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ABSTRACT

The Seattle Central Library project, which replaced the existing downtown library, consists of a twelve-story building with several below-grade levels. The excavation encompassed an entire city block and had plan dimensions of approximately 250 feet by 240 feet. The original excavation depth was up to 53 feet in height. The excavation was made in highly overconsolidated Seattle silts and clays (Lawton Clay). The Lawton Clay has been documented to exhibit expansive behavior along planes of weakness associated with stress relief upon excavation.

The original excavation was designed to be supported using a tieback soldier pile wall, typical of shoring systems retaining the Lawton Clay. A soil nail shoring wall design was submitted, and subsequently installed, as part of a design-build alternative. The soil nail shoring wall system consisted of temporary, top-down soil nail walls that utilized portions of the concrete basement walls of the existing library. Vertical elements and shotcrete facing were constructed in areas where the excavation extended beyond or below the existing basement walls. Soil nails were installed using self-boring grout-injected anchors consisting of hollow bars with sacrificial drill bits.

Displacement of the soil nail shoring walls was predicted to be less than 1 inch by the designer. The actual wall movements for three sides of the excavation were as predicted. However, the east wall on the uphill side of the excavation experienced 4 inches of lateral movement and over 2.5 inches of vertical movement, causing damage to the adjacent street and necessitating extensive design modifications during construction. In addition, the excavation depth was reduced to 47 feet because of significant movement occurring below the excavation. This paper describes the construction and observed behavior of the east wall and the applicability of soil nail walls in the Lawton Clay deposit.

INTRODUCTION

The Seattle Central Library site is located in the heart of downtown Seattle. As with most downtown sites, the ground surface slopes down to the west towards Puget Sound. The current project replaced the existing library and consists of a twelve-story building with several below grade levels. The excavation for the project encompassed an entire city block and had plan dimensions of approximately 250 feet by 240 feet. The original depth of the excavation varied from 15 feet along the west side of the site to up to 53 feet along the east side of the site.

The shoring system for the project consisted of temporary soil nails using either the existing basement walls or shotcrete as the facing element. In most areas where the existing basement wall was used as facing, the excavation did not extend below the existing wall footing. The exception was along the east side of the site where the excavation was planned to undermine the entire existing wall and extend up to 25 feet below the wall footing.

The excavation was completed in fine-grained glacially consolidated Seattle silts and clays (Lawton Clay). Conventional soil nail walls with shotcrete facing are used extensively in glacially consolidated granular soils in the Seattle and surrounding areas. However, few have been used in glacially consolidated silts and clays because of the possibility of shear zones (planes of weakness) and locked in lateral stresses. This paper presents the observed performance of a soil nail wall in Lawton Clay and the lessons learned during construction. This paper focuses primarily on the east side of the excavation where significant wall movements occurred. Figure 1 shows a photograph of a portion of the east wall when the excavation was about 40 feet in depth and seven of the planned nine rows of soil nails and shotcrete were in place.

Fig. 1. Photograph of east wall during construction.

INVESTIGATIONS AND SHORING DEVELOPMENT

Four borings were completed by a Seattle geotechnical engineering firm as part of the geotechnical study for the project. Soil units encountered consisted of fill and native soils. The fill is associated with development of the existing library and was locally present adjacent to the existing basement walls. The fill generally consisted of very loose to medium dense sand. The native Lawton Clay soils encountered at the site consist of sandy silt, clayey silt, silty clay and clay. The consistency of these soils ranged from very stiff to hard. Slickensides were noted in the Lawton Clay in one boring slightly above the base of the proposed excavation. Perched groundwater was also encountered near the bottom of the planned excavation. The static groundwater table was not encountered and was interpreted to be below the base of the excavation.

Lawton Clay is a lacustrine silt and clay with thin sand interbeds which has been overconsolidated by glacial ice. The soil varies from massive to laminated and varved. It generally contains numerous fractures, joints and slickensides which are generally thought to be the result of stress relief upon removal of the glacial ice (Galster et. al., 1991).

Numerous stability problems have occurred on natural slopes and in excavations in the Lawton Clay. The Lawton Clay has been documented to exhibit expansive behavior along planes of weakness associated with stress relief upon excavation. The excavation releases the high locked in lateral stresses in the soil, which results in elastic expansion that open the natural joint system. The most notable project that this type of expansive movement on weak planes occurred was construction of Interstate 5 east of the site (Peck, 1963; Palladino, 1971). Additionally, the presence of shear zones in the Lawton Clay has been a design consideration for numerous excavations in the city (Gurtowski et. al., 1989), particularly adjacent to the south and west sides of the Seattle Library site for the Bank of California project (Clough et. al., 1972) and the Seattle First National Bank project (Shannon et. al. 1970).

Two shoring options were provided by the geotechnical engineer, including conventional tieback soldier pile and lagging walls and soil nail walls. The geotechnical report warned that there was a potential for fractured soils in the Lawton Clay and this could have a significant impact on the construction and performance of a soil nail wall if selected. The shoring system selected and designed by the project team consisted of a tieback soldier pile and lagging wall.

Prior to construction, a design-build soil nail wall alternative was proposed by the shoring contractor. The wall design was completed by a second Seattle geotechnical engineering firm and submitted for permit. The shoring system consisted of temporary top-down soil nailed walls that utilized portions of the basement walls of the existing building for the soil nail facing. Shotcrete facing was constructed in areas where the excavation extended below or laterally beyond the limits of the existing basement wall. The design included strut nails at the bottom of the existing basement wall to support and underpin the wall as the excavation continued below the wall. Shotcrete facing, along with vertical elements for face stability, were constructed to support the fill soils adjacent to the basement walls. The proposed soil nail installation method consisted of using grout-injected anchors (hollow Titan bars installed with grout slurry) or post-grouted anchors. Grout-injected anchors were designed to be installed in the fill soils located behind the existing building walls. Post-grouted anchors were designed for all native soil (Lawton Clay) areas.

Because of time constraints and the nature of the soil conditions and the soil nail wall design, the City of Seattle contracted the primary author’s firm to peer review the geotechnical aspects of the shoring design. During the peer review, the primary author’s firm raised numerous issues related to the design and expected performance of the soil nail wall shoring system, including:

- The potential for slickensides and locked-in lateral stresses in the Lawton Clay material. Subsequently, the shoring designer completed additional explorations (borings and test pits) and they concluded that the slickensides at the site were minor and sporadic, in their opinion.

