Design and performance of a deep tiedback sheet pile wall in soft clay

Conception et performances d'un mur de pieux dans une argile molle

D.M.Cacoilo, P.H.Edinger & G.J.Tamaro – *Mueser Rutledge Consulting Engineers, New York, N.Y., USA*

INTRODUCTION

The excavation for the 760 meter long South Approach to the Third Harbor Tunnel in Boston, Massachusetts was 30 to 60 meters wide, 11 to 19 meters deep and extended through loose fills, into soft organic silts and clay.

Soldier pile and lagging, in-situ soil mixing and tangent pile alternatives were considered for the support of the excavation. However, a continuous steel sheet pile wall was selected considering schedule, constructibility and cost.

The geotechnical report in the Contract Documents deprecated use of tiebacks for sheeting support because of anticipated large brace loads and the presumed poor anchoring capacity in the soft clays. The report suggested that tiebacks could be feasible where anchored in soils other than soft clay. Multi-levels of rakers and cross-lot bracing were evaluated but found impractical because they obstructed excavation and concrete placement. A multi-level tieback system was ultimately selected to provide an open work area and eliminate brace or raker penetrations through the permanent structure. The tieback anchors required bond zones in the soft clay and capacities larger than previously achieved in this type of soil.

This paper discusses the design of the tiedback sheeting, the tieback test program and the performance of the completed system.

SUBSURFACE CONDITIONS

The subsurface profile on the alignment is generally 6 meters of fill overlying organic silt averaging 4.5 meters thick, underlain by marine clay and silt commonly known as the Boston Blue Clay. The clay is 15 to 25 meters thick, overlying thin glaciomarine soils and glacial till. Bedrock is 27 to 40 meters below ground surface.

The upper fill is loose to medium dense fine to coarse sand, with local inclusions of organic silt and construction debris. The lower fill is probably dredge spoil consisting of very soft silty clay. The uppermost natural soil is soft, dark gray organic silt with trace to little fine sand. The surface of the underlying marine clay is desiccated and very stiff to hard. Below this crust, the clay strength decreases from medium to soft with depth. The clay strength profiles along the alignment are shown on Figure 1.

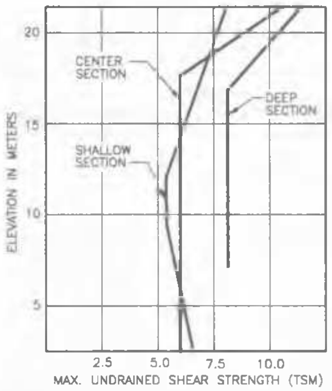

FIGURE 1 – CLAY STRENGTH PROFILE

EXCAVATION SUPPORT SYSTEM DESIGN CRITERIA AND METHOD OF ANALYSIS

The Contract Documents required use of a specified, empirical design pressure diagram on the entire alignment, regardless of variations in soil strata. However, review of actual soil profiles indicated that the empirical diagram and method of analysis would obtain very conservative and uneconomic designs.

As an alternative to the specified criteria, a detailed excavation staging analysis was performed using actual soil properties and stratifications. The pressure diagrams and bracing reactions obtained by the specified and the alternative procedures are compared on Figures 2 and 3 for an excavation to a depth of 16 meters.

The staging analysis was performed as follows:

1. The 1500 meters of excavation support wall were divided into 12 design sections based on the subsurface data.
2. Sheeting design pressures in the fill and organic silt above the top of the marine clay were calculated using Rankine soil pressures based on effective soil weight, plus ground water pressure. In the marine clay, soil pressures were computed on the basis of total overburden weights. The marine clay was assumed impermeable since there was no evidence of vertical fissures; thus, water pressures were not considered separately.

