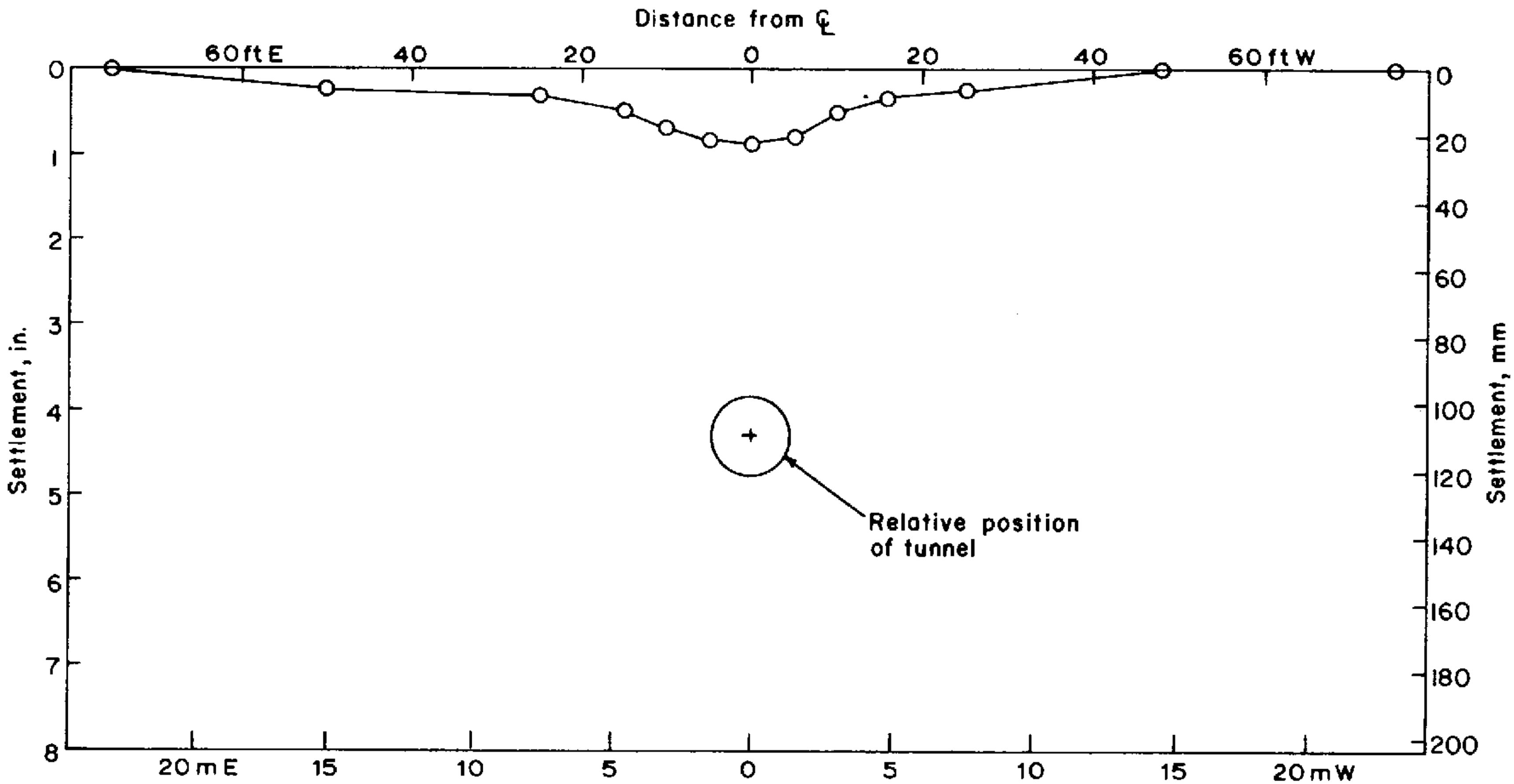


FIGURE C.14 GROUND SURFACE SETTLEMENT, CROSS-SECTION 39+50

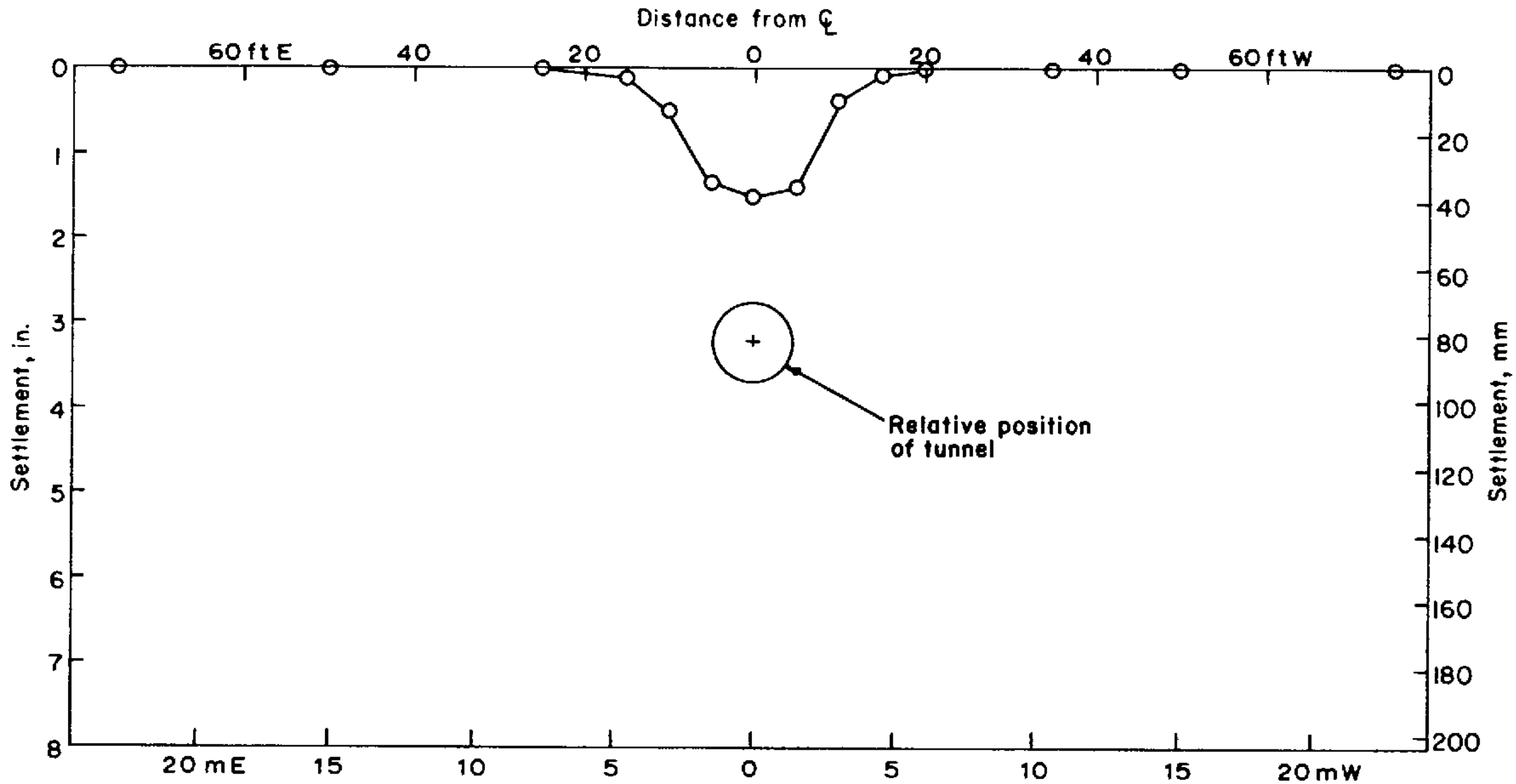


FIGURE C.15 GROUND SURFACE SETTLEMENT, CROSS-SECTION 43+47

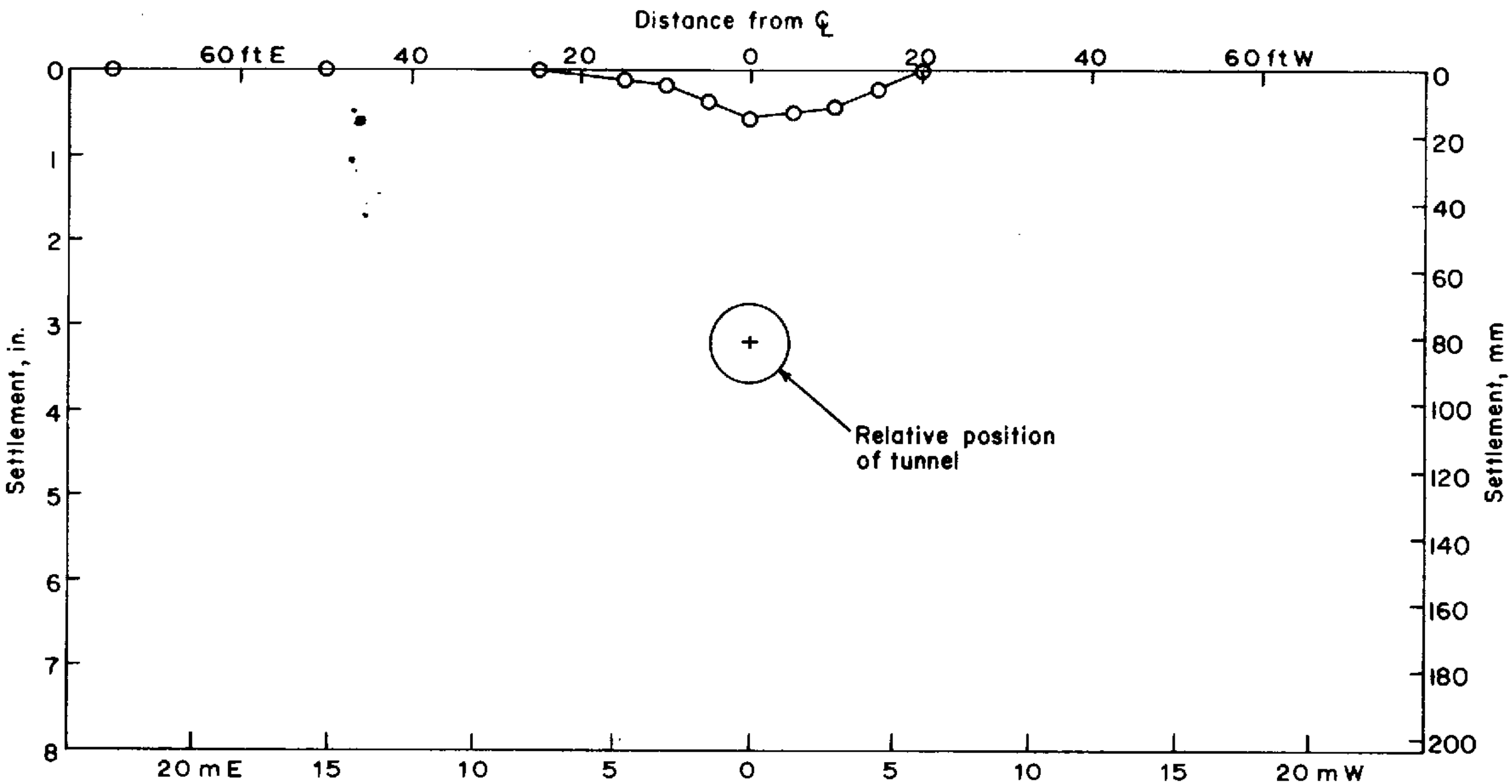


FIGURE C.16 GROUND SURFACE SETTLEMENT, CROSS-SECTION 45+10

TABLE C.3

VOLUMES AND DISPLACEMENTS - ESLLIRP TUNNEL

Case	$z/2R$	Vertical displacement δ_{max}, in.		Volume of movement ft^3/ft		Trough width		Slope of surface settlement trough	
		surface	deep	surface V_S	tunnel V_L	i/R	w, ft	cross-section	longitudinal section
1	2	3	4	5	6	7	8	9	10
Cross-section 33+00	3.6	1.0	—	1.0 1.5%	—	1.0 (β = 11°)	11	1:130	—
