Report No. CDOT-DTD-R-2001-5

DESIGN AND CONSTRUCTION GUIDELINES FOR MSE WALLS WITH INDEPENDENT FULL-HEIGHT FACING PANELS

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August 2001

COLORADO DEPARTMENT OF TRANSPORTATION RESEARCH BRANCH

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16. Abstract In 1996, the Colorado DOT completed the construction of a unique mechanically stabilized earth (MSE) wall with an independent full-height facing (IFF) for the ramp connecting northbound Interstate-25 to Interstate-70 in Denver, Colorado. The new MSE/IFF wall has four major components: 1) a self-stable welded wire fabric (WWF) reinforced soil mass, 2) full-height concrete facing panels not attached to the soil reinforcements (i.e., independent) that are allowed to tilt around their base, 3) flexible face anchors to provide for attachment of facing panels to the reinforced soil mass and accommodate movements of the wall system, and 4) a trench with flowfill to brace the panels during construction only (before the face anchors are placed). Since this MSE wall system is the first of its kind, it was considered experimental and a comprehensive instrumentation and monitoring program was performed. The main objective of this study was to upgrade the I-25/I-70 MSE/IFF wall for future standard use of this wall system by identifying modifications and additions to the design and construction of the I-25/I-70 MSE/IFF wall that would improve performance and save money and time. This report provides insight into material, construction, construction problems and corrective actions, monitoring, performance and design assessment of the I-25/I-70 MSE/IFF wall. The wall system performed as intended in the design. The flexibility of the MSE wall system smoothly accommodated the movements of the wall system, especially those induced by heavy compaction close to the facing, and allowed for the mobilization of tensile resistance in the WWF reinforcements, thus taking most of the lateral load off the facing panels. The average lateral earth pressure measured on the facing was a low value of 32 psf. After five years in service, the structure performance has been excellent with no signs of distress and the facing remained properly aligned.

Implementation: This report provides complete details and basis for future standard use of an MSE/IFF wall with facing either weakly braced or unbraced during construction. Recommendations for depths of embedment and setbacks of facing panels and to reduce wall deformations and increase stability of the facing anchors and WWF reinforcements were furnished. The details for the proposed standard MSE/IFF wall system are summarized in four parts: 1) description (layout and materials), 2) selection and unique features, 3) design guidelines, and 4) construction procedures.

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CONVERSION TABLE

U. S. Customary System to SI to U. S. Customary System

(multipliers are approximate)

Multiply (symbol)	by	To Get (symbol)	Multiply	by	To Get
		LENGTH			
Inches (in)	25.4	millimeters (mm)	mm	0.039	in
Feet (ft)	0.305	meters (m)	m	3.28	ft
yards (yd)	10.914	meters (m)	m	1.09	yd
miles (mi)	1.61	kilometers (km)	m	0.621	mi
		AREA			
square inches (in^2)	645.2	square millimeters (mm^2)	mm^2	0.0016	in^2
square feet (ft^2)	0.093	square meters (m^2)	m^2	10.764	ft^2
square yards (yd^2)	0.836	square meters (m^2)	m^2	1.195	yd^2
acres (ac)	0.405	hectares (ha)	ha	2.47	ac
square miles (mi^2)	2.59	square kilometers (km^2)	km^2	0.386	mi^2
		VOLUME			
fluid ounces (fl oz)	29.57	milliliters (ml)	ml	0.034	fl oz
gallons (gal)	3.785	liters (l)	l	0.264	gal
cubic feet (ft^3)	0.028	cubic meters (m^3)	m^3	35.71	ft^3
cubic yards (yd^3)	0.765	cubic meters (m^3)	m^3	1.307	yd^3
		MASS			
ounces (oz)	28.35	grams (g)	g	0.035	oz
pounds (lb)	0.454	kilograms (kg)	kg	2.202	lb
short tons (T)	0.907	megagrams (Mg)	Mg	1.103	T
		TEMPERATURE (EXACT)			
Farenheit (°F)	5(F-32)/9 (F-32)/1.8	Celcius (° C)	° C	1.8C+32	° F
		ILLUMINATION			
foot candles (fc)	10.76	lux (lx)	lx	0.0929	fc
foot-Lamberts (fl)	3.426	candela/m (cd/m)	cd/m	0.2919	fl
		FORCE AND PRESSURE OR STRESS			
poundforce (lbf)	4.45	newtons (N)	N	.225	lbf
poundforce (psi)	6.89	kilopascals (kPa)	kPa	.0145	psi

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by

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The authors would like to express their sincere gratitude to Mr. Robert Blanks and Mr. Dave Price who described the I-25/I-70 MSE wall system and the instrumentation program, installed all the instruments, collected all the performance data, and provided history of fill placement as presented in various parts of this report. Special thanks goes to Christina C.W. Ma and Jonathan Wu from the University of Colorado at Denver who documented some of the features of the I-25/I-70 MSE wall given in Chapter 1, documented the description and layout of all instruments as given in Chapter 3, and organized the collected surveying and inclinometer data presented in Figures 4.1 to 4.6 of this report. Without their professional and generous input, this study would not have come to fruitful conclusions.

Executive Summary

In 1996, the Colorado DOT completed the construction of a unique MSE wall with an independent full-height facing (IFF) for the ramp connecting northbound Interstate-25 to Interstate-70 in Denver, Colorado. The new MSE/IFF wall system has four major components: 1) a self-stable welded wire fabric (WWF) reinforced soil mass, 2) full-height concrete facing panels not attached to the soil reinforcements (i.e., independent) that are allowed to tilt about their base, 3) flexible facing anchors to provide for attachment of facing panels to the reinforced soil mass and accommodate movements of the wall system, and 4) a trench with flowfill to brace the panels during construction before the facing anchors are placed. Because the facing and MSE mass are independent, the design of the MSE mass and wall facing are effectively divorced. To the authors' knowledge, the design and construction of the I-25/I-70 MSE/IFF wall is the first of its kind in conventional highway practice. Therefore, the I-25/I-70 MSE/IFF wall system was considered experimental and a comprehensive instrumentation and monitoring program was incorporated into the construction operations. Two sections were instrumented with inclinometers and survey points to measure the facing movement and rotation, and with strain gauges to measure the lateral earth forces and moments on the facing panels and the tensile forces mobilized in the WWF reinforcements.

The main objective of this study was to upgrade the I-25/I-70 MSE/IFF wall to a standard wall system by identifying modifications and additions to the design and construction of the I-25/I-70 MSE/IFF wall that would improve performance and save money and time. This was achieved through the completion of the following tasks:

- Documentation and assessment of the I-25/I-70 MSE wall details, materials description and strength, construction procedure, problems encountered during construction, and corrective action implemented to alleviate these problems.
- Provide recommendations for wall setbacks from measurements of wall facing movements, and provide recommendations for depth of embedment for unbraced facing during construction.
- Assessments of the CDOT design for facing panel and anchors based on measurements of lateral earth loads and moments on the facing panels and pullout loads in the anchors.

- Assessments of the CDOT design for the reinforced soil mass from measurement of reinforcement tensile forces.

The I-25/I-70 MSE/IFF wall system functioned to a large degree almost as planned in the design. The flexibility of the wall system accommodated the deformation of the reinforcement soil mass, especially those induced by heavy compaction, and allowed for mobilization of the tensile resistance in the reinforcements, especially in the upper WWF layers, thus taking most of the lateral load off the facing panels. The average earth pressure on the facing, measured from 19 gauges, was 32 psf. The facing panels are at least four times stronger in bending than needed for lateral earth pressures. After almost five years in service, the structure performance has been excellent with no signs of structural distress and the facing remained properly aligned.

Implementation Statement

Chapter 8 of this report provides complete details and the basis for future standard use of an MSE/IFF wall system that would improve performance and save money and time in relation to other conventional and MSE wall systems. Three interrelated problems with the I-25/I-70 MSE/IFF wall were identified: larger than anticipated wall deformations, larger reinforcement tensile loads, and anchors pullout capacity smaller or close to pullout loads, all occurring in the upper zone of the wall. Measures to eliminate these problems in the proposed MSE/IFF wall system were furnished. CDOT bridge and geotechnical engineers should consider this type of wall system as a viable and standard alternative in the selection process of retaining walls. The details of the proposed MSE/IFF wall system are summarized in four parts:

- Layout and materials. Recommendations for MSE/IFF wall system with facing either weakly braced or unbraced during construction are different in terms of depths of embedment and setbacks, but similar for all other aspects. The new guidelines include changes to the vertical spacing of the WWF reinforcement and face anchors and new material specifications for the face anchors.
- Selection and unique features. The proposed MSE/IFF wall system is adequate for all field conditions found suitable for the use of MSE walls. The features and advantages of the MSE/IFF wall in relation to other conventional and MSE wall systems are described.

- Design guidelines. The proposed MSE/IFF wall system is fixed in term of materials and layout to ensure the internal stability of the facing panels, facing anchors, and reinforcements. The designer still needs to ensure the external stability of the wall under specific field conditions (e.g., retaining fill and foundation soil). Until track records for the movements of this kind of walls are established, the first six panels should be monitored closely.
- Construction procedure. This was developed based on the lessons learned during construction of I-25/I-70 MSE/IFF wall, including adjustment features for the facing panels that resulted in properly aligned facing and good performance.

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1.0 INTRODUCTION

1.1. Background

The technology of mechanically stabilized earth (MSE) has been used extensively in highway retaining structures to support the self-weight of the backfill soil, the roadway structure, and the traffic loads. The increasing use and acceptance of soil reinforcement has been triggered by a number of factors, including cost savings, aesthetics, simple and fast construction techniques, good seismic performance, and the ability to tolerate large total and differential settlement without structural distress. The aesthetics of a wall are often very important. Block facings and stacked panel facings are attractive, but some projects may need walls with monolithic fronts not broken by horizontal joints. In such cases, full-height facing units are required. In addition, the use of full-height facing significantly reduces construction time and may require less maintenance work than MSE wall with segmental facing (from comments of CDOT engineers). Full-height facing used in conventional MSE walls are attached to the soil reinforcement layers. This attachment can result in significant stresses in the panel, leading to the design of heavy panels and significant costs to the project. High stresses acting on full-height facing panels can be avoided if the facing is not attached to the soil reinforcement and is allowed to move to accommodate movements of the wall system. This is the concept of MSE walls with independent full-height facing (MSE/IFF, Hearn and Myers, 1993, and Hearn et. al., 1995). The construction method of this MSE wall system has been tested with full-scale loading tests conducted at the University of Colorado at Denver (Wu and Christiana, 1999; Hearn and Myers, 1993). The tests indicated that the reinforced soil mass exerted comparatively small lateral earth pressure on the full-height facing panel.

Hearn et. al. (1995) and Hearn and Myers (1993) described the design, materials, and construction of MSE walls with independent full-height facing (IFF). As shown in Figure 1.1, the IFF MSE wall system has three major components:

- A stable reinforced fill, typically with a wrapped reinforcement at the front. The system is compatible with many types of reinforcements, including geotextiles, geogrids, and steel meshes.

- Independent full-height facing panels allowed to move (mostly by tilting around their bases). Panels provide a forming surface and permanent facing for MSE walls, but are independent of the reinforced fill (i.e., not attached to reinforcement).
- Flexible facing anchors to attach panels to the stable reinforced soil mass and to allow movement of panels at moderate earth pressure. The facing anchors can take several forms as shown in Figure 1.1b. Yielding anchors impose an upper bound on the magnitude of lateral earth pressure acting on panels. Once this yield load is reached, the facing panels will tilt and will not accept higher pressures.

1.2 The I-25/I-70 MSE/IFF Wall

In 1996 the Colorado Department of Transportation (CDOT) completed the construction of a unique MSE wall with an independent full-height facing (IFF) for the ramp connecting northbound Interstate-25 to Interstate-70 in Denver, Colorado. Figures 1.2 and 1.3 show a picture and location of the I-25/I-70 MSE/IFF wall. Since the new retaining wall was the extension of an existing cantilever reinforced concrete retaining wall, it was deemed necessary for the new retaining wall to bear the same monolithic front as the existing wall which had full-height grooved concrete facing. Mr. Mike McMullen of CDOT, who was one of the inventors of the MSE/IFF wall, designed this wall. This wall system was designed to be very flexible to mobilize large tensile resistance in the reinforcements, thus allowing the reinforced fill to take most of the lateral earth load, and minimizing the lateral earth loads on the panel facings. The smaller earth pressure on the facing reduced the required facing strength capacity, and therefore significantly reduced the overall costs of this project when compared to the use of a CIP cantilever wall. The cost per square foot of the facing was estimated to be around $20, which is about ½ of the cost of a reinforced concrete cantilever wall (Christiana and Wu, 1999). In addition, this wall requires little or no over-excavation in front and beneath the wall. This feature allowed traffic to remain open or undisrupted along a busy section of I-25 throughout construction activities. It also alleviated the need to deal with excavation and disposal of possibly contaminated I-25 subsoil. The distinct features of MSE/IFF wall system presented before and the excellent past performance of full-scale tests (Hearn and Myers, 1995) convinced Mr. Mike McMullen to select this wall system instead of both cantilever CIP and conventional MSE retaining walls with full-height facing. It was expected that the foundation soil would support the new wall because it

safely supported I-25 and the nearby retaining walls. Design calculations indicated an adequate margin of safety for internal and external stability of the I-25/I-70 MSE/IFF wall.

1.3 Study Objectives and Work Plan

To the authors' knowledge, the design and construction of the I-25/I-70 MSE/IFF is the first of its kind in conventional highway practice. The performance of this unique MSE system had not been tested under actual service conditions to merit acceptance without reservation in normal highway construction. Therefore, it was important to this project and to future applications that CDOT measure the performance of this first production wall system. The I-25/I-70 MSE wall system was considered experimental and a comprehensive instrumentation and monitoring program was incorporated into the construction operations. Instrumentation was installed at two locations, stations 3116 and 3119. Each location was instrumented with inclinometers and survey points to measure facing's movement and rotation, and with strain gauges to measure lateral earth forces on the facing panels and tensile forces mobilized in the WWF reinforcements. The main objective of this study was to upgrade the I-25/I-70 IFF/MSE wall to a standard wall system by identifying modifications and additions to the design and construction of the I-25/I-70 MSE/IFF wall that would improve performance and save money and time. To achieve these objectives, this study attempted to:

- Document and assess the I-25/I-70 MSE/IFF wall details, materials description and strength, construction procedure, problems encountered during construction, and corrective action implemented to alleviate these problems (Chapter 2).
- Provide recommendations for wall setbacks from measurements of wall facing movements, and provide recommendations for depth of embedment for unbraced facing panels during construction (Chapter 4). Note that Chapter 3 describes the instrumentation and monitoring programs.
- Assess CDOT design procedures for facing panel and anchors based on measurements of lateral earth loads and moments on the facing panels and pullout loads in anchors (Chapter 5).
- Assess CDOT design procedures for the reinforced soil mass from measurement of reinforcement tensile forces (Chapter 6).

- Summarize the results of previous tasks and lessons learned for design and construction of future MSE/IFF wall systems (Chapter 7).
- Provides complete design and construction details and basis for future standard use of an MSE/IFF wall with facing either weakly braced or unbraced during construction (Chapter 8).

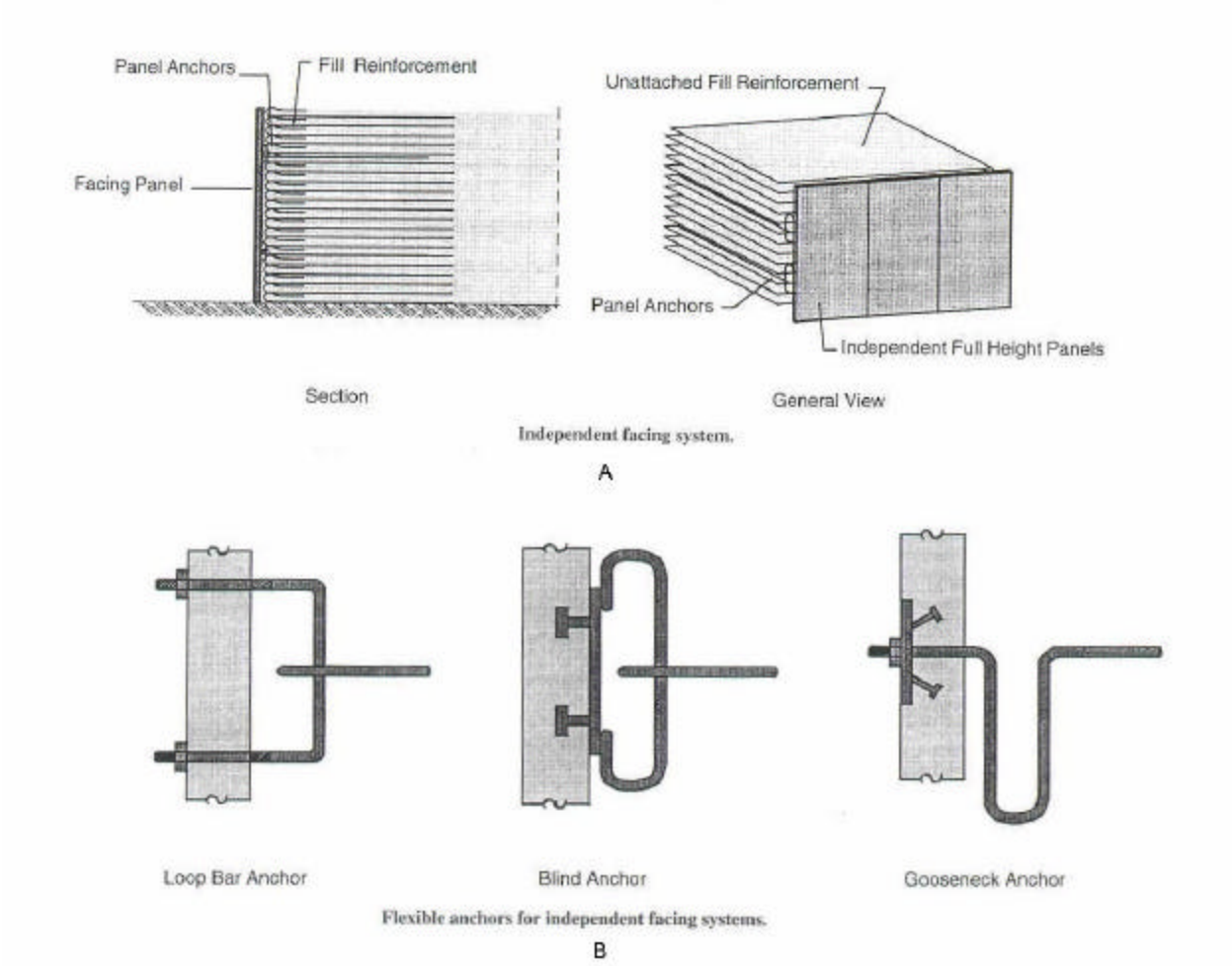

Figure 1.1. MSE Wall with Independent Full-Height Facing (from Hearn et al., 1995).

Figure 1.2. Picture of the Completed I-25/I-70 MSE Wall with Independent Full-Height Facing in Denver, Colorado.

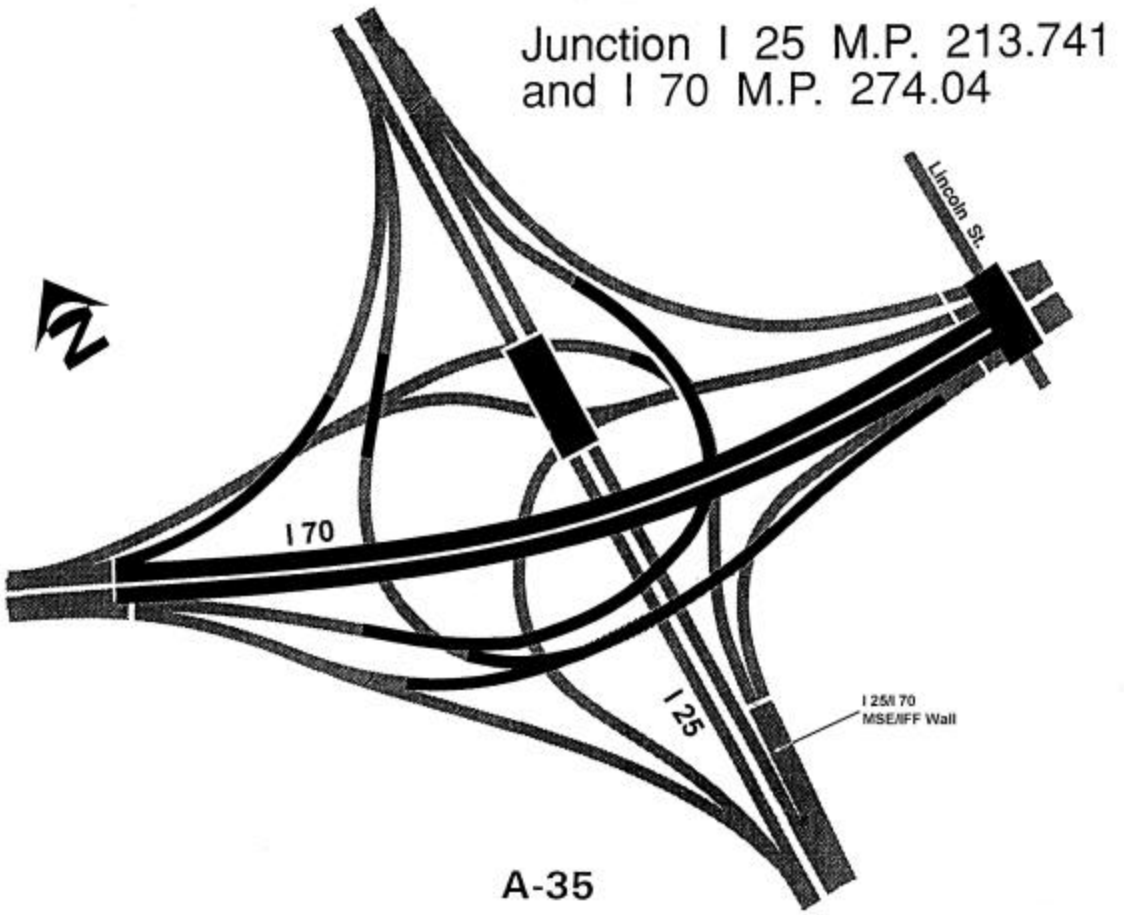

Figure 1.3. Map of the Location of the MSE Wall with Independent Full-Height Facing in Denver, Colorado.

2.0 MATERIAL AND CONSTRUCTION OF THE I-25/I-70 MSE/IFF WALL

2.1 Structure Layout

Typical cross-sections of the I-125/I-70 MSE/IFF wall are shown in Figures 2.1 and 2.2. The total wall length was over 1,400 ft. The facing total height (from bottom to top, see Figure 2.1) varied from 5.7 ft at the north end to 18.8 ft at the south end. The top of wall is typically 1 ft above the top of the acing (Figure 2.1).

2.2. Description and Strength of the Wall's Materials

The MSE wall has four major components (see Figures 2.1 to 2.5): 1) a self-stable welded wire fabric (WWF) reinforced soil mass, 2) full-height concrete facing panels not attached to the soil reinforcements (i.e., independent) that are allowed to tilt around their bases, 3) flexible face anchors to provide attachment of facing panels to the reinforced soil mass and accommodate movements of the wall system, and 4) a trench with flowfill to brace panel during construction before face anchors are placed (Figure 2.3). The backfill soil used in the project meets CDOT material specifications (i.e., gradation, liquid limit and plastic limit) and compaction requirements for CDOT class-1 backfill material. For a backfill meeting CDOT material and compaction requirements for class-1 backfill, the backfill unit weight (γ) and friction angle were taken as 125 pcf and 34°, respectively.

The facing panels were typically placed such that one 4 ft wide smooth panel is stacked next to four 8 ft wide grooved panels (Figures 1.2, 2.4a, and 2.5). The panel thickness ranges from 4' 1/8" for plain panels to 4' 3/8" for grooved panels (Figure 2.4b). When set, the panels have approximately 1" gap between adjacent panels. This gap is covered with fabric at the back of the panels for filtration purposes. The precast, prestressed facing panels conformed to sections 601(concrete), 602 (reinforcing), and 618 (prestressed components) of CDOT standard specifications. It was required that the initial concrete compressive strength of the concrete at the time of prestressing was transferred to the concrete, f'c, and the concrete 28 days compressive strength, f'c, were a minimum of 4,000 psi and 6,000 psi, respectively. The bending moment capacities of the panels in the vertical and transverse directions along the back of the panels (i.e.,

tension develops in the back side of the panels toward the fill) were 7800 pounds-ft/ft and 1200 pounds-ft/ft, respectively. The bending moment capacities of the panels in the vertical and transverse directions along the front of the panels (i.e., tension develops in the front side of the panels toward the highway) were 4300 pounds-ft/ft and 2000 pounds-ft/ft, respectively. The shear capacities near the edges of the panels were 6400 pounds/ft and 5300 pounds/ft, in the vertical and transverse directions, respectively.

The wires of the WWF sheets were galvanized, having a cross-sectional area of 0.029 in^2, and welded in a grid pattern of 1 ft x 1 ft squares (referred to as 12 x12 w2.9 x w2.9). Note that the plans call for WWF of wires having a cross-sectional area of 0.014 in^2 and welded in a grid pattern of 6"x6" squares (refer to as 6x6 w1.4 x w1.4 in Figure 2.2). Geomembrane was placed over the backfill to protect the steel reinforcement from the effects of road salt and water. The WWF mesh was manufactured in sheets 8 ft wide and 20 ft long. WWF sheets were placed with a vertical spacing of 1 ft (Figure 2.2). In the lower levels, the mesh was laid long-end (i.e., the 20-ft side) against the panels and in the upper levels the mesh was laid short-end (i.e., the 8-ft side) to the wall (Figure 2.2). The Young's Modulus (E) of the steel material of the WWF and face anchor rebars was taken as 30000 ksi, confirmed by tests performed at the University of Colorado at Denver. CDOT specifications require the steel yield stress for both anchor rebars and WWF to be at least 60 ksi, which corresponds to strain of 0.2% or 2000 microstrain. However, the typical yield stress for WWF sheets can reach 100 ksi. At 60 ksi, the steel yield force was calculated as 1740 pounds for the single wire of the WWF mesh.

The epoxy-coated #5 face anchor rebars were shaped as one half of a 12-sided symmetric polygon that starts at one panel gap and extends out in the backfill, then wraps around horizontally and connects to another panel gap three panels down (Figure 2.4a). The length of each facing anchor rebar loop was 30.33 ft when the anchor did not cross a 4 ft panel and 27 ft when the anchor crossed the 4 ft ft panel (Figure 2.4a). The facing anchor loop extends inside the reinforced soil mass 8 ft perpendicular to the wall and 20 to 24 ft parallel to the wall (Figures 2.2 and 2.4a). The project plans specify a maximum spacing of 8 ft for face anchors (Figure 2.2). The cross-sectional area (A) is 0.31 in^2 for the #5 face anchor rebars. At 60 ksi, the steel yield force was calculated as 18600 pounds for the face anchor rebars. The threaded ends of the facing

anchor rebars were attached to the facing panels in the gaps between adjacent facing panels (Figure 2.4b). As shown in Figure 2.5, at the panel gaps a rectangle plate (with a rebar size hole in the center) is slipped over the threaded end of the rebar and tightened down on the outside of the panels with a galvanized nut and a lock washer (Figure 2.4). This plate distributes forces on both adjacent panels, causing the panels to self-align (Figures 2.4 and 2.5). The rebar tiebacks pull the panel into the backfill material, helping to hold the panels up. To reduce the earth pressure on the facing, the nuts (Figure 2.5) can be loosened, thus taking the pressure off the panel and allowing the reinforced backfill to take the load. Loosing of nuts can also help the contractor to align the adjacent facing panels as will be discussed later.

The pullout resistance of facing anchors can be mobilized through interface friction and passive soil resistance. It can be obtained accurately from laboratory or field pullout tests performed in the specific backfill to be used on the project (Elias and Christopher, 1997), which unfortunately was not performed in this study. Pullout strength of the facing anchors (P) is taken, conservatively, as the friction developed along the facing bar surface in the undisturbed portion of the fill:

$$P = ALgz\tan j \qquad (2.1)$$

Where A is bar circumference, L is length of the resistive anchor in the undisturbed soil region, z is the height of the backfill above the anchor, g is the backfill unit weight taken as 125 pcf and j is the backfill friction angle taken as 34 degrees. The #5 bar circumference is 1.97 in or 0.165 ft . The resistive length of the facing anchor loop to pullout starts from near the facing and not beyond the potential failure surface of the reinforced backfill. Half of the facing anchor loop contributes to the pullout resistance, equals to 15.2 ft when the facing anchor did not cross the 4 ft panel and 13.5 ft when the facing anchor bar crossed the 4 ft panel (values from Figure 2.4). Fill immediately behind facing panels (disturbed zone) is at relatively low vertical earth pressure and makes an uncertain contribution to pullout capacity of anchor bars. For simplicity the length of this disturbed zone is taken as 1 ft and the length of the facing anchor in the undisturbed resistive zone to reinforcement pullout is taken as 14.2 ft when facing anchor bars do not cross 4 ft panel and 12.5 ft when the facing anchor bars cross the 4 ft panel. The pullout resistance of

the facing anchor in *pounds* can be related to the depth of the fill over the anchor (*z*) through the following equations:

$P = 198z$ …….. *(2.2)*

when facing anchor bars do not cross 4 ft panel, and

$P = 175z$ …….. *(2.3)*

when facing anchor bars cross 4 ft panel. Results for the face anchor pullout capacity from Eqs. 2.2 and 2.3 are presented in Chapter 5.

