

# UNDERPINNING AND SHORING FOR UNDERGROUND MRI RESEARCH FACILITY AT OHIO STATE UNIVERSITY

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**Abstract:** A case study involving underpinning and shoring activities associated with the new Underground MRI Research Facility at Ohio State University is presented. Particular challenges overcome on this project include excavation along existing shallow footing foundations, construction inside the existing medical research center building, weak soils and shallow ground water conditions, zero tolerance for noise and vibration, and a fast-paced construction schedule. A solution was found involving underpinning using a hydraulically driven steel push piers. Shoring of the excavation was achieved using helical tieback anchors and reinforced shotcrete facing. The shoring was designed using a finite element plate analysis. Different lateral pressure distributions were applied depending on the stage of construction. Potential buckling of steel push piers and the vertical component of earth pressure on shoring and underpinning were taken into account. Vertical and lateral movement of the shoring and underpinning system were not detected during and after construction. Noise levels were low and no significant vibrations were experienced. The project was completed safely, cost-effectively, and on time.

## I INTRODUCTION

Construction in urban environments often requires working in close vicinity to existing structures. Underpinning of adjacent buildings and shoring of excavations can be challenging components of these projects. A case study involving underpinning, excavation, and shoring activities associated with a new underground MRI Facility is presented. The new facility was constructed inside the existing Medical Research Building at Ohio State University in Columbus, Ohio (Figure 1). The proposed facility is intended to house a Philips Intera 7T MRI system and associated shielding for human and animal medical research. The facility has heavily reinforced concrete walls and floor slab with 5-foot thick concrete interior partitions closely spaced within the floor plan. The entire system is anticipated to weigh up to 1,700,000 lbs. Dwyer Concrete Lifting, Inc. of Cincinnati was contracted to provide underpinning, excavation, and shoring services. A geotechnical investigation was conducted by the H.C. Nutting Company. Secure Foundations, LLC of Fort Collins, Colorado, provided structural engineering and earth retention design services. The unique design and construction activities selected by Dwyer and its consultants to provide a safe excavation amidst challenging soil and site conditions proved to be economical and effective.

The proposed excavation was particularly difficult due to soft soils, its location within an existing building, and limited access for construction equipment. The excavation measured only 22 ft x 40 ft and was 13 ft deep. Sides of the excavation could not be sloped due to spatial constraints within the existing building. Conventional shoring, such as sheet pile or soldier pile walls, was impossible due to low overhead clearance and limited areas to maneuver. In addition, the excavation bordered several existing masonry building walls with shallow footing foundations and an interior column. Other areas of the building remained occupied during construction, so vibration and noise were intolerable. A solution to these problems was developed wherein the existing foundations were underpinned using hydraulically driven steel push piers. The excavation was stabilized using helical tieback anchors and reinforced shotcrete facing. Presented herein are a review of the soil and site conditions, engineering analysis undertaken to design the underpinning and shoring systems, and an account of the construction activities.



Figure 1: Medical Research Institute at Ohio State University

## II SUBSURFACE PROFILE

Exploratory soil borings shown in Figure 2 revealed 21 to 22 feet of moist, soft to stiff, sandy, silty clay (natural cohesive glacial till soils) overlying wet, medium dense to dense, silty, sands with gravel (granular outwash deposits) extending to the depth explored of 30 feet. Ground water was encountered at depths of 10 and 22 feet. Both borings caved at depths between 16 and 17.5 feet. Samples of the glacial till soils contained 28 to 30% clay size particles and 31 to 38% silt size particles; plasticity index ranged from 12 to 24 and plastic limit ranged from 17 to 21. Samples of the outwash deposits contained 5 to 6% clay size particles and 5 to 11% silt size particles. Based on correlations with average SPT results, the glacial till soils were assumed to have a total shear strength defined by an angle of internal friction of 20 deg and a cohesion of 250 psf. Given this, the upper soils were judged capable of standing vertically up to 4 feet for a short period of time.

