

## **Innovations and Advances in Tied-Back Soldier Pile Shoring in Seattle**

David G. Winter<sup>1</sup>, Garry E. Horvitz<sup>2</sup>, and Tom A. Armour<sup>3</sup>

<sup>1</sup>Senior Principal, Hart Crowser, Inc., Seattle, Washington

<sup>2</sup>Senior Principal, Hart Crowser, Inc., Seattle, Washington

<sup>3</sup>President, DBM Contractors, Federal Way, Washington

### **ABSTRACT**

Since the 1970s soldier pile and tieback anchor shoring in the Seattle downtown core has supported more than 40 deep excavations and has become substantially more economical and predictable in its performance. Design and construction efficiencies have been made possible using a growing database of performance results. Recent development sites have often been partial blocks surrounded by buildings with basements, vital utility corridors, and narrow streets. These conditions have challenged development teams and led to new solutions that have made tough site development practical. This paper analyzes published and unpublished data to demonstrate how soldier pile and tieback shoring in Seattle has changed, how a greater emphasis on observed behavior and calibrated predictive tools has led to more efficient designs, and how much more cost-effectively these designs can be installed in today's market.

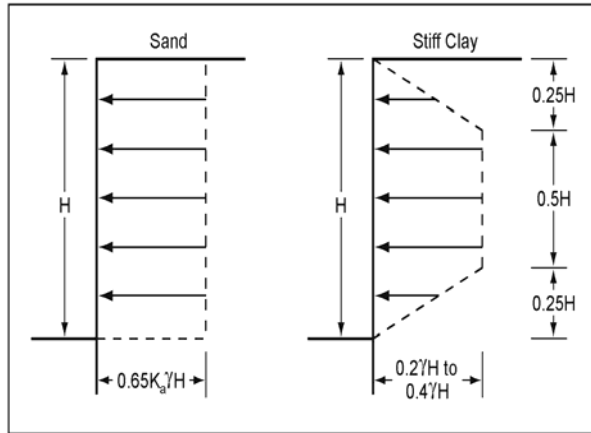
### **SHORING HISTORY IN SEATTLE**

In 1972 the Bank of California Center helped open an era of high-rise/deep excavation development in Seattle that continues today. Clough, et al (1972) documented the design and construction results, and provided some of the first published information on tied-back shoring in the region. The paper described the characteristics of the overconsolidated sand, silt, and clay that dominates the downtown Seattle core, and reminded the reader that not only was there meager soldier pile and tieback shoring wall deflection data available for comparison, but that the accepted design methods published by Terzaghi & Peck (1967) were developed for normally consolidated soils. The wall was nonetheless designed using the Terzaghi & Peck approach, with a maximum lateral pressure of  $0.4 * 120 \text{pcf} * H = 48H$  psf and a trapezoidal distribution. Based on laboratory test results and a finite element analysis the predicted maximum wall deflections were about 2 inches. The observed pressures were very close to the design values but the wall deflected only about  $\frac{3}{4}$  inch. The authors concluded that the wall pressures reflected the lock off load of the anchors, instead of the actual lateral pressures on the wall. After nearly 40 years of additional data we can conclude that the authors were correct about the wall pressures, and offer that the low observed deflections meant the wall was oversized.

Other Seattle engineers faced similar challenges. The authors reviewed the design criteria for five downtown excavations deeper than 50 feet and seven

excavations 30 to 50 feet deep (between 1963 and 1979) as reported by Gurtowski and Boirum, (1989). Of the five deep shoring walls, only one appears to have strayed from the Terzaghi & Peck approach, since all had design pressures typically  $40H$  psf or higher. The one excavation with a lower design pressure was in more of a sand profile,

with a uniform distribution of pressure calculated as  $0.65 \cdot K_a \cdot H = 21H$  psf.



**Figure 1. Terzaghi & Peck Design Envelope**

One designer had a particularly challenging problem. The geotechnical engineers prepared a design for the 50-foot deep 1111 Third Avenue excavation that encountered both layers of sand and layers of clay in the profile. The design called for a pressure of  $22H$  psf, based on a Terzaghi & Peck approach for sand, and a belief that the pressures used in the past for clay were conservative.