- The designers originally estimated wall displacements of less than 1 inch, which corresponded to about 0.15%H, where H is the height of wall. The estimate was subsequently revised to correspond to between 0.1%H to 0.3%H, with the lower bound being an "Action Level" when remedial measures would be considered. (The "Action Level" for remedial measures as noted on the plans was movement in excess of 1 inch).
- The designers used a design nail pullout capacity of 5 kips/foot (as much as 2.5 times higher than values typically used in the Seattle area for the anticipated soil conditions). The reason provided by the designer for the higher capacity was the grout-injection method or post-grouting method of nail installation would result in higher capacities. The nail pullout capacity was subsequently reduced to 3.5 kips/foot in the design. Further reduction, to as low as 1.4 kips/foot, occurred during construction due to the low pullout resistance during nail testing.
- The original design consisted of completing 2 verification tests for each soil type prior to construction and completing 1 proof test for every 20 production nails installed during construction. Creep testing of the anchors, which was proposed to be up to 1 hour in total hold time, was also included in the verification and proof testing. Because the designer's pullout capacity of the nails was still much higher than typically used, the testing criteria was subsequently modified to include 4 verification tests for each soil type and 1 proof test for every 10 production nails. Additionally, the creep test hold time for the validation nails was increased to 24 hours. The additional testing proved critical during construction as a high number of pullout failures occurred.

The permitted shoring design for the east wall consisted of five rows of soil nails through the existing basement wall and four rows of soil nails and shotcrete below the existing wall. At the north and south ends of the east wall, there were small sections that consisted of nine rows of soil nails and shotcrete. The typical soil nail spacing was 5 feet on-center horizontally. The maximum depth of excavation was up to 53 feet. An elevation and cross section showing the permitted soil nail wall design for the east wall are shown in Figs. 2 and 3. The darker area shown in the middle of Fig. 2 represents the existing basement wall.

EAST WALL SOIL NAIL WALL CONSTRUCTION

General

Construction of the soil nail wall shoring system was completed in phases. Phase I of construction consisted installing the soil nails through the existing building walls of the library. The soil nails installed during Phase I consisted of grout-injected anchors because of the presence of existing fill materials behind the building walls. Validation and proof testing of the grout-injected soil nails was completed and the testing indicated adequate pullout capacities. Strut nails were also installed at the base of the existing building walls (see Fig. 3). The strut nails were designed to support the weight of the existing walls in compression when future excavation occurred below the wall footing.

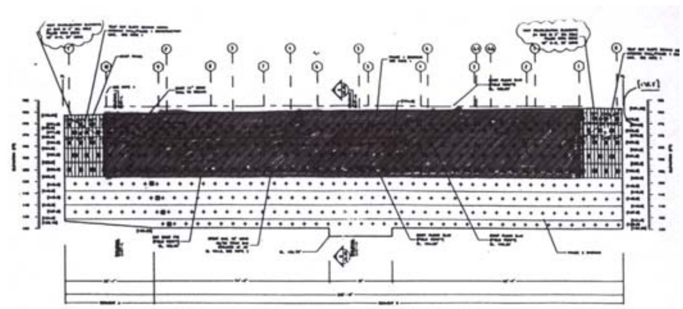

Fig. 2. Elevation view of permitted design for east wall

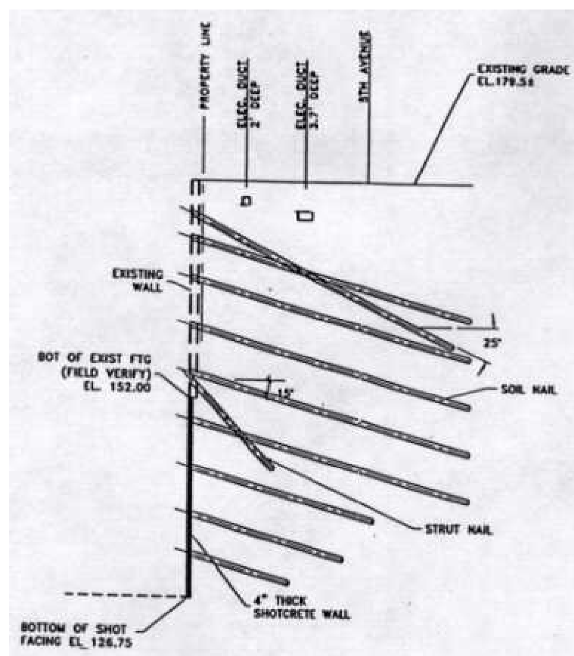

Fig. 3. Cross section of permitted design for east wall

Phase II of construction consisted of demolition of the existing library and installation of vertical elements. The floor slabs of the existing library were removed during demolition, which loaded the Phase I soil nails installed through the existing basement walls. Lateral wall deflections were measured at approximately 0.3 inches upon initial loading of the nails. Portions of the existing walls along the north and south sides of the excavation were also removed. Vertical elements, which consisted of vertical soil nails, were installed to provide face stability in those areas where loose, caving soils were anticipated (existing fill and disturbed native soils).

Phase III of construction consisted of installing soil nails and shotcrete facing for the portion of the excavation that extended beyond the limits of the previous building. The original design was to use post-grouted soil nails (open hole techniques) during this phase of construction. However, the shoring contractor decided to proceed with grout-injected anchors for all soil nails at the site to avoid additional validation and creep tests for the open-hole installation technique.

The second Seattle geotechnical engineering firm, who was part of design-build team, was contracted by the Seattle Library to complete construction observation and inspection services for the shoring walls on behalf of the city. The geotechnical engineering firm performed these services for Phases I and II of the project. During the initial portion of Phase III of construction, the city decided that there was a potential conflict of interest with having this geotechnical engineer perform the observation and testing services because they were under contract with both the city and the design-build team. The primary author's firm was subsequently contracted by the city to perform these services for the remaining portion of Phase III construction. When the primary author's firm began observing construction, installation of row 4 soil nails and shotcrete adjacent to the existing basement walls was underway.

