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ABSTRACT

Woodrow Wilson Replacement Bridge Project included widening the Washington Beltway (I-95/I-495) Outer Loop from three lanes to six-lanes. This required supporting two existing ramps that connect I-295 and MD 210 as well as the existing Mechanically Stabilized Earth (MSE) wall that supports the ramps. The MSE is about 17-ft tall, about 570-ft long, and at the top of a slope. A tied-back soldier pile and lagging wall with cast-in-place facing was selected to support the MSE and the ramps. The new wall will be about 1,376-ft long and will be as high as 37-ft. The closest approach of the wall to the existing MSE is about 3-ft.

Laboratory testing was supplemented with Dilatometer Test (DMT) and Cone Penetration Test (CPT) soundings. PYWall and PLAXIS were used to estimate wall deflections and bending moments in the soldier piles. This paper reviews the analysis techniques, describes the design and the construction methods, and the instrumentation used to monitor the wall and MSE movements.

The results of the computer simulations were compared to the inclinometer results. As work progressed simulations were updated by modifying the soil parameters to obtain calculated results that are more nearly consistent with the instrumentation readings.

INTRODUCTION

Soil movements and deflections are some of the most difficult predictions that designers must make; wall deflections and settlements of underpinned structures when due to excavations are perhaps the most complicated of all geotechnical structures. Factors that affect these movements include soil stratigraphy, soil strength and stress-strain parameters (actual variations & testing errors), stress history, support system details, construction sequence, and workmanship (Finno and Calvello 2005; Finno, et al. 2002; Mana and Clough, 1981).

Typically, simplified design charts and rules of thumb have been used to estimate wall movements and settlements. Combined with engineering judgment, these charts have served well over the years, but are limited in usefulness and are not capable of handling the increasingly complicated situations that we are being presented in practice. This is especially the case in infrastructure improvement projects where existing structures and facilities must often be in use serving the public during construction.

Because it has not been possible to accurately include all the factors in design the Observational Method has been used to verify that the construction is behaving as the rules of thumb predicted and to provide warning of excessive movements before any serious damage or injury is inflicted. This method is often used for situations too complex to fully characterize (Vick, 2002).

The Finite Element Method (FEM) and p-y techniques are being increasingly used to supplement these older methods so that more accurate predictions of deflections, settlements and other movements can be made. These methods are very sensitive to several of these factors listed above, and, therefore, these factors must be accurately incorporated into the model. In most circumstances the engineer can not control the actual construction means and methods or the detailed sequence of construction. These are left to the Contractor.

Because most of these factors are uncertain and the models introduce the addition of modeling errors, the use of FEM or p-y techniques still requires that the Observational Method during construction. The more complicated a construction project is the more a complicated analysis technique need to be, and therefore, the more room for error.

In addition to evaluating design and construction alternatives, FEM is a very useful tool to locate instrumentation, and establish instrumentation criteria. The following case study will illustrate how these methods combined with engineering judgment can work together to provide a cost effective and safe structure.