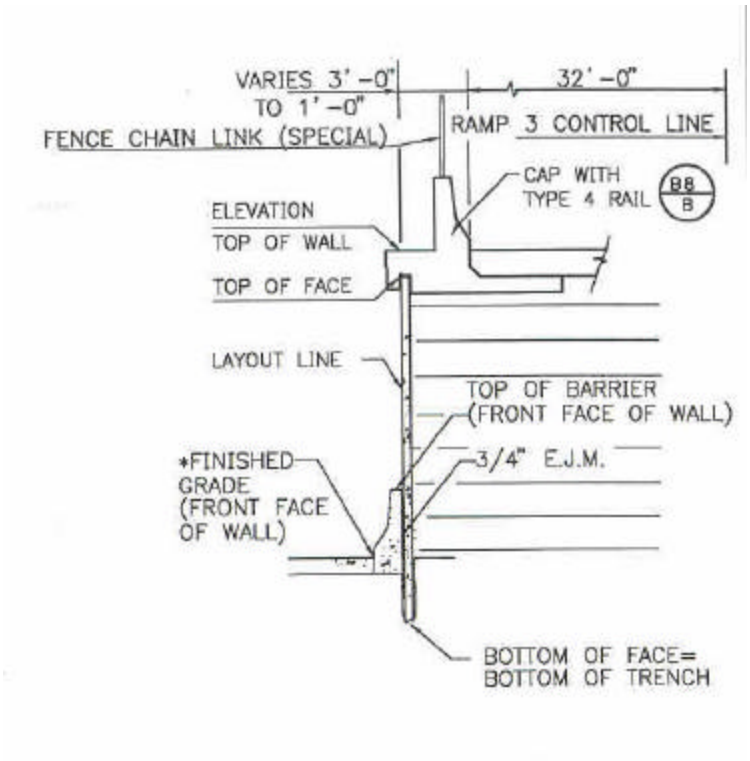

Figure 2.1. Typical Cross-Section along the I-25/I-70 Structure Showing the MSE/IFF Wall and Roadway Structure (from Construction Plans).

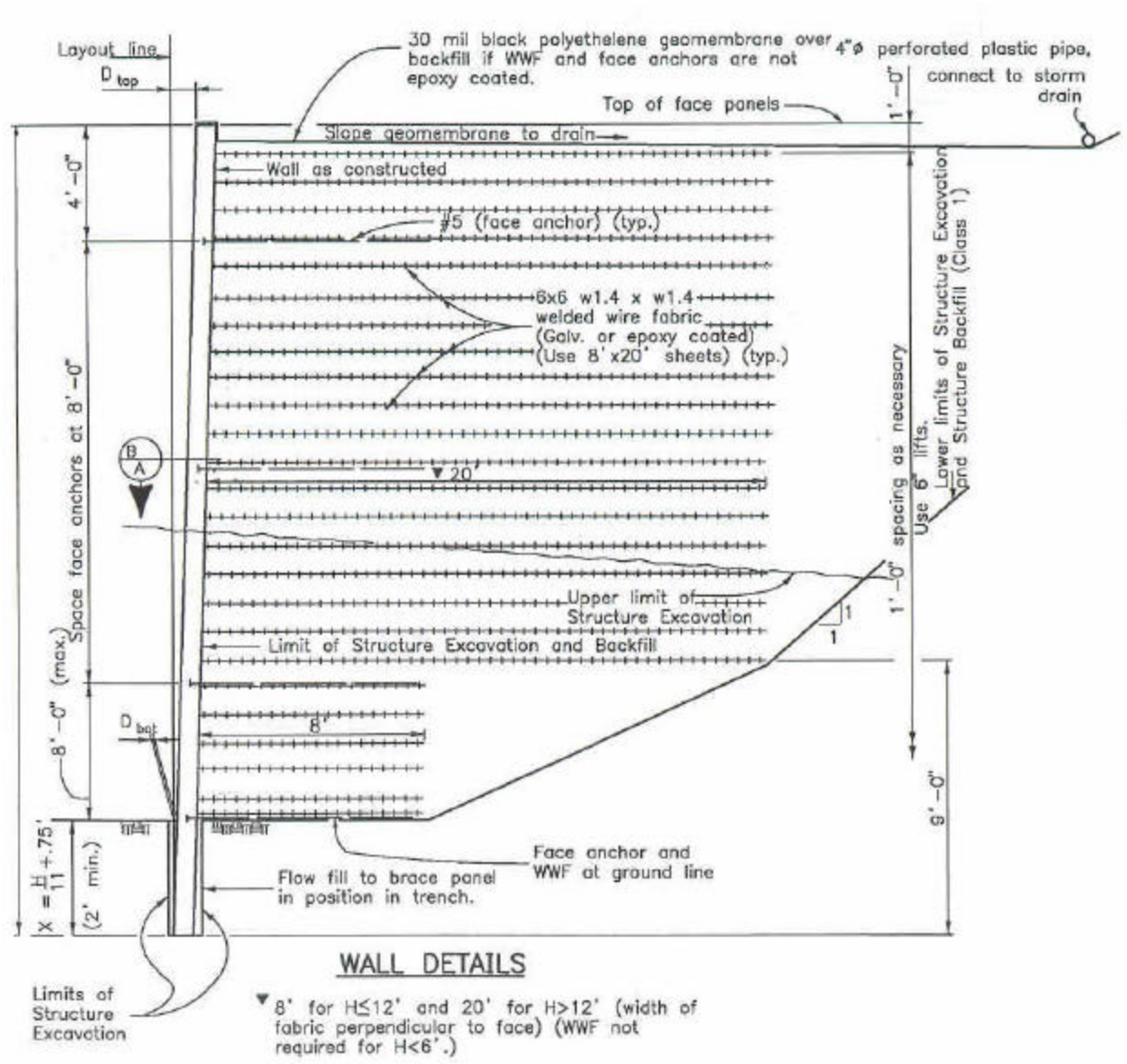

Figure 2.2 Details of the I-25/I-70 MSE/IFF Wall (from Construction Plans)

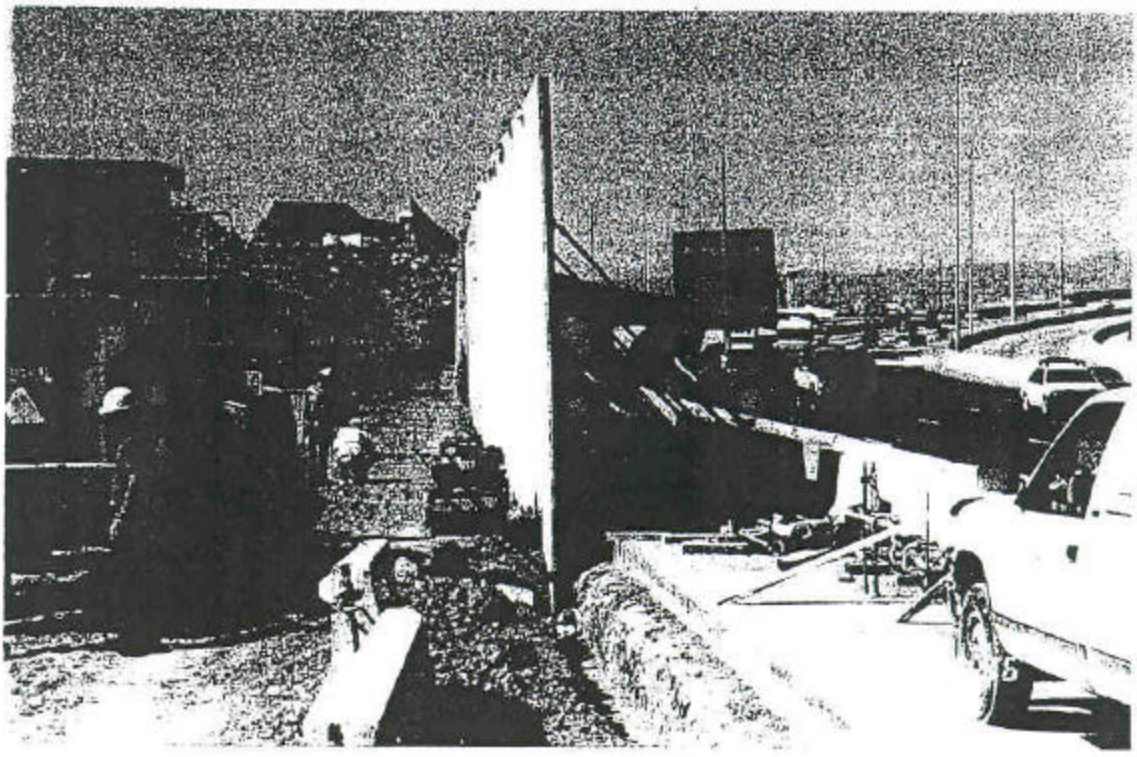

Figure 2.3. Placement of Facing Panel in Position in Trench (from Christina and Wu, 1999). Note: Bracing at Front (Right Side) of the Facing Panels.

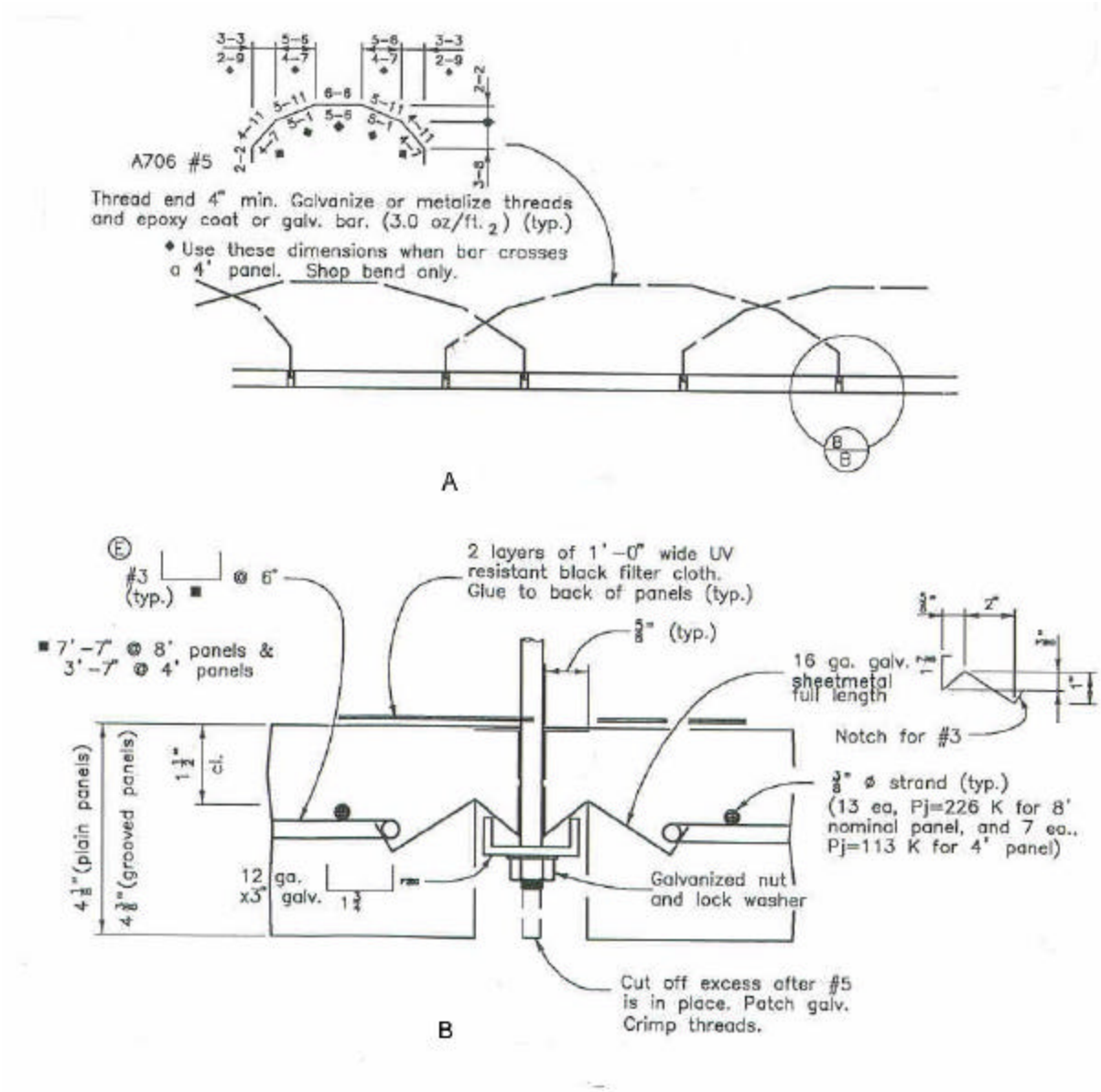

Figure 2.4. Details of the Facing Panels and Anchors (from Construction Plans)

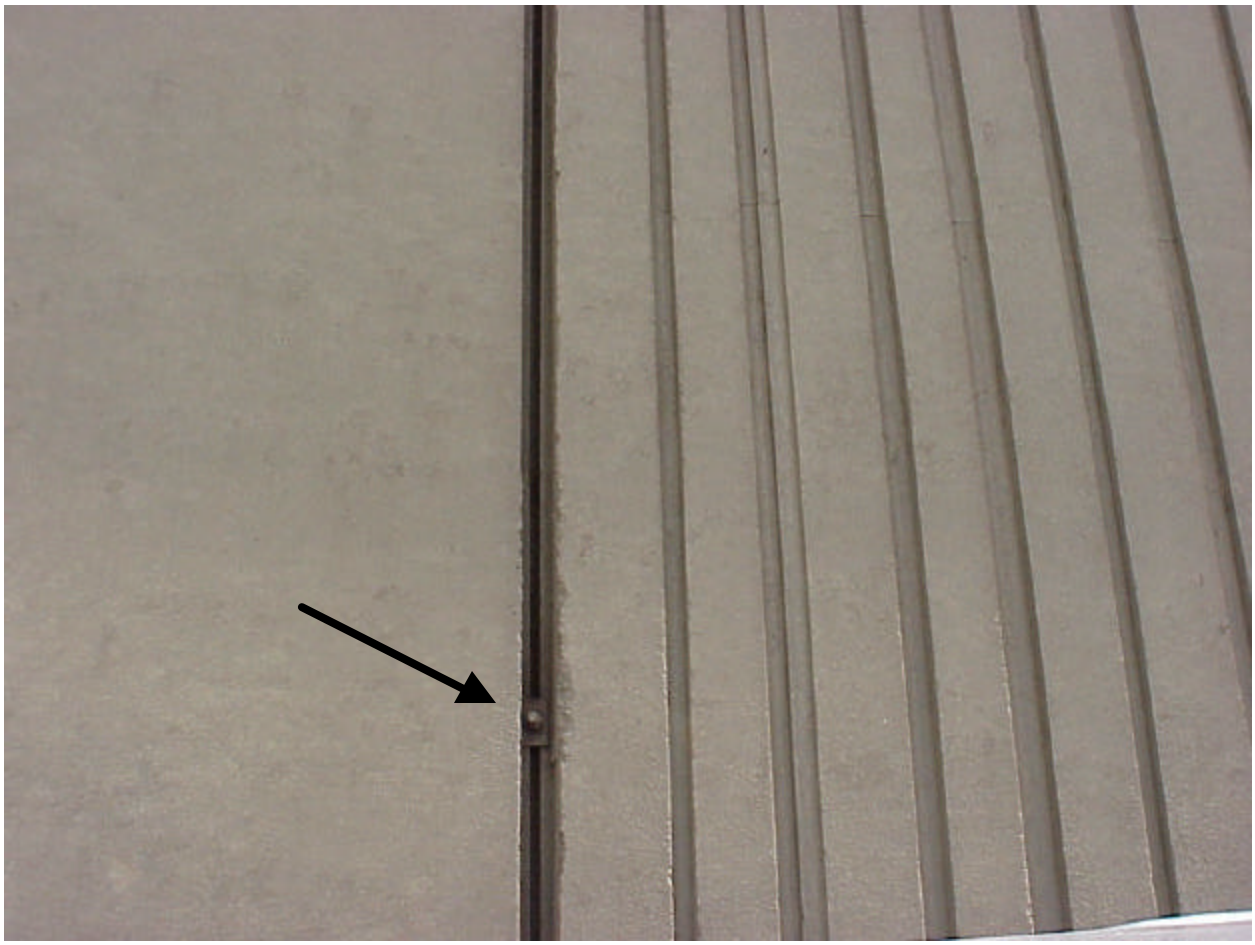

Figure 2.5. Details of the Face Anchor on the Outside Face of the Panel. Note Gap between Panels with Anchors, Nut, and Bearing Bar.

2.3 Construction of the Wall

2.3.1 Embedment and Bracing of Facing Panels

Bracing was utilized to hold the panel in position while the trench was being backfilled with flowfill. The trench was utilized to provide resistance against wind loads that might be experienced during construction before placements of face anchors. The depths of embedment employed for facing panels with different panel heights above ground level are shown in Table 2.1. In addition to the trench support, the facing panels were weakly braced during construction of the wall (see Figure 2.3). No guidelines were set for the timing to remove this bracing system. It was reported that the bracing was removed when the fill was 5 ft from the top. Others reported that shoring was removed when the fill was 2 ft from the top for the first 250 ft of the wall constructed in the south side, and was left in place until fill was at top for the remainder of the wall (1150 ft). It was also reported that most of the braces were loose while the structure was under construction. At least one brace buckled during construction but this did not seem to affect the panels.

2.3.2 Accommodating Movement of Facing Panels during Construction

Facing panels were allowed to move to accommodate the soil deformations as tensions in reinforcements were mobilized. There are no good methods for predicting soil deformation for this new MSE system and the objective of this research was to establish track records for future similar MSE walls. However, panel movements during construction may result in unacceptable facing alignment. Two measures were implemented during construction of the I-25/I-70 MSE/IFF wall to accommodate wall movements:

- First, at initial placement, facing was battered in anticipation of horizontal deformation away from the fill that would occur during construction. The initial bottom setback (D_{bot} in Figure 2.2) and top wall setback (D_{top} in Figure 2.2) from the layout line (see Figure 2.2) as recommended in the construction plans are shown in Table 2.1.
- Second, face anchor nuts were adjusted to improve wall alignment (described in next section).

To accommodate the post-construction wall movements, a loose surcharge of three ft of fill was placed on the top of the constructed wall (before placement of pavement structure) for a period of at least 60 hours. The three ft surcharge corresponds to the load expected from the roadway structure and traffic.

Table 2.1. Depths of Embedment and Setbacks Employed during Placement of Facing Panels in the I-25/I-70 MSE/IFF Wall

Facing Height above Ground-Level (feet)	4	10	12	20
Panels Embedment depth (feet)	2	2	2	2.8
Panels Bottom Setback (inch)	0.12	0.24	0.24	0.36
Panels Initial Top Setback (inch)	0.48	1.08	1.2	2.4
Panels Modified Top Setback during Construction (inch)	0.96	2.16	2.4	4.8

2.3.3 Construction Procedure as Described in the Construction Plans

1. Excavate and/or fill, and compact the area at the base of the walls such that the area is at the level of the future finished grade at the front of the wall and provides a firm base for construction equipment and allows for a vertical edged trench for the face panels.
2. Trench to the bottom of the panel facing elevation as specified in the plans (Table 2.1). The specified minimum depth and width of the trench were, respectively, 2 ft and 6".
3. Position facing panel in the trench with required top and bottom setbacks (Table 2.1), and use flowfill and bracing to brace the panel in the required position in the trench (see Figure 2.3).
4. Place backfill, wire mesh and face anchor rebars behind the facing as described in the plans (Figure 2.2). Lightly compact the 3 ft behind the facing with one pass of a hand-operated compactor. Keep heavy compaction equipment 2 ft minimum from the panels. Backfill compaction requirements (100% of AASHTO T-99 or 95% of AASHTO T-180A) will not apply within 4 ft of the panels.
5. Monitor the location of the top of the panel and the batter as backfill progresses. If it appears that the top of panel will deform beyond the layout line before backfill is complete, add supplemental layers of wire mesh and move the heavy compaction equipment further from the face of the wall.
6. When backfill placement is complete, place three ft of loose fill over the finished top grade for a period of at least 60 hours to reduce any potential long-term movements.

7. Remove the surcharge and loosen the nuts slightly on all face anchor bars. Back off the nuts (from bottom to top) on those nuts that are in areas that need to be brought out closer to the layout line for a smooth appearance of the face. Do not tighten nuts to attempt to bring the face back toward the reinforced backfill. Do not attempt to bring areas that are less than 1" behind the layout line out to the layout line, except to provide smooth appearance. If, from this adjustment, a gap develops between the reinforced backfill and the face, fill it with sand or flowfill.

2.3.4 Encountered Problems and Corrective Measures during Construction

The construction plans required close monitoring of the first six panels for movement before placement of additional panels. This section of six panels was built with the intent of adjusting as needed even if it meant tearing down the wall and reconstructing. For these panels, it was noticed that the facing movements were greater than anticipated during the compaction operations. Mr. Mike McMullen reported the following field observations: "The bracing struts were removed from the south six panels when the backfill was about 5 ft from the top of wall facing. Prior to removal, the struts were mostly loose with some carrying a small load. Following removal, the top of the wall facing moved outward slightly. When the next lift was compacted the top of the wall moved outward some more, so that when checked with a level it was very nearly plumb". The project engineer also reported that a portion of the wall facing moved outward about 2.25" with about 2 ft of backfill to go.

Possible reasons for the more than anticipated wall movements were:

- The equipment compacting and spreading the fill was somewhat larger than anticipated, especially close to the facing where the contractor used a heavy vibrating roller (maybe 5 ton class) rather than the light vibrating plate expected by the designer.
- In many cases, the first transverse wire of the mesh was 1 ft from the face (i.e., not close to the facing).
- Horizontal spacing of the wires of the WWF was 1 ft between the wires rather than 6" originally planned.
- The #5 face anchor rebars were not snug, did not seem to not take the up the load quickly, and did not seem to provide a stiff connection to the soil as expected in the design. This was

possibly due to: 1) slack in the bar positioning when placed, 2) possible movement of the bars when compaction was taking place immediately over the bars, and 3) the epoxy coating (which is very slick) reduced and weakened the interaction of face anchor rebars with soil.

Although the first six panels over-deflected slightly, it was judged to be aesthetically acceptable, so the panels stayed in place. The following corrective actions were recommended and implemented during construction of the remainder of the wall:

- Decrease the maximum spacing of the #5 face anchor rebars from 8 ft to 5' 4''.
- Place the first cross wire of the mesh as close as possible to the wall. Bend the tails at the face as needed to get the first cross wire within 2" of the facing.
- Snug the # 5 loop bars by pulling the #5 rebars and driving a # 4 L bar into the ground at each of the back ends (the bend with one side parallel to the wall). When one lift of fill has been placed on the face anchor rebars, tighten the nuts finger tight plus two full turns on both this layer of # 5 rebars and any lower layer.
- Increase the set back at the top of the panel to twice the plan values (See Table 2.1).
- Check the plumb of the panel just before the last lift is compacted. If the top of the panel is less than 0.5" behind plumb tighten all nuts 2 full turns past the finger tight, and leave the struts in place for the compaction of the last lift. If the top of the panel is between 0.5" and 1.0" behind plumb, tighten all the #5 nuts as above and remove the braces before placing and compacting the last lift. If the top of the panel is more than 1" behind plumb, tighten the nuts as above, then back off the nuts at all levels proportionally to achieve a setback of 1" at the top of the panel. For example if a 15 ft height panel is 3" back before the top lift is placed and the tieback bars are placed at 0 ft, 5 ft, and 10 ft, the bottom nut would be left snug, the middle and top nuts need to be backed off 2" x5/15 = 0.66" and 1.33", respectively.

After these corrective measures were applied, the remainder of the wall was never more than 1" behind plumb upon completion. Differentials tilts between panels were small, and in fact decreased as the panels loaded up, apparently due to the tieback connection equalizing the position of the edges of adjacent panels under load.

3.0 INSTRUMENTATION AND MONITORING PROGRAMS

3.1 Description of the Monitored Sections

Two sections of the retaining wall were instrumented to monitor the wall performance during construction stages. These sections were designated as Station 3116 and Station 3119. Each 56 foot long test section was composed of six 8 ft wide facing panels and two 4 ft wide facing panels (Figures 3.1 and 3.2). Figures 3.3 and 3.4 show cross-sections of the monitored sections at Stations 3116 and 3119, respectively. The total height of the panel facing was 17.7 ft at Station 3116, of which 15.2 ft was above ground level (Figure 3.3). The total height of the panel facing was 15.3 ft at Station 3119, of which 13.1 ft was above the ground level (Figure 3.4). The face anchors were installed at H = 0 (ground surface), 6 ft, and 11 ft above the ground surface for Station 3116, and H = 0 (ground surface), 5 ft, and 9 ft above the ground surface for Station 3119. The WWF mesh was placed with a vertical spacing of 1 foot. Figures 3.5 and 3.6 show the history of the placement of the reinforced fill behind the wall facing at Stations 3116 and 3119, respectively.

The backfill placement along Station 3116 started on June 14, 1996 and was almost completed by June 28, 1996 (fill height= 16.1 ft, 15.2 ft of backfill behind the panels and 1 foot of the loose surcharge fill). The backfill placement along Station 3119 started on July 1, 1996 and was almost completed by July 26, 1996 (fill height= 12 ft). No information on construction progress was available after June 28 for Station 3116 and after July 26 for Station 3119. Construction of the wall and placement of the roadway structure were completed at mid-August 1996.

3.2 Instrument Descriptions, Layouts, and Applications

A number of instruments and tools were employed to monitor the performance of the retaining wall. Instrument descriptions, layouts, and applications are given in the following subsections. The data reduction for the inclinometer, face anchor and WWF strain gages is described in detail in the Geokon Instruction Manuals for each instrument.

3.2.1 Surveying

Survey targets were mounted on the surface of selected facing panels in the test sections (Blanks, 1996). The survey equipment was placed across Interstate-25 in the parking lot of the Regency Hotel. A total of 48 survey targets (24 on Station 3116 and 24 on Station 3119) were placed. At each Station, 8 survey targets were placed at H=1 ft, 8 survey targets at H=4 ft, and 8 survey targets at H=7 ft (where H is height above ground level). A surveying device was used to measure the distances and elevations at the targets. At each level (i.e., H=8 ft) eight readings of elevation and eight readings of distance were collected and were represented by one average elevation reading and one distance reading. Changes in the average elevation and average distances with time at each level provide information for vertical and outward displacements of the wall facing at that level.

3.2.2 Vibrating Wire Inclinometer

Geokon Model 6300 vibrating wire in-place inclinometers were employed to monitor the outward lateral displacement of the wall at different times during construction. Two inclinometers were set in the back center of 8 ft wide facing panels as shown in Figures 3.1 and 3.2 and labeled with a 2-digit number. They were set after preparation of the base and flowfill footing but prior to the backfill placement. The actual total outward displacements of the facing panels cannot be obtained from the measured data of the inclinometers. The inclinometer data, however, can be used to determine the relative outward displacement between two selected points on the panels and the angles of rotation or tilt rate (changes in facing outward displacements per 1 unit of wall facing height) of the facing panels.

3.2.3 Thermistors

Thermistors were installed adjacent to the strain gages to monitor the temperatures at the time the gage readings were taken.

3.2.4 Vibrating Wire Rebar Strain Gages

Geokon Model 4911A vibrating wire strain gages were used. The strain gages were installed at the ends of the face anchors where the anchors were attached to the facing panels to measure strains. These strains can be used to estimate the forces exerted by the reinforced fill on the

facing panels. The rebar gages were delivered on their own 4 ft piece of #5 rebar. They were then cut down to 2 ft and threaded on one end. The 2 ft epoxy threaded end of the placed rebars was cut out to allow for placement of the gaged rebar. A coupling device was then used to connect in series the placed rebars to the gaged rebar section. The gaged rebar section was covered with asphalt type mastic tape for corrosion protection.

