Emergency Underpinning and Repositioning of a Four-Story Office Building

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EMERGENCY UNDERPINNING AND REPOSITIONING OF A FOUR-STORY OFFICE BUILDING

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ABSTRACT

A three-story office building was located above a 35-ft slope and was supported on spread footings. The corner of the building nearest the crest of the slope experienced several inches of settlement. A subsurface exploration revealed that this portion of the building was founded on recently placed, soft fill soils. Apart from being easily noticeable on the brick veneer façade, the distress also caused serious concerns on the part of the occupants and owner regarding the continuous operability of the structure, which served as a main hub for wireless communications.

A repair plan was developed that consisted of installation of micropiles outside and inside the building. The micropiles were installed around the existing footings, penetrating through the fill and into rock. Connection between the micropiles and the structure was achieved through a steel grillage attached to the column piers. Hydraulic jacks were installed between each grillage and the top of the micropiles to allow repositioning of the structure. Once the building was repositioned, the micropile-to-column connection was embedded in concrete. The work was completed in a period of approximately three weeks after the start of construction. The entire operation took place without disrupting business operations inside the building and was entirely successful, as evidenced by the settlement records of the structure. This paper describes the process of design and construction of the underpinning, the materials and equipment used for micropile installation and stressing, and the problems encountered during implementation of the proposed plan.

INTRODUCTION

A three-story office building located north of Harrisburg, Pennsylvania, with a partial unfinished basement, experienced severe settlement that affected not only the exterior finishes but also the interior usability of this portion of the building. Displacement of the first full floor and the basement column was visible. Distress was noticeable along both the southern and eastern walls to distances of about 50 ft from the southeast corner of the building. Emergency actions were required to restore the integrity of the building and maintain operations within the facility.

The structure is steel frame with brick and glass veneer, as well as cast in place concrete walls in below-grade areas (see Fig. 1). The ground surface slopes from the front of the building to the back with a change in elevation of about 14 ft.

Available construction records included a preconstruction geotechnical investigation, and limited construction testing reports. According to civil site plans, the original ground surface before construction of the building was about EL 391 at the present location of the southeast corner of the structure.

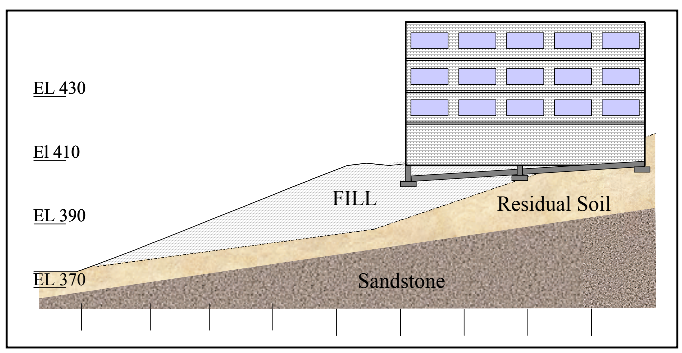

Fig. 1. Interpreted subsurface conditions.

A cast-in-place concrete wall on the east side of this building supported one edge of the first floor slab. This wall was supported on a shallow strip footing independent from the column footings. Therefore, differential settlement was taking place between the wall and the rest of the structure. This caused deformation and loss of support of the first floor slab.

A local soil testing agency provided site service during construction. According to available records, the subgrade within the building area and portions of the fill were tested. However, no test data was available for the material between 6.5 and 15 ft below subgrade.

According to the testing agency's field reports dated approximately four months after the date of fill placement, soft soils were observed at the bottom of the excavations for the footings at the southeast corner. Recommendations were made that the loose materials be removed from the bottom of the excavations before construction of these footings. Yet, there were no records of this work being completed.

Given the concerns about the limited construction documentation and the visible distress, a program of exploration and development of remediation alternatives were initiated.