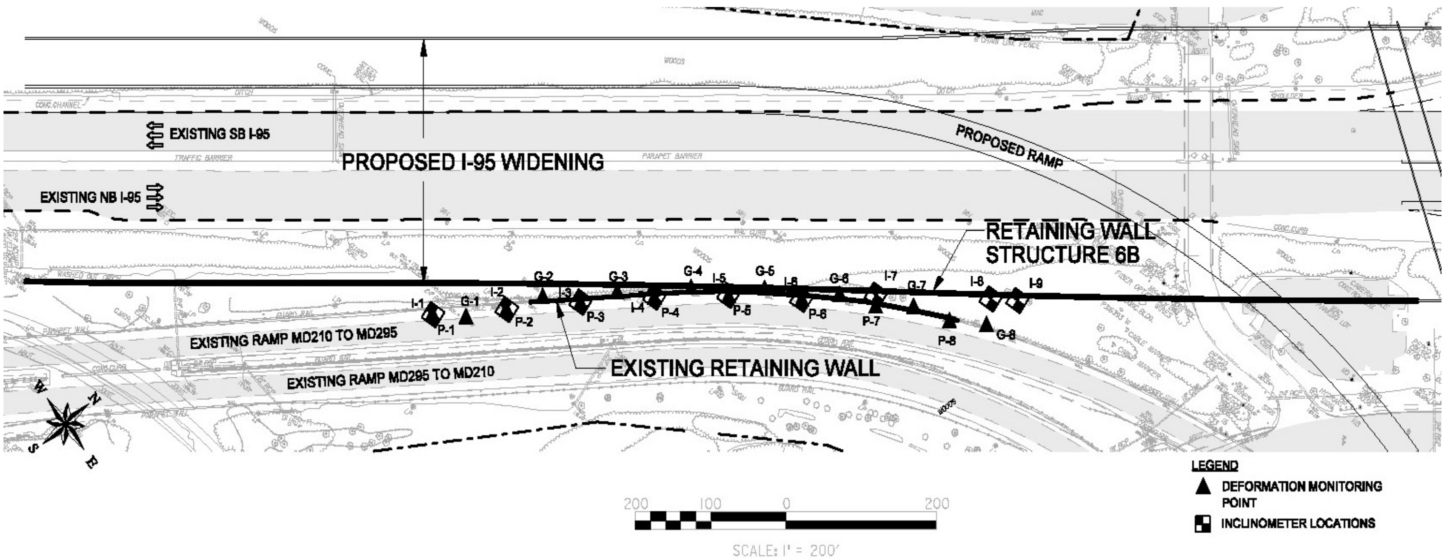

Fig. 1. Site Plan

PROJECT DESCRIPTION

Part of the Woodrow Wilson Replacement Bridge Project included widening the Washington Beltway (I-95/I-495) Outer Loop from three lanes to six-lanes. This required supporting Ramp F that carries traffic from north bound MD 210 to north bound I-295 and Ramp E that carries traffic from south bound I-295 to south bound MD 210. These ramps are supported by a Mechanically Stabilized Embankment (MSE) located at the top of a 2(H):1(V) slope. The MSE is about 17-ft tall at the tallest and about 570-ft long. The original Beltway (I-495/I-95) was completed in the mid-1960's and the I-295 ramps and MSE were built in the early 1990's. The proximity of the MSE to the new edge of pavement created design and construction difficulties. A top-down wall would be needed to support the existing MSE while maintaining traffic on the ramps, and the steep slope and proximity to the outer loop of the beltway limited the amount of space that was available for constructing benches and staging equipment.

A tied-back soldier pile and lagging wall with cast-in-place facing was selected. The new wall is about 1,376-ft long and is as high as 37-ft. The closest approach of the new wall to the existing MSE is about 3-ft. Although MSE's can tolerate large differential settlements, if the highway ramps experience large differential settlements, it could cause drainage and traffic difficulties. Therefore, it was important to monitor the deflections of the MSE and the new wall.

The new wall is a Contractor designed wall. The wall was constructed by driving twin HP 12x53 H-piles at each soldier pile location. Typically, the pile pairs were located at 6-ft intervals, but in some areas the spacing was as much as 8-ft.

For most of the length of the wall three rows of tiebacks were used as described below. Studs were welded to the outer flange of the soldier piles as shear connectors for the cast in place facing for the wall. Form liners were used to provide an appearance similar to the existing MSE.

Figure 1 shows the plan area and the inclinometer and Deformation Monitoring Point (DMP) locations. A total of nine inclinometers and eight deformation monitoring points were installed to monitor the wall deflections and soil movements during construction and for about a year after construction was complete. The monitoring is scheduled to stop in September 2008. Because of the limited space between the back of the new wall and the facing of the existing MSE several of the inclinometers were located in a flush mount casing on the shoulder of the ramp at the top of the MSE.

Figure 2 shows the cross section near the location of Inclinometer I-6. Some soil was excavated and a bench formed to provide access for the pile driver and other construction equipment. The soils were excavated in stages to allow for installation of the tiebacks as follows.

Stage 1: cut 8-ft (EL 187)
Stage 2: cut 19-ft (EL176)
Stage 3: cut 26-ft (EL169)
Stage 4: final cut to pavement subgrade EL (163)

The Contractor started driving the soldier piles May 2006. The bench at Stage 1 was excavated by August 1, 2006 and the tiebacks for Row 1 near Inclinometer I-6 were installed September 20, 2006. The bench at Stage 2 was excavated by

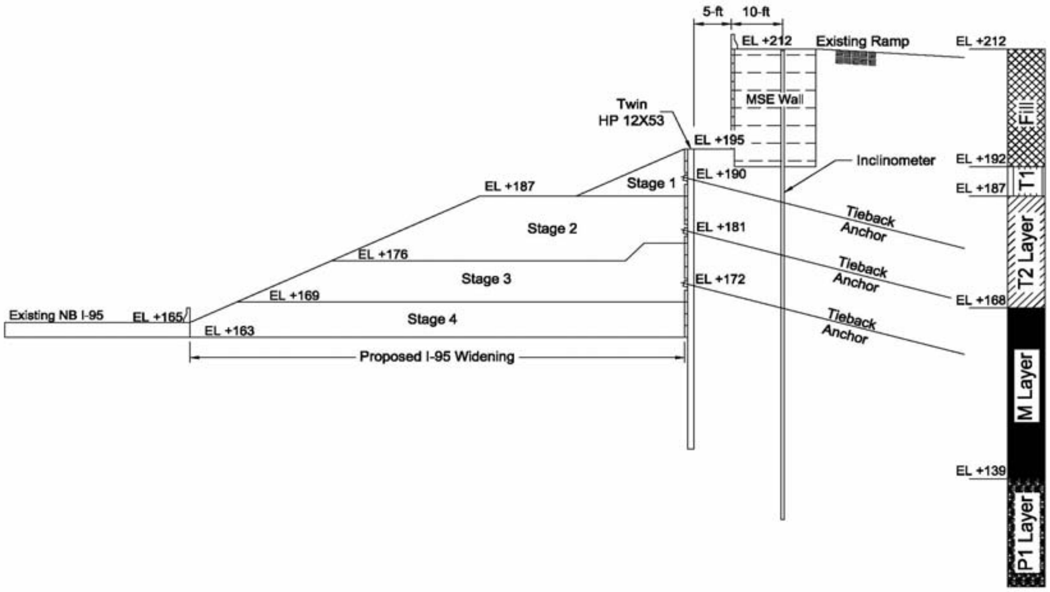

Fig. 2. Wall Cross Section near Inclinometer I-6

November 29, 2006 and the tiebacks for Row 2 near Inclinometer I-6 were installed February 20, 2007. The bench at Stage 3 was excavated by 3/27/2007 and the tiebacks for Row 3 near inclinometer I-6 were installed April 18, 2007. The final excavation was reached April 23, 2007 and the cast in place facing in this area was complete by May 2007.

