SUPPORT OF AN EXCAVATION FACE WITH MICROPILES

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Abstract

The use of micropiles to stiffen and support an open cut face in lieu of using conventional caissons or soldier piles with structural steel offers a technically sound method to construct excavations. This technique can solve limited access problems as well as provide load bearing and better face deflection control. Micropiles at the face can be used with soil nails to construct an excavation which has to be adapted to difficult conditions as the construction progresses. This can provide additional lateral and vertical deflection control, particularly when used with post-tensioned soil nails. The design, use, and performance of a case history is presented to illustrate this approach and excavation design.

Introduction

Design of shoring systems for excavation support has evolved in the last fifty years. Sheet piling, soldier pile and secant walls have grown up, and in turn given way to soil nail construction where conditions allow. The increasing demand for more cost-effective methods has pushed the use of soil nailing for increasingly difficult applications. As the construction industry continues down this path, geo-structural designers and builders are asked to look more and more at soil as a scalpel instead of the blunt instrument it was considered to be fifty years ago.

Cost of construction often takes a second row seat to space requirements. Every available square foot in a structure footprint is now being used to generate revenue, and geo-structural designers are frequently presented with extremely narrow corridors to construct shoring systems. Often the constraints are increased by the presence of an existing structure bounding this narrow corridor. This problem often requires a structural face system to limit movements and yet does not allow installation of soldier piles, steel sheet piles, or any type of slurry wall.

Innovations in recent years help fill this technology gap. One approach is the use of micropiles, installed to provide a simple structural face, and several problems have been solved this way. Most applications of a micropiles face can be classified under at least one these scenarios:

- Limited access shoring zones which cannot be drilled with large equipment.
- Support of existing structures along the shoring line.
- Poor soil conditions which require face stability improvement for excavation.
- The need for a structural face stiffer to limit vertical deflection of a soil nail wall.

Figures 1 and 2, Face Support with Micropiles, present cross-sections of four projects which have been successfully constructed for each of these reasons.

Presently, there is no methodology developed or adapted to the design of a composite system of soil nails and micropiles. Most of these applications have been done by designers simply applying good engineering judgement and fitting approximate methods of analysis to provide a sufficient “reality check” for the design. The purpose of this paper is to present observations made on different projects and to look specifically at a recent design and performance of a composite wall.

Micropiles as discussed here are micropiles constructed by drilling and grouting a reinforcing bar into the center of a drilled hole (center reinforced), and not a micropile constructed with driven casing (cased micropile).

Soil-Anchor Interaction

A group of anchors placed vertically as micropiles or fully grouted near-horizontal anchors placed as soil nails interact in the same fundamental manner, by soil arching. Essential to arching is the non-slip bond. Without this mechanism to ‘connect’ the elements, each one would be independent and unable to improve support characteristics of the surrounding soil.

Figure 3, Non-Slip Bond, shows an idealized cross-section of anchors and soil. This depicts the transfer of load from anchor to anchor and a simplified view of how the soil is strengthened by the anchors in the process. The non-slip bond of the soil particles against each reinforcing element is essential, as a boundary layer. If movement begins then the load has exceeded the composite-structure capacity and it will yield and fail.

There is a parallel application which gives an opportunity to see this interaction at work. Reinforced soil (MSE) depends wholly upon arching between horizontal reinforcing elements. One of several types of MSE reinforcement is welded wire mesh. High walls can require reinforcement lengths greater than can be manufactured or shipped, so the problem of splicing reinforcement has been addressed. An early method of splicing steel mesh reinforcement used overlapping sections of mesh so that the transverse wires would bear directly on each other. However direct bearing does not allow for the reality of uneven wire alignment and high stress points which risk localized yield of either the wire or welds.

Another approach to splicing is to make the transfer entirely by soil arching. This is done by overlapping the welded wire mats a length which is sufficient for full pullout development (about two meters / six feet) plus some safety factor (one meter / three feet, or so). The mats can either lay directly on top of each other or be separated by soil. Soil is compacted around the reinforcing elements and as the wall is built, the mats engage the soil and the load transfer from segment to segment is done through the arching of the soil from mat to mat.

This concept was field verified with two large MSE walls constructed to a height of 38.4 meters (126 feet) using welded wire mesh. The upper half required reinforcing mats which were 27.5 meters (90 feet) long. It was not possible to ship mats longer than 15.25 meters (50 feet), so the total reinforcement length was obtained by overlapping two 15.25 meter (50 feet) mats a total of three meters (ten feet). This provided the correct total reinforcement length of 27.5 meters (90 feet). Strain gages and pressure gages measured load transfer at selected locations and found this connection to have an efficiency well over ninety percent (1). The vendor/wall manufacturer has since switched to the use of this overlap splice for all wall construction, with two and one-half meters (eight feet) being the standard overlap. A small project to check this concept by centrifuge testing has begun, and preliminary results confirm the empirical results.

An issue with drilling and placement of micropiles and soil nails is the need to keep the soil particles engaged insofar as possible during placement. Then the soil arching continues with minimal opportunity to lose confinement around the reinforcing elements, and subsequent debilitating displacements. Any yield of the soil must be countered with displacement to re-engage the soil particles in intimate particle to particle contact necessary for arching to take place. In practice, this can be accomplished by casing drilled and grouted (especially post-grouted) anchors or with hollow-bar anchors which are continuously grouted during drilling.

