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Tied-Back Wall Failure, Boston, MA

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TIED-BACK WALL FAILURE, BOSTON, MA

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ABSTRACT

Excavation for Boston's Central Artery project included one of the most interesting temporary excavation support system failures in recent history (1993). The wall moved much more than predicted at a depth of 41 feet, approximately 2/3 of the final 60 foot excavation depth. Jet grouting was used to stabilize the bottom of the excavation before proceeding to the full depth.

The excavation support system provided space for a cut and cover section of Interstate I-90's Third Harbor Tunnel approach to Logan Airport. The permanent structure is a concrete box section of the tunnel. The excavation system consisted of a tied-back soil mix wall (SMW) penetrating a thick zone of over-consolidated Boston Blue Clay.

Previous published papers and published discussions explore the possible causes of the failure. This paper investigates some of the key issues and questions raised from this case history as the project remains a fertile topic for reflection, re-examination of the issues related to bottom stability, the common use of the assumption of undrained conditions, selection of appropriate soil parameters and methods for the prediction of movements associated with excavation support systems.

INTRODUCTION

Boston's Central Artery project included a so-called Third Harbor Tunnel (Ted Williams Tunnel) connecting I-90 from Boston to East Boston's Logan Airport. The locations of the project and area of interest are shown on Fig. 1. The tunnel transitioned from an immersed tube (abutting contract to the south) below Boston Harbor into a buried box section ultimately daylighting at portals to the airport. This required below-grade "cut and cover" construction of a heavy reinforced concrete base slab, walls and roof for mainline tunnel and ramps.

This tunnel section contract was about 3350 feet long (Mainline ML Station 134+50 to 168+05) including ramps, and required cuts up to depths of 88 feet at the junction of the immersed tube section of the project (ML Station 134+50). Portions of the new construction were adjacent to the airport taxiway. Detail for the area of interest is shown on Fig. 2.

A temporary excavation system was required to construct the box section. Project geotechnical design reports were developed to provide subsurface conditions and geotechnical information. Concerns for bottom stability in the marine clay and potential for ground movements near the taxiway were identified in advance by the designers. At the time of design a structurally stiff, braced concrete diaphragm wall system was identified as a suitable support system. As is often the case, contract specifications permitted submission of proposed alternates.

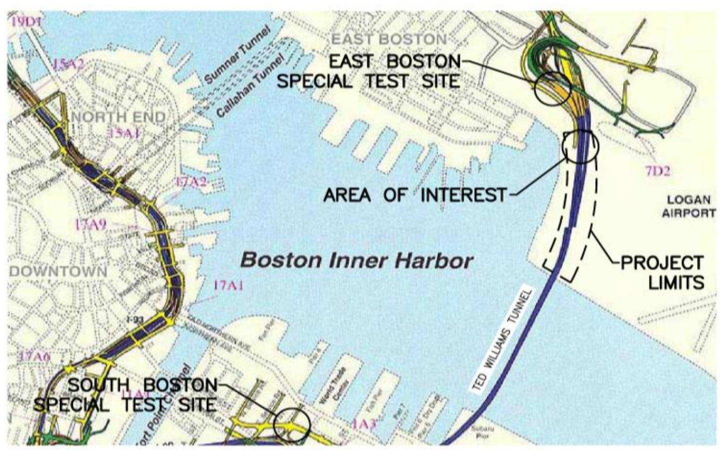

Fig. 1. Project Location, East Boston, Massachusetts

The system selected by the contractor was a tied-back soil mix wall (SMW); this was a relatively new technology at the time.

During excavation at a depth of about 41 feet, or about 2/3 of the final excavation depth of 60 feet, monitoring instrumentation indicated almost 9 inches of horizontal movement at an inclinometer positioned at ML Station 157+50 along the south side of the excavation (Ramp T/D). Ramp T/D is lower than the mainline tunnel and was the deepest part of the excavation in this area.