EXPLORATION PROGRAM

Standard test borings with continuous split spoon sampling were performed in the area of concern. Three test borings were completed to depths of about 30 ft. The conditions encountered were generally silty and clayey sand (SC, SM) fill to depths of about 17 to 23 ft (see Fig. 2). Laboratory testing indicated that these soils had Liquid Limits of about 25 to 35, and Plasticity Indices of 8 to 15. Moisture contents were in the range of 14 to 23%. Beneath the fill were several feet of a residual silty and clayey sand derived from the underlying rock. The rock consisted of sandstone with frequent shale seams. No groundwater was encountered in the test borings.

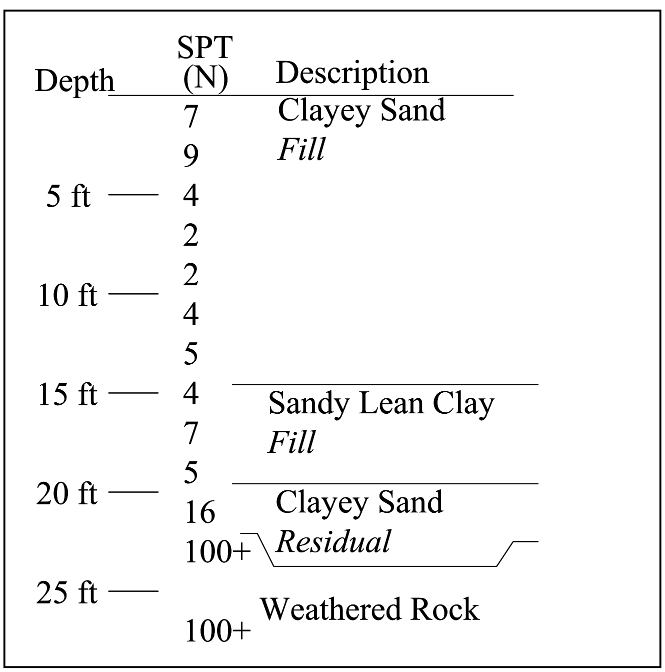

Fig. 2. Subsurface conditions under southeast corner.

INSTRUMENTATION

Six standard crack monitor gauges were installed to supplement several already installed by the structural engineer for the project. A graph of typical data is shown in Fig. 3.

In addition to the crack monitors, spot checks on the level of the first floor interior and exterior were made from the basement area. Based on measurements of the vertical crack widths inside the building, the total settlement of the southeast corner was estimated to range between 3 and 3½ inches. Optical observation of the southern and eastern exterior walls suggested that settlements of two to 2½ inches may have occurred after installation of the brick veneer. Given these measurements, it appears as though there was an original settlement of about one inch shortly after construction of the wall, and an additional two to 2½ inches of settlement since placement of the brick veneer.

Visual inspection of the existing cracks, and the separation of the south wall from the first floor slab, indicate that the movement observed in the building had both a horizontal and a vertical component. Global stability analyses of the slope were performed to try to assess the causes for the observed movements. The analyses suggested that compression of the fill materials, and not instability of the slope, was taking place. This was supported by the absence of evidence of movements on the slope itself and by the surveying data collected during installation of the micropiles.

It is conceivable that, due to the soft condition, the fill materials may have compressed over time inducing the observed movements of the structure. During such a process, lateral movements may be a natural consequence of compression of the unconfined fill lying on the inclined incompressible substratum. Lateral displacements may be similar in magnitude to the vertical compression of the fill. Configuration of the connections of structural elements may have also contributed to the observed horizontal movements.

REMEDIAL RECOMMENDATIONS

The columns and walls subject to movement required underpinning. Consideration was given to the use of micropiles, compaction grouting, and soil nails to provide for this underpinning of the structure, as well as stabilization of possible slope movements, and traditional underpinning pit excavations.

As indicated previously, movement of the slope was not considered to be the cause of the distress. The localized condition of the fill beneath the southeast corner foundation, resulting primarily in vertical compression under self weight as well as the weight of the building, was believed to be the most critical issue.