Observations - Overview

Numerous issues occurred during Phase III construction of the east shoring wall, particularly when the existing wall was undermined to extend the excavation deeper. The existing wall dropped vertically over 1 inch when the footing was initially undermined. Additional vertical support was provided by installing micropiles. By the end of construction, the east wall had dropped vertically over 2.6 inches, with over 0.5 inches occurring after the micropiles were installed. Additionally, significant lateral movement of the wall began to occur as excavation below the wall footing occurred. The total horizontal movement of the east wall that occurred during construction was 4 inches. A significant amount of this movement occurred along planes below the bottom of the excavation, generally within 4 to 12 feet of the current excavation bottom. This type of movement is consistent with stress relief within the Lawton Clay, resulting in the opening of fractures and movement along planes of weakness (Peck, 1963; Palladino, 1971). Water, which tended to collect in the fill behind the existing wall, may have exacerbated the lateral wall movements on the weak planes.

Several other modes of wall movement in addition to that attributed to stress relief were also observed. These included typical wall deflections required to mobilize forces in the soil nails, response to grouting pressures and/or construction activities, and creep movement after the excavation had bottomed out.

The movement of the east wall resulted in significant cracking of the pavement along 5th Avenue behind the wall. Large longitudinal cracks developed which required continual maintenance. One lane of 5th Avenue, which is a major arterial through downtown Seattle, was closed for several months because of the perceived risk to health and human safety. Additionally, the main fiber optic duct bank that supplies the Seattle financial district was located within 5 feet of the east wall. The duct bank was evaluated several times by the contractor during construction to ensure that the movements were not damaging the duct bank. Because of the large movements and damage to the adjacent infrastructure, the east soil nail wall design was revised numerous times during construction in an attempt to control the movements occurring.

A detailed summary of the events that occurred at the east wall while the primary author's firm was performing construction observation and inspection services is presented below. Table 1 presents a summary of the optical survey data collected at the top of the east wall during construction. Table 2 presents a summary of the inclinometer data at various times during construction, with particular emphasis on movement along planes of weakness below the excavation. The inclinometer was located along 5th Avenue, approximately 8 feet behind the east wall.

Table 1. East Wall Optical Survey Data

Completed Construction Activity	Excavation Depth (feet)	Horizontal Movement (inches)	Vertical Excavation (inches)
Row 5	28	0.24	0.00
Row 6 Nail Installation	34	0.84	1.20
Row 6 Shotcrete	34	1.20	1.56
Row 6 Replacement Nails[1,2]	34	1.56	1.92
Row 7 Nails & Shotcrete	40	2.10	2.22
Row 8 & 9 Nail Installation	40	2.64	2.28
Row 8 Replacement Nails[1]	40	3.06	2.28
Row 8 Shotcrete	43	3.18	2.34
Mass Excavation at Wall	46	3.42	2.46
Footing Excavation	49	3.54	2.52
Creep After Construction	46	3.90	2.64

[1]High grout communication
[2]Installation of micropiles complete

Table 2. East Wall Inclinometer Data

Completed Construction Activity	Excavation Depth (feet)	Ground Surface Movement (inches)	Cumm. Movement Below Excavation (inches)
Row 5	28	0.33	0.00
Row 6 Nail Installation	34	1.03	0.55
Row 6 Shotcrete	34	1.38	0.82
Row 6 Replacement Nails[1]	34	1.71	1.09
Row 7 Nails & Shotcrete	40	2.39	1.56
Row 8 & 9 Nail Installation	40	2.82	2.13
Row 8 Replacement Nails[1]	40	3.19	2.35
Row 8 Shotcrete	43	3.43	2.41
Mass Excavation at Wall	46	3.74	2.49
Footing Excavation	49	3.80	2.49
Creep After Construction	46	3.99	2.52

[1]High grout communication

Row 5 Observations. The row 5 soil nails and shotcrete at the north and south ends of the existing east wall were installed. The strips of drainage material installed behind the shotcrete were not connected with the existing footing drain behind the existing wall. Placement of shotcrete essentially cutoff the drainage path for the footing drain and water began building up behind the existing basement wall. The footing drain was also likely compromised by the installation of the Phase I strut nails and row 5 nails as they generally passed through the wall drainage zone. Several 1-inch diameter holes were subsequently drilled at the base of the wall to help relieve the hydrostatic water pressure building up behind the wall.

Based on optical survey and inclinometer data, the existing basement wall had moved approximately 0.3 inches horizontally at the completion of soil nail row 5. No vertical wall movement had occurred.

Row 6 Observations. Soil nail row 6 was the first lift of soil nails with temporary shotcrete facing across the entire east wall. The construction of row 6 required undercutting the concrete footing of the portion of the existing wall that was incorporated into the shoring system. Construction of row 6 was completed over a period of 2 weeks.

The construction sequence consisted of mass excavation in front of the wall to allow soil nail installation. The nails were installed through a 1H:1V (horizontal to vertical) native soil drill berm which temporarily supported the existing wall footing. Once the soil nails were installed, excavation to vertical wall face was completed. The northern half of row 6 was excavated vertically 6 feet for shotcrete placement while the southern half was left with the 1H:1V drill berm below the wall. During excavation, two vertical cracks developed in the existing concrete wall. The cracks were approximately ¼-inch at the bottom of the wall and became hairline about mid-height. Water was observed flowing through the cracks and onto the recently cut soil face. Separations between the curb and gutter along 5th Avenue were also observed.

Water seepage on the soil face from the cracks made shotcrete placement difficult. There were several locations where the fresh shotcrete peeled away from the face and required repair. To reduce seepage at the face and buildup of hydrostatic pressure behind the wall, the 1-inch-diameter weep holes were supplemented with a series of 4-inch-diameter holes that were cored through the existing basement wall at a 12-foot spacing. Additionally, several horizontal drains, consisting of 20-foot-long drilled holes with a 2-inch slotted well casing, were installed along the face of the wall at a 20-foot spacing. The core holes and drains were located about 2 feet above the footing bottom to facilitate drainage of water above this level.

Significant horizontal and vertical wall movements were measured on both the optical survey points and the inclinometer when the north half of the existing wall was undermined (row 6 nails). The vertical and horizontal movements of the east wall were about 1.2 and 0.6 inches, respectively, as shown in Table 1. The inclinometer data, as shown in Fig. 4 and Table 2, showed that almost all of the incremental horizontal wall movement occurred 6 feet below the bottom of the excavation.