The layouts of the face anchor strain gages are depicted in Figures 3.1 and 3.2 for Stations 3116 and 3119, respectively, each labeled with a 5-digit number. Gages at both Stations were placed at three elevations, layer 1 strain gages placed at the lowest level, layer 2 at the intermediate level, and layer 3 at the highest level. Nine gages were placed at Station 3116 and 10 gages at Station 3119.

The basic units utilized by Geokon for measurement from the face anchor strain gages (Geokon Model 4911A) are "digits", read manually in mode B of Geokon Readers 403. The general equation employed to estimate the temperature corrected strains from reading collected from the face anchor strain gages is

$$e_{corrected} = [(R_1 - R_0) \times C] + [(T_1 - T_o) \times K] \quad \text{(3.1)}$$

where $\varepsilon_{corrected}$ is the temperature-corrected strain, R_0 is the reference reading in "digits", R_1 is the subsequent reading in "digits", C is the calibration factor ($C = 0.38$ microstrain per digit), T_0 is the reference temperature, often recorded at the time of installation, T_1 is the subsequent temperature reading, and K is the thermal coefficient, equal to 0.9 microstrain/^oC.

Once the strains were obtained, the following equation was used to calculate the corresponding forces:

$$F = e\,E\,A \quad \text{(3.2)}$$

Where E is the Young's modulus of the material (30 x 10^6psi for steel), A is the cross sectional area. The EA is 9300 kips for the face anchor rebars.

3.2.5 Vibrating Wire WWF Strain Gages

Geokon VK4100 strain gages were mounted at selected points on the welded wire mesh reinforcement. The mesh strain gages are custom fabricated for that particular mesh. They were tightened down onto the wire having sensors on opposite sides of the mesh wire (top and bottom). If the wire mesh is placed on uneven ground or experiencing any flexural loading, the two sensors offset will cancel out, one will be in tension and the other in compression, giving true change in axial wires strains.

As the backfill was placed, several wire mesh strain gages went off the scale and maxed out, yielding no reading. This problem is a result of the highly sensitive nature of the gages themselves. The gages were ordered with high sensitivity because the expected strains seen on MSE wall were small (Blanks, 1996). In addition, the welded wire fabric mesh is a lightweight flimsy material that can easily experience vertical deformation. This potential for large vertical deformations works against the highly sensitive gages. If the mesh is resting on uneven material or a hole, that alone can max out the sensor. To alleviate the problem, fine grain sand was placed around the gages. The combination of gage sensitivity, flexibility of the WWF, and installation resulted in the loss of 3 WWF strain gages.

The layouts of the WWF strain gages for Stations 3116 and 3119 are depicted in Figures 3.3 and 3.4, respectively. Three strain gages were attached to the WWF of Station 3119 (R-1 at the lowest level, R-2 at the intermediate level, and R-3 at the highest level). These gages were placed behind Panel # 60 (Figure 3.2) and their position from the facing is shown in Figure 3.4. Twenty-six strain gages were placed along Station 3116, 13 gages behind panel #22, and 13 gages behind panel # 23, at four elevations above ground level (see Figure 3.3): layer 1 gages at the lowest level (H= 1 ft), layer 2 at the second lowest level (H= 4 ft), layer 3 at the second highest level (H= 7 ft), and layer 4 at the highest level (H= 11ft). WWF strain gages for Station 3116 were labeled with one digit that refers to the number of the layer (1, 2, 3, or 4) and two letters (e.g., 2-A-C). The first letter characterizes the position from the facing where A is closest to the facing, B is second closest to the facing, C is third closest to the facing, and D is the farthest from the facing. The distance from the facing for each gage is shown in Figure 3.3. The second letter characterizes the gage location parallel to the wall, N (North side of the control section) if placed

behind panel # 23 and S (south of the control section) if placed behind panel 22 (Figure 3.1). For example, six gages were placed at layer 2 (H= 4 ft, Figure 3.3) labeled as: 2-A-S, 2-A-N, 2-B-S, 2-B-N, 2-C-S, and 2-C-N.

Data was collected from 26 WWF strain gages (23 at Station 3116 and 3 at Station 3119). There were three problems with this data:

- It was indicated in the field notes that the data for WWF strain gages were collected on Mode E of Geokon Reader 403, in which readings are displayed directly in microstrains. The typical range for microstrain readings is between 1000-4000 (see Geokon Manual). It was unexpected that some of these reading were larger than 6000 (Gages 1-A-S, 1-B-S, 1-C-N, 2-C-N) and one gage reading was even larger than 7000 (4-C-S). After lengthy discussion with Geokon Inc., Geokon engineer, Mr. Tony Simmonds, indicated that such high microstrain readings although very unusual are possible. Therefore, and to be more conservative, it was decided to treat the collected data as microstrain readings.
- A pair of strain gages were attached to the top and bottom of wire mesh at each location so that the average of their readings will reflect changes in wire mesh axial strains and cancel out any bending effect. However, one reading was collected and reported at each location.
- Most of the gages where the first reading was taken with no backfill on the gages experienced a large increase in the second reading that was much larger than the entire change from the second reading to the final reading. For some of these gages, results for calculated changes in wire mesh strains (assuming correct first reading) due to the placement and compaction of one to two ft of backfill over these gages are shown in Table 3.1. The large changes in strains shown in Table 3.1 can be attributed to large locked-in strains induced by compaction operations (Abu-Hejleh et. al., 2000), bending of the wire mesh, and non-stable or erroneous first reading due to the sensitivity of the gages and flexibility of the WWF. These changes in WWF strains are questionable because they are very large and exceed in some cases the typical steel yielding strains of 2000 microstrains (4-B-N, 4-C-S, and 4-C-N).

Table 3.1. Measured Changes in Wire Mesh Strains following Placement and Compaction of 1 to 2 ft of Backfill over the Gages.

Gage	1-A-S	1-C-N	4-A-S	4-A-N	4-B-N	4-C-S	4-C-N
Measured Changes in WWF Strains between the first two readings	1119	1259	870	910	2718	2447	3563

To address and alleviate the consequences of the last two problems in the collected data, data analysis was performed by:

- Taking reference readings when at least 1 foot of backfill is placed and compacted over the gages. It is expected that most of the bending effect will occur during placement and compaction of the first 1 foot of backfill.
- Averaging the readings of two gages placed at the north and south sides of the control section (but at the same elevation and distance from the facing). This reading will be a better representative of the overall straining behavior of the wire mesh. For example, results for gage 1-B will be the average of readings taken from Gages 1-B-N and 1-B-S.

The thermal expansion of gages is equal to thermal expansion of mesh and therefore no correction for temperature-induced strains was needed. It was concluded that the data for WWF strain gages were collected on Mode E of Geokon Reader 403, in which readings are displayed directly in microstrains. According to Geokon, the true changes in WWF microstrains $\mu\varepsilon_{true}$, can be calculated by:

$$me_{true} = (E_1 - E_0) \times B \qquad (3.3)$$

Where E_0 is the reference reading on Channel E, E_1 is the subsequent reading, and B is a batch gage factor, typically 0.91. Once the strains are obtained, the Equation 3.2 was used to calculate the corresponding tensile forces in the wires of the WWF using EA of 870 kips.

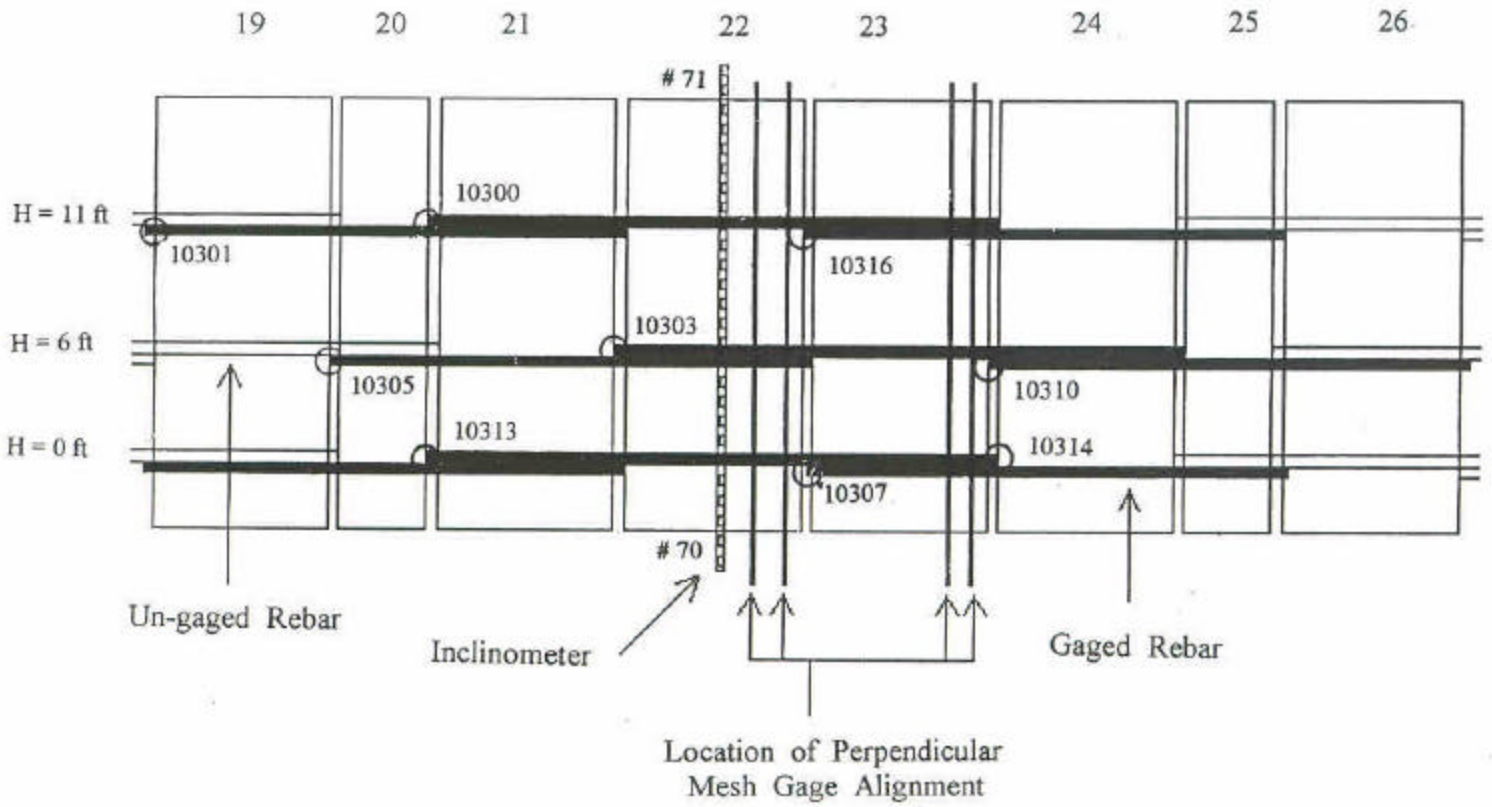

Figure 3.1. Layout of the Monitored Test Section for Station 3116, Showing Location of Face Anchor Strain Gages and Inclinometer (from Blanks, 1996).

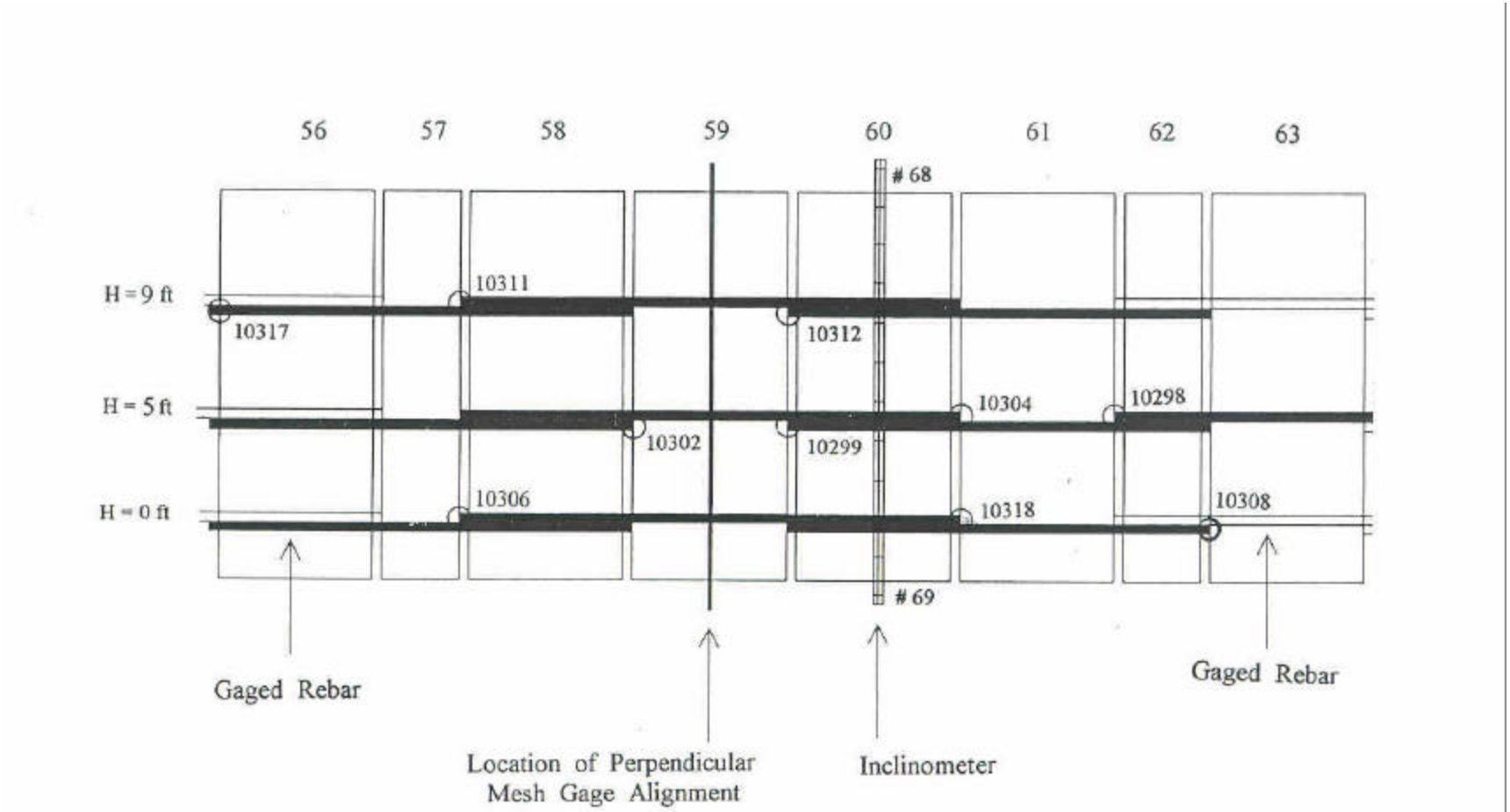

Figure 3.2. Layout of the Monitored Test Section for Station 3119, Showing Location of Face Anchor Strain Gages and Inclinometer (from Blanks, 1996).

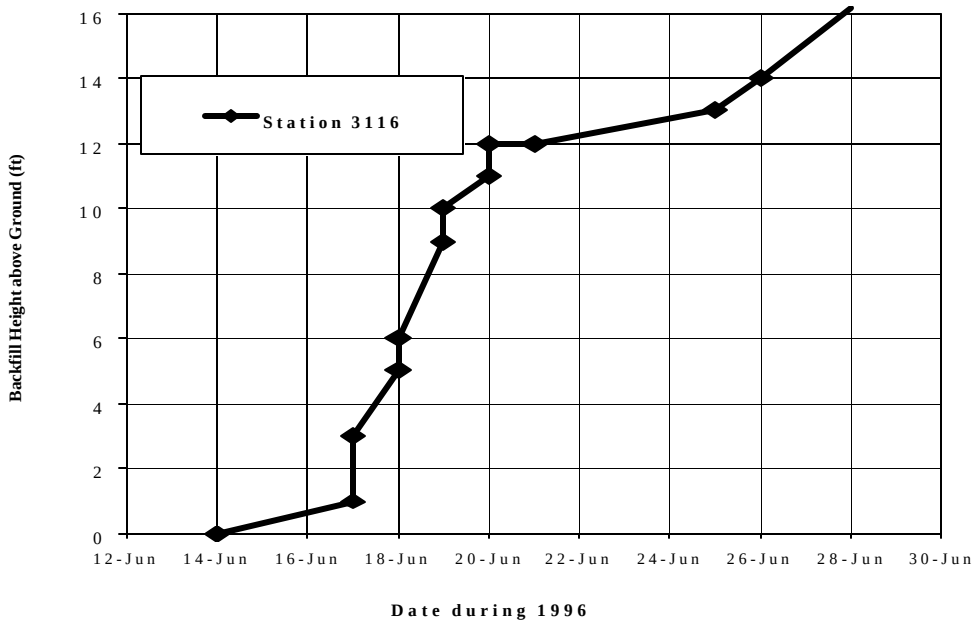

Figure 3.3. Cross-Section for Station 3116, Showing Locations of Wire Mesh and Face Anchor Strain Gages (Source for Locations: Blanks, 1996).

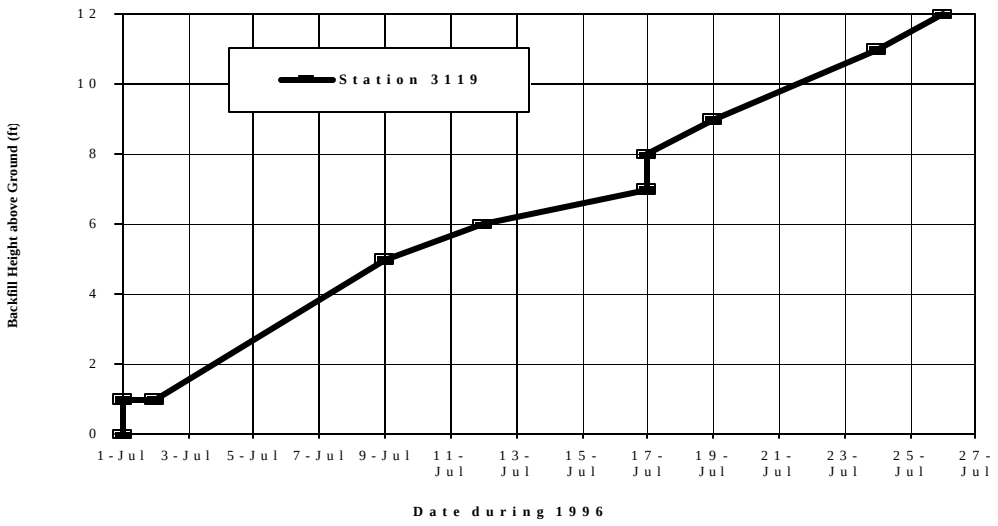

Figure 3.4. Cross-Section for Station 3119, Showing Locations of Wire Mesh and Face Anchor Strain Gages (Source for Locations: Blanks, 1996).

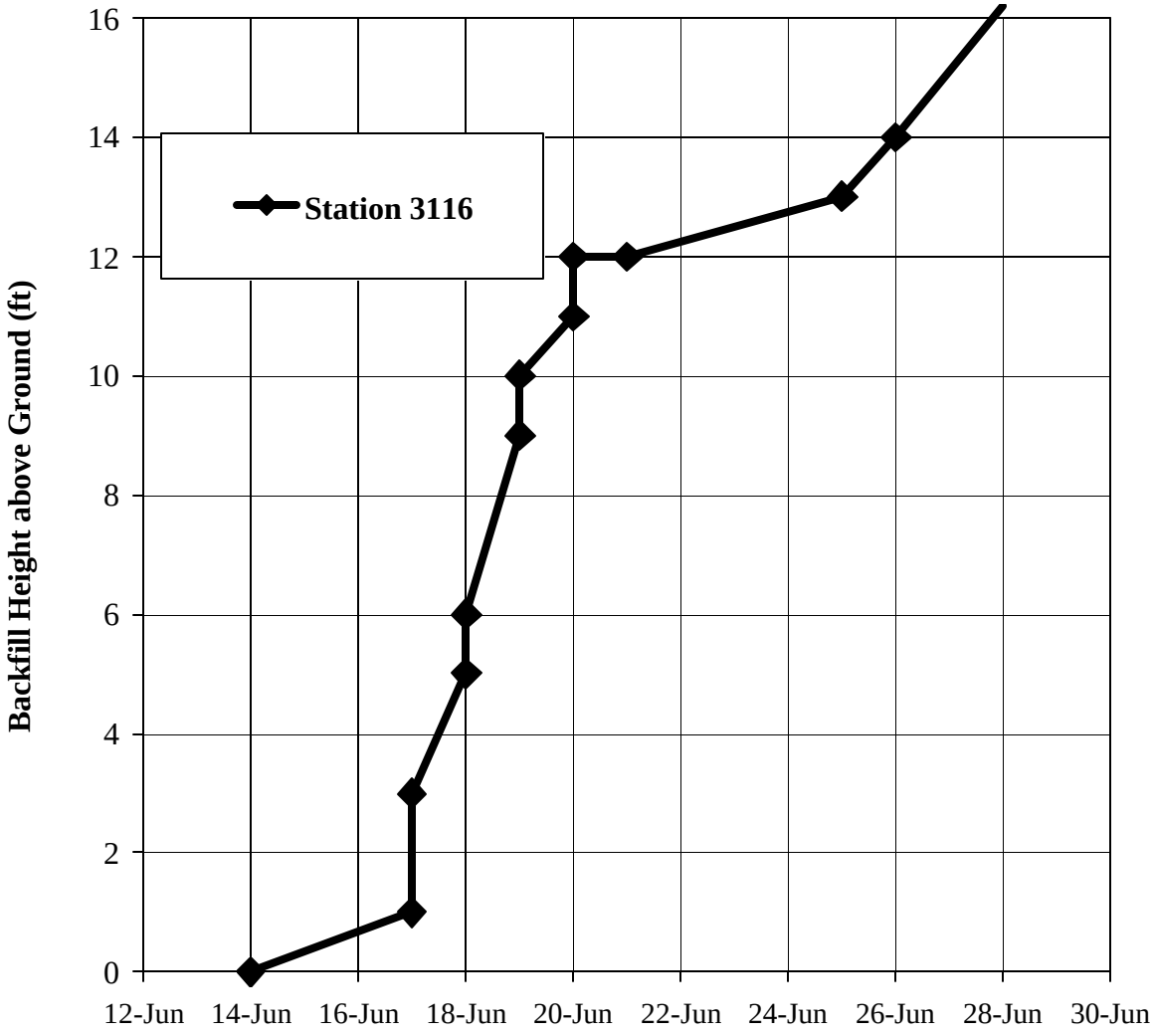

Figure 3.5. Fill History of Station 3116 (Based on Data from Blanks, 1996). Note That Top of Panel is at 15.2 ft.

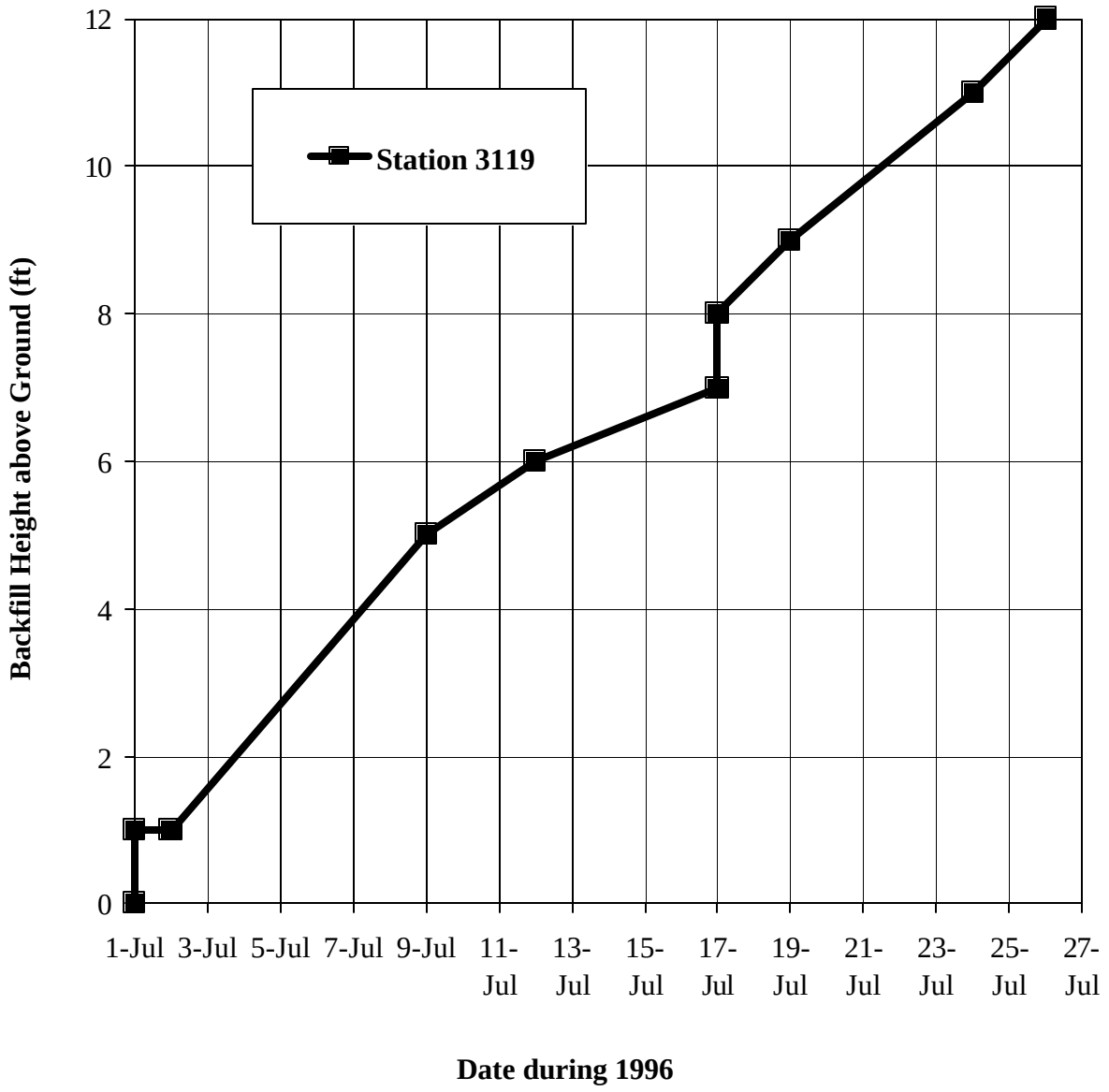

Figure 3.6. Fill History of Station 3119 (Based on Data from Blanks, 1996). Note That Top of Panel is at 13.1 ft.

4.0 DEPTHS OF EMBEDMENT AND SETBACKS FOR THE FACING PANELS

4.1 Introduction

The employed depths of embedment for the I-25/I-70 MSE/IFF wall structure, where the facing was weakly braced during construction, seem to be adequate (Table 4.1). However, using unbraced panels during construction speeds construction, allows for construction in more confined spaces, and reduces construction costs. Recommendations for depth of embedment for future MSE/IFF wall system with unbraced facing during construction are presented in Section 4.2.

Measurements of the facing movements for the I-25/I-70 MSE/IFF wall structure were employed to estimate the top setback (D_{top} in Figure 2.2) and bottom setback (D_{bot} in Figure 2.2) from the layout line (Figure 2.2) that should be employed for construction of future MSE/IFF wall structures. These movement measurements are presented in Section 4.3 and analyzed in Section 4.4. Recommendations for setbacks in future MSE/IFF wall applications with facing panels either weakly braced or unbraced during construction are presented in Section 4.5.

4.2 Depths of Embedment for Unbraced Facing Panels during Construction

For the I-25/I-70 MSE/IFF wall, the depth of embedment for the trench was calculated to provide enough moment resistance to support a wind load of 8 psf on facing panels. For future MSE/IFF wall systems with unbraced facing during construction, the assigned serviceable wind pressure was 12 psf, as recommended by AASHTO for serviceable wind pressure acting on sound barrier walls (permanent structure). This higher design value of wind pressure was selected to increase confidence of construction personnel in stability of the wall during construction. The method adopted in this study to estimate the proper depth of embedment is similar to that employed in geotechnical engineering to estimate depth of embedment for cantilever sheetpiling. For a facing with a height H above ground level and wind pressure of w, the total wind load acting on one unit width of the wall is Hw. Under this load, the facing tend to rotate around a pivot point below the ground level (not necessary at the base of the facing), developing active earth pressure where the soil is stretched outward and passive earth pressure where the soil is compressed laterally.

