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Design and Construction Considerations for Anchored Earth Retention Systems in Urban Environments

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ABSTRACT
Anchored earth retention (AER) wall systems have been used when an excavation is required to provide a grade separation and/or for the stabilization of slopes in urban environments. Tiedback walls and soil nailed walls are the most common AER wall systems utilized for support of excavation systems in urban construction environments. These wall systems have become increasingly cost-effective due to advancements in design methods, computational software, construction techniques and equipment, materials, and use of instrumentation. This paper discusses various design and construction considerations for AER wall systems and presents mini-case histories of different wall systems and their performance.

1. INTRODUCTION
Anchored earth retention (AER) systems are typically installed prior to commencing an excavation (i.e., cut situations), where vertical walls are constructed in a top-down manner as the in-situ ground is excavated on one side of the wall, ultimately resulting in a grade separation. AER systems are required because the native ground cannot be sloped back at an angle to facilitate the stable and safe deep excavation and subsequent construction of the proposed structure(s) because the site constraints or because the required easement is uneconomical to purchase.

An AER system can be constructed using discrete structural elements that are spanned by a structural facing, using individual elements that interlock or overlap each other to form a continuous wall, or using reinforcement elements installed into the in-situ soil mass that resists loading by the interaction between the soil and reinforcement. Common types of AER wall systems used in urban environments include soldier pile-and-lagging walls, sheetpile walls, tangent and secant bored pile walls, soil mixed walls, and diaphragm walls (Fig. 1).

The widespread use of these AER systems, especially for tall walls, is attributed to lower costs and/or expedited construction compared to older earth retaining wall systems (e.g., concrete gravity and cantilever walls). Technological advancements in computer software and hardware and in equipment and tooling has facilitated the growth of these techniques and their increased use and application.

Advantages of using AER systems include using top-down construction to provide both temporary and permanent support, which results in faster overall construction time and lower costs. Most of the available AER wall systems can be designed to provide support to vertical loading imposed

from buildings, superstructures, and bridge abutments, wingwalls, etc. Continuous wall systems can also be used to provide a seepage barrier. Lateral support of the wall is used when the exposed height approaches or exceeds about 3.7 m (12 ft) or when excessive horizontal deformation must be limited (Fig. 2).

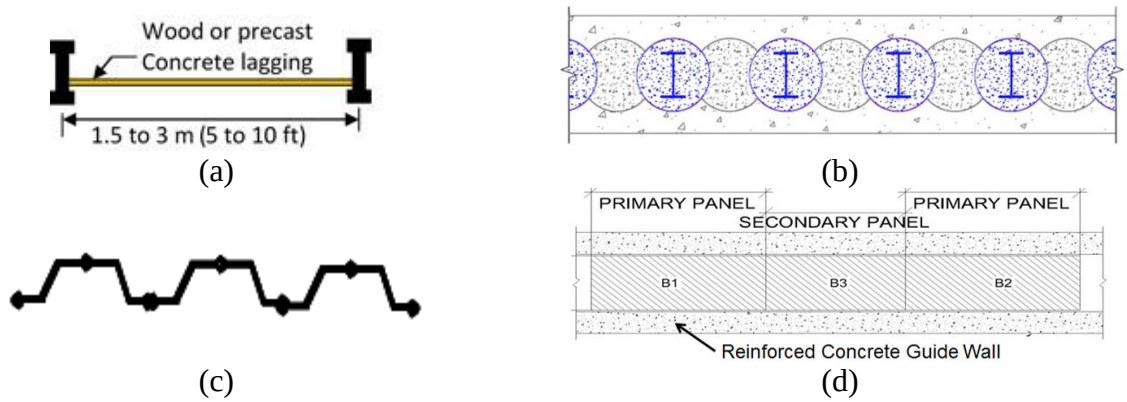


Fig. 1. Plan view of different types of AER wall systems: (a) soldier pile-and-lagging wall, (b) secant bored pile wall, (c) sheetpile wall, and (d) diaphragm wall (mod. after Tanyu et al, 2008)