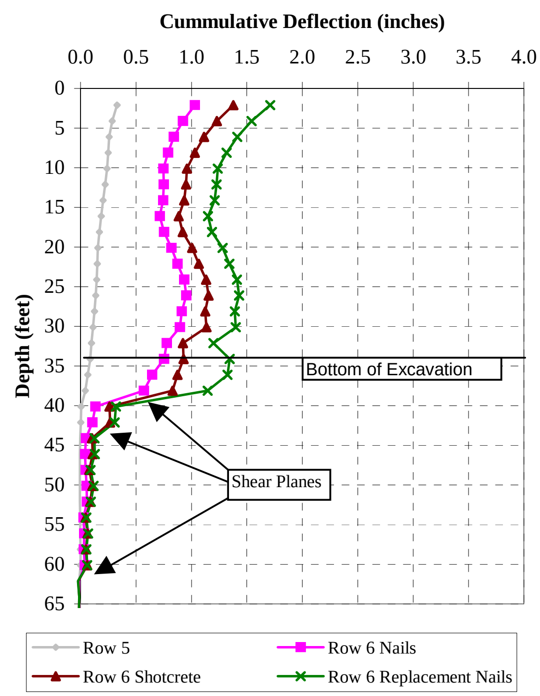

Fig. 4. East wall inclinometer data after completion of row 6.

As a result of the rapid movement of the wall, the southern half of the wall was shotcreted using a staggered excavation approach. Each excavation, or slot, was approximately 20 feet in length. The slots were spaced such that the adjacent slots were either protected by a soil drill berm or completed shotcrete wall.

During the shotcreting of the southern half of the wall, a visual survey of the wall and 5th Avenue was completed. The survey revealed that the several of the row 1 through 5 soil nail nuts were loose. In addition, the accessible strut nail locations were observed to determine if the strut nails had been damaged as a result of the vertical movement. A detail of the strut nail connection is shown in Fig. 5 (strut nail is shown inclined at 45 degrees from horizontal). Half of the strut nails evaluated showed either relative slipping/movement between the grout and wall or very weak grout connection (soil/grout mixture) that could easily be removed with hand tools. The relative movement was small and did not account for the 1.2 inches of vertical wall movement measured, but did call into question the integrity of the connection of the strut nails to the existing wall.

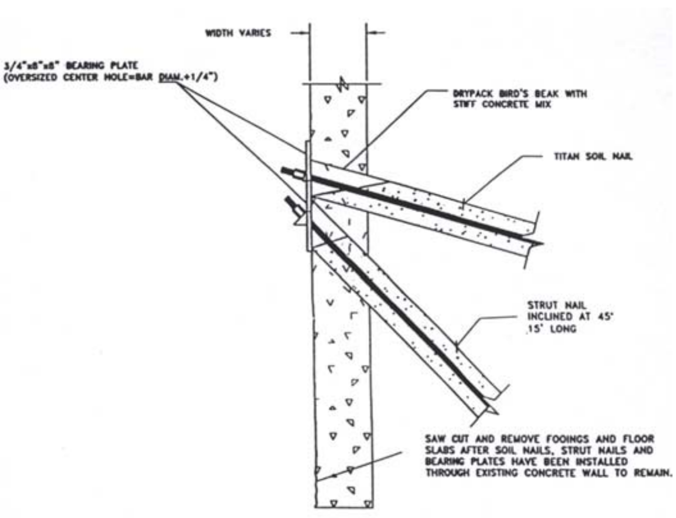

Fig. 5. Strut nail connection to the existing concrete wall

During the installation of core holes in the existing wall, it was observed that the gravel drain at the base of the wall was partially filled with grout. It appeared that the neat cement grout from the strut nail installation had contaminated the gravel drain, contributing to the buildup of groundwater behind the existing wall (the continuity of the drain pipe had already been compromised as discussed above). This also may have contributed to a poor structural connection between the strut nail and the existing wall. Without a continuous grout column around the strut nail, the nail would not be able to support the weight of the existing wall by functioning as a compression member as intended by the design. It is the author's opinion that this poor structural connection led to the failure of the strut nail system and the resulting vertical wall movement. In essence, the soil nails (both regular and strut nails) were supporting the existing wall in a bending mode (similar to nails supporting a picture frame on a wall).

When the shotcrete for row 6 had been completed, the total wall movement was 1.2 inches horizontal and 1.6 inches vertical (see Table 1 and Fig. 4). The proof tests were then completed on the row 6 soil nails. All four of the proof test nails failed in pullout (between 65 and 130 percent of the design pullout of 3.5 kips/foot). Because the design required a factor of safety of 2 on the pullout, replacement soil nails were installed. The replacement nails were installed in between the original nails. With the additional of the replacement soil nails, the effective nail spacing was reduced from 5 feet to 2.5 feet. The replacement nails were installed through the existing shotcrete facing. A rock hammer was used to break through the existing shotcrete. The vibrations of the rock hammer resulted in an additional 0.4 inches of horizontal wall movement and 0.4 inches of vertical wall movement. Proof testing of the replacement nails indicated that 4 out of 5 nails failed in pullout (again between 65 and 130 percent of the design pullout of 3.5 kips/foot), but the sum pullout capacity of the initial nails plus the replacement nails met the required row 6 pullout capacity.

At the completion of row 6, the total vertical and horizontal wall movements were 1.9 inches and 1.6 inches, respectively. At the inclinometer, the total horizontal movement was 1.7 inches, of which 1.1 inches occurred on three shear planes located at depths of 6, 10 and 28 feet below the bottom of the excavation (depths of 40, 44 and 62 feet below the top of the wall)(see Table 2 and Fig. 4).

<u>Row 7 Observations</u>. Because of the continued vertical movement of the wall, the contractor proposed installing a series of micropiles to replace the strut nails. The micropiles were installed at the bottom of the row 6 shotcrete. The micropiles consisted of near vertical, 6-inch diameter, 32-foot long, post-grouted nails. After installation of the micropiles, the vertical wall movements slowed significantly, but did not stop.