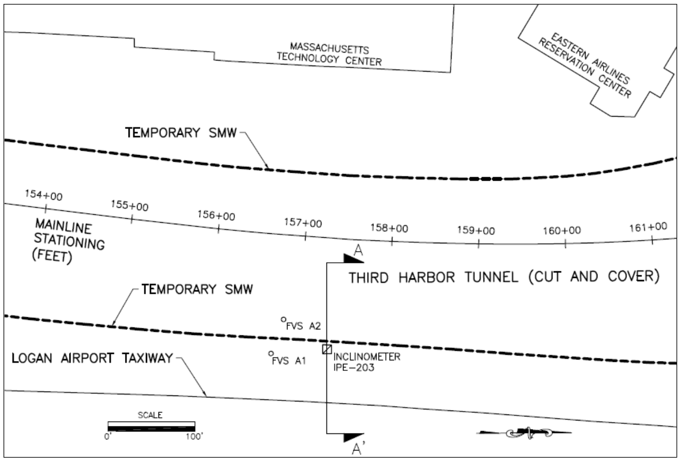

Fig. 2. Plan

Figure 3 shows a cross-section of the SMW and relative movements at the time of failure (Stage 3 excavation). Because of continuing movements, the excavation was promptly backfilled to just below the second level of tiebacks and a jet grouting remediation program for strengthening the soils at the bottom of the excavation was implemented as described by O'Rourke et al. (1997b).

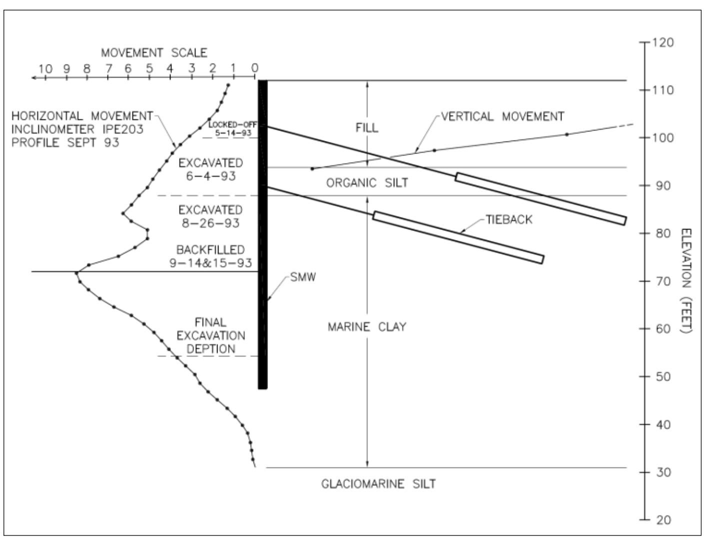

Fig. 3. Soil Mix Wall Movement

The amount of movement that would have occurred if the excavation was not quickly backfilled is unknown. Remedial jet grout columns were installed to buttress the passive zone at the base of the excavation. Construction then resumed with excavation and installation of the third through sixth levels of tiebacks.

SUBSURFACE CONDITIONS

Subsurface conditions and the bottom of structure elevation along Ramp T/D are shown on Fig. 4 (note that ML Station 157+50 corresponds to T/D Station 257+50).

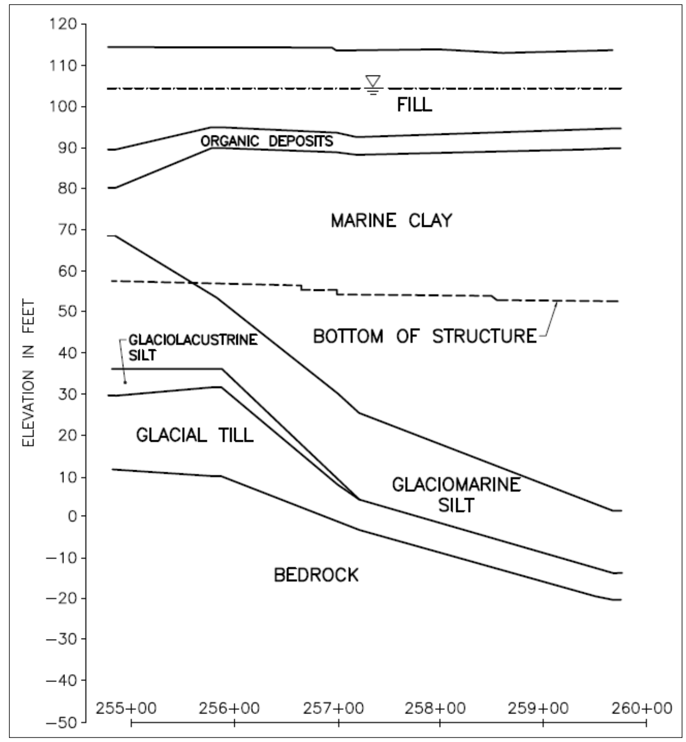

Fig. 4. Subsurface Conditions at Ramp T/D (after Haley and Aldrich 1991)

PREDICTED VERSUS ACTUAL WALL MOVEMENTS

Wall movements were predicted in advance of excavation as part of the contractor's design process and submitted for review. Predicted wall movements were calculated (O'Rourke 1993) using the system stiffness method (SSM). SSM is a semi-empirical procedure developed by Clough et al. (1989).