Reinforcement spacing should be also considered. The question is, how far apart can elements be placed and still have load transfer by arching. One possible indication is the performance of a shoring system as a soil nail wall with a structural face of soldier pile and lagging. The shored depth typically ranged from 18 to 19 meters (60 to 65 feet), while the soil nails ranged from12 meters (38 feet) to 14 meters (46 feet) in length. The nails were constructed with hollow-bar anchors, and were fully grouted. Inclinometer measurements disclosed a maximum lateral deflection of 12 mm (1/2-inch) for the 19 meter (65 feet) height segment, and 3 mm (0.1 inches) for a short segment which was 7.5 meters (25 feet) deep and carried one side of a ten-story historical building. The movements were very slight throughout with a few paper-width cracks appearing in isolated places behind the wall. In general it appeared to behave as a contiguous mass of reinforced soil, much as an MSE wall. The horizontal and vertical soil nail spacing was 2.6 meters (8 feet) and 3 meters (10 feet), respectively (2).

The lateral load was back-calculated by the deformation of the structural steel beams in the soldier piles. The load carried by the soldier pile face was estimated to be between one-third to one-half of the load estimated by conventional shoring analyses. This is also a consequence of the arching, with the composite soil and soil nails taking the remaining load. This is much like an MSE wall, which has negligible face stress with closely spaced reinforcing. As the spacing grows, so does the face stress. This is also consistent with typical face load reductions for soil nail design, in which the face stresses are estimated to be 40 to 60 percent of the maximum pressure, depending on spacing and soil type.

One way of considering reinforcement spacing necessary to still achieve arching is to normalize the spacing into an aspect ratio. In this case, the aspect ratio would be the length of reinforcing to the distance apart. In order to have efficient arching occur, GDSI has observed that an aspect ratio of three is not an unreasonable minimum. This is, however, strictly intuitive and will require instrumented projects to verify the concept as the engineering of composite structures is refined. It is important to realize that this aspect ratio is for ideal conditions. These conditions would be defined as dense granular soil with hollow-bar anchors (or casing-drilled anchors of solid bar, preferably post-grouted). More fine-grained soils with anchors less inclined to limit yield (such as open hole, grouted nails) will not perform as well.

Composite Structure Design

There are a number of analyses which are common to any type of earth retaining system. GDSI normally sizes any lateral reinforcement (such as soil nails) with basic lateral earth pressure theory, mostly using Terzaghi's apparent pressure formula, as follows:

$$P_{max} = 0.65\ (k_a)\ H$$

The top and bottom 20 percent are truncated where appropriate and global stability analyses and soil nail structural programs (such as Goldnail or SnailWin, etc.) are used to refine the design.

Inserting vertical structural elements like micropiles into the structure cannot be conveniently done with any of these analyses. Some global stability analyses have limited capability for vertical pier elements (such as SLOPEW by GeoSlope International), but these should not be used for primary element design, only for overall stability, and then with some conservatism in shear resistance values (3).

The use of micropiles at or near the face of an excavation provides the opportunity to design for any of several possible requirements, which include:

- Vertical load from an existing or planned structure.
- Limited lateral load, provided the load is either in the plane of the wall or there are micropiles paired up as "A" frames to carry the load in tension and compression.
- Uplift load.
- Provide additional flexural stiffness to the wall face. This can be done best with multiple rows of micropiles placed behind the face, to the depth required. The purpose would be to control deflection and provide stability at the face.

Conventional analyses for sizing micropiles to provide axial load capacity do not address any type of flexural load capacity. Taken as an individual element, a center reinforced micropile has an extremely low moment of inertia. Very little stiffness can be added with a single element, considering the preponderance of soil which surrounds it, unless a micropile constructed with driven casing is used. However, using center reinforced micropiles placed in pairs as a front row and a back row, gains some advantage through soil-structure interaction.

A further distinction can be made here between vertical soil nails and micropiles. Micropiles are usually intended to carry some type of structural load, as outlined above. Vertical soil nails, as we have come to define them, are usually constructed with lighter weight reinforcement and employed primarily to enhance face stability. They are sometimes incorporated into the back of the shotcrete face and usually placed close enough (0.6 meters / 2 feet, or less) to support layers of loose or soft material during excavation, anchor installation and shotcrete placement. It is possible to combine the two by placing the front row of micropiles very close to the face at close enough spacing for face stability, but combined loading should be factored into element design.

A simple approach to a design methodology for flexural stiffness is to use two rows of center reinforced micropiles. They may be parallel, or allowed to spread apart with depth, beginning with a separation of 0.3 to 0.6 meters (1 to 2 feet). A structural beam may be considered to form with this configuration. The flanges are represented respectively by the front and back rows of micropiles, while the web is the soil which arches between them, transferring the load. The distance apart provides a lever arm which can be used to calculate a section modulus. Points of support can be provided by soil nails crossing the micropiles at design intervals. A hard connection can be made (if needed) as the soil nails are installed, by drilling them close enough for the nail grout to mix with the micropile grout mass.

This method is shown on Figure 4, Micropile Stiffness Design. It should be used with a significant factor of safety, at least 3.0, as this is a very approximate approach for determining face stiffness. The high factor of safety is to account for poor load transfer at the soil nail points and general inefficiency of arching between horizontal and vertical elements. There has been no comprehensive study to show that this method actually describes what the micropiles do or that any stiffness is actually imparted to the soil face by micropiles. We have used it as a starting point to approach the problem with a simple limit equilibrium analysis. If these micropiles are intended to also carry a significant vertical load, the micropiles must be designed for the combined loading. This can be accomplished by breaking down the flexural load into tension and compression components and combining them to the vertical load. Once again, significant conservatism is necessary.

One aspect of micropile design becomes more important when designing a micropile installed near the excavation face to carry significant vertical load. Load shedding and the question of how close to the face a micropile can be placed without affecting the face stresses is significant and has not really been addressed. As a general micropiles carrying significant loads should be designed to pass them through the wall itself and to the base of the excavation, on down. Sufficient capacity should be developed at depth for this purpose.