When evaluating the alternatives, this reduced the options to micropiles, compaction grouting, and pit underpinning. Injection of the compaction grout at very high pressures, and with the anticipated large volumes that could be necessary to improve the

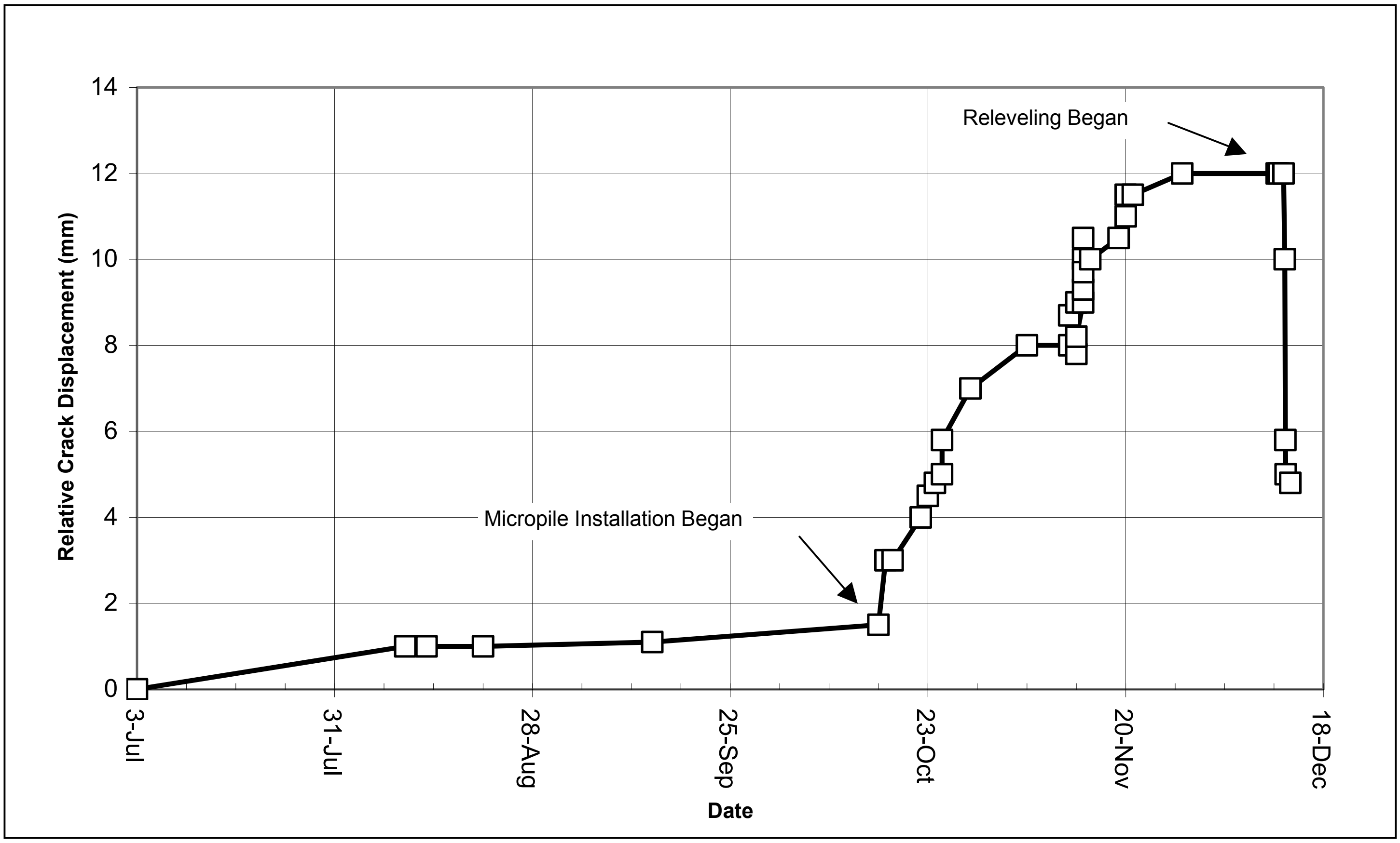

Fig. 3. Crack monitoring data.

fill, caused concern that slope movements may be induced. Excavation of pits to about 20 ft was also considered to be more difficult and intrusive than the micropile alternative. As such, the use of micropiles was selected for the stabilization effort, and continued monitoring of the slope was to be made.