Assuming that the facing is embedded in granular soil and the water level is below the bottom of the facing, the depth of embedment (*D*) can be obtained as a solution to the following fourth-degree equation (from Powel, 1988):

$$D^4 - C_1D^2 - C_2D - C_3 = 0 \quad \text{.....(4.1)}$$

Equation 4.1 constants can be calculated as $C_1 = 8Hw/(g*(K_p-K_a))$, $C_2 = 6H^2w/(g*(K_p-K_a))$, and $C_3 = 4(Hw)^2/(g*(K_p-K_a))^2$ where g is the backfill unit weight and K_a and K_p are, respectively, the active and passive earth pressure constants of the backfill. For a soil below ground level with friction angle of 30 degrees, a unit weight of 100 pcf, and interface friction angle between the facing and the surrounding material (flowfill in this case) of 20 degrees, the depths of embedment from Equation 4.1 were calculated. The values calculated from Equation 4.1 for depth of embedment were increased by a factor of 1.3 as recommended for the cantilever sheetpiling (Powel, 1988). As expected, Table 4.1 shows higher depth of embedment for MSE/IFF with unbraced facing during construction than those employed in the I-25/I-70 MSE Wall project with facing weakly braced during construction.

Table 4.1. Recommended Depths of Embedment for Facing Panels Either Weakly Braced or Unbraced during Construction.

Facing Height above ground (ft)	4	10	12	16	20
Embedment for Weakly Braced Facing (ft)	2	2	2	2.4	2.8
Embedment for Unbraced Facing (ft) obtained from Eq. 4.1	2	3.4	3.8	4.4	5.2

For a 20-ft height MSE/IFF wall with unbraced facing subjected to a wind load of 12 psf, the maximum moment in the vertical direction is 2400 foot-pound/ft. This is smaller than the ultimate panel moment capacity (front side of the panel) of 4300 ft-pound/ft (factor of safety of 1.8).

4.3 Surveying and Inclinometer Results for Movement of Facing Panels

The movements of the wall facing panels were measured by a precision surveying method and two inclinometers placed along Stations 3116 and 3119 (Figures 3.1 and 3.2). Surveying and inclinometer data were collected during all construction stages of the wall and roadway structure. No displacement data for facing panels were obtained after opening the structure to traffic. The

backfill placement along Station 3116 started on June 14 and was almost completed by June 28 (fill height= 16.1 ft). The backfill placement along Station 3119 started on July 1 and was almost completed by July 26 (fill height= 12 ft). By mid-August 1996, the construction of the wall and the roadway structure was completed.

For Station 3116, the reference for the survey data was the survey reading collected on June 18, at which time the fill height was 5 ft above ground. Average cumulative changes in elevations (i.e., measure of facing settlements) and distances (measure of facing outward displacements) at different levels of the facing panels are shown, respectively, in Figures 4.1 and 4.2 (from Christina and Wu, 1996). For Station 3119, the reference for the survey data was the reading taken on July 1, 1996, at which time the fill had not begun. Average cumulative changes in elevations and distances at different levels of the facing panels during the entire wall construction history are shown in Figures 4.3 and 4.4 (from Christina and Wu, 1996). The displacement curves follow a fairly consistent trend as expected. The negative values indicate that the panel settled vertically and moved outward (i.e., toward Interstate-25).

Results in Figures 4.1 and 4.3 indicate that settlement of the facing were almost uniform at different heights. This indicates that the settlement of the facing was primarily due to settlement of the foundation soil. The total measured facing settlement occurred along Stations 3116 and 3119 were, respectively, 0.055 ft (0.66”), and 0.1 ft (1.2”). The facing settlement at both Stations continued after most of the backfill behind the facing was placed (after June 28 for Station 3116 and after July 26 for Station 3119). For Station 3116, there was a slight "rebound" past August 5 (Figure 4.1).

As expected, the facing outward displacement was larger at the top of the panels and smaller near the bottom (Figures 4.2 and 4.4). Most of this displacement occurred during placement of backfill behind the facing and very small displacement occurred after that (after June 28 for Station 3116 and July 26 for Station 3119). The total measured facing outward displacements at H= 7 ft measured at Stations 3116 and 3119 were, respectively, 0.21 ft (2.52”) and 0.19 ft (2.28”).

Following the discussion in the previous chapter, the inclinometer results were obtained in term of facing tilt defined as changes in facing outward displacements per 1 unit of wall facing height. The histories of the facing tilt as deduced from inclinometers for Stations 3116 and 3119 are shown in Figures 4.5 and 4.6 (from Christina and Wu, 1996, referred to as deflection rate). The reference inclinometer reading was taken on June 18 for Station 3116 (fill height = 5 ft), and on July 14 for Station 3119 (fill height= 5.5 ft). The maximum measured facing tilt (in/ft) were 0.17 at Station 3116 and 0.11 for Station 3119.

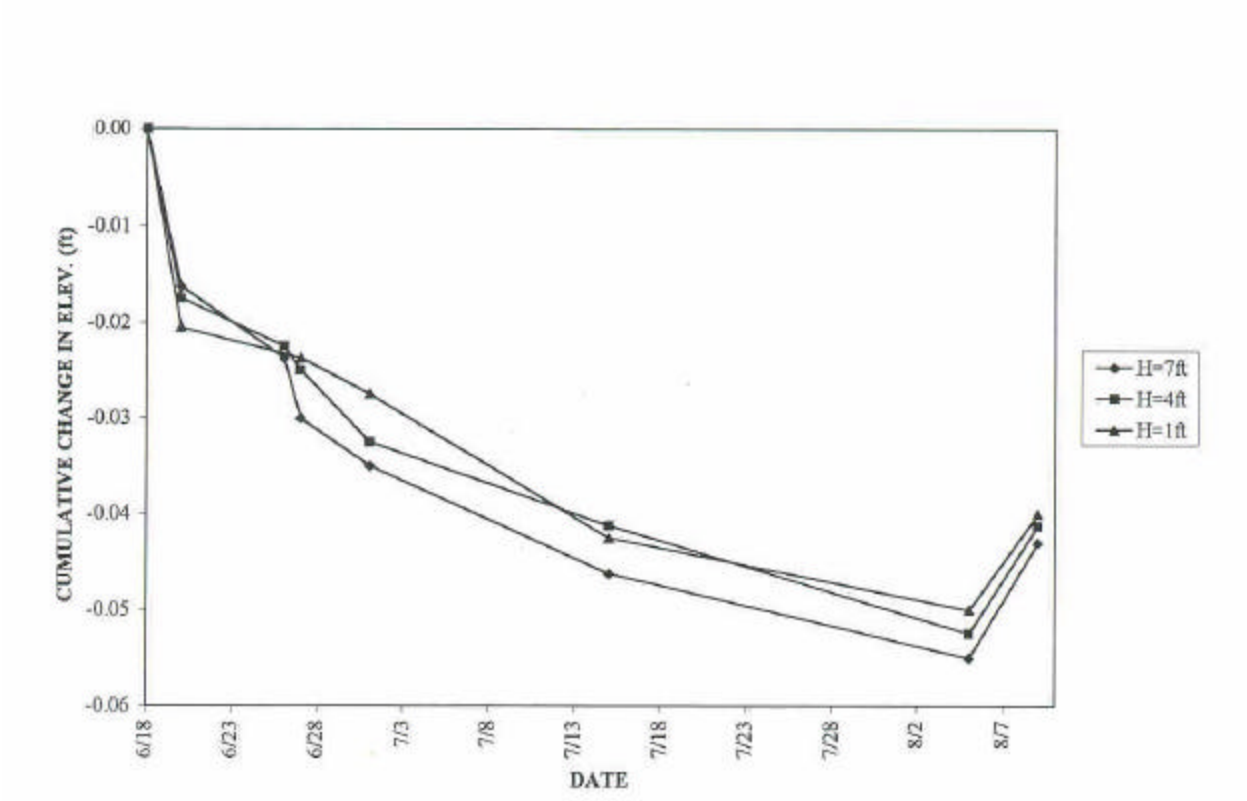

Figure 4.1. Average Cumulative Change in Elevation of Facing Panels with Time at Station 3116 (from Christina and Wu, 1999).

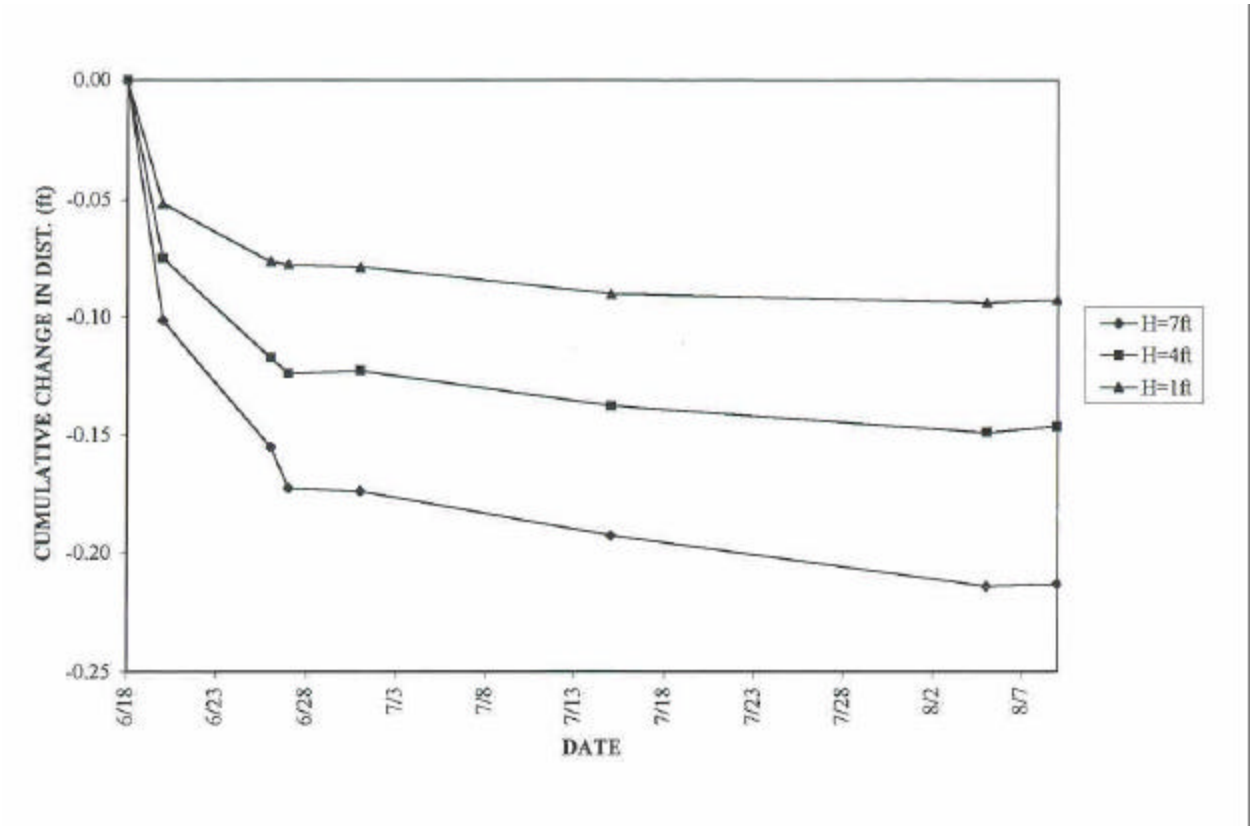

Figure 4.2. Average Cumulative Change in Distance from Facing Panels with Time at Station 3116 (from Christina and Wu, 1999).

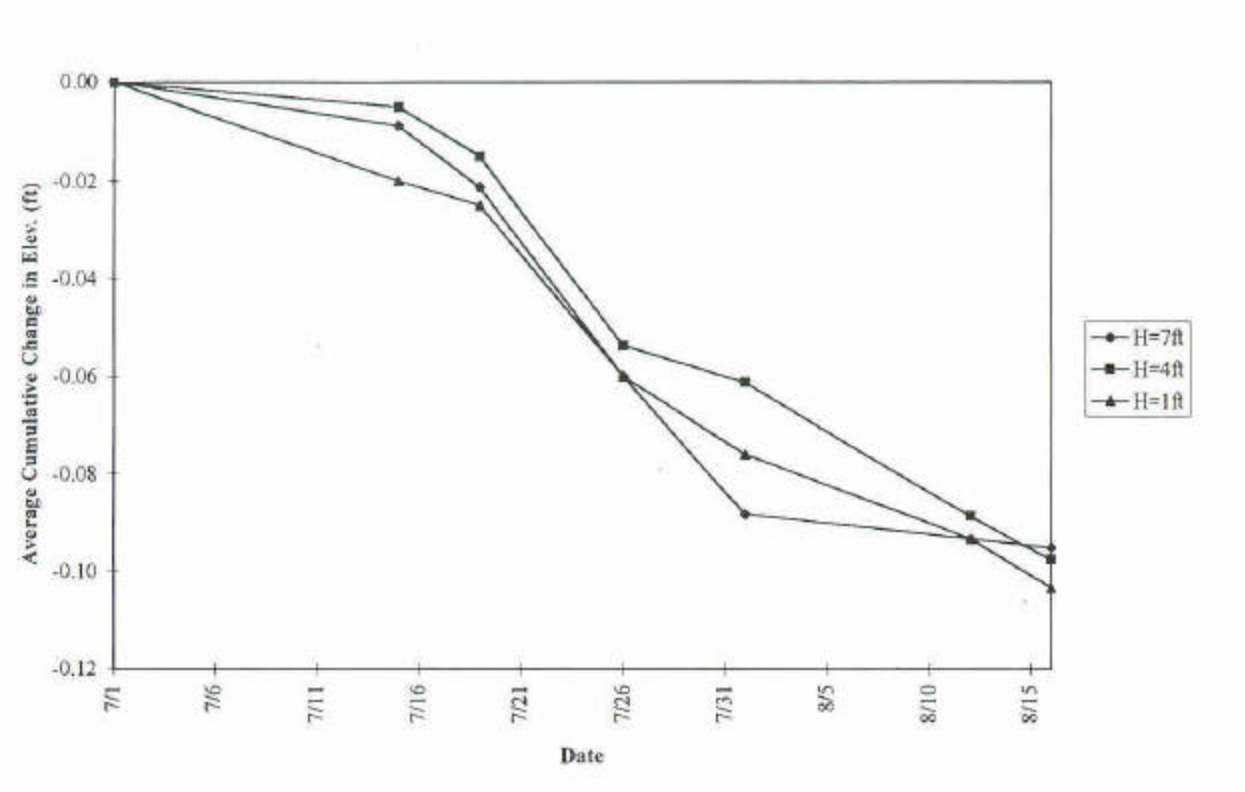

Figure 4.3. Average Cumulative Change in Elevation of Facing Panels with Time at Station 3119 (from Christina and Wu, 1999).

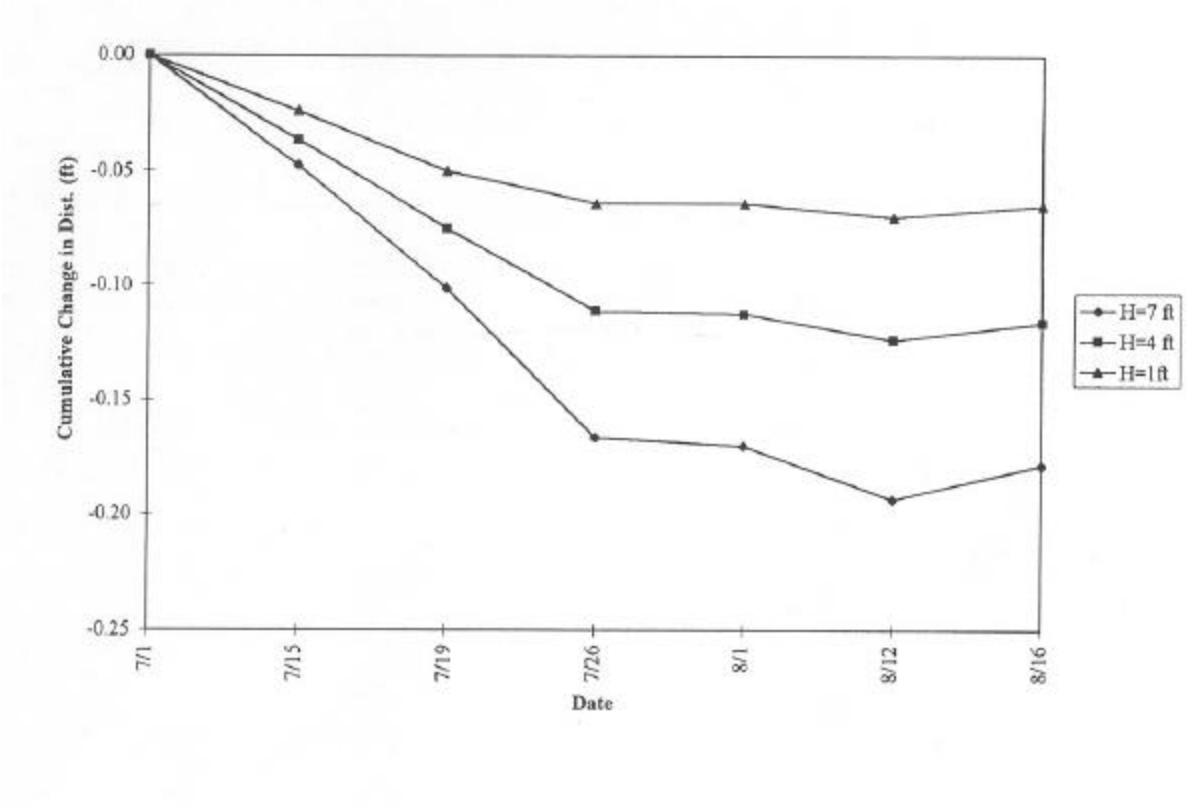

Figure 4.4. Average Cumulative Change in Distance from Facing Panels with Time at Station 3119 (from Christina and Wu, 1999).

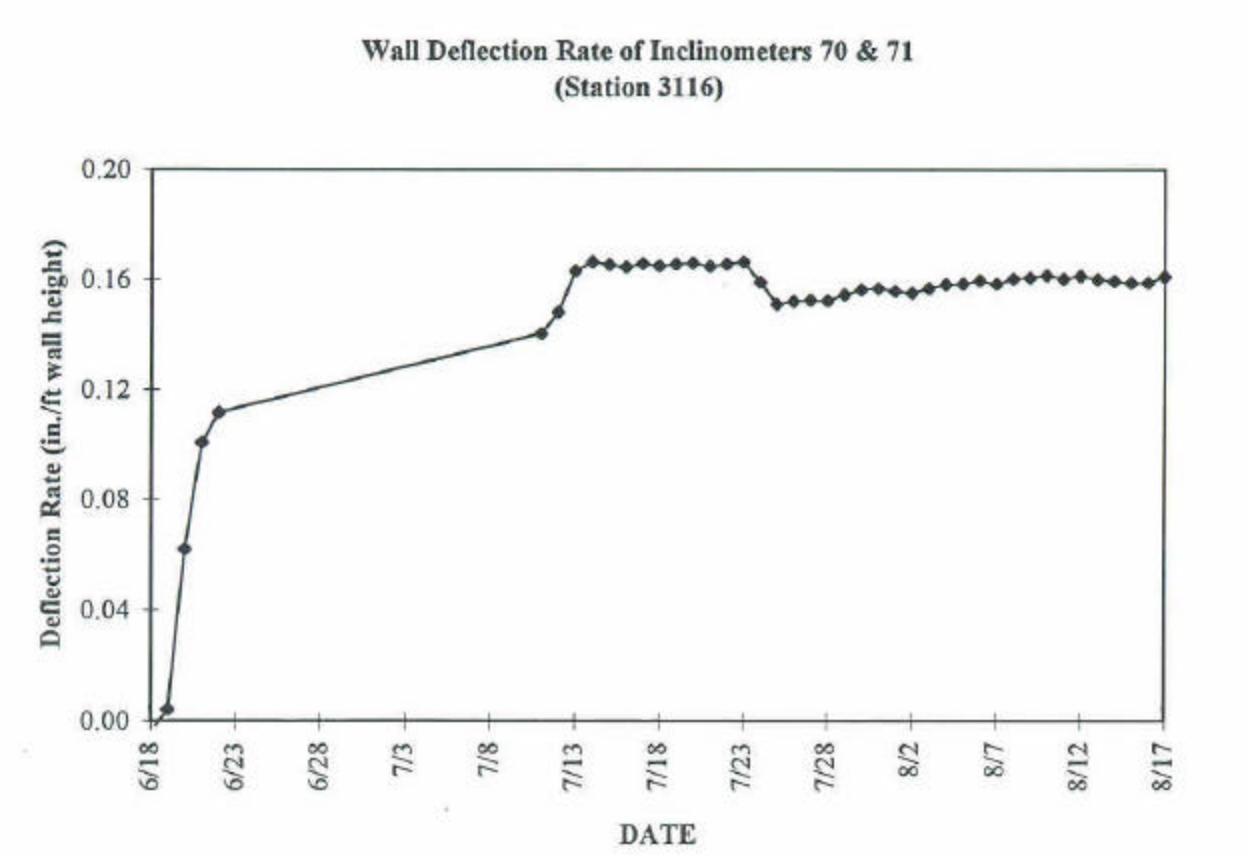

Figure 4.5. Facing Tilt vs. Time for Facing Panel at Station 3116 (from Christina and Wu, 1999).

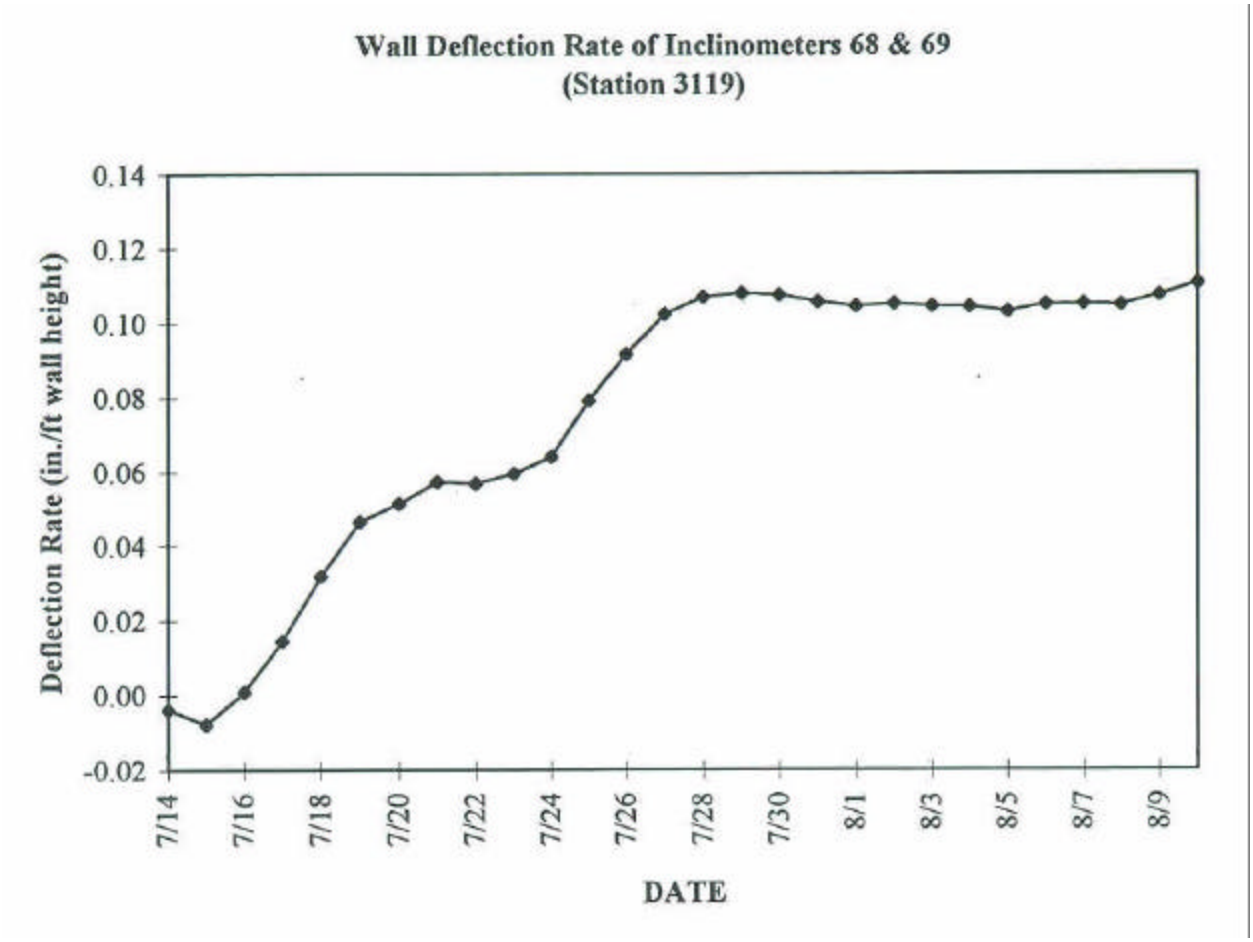

Figure 4.6. Facing Tilt vs. Time for Facing Panels at Station 3119 (from Christina and Wu, 1996).

4.4 Analysis of Movement Results for Facing Panels

By the end of the first half of August 1996, the construction of the wall and the roadway structure was completed and the equivalent applied backfill height above ground level was 18.2 ft at Station 3116 (15.2 ft of backfill behind the facing panels and 3 ft of surface surcharge fill), and 16.1 ft at Station 3119.

All the panel facing displacement results and the corresponding changes in applied vertical loads in term of equivalent applied backfill height are summarized in Table 4.2. The facing tilt from surveying data was calculated as shown in Table 4.2 so that it could be compared with the inclinometer results. In the last column of Table 4.2, the displacement results for the wall facing (settlement or tilt) are normalized with respect to the corresponding applied backfill height. With this normalization, the facing movements results from surveying and inclinometer at Stations 3116 and 3119 could be compared. Results in the last column of Table 4.2 for Stations 3116 and 3119 indicate very close surveying normalized results at Station 3116 and 3119 (0.05 vs. 0.07 and 0.019 vs. 0.016), and inclinometer normalized results (0.013 vs. 0.012). Surveying results seem to suggest larger facing outward displacements than inclinometer, but it can be concluded that the inclinometer results correlate very well with the surveying results. The results in Table 4.2 suggest, also, larger facing outward displacements at Station 3116 than at Station 3119.

Measured vertical profiles of the facing outward displacements along Stations 3116 and 3119 from surveying data collected by the end of construction operations (data shown in Table 4.2) are shown in Figure 4.7. In this figure, the measured outward displacements at different heights were extended linearly to estimate the facing outward displacements at the base and top of the wall facing. The facing top displacements are estimated as 4.5” at Station 3116 and 3.7” at Station 3119. Higher facing top displacement at Station 3116 would be recorded if the reference reading was taken at the beginning of the backfilling operations. Results of Figure 4.7 suggest that the rotation point of the facing panels is close to the wall base for Station 3119 and a little bit below the base for Station 3116. Most likely, small facing translation displacement was experienced at Station 3116.

Table 4.2. Summary of Measured Movements Results for Facing Panels and the Corresponding Applied Backfill Height.