But city reviewers believed that the clay layers ought to have higher design values than the sand, and thus a pressure envelope that jumped between  $22H$  and  $36H$  resulted. Not only was such an envelope difficult to translate into an easily constructed tieback system (because of the changing loads on each tieback) but it was also based on an assumption of soil layering consistency that was impossible to verify until the 120 or so soldier piles were installed. Even though over the next 10 years there were a few projects with similar variable pressure envelopes designed and constructed most designers understood that variable design from layer to layer suggested a precision in both the methodology and the understanding of the subsurface profile that didn't exist. The more reasonable approach, and the one that increasingly gained favor among the primary downtown geotechnical engineers, was to:

- Estimate a single combined representative or average soil profile
- Design the pressure envelope for the soil profile, based on the results of past successful projects, and check against the established empirical methods, and
- Include a monitoring program for wall deflections and tieback pressures.

With this approach various elements of tied-back shoring design advanced.

## **ADVANCES IN PRESSURE ENVELOPE DESIGN**

Just as important as a realization that lateral soil pressure envelopes for overconsolidated soil profiles couldn't be directly developed using methods based on normally consolidated soil data, was the growing recognition that pressures for very deep excavations were not directly comparable to those for shallow excavations. In other words, a pressure envelop of  $36H$  for a 40 foot deep cut might be reasonable and result in an economical design in a mixed clay and sand profile, but  $36H$  for an 80 foot deep cut resulted in huge pressures; and very large soldier piles, long tiebacks, and thick

lagging boards, thus making construction unnecessarily expensive. On several very deep excavations in the 1980s (Seafirst 5<sup>th</sup> Avenue Plaza – 75 feet, Madison Hotel – 70 feet, First Interstate Bank – 80 feet, Columbia Center – 120 feet, 1201 Third Avenue – 85 feet, and Pacific First Center – 80 feet), consideration of maximum pressures regardless of depth began. At the First Interstate Bank excavation of a predominantly clay profile resulted in a pressure envelope of 30H psf. The trapezoidal shape of the envelope allowed elimination of the bottom row of ties during design. The City of Seattle reviewer objected to the bottom truncation of the trapezoid and required another row of ties to be designed within just a few feet of the bottom of the excavation. The designers used load cells, inclinometers, and optical survey monitoring to document the small movements of the wall during the excavation. By the time the excavation reached the bottom row they had a convincing case for the City that the wall was performing better than expected and the last row of tiebacks was eliminated without issue.

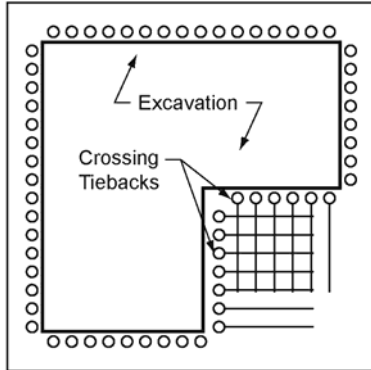
Five years later designers working on the 1201 Third Avenue project just three blocks from First Interstate built on the observations of that earlier instrumented excavation. They advanced the pressure envelope design by instrumenting tiebacks with hydraulic load cells and trying to isolate a section of the instrumented wall to get results more representative of the true pressures. The instrumented section was comprised of the soldier pile with the load cells on each tieback plus the two soldier piles on each side of the instrumented pile. All of the tiebacks on the five soldier piles in this 35 foot wide section were locked off at 50% of the design load, instead of the usual 100%, after proof testing. By isolating a section of the wall at 50% of the design load there was a better chance of recording actual pressures instead of just lock-off values. The field observations verified the validity of the approach and the designers concluded that pressures even lower than the 20H design were appropriate for this soil profile and depth. They concluded that a pressure envelope as low as 17H psf would have been a reasonable design for this deep excavation (Winter, et al, 1987).

Note that 17H for an 85 foot deep excavation gives a maximum pressure of 1,445 psf, about the same as 36H on a 40 foot deep cut. The First Interstate and 1201 Third Avenue results supported a belief that overconsolidated soils reached a maximum pressure at a relatively shallow depth, and if the shoring system was properly designed and constructed, the same pressures could be applied to much deeper excavations resulting in a significant construction cost savings. Subsequent excavations and different Seattle designers verified this conclusion and shoring walls for just about any overconsolidated soil type and depth greater than 50 feet were designed for a maximum pressure envelope of less than 30H psf and typically 20H to 25H, representing about a 60% reduction in pressures from the conventional designs of the 1970s.

Other elements of the wall pressure and soldier pile/tieback design were similarly considered and adjusted for observed conditions. They included:

- Wall pressures in corners were reduced in recognition of three-dimensional resisting effects of the adjacent perpendicular wall. Often pressures within 30 to 50 feet of corners were reduced by as much as 50% (4<sup>th</sup> and Madison, Chase Center).

- The upper section of the shoring wall that includes the cantilever portion and the first tieback often deflected away from the excavation, suggesting the prestressing of the upper tieback was jacking load into the soil rather than resisting pressures from the soil. These observations allowed the location of the first tieback to be deepened thus reducing interference with adjacent shallow utilities (Olive 8).