The HP 12x53 piles were driven without pre-drilling. The tiebacks were preassembled and pre-grouted when delivered to the site speeding construction and so as not to waste valuable room. The tiebacks that did not comply with the proof test criteria were re-grouted. The few that still did not comply with the proof test criteria were abandoned and a new tie-back installed. Most of those were in the upper row while installation procedures were still being developed.

Figure 3 is a photograph showing the soldier piles and lagging during construction in the area of Inclinometer I-6. The excavation is near the final pavement subgrade (Stage 4). The piles can be seen to be slightly out of plumb, but not to an excessive amount. The black material between the soldier piles to the right is the single-sided drainage board. The space between the twin piles is filter fabric reinforced with welded wire fabric.

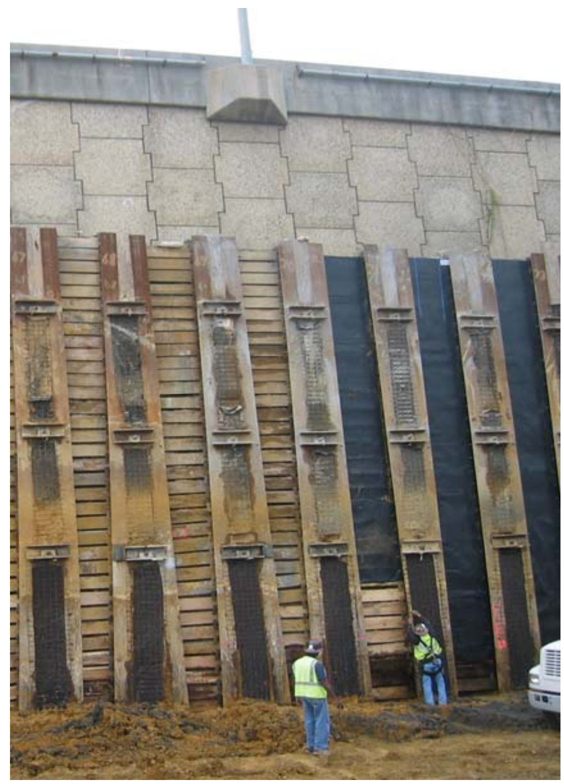

Fig. 3. Photograph of soldier piles showing the existing MSE

SUBSURFACE CONDITIONS

To facilitate and standardize the subsurface characterization of all contracts in the project all soil strata descriptions were based on the Washington Metropolitan Area Transit Authority (WMATA) strata designations. These were developed for the Washington, D.C. metro subway system and most local consultants are at least somewhat familiar the system.

The existing highway ramp consists of fill associated with construction of Ramps E and F in the late 1980's. This material generally consists of medium stiff to hard clay with varying percentages of sand and gravel. The liquid limit for this layer ranged from 16 to 40 and the Plasticity Index ranged from 11to 25. The natural moisture content of the soil ranged from 2.4 to 23.4 percent. The SPT N-values typically ranged from 6 blows per foot (bpf) to 138 bpf.

The plans for the shop drawings for the MSE were not available. It was assumed that the material in the reinforcement zone consisted of select fill or #57 stone and that galvanized steel straps were used for the reinforcement.

The fill overlies T1 and T2 strata. T1 consists of low plasticity silty soils thought to possibly be aeolian deposits (USDA, MAES, 1967). This layer was thin and not encountered in most borings. The T2 consists of medium dense silty and clayey sand and fine to coarse rounded gravel and cobbles thought to have been deposited by the ancestral Potomac River in the Pleistocene Epoch. The T1 stratum overlies the T2 stratum, but in many instances it is not possible to distinguish between them as the T1 can be somewhat sandy and the T2 can contain a fair amount of fine-grained material.

The SPT N-values of the T2 stratum ranged from 4-bpf to 100/3-inches. The liquid limit ranged from 20 to 53 and the plasticity index ranged from 3 to 25. The natural moisture content ranged from 3.7 to 35.5 percent. The SPT N-values were likely exaggerated by the gravel and cobbles.

The Monmouth formation (M Layer) is believed the have been deposited on a continental shelf during the Upper Cretaceous period and underlies the T1 and T2 strata. The natural soil in the M Layer generally consisted of soft to hard fine grained material. Silt and Clay were predominantly found in this stratum with frequent sand seams. The lower portions of the stratum seemed to contain more sand than the upper portions.

The SPT N-values ranged from 3 to 38-bpf. The liquid limit and the plasticity index ranged from 23 to 45 and 4 to 25, respectively. The natural moisture content ranged from 11.8 to 42.5 percent. Based on laboratory consolidation tests and DMT soundings, the OCR for this layer ranged from 3.2 to 10.7.The undrained shear strength increased with depth from approximately 1-tsf at EL 164 to about 2.5-tsf at EL 140 as shown in Fig. 4.

This M Layer overlies the Patapsco Formation and Arundel Clay (P1 Layer). The P1 Layer typically consisted of very stiff to hard silty clay. The SPT N-values ranged from 7 to 77 bpf and the natural moisture content ranged from 13.5 to 27 percent. The liquid limit and the plasticity index ranged from 24 to 67 and 10 to 43, respectively. The undrained shear strength for this layer ranged from approximately 2.86-tsf to 4.18-tsf.

The Patuxent Formation (P2 Layer) is interbedded with the P1 Layer. The P2 Layer typically consisted of medium dense to dense sand with varying percentage of silt and clay. The SPT N-values typically ranged from 23 bpf to 100/3-inches and the natural moisture content ranged from 19.2 to 24.5 percent. The liquid limit and the plasticity index for the layer ranged from non-plastic to 33 and non-plastic to 11, respectively.