A. Vertical Settlement of the Wall Facing				
Station	Date	Settlement (inch)	Change in Backfill Height (feet)	Average Settlement (inch) per 1ft of Applied Backfill
3116	From 6/18 to 8/5	0.66	13.2	0.05
3119	From 7/1/ to 8/16	1.2	16.1	0.07
B. Outward Displacements of the Wall Facing				
1. Station 3116, Reference Date is 6/18, Change in Backfill Height is 13.2 ft				
From	Location (feet)	Displacement (inch)	Facing Tilt (inch/feet)	Average Facing Tilt Rate ((inch/feet)/feet)
Surveying	H=1	1.02		
	H=4	1.74		
	H=7	2.52		
			0.25	0.019
Inclinometer			0.17	0.013
2. Station 3119, Reference Date is 7/1, Change in Backfill Height is 16.1 ft				
Surveying	H=1	0.78		
	H=4	1.56		
	H=7	2.28		
			0.25	0.016
3. Station 3119, Reference Date is 7/14, Change in Backfill Height is 9.6 ft				
Inclinometer			0.11	0.012

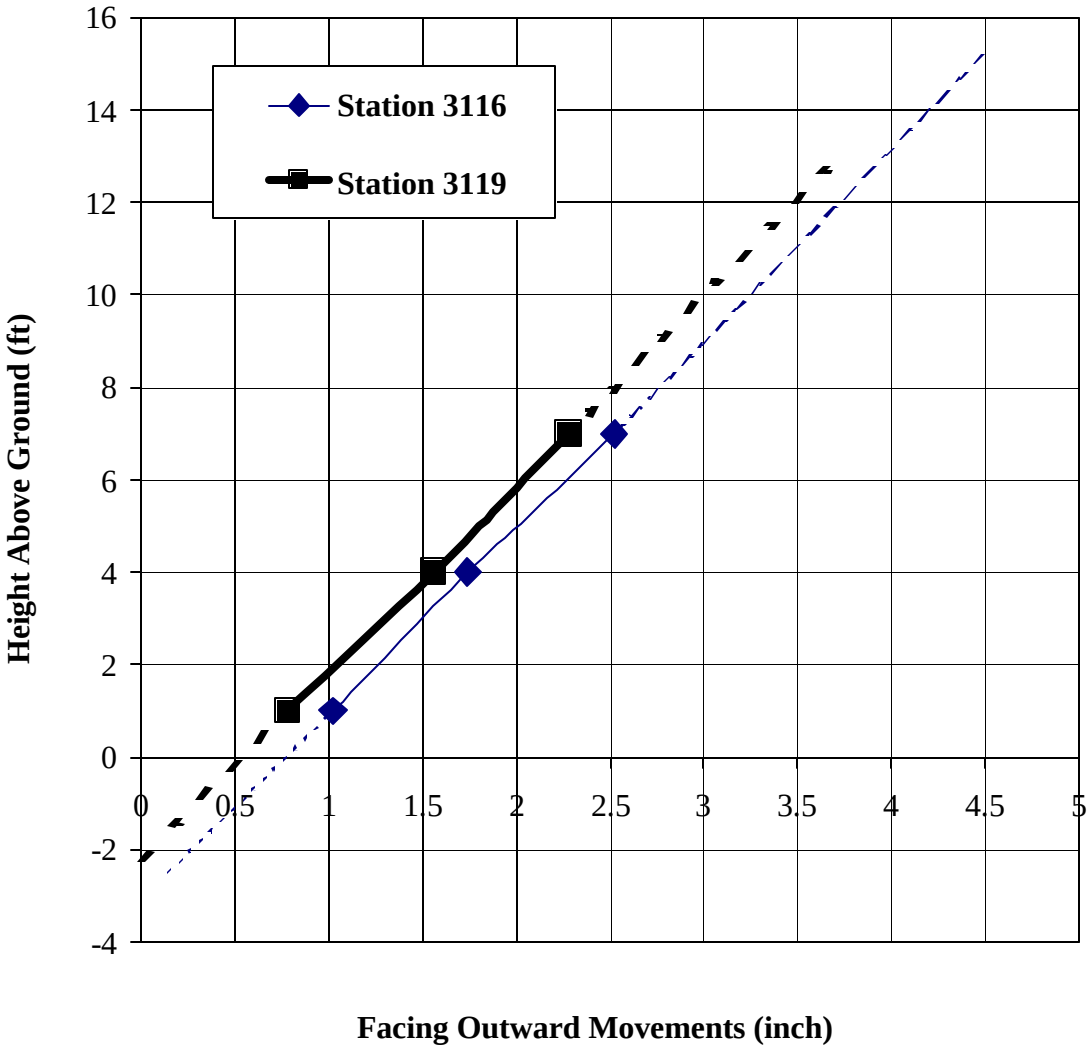

Figure 4.7. Profile of Measured Facing Outward Displacements for Facing Panels at Stations 3116 and 3119.

4.5 Recommended Setbacks for Facing Panels

The discussion presented next assumes that the measured facing settlement and tilt are related linearly to the applied load, the facing panel rotates around the base as a rigid body, and the equivalent backfill height of the roadway structure placed on top of the wall is 3 ft. It should be

also noted that the recommended setbacks in this chapter are valid for field and loading conditions similar or close to those encountered in the I-25/I-70 MSE/IFF wall.

The results in Table 4.2 suggest that the wall facing experienced an average vertical facing settlement of 0.07" due to the placement of 1 ft of backfill. Based on this measured value, expected vertical settlement of panel facing with different heights above ground level is provided in Table 4.2.

It is clear from the results in Figure 4.7 that the specified top setbacks in the plans for the I-25 MSE/IFF wall were much smaller than the measured values. The average measured facing tilt (inch/feet) due to the application of 1 ft of backfill from all the results shown in Table 4.2 is 0.015 (inch/feet)/feet. Using this data and previous assumptions, the following empirical equation can be used to estimate the facing outward displacement (in inches) at the ground level, $D_{bot,}$(bottom setback, see Figure 2.2), and at the top of the wall, D_{Top}, (top setback, see Figure 2.2):

$$D_{bot} = 0.015\ (H+3)*(D) \quad (4.2)$$

$$D_{top} = 0.015\ (H+3)*(D+H) \quad (4.3)$$

Where D is the depth of embedment in ft, H is the facing height above ground level in ft. For different facing heights, and using depth of embedment as suggested in Chapter 2, the recommended bottom and top setbacks from Equations 4.2 and 4.3 are shown in Table 4.3. The recommended top and bottom setbacks in Table 4.3 are larger than what were required in the plans and even during construction. Note that the nature of this wall allows correcting for any excessive setbacks. Therefore, it is recommended to use the values shown in Table 4.3 in future similar projects, until more displacement data become available.

Since there are no facing movement data for an MSE/IFF walls with unbraced facing during construction, it is recommended for such walls to use setbacks equal to 130% of the setbacks recommended for MSE/IFF walls with weakly braced facing (Table 4.4). The close deformation

monitoring of the first six panels for possible adjustment during construction or even reconstructing must be performed.

Table 4.3. Recommended Setbacks for MSE/IFF Wall with Facing Weakly Braced during Construction Operations.

Panel Height above ground Level (feet)	4	10	12	16	20
Expected Panel's Settlement (inch)	0.5	0.9	1.1	1.4	1.6
Panel Bottom Setback (inch)	0.2	0.4	0.5	0.7	1.0
Panel Top Setback (inch)	0.6	2.4	3.2	5.3	7.9

Table 4.4. Recommended Setbacks for MSE/IFF Walls with Unbraced Facing during Construction Operations.

Panel Height above ground Level (feet)	4	10	12	16	20
Expected Panel's Settlement (inch)	0.7	1.2	1.5	1.8	2.1
Panel Bottom Setback (inch)	0.3	0.9	1.1	1.6	2.3
Panel Top Setback (inch)	0.8	3.4	4.6	7.6	11.3

5.0 ASSESSMENT OF THE DESIGN PROCEDURE FOR FACING PANELS AND ANCHORS

5.1 Background

In conventional MSE walls with stiff connection between facing and soil reinforcements, the earth loads on the facing can be as high as the maximum tensile forces in the soil reinforcements (Elias and Christopher, 1997). However, the facing in the I-25/I-70 MSE/IFF wall is not attached to the fill reinforcements. Because the facing and MSE mass are independent, design of the reinforced soil mass and of the facing are effectively divorced, and using the procedure described above is not appropriate. The I-25/I-70 MSE/IFF wall system was designed to be very flexible to mobilize large tensile resistance in the soil reinforcements. The facing anchor is very flexible due to the curvature shape of the anchor bars, which may straighten a bit under load. The facing is not attached to the reinforcements and is allowed to rotate as a rigid body around the base. The adjustable feature of the facing (i.e., loosening the nuts) can be used to control facing movement in order to minimize lateral earth pressure on the wall facing. The flexibility of the wall system allows the reinforced soil mass to support most of the lateral earth load and minimize the lateral earth pressure on the facing panels. According to McGown et. al. (1998), if the lateral boundary of the MSE wall is allowed to yield sufficiently, large tensile resistance in the reinforcements will be mobilized to balance the lateral earth loads and theoretically there will be no earth pressure on the facing. This provides the lower limit case for earth pressure on the facing of flexible MSE wall systems (McGown et. al., 1998). For this case, however, there will be only very small localized stresses near the facing that will develop because each soil layer between the reinforcements layers tends to act independently, causing the wall to be subjected to active earth pressure over the vertical spacing between reinforcement layers (McGown et. al., 1998). A lower limit and uniform earth pressure of 30 psf against wall facing was anticipated by the designer and assumed to be the active and design earth pressure against the wall facing.

5.2 Instrumentation and Analysis Results and Discussion

The locations of the face anchor strain gages are depicted in Figures 3.1 and 3.2 for Stations 3116 and 3119, respectively. Gages at both Stations were placed at three elevations, layer 1 strain gages were placed at the lowest level, layer 2 at the intermediate level, and layer 3 at the

highest level. Tables 5.1 and 5.2 list the measurements collected from face anchor strain gages placed along Stations 3116 and 3119, respectively, at different times and backfill heights above ground level (raw data from Blanks, 1996). Collected digit readings and temperatures (see Equation 3.1) are listed for Station 3116 (Table 5.1) and just digit readings (temperatures data were missing) are listed for Station 3119 (Table 5.2). For layer 1 gages along Stations 3116 and 3119, the first reading was taken when there was 1 ft of backfill over the face anchor gages (see Tables 5.1 and 5.2). For all other gages, the first reading was taken when there was no backfill over the gages (see Tables 5.1 and 5.2). The data were collected manually until backfill height of 16.1 feet for Station 3116 (0.9 ft above top of facing panel) and 12 ft at Station 3119 (1.1 ft below top of facing panel). No reliable data were found after that. The first reading for each gage was taken as the reference reading (R_o and T_o in Equation 3.1), and the backfill height that corresponds to that reading is referred to as the reference height. For Station 3119, because temperature data were not available, the strains developed due to temperature changes (estimated to be very small) were ignored. The microstrain changes in the face anchor rebars at the wall facing from the reference readings were calculated from Equation 3.1 and the calculated results are listed in Tables 5.1 and 5.2. It is clear from Tables 5.1 and 5.2 that the measured microstrains are well below the yield strain of at least 2000 microstrains. Therefore, changes in face anchors forces in units of pounds were calculated using Equation 3.2 by multiplying the measured changes in microstrains values by 9.3.

The tributary widths, heights, and areas for all gages are shown in Table 3.3. The portion of lateral earth pressure that is in equilibrium locally with anchor forces is assumed to be distributed uniformly over the tributary areas. The changes in this lateral earth pressure with changes in applied backfill height (from reference backfill height) over the gages from all gages are summarized in Figures 5.1 through 5.6. Each figure shows also the average measured changes in earth pressure for each layer of gages. Figures 5.1 to 5.6 suggest the results from different gages are very close to each other and close to the average values. This indicates that the measured values are reliable and representative of the actual earth pressures resisted by the face anchors.

The average earth pressures at the bottom of the wall (average of layer 1 gages), middle of the wall (average of layer 2 gages), and top of the wall (average of layer 3 gages) at Stations 3116

and 3119 versus the backfill heights over the gages are shown in Figures 5.7 and 5.8, respectively. In the early stages of backfilling over layer 1 gages (see Figures 5.7 and 5.8), it seems that the face anchors supported smaller earth loads. This could be attributed to the trench support for some of the applied backfill loads placed close to the ground level. Results at Station 3116 indicated that earth pressure on the facing in the last stages of backfill placement became almost constant (after 8 ft of backfill for the middle and upper layers and 3 ft for the bottom layer, see Figure 5.7), but continued to increase at Station 3119 (Figure 5.8). Results of Figure 5.7 for Station 3116 indicate larger earth pressures were experienced in the middle zone of the wall (layer 2) than in the upper or lower zones (i.e., almost uniform distribution of earth pressures). Results of Figure 5.8 for Station 3119 support the triangular distribution of earth pressure on the facing where the measured earth pressure was largest at the bottom of the wall and lowest at the top of the wall. The average earth pressure on the wall facing measured from face anchor strain gages was 28 psf for Station 3116 and 35 psf for Station 3119. Larger earth pressure at Station 3119 than at Station 3116 could be attributed to the smaller facing outward displacement at Station 3119 (see Chapter 4).

Table 5.1. Results Obtained from Face Anchor Strain Gages Placed along Station 3116.

Date in '96	Fill Ht. (ft)	Layer 1 Gages, Placed Ground Level								
		10314			10307			10313		
		Digits	T ^{o}C	Micro Strain	Digits	T ^{o}C	Micro Strain	Digits	T ^{o}C	Micro Strain
6/17	1	6459	20	0	6545	21	0	6551	21	0
6/18	5	6521	23	26	6561	23	8	6568	23	8
6/18	6	6540	23	34	6565	23	9	6598	23	20
6/19	9	6581	23	50	6644	23	40	6646	24	38
6/19	10	6580	24	50	6643	23	40	6652	24	41
6/20	11	6590	24	54	6654	24	44	6647	24	39
6/21	12	6581	24	50	6662	24	47	6648	24	39
6/26	14	6572	23	46	6668	23	49	6633	23	33
6/28	16	6572	23	46	6667	23	49	6630	24	32
		Layer 2 Gages, Placed at Backfill Height of 6 ft								
		10310			**10303**			**10305**		
6/18	6	6442	43	0	6452	37	0	6336	43	0
6/19	9	6643	24	59	6761	24	105	6492	24	42
6/19	10	6707	24	84	6827	24	130	6557	23	66
6/20	11	6756	23	102	6901	23	158	6623	23	91
6/21	12	6801	24	119	6966	23	183	6697	24	120
6/26	14	7015	24	201	7125	24	244	6890	24	194
6/28	16	7005	30	202	7090	26	232	6870	25	187
		Layer 3 Gages, Placed at Backfill Height of 11 ft								
		10316			**10300**			**10301**		
6/20	11	6424	33	0	6301	34	0	6391	24	0
6/21	12	6480	24	13	6428	23	39	6467	23	29
6/26	14	6655	25	80	6560	25	91	6555	25	64
6/28	16	6625	26	70	6540	26	84	6535	26	57

Table 5.2. Results Obtained from Face Anchor Strain Gages Placed at Station 3119.

Date in '96	Fill Ht. (ft)	Layer 1 Gages, Placed at Ground Level							
		10308		10318		10306			
		Digits	Micro Strain	Digits	Micro Strain	Digits	Micro Strain	Digits	Micro Strain
7/1	1	6414	0	6442	0	6438	0		
7/2	1	6422	3	6452	4	6447	4		
7/9	5	6438	9	6450	3	6449	4		
7/12	6	6461	18	6531	34	6542	40		
7/17	7	6528	43	6649	79	6599	61		
7/17	8	6557	54	6679	90	6617	68		
7/24	11	6627	81	6786	131	6653	82		
7/26	12	6660	93	6818	143	6658	84		
		Layer 2 Gages, Placed at Backfill Height of 5 ft							
		10298		10304		10299		10302	
7/9	5	6333	0	6434	0	6317	0	6383	0
7/12	6	6363	11	6489	21	6341	9	6406	9
7/17	7	6427	36	6609	66	6404	33	6462	30
7/17	8	6482	56	6637	77	6431	43	6500	45
7/24	11	6642	117	6784	133	6572	97	6725	130
7/26	12	6718	146	6853	159	6611	112	6780	151
		Layer 3 Gages, Placed at Backfill Height of 9 ft							
		10317		10312		10311			
7/19	9	6431	0	6447	0	6465	0		
7/24	11	6458	10	6515	26	6607	54		
7/26	12	6522	35	6628	69	6720	97		

Table 5.3 Tributary Widths, Heights, and Areas for all Gages.

Gage	Width, (ft)	Height, (ft)	Area, (ft^2)	Gage	Width, (ft)	Height, (ft)	Area (ft^2)
Station 3116				Station 3119			
10300	6	6.7	40.2	10298	6	4.5	27
10301	8	6.7	53.6	10299	8	4.5	36
10303	8	5.5	44	10302	8	4.5	36
10305	6	5.5	33	10304	8	4.5	36
10307	8	3	24	10306	6	2.5	15
10310	8	5.5	44	10308	6	2.5	15
10313	6	3	18	10311	6	5.0	36.6
10314	8	3	24	10312	8	5.0	48.8
10316	8	6.7	53.6	10317	8	5.0	48.8
				10318	8	2.5	20

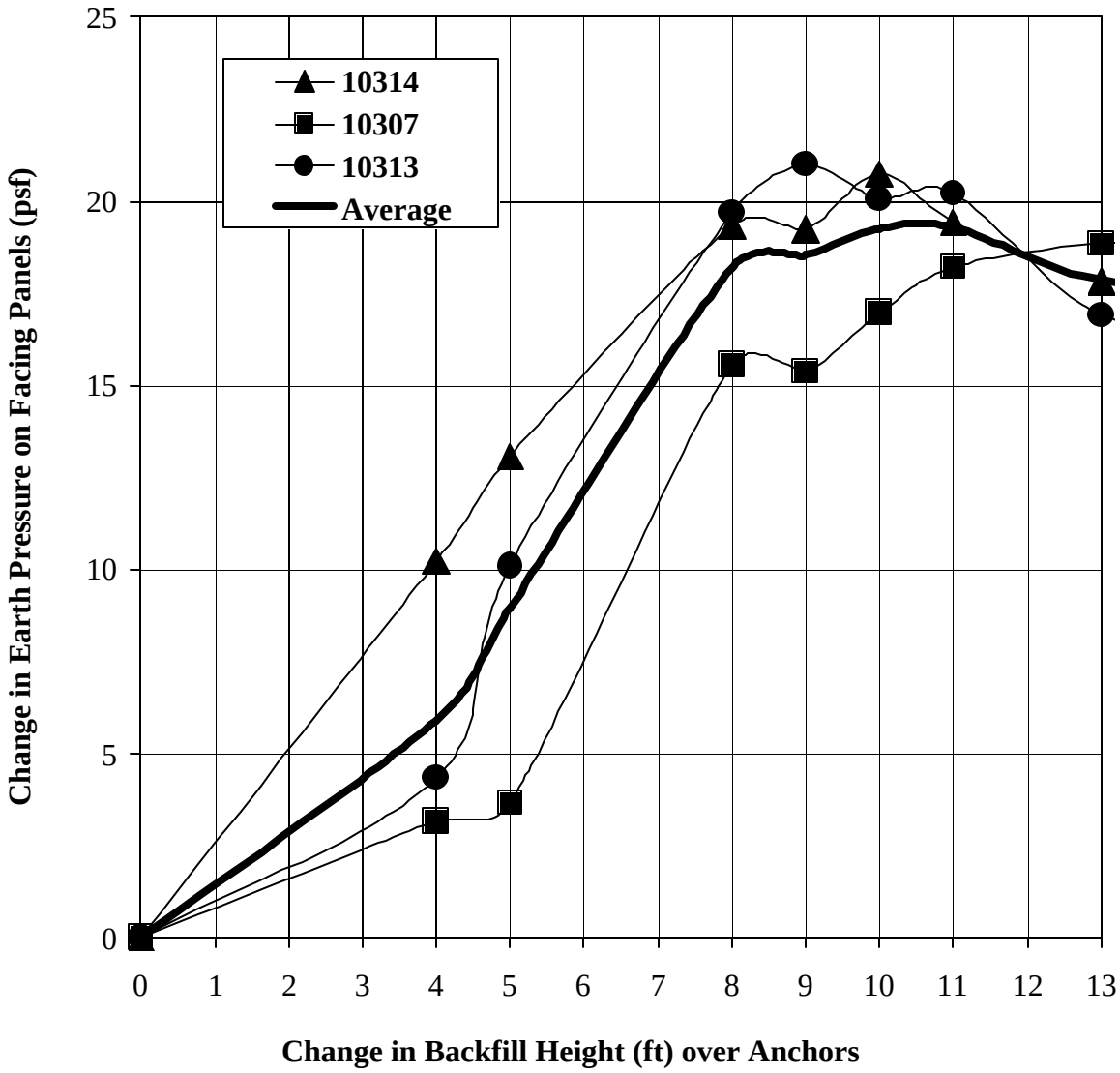

Figure 5.1. Results for Earth Pressures on Facing Panels Measured from Layer 1 Face Anchor Strain Gages Placed along Station 3116 at H= 0 ft (Reference Reading at H= 1 ft).

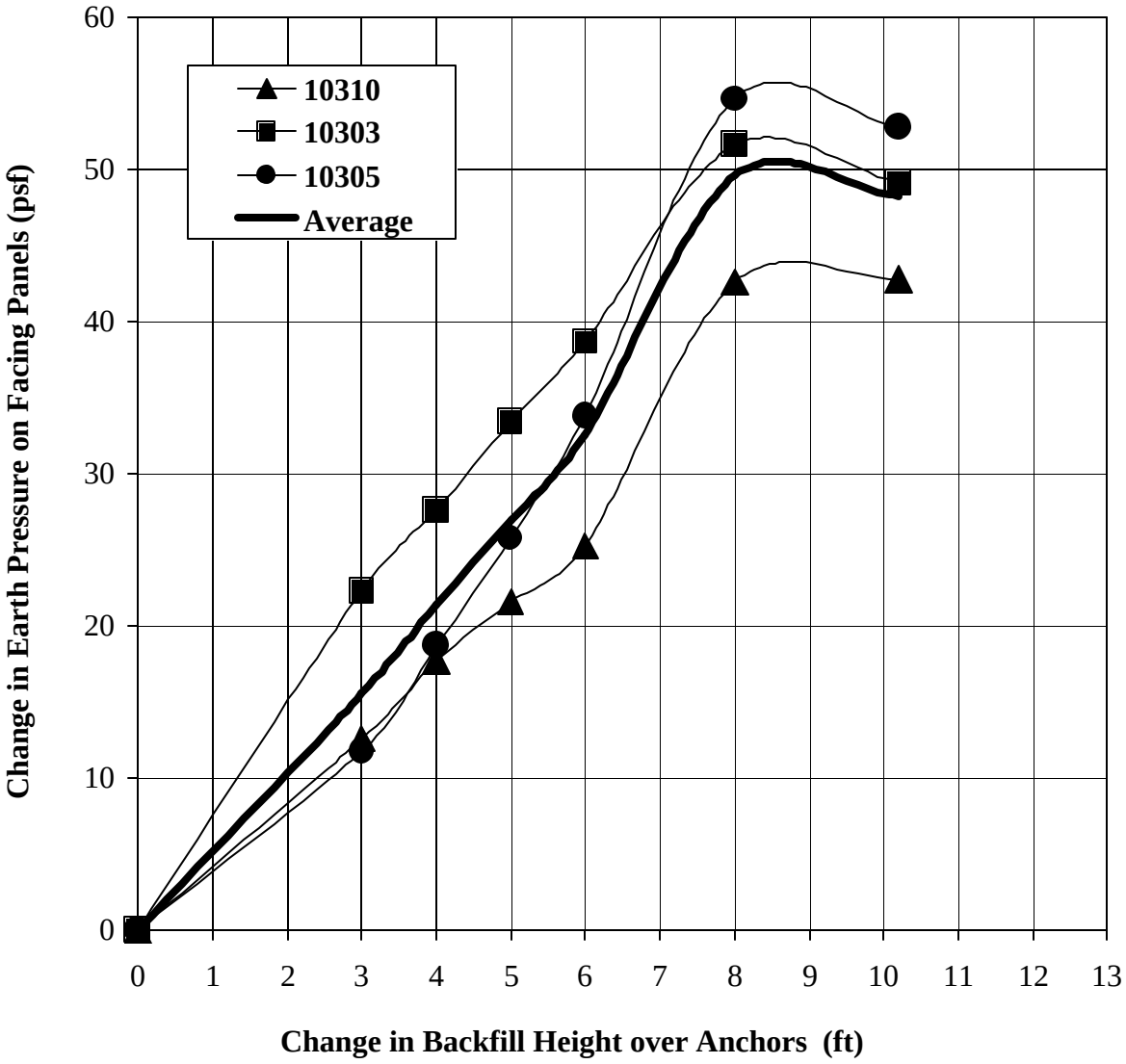

Figure 5.2. Results for Earth Pressures on Facing Panels Measured from Layer 2 Face Anchor Strain Gages Placed along Station 3116 at H= 6 ft (Reference Reading at H= 6 ft).

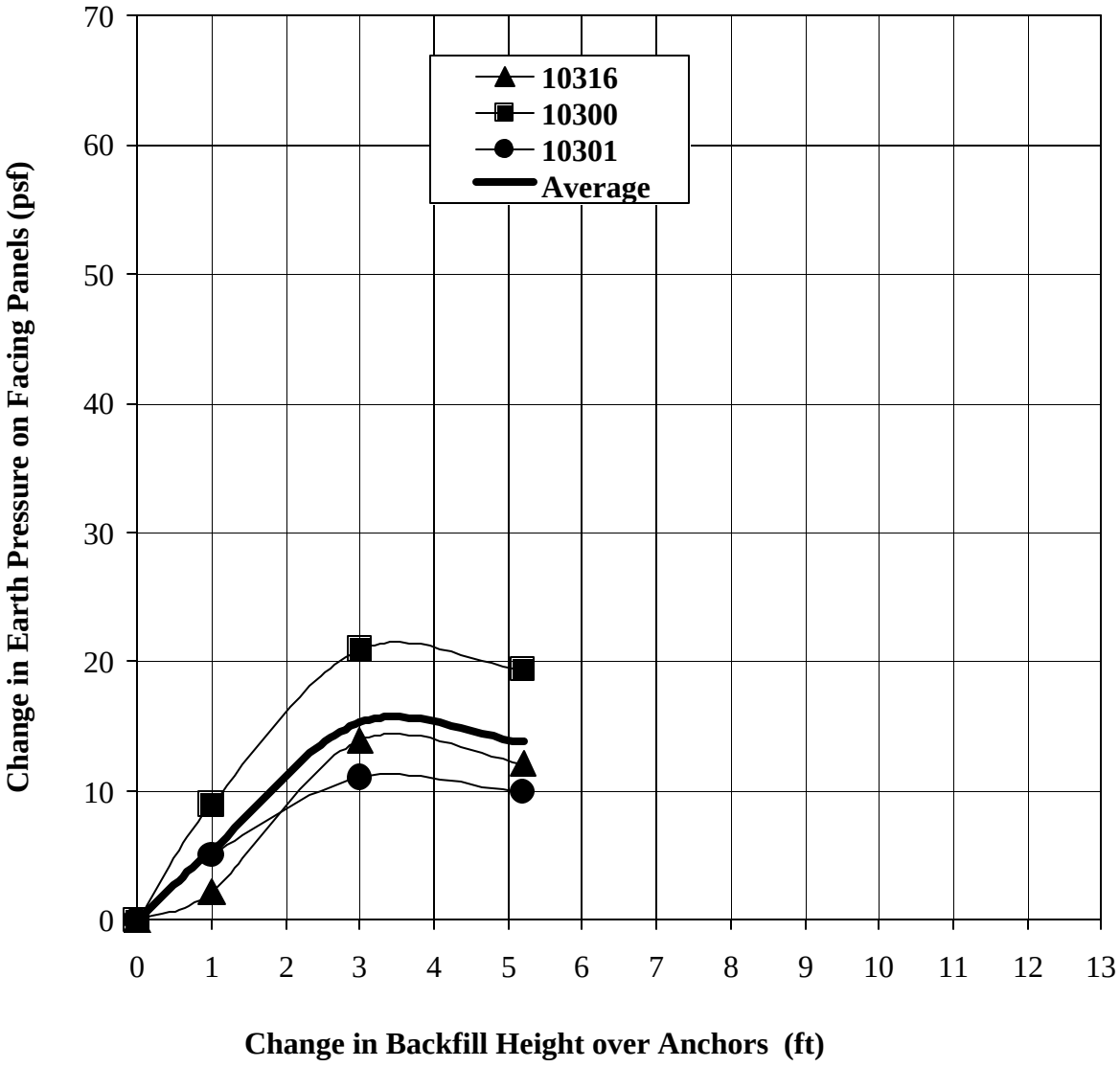

Figure 5.3. Results for Earth Pressures on Facing Panels Measured from Layer 3 Face Anchor Strain Gages Placed along Station 3116 at H= 11 ft (Reference Reading at H= 11 ft).