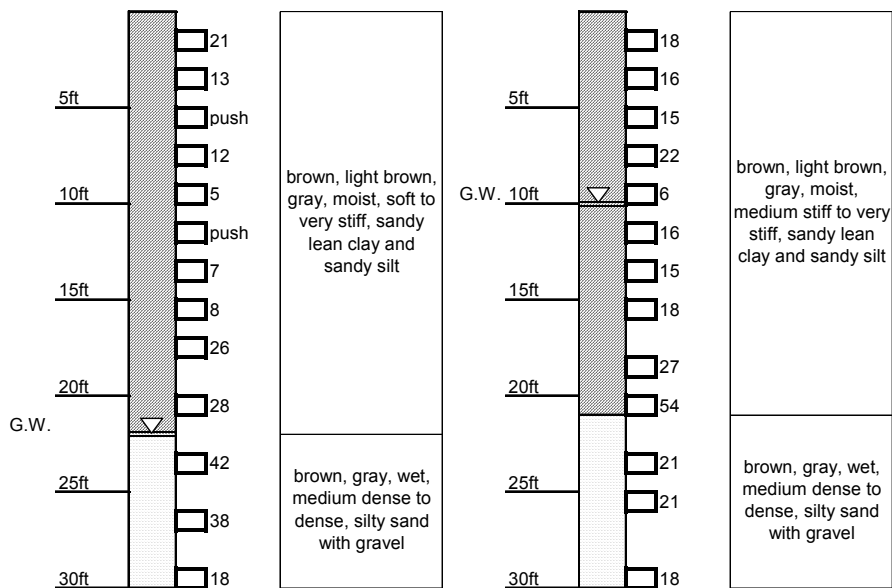


Figure 2: Logs of Exploratory Borings

### III UNDERPINNING DESIGN

A hydraulically driven steel push pier system manufactured by Magnum Piering, Inc. was selected to underpin areas of the existing foundation that would be undermined. This system consists of steel angle brackets attached to the existing footings. A hydraulic ram capable of exerting up to 80 kips of force on the steel pier is attached to the bracket. Dead load from the existing structure is used as a reaction to drive the piers, which are comprised of 36-inch sections of 3.00 inch diameter, 1/8" thick wall, high strength structural steel tubes with male/female sleeve connections. The piers are installed individually to the design capacity times a factor of safety of 1.5, and a load test is conducted at each pier location. The load test consists of applying the design load for a period of 30 minutes and monitoring pier deflection. The test is deemed successful if pier deflection is less than 1/16<sup>th</sup> inch. Since the test load is applied using rams connected to the underpinning bracket, the test confirms, not only the capacity of the pier, but also the capacity of the bracket, the connection of the bracket to the structure, and the structure's integrity at the pier location. Although the test load is generated using the dead load of the structure in the area of the pier, a factor of safety can be achieved by installing more than one pier individually using the same reaction area.

The final excavation, shoring, and underpinning plan is shown in Figure 3. As can be seen, one of the more critical areas where the existing shallow footing foundation would be undermined was the 22-foot length of existing building at section A/S1. The required underpinning was designed by first determining the maximum installation force that can be safely applied using the existing foundation as a reaction. In order to determine this, the existing foundation is modeled with estimated dead loads. A point load is applied to the bottom of the footing individually at critical pier locations. It was determined that a point load of 33 kips could be applied without lifting the structure or exceeding its allowable flexural and shear resistance. In most instances, the existing footing is chipped flush with the foundation stem wall to reduce torsional stresses applied to the foundation wall. Here, it was possible to install push piers on both sides of the foundation so that the footing could be left intact. The allowable capacity of each pier is found by reducing the maximum installation force by a factor of safety of 1.5. Hence, the allowable capacity of the push piers was 22 kips each on this project.

The final step in underpinning design was to determine the required pier spacing by estimating the total dead and live load of the existing structure plus any new loads due to excavation and shoring. The total dead and live load on the structure in the area of section A/S1 was 2.3 kips/ft. A static force analysis revealed that the planned shoring and excavation would exert an additional downward component of force on the underpinning equal to 6.5 kips/ft. More details regarding this static force analysis are described in the next section. The total load on the underpinning system is the sum of the building loads and the shoring load, which equals 8.8 kips/ft. The required pier spacing is the allowable capacity of the piers divided by the total load per lineal foot or 22 kips divided by 8.8 kips/ft equals 2.5 feet on-center.

### IV SHORING AND EARTH ANCHOR DESIGN

Given the restrictions of the site and capabilities of the contractor, a shoring system consisting of helical tieback anchors and reinforced shotcrete facing was selected to stabilize the excavation. Due to the nature of the anticipated soil conditions, it was determined that the excavation would be conducted in three phases. The first phase consisted of excavation to the bottom of the existing footing foundation and installation of push piers. Once the footing was supported, it was determined that a top row of helical tiebacks would be needed to retain the earth prior to subsequent excavation.