Case History: Description

GDSI and Malcolm Drilling Company Inc (MDCI) have completed the construction of a composite wall to a depth of 13 meters (42 feet) below foundation grade, which was in turn 6.5 meters (21 feet) below street grade. The structure to be supported was a six story settlement sensitive building occupied with extensive electronics equipment. There was a very narrow zone allotted for the construction of this shoring system. The face of shoring was required to be 0.61 meters (2 feet) from the building wall. The structure design surcharge was 182.7 kPa (3,815 psf).

The closest borings at the time of design showed that the subgrade soils were a conglomeration of lakebed and stream deposited soil. At foundation grade, the soil encountered was a gravelly sand with some silt, which extended to a depth of about five meters (16 feet) below the building. This soil was medium dense to dense and exhibited high strength and low compressibility characteristics. Beneath this soil, a layered, stiff to very stiff silty clay with some zones of silty sand was logged to a depth of about seven meters (23 feet) beneath the base of the excavation. This material exhibited moderate strength and moderate compressibility characteristics. The ratio of horizontal to vertical permeability is typically at least 10 with the sand and sandy silt layers which are apparent at different levels.

Groundwater was recorded to be at a depth of about five meters below foundation grade. This is more or less coincident with the interface of the clayey soil. Groundwater flow was upgradient from the wall. At the west end of the wall is the corner of the existing building. The shoring turns south to support the west side of the building. The east end of this wall segment turns a corner to go north for a short length, into a three-sided alcove. The general plan layout of the wall for the Case History Wall is shown on Figure 5 - Case History Project Plan View. Wall A is the adjacent wall to the east, Wall B is the Case History Wall and Wall C is the adjacent wall to the west.

During the project bidding, GDSI participated with Malcolm Drilling Company in performing extensive pre-bid design studies and layouts. As part of this work, we determined a preliminary shoring wall design which was a composite wall, using micropiles to stiffen the face and carry some of the vertical load of the perimeter footing for the building on top. The excavation would proceed from the top down, with soil nails which were post-tensioned. The face of the shoring was required to be smooth to allow for forming against the shoring for the permanent structures. There was no room for the installation of any other shoring system at this location, because of the nominal six-inch shotcrete thickness and the close proximity of the building which precluded the use of large drilling equipment. Based upon our preliminary studies, the agreed upon standard of performance for the shoring system was that total settlement would not exceed 13mm (1/2-inch).

The length of this shoring segment was 21 meters (70 feet) from corner to corner. Around the corner on the east end (Wall A), a wholly different design was used and there was no structure supported on top. At the west corner, the shoring turned south (Wall C). This wall was constructed in a similar manner, but stepped up to a higher level within about 13 meters (42 feet). Wall C was able to incorporate some of the original driven piles for the former structure which had occupied the excavation area. On Wall B (Case History Wall), the use of existing piling was precluded because these piles were set in front of the contract face of shoring.

Case History Design

The design of the shoring for this segment of the project is presented on Figure 6 - Case History Typical Cross-Section. A plan view for the various elements is presented in Figure 7 - Shoring Layout to detail the element layout.

The cross-section was sized initially by lateral pressure theory. SnailWin (a 2D limit equilibrium program for soil nail analysis/design developed by CALTRANS) was used to finalize the nails. SlopeW (by GeoSlope International) was used to evaluate global factor of safety scenarios (1.4 for the static, operating case).

To stabilize and stiffen the face, a pair of micropiles were placed at 0.91 meter intervals (3 feet). These micropiles considered to add some flexural stiffness and overall deflection control to the wall face. They were designed in two steps. An initial soldier pile and tieback anchor shoring analysis was run on the wall face, using the projected soil nail spacing of 1.8 meter (6 feet) horizontal by 1.5 meter (5 feet) vertical as the spacing for the soldier piles and anchors, respectively. This was done to determine the required section modulus per unit wall length for a soldier pile / tieback anchored wall. Because this will be a soil nail supported structure, the section modulus was reduced by half, assuming this structure will behave as other soil nail structures with face load at one-half or less of the design for a tieback wall system.

Because of the short vertical spacings, the required section modulus was relatively low. The actual values were 75 cubic centimeters/meter (1.39 cubic inches/foot) at the top, increasing constantly to 225 cubic centimeters/meter (4.2 cubic inches/foot) at the base. These calculations were made for structural steel having the same yield value as the micropile steel reinforcing, which was approximately 517.1 mPa (Grade 75 steel). All of the calculations were based on allowable strength and not yield strength, which gives additional factor of safety.

The available section modulus was calculated using the micropile center reinforcing bars, according to the methodology outlined previously. Calculations were based on the reinforcing steel, without the grout casing. The calculated values are presented in the following table:

Span	Required Sx	Available Sx	Factor of Safety
First Span (top)	75 cubic centimeters/meter (1.39 cubic inches/foot)	427.3 cubic centimeters/meter (7.95 cubic inches/foot)	5
Bottom Span	225 cubic centimeters/meter (4.2 cubic inches/foot)	3244 cubic centimeters/meter (60.4 cubic inches/foot)	15

Another measure which was included in the design to help control settlement was to post-tension each level of soil nails against the shotcrete face following installation. Some of the inherent deflection of the nails is taken out by this process, which improves the overall performance.

An additional micropile was placed immediately in front of the building footing. While drilling this micropile, a grout mass was built up around the top of the pile underneath the base of the footing to provide some load support and consequent deflection control. This micropile was not extended to a depth beneath the excavation because it was felt that such a design would make an undesirable “hard spot” for the footing. Instead it was allowed to ride with the wall while dissipating some of the load through the reinforced soil mass down to the base of the excavation.