As discussed above, 8 out of 9 proof test nails in row 6 failed in pullout and the planes of movement below the base of the excavation suggested soil conditions with strength parameters less than those originally assumed for design. The horizontal spacing of the row 7 soil nails was decreased from 5 feet to 4 feet and the nail length was increased by 8 feet to account for the low pullout values in row 6 and lower soil strength parameters along the planes of movement. In an attempt to further increase the pullout capacity of the row 7 nails, the standard 4.3-inch diameter drill bit for the soil nails was modified by welding small steel plates to the bit. The modified drill bits were approximately 6.5 inches in diameter.

Per the specifications, the modified drill bits constituted a new drilling method and additional validation tests were required.

The validation test nails and production nails were installed through a 1H:1V native drill berm, similar to that described for row 6. The soil nails were tested prior to shotcrete placement. The initial five test nails (two validation nails and three proof nails) were tested using a design pullout value of 3.5 kips/foot. Of these five nails, one validation nail and one proof nail failed in pullout at 175 percent and 125 percent of the design pullout value. Based on these test results, the design-build team reevaluated the soil nail design and determined that a design pullout capacity of 2.8 kips/foot was sufficient for the row 7 nails. An additional validation test nail was installed to validate this reduced value. The remaining two proof test nails were also tested with this lower value. The results of the testing, taking into account the lower design pullout capacity, indicated that the three validation test nails and five proof test nails were successfully tested to the lower value.

During soil nail installation, it was observed that the drilling method with the modified bit did not clean out the holes completely. A remolded soil plug would tend to form in the hole, causing little grout return at the drill hole face and a highly pressurized drilling fluid. A significant amount of grout communication was observed between adjacent nails. In addition, fractured soils were observed at the soil face.

Excavation and shotcrete placement for row 7 proceeded similarly to row 6 by excavating in slots. The slots were typically between 20 feet and 25 feet in length. The final slot was completed approximately 2-1/2 weeks after construction on row 7 began.

During construction of row 7, the movement of the wall continued. The wall moved vertically approximately 0.3 inches, for a total cumulative movement of 2.2 inches. The horizontal wall movement increased from 1.6 inches to 2.1 inches. As with row 6, a significant amount of the additional movement occurred below the bottom of the excavation. A fourth shear plane also developed 12 feet below the excavation (52 feet below the top of the wall) and accounted for about one-half of the horizontal movement observed during row 7 installation, as shown in Fig. 6.

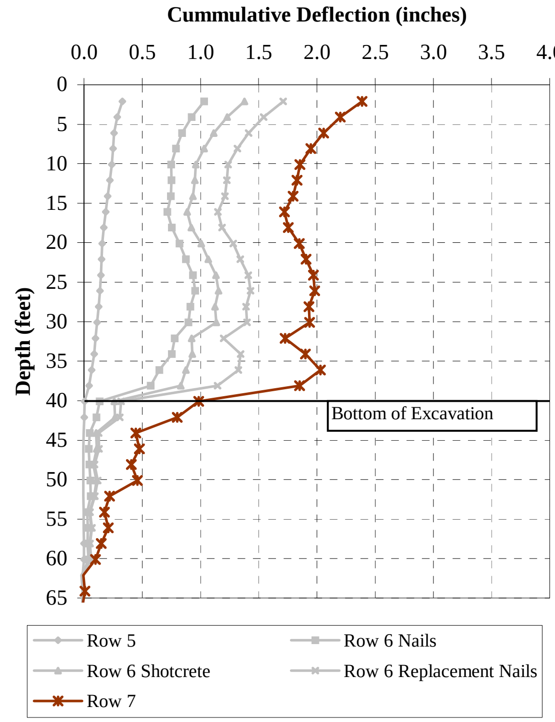

Fig. 6. East wall inclinometer data after completion of row 7.

Row 8 and 9 Observations. Due to the continued movement of the wall in response to excavation and the estimated final movements at the end of construction, the below-grade design for the project was modified to remove the bottom level of excavation. The row 9 excavation was completely removed from design to limit depth of excavation. This resulted in structural, architectural, and mechanical redesign for this portion of below-grade structure.

Installation and construction techniques were again modified for the row 8 shotcrete. The row 8 and 9 nails were installed together through a narrow trough excavated in a staggered pattern. The trough was approximately 3 feet deep and 20 feet in length. The row 8 nails were installed at the bottom of the vertical cut near the wall face while the row 9 nails were installed through the bottom of the trough (approximately 1 foot away from the row 8 nails). The row 9 nails were installed to provide increased capacity for the wall and to account for the lower 2.8 kip/foot design pullout value. Each staggered trough area was backfilled immediately after nail installation and prior to excavating for the adjacent trough slot.

The soil nails were proof tested prior to shotcreting the wall, similar to the row 7 nails. This method was chosen to reduce the risk of vibration caused by breaking through shotcrete during replacement nail installation, if required. Again, two out of two test nails failed in pullout before reaching the required pullout capacity. After further stability analyses by the design team, the design pullout capacity was reduced in half to 1.4 kips/foot. Replacement nails were installed, reducing the nail spacing from 5 feet to 2.5 feet. All subsequent test nails passed with the 1.4 kip/foot pullout capacity.

During the replacement nail installation, a significant amount of grout communication occurred. As discussed previously, the soil plug that tended to form during drilling would hinder the grout return. It appeared that the grout would then pressurize and begin flowing through the cracks and fractures within the Lawton Clay. Grout was observed flowing from the wall face through fractures in the silt as far away as 25 feet from the drilling location. Based on the survey and inclinometer data (Tables 1 and 2 and Fig. 7), approximately 0.4 inches of horizontal wall movement occurred during installation and grouting of the replacement nails. Approximately one-half of this movement occurred below the bottom of the excavation.