This row of tiebacks was attached to the existing footing and stem wall by installation of reinforced concrete haunches. Shear, flexural, and torsional loads were transferred to the existing foundation by reinforcing steel dowels drilled and epoxied into the bottom of the footing at each haunch. The second phase of construction consisted of excavating 52 inches below the existing foundation, installing a second row of helical tie-backs at the base of the excavation, and placing a reinforced shotcrete facing. The third and final phase of construction involved completing the excavation to the desired depth and installation of the bottom section of reinforced shotcrete.

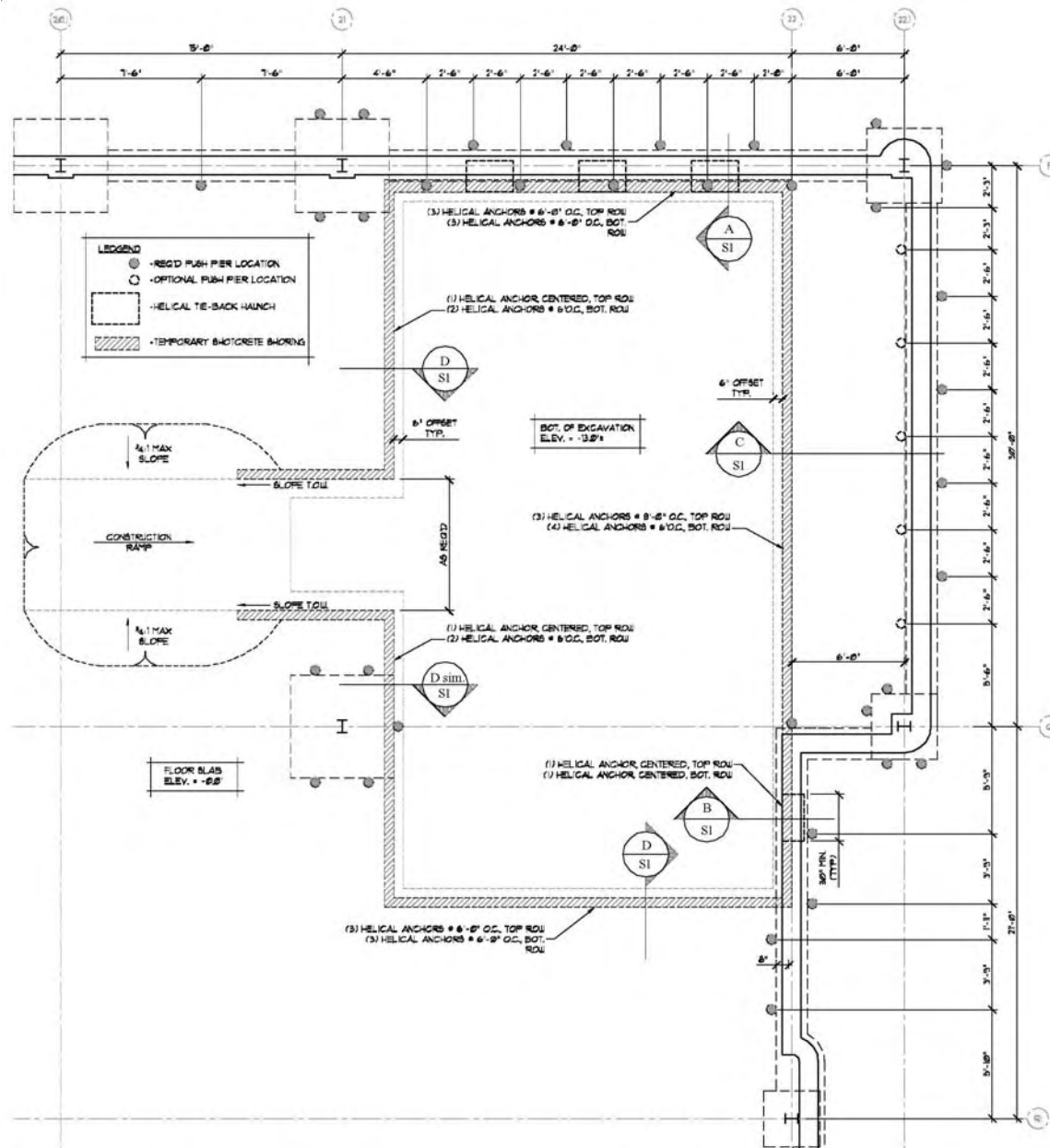


Figure 3: Underpinning and Shoring Plan

The stability of the shoring system during each phase of construction was checked using a static force analysis. A schematic diagram of the shoring system for two phases of construction is shown in Figure 4. Although the upper soils were believed to be slightly cohesive, stability was checked by conservatively ignoring cohesion and conducting a Rankine-type sliding wedge block limit state analysis. The configuration of each wedge and resulting force diagram is given in the figure. Through this type of analysis, the required minimum resistance of the tie-back anchors and the downward component of force on the push piers could be determined per lineal foot of excavation. The highest downward component of force exerted on the push piers occurs after excavation for the third phase of construction and prior to installation of the bottom layer of reinforced shotcrete.

The buckling resistance of the piers was also checked. The most critical buckling condition is the intermediate stage between phases two and three of construction when the push piers are laterally unsupported between the bottom row of tie-back anchors and the bottom of the excavation. When one examines the probable mode of buckling of the piers at this location, it can be seen that the unsupported length of pier shaft can move laterally away from the excavation. The pier shaft will be restrained from movement by embedment above and below this length. A best-fit Euler wave equation is one that is fixed on both ends with inflection points near the top and bottom of the unsupported length. This is mathematically equivalent to an effective column length equal to the depth of the unrestrained section of excavation. A conventional check of this buckling problem revealed that it was necessary to completely fill each push pier shaft with non-shrink cement grout.