The Clough SSM is depicted on Fig. 5. The Clough method was combined with modifications by a method developed by O'Rourke (1992) to predict the range of movements. The analysis incorporated undrained shear strengths.

This method uses inputs of excavation geometry, wall system stiffness and factor of safety against basal heave for prediction of maximum lateral wall movements. A key component in the computation for factor of safety against basal heave is undrained shear strength, in this case the undrained shear strength of the marine clay.

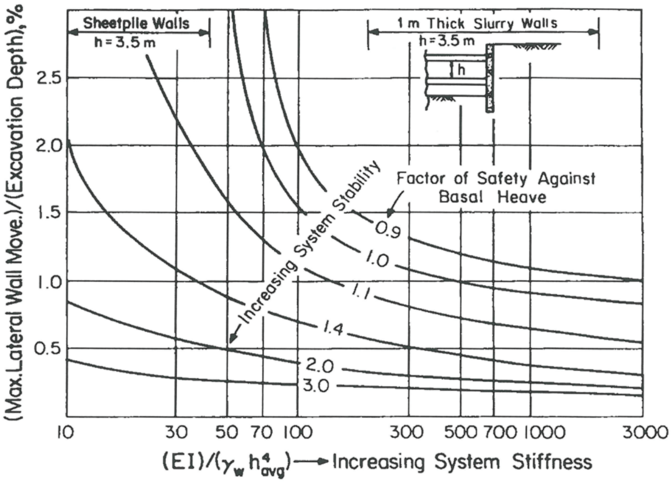

Fig. 5. System Stiffness Method (from Clough et al. 1989)

Predicted maximum wall movements (using the Clough method) for the final excavation depth in the failure area were estimated to be about 4.6 to 5.4 inches and 3.0 to 4.8 inches with the O'Rourke (1992) modification. Ground surface settlements of similar amounts were estimated for the area behind the excavation.

Thus, the prediction differed from the actual performance by a wide margin as measured horizontal movement at the inclinometer was almost 9 inches at about 2/3 of the final excavation depth (Stage 3) versus a predicted 3 to 5 inches estimated at final excavation depth.

POST FAILURE ANALYSES

The O'Rourke et al. (1997a) post failure analysis attributes the failure to deep rotational stability on the basis of limit equilibrium (LE) and finite element (FE) analyses, and post-failure vane shear strength tests.

The paper was followed by published discussions by Schnabel (1998) and by Whittle and Ladd (1998), and then by a closure from O'Rourke (1998).

The Whittle and Ladd discussion raises issues regarding the limitations of LE calculations, proper selection of stress-strain parameters for the LE and FE analyses, effects of partial drainage, progressive failure mechanisms and anisotropic characteristics of the clay.

The following extends the previous discussions with some practical considerations for future designs. This will be done by reviewing the assumptions that we typically use in such analyses and also as they relate to this case study.

ASSUMPTIONS

We commonly make four general assumptions during lateral support system design in cohesive soils:

#1 Undrained Conditions

Undrained soil conditions are normally assumed for the "short term" construction case for excavations in clay. The validity of this idealized assumption should be reviewed for each case. Some questions may include…….How long will the excavation be open? Are there conditions that may cause or accelerate drainage for portions of the excavation (e.g., swell at the base of excavation)?

#2 Clay Shear Strength

A strength for the clay is assumed in design. How should we select representative stress-strain parameters? How shall we account for anisotropic stress conditions, strain rate (i.e., lab testing versus actual construction), drainage, stress history and other factors (e.g., sample disturbance)?

#3 Mode of Failure

The likely modes of failure must be assumed.

#4 Deformation Prediction Model

For a major project with complex soil profiles and multiple stages of excavation – is the assumption of a semi-empirical approach (such as SSM) sufficient for predicting movements? When should FE modeling be used?

REVIEW OF ASSUMPTIONS

Each of the foregoing assumptions are discussed below:

Assumption #1 - "Undrained"

The assumptions of "undrained" and "drained" conditions for excavations in a cohesive soil are the idealized "short term, end of construction" and "long term" cases, respectively. In reality, the actual condition is almost always one of some degree of partial drainage.