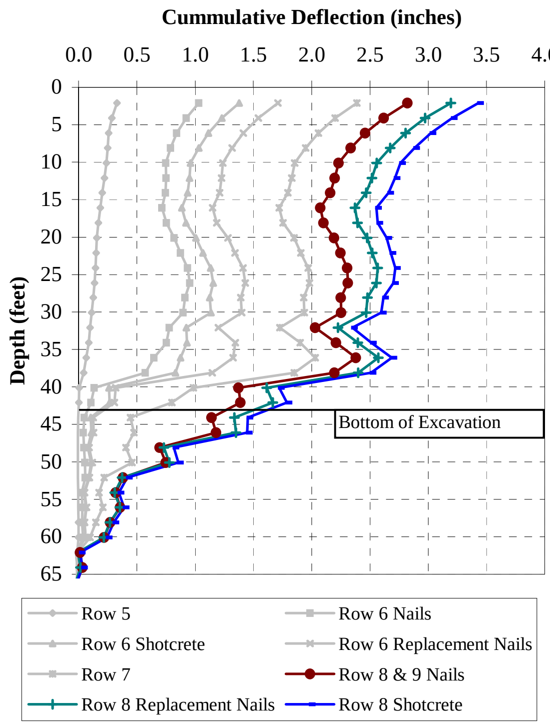

Fig. 7. East wall inclinometer data after completion of row 8.

The shotcrete for rows 8 and 9 was placed in a similar manner to row 7 using the staggered slot excavation approach.

At the completion of rows 8 and 9 shotcrete, the total vertical wall movement was 2.34 inches and the total horizontal movement was 3.18 inches. As with previous rows, the majority of the movement occurred on shear planes below the bottom of the excavation. During this stage, a fifth shear plane occurred at a depth of 5 feet below the base of the excavation (depth of 48 feet below the top of the wall).

Once the shotcrete was installed in the trench slots, mass excavation of 6 feet of soil in front of the wall occurred to bring the site grade down to the bottom of wall elevation. This mass excavation occurred between 0 and 50 feet in front of the wall and resulted in an additional 0.25 inches of horizontal movement and 0.1 inches of vertical movement, as presented in Tables 1 and 2 and Fig. 8.

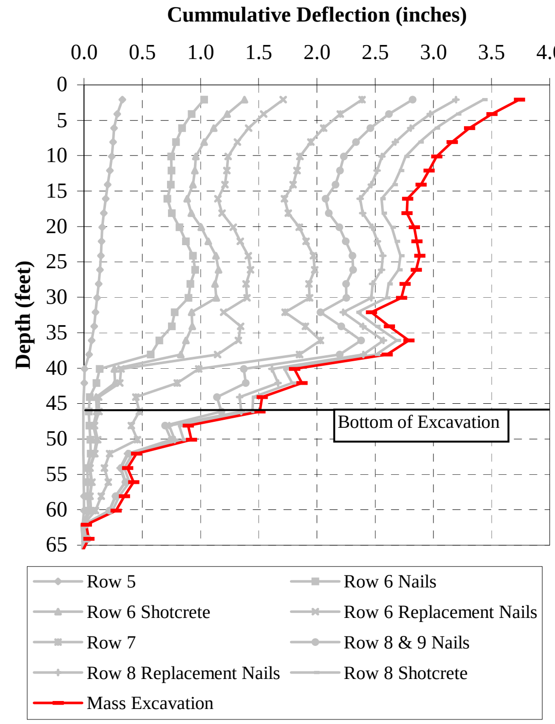

Fig. 8. East wall inclinometer data after completion of mass excavation to bottom of wall elevation.

<u>Footing Excavations in Front of the Wall</u>. Interior footing excavations were completed immediately after the completion of the east shoring wall installation. Instead of completing separate excavations for each column footing, the contractor decided to excavate the column footings using a trench that extended along the length of the east wall. The trench excavation was approximately 3 feet deep and 15 to 20 feet wide. The centerline of the footing trench was located approximately 28 feet west of the completed east shoring wall.

During the footing excavations, the wall movement continued. The horizontal movement of the wall increased 0.12 inches to 3.54 inches while the vertical movement increased 0.06 inches to 2.52 inches, as presented in Tables 1 and 2 and Fig. 9.

<u>Following Construction</u>. Movement of the east wall, both horizontal and vertical, continued to occur after all earthwork construction activities were completed in front of the wall. Movement was measured over a period of several months. The movement essentially stopped when enough building structure (i.e. permanent walls and floor slabs) had been constructed in front of the shoring wall. The horizontal and vertical wall movements that occurred during this period were 0.36 inches and 0.12 inches, respectively (see Tables 1 and 2 and Fig. 9). Very little of the horizontal movement (0.03 inches) occurred below the bottom of the excavation. The majority of movement was outward rotation of the wall.

These movements were occurring with little or no additional load being applied to the wall, which is the definition of creep movement.

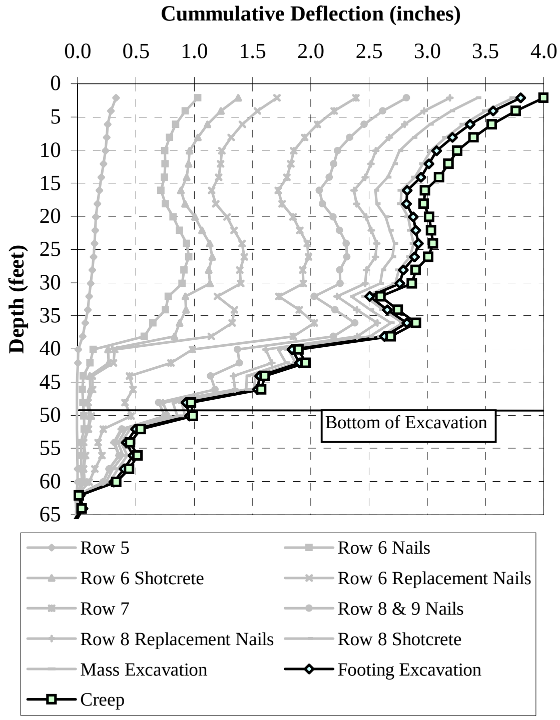

Fig. 9. Final east wall inclinometer data.

The design-build team's interpretation of the data following construction was slightly different. They concluded that hydrostatic water pressures were periodically building up in the backfill behind the existing concrete wall due to rainfall and this cyclic hydrostatic pressure was causing the wall movements. The design-build team subsequently designed and constructed an active dewatering system by applying a vacuum suction to the existing horizontal drains. In addition, seventy-two 4-inch-diameter core holes were installed throughout the height of the existing basement wall. At no time during this period was groundwater seepage observed to occur through these core holes. Also, when the active dewatering system was initiated, the maximum amount of water removed from behind the east wall was 23 gallons per day.