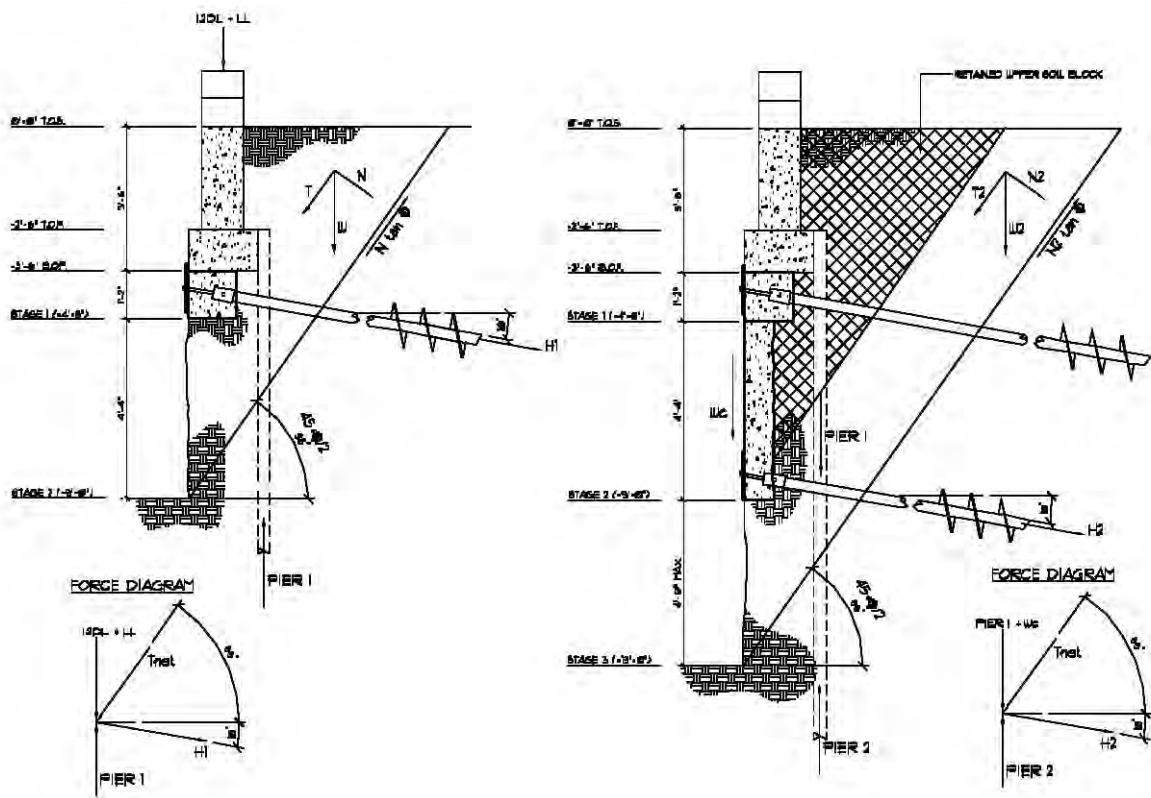


Figure 4: Excavation Stability during Construction

The stability of the final shoring configuration was checked along each side of the excavation using a simple finite element plate analysis. A trapezoidal distribution based on Peck's apparent earth pressure was input over the wall area. The most effective arrangement of tiebacks was determined by trial-and-error using the required anchoring resistance determined through static analysis as a guide. An example of the final configuration of tiebacks and resulting stresses within the shotcrete wall is shown in Figure 5. The section of wall shown is intersected by section A/S1 in Figure 3.