Figure 4 compares the WMATA Strata with the undrained shear strength as measured using the DMT and Unconsolidated Undrained (UU) and Consolidated Isotropic Undrained Compression (CIUC) triaxial shear test results, and the Atterberg and natural moisture contents, modified from Klein and Bathe (2006).

FEM PROCEDURES

The soldier pile and lagging wall was modeled using the finite element software PLAXIS 8.2 Professional version to compute the displacement of the soil and the wall due to the excavation. A plain-strain condition was assumed for simulating the problem. Considering the length of the wall this was certainly a reasonable assumption. The stratification lines between the different material types were assumed to be horizontal although they are likely slightly dipping to the south east. Five soil layers were included in the model. The hardening-soil model was used to characterize the soil layers in the PLAXIS simulation. Horizontal restraints were set as the mesh boundary condition for the left and right boundaries and total restraints were used for the bottom boundary in the finite element mesh.

Fourteen calculation and construction phases were used to simulate the wall in the finite element model. The construction of interstate I-95 and the MSE wall with the ramps were also modeled into the simulation to fully simulate the stress history. The stages are summarized in Table 1. During the design phase some analysis using FEM was performed to evaluate options. Once the Contractor's design was accepted the model was revised to reflect the Contractor's design and sequence of construction.

Table 1. PLAXIS Simulation Stages

Calculation Phase	Simulation Stages
0	Initial conditions
1	Original I-95/I-495 Construction
2	Consolidation Stage
3	I-295 Ramp Construction
4	Consolidation Stage
5	Pile Driving
6	Excavation to First Row of Tiebacks
7	Prestress 1st Row of Tiebacks (Stage 1)
8	Excavation to Bench
9	Excavation to 2nd Row of Tiebacks
10	Prestress 2nd Row of Tiebacks (Stage 2)
11	Excavation to 3rd Row of Tiebacks
12	Prestress 3rd Row of Tiebacks (Stage 3)
13	Final Excavation (Stage 4)
14	Consolidation Stage

Stages 1, 2 and 3, 4 model the original construction of I-95/I-495 and the ramps, respectively. Consolidation stages were added after the construction of I-95 and the ramps to allow for the pore water pressure to equalize. The wall face was modeled as a beam element with the properties of the soldier pile. The hardening-soil model was used to characterize the soil in the simulation. The input soil parameters are the friction angle, φ, cohesion, c, dilation angle, ψ, the reference secant Young's modulus at the 50% stress level, E_{50}^{ref}, the reference oedometer tangent modulus, E_{oed}^{ref}, and the exponent m which relates reference moduli to the stress level dependent moduli.

$$E = E^{ref}\left(\frac{c\cot\varphi - \sigma_3'}{c\cot\varphi + p^{ref}}\right)^m \tag{1}$$

where p^{ref}= reference pressure equal to 100 stress units; and σ_3'= minor principle effective stress (Brinkgreve and Vermeer 1998)

The moduli for the soils were the only parameter updated during the optimization. According to Brinkgreve and Vermeer (1998) the correlations between the moduli are,

$$E_{oed}^{ref} = 0.7\ E_{50}^{ref} \tag{2}$$

$$E_{ur}^{ref} = 3\ E_{50}^{ref} \tag{3}$$

The computed displacements were compared to the inclinometer field data. The inclinometer readings were taken every 2-ft at different stages of the construction. The computed results and the actual field data were compared at Construction Stages 1, 2, 3 and 4.

P-Y Analysis

A computer software application for the analysis of Flexible Retaining structures PYWALL version 2.0 was used to compute the deflection of the soldier pile. A trapezoidal pressure distribution according to the FHWA (1999) manual was used to determine the lateral load the soldier pile wall as well as to confirm the tieback prestress levels proposed by the Contractor. The tieback resistances in PYWALL were modeled by specifying lateral springs at the tieback locations. The MSE wall on top of the soldier pile wall was modeled as a surcharge dead load equivalent to the dead weight of the wall.

The deflection of the soldier pile was also computed using PLAXIS using the optimized soil parameters as described below. The lateral displacements of the soldier pile computed using the two computer programs was compared. The results of the comparison are described below.

SOIL PARAMETERS

Table 2 summarizes the initial soil parameters used for the FEM and P-YWall analysis. The actual soil parameters used for design of the wall by the Contractor were specified in the contract and were limited to shear strength and unit weights. The Contractor was directed to use limit equilibrium methods as described in the FHWA Manual Circular No 4 Ground Anchors and Anchored Systems (FHWA, 1999). These parameters were determined based on conservative assessments of the laboratory test results and the stress history and shear strength parameters from the DMT soundings. The soil parameters for the fill, T1 and T2 layers were based on SPT N-values and engineering judgment. The M and P1 layers were mostly fine grained, but were overconsolidated and the M layer did contain several sandy seams.