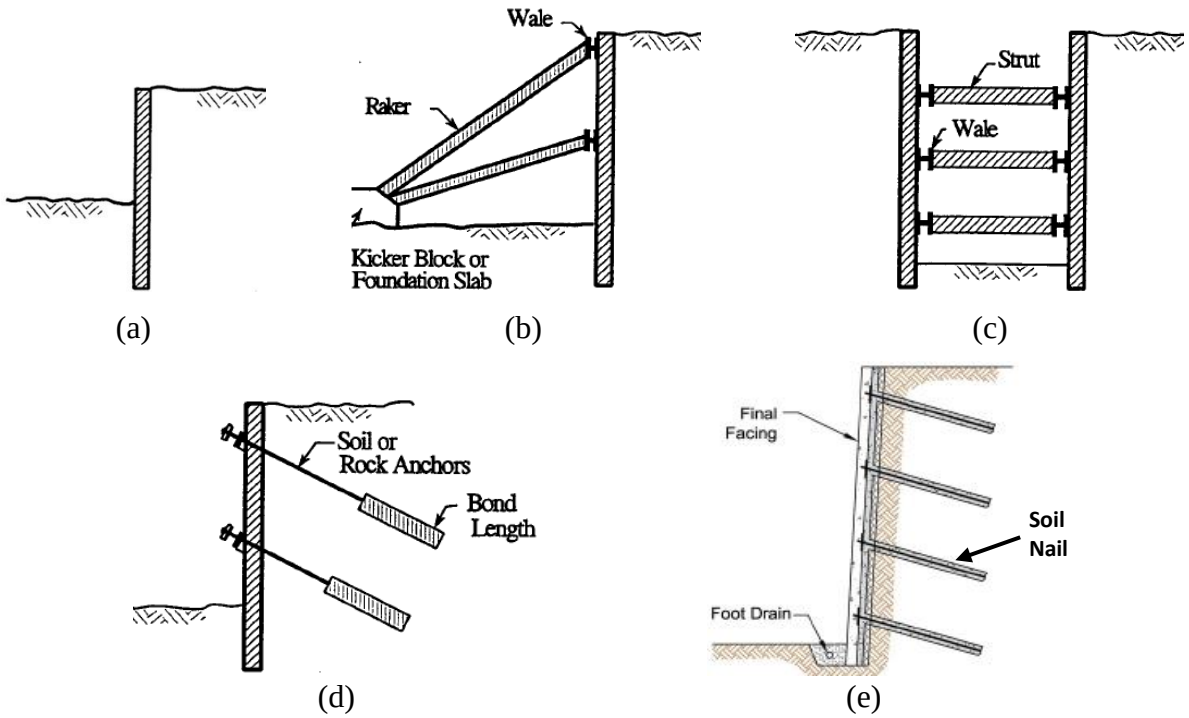
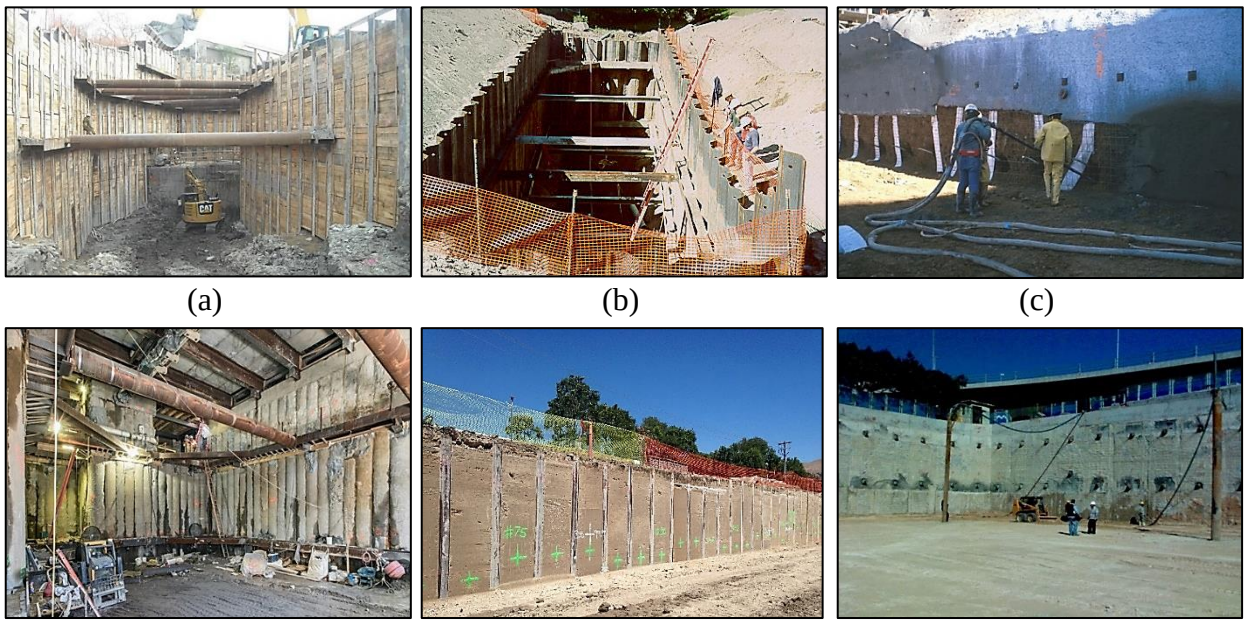