A dewatering scheme was used along this length of shoring. This included two tiers of vacuum wells set at 1.6 meter (5 foot) intervals drilled in about 2.5m and 6.7m below top of wall, respectively. The dewatering wells were drilled in from the face to a depth of about 6.5 meters, at an angle of about 20 degrees up from the vertical. Each well was sealed off about a meter below the top; below that level the wells were screened and packed in sand. Each level drained its respective wells into a pipe manifold which was piped offsite.

The intention of the dewatering scheme was to control groundwater by keeping it off the excavation face and out of any zones of sloughing soil encountered below the groundwater level at the excavation face. At the same time it was important to avoid consolidation settlement induced by dewatering. This was the basis for design of the wells so close to the face, so that there would be minimal dewatering at depth in from the face.

As the construction of the shoring along Wall B began, MDCI added a grade beam all along the top of the wall and the top of the adjoining segment to improve fixity of all the elements drilled in from the top. The reinforced grade beam was poured to a depth of about 0.6 meters (2 feet).

Case History Construction and Performance

Different types of instrumentation were used at throughout the project to monitor and measure shoring performance. An inclinometer casing was placed at the west end of Wall B, right at the northwest corner of the building support by the excavation in this area. There were also targets for high resolution surveying placed on this wall and on the adjacent walls. Vibrating wire piezometers were installed at several locations and depths to monitor dewatering.

After construction began, additional borings were drilled near this wall segment. The original borings which were the basis for the design were installed around the perimeter of the project because there was no interior access. After demolition additional borings were advanced to clarify the subgrade conditions in several areas of the project.

The nearest borings showed that the subgrade soils included a zone of layered silty sand/sandy silt. This was non-plastic soil which is loose to medium dense in consistency. The natural water content is typically 20 to 30 percent (saturated). Layering is typically one to two centimeters (3/8-inch to 3/4-inch) thick. This zone exhibits a horizontal to vertical permeability ratio of at least ten or more, and is generally a moderate to moderately low strength and medium compressibility material. This silt / sand zone was typically six to seven meters thick, extending to a depth of about 1.6 meters (five feet) above the base of the excavation, roughly 4 m to 5 m (13 to 16 feet) below top of wall and down to a depth of about 1.5m above the bottom. Beneath this soil, the layered, stiff to very stiff silty clay was encountered to well below the base of the excavation.

This softer / looser material meant that excavation face stability would be more problematic and that the expected soil stiffness would not be present over much of the excavation face. The excavation was deep enough by then that design changes were more difficult to implement.

Most of the groundwater flow was apparently perched on top of the silt / sand zone of material and was effectively managed by the well point system. However the sand / silt was saturated and water seeped from intermittent sand layers. Groundwater began seeping through the shotcrete walls in the alcove area. An additional row of dewatering wells was added but the silty soils remained saturated.

Originally, the shoring was not designed to support hydrostatic pressure. Groundwater and soil conditions observed served as the basis to modify the design to support hydrostatic pressure at the excavation face below a depth of 6 meters. This design change included the addition of more soil nails and thicker shotcrete to the depth of the excavation. The shoring wall was then completed to design depth within a period of about four weeks.

The performance of the wall during the excavation is shown by the inclinometer record of the casing at the west end of the Case History Wall. This is presented on Figure 8, Inclinometer Results. This is a cumulative displacement plot, with the origin of the plot being set 4.6 meters (15 feet) below the bottom of the excavation). It is important to make this distinction, as typical cumulative displacement plots showing movement at depth show the inclinometer trace going vertical from the level where movement occurs.

Outward movement began about a month after the baseline readings were taken as the excavation started. There is an abrupt outward movement between August 14 and August 22, 2008 of nearly 8 mm (0.3 inches), coincident with commencement of dewatering. This movement did not have a corresponding change show up in the daily high resolution survey measurements. This particular kind of movement occurred in other inclinometer casings on the project when dewatering operations were turned on. This movement appears to have been a slight movement of the casing which was being subjected to an increased downward load, probably induced by the additional weight of the dewatered soil. The casing 'bulged' outward in the soft soil, making a slight adjustment in the direction of least resistance.

Also apparent is the impact of the post-tensioned soil nails. In this project, some of the soil nails were lightly post-tensioned by loading them to 89 kN (20 kips) against the shotcrete face following installation. The purpose was to limit deflections. The results were noticeable in the inclinometer measurements. Instead of the inclinometer trace going vertical from the level of disturbance, the trace returns back towards the baseline, indicating slight inward movement which was probably caused by the post-tensioned soil nails.

From this first 'adjustment' of the casing, the bottom half of the excavation was completed (between early November and early December, 2008). During this time, the limited access area of the alcove brimmed with intense construction activity and very wet weather conditions. Access to the inclinometer was not possible (a man-lift was required to access the casing) until the wall was completed in early December. Traces made on December 4, 2008 and since are comparatively consistent and show outward movement of about 9mm (0.35 inches). There were also comparable movements to the west, as shown by this same casing.

This inclinometer record and the overall performance shows that the combination of the post-tensioned soil nails and the micropile rows at the face maintained sufficient stiffness to perform satisfactorily, even with conditions much worse than expected. While some minor face sloughing occurred during construction, once completed the wall has not deflected. Total settlement, as measured with targets set at the top of shoring wall for the high resolution survey was 13mm (0.50 inches), which was the original maximum limit set for the project performance.

At a few locations it was necessary to cut one of the micropiles installed at the face of the wall to accommodate the supplemental dewatering wells. Almost immediately a single fine crack appeared on the face of the shotcrete over the micropile alignment. These cracks were vertical and extended up to the top of the wall. This event appeared to signal a release of tension in the outer micropile and transfer of that load to the shotcrete face. These micropiles were reconnected after drilling using couplers. The face was repaired and crack monitors did not show any additional widening of the cracks as the excavation was completed.

Photos of the Case History Wall are presented on Figures 9, 10, and 11 - Project Photographs.