However, at the practice level, the choice of either the undrained or the drained case remains prevalent, primarily as this simplifies analyses considerably. The partially drained condition is largely ignored in practice. Typically, either one case or the other is assumed.

To help address this issue, the Stress Path Method (Lambe 1967) has been applied for partially drained conditions to model undrained shear and consolidation (von Rosenvinge 1980). Now, sophisticated numerical models can be used to simulate time-dependent behavior of excavations.

Use of stress paths to help understand partially drained behavior remains a powerful tool when used in conjunction with SSM and computer models.

Figure 6 illustrates the anisotropic loading conditions associated with a braced (or tied back) excavation.

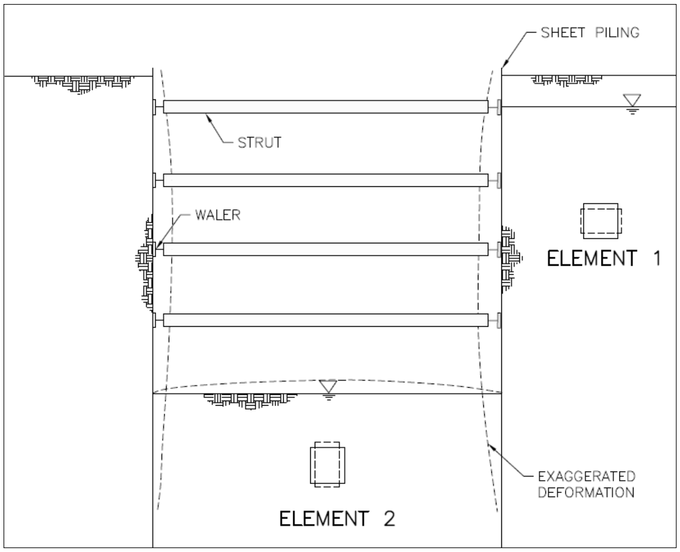

Fig. 6. Anisotropic Stress Conditions

Stress paths for a typical partially drained behavior for Element 2 at the excavation base are shown on Fig. 7 (K_o=1 case).

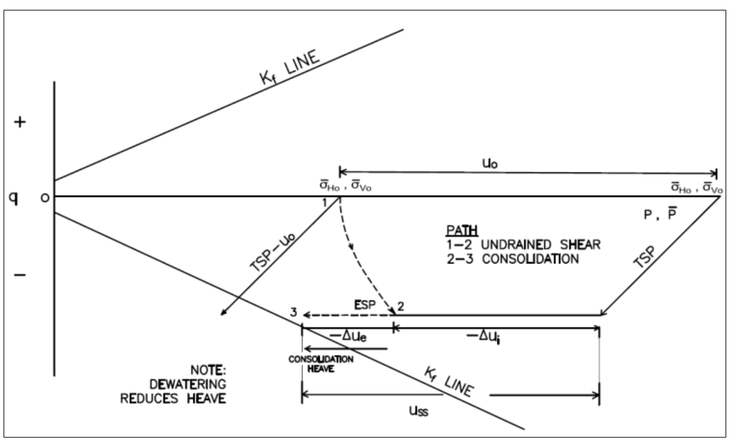

Fig. 7. Stress Path for Element 2 at Base of Excavation

For the base of an excavation in clay, a partial drainage condition may be critical. The excavation may be left open for an extended period of time, drainage paths may be altered, and/or new sources of water may be introduced at the surface. The bottom of the excavation may swell and the shear strength may be reduced in the process.

Given that the base of an excavation essentially acts as part of the support system, the possibility of strength loss due to drainage should be considered.

There are numerous past case histories and studies where the effect of time and drainage has been observed. Three particular studies are briefly mentioned herein to strengthen the point that this phenomenon is not a recent discovery.

Lambe (1968) reported dissipation of negative excess pore pressures in an excavation in Boston Blue Clay during a 24 day period while the cut was open.

Clough and Davidson (1977) presented a case where sheet pile movement continued and more than doubled after excavation was made to subgrade and construction was delayed by a sixteen day strike.

Osaimi and Clough (1979) investigated pore pressure dissipation for excavations using FE modeling. They concluded that dissipation in clay is likely to occur to a greater degree than previously believed.