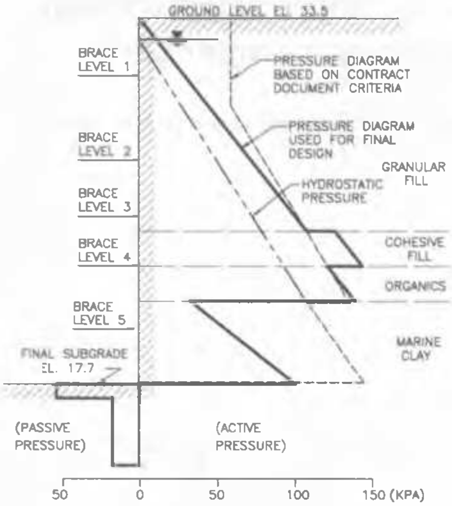

FIGURE 2
COMPARISON OF SOIL PRESSURE DIAGRAMS

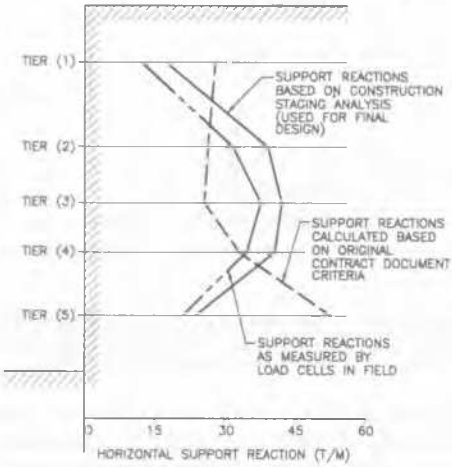

FIGURE 3
COMPARISON OF SUPPORT REACTIONS

3. No factor of safety was applied to passive pressures. Instead, stresses in the sheet piles, wales and tieback tendons were limited to 60 percent of the steel yield strength, and the tieback working load was limited to one half the lowest tested ultimate anchor capacity.

4. Sheeting bending moments and support reactions were calculated for the following stages:

    a. Sheeting acting as a cantilever, prior to installing the first wale level.

    b. Sheeting supported at the first wale level and by soil below subgrade.

    c. In all subsequent excavation stages the sheeting was analyzed as a continuous beam spanning over all wale levels and supported by soil below subgrade.

The final design reaction at each support level is the maximum value obtained from the staging analysis.

As shown on Figure 3, field measurement of tieback forces using load cells generally obtained forces in agreement with the values calculated by the staging analysis. Tieback reactions calculated using the Contract Document empirical criteria obtained much higher forces than the measured values at the upper and lower tiers and lower forces in the middle tiers.

TIEBACK SYSTEM DESIGN AND TESTING

Analyses indicated that tieback reactions on the order of 42 tons per meter had to be developed at the lower wales to support the sheeting. Thus, a minimum anchor working capacity of 77 tons was required. Although this capacity had not been previously achieved in clays similar to those at this site, it was judged that it should be feasible if special drilling and postgrouting procedures were used during anchor installation.

A tieback test program was undertaken to evaluate anchor performance, including group action effects. Group action was critical because of required close anchor spacing. The test program simulated actual excavation staging and anticipated tieback spacings. The test set up included 10 anchors, installed in two tiers through temporary steel sheet piles driven specifically for the test. The target maximum test load was 182 tons.

The upper tier contained 5 tiebacks (Numbers 1 through 5) anchored in the desiccated marine clay crust. These were installed after the area in front of the sheeting was excavated to a depth of 0.6 m below the anchor level to simulate the cantilever stage. The tiebacks were approximately 28 m long with a 12 m bond zone, installed at 1.5 m spacing and at a 30 degree angle below the horizontal. The lower tier of 5 tiebacks (Numbers 6 through 10) were anchored in the soft clay below the crust. These were installed 4 m below the upper tier after the upper tier was tested and locked off and the area in front of the sheeting was excavated to a depth of 0.6 m below the lower tieback level. This sequence simulated the second phase of the proposed excavation sequence. The lower tiebacks were approximately 32 m long with a 12 m bond zone, installed in vertical alignment with and parallel to the upper tiebacks.

Each tieback was initially postgrouted approximately 24 hours after tendon installation, followed by one or two additional groutings at approximately 24 hour intervals.

Pull out tests were performed on each end tieback in each tier to determine ultimate capacity. The test results are shown on Table 1. Based on the pull out tests, the tieback design capacity was established as 77 tons, one half of the maximum test load as measured by the load cell.

The remaining 6 tiebacks were performance and creep tested to 116 tons, 1.5 times the design load established by the pull out tests. To assess the influence of group loading, the center tieback in each tier was tested last, after all adjacent tiebacks were locked-off at the design load. The tieback performance and creep test results are shown on Table 2.