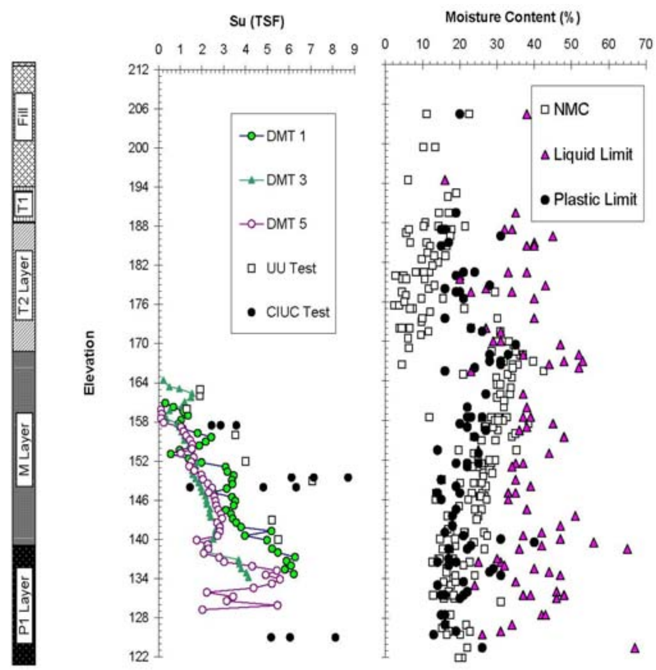

Fig. 4. Soil Strata

The constrained modulus as estimated by the DMT was also used to develop the stiffness parameters to compare with the stiffness parameters derived from the laboratory testing. The parameter E_{oed}^{ref} is similar to the constrained modulus. It was thought that perhaps since the DMT also loads the soil in a lateral direction the measured modulus might be a good indictor of the actual modulus values. Given the density and the presence of oversized gravel and cobble material in the fill and T2 layers meaningful DMT results were difficult to obtain from the Fill and T2 layers. Table 2 lists the initial soil parameters derived only from the laboratory testing and Table 3 lists the soil parameters assuming the constrained modulus from the DMT and then back calculating the E_{50}^{ref}.

Table 2. Initial Soil Parameters

Parameter	Fill	T1	T2	M	P1
φ(°)	32	31	32	32	17.7
c (ksf)	0	0	0	0.24	0.70
ψ(°)	2	1	1	2	0
E_{50}^{ref} (ksf)	57	22	92	80	427
E_{oed}^{ref} (ksf)	38	16	64	56	299
E_{ur}^{ref} (ksf)	171	67	277	240	1281

Table 3. Initial Soil Parameters Based on DMT

Parameter	Fill	T1	T2	M	P1
φ(°)	32	31	32	32	17.7
c (ksf)	0	0	0	0.24	0.70
ψ(°)	2	1	1	2	0
E_{50}^{ref} (ksf)	57	22	92	176	892
E_{oed}^{ref} (ksf)	38	16	64	123	625
E_{ur}^{ref} (ksf)	171	67	277	527	2677

RESULTS

The shear strength parameters are usually well characterized using laboratory testing and in situ testing. It is the parameters that relate stress to strain that is often difficult to characterize. While this relationship is difficult, at the same time it is the most crucial for estimating deflections and settlements. For this back analysis we, therefore, held the most of the soil parameters constant and varied only the modulus values. This also simplified the analysis and made it manageable.

The FEM results of using Equation (2) to estimate E_{oed}^{ref} are shown in Fig.5. The FEM results using the DMT to estimate E_{oed}^{ref} is shown in Fig.6 and the results discussed below. Equation (2) was used to estimate E_{50}^{ref} when the DMT was used to estimate E_{oed}^{ref}.

During construction we were able to observe the soils as they were exposed and update the elevations of the contacts between the strata. An old top soil layer was exposed indicating the fill extended to a lower elevation than previously thought and the T1 and C strata did not seem to exist. The material thought to be these were actually fill material. In the area of Inclinometer I-6 no C layer material was encountered in the borings.

The three inclinometers that were in the area of the closest approach to the existing MSE were I-4, I-5, and I-6. For brevity, the rest of this case study describes the results for Inclinometer I-6; the results from Inclinometers I-4 and I-5 and are not significantly different, but I-6 did show the most movement in the inclinometer, although not by very much.

The top of casing in the ramp shoulder at I-6 is at EL 212.0, and the top of the new wall elevation is at EL 196.6: the MSE height in this area is 15.5-ft. The depth from the top of wall to the bottom of cut or pavement subgrade is 33.45-ft and the face of the MSE is about 5-ft behind the face of the new wall. Inclinometer I-6 is about 10-ft from the face of the MSE. This inclinometer was installed in the shoulder of the ramp because of access limitations.

Figures 5 and 6 show the predicted deflection of Inclinometer I-6 using the laboratory soil parameters and the DMT soil parameters. Some analysis using FEM was performed during the design phase to aid in locating the instrumentation and estimate rough estimates of the displacement of the MSE. This was updated based on the actual shop drawings prepared by the Contractor and the actual as-drilled location of the inclinometer. The actual and the predicted deflections are show together on these figures.