<u>Assumption #2 - Clay Strength</u>

One fundamental lesson learned early in a geotechnical engineering education is that soils are rarely homogenous or isotropic. To make matters more complicated, there is stress anisotropy; different strengths result due to the manner in which the soil is stressed and sheared.

As shown in Fig. 6, such is the case for braced excavations. Extension-unloading is the simplified mode of applied stress for the base of the excavation (Element 2), compression-unloading is the mode of applied stress behind the excavation (Element 1). Direct simple shear (DSS) may represent the horizontal shear surfaces (e.g., at the base of a global slip circle).

Figure 8 shows example stress paths from triaxial extension unloading tests with superimposed contours of strain. If these same samples were loaded by triaxial compression, significantly higher shear strengths should result as shown on Fig. 8.

<u>Assumption #3 - Failure Mode</u>

Rotational, translational block-wedge and progressive failure modes may be applicable. For the subject case, consideration should be given to a soft base where the soil provides lateral stability to the wall at the bottom of the excavation.

This is a critical zone expected to have the weakest strength due to stress anisotropy, and is most susceptible to weakening during unloading, drainage, swell and disturbance from construction activity.

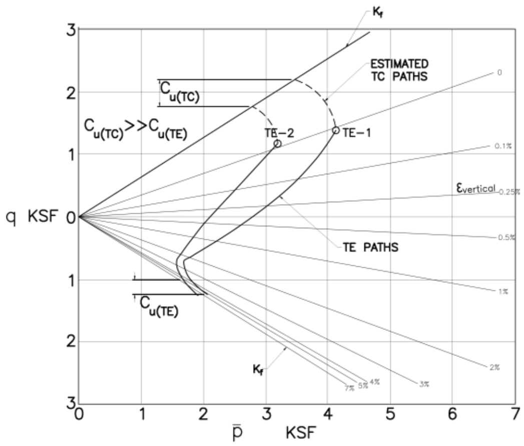

Fig. 8. Effective Stress Paths for Triaxial Extension Unloading Tests on Kawasaki Clay modified after von Rosenvinge (1980)

Introduce the element of time and the ensuing drainage of excess pore water pressure, and additional strains may occur compared to the undrained shear strains shown on Fig. 7.

Assumption #4 – Deformation Prediction Model

Sophisticated numerical FE models have been available for over three decades to help model soil-structure interaction, including staged construction and non-linear soil behavior.

An advantage to FE models over SSM and LE models is the ability to address strain compatibility. Characterizing the soil and selecting the appropriate soil stress-strain input is the challenge for each of these methods.

REVIEW OF CENTRAL ARTERY CASE HISTORY

These four assumptions are reviewed for this case history.

Undrained

Excavation in the failure area began in spring of 1993 and continued through the summer until mid-September when the excessive movements paused excavation. The excavation was effectively open for about four months. It was reported that surface water collected and ponded at the base of excavation. Thus, there was an opportunity for the clay to swell and lose strength.

Shortly after the failure, vane shear tests were performed both inside the excavation and just outside the soil mix wall. These results are plotted in Fig. 9. Lower undrained shear strengths were documented inside of the excavation. This suggests that some drainage and softening occurred. McGinn and O'Rourke (2000) present an analysis that significant drainage and strength loss (average of 22%) did occur at the base of the excavation.

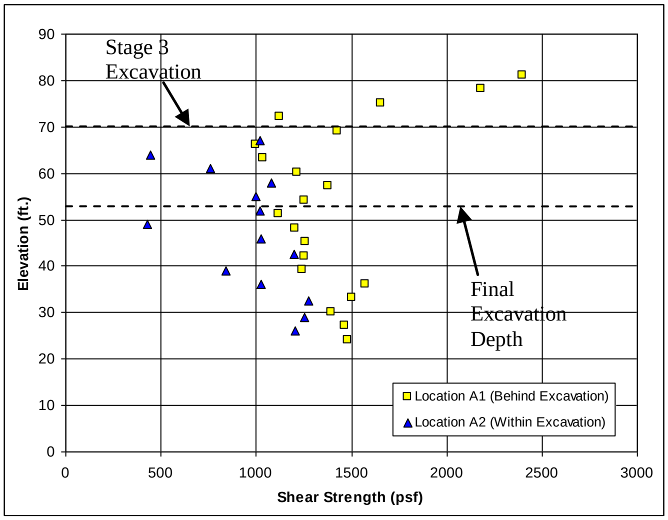

Fig. 9. Field Vane Shear Test Results, Shear Strength vs. Elevation – Inside and Outside of the Excavation after Failure

Clay Strength

Figure 10 is a plot based upon the geotechnical design report. The plot shows raw laboratory undrained shear strength results from CIU and UU compression tests, as well as a SHANSEP (Ladd and Foott 1974) DSS strength profile based on over-consolidation ratio (OCR) data and the empirical SHANSEP correlation below.

$$Cu = \sigma_v \,(S) OCR^m \qquad (1)$$

Where S=0.2 and m=0.8 in Fig. 10

The CIU and UU tests are from samples recovered from borings in the vicinity of the subject wall failure.