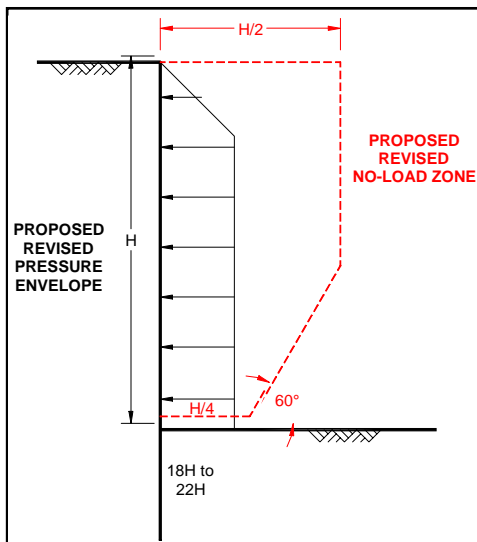


**Figure 2. Re-entrant Corner.**

- Design of crossing tiebacks supporting re-entrant corners was subject to debate. Four significant excavations (Columbia Center, Westlake Center, Pacific First Center, and 4<sup>th</sup> and Madison) included re-entrant corners. Observations at these sites supported three conclusions: 1) each of the “re-entrant walls” can be designed as if the other wasn’t present; 2) lower pressures exist near the corner so prestressing required caution; 3) pressure grouted ties required careful pressure monitoring to avoid overstressing the perpendicular wall.

### ADVANCES IN NO LOAD ZONE CONFIGURATION

The designers of the Columbia Center excavation faced a significant challenge: The conventional no load zone design for this 120 feet deep excavation extended as far as 90 feet from the excavation face at the ground surface. However, 5<sup>th</sup> Avenue, on the high side of the excavation was only about 70 feet wide. The upper rows of tiebacks would thus extend as much as 20 feet beyond the property line, requiring easements and often intersecting basements. To address this problem the designers proposed a



**Figure 3. Revisions proposed by Winter, Loesch, and Hollister (2001)**

truncated no load zone, set back from the base of the excavation a distance of about 40 feet. The no load zone represents the area of potentially unstable soil just behind the wall behind which all tiebacks must attain their support. The practical effect of a truncated no load zone is shorter tiebacks for the upper rows. The uncertainty is how much additional wall deflection could result. At Columbia Center the wall deflected only about 1/2 inch. The designers concluded that the shorter ties in the truncated no load zone “provided satisfactory stability for the shoring wall without excessive movement” (Grant, 1985).

Similar no load zone truncations with similar results occurred at 4<sup>th</sup> and Madison (95 feet deep) and Olive 8 (85 feet deep). The maximum deflections at 4<sup>th</sup> and Madison

were about 2 inches and minor repair work was required for cracks opened at the property line across the street (Winter, et al, 2001). At Olive 8 deflections were less than 1 inch. At none of the sites was stability of the wall threatened by the truncation.

## **ADVANCES IN ESTIMATING AND MEASURING WALL DEFLECTIONS**

The success of a shored excavation is not solely expressed in the ability of the system to safely retain the earth loads imposed on the system. Success, in the context of a design acceptable to the reviewing agencies, is the avoidance of adverse impacts to adjacent utilities, rights-of-way and buildings. Thus the attention given to deformation of the shoring system and potential settlement behind the wall has justifiably increased.

In early shoring system designs for excavations shallower than 60 to 70 feet, empirical approaches to estimating deflections were accurate enough. But non-standard site geometry and deeper excavations challenge traditional limit equilibrium models. Recently the state of the practice has evolved to include the use of soil/structure interaction models to predict not only the deformations of the shoring system but the deformations of the adjacent soil mass within the right-of-way and the potential vertical and lateral movement of nearby building foundations. The use of finite element models such as PLAXIS or finite difference methods such as FLAC have enhanced the industry's ability to more reliably predict these deformations. With repeated use we can "calibrate" the models' predictive ability by back-calculating input parameters for results that match past observations, and use those adjusted parameters at future sites.