INSTALLATION OF MICROPILES

The initial design required the installation of ten 5.5-inch diameter micropiles with a design capacity of 60 kips. After excavating inside the building at the southeast corner, it was determined that additional micropiles were necessary for support of the east wall. Consequently, additional piles were added along the east wall.

Drilling of each micropile was accomplished by end of casing flushing a 5.5 inch diameter casing under air pressure until reaching top of rock. Drilling through rock was performed with a downhole hammer to create a 4.5 inch diameter bond zone. The micropiles were filled by the tremie method with a neat water-cement grout with a specific gravity of about 1.8 and a design strength of 5,000 psi. Each of the micropiles was reinforced through the bond length with two No. 8 bars, Grade 60 ksi, with centralizers spaced at approximately 10 ft centers. Figure 4 shows a typical Case I (Bruce, 1988, 1989) micropile used for this project.

During drilling of the first micropile on the exterior of the east wall, the cuttings observed suggested that the upper 10 ft of rock were highly weathered and fractured. Given this information, micropile bond length was extended to 20 ft into rock since there was not a test program to confirm micropile capacities.

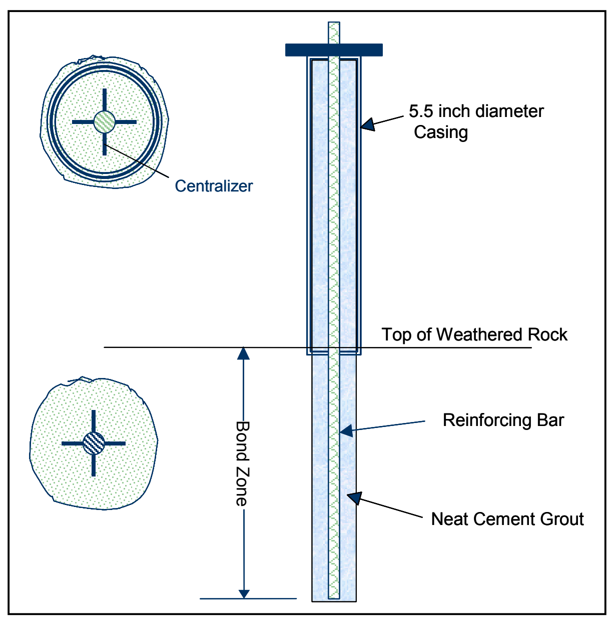

Fig. 4. Case I micropile (adapted from Bruce, 1988).

Movements were observed in the structure as the first three micropiles were being installed. To further limit potential disturbances from the drilling process, the installation method was adjusted to utilize duplex drilling methods where flushing of the cuttings occurs within the outer casing.

Crack gauge measurements revealed that lateral movement of the top of the east wall had apparently accelerated during installation of micropiles. In addition, some of the existing cracks in the brick veneer on the east wall had visibly opened farther. It is possible that micropile installation may have impacted the rate of settlement of the wall, and consequently increased the loss of lateral restraint provided by the first floor slab. However, the reasons for this accelerated movement were not fully known. Consequently, temporary bracing and shoring were installed to limit further movements of the east wall.

LIMITED MOBILITY GROUTING (LMG)

Movements in the east wall occurring during pile installation raised concerns about continued stability in this area. As such, limited mobility grouting (LMG) injections were installed along the east wall to provide temporary vertical support to the wall during installation of the micropiles. A total of ten LMG grout columns was installed at about eight foot center to center spacing.

The grouting plan considered extending each grout hole to very dense weathered rock. The grout holes were battered slightly so that the soils under the existing wall footing could be treated. Grouting was performed using the bottom up procedure at all locations.

Grout pipe installation depths ranged from about 16 to 22 ft below the existing ground surface. The bottom of the footings in the area was nine to 12 ft deep. Therefore, grouting depths ranged from seven to 13 ft below the level of the footings.