There is significant scatter within the data (especially, the UU results). Note that the CIU strengths plot to the left of the DSS profile above Elev. 55 and to the right below Elev. 55.

The SHANSEP curve was based on empirical strength relationships between DSS and OCR. OCR was developed from consolidation testing of samples from vicinity borings.

The SHANSEP parameters were largely confirmed by Haley and Aldrich (1993) by a Special Test Program (STP) in South Boston and East Boston (both sites shown on Figure 1). Typical values of S and m from the STP are shown in Table 1.

Table 1
Summary of Normalized Cu Parameters

Test	S	M
CK_oUC	0.28	0.68
CK_oUE	0.14	0.83
CK_oDSS[1]	0.20	0.77
CK_oDSS[2]	0.18	0.66

[1] Overconsolidated
[2] Slightly Overconsolidated

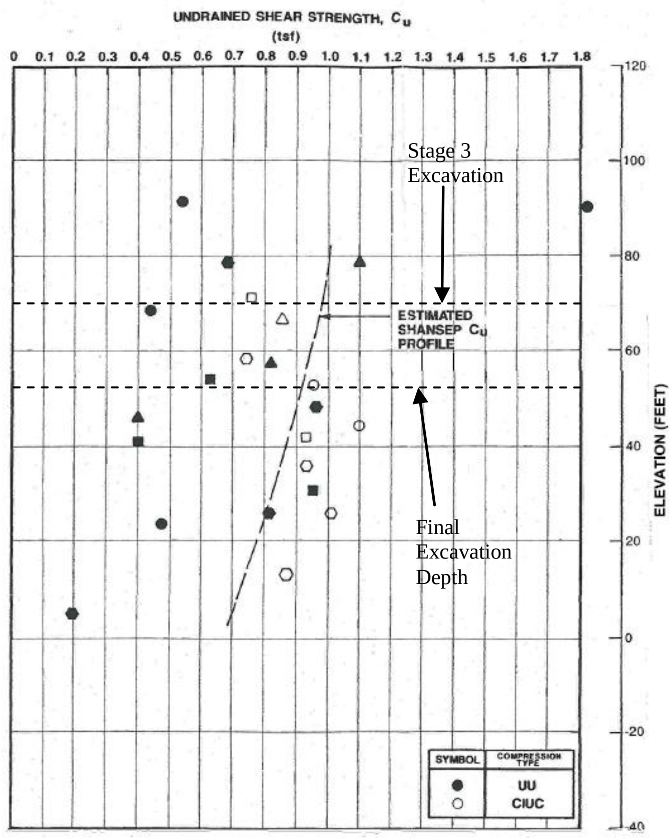

Fig. 10. Cu Profile (modified after Haley and Aldrich 1991)*

**added excavation depths and removed reference to "see note 6" referring to equation 1*

Strengths selected and used in the previously mentioned SSM prediction were very close to the SHANSEP strength DSS profile shown on Fig. 10 with a range of 1 tsf to 0.7 tsf decreasing with depth.

For depths of 33 to 64 feet, the assumed average strength was about 0.85 tsf.

Individual CIUC strength testing data from Fig. 10 tests are provided in Table 2 and re-plotted on Fig. 11.

Also provided in Fig. 11 are estimated DSS and CIUE strengths adjusted by their respective SHANSEP parameters to the CIUC results.

CIUE and DSS strengths would be expected to be about 55% to 62% of CIUC strengths, respectively.