Conclusions

The micropile supported excavation face for the Case History Wall was constructed under relatively difficult conditions and performed well. There was some outward bulging at the base during the construction of the bottom five meters (17 feet) which caused the total settlement to go to the maximum limits of the predicted performance of 13mm (0.50 inches). However the softer nature of the saturated silty sand / sandy silty soil was contained by good construction practice and the shoring system as designed. Considering the unexpectedly difficult conditions encountered in the excavation, the movements which happened were probably near the minimum of the movements which could have occurred. The overall settlement was limited to 0.1% of the wall height.

The micropile supported face is a viable approach to solving shoring problems fitting the criteria outlined at the beginning of this paper. When access is difficult and/or when applied loads or subsurface conditions demand additional stiffness for deflection control, this approach can be useful. Because the demand on geo-structure performance is increasing, designers are going to inevitably reach for this solution more frequently. Construction problems are no different from any other soil anchor or soil nail construction and this solution can be implemented with personnel already experienced in anchor and micropile construction.

The greatest need is to gather good empirical performance data so that a more direct design methodology can be documented. While a likely solution can be found by modeling the problem using the finite element method, what is really needed is a simple limit equilibrium method so that the complexities of FEM design do not have to enter in to simple projects. There are undoubtably many other approaches to this type of design, derived by the several engineers who have already used micropiles for face support. In order to take advantage of this construction technique on a broader scale, it is going to be necessary to more clearly understand the actual mechanics and have more well-established methodology.

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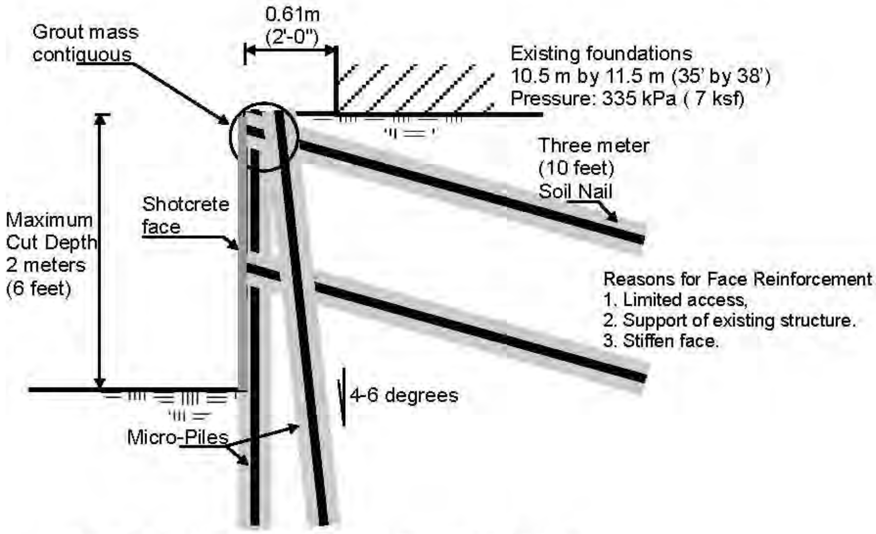

Figure 1a - Micropile Stiffened Face (2004-2006)

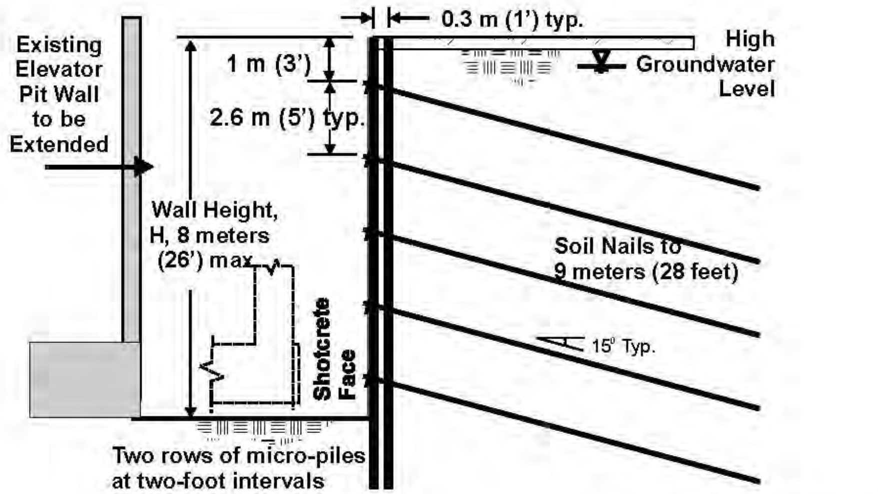

Figure 1b - Micropile Stiffened Face (2005)

Reasons for Face Reinforcement
1. Limited access,
2. Stabilize poor soils.
3. Stiffen face.

Figure 1 - Face Support With Micropiles - Begin

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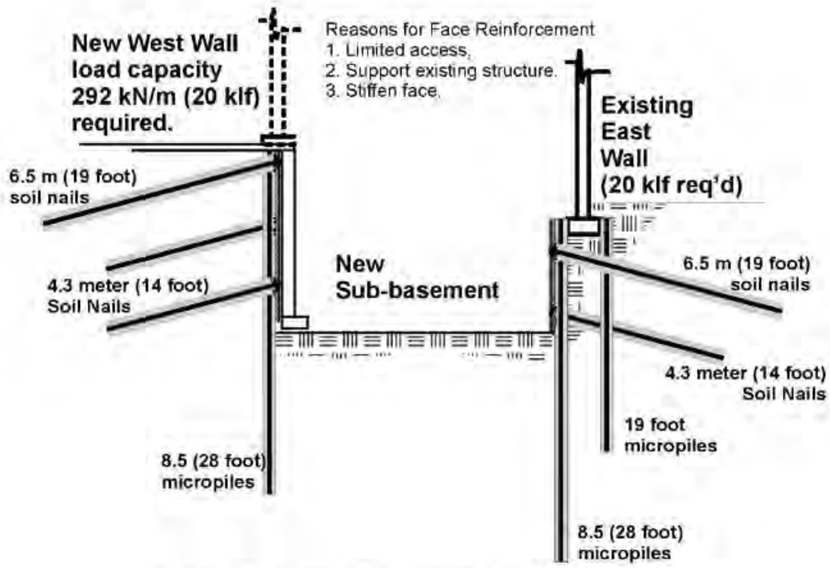