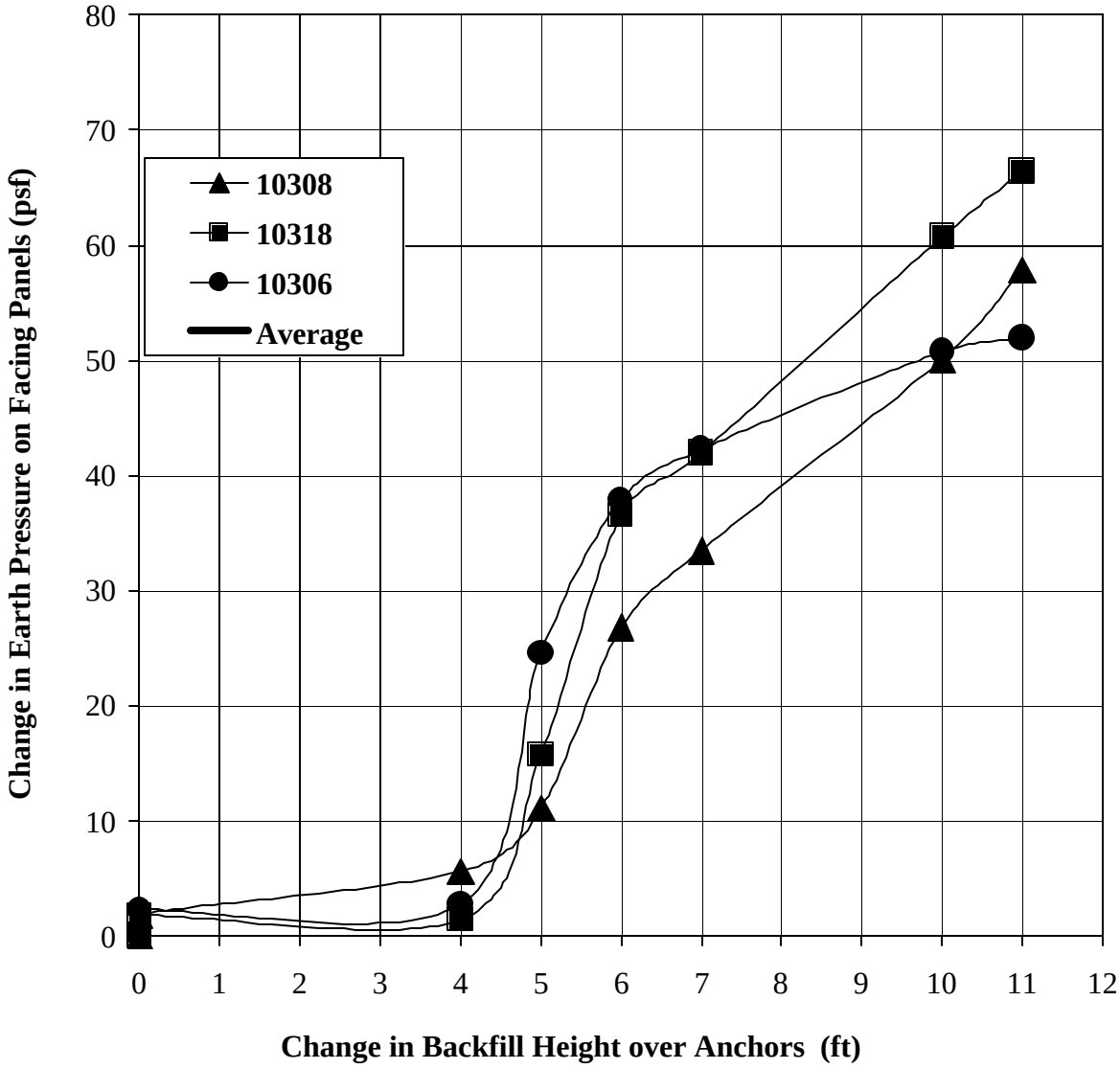

Figure 5.4. Results for Earth Pressures on Facing Panels Measured from Layer 1 Face Anchor Strain Gages Placed along Station 3119 at H= 0 ft (Reference Reading at H= 1 ft).

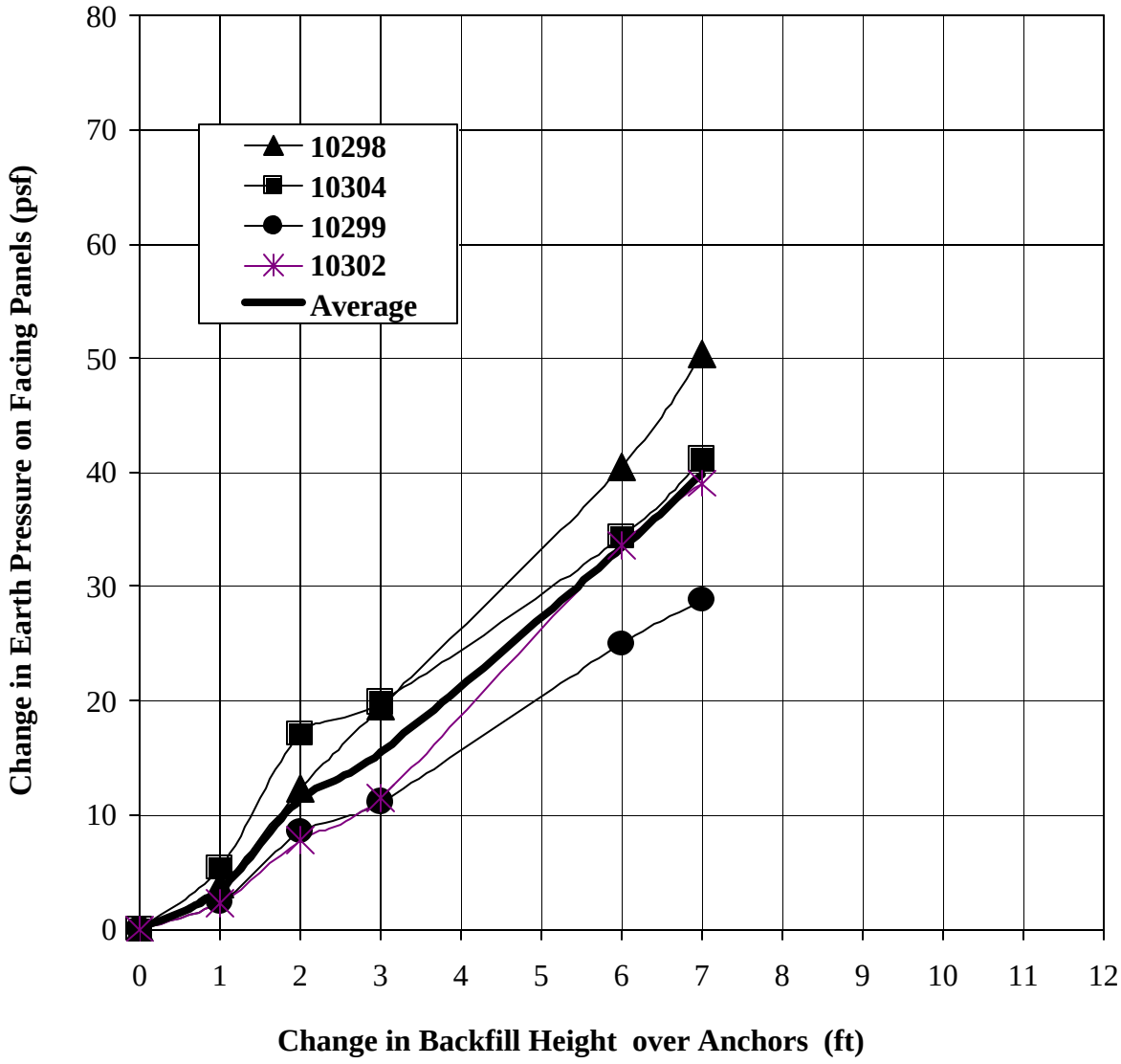

Figure 5.5. Results for Earth Pressures on Facing Panels Measured from Layer 2 Face Anchor Strain Gages Placed along Station 3119 at H= 5 ft (Reference Reading at H= 5 ft).

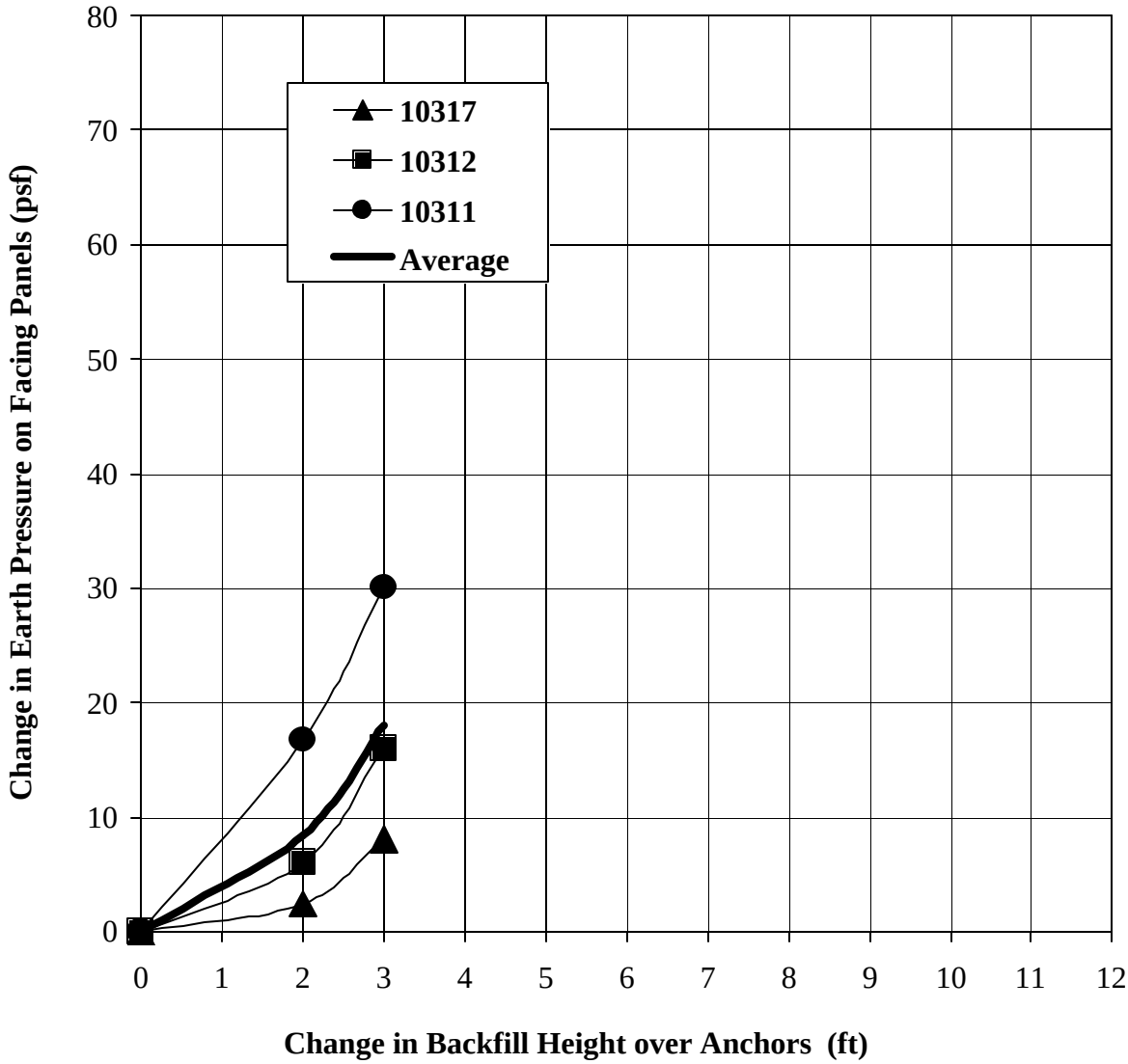

Figure 5.6. Results for Earth Pressures on Facing Panels Measured from Layer 3 Face Anchor Strain Gages Placed along Station 3119 at H= 9 ft (Reference Reading at H= 9 ft).

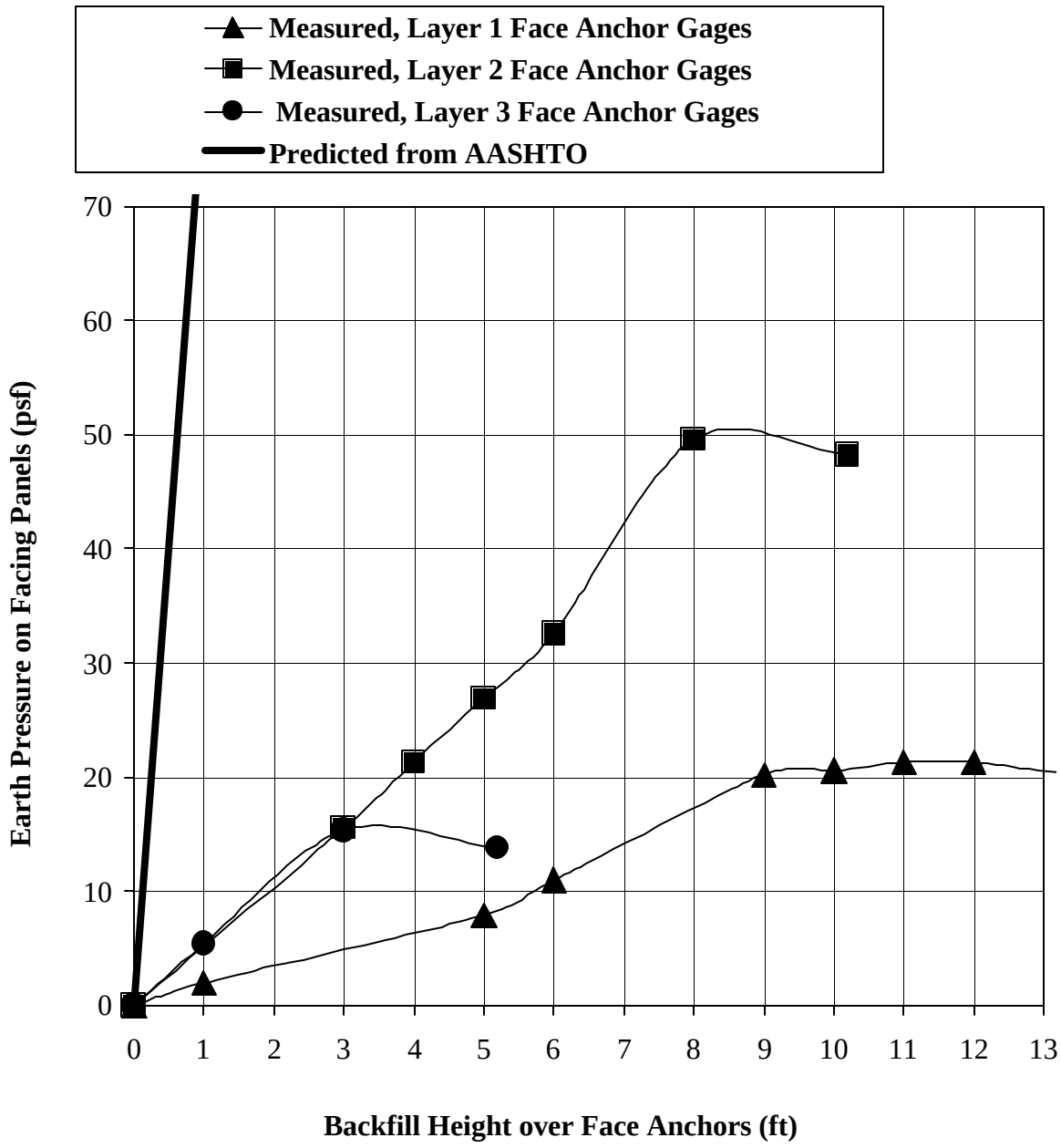

Figure 5.7. Measured and AASHTO Predicted Earth Pressure on Facing Panel of Station 3116.

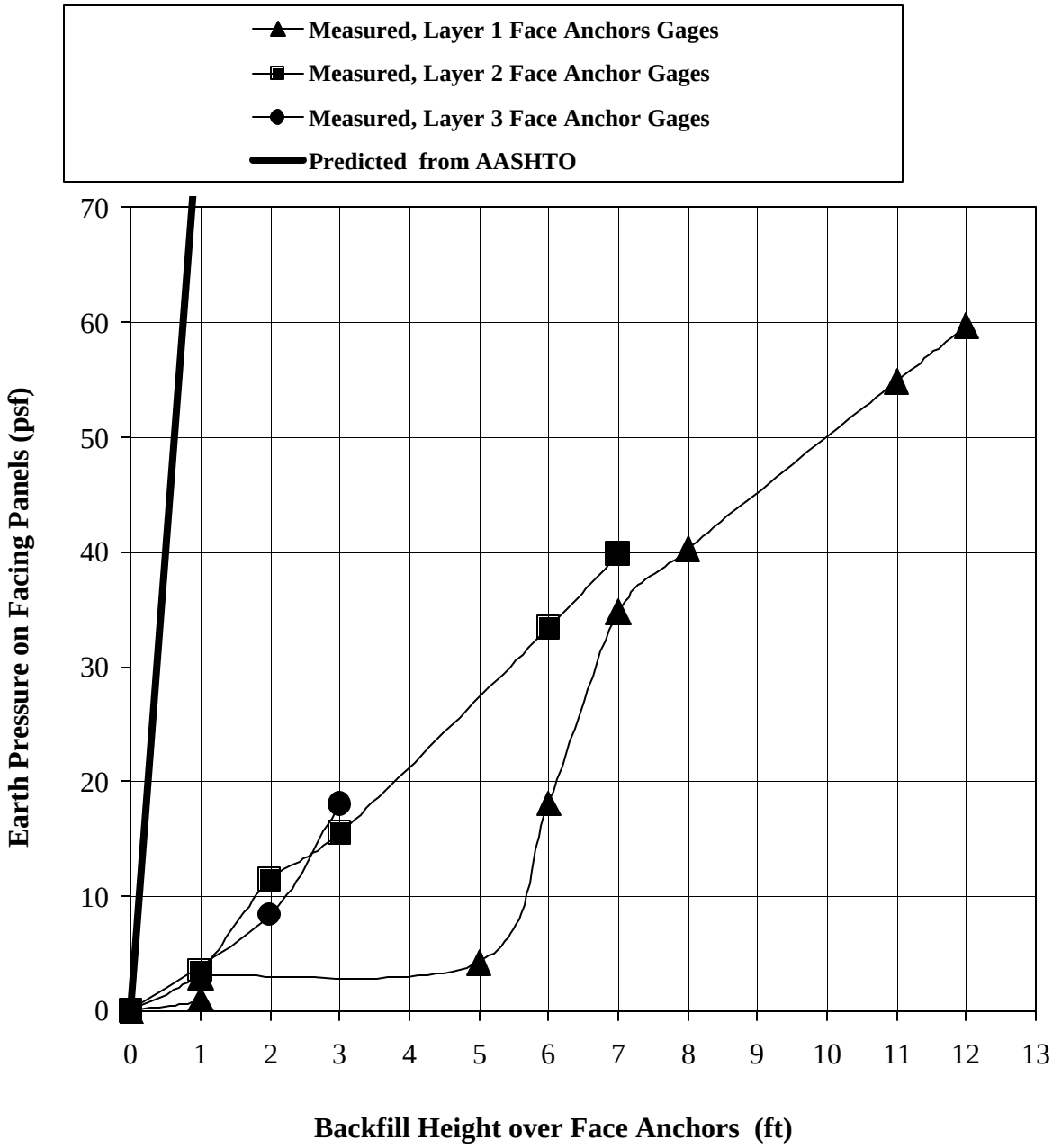

Figure 5.8. Measured and AASHTO Predicted Earth Pressure on Facing Panel of Station 3119.

5.3 Assessment of the Design Procedure for Facing Anchors

At the end of the monitoring stage (backfill height is 16.1 ft at Station 3116 and 12 ft at Station 3119), Table 5.4 shows the measured tensile forces in face anchors, depth of backfill over the face anchor (z in Equations 2.2 and 2.3) and estimation of the facing anchor pullout capacity (see Eqs. 2.2 and 2.3). Note that the yield strength of the facing anchor rebars (18600 pounds) is much higher than the pullout capacity of anchors and therefore will not govern the stability analysis. Also shown in Table 5.4 is the estimated local factor of safety for each gage against pullout calculated as the estimated anchor pullout capacity over the measured tensile load in that anchor. All the bottom anchors of Stations 3116 and 3119 (layer 1) show a very high factor of safety against pullout of face anchors. Most of the middle (layer 2) and top anchors (layer 3) at Stations 3116 and 3119 show a factor of safety less than the tolerable value of 1.5. The factor of safety is highlighted for anchors with a factor of safety less than or close to one. Seven anchors have a factor of safety less than 1 and an additional two have a factor of safety very close to one. For these 9 anchors, with a factor of safety less than or close to 1, the results suggest local pullout problem at these anchors. This response correlates very well with the significant deformations of the wall facing noticed during construction of the upper zone of the wall, attributed to possible slippage of the anchors. The gross factor of safety against complete pullout failure of all anchors, calculated as total pullout capacity of all instrumented anchors over total measured loads in all instrumented anchors, is 1.89 at Station 3116 and a low value of 1.35 at Station 3119.

Table 5.4. Measured Tensile Forces and Pullout Capacity of Facing Anchors at the Ed of the Monitoring Program

Measured Tensile Load (pounds)	Pullout Capacity (pounds)	Factor of Safety	Measured Tensile Load (pounds)	Pullout Capacity (pounds)	Factor of Safety	Measured Tensile Load (pounds)	Pullout Capacity (Pounds)	Factor of Safety
Sta. 3116, Layer 1 of Instrumented Facing Anchors, Depth of Fill above anchors is 15 ft								
10314			**10307**			**10313**		
480	2970	6.2	504	2970	5.9	342	2625	7.7
Sta. 3116, Layer 2 of Instrumented Facing Anchors, Depth of Fill above anchors is 10 ft								
10305			**10303**			**10310**		
1740	1750	1.0	2159	1980	0.92	1881	1980	1.05
Sta. 3116, Layer 3 of Instrumented Facing Anchors, Depth of Fill above anchors is 5 ft								
10301			**10300**			**10316**		
531	990	1.9	779	875	1.1	650	990	1.5
Sta. 3119, Layer 1 of Instrumented Facing Anchors, Depth of Fill above anchors is 12 ft								
10306			**10318**			**10308**		
795	2100	2.6	1348	2376	1.8	885	2100	2.6
Sta. 3119, Layer 2 of Instrumented Facing Anchors, Depth of Fill above anchors is 7 ft								
10298 (10304)			**10299**			**10302**		
1361 (1480)	1225 (1386)	0.9 (0.9)	1037	1386	1.3	1405	1386	1.0
Sta. 3119, Layer 3 of Instrumented Facing Anchors, Depth of Fill above anchors is 3 ft								
10317			**10311**			**10312**		
322	594	1.8	901	525	0.6	640	594	0.93

5.4 Assessment of the Design Procedure for Facing Panels

The induced lateral earth pressures against the wall facing are resisted by both facing anchors and the trench. The portion of lateral earth pressure from fill that is in equilibrium locally with anchor forces only will be considered in this section. Therefore, this lateral earth could be less than the actual earth pressure against facing panels.

Shown in Figures 5.7 and 5.8 predictions for earth pressure on facing as given in AASHTO 5.8.4.1C for MSE walls with earth loads on the facing panel equal to the reinforcement maximum tensile forces. The measured lateral earth pressures were way below the predicted values from AASHTO. The maximum measured earth pressure on the wall was 67 psf and the average value was 32 psf. These results indicate the measured earth pressures are very close to

the lower limit earth pressure of 30 psf, expected in the design. The very low measured earth pressure on the facing implies that the I-25/I-70 MSE/IFF wall functioned as planned in the design. The flexibility of the MSE wall system accommodated the wall deformations and thereby reduced earth pressures on panels to a very small value. The flexibility of the MSE/IFF wall system accommodated the large facing deformation resulting from heavy compaction of backfill close to the facing. This also minimized the development of large compaction-induced pressures on the facing.

Vertical and transverse moments in the panels are estimated from anchor forces and the portion of lateral earth pressure that is in equilibrium locally with anchor forces. For a 1 ft wide strip along the panels of Station 3116, Figure 5.9 shows the average anchor support per 1 ft at different levels (from Table 5.4 results) and average earth pressure, *w*, (last data points of Figure 5.7) within the tributary height of each layer of gages (see Table 5.3). For a 1 ft wide strip along the panels of Station 3119, the average anchor support was 150 pounds/ft at the lower level (H= 0 ft), 180 pounds/ft at the middle level (H= 5 ft), and 90 pounds/ft at the upper level (H= 9 ft). The earth pressure, *w*, was 60 psf from H=0 to 2.5 ft, 40 psf from H=2.5 to 7 ft, and 18 psf from H= 7 to 12 ft.

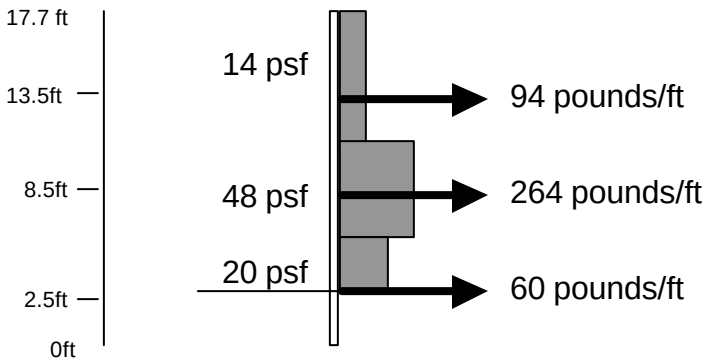

Figure 5.9. Average Anchor Forces and Earth Pressures along Panels of Station 3116.

A peak moment in the vertical direction due to earth pressure is expected to occur at the ground level in the backside of the panels. Based on Figure 5.9, this moment can be calculated as 94x 0.85-264x 0.25+60 x 1.5= 104 pounds-ft/ft at Station 3116 and 188 ft-pound/ft at Station 3119. These values are way below the panel moment capacity of 7800 pounds-ft/ft. For the I-25/I-70

wall, the depth of embedment for the trench was calculated to ensure adequate moment resistance to support a wind load of 8 psf on facing panels. The trench moment resistance is at least 924 pounds-ft/ft at Station 3116 and 687 pounds-ft/ft at Station 3119. The estimated moments due to earth pressures are also way below the moment support of the trench. This will ensure adequate global margin of safety against overturning of the entire facing system (anchors and panels).

Transverse bending is greatest at the vertical centerline of a panel. The peak transverse moment along an 8 ft wide panel can be calculated as $wl^2/8 = 8w$. The peak transverse moment in the panels is 8 x 48= 348 pounds-ft/ft at Station 3116 (middle level) and 480 pounds-ft/ft at Station 3119 (lower level). These values are much smaller than the moment capacity of the panels in the transverse direction of 2000 pounds-ft/ft (in the front side of the panel). The facing panels are at least four times stronger in bending due to lateral earth pressure than needed. This large margin of safety is needed to cover for uncertainties regarding the earth pressures that might develop against the facing panels. As discussed before, the actual earth pressure against the facing panels could be larger than those estimated in this study.

The greatest bending moment in panels probably occur during their handling and placement in the trench. Under worst conditions, the safety factor for construction handling is about 1.7. This is still quite good.

6.0 ASSESSMENT OF THE DESIGN PROCEDURE FOR THE REINFORCED SOIL MASS

6.1 Background

The reinforced soil mass of the I-25/I-70 MSE/IFF wall with no connections to the facing must be self-stable. This is because the soil reinforcements are not attached to the facing, and therefore, the panels do not provide anchorage for reinforcement tension. The design of the reinforced soil mass followed AASHTO guidelines for conventional MSE walls with inextensible reinforcements (see Elias and Christopher, 1997). The vertical earth pressure, σ_v, within the reinforced soil mass is induced by gravity forces due to the backfill self-weight and uniform surcharge load, q, due to the traffic load (taken as 2 ft of backfill) and the roadway structure (assumed 1 ft of backfill). The vertical earth pressure, σ_v, and horizontal earth pressure, σ_h, at a depth z below the top of the facing were conventionally estimated as follows:

$$s_v = g\,z + q \quad \text{.....(6.1)}$$

$$s_h = K_a\,K\,s_v \quad \text{.....(6.2)}$$

where K_a is the active earth pressure coefficient, estimated as 0.28 for a soil having a friction angle of 34 degrees, and K is a multiplier. The value of K for WWF changes linearly from 2.5 at the top of the facing panel to a constant value of 1.2 at 18 ft below the top of the facing panel (Elias and Christopher, 1997). Note that this multiplier is 1 for geosynthetic reinforcements. The values for K were obtained based on measured field data (Elias and Christopher, 1997) suggesting high tensile forces in inextensible reinforcements especially near to the surface of the soil layer. Values of K larger than 1 can be attributed to the influence of compaction and high stiffness of the WWF reinforcements. Compaction creates large lateral strains within the soil mass and reinforcements that remains locked-in after the compaction loads are released (Abu-Hejleh et al., 2000). The influence of compaction is the highest at the soil layer surface and decreases with the depth. The compaction influence is expected to be relatively higher with inextensible reinforcements (such as WWF) due to their higher stiffness values. The high reinforcement stiffness does not allow the development of large enough lateral tensile soil strains

(see McGown et al., 1998) and mobilization of the soil friction resistance, resulting in higher earth loads carried by the reinforcements.

The maximum estimated tensile force in the WWF in the design was 894 pounds, which is less than the WWF allowable tensile steel load of 1008 pounds (48% of the yield force). The extended reinforced soil zone in the upper zone of the wall (Figure 2.2, 20 ft length of reinforcements larger than 11.3 ft required in the design specification) was considered to enhance the overall stability of the structure (i.e., sliding, overturning, bearing, and reinforcement pullout) and to compensate for the use of less reinforcement (8 ft) in the lower zone.