When the wall movements finally stopped the total horizontal and vertical movements for the east shoring wall were 3.9 inches and 2.64 inches, respectively. The total movement in the inclinometer was 4 inches, of which 2.5 inches occurred on planes below the base of the excavation.

CONCLUSIONS AND LESSONS LEARNED

Vertical Movements

It is the opinion of the authors that the vertical wall movements were the result of failure of the strut nails installed along the bottom of the existing east concrete basement wall to perform as intended. The strut nails were designed to support the full weight of the existing wall. However, when the wall footing was initially undermined, the wall dropped vertically 1.2 inches over a short period of time (3 days). Vertical wall movements continued throughout construction, even after micropiles were installed to resupport the wall. The vertical wall movement at the end of construction was 2.64 inches.

It appears the strut nails failed to support the existing basement wall because of a poor connection with the existing wall. The design theory of strut nails is similar to a truss system. With a rigid connection at the wall face, the strut nails (which were installed at 45 degrees from vertical) are a compression element. A rigid connection would require a continuous column of grout in the annulus between the nail bar and the hole drilled through the existing wall and around the nail bar directly behind the wall. A continuous grout column was probably not achieved because the strut nails were grouted with a neat cement grout, which likely flowed into the gravel drain rock directly behind the wall before setting up around the nail bar. As discussed above, we also observed a considerable amount of soil within the grout of the strut nail connection. A photograph of one of the connections observed in shown in Fig. 10. When the existing wall was undermined, the poor connection did not allow full compression of the strut nail and the strut nails, along with the regular soil nails, were loaded in bending.

Fig. 10. Observed strut nail connection at existing wall.

As discussed above, vertical wall movements, on the order of 0.4 inches, occurred after installation of the micropiles below the row 6 shotcrete. During construction of rows 7 and 8, the micropiles were exposed and grout around several of the micropiles had to be removed in order to install the required thickness of shotcrete. Additionally, as lateral movement occurred below the base of the excavation, the authors postulate the eccentric load on the micropiles increased. It is believed that these factors resulted in the additional vertical movement of the wall after the micropile installation.

Horizontal Wall Movements

General. The horizontal movement at the top of the east wall was 3.9 inches based on the optical survey and 4.0 inches based on the inclinometer data. This movement can be attributed to three different components of movement, including 1) lateral translation and movement of blocks along planes of weakness in the Lawton Clay 2) outward rotation of the wall to mobilize force in the nails as the excavation proceeds and 3) creep movement following construction. Each of these is discussed in more detail below.

Lateral Translation. Lateral translation of blocks of soil along planes of weakness in the Lawton Clay was observed throughout construction. This mechanism of movement is initially an elastic rebound response of the overconsolidated silt and clay as the lateral confining stress is removed (excavation). This movement is consistent with lateral block movements within the Lawton Clay in other areas of the city (Peck, 1963, Palladino, 1971, Clough et. al. 1972).

Generally, the lateral translation movements are relatively small (a few inches or less) because of the overall high strength of the Lawton Clay mass. However, these small movements can result in large scale stability issues, depending on the joints and fractures in the clay and the groundwater conditions. The movements allow joints and fractures in the soil to open. With the presence of groundwater in the open joints and fractures, the intact material adjacent to the joints and fractures begin to swell, resulting in higher movements and a significant loss of soil strength. Additionally, free hydrostatic pressure, or even grouting pressures as evidenced during row 6 and 8 soil nail installations, can build up in the joints and fractures, resulting in block movement along the weakened joints.

During construction of each soil nail lift below the existing basement wall, lateral translation along planes of weakness were observed in the inclinometer data (see Table 2 and Fig. 4, 6 through 8). The planes of weakness were located below the bottom of the current excavation. Movement initially occurred on three separate planes of weakness. However, as the excavation was deepened, two additional planes of weakness developed. During each wall lift, the majority of the lateral translation movement occurred on the plane of weakness nearest the bottom of the excavation. This plane was typically located between 5 to 12 feet below the current bottom of excavation.

The total amount of lateral translation movement was calculated by summing the total incremental movements that occurred on the five planes of weakness. Of the 4.0 inches of total wall movements, 2.5 inches of this movement was attributed to lateral translation along preexisting planes of weakness. It is the authors' opinion that the block movement observed is related primarily to elastic rebound (2 inches of the 2.5 inches of block movement) with some contribution (0.5 inches) from the grouting pressures. Luckily, large scale sliding along the planes of weakness did not develop.

Creep. Creep is defined as continued movement in response to little or no additional applied load. No appreciable additional load was applied to the east shoring wall after the excavation for the footing trough was completed. However, movement of the wall continued for a period of several months following this footing trough excavation. The optical survey data indicated that the top of the wall moved approximately 0.35 inches during this period while the inclinometer data showed that the top of the wall moved approximately 0.2 inches. Additionally, the inclinometer data showed that little of this movement occurred on the planes of weakness below the excavation. The trend of movement was outward rotation (greatest movement at the top of the wall and reducing to near zero movement at the bottom of the wall).

The creep movement of the wall was a significant concern during construction because the wall continued to move after it was completed. The creep movement implies that the soil nails had a pullout capacity at or near a factor of safety of 1.0 as opposed to the design safety factor of 2.0, which essentially eliminated the shoring systems capacity to accommodate additional unforeseen loading or events. The soil nails were tested to a factor of safety of 2.0 during construction. The authors believe that numerous factors may have contributed to the creep movement of the wall following construction, including:

- The pullout capacity of the soil nails continually reduced as lateral translation movements occurred during construction. As these nails softened, the loads were transferred to other soil nails, which likely contributed to the creep movements observed.
- High variability of pullout capacities of the proof test nails. The test nail program, which consisted of testing 1 out of every 10 nails, may not have been representative due to high variability.
- Testing of proof nails in rows 6 through 8 always lowered the design pullout capacity. It is likely that the upper 5 rows of soil nails were given more credit that they deserved. Additionally, as discussed further below, the length of the proof test nails was about ½ the length of the

production nails. For the upper 2 to 3 rows of soil nails, the proof test nails would have been fully within the granular backfill behind the existing basement wall whereas the production nails would have extended through this backfill into native Lawton Clay. Therefore, these proof tests were not representative of the production nails and likely overstated the capacity of these production nails.