Fig. 2. Different types of AER systems: (a) cantilever wall (height ≤ 3.7 m (12 ft)), (b) internal raker bracing, (c) cross-lot bracing, (d) anchored wall, and (e) soil nail wall ([a-d] after Tanyu et al, 2008; [e] after Lazarte et al, 2015)

If the wall system is designed for a permanent application, the quantities of both excavation and backfill materials will be reduced in addition to a narrower work area required since a temporary wall and then a separate permanent wall will not be needed. In general, good quality workmanship

during construction of the wall system and its lateral support reduces both horizontal and vertical ground displacement and wall movement.

Depending on the required function and performance criteria, multiple types of wall systems are available for consideration. AERs are classified as externally or internally stabilized/supported cut walls, are relatively flexible, and are either temporary or permanent depending on the design life of the wall system. Some of the applications for AERSs in commercial and infrastructure projects include support of excavation for buildings and utilities; earth retention and/or structural support for bridge abutments, wing walls, and grade separations for new or widening of highways; watertight barriers for groundwater cutoff, flood walls, and cofferdams; and slope stabilization and landslide mitigation (Fig. 3).



(a) (b) (c)

(d) (e) (f)

Fig. 3. More common types of cut wall systems used in urban environments: (a) internally braced soldier pile-and-lagging wall, (b) internally braced steel sheetpile wall (for a deep utility installation), (c) temporary soil nailed wall, (d) internally braced and tiedback secant bored pile wall (for a deep subway station), (e) cantilever soil mixed wall, and (f) tiedback diaphragm wall

One of the main concerns with exposed and buried steel elements (i.e., ferrous metals) is corrosion; therefore, the corrosion susceptibility of the metallic components in the AER wall systems (e.g., vertical wall elements, bracing, ground anchor tendons, etc.) needs to be carefully evaluated and properly protected (e.g., use of epoxy coating, galvanization, encapsulation using corrugated plastics, and sacrificial cross-section of steel to account for future section loss through corrosion) to ensure the required performance of the structure throughout its expected design life is achieved. Compared to other types of earth retention systems (e.g., mechanically stabilized earth walls, reinforced concrete cantilever walls, etc.), a limitation of using AER systems is that the construction of these wall systems typically requires specialized construction techniques, skilled labor/operators, and sophisticated construction equipment. Correspondingly, the overall performance of the wall system and behavior of surrounding structures and ground will depend

on, among others, the robustness and accuracy of the design, quality of the workmanship, and type of method used to construct the wall, as each are integral to performance.

The following sections will present design and construction considerations for more common AER wall systems used in urban environments. In addition, the applicability, advantages, and limitations of as well as common concerns with each wall system will be discussed.

2. TYPES OF WALL SYSTEMS

2.1 SOLDIER PILE-AND-LAGGING WALLS

In dry conditions (i.e., absence of groundwater or seepage), a soldier pile-and-lagging wall may be applicable for use. Discreet vertical wall elements comprising a vertical steel section (e.g., H-pile, W-pile, double channels, etc.) are either driven into place or inserted into drilled holes, which are then filled with concrete. Between the vertical steel elements, lagging (e.g., wood boards, concrete panels, or shotcrete) is used to retain the in-situ soil. Wood lagging may be mounted either onto the face of the steel section (i.e., contact lagging, Fig. 4a) or placed behind the outer flange of the steel section (Fig. 4b). Face-mounted lagging is best suited for excavations in loose soil conditions. Depending on the final height of the wall, a single level or multiple levels of horizontal support may be needed for stability of the wall, which is provided by external bracing and/or ground anchors.