The performance/creep test data indicates that anchors post grouted with the use of a packer in both soft and stiff

TABLE 1
ANCHOR PULL-OUT TEST CAPACITIES

Tieback Number	Tieback Anchor Zone Clay Consistency	Number of Post Grouts	Type of Postgrouting	Tieback Pull Out Test Ultimate Capacity (Metric Unit)	
				Jack Reading	Load Cell Reading
1	STIFF	2	Packer	175	156
5	STIFF	3	Packer	188	(INOPERATIVE)
6	SOFT	3	Packer	191	163
10	SOFT	2	Packer	144	155

TABLE 2
TEST ANCHOR PERFORMANCE DATA

Tieback Number	Tieback Anchor Zone Clay Consistency	Number of Postgrouts	Type of Postgrouting	Maximum Test Load (Metric Tons)		Anchor Elongation Per Log Cycle (mm)	
				Jack Reading	Load Cell Reading	1 - 10 min.	10 - 100 min.
2	STIFF	2	Packer	135	115	.79	1.30
3	STIFF	2	Packer	134	116	.25	.36
4	STIFF	3	No Packer	124	116	.46	.74
7	SOFT	3	Packer	129	116	.23	.69
8	SOFT	3	Packer	132	116	.23	.74
9	SOFT	3	No Packer	129	116	(FAILED)	--

clay (Nos. 2, 3, 7 and 8) and the anchor post grouted without a packer in the stiff clay (No. 4) performed satisfactorily at the design load and met the limiting creep criteria. However, anchor No. 9 which was post grouted in soft clay without a packer failed to meet the creep criteria.

The tieback test program demonstrated that anchors with a working capacity of 77 tons are feasible in the marine clay if the clay in the bond zone is not excessively disturbed during drilling, the bond zone is properly flushed and cleaned with grout, and tiebacks are post grouted twice in the stiff clay crust and 3 times in the underlying soft zone using a packer.

TIEBACK INSTALLATION PROCEDURE DURING CONSTRUCTION

The final tieback installation specification permitted alternative installation procedures for production tiebacks provided that the working load and factors of safety estimated in the test program were maintained. The alternative production anchor installation procedure accepted by the General Contractor differed from the test anchor installation procedure, and was generally as follows:

1. A casing was advanced to the bottom of the hole using internal flush, rotary drilling. The casing was cleaned out with a roller bit and flushed with water. Initially, air was introduced in the flush water and the drill bit was maintained ahead of the casing. This procedure was discontinued because return of air and water outside of the casing and disturbance of adjacent, previously drilled anchors occurred.

2. After the casing was cleaned, the drill string was withdrawn and grout tremied into the casing. The tendon assembly was then inserted and the casing withdrawn.

3. Post grouting was initially performed without the use of a packer through a single grout pipe with valves located on 1.2 m centers. Typically, 3 postgrouts were performed on each anchor prior to testing.

This installation method for production tiebacks obtained about a 20 percent test failure rate. Ultimately, a total of 4 to 6 postgrouts were required to achieve design capacity. The postgrouting method was later modified to include two postgrout tubes. One tube grouted the lower half of the anchorage zone, and the second tube grouted the upper half. The revised procedure reduced the frequency of test failures; however, it was not as effective as drilling open-hole in the clay, flushing the bond zone with grout and postgrouting with a packer as was done during the test program.

EXCAVATION SUPPORT SYSTEM STABILITY CONSIDERATIONS

The presence of deep, soft clay significantly affected the overall stability of the excavation. Two-dimensional mass stability analyses were performed for each sheeting design section. Safety factors against mass stability failure between 1.1 and 1.4 were calculated. Figure 4 shows a cross section at a 16 m deep excavation design section where the factor of safety was 1.2. Parametric studies determined that the safety factor was essentially a function of excavation depth, and increased depth of sheeting, increased length of tiebacks or the use of internal rakers or cross-lot bracing would not significantly improve the mass stability.

The calculated factors of safety were generally lower than 1.5, the value typically desired to limit soil displacements. Therefore, soil deformations exceeding limiting values established in the Contract Documents were anticipated. The Contract Documents required implementation of remedial measures if sheeting movements exceeded a threshold value of .0075 times the depth of excavation, H; or, 0.12 meters for a 16 m deep section. Maximum sheeting movements were limited to .01 H. Measured sheeting movements at the 16 m deep section were about 0.16 meters, exceeding the threshold.

PERFORMANCE OF THE TIEDBACK SHEET PILE WALL SYSTEM AS RECORDED BY MONITORING DATA

Clusters of inclinometers, piezometers, settlement monitoring points, extensometers and load cells were installed to monitor the behavior of the excavation and the support system. An "observational approach" was implemented, with the Contractor, Designer and Owner cooperating to evaluate the monitoring data to establish acceptable excavation procedures. Because of the known marginal stability of the excavation, monitoring was performed on a daily basis until the 3 m thick base mat of the roadway structure secured the excavation bottom.

Figure 4 shows a summary plot of lateral movements recorded at a 16 meter excavation. Figure 5 is a plot of sheeting movement versus time at each tieback level and at the toe of the sheeting. The total lateral movement at this section was approximately 160 mm.