6.2 Instrumentation Results from the WWF Strain Gages

The locations of the wire mesh strain gages are depicted in Figures 3.3 and 3.4 for Stations 3116 and 3119, respectively. Strain gages along Station 3116 were placed at four levels (or elevation above ground level defined as H): layer 1 gages at the lowest level (H= 1 ft), layer 2 at the second lowest level (H= 4 ft), layer 3 at the second highest level (H= 7 ft), and layer 4 at the highest level (H= 11 ft). Three gages (R-1, R-2, and R-3) were placed along station 3119 (Figure 3.4). Tables 6.1 and 6.2 list the location of each gage above ground level (H), and the microstrains readings (raw data from Blanks, 1996) collected from each gage along stations 3116 and 3119, respectively. These data were collected at different times and for different backfill height as shown in the tables. Note that readings for each gage are stored in rows and the column headings refer to the backfill height and date at which these readings were collected. For layer 1 and layer 4 gages along station 3116 and Gage R-3 along Station 3119, first reading was taken when there was no backfill over the gages (Tables 6.1 and 6.2). For layer 1 gages, the second reading was collected when 2ft of backfill was placed over that layer. For layer 4 gages and Gage R-3, the second reading was collected when 1 ft of backfill was placed over that layer. For layers 2 and 3 gages (station 3116) and Gage R-2 (station 3119), the first reading was taken when there was 1 ft of backfill over the gages (Tables 6.1 and 6.2). For Gage R-1, the first reading was taken when the backfill was 4 ft over that gage. Microstrain data from the wire mesh strain gages were collected till the backfill height was 16 ft for station 3116 (~1 ft above top of the facing panel) and 12 ft for station 3119 (1 ft below top of facing panels). No reliable data were found after that.

Table 6.1. Measured Results Collected from WWF Strain Gages Placed along Station 3116.

When Readings were Collected	Date	6/17	6/17	6/18	6/18	6/19	6/19	6/20	6/20	6/21	6/25	6/26	6/28
	Backfill Height (ft)	1	3	5	6	9	10	11	12	12	13	14	16
Gage #	Loc. Above Ground (ft)	Readings in Microstrains											
1-A-S	1	4942	6172	6279	6327	6432	6452	6466	Missing Data			6457	6455
1-B-N	1	5016	5089	5204	5253	5334	5367	5400	5420	5428	5479	5497	5512
1-B-S	1	6264	6291	6315	6387	6411	Range Exceeded						
1-C-S	1	3925	4206	4215	4237	4269	4267	4260	4262	4269	4263	4265	4271
1-C-N	1	4904	6288	6245	6212	6244	Range Exceeded						
2-A-S	4			2139	2060	2134	2150	2175	2189	2208	2249	2277	2290
2-A-N	4			4170	4193	4333	4365	4373	4401	4387	4445	4468	4461
2-B-S	4			2012	2017	2026	2023	2025	2016	2024	2040	2032	2034
2-B-N	4			1543	1595	1677	1687	1693	1700	1696	1728	1724	1719
2-C-S	4			5279	5216	5175	5174	5184	5185	5185	5190	5187	5182
2-C-N	4			6403	6528	6681	6700	6696	6699	6692	6707	6700	6691
3-A-N	7				725*	854	875	892	905	914	859	860	858
3-B-S	7				765*	805	908	996	1022	1034	1091	1099	1108
3-B-N	7				1090*	1105	1155	1190	1194	1211	1250	1251	1255
3-C-N	7					2117	2200				2220	2230	2231
4-A-S	11							722	1678	1686	1670	1606	1587
4-A-N	11							1644	2644	2611	2923	2978	2965
4-B-S	11							2810	2541	2596	2679	2766	2797
4-B-N	11							1978	4965	4912	4724	4717	4717
4-C-S	11							2672	6588	6563	7286	7450	7443
4-C-N	11							2245	4934	4965	5004	5087	5091
4-D-S	11							3633	3842	3846	4029	4063	4043
4-D-N	11							2431	2976	2984	3032	3055	3060

* These readings were taken on June 18, 1996 when the Backfill height was 8 ft not 6 ft shown in the column heading

Table 6.2. Measured Results Collected from WWF Strain Gages Placed on Station 3119.

When Readings were Collected	Date	6/17	6/17	6/18	6/18	6/19
	Backfill Height (ft)	6	7	8	11	12
Gage #	Gage Location above Ground (ft)	Readings in Microstrains				
R-1	1	2641	2642	2636	2575	2538
R-2	5	1076	1023	1007	980	933
R-3	7		1413	1972	2485	2635

The reference reading for each gage is highlighted in Tables 6.1 and 6.2 and the reference backfill height corresponds to that reading. The calculated changes in wire mesh strains using Equation 3.3 versus the changes in backfill height are shown in Figures 6.1 through 6.5. Most of the gages experienced increasing strains readings value as backfill was placed, indicating development of tensile forces in the wire mesh as expected. At the end of the monitoring program, all gages registered net increase in wire mesh tensile load (except gages R-2 and R-3). Gages 1-C and 4-B suggest that the wire mesh at the location of these gages experienced *very small* compressive strains changes during the first few measurements only (see Figures 6.1 and 6.4). If the initial readings for these gages were taken as the reference (available in Table 6.1), these gages would suggest an overall large net tensile load in the wire mesh. Gages R-2 and R-3 (Figure 6.5) measurements and Gage 2-A in the third measurement also suggest that the wire mesh experienced *very small* compressive strains changes. For these gages, the initial readings (with no fill over the gages) are not available. Therefore, it is hard to conclude that wire mesh at the location of these gages was subjected to net compressive strain at any stage. The reduction of the wire mesh tensile load at some stages could be attributed to bending of the wire mesh, redistribution of wire mesh strains, and slippage of the reinforcements.

6.3 Data Analysis

The analysis of the measured WWF strain data require the determination of the value and location of maximum tensile axial strain experienced by each WWF layer. The results shown in Figures 6.1 through 6.5 can only be used to find changes in WWF strains after the placement and compaction of the first 1 ft (2 ft for layer 1 gages of station 3116). Changes in wire mesh strains

during the first 1 ft (2 ft for layer 1) are still needed to estimate the total maximum tensile strains experienced by the WWF layers. Judgment based on the results shown in Figures 6.1 through 6.5 was made to estimate axial strain changes in WWF layers during placement and compaction of the first 1 ft of backfill (2 ft for layer 1 gages) over these layers. For layer 1 gages, it was concluded that wire mesh at the location of Gage 1-A-S experienced the largest tensile strain with up to 200 microstrains occurring during placement and compaction of the first 2 ft of backfill over that layer. For layer 2 gages, it was concluded that wire mesh at the location of Gage 2-A experienced the largest tensile strain with up to 100 microstrains occurring during placement and compaction of the first 1 ft of backfill over that layer. For layer 3 gages, it was concluded that wire mesh at the location of Gage 3-B experienced the largest tensile strain with up to 250 axial microstrains occurring during placement and compaction of the first 1 ft of backfill over that layer. For layer 4 gages, it was concluded that wire mesh at the location of Gage 4-C experienced the largest tensile strains with up to 600 microstrains occurring during placement and compaction of the first 1 ft of backfill over that layer. Along station 3119, it was concluded that that the wire mesh at the location of Gage R-3 experienced up to 300 microstrains during placement and compaction of the first 1 ft of backfill over that gage.

The value and location of the maximum total strains experienced by the wire mesh layers versus the total backfill height over these layers are shown in Figure 6.6. Figure 6.6 suggests that the measured changes in wire mesh axial strains are well below the yield strain of at least 2000 microstrains. Therefore, the maximum tensile forces in wire mesh layers in pounds/ft were calculated by multiplying the measured microstrain values shown in Figure 6.6 by a factor of 0.87.

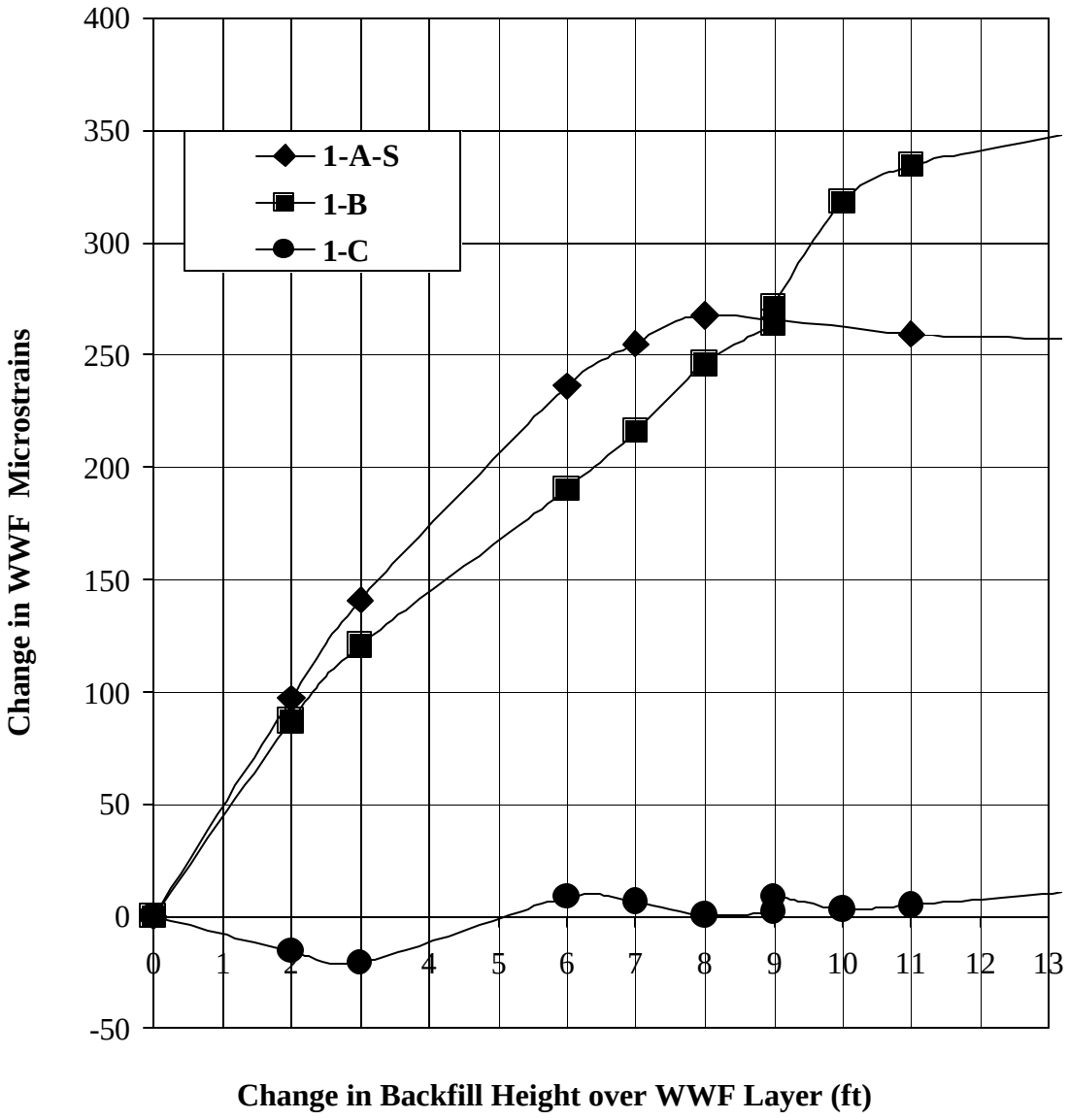

Figure 6.1. Results of WWF Layer 1 Strain Gages Placed at H= 1 ft (Reference Reading at H= 3 ft).

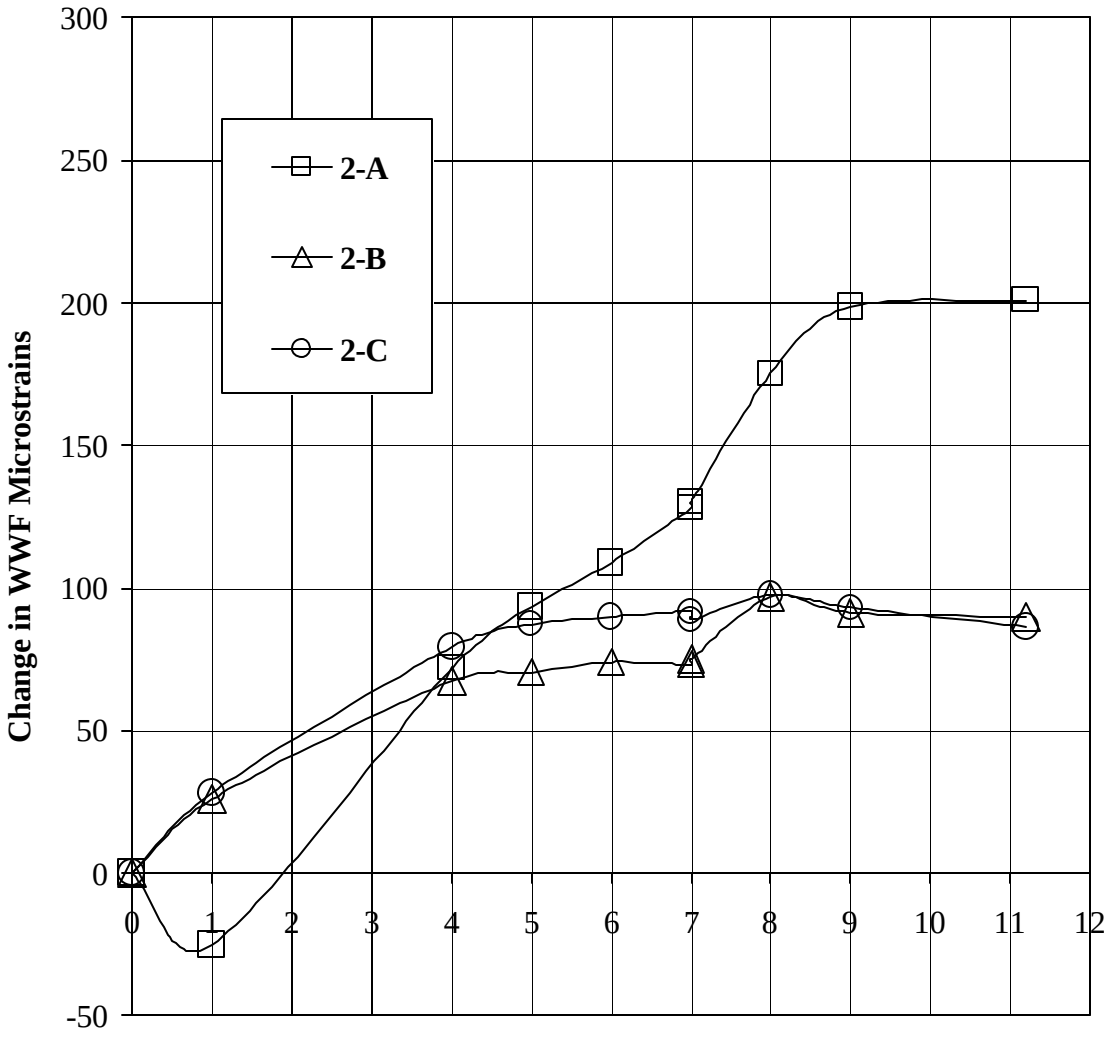

Figure 6.2. Results of WWF Layer 2 Strain Gages Placed at H= 4 ft (Reference Reading at H= 5 ft).

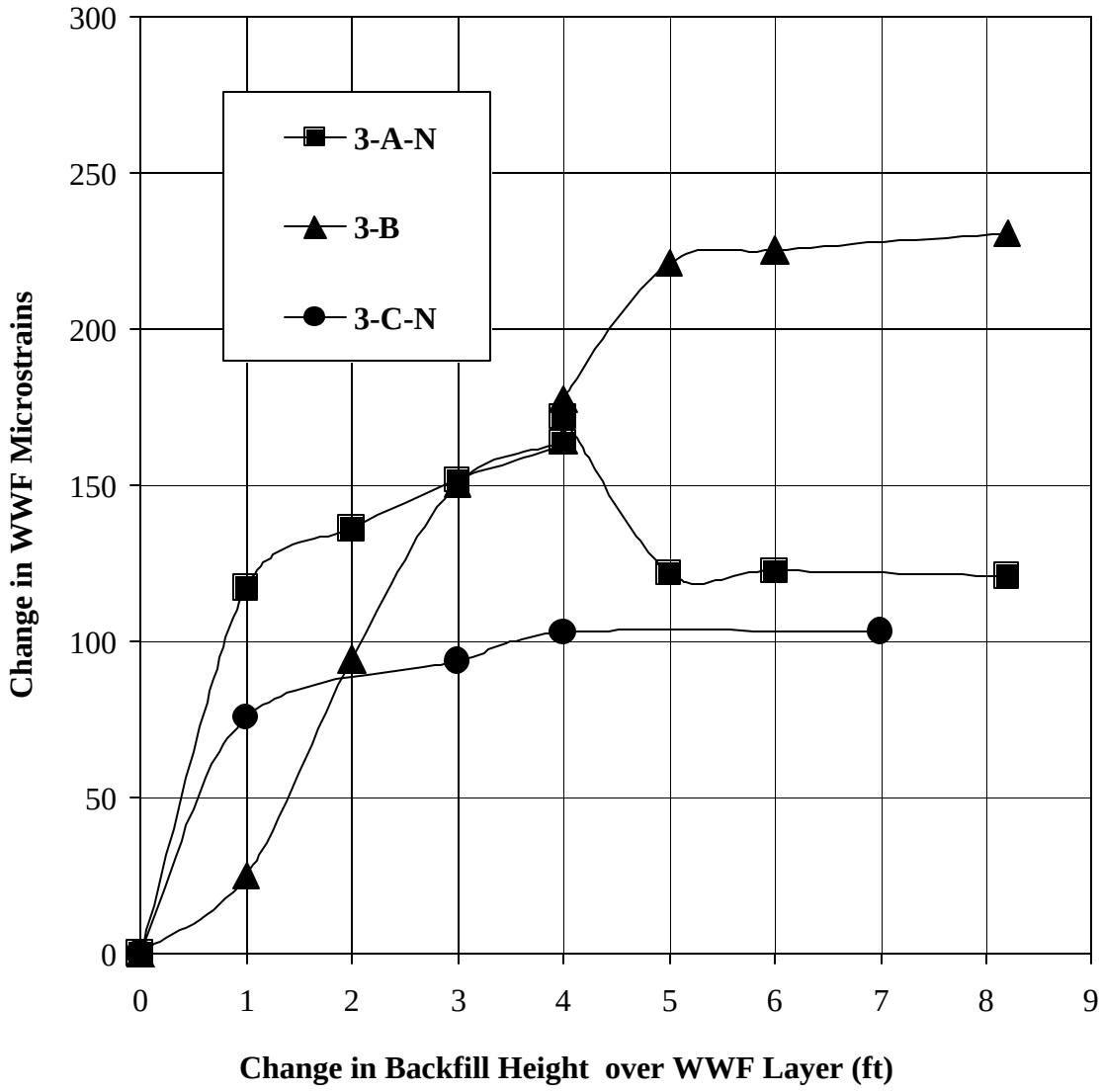

Figure 6.3. Results of WWF Layer 3 Strain Gages Placed at H= 7 ft (Reference Reading at H= 8 ft).

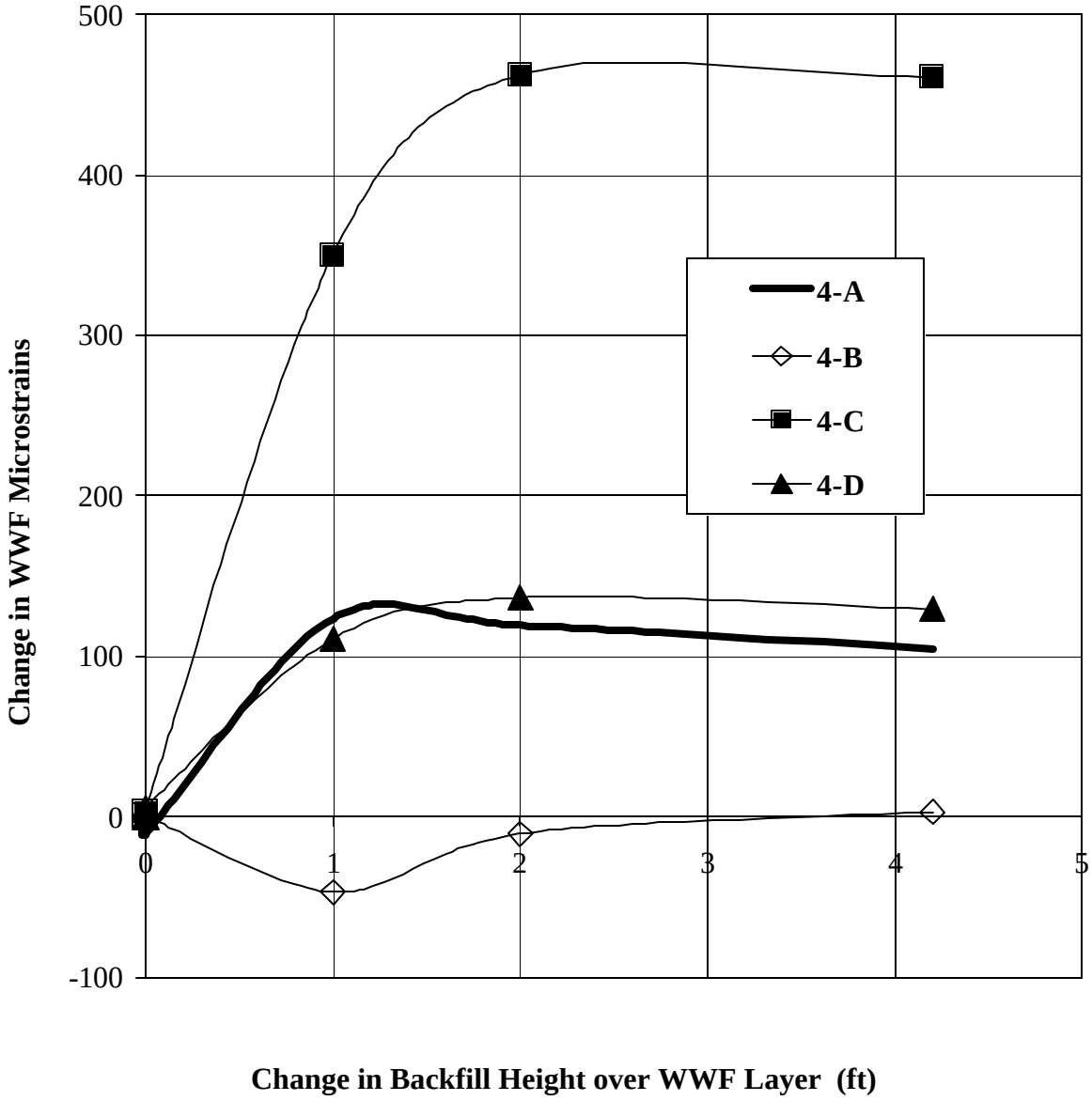

Figure 6.4. Results of WWF Layer 4 Strain Gages, Placed at H= 11 ft (Reference Reading at H=12 ft).

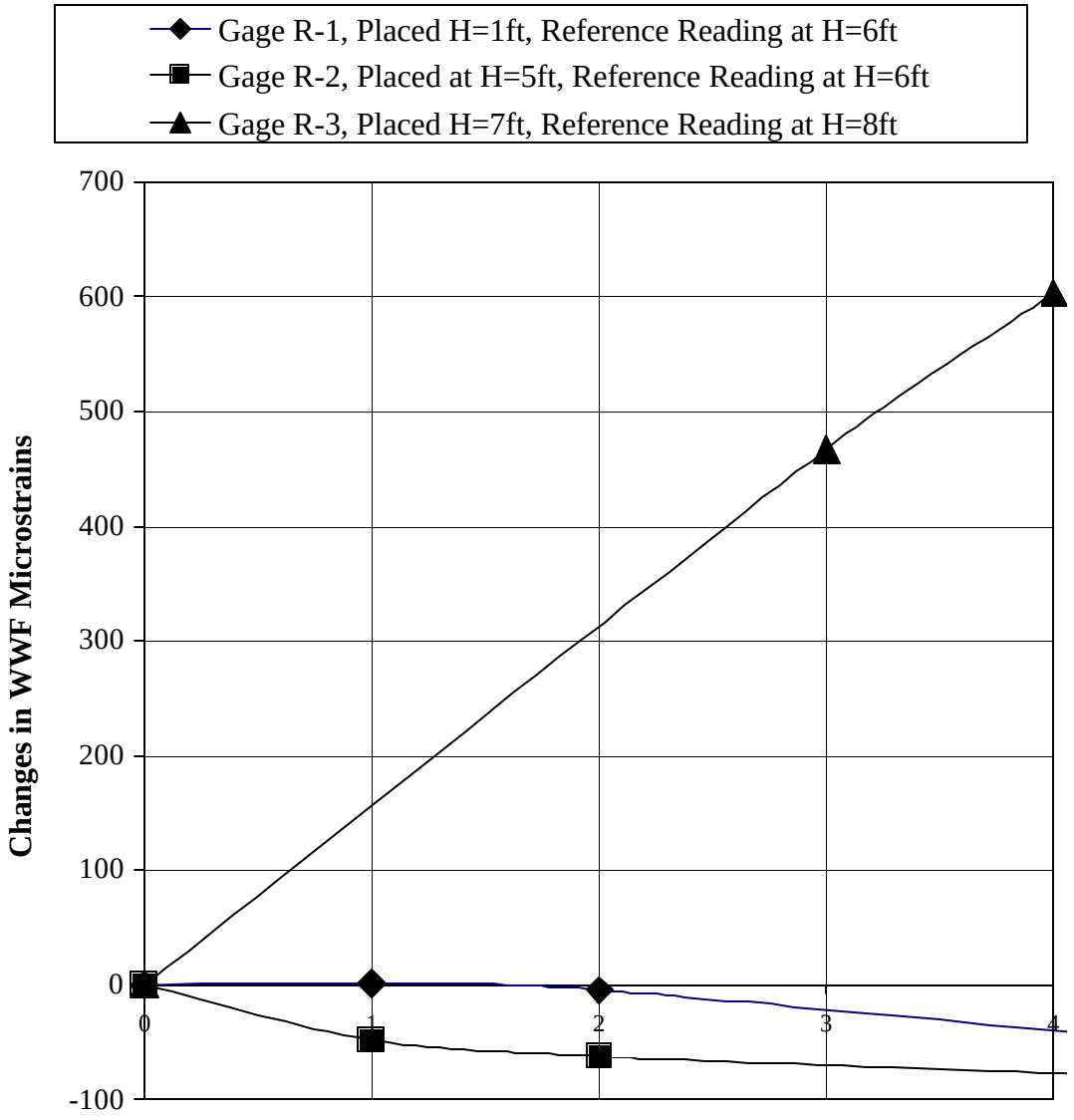

Figure 6.5. Results of WWF Strain Gages Placed at Station 3119.

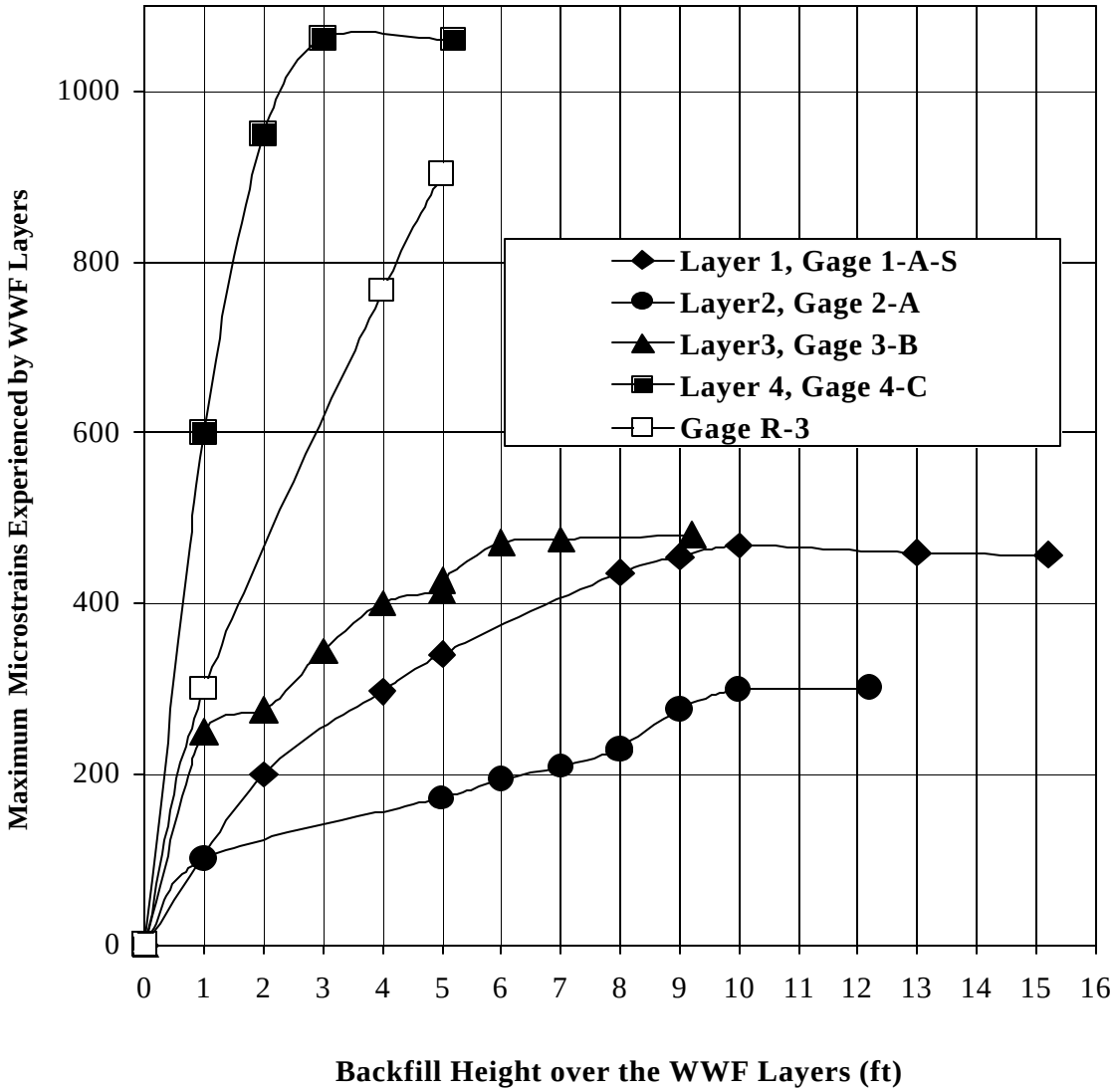

Figure 6.6. Location and Value of the Maximum Axial Strains for WWF Layers Placed at Different Levels.