- There may have been a small increase in the weight of the backfill during high precipitation events (increase in soil moisture content), which slightly increased the driving forces from a stability standpoint.

Outward Rotation. Outward rotation of a soil nail wall is required to mobilize force in the nails as the excavation proceeds. The magnitude of outward rotation is dependent on the type of soil being retained. Typical movements for soil nail walls in stiff clays, residual soils and sands are estimated to be between 0.2%H to 0.3%H (Clough et. al. 1991) and in fine-grained clay type soils are estimated to be 0.4%H, where H is the height of the wall (Clouterre, 1991, FHWA, 1996).

The total amount of outward wall rotation is determined by subtracting the lateral translation movement (inclinometer data) and the creep movement from the total wall movement. The outward wall rotation for the east wall is estimated to be between 1.1 to 1.3 inches. For a completed wall height of 47 feet, this corresponds to 0.2%H to 0.24%H, which is consistent with the above published movement and the designers estimate of 0.1%H to 0.3%H.

Causes of Low Pullout Values

A significant number of tested soil nails on the east wall (proof and verification test nails) demonstrated extremely low pullout capacities. The initial design pullout capacity of the nails was 3.5 kips/foot. During construction, the design pullout capacity was reduced to as low as 1.4 kips/foot. There are several reasons, in our opinion, for the low pullout values:

- Fractures and planes of weakness in the soil unit as stress relief and lateral translation movement occurred. This resulted in softening and loss of soil strength near the planes of weakness. It also resulted in loading of the nails at the plane of weakness. If the plane of weakness was located outside (beyond) the failure plane assumed in design, the nails would be attracting more load than assumed for design. This would also result in a reduced pullout capacity of the nails because the Lawton Clay is a strain-softening soil material.
- Inability to completely clean out the silt and clay cuttings during the grout-injected nail installation. This may have resulted in plugs of remolded, disturbed soil left in the hole and thus incomplete grouting of the nails. Drilling spoils were observed in the nail hole annulus of several nails upon cutting the soil face in preparation of shotcrete, as shown in Fig. 11 and 12.

Fig 11. Nail hole annulus. Majority of 6-inch-diameter hole is soil cuttings, with some grout around nail bar

Fig 12. Nail hole annulus with no grout in drill hole or around nail bar

- Communication of grout between adjacent nails resulted in grout pressure losses for nail installation, which further compromised the ability to remove the soil cuttings from the hole.
- The modified drill bits likely had a negative impact to the pullout capacity. The intent of the modified drill bits was to increase the diameter of the drilled hole, thereby increasing the pullout capacity for a given soil/grout adhesion. However, it is likely that cuttings broke off in larger chunks and could not be adequately flushed from the hole, resulting in significantly lower adhesion values. Additionally, the grout pressures were reduced because of

the size of the drill hole, further reducing the ability to remove drill spoils from the holes.

- The length of the proof test nails was typically ½ the length of the production nails. The reason behind this was because a large free length (unbonded length) of nail would have been required to adequately proof test the production nails, which was difficult to accomplish because the nails were installed using grout injection methods. The proof test nails were likely not representative of the production nails because as the nail length increased, the ability to remove soil cuttings from the hole was significantly impacted. The author's believe that the production nails actually had a much lower pullout capacity than the test proof nails indicated.

Drilling methods

The installation of grout-injected anchors longer than about 20 feet in Seattle Lawton Clay may not be appropriate drilling technique in the opinion of the authors. It was very difficult to remove the soil cuttings from the anchor holes, particularly at depths greater than 20 feet. The soil cuttings appeared to form a plug in the hole and would hinder the grout return during installation. The grout in the hole would then pressurize and often begin flowing through cracks and fissures within the soil unit. In one instance, grout was observed flowing from a location 25 feet laterally and 5 feet above the drilling location. Also, the grout pressure became high enough to actually cause horizontal wall movements on the order of 0.4 inches during the row 8 replacement nail installation. The incomplete removal of cuttings from the nail holes is the primary reason for the highly variable pullout capacities attained, in the opinion of the authors.

Performance of Other Walls

The monitoring data for the west, south, and north shoring walls generally showed only outward rotation of the walls during construction, typical of soil nail shoring walls. The horizontal and vertical wall movements were less than 1 inch and the walls performed as expected. The main difference between these sides of the excavation and the east side was previous development activity. Deep excavations had not been completed adjacent to the east side of the site. However, several deep excavations had occurred along or adjacent to the other three sides and these excavations likely allowed sufficient movement and elastic rebound to release the high locked in lateral stresses in the Lawton Clay. The maximum horizontal movement of the west, south and north sides of the excavation varied between 0.13%H to 0.2%H, which compares well with the published correlation's and the designers estimate. In addition the topography (cross slope), wall height (shorter than east wall), and orientation of the fractures and bedding planes of the Lawton Clay appeared to be more favorable for the other three sides of the excavation.

In addition, the east wall completely undermined the existing basement wall that was incorporated into the shoring design. The other sides of the excavation did not incorporate the existing basement walls into the shoring system, or if they did, these walls were not completely undermined as the east wall was.

Applicability of Soil Nail Shoring in Seattle Lawton Clay

In our opinion, extreme caution should be used when considering typical soil nail walls in the Lawton Clay or similar over-consolidated fine-grained soils. This is particularly true if the excavation will occur in areas where stress relief has not previously occurred, such as the east side of the site for the Seattle Library Project. Typical soil nail wall design consists of passive soil anchors which require soil movement to load the nails and mobilize resisting forces. Generally, little movement is required to mobilize the resisting forces but the passive anchors do not restore a portion of the locked in lateral stress, as a pre-stressed anchor might. And unlike soldier piles that would act as a dowel and provide shear resistance where they intercept planes of weakness, the small soil nails provide little shear resistance along these planes. The release of the previously locked in lateral stresses can trigger horizontal translation of blocks within the soil unit. These movements occur on preexisting planes of weakness or on new planes of weakness caused by the stress relief. The movement can result in significant loss of strength at joints and fractures within the clay mass, which in turn result in additional translation of the blocks. The observations and evaluation of the inclinometer and survey data for the Seattle Library project support this conclusion.

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