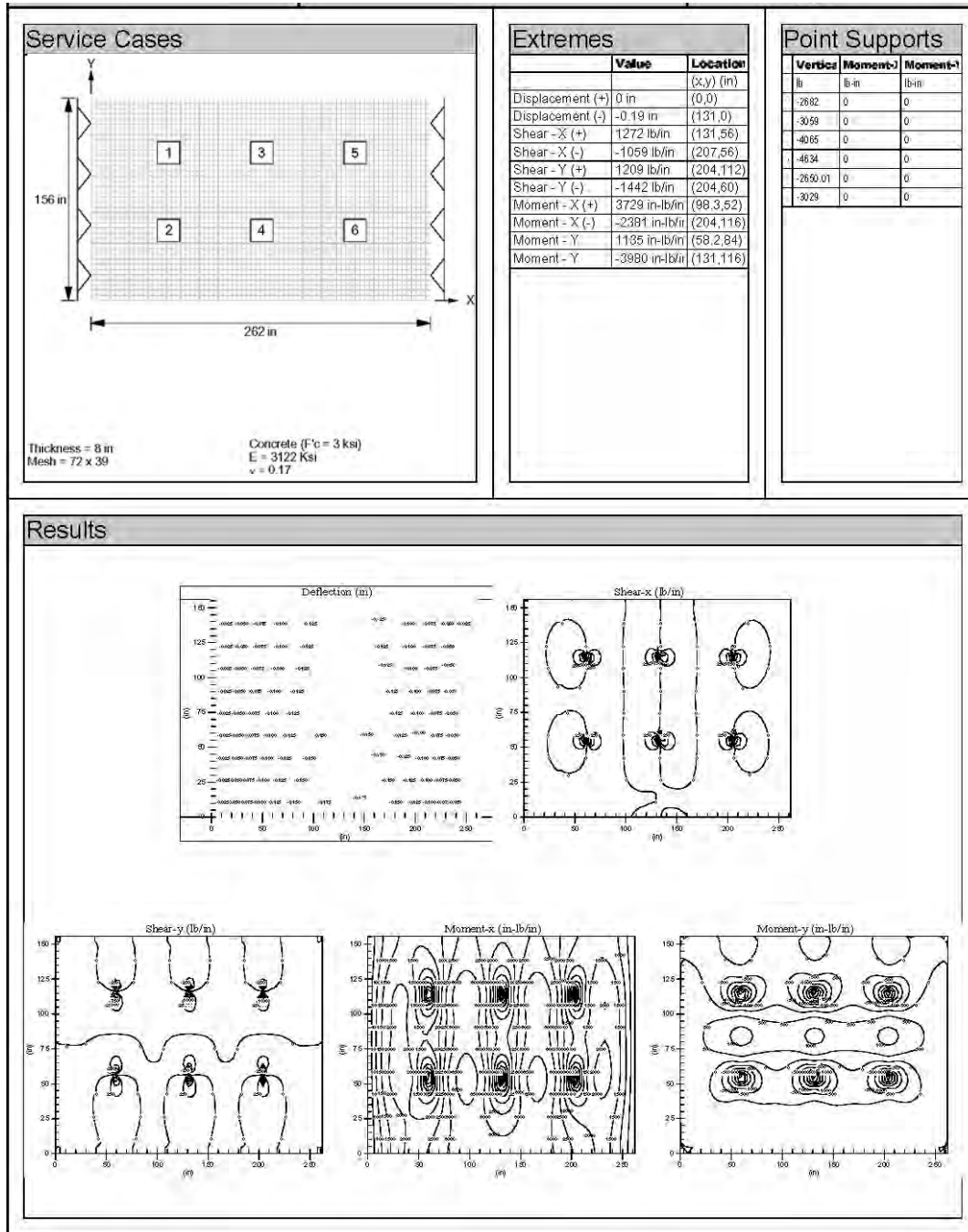


Figure 5: Example Finite Element Analysis of Shoring Wall

As can be seen in Figure 5, the boundary conditions used in the analysis included pinned edge conditions at the corners of the wall and free end conditions at the top and bottom of the wall. Experience modeling foundation walls in this way indicates that a more economical reinforcing steel design can be obtained by allowing the corners to crack and exhibit pinned behavior without aesthetically displeasing or structurally problematic results. Wall thickness was set at 8 inches, which is the maximum thickness that the contractor believed could be applied in one application. The contractor desired to utilize helical tieback anchors, because they could be installed using the same machinery that was used to make the excavation, namely a small, articulated track-hoe. Standard-duty round-shaft helical anchors with a maximum working capacity of 32 kips as manufactured by Magnum Piering, Inc. were selected as the helical anchor of choice due to their unique cutting blades and capability to penetrate glacial till with gravel and possible cobbles. In the finite element program, helical tieback anchors were modeled as spring supports with a spring constant of 32 kips/inch. This spring constant was used based on the author's experience with pullout tests on anchors of this type in similar soil conditions.

The highest shear force and maximum flexural moment in the shoring wall occurred near the tieback anchors and was similar in both the horizontal and vertical directions. It should be noted that the program used for the analysis produces misleading results in that the Moment-x is actually the moment about the vertical axis, which governs the horizontal reinforcing in the wall. The opposite is true for the results given by the software in terms of Moment-y. It was found that applying a moderate post-tension load to each anchor could reduce wall deflections and stresses within the wall significantly. Post-tension force was modeled in the finite element program by applying an equivalent point load at each tieback location. For this project, a post-tension force of 15 kips was applied to each anchor. This is the force that was determined could be applied fairly reliably by hand using an impact wrench.