Figure 2a - Micropile Stiffened Face (2005)

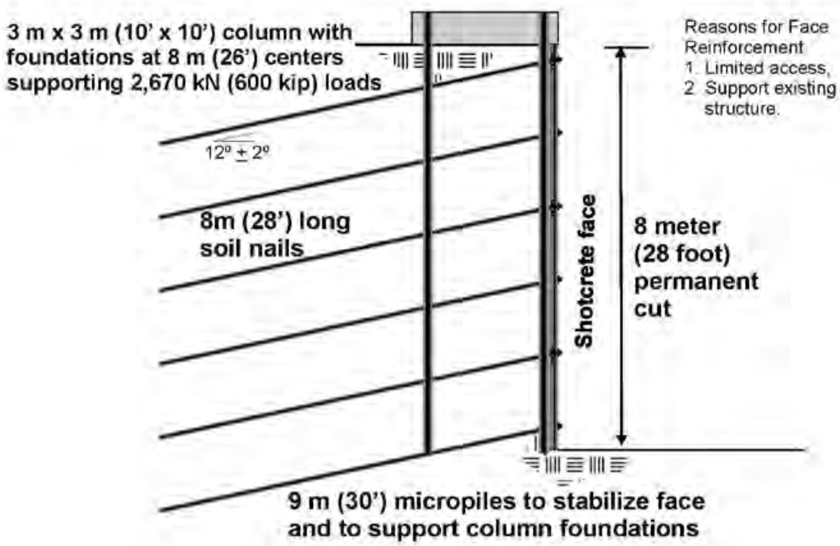

Figure 2b - Micropile Stiffened Face (2006)

Figure 2 - Face Support With Micropiles - End

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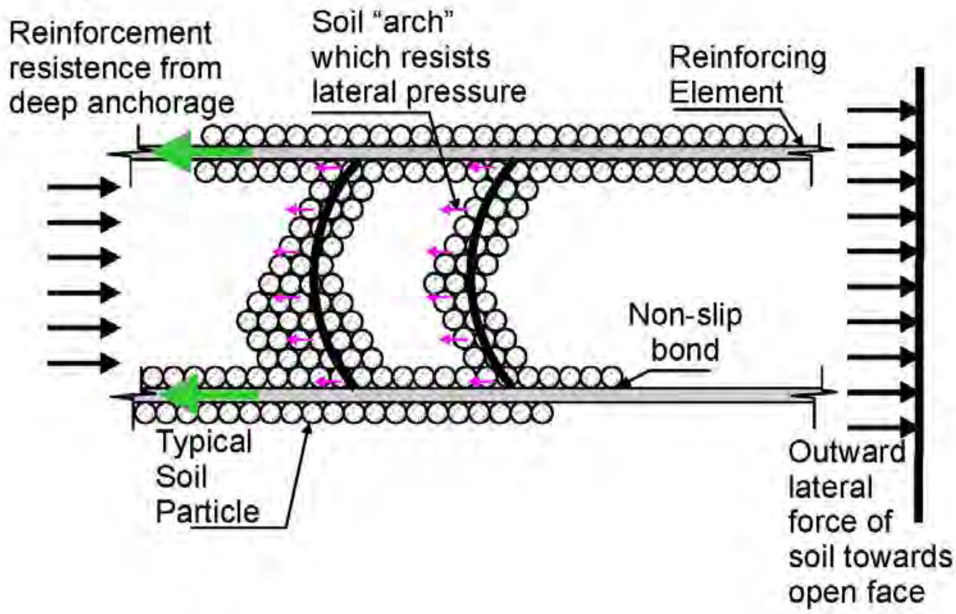

Arching = f (soil friction, aspect ratio, installation etc.)

Figure 3 - Non-Slip Bond

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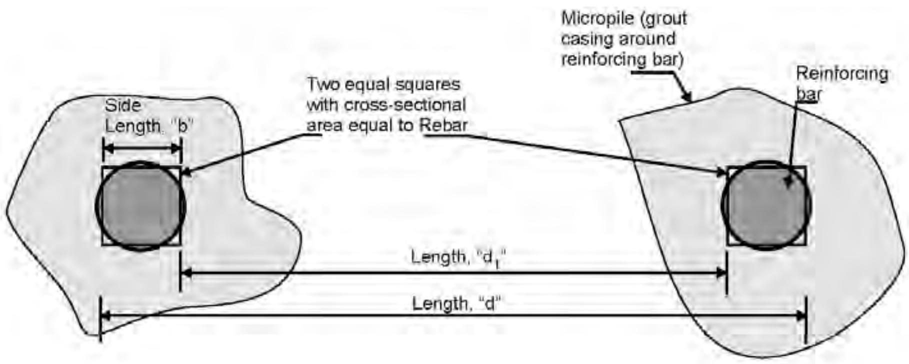

Section Modulus for Composite Structure of Two Equal Squares is equal to

$$S_x = \frac{b\,(d^3 - d_1^3)}{6\,d}$$

This provides a section modulus for a pair of micropiles separated by a design distance.

This value should be greater than the desired section modulus determined by conventional analysis (F.O.S. 3 or greater, with both *Sx* being determined for steel with the same yield strength).