Cross-section 34+94	4.5	1.4	7.3	2.9 4.2%	9.4 13.7%	1.3 - 1.4 (β = 26°)	25	1:210	1:375
Cross-section 37+50	4.8	1.8	1.9	4.3 6.3%	2.4 3.5%	1.5 - 2.9 (β = 28°)	29	1:190	1:375
Cross-section 39+50	4.6	0.9	—	3.5 5.1%	—	1.4 - 1.6 (β = 35° - 44°)	35 - 47	1:470	—
Cross-section 43+47	3.6	1.5	4.5	2.2 3.2%	6.0 8.7%	2.0 - 2.9 (β = 21°)	18	1:140	1:275
Cross-section 45+10	3.4	0.6	3.6	1.0 1.5%	5.0 7.3%	1.5 - 2.5 (β = 27°)	21	1:420	1:650

Conversion factors: 1.0 in. = 25.4 mm, 1.0 ft = .03048 m, 1.0 ft^3/ft = 0.0929 m^3/m

volume of surface settlements and geometric parameters for each cross-section.

The maximum settlement, δ_{max}, occurred directly over the tunnel.

C.5.3 COMPARISON OF V_S AND V_L

Estimated values of surface settlement volume, V_S, and volume of lost ground, V_L, are compared on Fig. 4.1. The solid line represents a condition where no soil volume change occurs, and V_S equals V_L. As shown the ESLLIRP tunnel data plots well above this line (except for test section at Sta. 37+50, point D), indicating significant volume expansion during tunnel excavation. At Sta. 37+50, the fact that the surface settlement and deep settlement measurements were nearly equal (refer to Fig. C.7) suggests that the settlement rod was probably jammed inside the casing. The rod would then move with the casing rather than indicate the deep settlement at the lower end of the rod.

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