The required reinforcing steel within the shotcrete wall was determined using conventional design procedures as outlined by ACI 318. A combination of heavy gauge welded wire fabric and No. 5 reinforcing steel bars was selected. The heavy gauge wire can be molded to the configuration of the excavation and is easier to keep stationary during the shotcrete process. The No. 5 reinforcing steel bars are small enough where they can be cut by hand in the field to accommodate the randomness of an excavation. They also can be driven more easily into the ground to provide bond steel for subsequent phases of excavation and shotcrete placement.

## **V CONSTRUCTION SEQUENCE**

Construction began by excavation along the existing foundations on both the interior and exterior of the building. Steel angle brackets were attached to the outside of footings in accordance with the plan. Steel piers were hydraulically forced into the soil at the bracket locations. The force applied to each push pier as a function of depth is given in Figure 6. All piers were installed individually to the design pressure and a load test was conducted at each location. The majority of push piers were installed to a depth of 18 feet below the existing footing. A photograph of push pier installation is shown in Figure 7. As can be seen, the equipment used to perform this type of work is ultra-portable. Main structural components of the ram frames are made of aluminum. A single worker can lift, haul, and place a ram assembly. A small electric pump with 2-gallon reservoir provides hydraulic pressure. After load testing, the piers are unloaded and locked in-place with two  $\frac{3}{4}$ " diameter SAE grade 8 bolts that penetrate the pier shaft through the bracket sleeve. For this project, the pier shafts were checked for plumbness after installation and completely filled with cement grout. Buckling was ruled out by the absence of "spring back" (upward movement) of the pier shaft after removing the test load.