Figure 4 - Micropile Stiffness Design

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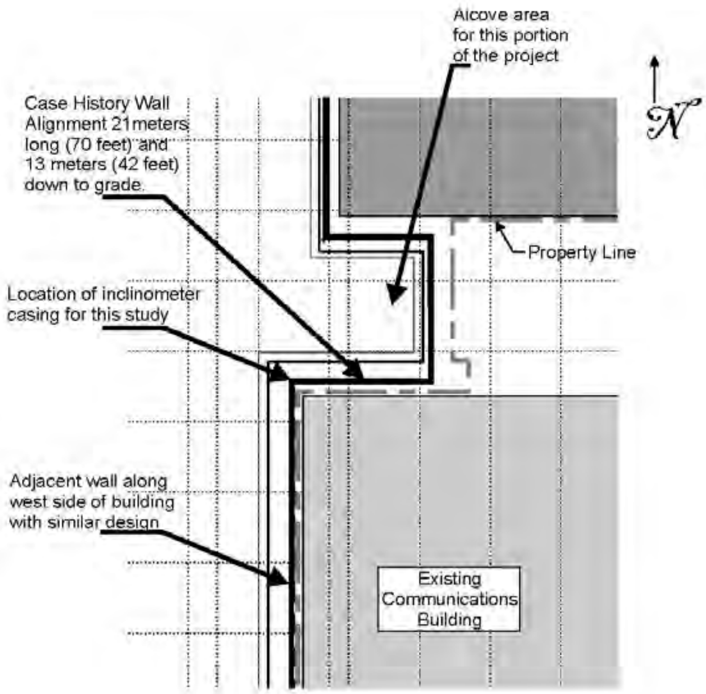

Figure 5 - Case History Project Plan View

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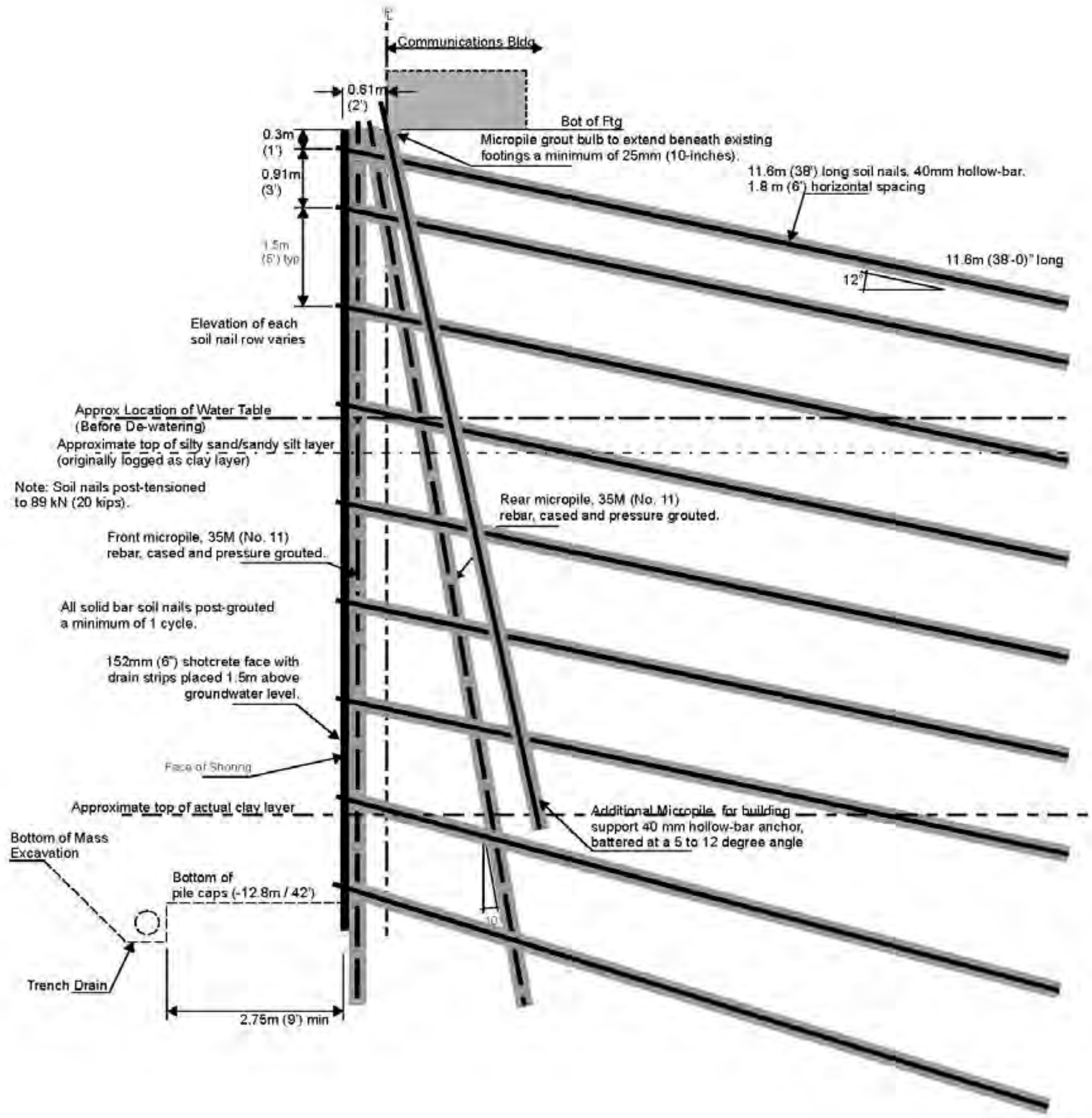

Figure 6 - Case History Typical Cross-Section

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MALCOLM DRILLING CO., INC.