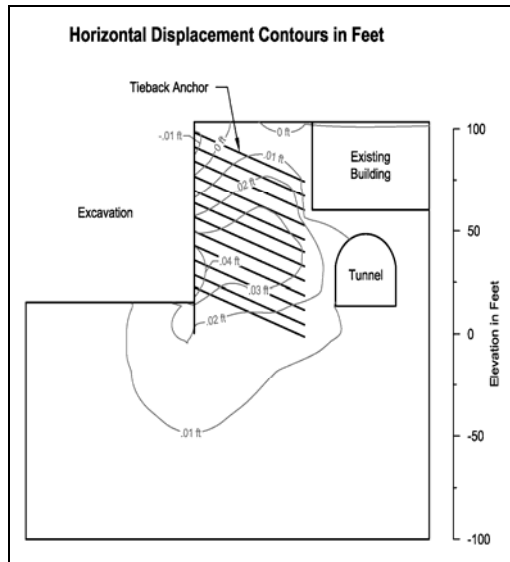
An example of this approach is the Chase Center. The excavation for this building extended to deeper than 100 feet at the corner of 2nd Avenue and Union Street. This depth required the use of a truncated no load zone which increased the likelihood of greater deflections. In addition the mainline tracks of the BNSF Railroad travel through a century-old tunnel located within 25 feet of the edge of the excavation which extends down to the invert of the tunnel. The tunnel is highly settlement sensitive and a design focus was to avoid adverse impacts to the tunnel and structures across the street. FLAC was used to model the interaction of the shoring elements, adjacent soil mass, tunnel and existing building foundations. The model predicted 0.9 inches of deflection at the tunnel and 1.2 inches at the shoring face. Optical survey monitoring and inclinometer casing attached to the soldier piles showed magnitudes of vertical and horizontal displacement matching those predicted.

At Olive 8 (Winter, et al, 2009) the site was surrounded by narrow streets and existing buildings. One of those buildings had a deep basement (about 50 feet) that could not be accessed, and was separated from the excavation by a narrow alley with utilities that could not be moved or abandoned. The Olive 8 excavation was to extend about 85 feet deep. On the wall facing the 16-foot wide alley and adjacent deep basement the designers developed a hybrid shoring solution consisting of:

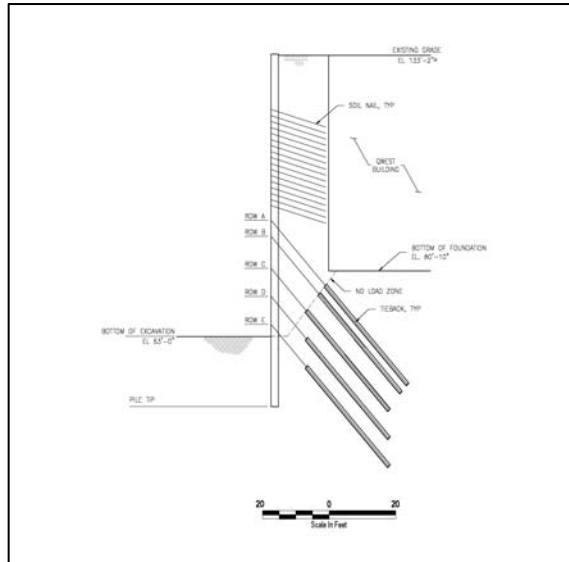
- Soldier piles on a normal spacing,
- Short soil nails on a tight grid pattern between the soldier piles and extending just back to the adjacent basement wall, and

- Tieback anchors extending below the adjacent basement and designed for the surcharge loads from the adjacent footings.

The PLAXIS model and the design and performance of the shoring wall showed that the hybrid system actually performed better than predicted by the model, with deflections well under 1 inch for most of the instrumented areas. These two results demonstrate that creative solutions to difficult site problems are possible, and made easier and more dependable by the competence of the overconsolidated Seattle soils, and the confidence recently gained in predictive modeling of deflections.



**Figure 4. Chase Plaza FLAC Model**



**Figure 5. Olive 8 Hybrid Shoring**

## ADVANCES IN CONSTRUCTION METHODS AND APPROACH

In addition to the consistent and incremental advances in wall design, contractors, equipment manufacturers and material suppliers have improved construction elements to cut costs and meet schedule demands. Addressing site, soil, and neighborhood challenges require foundation contractors working closer than ever with their project partners during design development as well as construction. Since the Bank of California Center Project in 1972 innovations in procurement, equipment, tooling, and field process have helped owners and engineers “pencil out” difficult to build on properties that at one time seemed too expensive to develop. In the early 1980’s foundation contractors introduced smaller European hydraulic drill rigs. More efficient anchor designs were now possible with this new ability to install longer anchors, pressure grout bond zones, and anchors at varying angles - all in less required space and at a faster rate. Continuing this European trend, specialty soldier pile drill rigs were introduced in the late 1980’s. These more compact and powerful drills, designed for the tight urban confines of European cities allowed more efficient installation of deeper soldier piles enabling the project owners to add more usable space to their buildings. Tooling innovations have played an integral role as well. Drill augers, core barrels and cleanout buckets are constantly being improved to process a hole faster and

safer. Auger flight and drill tooth designs are specific to ground conditions. Much improved overburden drill tool systems are more readily available allowing higher quality lower cost anchor installations in difficult ground. Dual rotary systems, oscillators, and casing rotators perfected in the late 1990's allow independent and synchronized rotation and advancement of a casing and a drill tool.