Table 2
Summary of Undrained Shear Strength, Cu (tsf)

CIUC Test No.- Depth (ft)	Cu	Estimated Cu DSS/CIUE[1]
#5 - 42.8	0.760	0.47/0.42
#4 - 45.3	0.854	0.53/0.47
#6 - 54.3	0.742	0.46/0.41
#1 - 60.5	0.955	0.59/0.53
#14 - 65.6	0.966	0.60/0.53
#2 - 69.3	1.099	0.68/0.60
#13 - 71.8	0.93	0.58/0.51
#7 - 77.3	0.932	0.58/0.51
#15 - 86.6	1.066	0.66/0.59
#16 - 107.9	1.227	0.76/0.67

[1] Slightly Overconsolidated (OCR=2)

The adjusted "equivalent" DSS and CIUE strengths are about 60% of the DSS values assumed for design. Also, both the trend and magnitude of these strengths compare favorably to the vane shear tests shown on Fig. 10.

Assumption of the lower strengths would have a profound impact on the SSM prediction. Estimated basal stability would be lower; horizontal wall movements would increase.

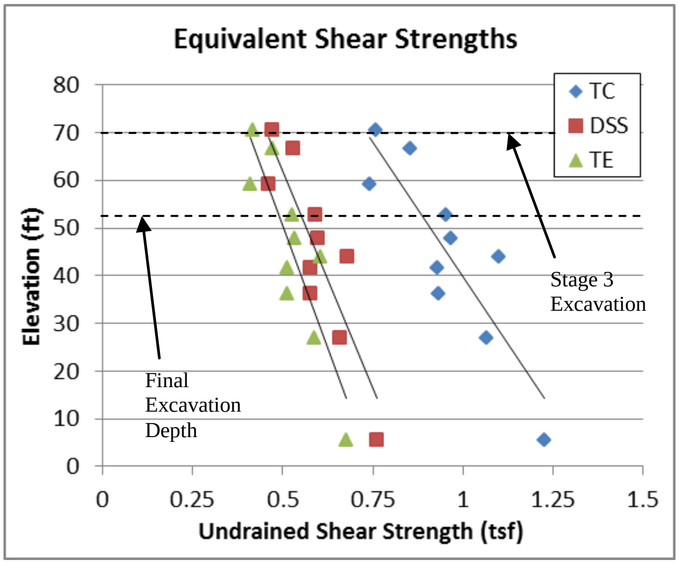

Fig. 11. CIUC Strength Testing Data and Estimated DSS and TE Strengths (Cu)

<u>Failure Mode</u>

O'Rourke (1997a) suggests that the principal mode of failure was circular. Ladd and Whittle (1998) highlight a number of issues associated with assumptions used in the analyses. Many of those issues are also discussed in this paper, including the effects of stress anisotropy, limitations of LE calculations, partial drainage and progressive failure mechanisms.

It is the author's opinion that the observed mode of failure is progressive. The clay at the base of the excavation became overstressed in the passive wedge at the toe of the wall. This is supported by FE analyses discussed below.

A review of plastic points in the FE model discussed below suggests that a deep rotational slip surface had only partially developed at this point in the excavation.

Prediction of Movements

The STP also provided Young's Modulus data. Figure 12 shows the variation of Young's Modulus between TC, DSS and TE from the STP. Note that TE has the lowest modulus. As such, a lower modulus should be considered for the base of the excavation for movement predictions.

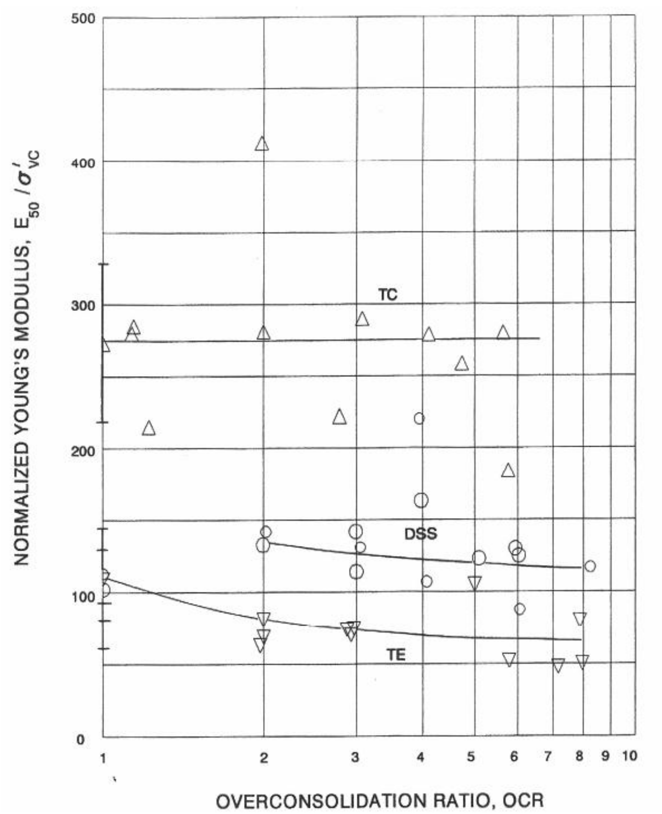

Fig. 12. OCR vs. Normalized Young's Modulus (after Haley and Aldrich 1993)