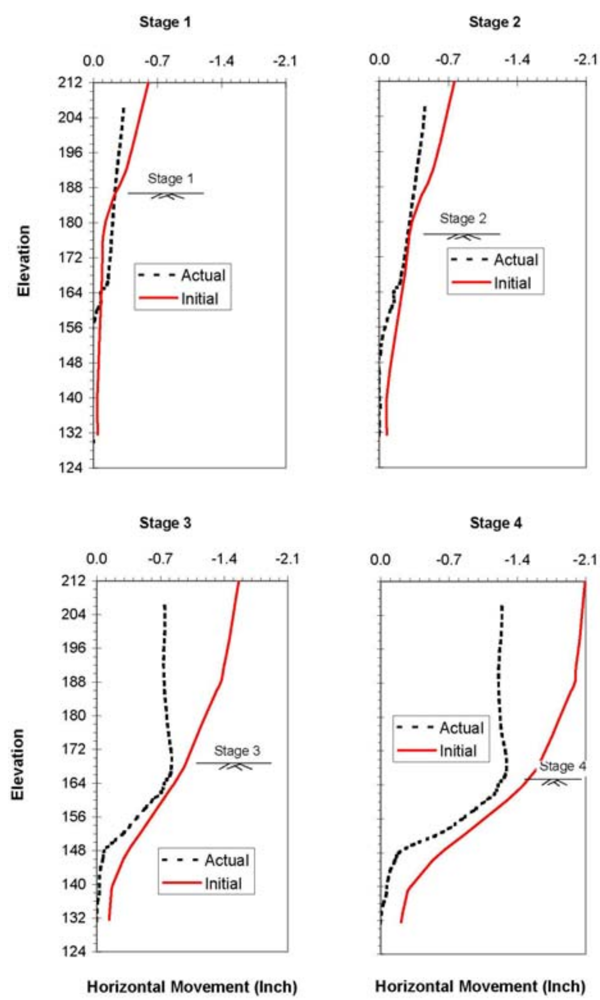

Fig. 5. Inclinometer Deflections Using Initial Soil Parameters

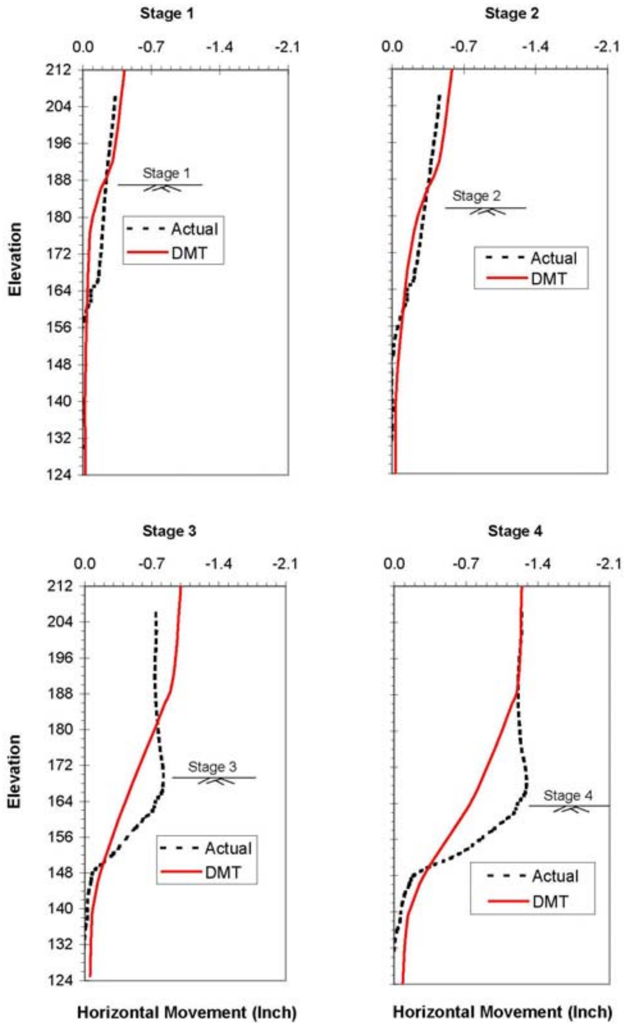

Fig. 6. Inclinometer Deflections Using Initial DMT to Estimate Soil Parameters

After Stage 1 the actual deflections above the bench were slightly smaller than estimates by about 0.15-inch at the top of the wall. Larger discrepancies were observed at the elevation of the MSE. This is likely due to the difficulty in modeling the reinforcement zone. Near the bench elevation the estimate matched the actual deflection rather well and below the bench the actual deflections were slightly larger than the estimate by about 0.1-inch. These deviations from the predicted deflections are rather small and well within our ability to make predictions, but soil parameters were adjusted to be more consistent with the measured deflections.

Of more interest is the deflected shape of the inclinometer casing. The actual deflected shape shows a small S-shaped deflection from about EL 156 to about 166. The initial predictions also indicated an S-shaped deflection, but at between EL 182 and 190, closer to the elevation of the bench near EL 187. Also, the upper portions of the MSE were predicted to deflect more than it actually did.

The process of revising the stiffness parameters was repeated for each stage. Not surprisingly, the largest discrepancies between predicted and actual were after Stage 4 when the activating loads would be at the greatest and the resistance from the clay in the M layer would be fully mobilized.

The initial estimates of the stiffness parameters based on the laboratory testing appeared to be much smaller than the actual values back calculated. This can be expected because the laboratory testing consisted of triaxial undrained compression testing on Shelby tube samples. There may have been better agreement between predicted and observed if triaxial extension tests had been used.

After the first two stages there was not much difference in either the laboratory derived stiffness parameters, the DMT derived stiffness parameters or the observed behavior. At Stage 3, the predicted deflections based on the DMT derived parameters were larger than the observed values by about 0.3-inches at the top of the wall near EL 196. However, below El 180 the actual deflections exceeded the predicted values by as much as 0.45-inches at an elevation just above the bench at EL 169. The discrepancy between the predicted and observed eventually disappeared near EL 148, about 20-ft below the bench and near the top of the P1 layer. It appears that the DMT over estimated the stiffness of the M layer.