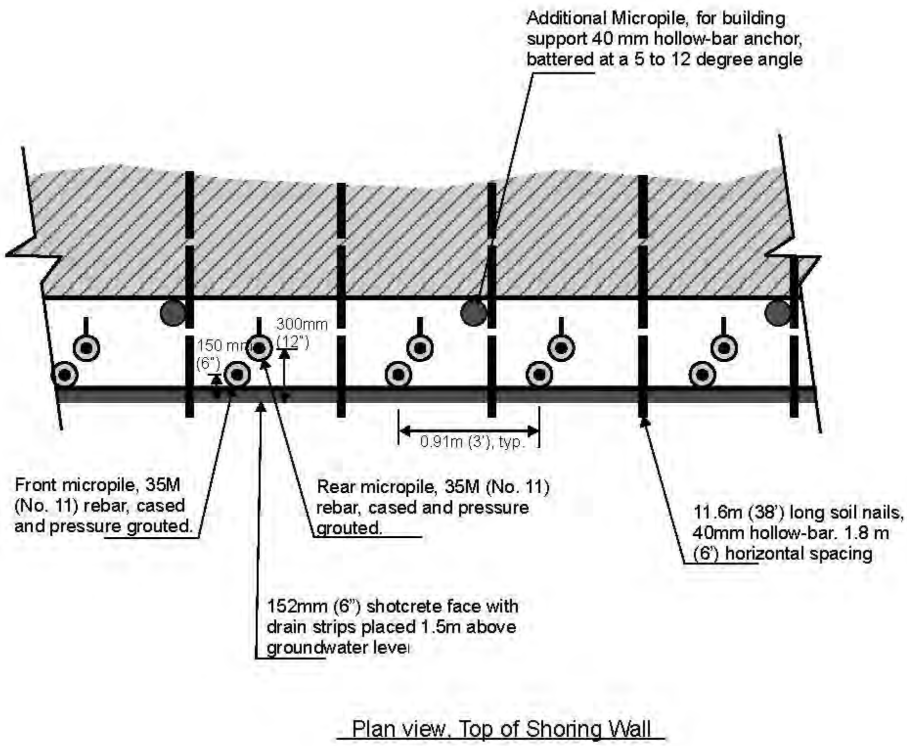

Figure 7 - Case History Shoring Layout

GEOTECHNICAL DESIGN SYSTEMS INC.

MALCOLM DRILLING CO., INC.

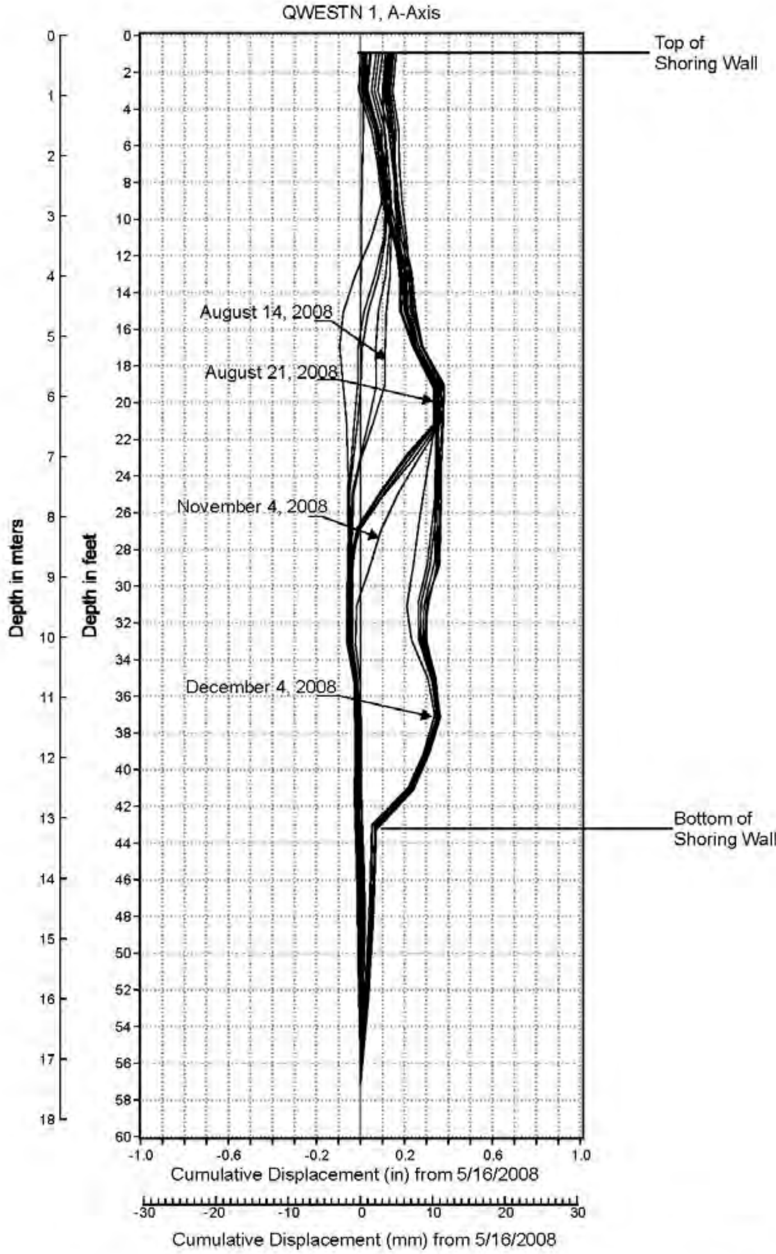

Figure 8 - Inclinometer Results
Case History Wall - Inclinometer Casing

GEOTECHNICAL DESIGN SYSTEMS INC.

MALCOLM DRILLING CO., INC.