The author applied a CIU-derived DSS strength to an SSM analysis. This resulted in a predicted maximum horizontal deformation of about 7 inches for the failure depth Stage 3 (Fig. 3).

Post-failure FE modeling by O'Rourke (1997a) estimates movements more consistent with performance using a lower strength based on the post construction field vane shear tests (Fig. 9).

Post-failure FE modeling (Fig. 13) was performed by the author using PLAXIS for the Stage 3 geometry shown on Fig. 3. In the model TE strengths were used inside of the excavation and TC strengths were applied outside of the excavation. FE results found maximum movements of about 9 inches at the bottom of the excavation.

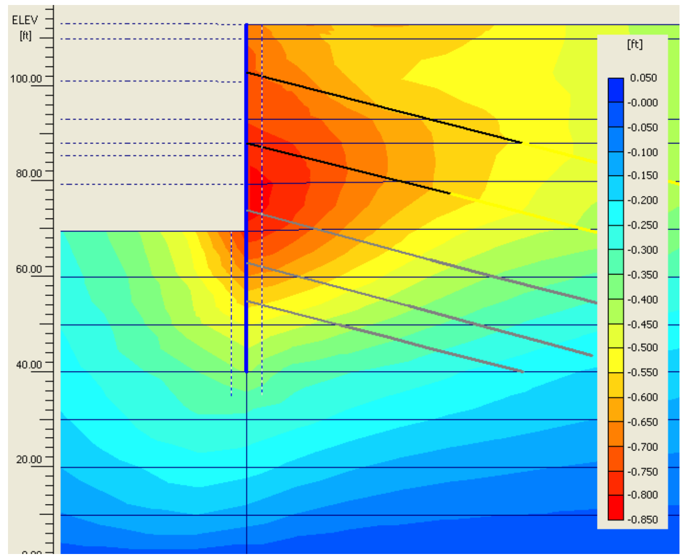

Fig. 13. Post-failure FE Model Showing Contours of Predicted Horizontal Displacements

A comparison of predicted movements by the author using various Cu assumptions, and FE and SSM methodologies compared to the observed movements is provided in Table 3.

Table 3
Summary of Movement Estimates

Case	Method	Notes	Maximum Horizontal Movement (in.)
TC	FE	CIU Cu Strength Profile[1]	5.1
TC/TE	FE	CIU Cu - outside excavation TE Cu - inside excavation[1]	9.3
DSS	FE	DSS Cu[1]	12.1
DSS	SSM	DSS Cu[1]	7
Actual	Measured	Inclinometer	8.5

[1] Based on Table 2 and Figure 11: triaxial extension (TE) and DSS Cu estimated from TC (CIU) tests

Notice that the TC/TE movement is reasonably close to the measured inclinometer readings before backfilling. The revised SSM estimate is close but less than what was observed. As discussed earlier, had the excavation not been quickly backfilled, higher movements would likely have been measured. Moreover, the above predictions do not account for softening of the base of the excavation due to heave.

CONCLUSIONS

1. DSS strengths derived from OCR correlations were interestingly higher or similar to the expected CIU strengths at this location. The expected result would be CIU strengths that exceeded DSS values.

2. Post-failure analysis indicates that strengths derived from vane or CIU tests (adjusted to DSS levels) provided a better match to actual performance.

3. Vane shear values obtained within the cut were lower than those behind the wall. This suggests that the excavation was open long enough to behave in a partially drained manner and lose some strength by swelling.

4. Movement Predictions based on SSM and FE models are only as good as the input. This case underscores the need to scrutinize soils testing data and consider respective modes of shear.

4. For FE modeling, consideration should be given to using triaxial extension stress-strain parameters within the excavation and triaxial compression stress strain parameters behind the excavation.

5. For complex excavations, such as this case history, it is prudent to consider the effects of partial drainage and the potential effects on strength and base stability.

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