Table 4. Soil Parameters – Stage 1 Optimized

Parameter	**Fill**	**T1**	**T2**	**M**	**P1**
φ(°)	32	31	32	32	17.7
c (ksf)	0	0	0	0.24	0.70
ψ(°)	2	1	1	2	0
E_{50}^{ref} (ksf)	57	38	184	107	427
E_{oed}^{ref} (ksf)	38	26	129	75	299
E_{ur}^{ref} (ksf)	171	113	553	321	1281

Table 5. Soil Parameters – Final Optimized

Parameter	**Fill**	**T1**	**T2**	**Ma**	**Mb**	**P1**
φ(°)	32	31	32	32	32	17.7
c (ksf)	0	0	0	0.24	0.24	0.70
ψ(°)	2	1	1	2	2	0
E_{50}^{ref} (ksf)	57	38	184	107	134	759
E_{oed}^{ref} (ksf)	38	26	129	75	93.5	531
E_{ur}^{ref} (ksf)	171	113	553	321	401	2280

To provide a more accurate reflection of the subsurface conditions and the observed performance of the system, we divided the M layer into two separate layers in the optimized model. The upper layer was re-labeled Ma and the lower was re-labeled Mb. This allowed us to use a separate range of soil parameters and increase the stiffness of the layer with depth. The increase in stiffness in the M layer with depth was underestimated during design. The optimized soil parameters for both M-layers tended to be between the parameters derived from the laboratory testing and the DMT. The optimized value was significantly larger than estimated for the Mb layer. The stiffness parameters using all three methods are summarized in Table 6.

Table 7 shows the initial and optimized values for the P1 layer. As with the M layer the optimized values are between the values derived from the laboratory tests and the DMT, although the DMT values are closer the optimized values.

Figure 7 shows the predicted deflection using the optimized soil parameters of the Inclinometer I-6 and compares it with the actual inclinometer reading taken in the field.

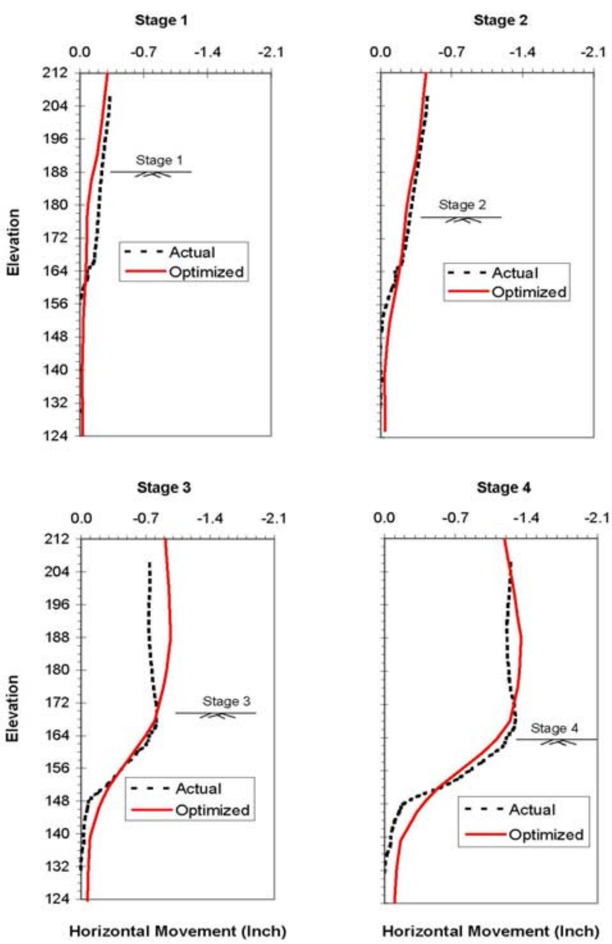

Fig. 7. Inclinometer Deflections using Optimized Soil Parameters

Table 6. M Layer Stiffness Parameters

	Estimated		Optimized	
Parameter	**Laboratory**	**DMT**	**Ma**	**Mb**
E_{50}^{ref} (ksf)	80	176	107	134
E_{oed}^{ref} (ksf)	56	123	75	93.5
E_{ur}^{ref} (ksf)	240	527	321	401

Table 7. P1 Layer Stiffness Parameters

	Estimated		Optimized
Parameter	**Laboratory**	**DMT**	**P1**
E_{50}^{ref} (ksf)	427	892	759
E_{oed}^{ref} (ksf)	300	625	531
E_{ur}^{ref} (ksf)	1281	2677	2280

Again for the P1 layer the optimized values were between the Laboratory derived values and the DMT.

Table 8 summarizes the differences between the optimized values and the values derived from laboratory testing. The positive values indicate that the optimized values are stiffer than the values derived from the laboratory testing. Table 9 summarizes the differences between the optimized values and the values derived from the DMT probes. The positive values indicate that the optimized values are stiffer than the values derived from the laboratory testing, and negative values imply that the estimated values were larger than the observed values and therefore unconservative.

Table 8. Differential between Optimized and laboratory Derived Soil Parameters

Parameter	Fill	T1	T2	Ma	Mb	P1
E_{50}^{ref} (ksf)	29	25	62	0	54	332
E_{oed}^{ref} (ksf)	21	17	43	0	37	232
E_{ur}^{ref} (ksf)	85	74	184	0	161	999

For the granular T2 layer both estimates of the stiffness parameters were underestimated, and, not surprisingly, the DMT seemed to provide a better estimate of the soil parameters than the SPT method.