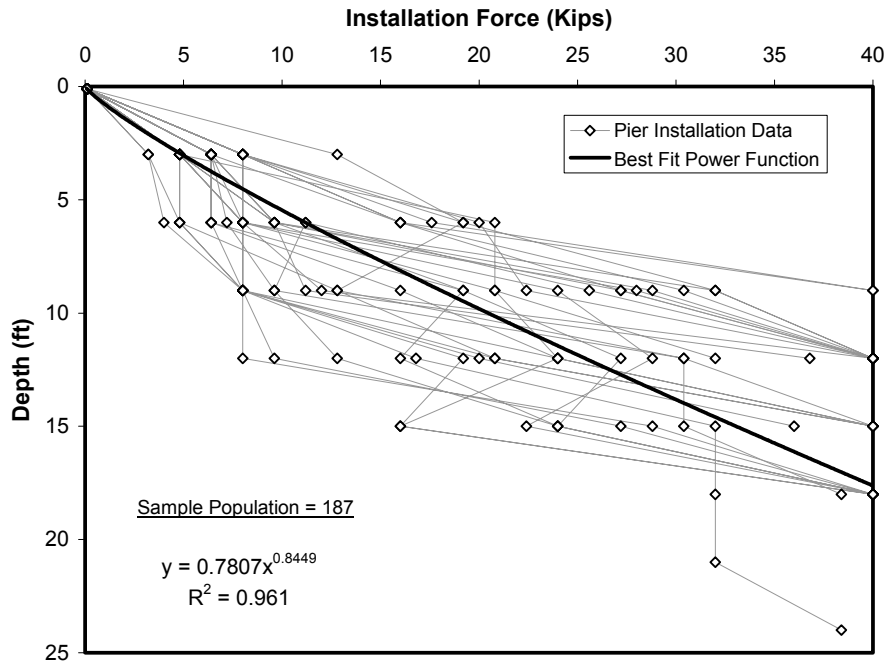


Figure 6: Push Pier Installation Data

After all piers were installed, small excavations were made under the footing at designated haunch locations. The contractor installed the top row of helical anchors at the haunch locations (Figure 8) and areas of the site that were not immediately adjacent existing foundations. Reinforcing steel was placed in the haunch form and concrete was cast about the anchor. A photograph a completed haunch is shown in Figure 9.



Figure 7: Push Pier Installation



Figure 8: Tieback Installation



After the haunches were allowed to cure, a steel plate, wedge washer, and hex nut was placed and fastened over each tieback anchor rod. Construction proceeded with the excavation of the site to a depth of roughly 4 feet 4 inches below the bottom of the existing footing foundation. The second row of tiebacks was placed at the base of the excavation according to the shoring plan. Reinforcing steel was placed over the vertical face of the excavation and the entire area received an 8-inch thick layer of shotcrete. An accelerant add-mixture was added to the mix design to ensure minimum 3,000 psi compressive strength in 48 hrs. Photographs of reinforcing steel placement and shotcrete activities are shown in Figures 10 through 12.



Figure 9. Completed Tieback Haunch



Figure 10. Reinforcing Steel Placement

The next step in construction involved excavation for the final phase. The photograph in Figure 13 shows some of the excavation activities. Similar to the prior phase, reinforcing steel was placed over the vertical excavation and the final portion of shotcrete being applied (see Figure 14). Photographs of the completed excavation are shown in Figures 15 and 16. After completing the work, the trackhoe was crane lifted from the excavation and the concrete contractor began building the new facility.

## VI CONCLUSIONS

The excavation was completed on time and on budget. Survey markers were placed about the perimeter of the existing building and at the top edge of the excavation. The contractor's quality assurance personnel were unable to detect any appreciable movement in the existing foundation or the top of the shoring wall during or after the work was completed. The shoring and excavation activities had minimal impact on the daily operations of adjacent areas of the medical research building. The total budget for the project was similar to other more conventional shoring alternatives used for larger more accessible excavations on a unit area basis for the shoring walls.



Figure 11. Shotcrete Operations



Figure 12. Shotcrete Application



Figure 13: Excavation of Final Phase



Figure 14: Final Shotcrete Application

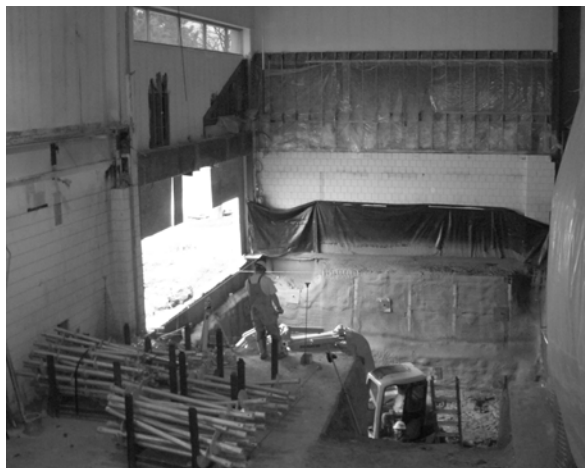


Figure 15. Context of Excavation



Figure 16. Completed Excavation