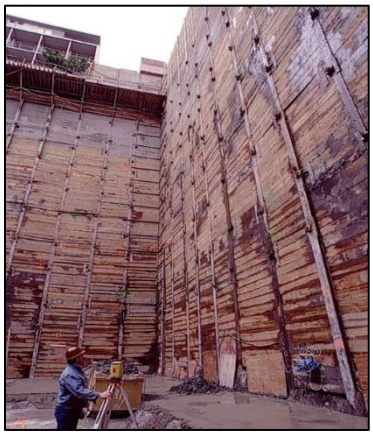
(a)  (b) 

Fig. 4. Examples of (a) contact wood lagging and (b) lagging placed behind the front flange (courtesy of Condon-Johnson & Assoc.)

Due to the discrete nature of the wall system, irregular geometries of the alignment and utilities crossing into or through the excavation can be accommodated relatively easily. Furthermore, the installation of the soldier piles and the wood lagging is relatively fast and quite economical, and the soldier piles can be installed in a wide range of geologic conditions (e.g., soft soils though hard soil/rock). In granular soils, the soldier piles are typically installed using vibratory or impact means. In harder soils and rock, the soldier piles are installed into bored holes and then filled with concrete. The installation produces a moderate amount of noise and vibration, though usually not excessive for an urban environment. A key benefit to this wall system is that the materials are generally available, and the soldier piles can be sized to resist high bending moments and axial loading and can be spliced, if needed, to accommodate the wall height. When incorporated into

the permanent wall, headed shear studs and one-sided formwork can be easily attached to the soldier pile face.

As the wall system is very permeable and free draining (i.e., water can escape through the lateral edges of each section of lagging), this wall system is not suitable when it is necessary to maintain the groundwater level behind the wall. When the groundwater table can be lowered, a dewatering system would be required during construction. Depending on the type and consistency of the in-situ soils, ground loss may occur from the time a lift of soil is excavated between the soldier piles and the lagging installed. Furthermore, as the soil is excavated between the soldier piles in order to install the lagging, the void space between the lagging and the in-situ soil must be filled or back-packed to limit the amount of settlement/movement of the ground behind the wall and at the surface. In addition, since the wall is free draining, it is important to ensure that the back-packing of fill soil does not impede the flow of groundwater through the lagging. As steel sections are used for the soldier piles, bracing, connections, etc., there is a concern with the corrosion susceptibility of the steel elements in direct contact with soil, thereby necessitating the possible need for periodic monitoring and maintenance and/or a mitigation plan. As this wall system is more flexible than other wall systems introduced below (e.g., diaphragm walls and tangent/secant pile walls), deflection of the wall may occur, which may result in excessive movement of the ground behind the wall system.

2.2 SHEETPILE WALLS

A sheetpile wall consists of a series of interlocking sheetpiles that are driven into the ground using vibratory or impact means to create a continuous vertical wall (Fig. 5). Steel is the most common material used for sheetpiling due to its inherent strength, but vinyl sheetpiling has also been used, especially in aggressive environments and areas of stray currents. Various manufacturers of sheetpiles have a wide range of sizes and shapes available to accommodate the loading imposed on the wall. Sheetpiling can accommodate a wide range of wall geometries (e.g., straight, circular, and irregular); however, utilities crossing into or through the excavation and wall can be accommodated only by cutting a hole in the sheetpiling. Depending on the final height of the wall, a single level or multiple levels of horizontal support may be needed for stability of the wall, which is provided by external bracing and/or ground anchors.

Sheetpile walls are relatively quick to install (i.e., economical) depending on the consistency and strength of the in-situ soil and can be installed in circular or box-shaped pattern to create a watertight wall. Unlike soldier beam-and-lagging walls, the facing of sheetpile walls is installed prior to excavation, which prevents loss and raveling of the retained soil thereby minimizing ground movements behind the wall. If the wall is temporary in nature and after any bracing/ground anchors are removed, sheetpiling can be removed if the sheetpiles were not installed into soils that were too stiff or dense or can be left in place but cut down to a depth below the ground surface.