Table 9. Differential between Optimized and DMT Derived Soil Parameters

Parameter	Fill	T1	T2	Ma	Mb	P1
E_{50}^{ref} (ksf)	29	25	62	-96	-42	-133
E_{oed}^{ref} (ksf)	21	17	43	-67	-30	-94
E_{ur}^{ref} (ksf)	85	74	184	-287	-126	-397

The deflection of the soldier pile wall using the PYWALL and PLAXIS software applications are compared in Fig.8. The deflections using PLAXIS are based on the optimized soil parameters. No inclinometers were installed in the face of the wall. The deflection of the soldier pile computed by PYWALL is approximately one half of that computed by PLAXIS. Since the PLAXIS simulation is based on the optimized soil values it is assumed that the PLAXIS results are close to the actual deflection of the wall. If twice the assumed lateral load is applied to the wall the PYWall results are similar to the PLAXIS results adjusted for zero deflection at the base of the soldier pile.

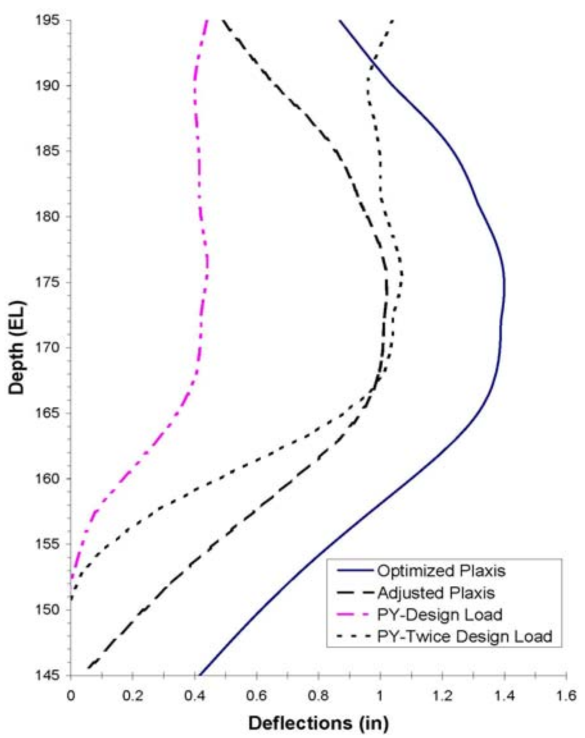

Fig. 8. Deflection of the soldier pile wall comparing PYWall and PLAXIS results

The condition of the pavement and parapet on the I-295 ramps were frequently inspected during construction. No noticeable movements were detected. There was no gap between the pavement and the parapet as would have been expected, and there did not seem to be any differential movement of the wall panels even though the surveyed deformation monitoring points indicated over an inch of movement. There were no signs of distress in the ramp areas in the pavement or median. The incremental differential movements were small enough that there was little to no visible signs of excessive deflections.

CONCLUSIONS

The FEM and p-y methods are reasonable and useful tools to use not only during design but also during construction to provide additional predictions and evaluations of the behavior of the structure and to possibly adjust instrumentation criteria based on observations. Using the DMT overestimated the stiffness of the soils, but the estimated deflections were not underestimated by very much and the results were more accurate than the parameters based on laboratory testing. Using the laboratory testing to estimate the soil parameters provided more conservative estimates of the wall deflections. The larger portion of the errors in estimating deflection probably had to do the choice of stratification by the authors than from the choice of parameters.

Fig. 11. Finished Wall with MSE

Combined with evaluations of the more traditional methods a sound design and set of contract documents were developed. These newer techniques can include several factors in the analysis that more traditional methods cannot include. The range of some of these parameters can be narrowed down during design. Therefore, the engineer should consider a range of likely parameters for these factors in the design to predict a range of outcomes. For the soil parameters, different testing methods, knowledge of the local geology and engineering judgment can be a guide to selecting the appropriate range of values and the drainage case. The instrumentation plan can then be developed to measure these uncertain outcomes.

Other factors are beyond the control of the engineer and are determined during construction by the Contractor. The engineer should have a sense of what is possible and likely based on experience and a knowledge of local construction practices to develop a range of likely construction techniques to be used on a project. The engineer should also make some attempt to predict how these construction practices and sequences could affect the final structure and nearby existing structures.

If any of the possible outcomes are undesirable it is usually better to simply develop an instrumentation plan with the appropriate threshold and limiting values than to try to limit the means and methods of the Contractor. In some cases however, the specifications may need to preclude specific construction methods or sequences that could lead to undesirable outcomes.

Both the FEM and p-y methods can be useful in design and can be used for the following tasks.

1. FEM and p-y techniques can be used to estimate effect of locating instrumentation in the actual areas that are available and accessible.
2. Developing threshold and limiting criteria for the instrumentation
3. Comparing alternatives during design
4. Comparing the range of soil parameters possible
5. Evaluating different sequences of construction or construction techniques

Based on this study, the following can be concluded.

1. Recalibration using back analysis techniques can be useful during construction to monitor and give some context to the instrumentation results as suggested by Finno and Calvello (2005).
2. Back analysis could be used to update and possibly revise the threshold and limiting values contained in an instrumentation plan.
3. DMT can be used to provide more accurate deflection estimates but could be unconservative.
4. No one test or method should not use in isolation: In situ testing, SPT, laboratory testing using different tests and stress paths, and knowledge of the local geology should all be used to develop the range of possible soil parameters. The is no substitute for understanding the local geology.
5. Laboratory testing using compressive strength tests underestimated deflections - consider triaxial extension tests for more accuracy since it would tend to mimic the actual stress path that the in situ soils will undergo during construction.
6. Given that the laboratory based parameters led to reasonably accurate results consistent with the observations, the traditional method of estimating parameters is reasonable, but greater accuracy could be obtained by also evaluating the DMT derived estimates of stress-strain parameters.

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