Sheeting movements appear to have been influenced by tieback grouting operations as well as by soil deformations resulting from the marginal mass stability. As shown in Figure 5, there were only slight movements recorded between anchor lock-off at each tier and excavation for the next tieback, including initial drilling and tremie grouting. However, significant sheeting movements occurred during post grouting of tier 3 anchors, where 8 mm at tier 1, 16 mm at tier 2 and 25 mm at tier 3 were recorded. Postgrout effects apparently extended to the tip of the sheeting as evidenced by the 25 mm of movement recorded at this location. Similar movements occurred at the upper tiers during postgrouting of tier 4. Tier 5 postgrouting effects were most pronounced at tiers 4, 5 and the tip of the sheeting. Only minor movements were recorded at tiers 1, 2 and 3 during postgrouting of tier 5.

As shown on Figure 4, the recorded movements between completion of anchor installation and final excavation to subgrade indicate that the anchored soil mass translated as a unit. As shown on Figure 5, the retained soil mass moved about 50 mm, at a constant rate for 30 days after excavation to subgrade. The system then stabilized.

Tiebacks at tiers 3, 4 and 5 were monitored with load cells. These data indicated that there was a 5 to 15 percent loss of load within 2 to 3 days after each anchor was locked off, after which the load remained constant. The load loss typically occurred prior to the postgrouting of lower anchors. There was no evidence of tieback load loss during or subsequent to postgrouting.

It is difficult to isolate the amount of movement attributable to postgrouting. However, the data plotted in Figure 5 suggests that it may have been as much as 65 mm.

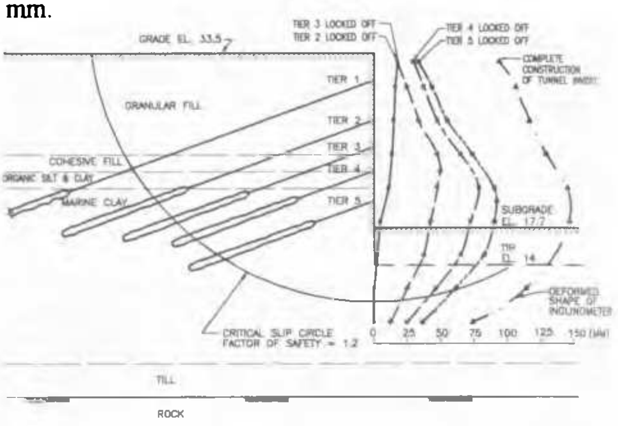

FIGURE 4 – SUMMARY OF WALL MOVEMENTS AT EACH CONSTRUCTION STAGE

CONCLUSION

The design and construction of the South Approach to the Third Harbor Tunnel demonstrates that tieback sheetpile systems can provide cost and scheduling advantages in marginal soil conditions if carefully analyzed, designed, installed and monitored. Relatively high tieback capacities are achievable in soft clay if care is taken during installation; but, poor drilling and grouting procedures will have a significant effect on capacity. Cooperation between the Designers, Contractor and Owner is essential, and all parties must be prepared to modify Procedures when the monitoring indicates behavior contrary to design assumptions.

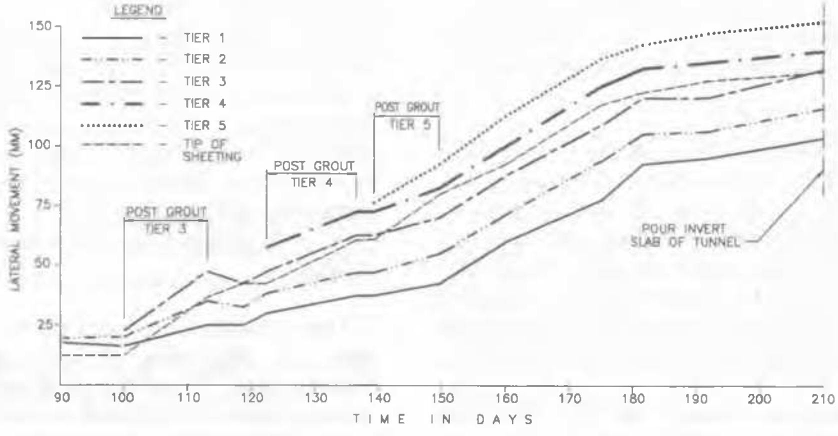

FIGURE 5 – RATE OF SHEETING MOVEMENT