6.4. Maximum Axial Tensile Forces in the Wire Mesh Reinforcements

Results for the maximum reinforcement tensile forces experienced by different WWF layers versus the applied backfill height over these layers are shown in Figure 6.7. Predictions for maximum tensile forces in these WWF layers as given in AASHTO 5.8.4.1C (using Equation 2.3), are also shown in Figure 6.7. If AASHTO predictions are reasonable, all measured curves shown in Figure 6.7 should be close to the AASHTO curve. The results in Figure 6.7 suggest the following:

- Most of the increase in reinforcement tensile forces occurred during placement and compaction of the first three feet of backfill (5 ft for Gage R-3). After that the reinforced soil mass responded with very small strains to increasing level of vertical loads. This behavior could be attributed to the influence of compaction operations that created large locked-in strains in the wire mesh layers during the placement of the first 3 feet of backfill over the gages, stiffening the reinforced soil mass response in subsequent stages.
- The maximum tensile forces for the lower WWF layers seem to agree with AASHTO predictions in the initial stages of backfilling but to be much lower than AASHTO predictions when the backfill height exceeded 5 ft. This could be attributed to the influence of the trench support in the lower zone of the wall.
- The largest wire mesh maximum tensile forces occurred in the upper WWF layers, almost six times higher than AASHTO predictions, and much larger than those developed in the lowest WWF layers. The maximum tensile force measured for layer 4 (925 pounds) is slightly larger than the steel allowable tensile force of 835 pounds (0.48 of the WWF yield tensile force). The higher tensile load in the upper WWF layers could be attributed to two factors: 1) the rotation of the facing to accommodate deformations of the reinforced soil mass, resulting in higher wall displacements and larger mobilization of the WWF tensile resistance at the higher locations of the wall, and 2) influence of compaction and high stiffness of the WWF reinforcements as discussed in Section 6.1. Note the selection of the reinforcement grade is uniform across the entire wall and based on the high reinforcement tensile force expected in the design procedure to occur at the lowest wire mesh layer.

6.5. Location of the WWF Maximum Tensile Force Line

It is required in the design that the reinforcement pullout resistance be less than the WWF applied maximum tensile force. The measured maximum tensile forces in the upper WWF layers were almost 6 times higher than those assumed in the AASHTO design procedure (Figure 6.7).

Location of the maximum tensile force line is needed to estimate the reinforcement embedment in the resistance zone to reinforcement pullout failure. The pullout resistance is linearly related to the reinforcement embedment length. Location of the potential failure surface is assumed to coincide with the maximum tensile forces line. According to FHWA Demo 82 (Elias and Christopher, 1997), the location of the potential failure surface for MSE walls with extensible and inextensible reinforcements are shown in Figure 6.8. Also shown in this figure is the estimated potential failure surface based on WWF strain measurements. It is clear from Figure 6.8 that the maximum tension line in the upper two thirds of the wall is shifted 3 to 6 ft (pushed into the backfill) beyond the line suggested by AASHTO for MSE walls with inextensible reinforcements. That could be attributed to the larger than anticipated WWF tensile loads (almost six times the design values, see previous section) and the relatively lower pullout capacity of the WWF layers in that zone. According to AASHTO, the reinforcement length should be 11.5 ft (70 % of the wall height). The placed reinforcement length in the upper zone of the MSE wall is 20 ft, 8.5 ft larger than the design required value. Therefore, the extended reinforced soil zone employed in this project was very beneficial in reducing the potential pullout failure of the reinforcements.

6.6 Assessment of the Design Procedure for the Reinforced Soil Mass

- The flexibility of the I-25/I-70 MSE/IFF wall system, allowing for rotation of the facing panels around the base of the wall, and heavy compaction most likely caused the larger than anticipated mobilization of reinforcement tensile forces and expansion of the potential failure zone in the upper zone of the wall. This response correlates very well with the significant deformations of the wall noticed during construction of the upper zone of the wall.
- The I-25/I-70 MSE/IFF wall system functioned to large degree almost as planned in the design. The flexibility of the systems allowed for significant mobilization of the tensile

resistance in the reinforcements, especially in the upper WWF layers, thus taking most of the lateral load off the facing panels.

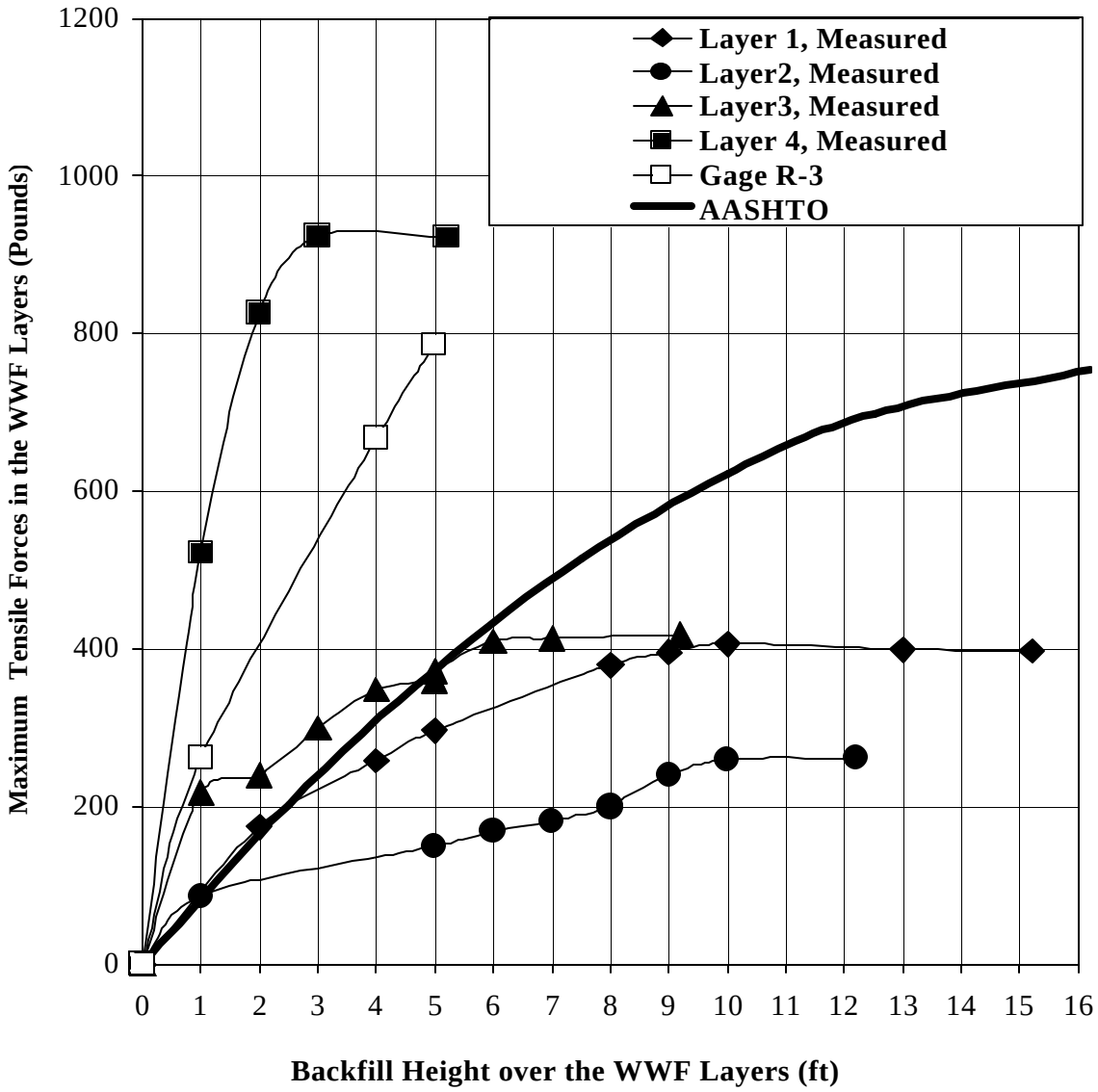

Figure 6.7. Measured and AASHTO Predicted Maximum Tensile Forces in WWF Layers.

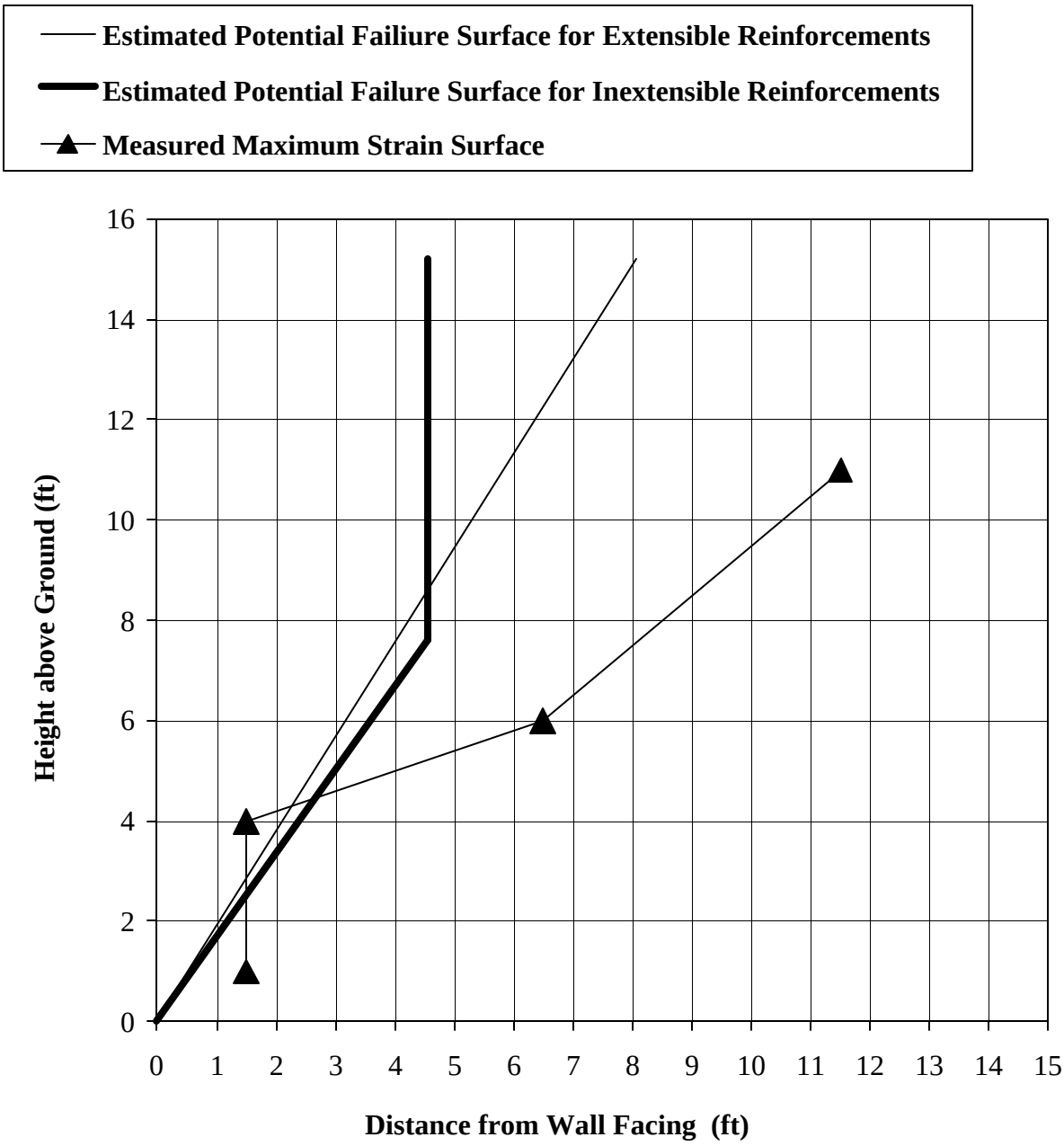

Figure 6.8. Measured and AASHTO Predicted Location of the Potential Failure Surface.

7. SUMMARY AND CONCLUSIONS

7.1 Overview

In 1996, the Colorado DOT completed the construction of a unique MSE wall with an independent full-height facing (IFF) for the ramp connecting Northbound Interstate-25 to Interstate-70 in Denver, Colorado. To the authors' knowledge, the design and construction of the I-25/I-70 MSE/IFF wall is the first of its kind in conventional highway practice. Therefore, the I-25/I-70 MSE/IFF wall system was considered experimental and comprehensive instrumentation and monitoring programs were incorporated into the construction operations. Instrumentation was installed at two locations, Stations 3116 and 3119. Each location was instrumented with inclinometer and survey points to measure movements of the facing panels, and with strain gauges to measure lateral earth forces and moment developed on the facing panels and the tensile loads mobilized in the WWF reinforcements. The main objective of this study was to upgrade the I-25/I-70 MSE/IFF wall for a future standard use of this wall system by identifying modifications and additions to the design and construction of the I-25/I-70 MSE/IFF wall that would improve performance and save money and time. This report provides insight into materials description and strength, construction procedure, monitoring, performance and design assessment of the I-25/I-70 MSE/IFF wall, and provides recommendations for construction of future MSE/IFF walls.

7.2 Description of the I-25/I-70 MSE/IFF Wall

The new MSE/IFF wall has four major components: 1) a self-stable welded wire fabric (WWF) reinforced soil mass, 2) full-height concrete facing panels not attached to the soil reinforcements (i.e., independent) that are allowed to tilt around their bases, 3) flexible face anchors to provide for attachment of facing panels to the reinforced soil mass and allow movements of the facing panel, and 4) a trench with a flowfill to brace the panels during construction before face anchors are placed. The MSE/IFF wall system was designed to be very flexible to mobilize large tensile resistance in the reinforcements, thus allowing the reinforced fill to take most of the lateral earth load and taking most of the lateral loads off the panel facings. A lower limit earth pressure on the facing of 30 psf was anticipated in this wall system and assumed to be the design lateral earth pressure on the facing panels. Weak bracing was utilized to support this wall during placement of the backfill and face anchors. Facing was battered with bottom and top setbacks in anticipation

of a horizontal movement that will occur during construction. The flexible face anchor connections were adjusted to improve wall alignment.

7.3 Study Findings and Recommendations

- It was noticed during construction of the first six panels that the facing deflections were greater than anticipated. Possible reasons for this excessive movement were identified and suitable corrective actions to reduce the wall movements and adjust the facing were implemented during construction of the remainder of the wall. These *recommendations* were successful in controlling the excessive movements of the wall as the remainder of the wall was never more than 1" behind plumb upon completion and the facing was properly aligned.
- Recommendations for depth of embedment for future MSE/IFF wall system with unbraced facing during construction are presented. These were based on analysis employed in geotechnical engineering to predict depth of embedment for cantilever sheet piling.
- Measurements of the facing outward displacements after the corrective actions had been in place were almost four times the values expected in the design, which is relatively large.
- Recommendations for setbacks in future MSE/IFF wall applications with facing panels either weakly braced or unbraced during construction are presented. These were based on analysis of the measurements of the facing movements for the I-25/I-70 MSE/IFF wall structure.
- The average earth pressure on the facing measured from 18 anchors strain gages was a low value of 32 psf, very close to the lower limit earth pressure of 30 psf, anticipated in the design.
- The I-25/I-70 MSE/IFF wall system functioned to a large degree as planned in the design. The flexibility of the wall system: 1) smoothly accommodated the deformation of the reinforcement soil mass, especially those induced by heavy compaction, 2) allowed for significant mobilization of the tensile resistance in the reinforcements, thus taking most of the lateral earth loads (including the compaction induced pressures) off the facing panels.
- The facing panels are at least four times stronger in bending than needed for lateral earth pressures. Under worst conditions, the safety factor for construction handling is about 1.7.
- There are concerns with the pullout capacity of facing anchors. The local factor of safety against anchors pullout was less than 1 for seven anchors and was less than 1.5 for almost all

middle and upper anchors. This response correlates very well with the significant deformations of the wall facing noticed during construction of the upper zone of the wall.

- Most of the increase in reinforcement tensile forces occurred during construction of the first three feet of backfill, which was attributed to the compaction influence.
- The largest wire mesh maximum tensile forces occurred in the upper WWF layers, almost six times higher than the design predictions and 100 pounds higher than the allowable tensile force.
- The maximum tension line in the upper two thirds of the wall was shifted 3 to 6 feet (pushed inside the reinforced soil mass) beyond the line suggested by AASHTO.
- The flexibility of the I-25/I-70 MSE/IFF wall system, allowing for rotation of the facing panels around the base of the wall, and the heavy compaction most likely caused the larger than anticipated mobilization of reinforcement tensile forces and expansion of the potential failure zone in the upper zone of the wall. This response correlates very well with the significant deformations of the wall noticed during construction of the upper zone of the wall.
- A field inspection of the I-25/I-70 wall on March 2001 (after almost 4.5 years of service) suggests excellent long-term performance of the wall with no signs for facing bulging or excessive movement.

7.4 Recommendations for Future Research

1. To monitor the performance of future MSE walls with independent full-height facing for possible refinements of the recommendations furnished in this study and to allow the future use of different soil reinforcements.
2. To perform pullout resistance test of face anchors to determine strength of soil interaction. Pull a few anchor bars to get an idea of the pullout resistance and the stiffness of the soil mass and face. This can be done using a center hole jack. Determine movement vs. force for deflections not to exceed 0.25" and forces not to exceed 12 K for fill over the anchor of 1 ft, 2 ft, 4, and 8 ft, and 12 ft.
3. To perform a large-scale experiment to verify that the panels will resist the wind loads. This is an essential for acceptance of the unbraced facing into MES wall with IFF system. A point load equal to a pressure of 16 psf should be applied to the center of the facing panel using the

depth of embedment recommended in this study. This test will increase confidence in the trench support scheme that could be also used for permanent noise barriers in the future.

8. BASIS FOR FUTURE STANDARD USE OF MSE WALLS WITH INDEPENDENT FULL-HEIGHT FACING PANLES

8.1 Overview

This chapter provides the design and construction basis and details for future standard use of an MSE/IFF wall system that would improve performance and save money and time. These recommendations are based on the lessons learned from design and construction of the I-25/I-70 MSE/IFF wall and its measured performance results (all summarized in Section 7.3). There were three interrelated problems with the I-25/I-70 MSE/IFF wall that need to be addressed in future applications: 1) larger than anticipated wall deformations, 2) higher reinforcement tensile loads and larger potential failure zone, and 3) and anchors pullout capacity smaller or close to measured anchor pullout loads. These problems were concentrated in the upper zone of the wall. To eliminate these problems, the density of WWF reinforcements and face anchors are increased among other measures in the proposed standard wall system. The furnished recommendations herein are expected to be conservative because of lack of sufficient performance data (i.e., no data when facing is unbraced) for this kind of MSE walls.

CDOT bridge and geotechnical engineers should consider this kind of wall system as a viable and standard alternative in the selection process for retaining walls.

8.2 Layout and Materials of the Proposed MSE/IFF Wall System

The proposed MSE/IFF wall system has four major components: 1) a self-stable welded wire fabric (WWF) reinforced soil mass, 2) full-height concrete facing panels not attached to the soil reinforcements (i.e., independent) that are allowed to tilt around their bases, 3) flexible face anchors to provide for attachment of facing panels to the reinforced soil mass and allow movements of the facing panel, and 4) a trench with a flowfill to brace the panels during construction before face anchors are placed. Recommendations for the proposed MSE/IFF wall with facing either weakly braced or unbraced during construction will be different in terms of depths of embedment and setbacks, but similar for all other aspects. Layout and details for the proposed MSE/IFF wall system are same as those described in Chapters 1 and 2 for the I-25/I-70

MSE/IFF wall (Figures 1.2, 2.1, 2.2, 2.3 and 2.4), but with the following modifications and additions:

- Height of facing panels is limited to a maximum of 20 ft above ground level. The applied design surface surcharge load on the top of the panel (due to roadway structure and traffic) is expected to be 375 psf (equivalent to 3 ft of fill surcharge).
- Tables 8.1 and 8.2 list the recommended facing settlements, setbacks (see Figure 2.2) and depths of embedment (Figures 2.2 and 2.3) for MSE/IFF wall with different heights above ground level and with the facing either braced or unbraced during construction.
- The specification to the WWF will be changed to wires having a cross-sectional area of 0.029 in^2 and welded in grid pattern of 6"x6" squares (referred to 6x6 w 2.9 x w2.9 welded wire fabric) with a vertical spacing of 1 ft (Figure 1.5). Reduce the zinc thickness to standard values for galvanized WWF to compensate for the increasing costs of the WWF.
- Material specifications for facing panels (up to 8 ft wide) will not be changed.
- The facing anchor bars will be changed to galvanized #4 steel bars placed at 4 ft maximum vertical spacing. The length of the bar threads at the facing side should be at least 8.5" with 6" extended beyond front of panel to furnish adjustment of panels. The length and configuration of the facing anchor loop will stay the same.

Table 8.1. Recommended Depths of Embedment and Setbacks for MSE/IFF Walls with Facing Weakly Braced during Wall Construction

Panel Height above ground Level (feet)	4	10	12	16	20
Embedment (feet)	2	2	2	2.4	2.8
Expected Panel's Settlement (inch)	0.5	0.9	1.1	1.4	1.6
Panel Bottom Setback (inch)	0.2	0.4	0.5	0.7	1.0
Panel Top Setback (inch)	0.6	2.4	3.2	5.3	7.9

Table 8.2 Recommended Depths of Embedment and Setbacks for MSE/IFF Walls with Unbraced Facing during Wall Construction.

Panel Height above ground Level (feet)	4	10	12	16	20
Embedment (feet)	2	3.4	3.8	4.4	5.2
Expected Panel's Settlement (inch)	0.7	1.2	1.5	1.8	2.1
Panel Bottom Setback (inch)	0.3	0.9	1.1	1.6	2.3
Panel Top Setback (inch)	0.8	3.4	4.6	7.6	11.3

8.2 Selection and Features of the Proposed MSE/IFF Wall

The proposed MSE/IFF wall system is adequate for all field conditions found suitable for the use of MSE walls. The features and advantages of this wall system over conventional CIP cantilever and MSE retaining wall systems are:

- This kind of wall is necessary to match the same monolithic appearance of an existing wall that had full-height concrete facing.
- This wall requires little or no over-excavation in front and beneath the wall. This feature allows traffic to remain open throughout the construction and alleviate the need to deal with excavation and disposal of subsoil.
- The small lateral earth pressure against the facing reduced the costs of this wall system and makes it less costly than other conventional walls
- More flexible and tolerable to differential settlements than cantilever CIP walls.
- The use of full-height facing significantly reduces the construction time and may require less maintenance work than MSE wall with segmental facing.
- The flexibility of the wall system can very smoothly accommodate the movements and stresses of the reinforced soil mass, especially those induced during compaction.
- The facing was properly aligned because it could be controlled by the user, a feature not available in conventional MSE walls, especially those with segmental facing panels.
- Finally, erecting the panels unbraced during backfilling operations speeds construction, allows for construction in more confined spaces, and reduces construction costs.

8.3 Design Guidelines of the Proposed MSE/IFF Wall

- The proposed MSE/IFF wall system is fixed in term of the materials properties and layout to ensure the internal stability of all wall components (facing panels, facing anchors, and reinforced soil mass).
- The designer still needs to ensure the external stability of the MSE wall (e.g., bearing capacity and overturning) of the wall under different but specific surrounding field conditions (e.g., retaining fill and foundation soil). The stability analyses should be performed in

accordance with AASHTO guidelines for conventional MSE walls with inextensible reinforcements (see Elias and Christopher, 1997).

- Recommendations for depth of panel embedment were developed for a granular foundation soil with a friction angle of 30 degrees, a unit weight of 100 pcf, and interface friction angle between the facing and the surrounding material (flowfill in this case) of 20 degrees. Judgments had to be made in the case of different foundation soil (see Chapter 2).
- Until more track records for the performance of MSE/IFF walls are established, it should be required in the design plans to closely monitor the movements and anchor loads of the first six panels before placement of additional panels. This section of six panels should be built with the intent to adjust the recommended setbacks as needed even if it means reconstructing. Based on the results of these first six panels, revised adjustment procedure for anchor nuts and changes to the setbacks could be made.

8.4. Construction Procedure of the Proposed MSE/IFF Wall

- ❑ Excavate and/or fill, and compact the area at the base of the walls such that the area is at the level of the future finished grade at the front of the wall and provides a firm base for construction equipment and allows for a vertical edged trench for the face panels.
- ❑ Trench to the bottom of the panel facing elevation as specified in Tables 8.1 and 8.2. The minimum depth and width of the trench are, respectively, 2 ft and 6".
- ❑ Position facing panel in the trench with required top and bottom setbacks (Tables 8.1 and 8.2), and use flowfill and temporary bracing (permanent bracing if the facing panels are to be supported during construction) to hold the panel in the required position in the trench.
- ❑ Place backfill, wire mesh, and face anchors behind the facing panel as required in the plans.
- ❑ Lightly compact the 3 ft behind the facing with one pass of a hand-operated vibrating plate or vibrating roller not exceeding 2500 pounds per roller. Keep heavy compaction equipment 2 ft min. from the panels. Backfill compaction requirements (100% of AASHTO T-99 or 95% of AASHTO T-180A) will not apply within 4 ft of the panels.
- ❑ Lay the mesh tightly against the back of the facing panels. Place the first wire of the mesh as close as possible to the wall. Bend the WWF tails at the face (fabricated in the plant) as needed to get the first cross wire within 2" of the facing.

- Snug the # 4 face anchor rebars by pulling the rebars and driving a # 4 L bar into the ground at each of the back ends (the bend with one side parallel to the wall). When one lift has been placed on the face anchor rebars, tighten the nuts finger tight plus two full turns on both this layer of # 4 anchor bars and any lower layer.
- Monitor top of panels location and batter as backfill progresses. If it appears that the top of panel will deform beyond the layout line before backfill is complete, add supplemental layers of wire mesh (decrease WWF vertical spacing to 6") and move the heavy compaction equipment further from the face of the wall.
- Check the plumb of the panel just before the last lift of backfill is compacted. If the top of the panel is less than 0.5" behind plumb, tighten all the anchor nuts 2 full turns after finger tight, and leave the struts in place (note no struts for the unbraced facing) for the compaction of the last lift. If the top of the panel is between 0.5" and 1.0" behind plumb, tighten all the anchor nuts as above and remove the struts before placing and compacting the last lift. If the top of the panel is more than 1" behind plum, tighten the anchor nuts as above, then leave the bottom nut snug and back off all other nuts at all levels proportionally to achieve a setback of 1" at the top of the panel
- When backfill is complete, place three feet of loose fill over the finished top grade for a period of at least 60 hours. Then, remove the surcharge.
- Back off the nuts (from bottom to top) on those nuts that are in areas that need to be brought out closer to the plumb line for a smooth appearance of the face. Do not tighten nuts to attempt to bring face area back toward the reinforced backfill. Do not attempt to bring areas that are less than 1" behind the plumb line out to the plumb line, except to provide smooth appearance. If, from this adjustment, a gap develops between the reinforced backfill and the face, fill it with sand or flowfill.

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