Over the last thirty years process improvements have also played a critical role in the



**Figure 6. 4<sup>th</sup> and Madison**

Seattle excavation support market. Engineers, contractors and manufacturers have become more knowledgeable of concrete mix designs, mineral and polymer slurry and shotcrete placement. Advances have improved the constructability of excavation soldier pile/tieback systems allowing more economical and safer designs to be installed. As an example, for a typical moderate depth soldier pile and tieback excavation and comparing costs from the 1970's to today (assuming the same labor and material unit prices), by incorporating today's advancements (both design and construction) owner unit cost savings are in the range of \$8/s.f. to \$12/s.f. of wall face. This represents a 30% to 45% savings in shoring costs.

Alternative design delivery and contract procurement methods are becoming more common. Owners often request design/build proposals based on stated wall performance criteria or allow foundation contractors to provide design/build options (as cost reduction incentives) to owner provided designs. Two recent project examples include the 505 1<sup>st</sup> Avenue and 635 Elliott Avenue projects. At 505 1<sup>st</sup> Avenue the owner provided designs for an anchored secant pile wall to provide the necessary temporary groundwater cutoff barrier and a constructible wall system through buried wood debris. The successful shoring contractor proposed an alternate design which relied on pre-trenching through the wood debris before installing a deep anchored cutoff wall. On the 635 Elliott Avenue Project the owner designed a conventional permanent anchored secant pile wall system due to the difficult soil conditions with a high water table. The selected contractor designed modifications that reduced costs and tightened a tough schedule. Congested sites and challenging ground conditions require early and active contractor participation in order to maximize efficiency.

## **CONCLUSIONS AND FUTURE IMPROVEMENTS**

Summarizing the conclusions reached previously and supported herein:

- Lateral pressures on shoring walls in overconsolidated soils can be more accurately predicted from previous observations than from established design methods.
- Lateral pressures on excavations deeper than about 40 feet appear to reach a limiting value, and do not necessarily increase with depth.

- No load zones can be safely truncated.
- Numerical modeling should be routinely used to predict wall deflections and area settlements.
- Instrumentation to measure wall performance is key to continuing improvement in wall design.
- Early participation from contractors can optimize designs.

The authors believe future studies should emphasize:

- Lighter soldier piles, shorter anchors, and thinner lagging – all are indirectly tied to lower wall pressures.
- Additional modeling of hybrid systems – combinations of soldier piles, tiebacks, soil nails, and corner bracing to produce the most efficient solution to variable site conditions.
- Adaptation of conventional construction equipment for specific site conditions to allow more efficient installation of shoring elements.

## REFERENCES

- Clough, G., Weber, P., and Lamont, J. (1972). “Design and Observation of a Tied-Back Wall.” *Proc., Performance of Earth and Earth Supported Structures*, ASCE, Purdue University.
- Grant, W.P. (1985). “Performance of Columbia Center Shoring Wall.” *Proc., 11<sup>th</sup> International Conf. on Soil Mechanics and Foundation Engr.* San Francisco.
- Gurtowski, T.M. and Boirum, R.N. (1989). “Foundations and Excavations for High-Rise Structures in Downtown Seattle.” *Engineering Geology in Washington, Vol. II*, Washington Division of Geology and Earth Resources Bulletin 78.
- Terzaghi, K. & Peck, R. (1967). *Soil Mechanics in Engineering Practice*. John Wiley & Sons, New York.
- Winter, D., Loesch, D. and Hollister, R. (2002). “Shoring Wall and Subsurface Structural Systems, IDX Tower, Seattle, Washington.” *Proc., Challenges and Future Directions for Tall Buildings*. Los Angeles Tall Buildings Structural Design Council.
- Winter, D., Macnab, A. and Turner, J. (1987). “Analysis of 85-Foot-Deep Shored Excavation in Seattle.” *12th Annual Meeting and Seminar*. Deep Foundations Institute. Hamilton, Ontario.
- Winter, D., Smith, M., Chin, K. and Carnevale, R. “Creative Shoring Solutions for a Tight Urban Site.” *34<sup>th</sup> Annual Meeting and Seminar*. Deep Foundations Institute. Kansas City, Missouri.