Grout was injected in two-foot stages. Grout was pumped for each stage until: a maximum injection pressure was achieved for each stage; until soil heave or structural movement was observed; or until grout appeared at the ground surface. The maximum injection pressure was limited to 200 psi for the bottom stage, and decreased in 20 psi increments for each subsequent stage. This pressure is lower than typically used for soil densification due to the concern about the stability of the adjacent slope. The grouting was accomplished using a duplex piston positive displacement pump with a calibrated volume of 0.5 cft/piston stroke.

The grout takes per stage ranged from about 1.5 to 68.0 cft/stage. The higher grout takes were generally encountered at depths of 14 to 22 ft below the ground surface.

In addition, the east wall was braced using high strength strands to tie the wall to the first floor girders.

MICROPILE-TO-STRUCTURE CONNECTIONS

A grill made of steel channel sections was designed to be slid under the existing wall and span across the top of the new piles (see Fig. 5). These channels were then bolted to the existing column pedestal. Steel bearing plates and steel keeper rings were welded to the top of the micropile and bottom of the steel channel at each micropile location. Twenty-five ton capacity hydraulic jacks were installed at each micropile location (see Fig. 6). Jacking manifold circuits were installed at each group of piles.

Fig. 5. View of completed piles and steel grillage.

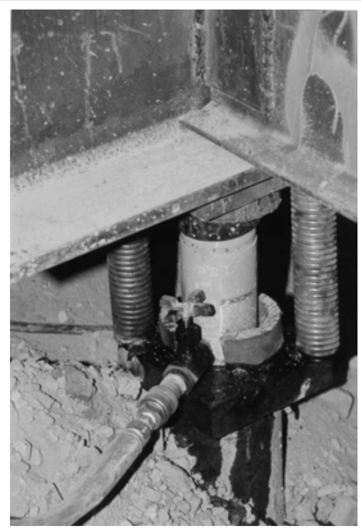

Fig. 6. Twenty-five ton jack after repositioning and lock-off.

Initially, the jacks were loaded to about 4,000 to 5,000 lbs to prestress the piles to the connection frame.

STRUCTURAL REPOSITIONING

The jacking process consisted of systematically loading the walls and then the corner incrementally. Final loads applied to each jack were about 50 kips at the column locations and 40 kips at the wall. Jacking was performed during the course of a regular office work day while the building was occupied (except in the area of the first floor where the slab had undergone distress).

Surveying of four control points was maintained during the structural repositioning. The control points were located directly above each pile cap location. In addition to the survey readings, personnel monitored the crack gauges, as well as dial gauges, piano wire, and mirrors mounted around the pile jacks.

Total settlement recovery measured by the survey was approximately one inch at the southeast column, and ¼ inch at the south column location. The recovery is depicted in the deflection plot of Fig. 3.

Wall settlement recovery values were 16 mm on the southern end of the east wall (near Column A-1.6), and seven mm at the midpoint of the east wall. Inward movement of the east wall was also observed in the range of 10 to 11 mm (east-west direction).

The jacking operation was considered complete after reaching the above settlement recovery values. No further jacking was attempted once the jack loads exceeded the estimated loads transmitted by the structure. The valves were closed on all of the pile jacks and two Williams All-Thread bars were welded between each pile bearing plate and the steel grillage to serve as a temporary lock-off mechanism until concrete was placed around the connection beams (see Fig. 6). The reinforcing steel was installed for the new pile cap, and concrete placement proceeded.

CONCLUSION

Completing this project, although not the largest remediation, required close coordination between the building owner, occupants, original structural engineer, remediation geotechnical engineer, and specialty contractors. Working with unknown conditions, as is typical of remediation work, requires flexibility and quick response from the contractor and the engineers. On this project, this meant developing revisions to the pile installation plan, use of low mobility grout and temporary shoring to stabilize a portion of the structure during pile installation, and adapting the pile cap connections to the existing structure.

The past experience that the geotechnical engineer had with the specialty foundation contractor allowed optimal use of the tools at hand to meet these challenges. The end result was a successful lifting of this structure back to a usable condition, and no further movements noted as of two years following completion of the project.

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