The sheetpiles will not be able to penetrate hard or dense layers (layers with standard penetration test (SPT) N-values greater than about 35 blow/0.3 m) and/or may even be damaged during the process. In addition, the sheetpiles cannot penetrate hard obstructions, and will likely be damaged during the process. If sheetpiling is necessary, pre-drilling through the hard layers and/or

Fig 5. Permanent sheetpiling and ground anchors installed in dense sands and gravels in Queenstown, New Zealand (Mothersille, 2010)

obstructions may be required, which would reduce the productivity while increasing the cost of the work. In cohesionless soils, the installation of the sheetpiling using vibratory means will likely cause densification of the ground around the sheetpiling, which could result in some ground deformations near/at the ground surface. Furthermore, vibratory installation has been known to cause localized liquefaction of loose sands and non-plastic silts, which could be problematic for nearby structures. Lastly, since steel sheetpiling is most commonly used, corrosion susceptibility and protection of the sheetpiling in aggressive soil conditions are important concerns.

2.3 SOIL NAILED WALLS

Soil nail walls are constructed in a top-down manner using steel reinforcement bars (i.e., soil nails) installed subhorizontally in a relatively close spacing (i.e., 1.2 m to 1.8 m [4 ft to 6 ft] on center horizontally and vertically) in conjunction with a facing consisting of shotcrete and steel reinforcement to create a composite earth mass to provide stability and support to the retained soil. For most applications, solid steel reinforcement bars are used to resist the tensile forces and improve the shearing resistance of the soil mass, can be inserted into a drilled hole that is then filled with grout (most common) or can be driven into place. Alternatively, fully-threaded hollow-core steel bars may be used, which are drilled and grouted simultaneously in one operation. A key advantage of hollow bar soil nails is that they are particularly well suited in soils where an open drill hole cannot be maintained during the installation of solid bar soil nails, thereby requiring the use of temporary casing. Soil nail walls, similar to soldier beam-and-lagging and sheetpiling walls, can be used for temporary and/or permanent applications (Fig. 6).

The soil nails and the facing are installed as the excavation proceeds in vertical lifts of about 1.2 m to 1.8 m [4 ft to 6 ft] in height, the soil nails and the facing are installed for each lift, the order of which depends on the ground conditions at the particular site. The function of the components of the facing are as follows: the shotcrete is applied to the exposed face of the soil and is used to prevent raveling of the soil; the steel wire mesh and reinforcement bars are used to provide tensile and shear strength to the shotcrete; and the drainage fabric placed behind the shotcrete facing is used to provide a relief of water pressure and a means for moving water from behind the wall.

Soil nail walls are most applicable in ground conditions where the soil and rock (i.e., weathered and jointed/fractured rock) can be excavated in vertical lifts of about 1.2 m to 1.8 m [4 ft to 6 ft] in height and remain unsupported for a duration of one to two days. In practice, it is customary to

(a)

(b)

Fig. 6. (a) Temporary and (b) permanent soil nail walls
(courtesy of Schnabel Foundation Co.)

not leave the excavated lift open even overnight; therefore, the amount of soil excavated in one day is about equal to the amount of wall that can be constructed (i.e., installation of the soil nails, reinforcement, drainage fabric, and shotcrete) during that same time. Furthermore, soil nail walls are relatively fast to install resulting in good production rates if the holes for the soil nails can be drilled without the use of casing (i.e., maintain a stable open hole) throughout its installation.

Applicable ground types for soil nail walls include stiff-to-hard fine-grained soils, dense-to-very dense granular soils with apparent cohesion, weathered rock, engineered fill, residual soils, and glacial soils. These walls are able to accommodate various alignments and irregular wall geometries relatively easily. In weathered/fractured rock conditions and where a tight wall alignment (i.e., relatively straight excavation cut of the slope face of the soil) is required, additional small diameter steel reinforcement bars (i.e., #19 or #25 [No. 6 or No. 8]) are sometimes drilled and grouted into place vertically at relatively tight spacing (1.5 m to 1.8 m [5 ft to 6 ft]) along the wall alignment to aid in maintaining the vertical face of the wall and to provide additional support to the ground behind the wall during installation.

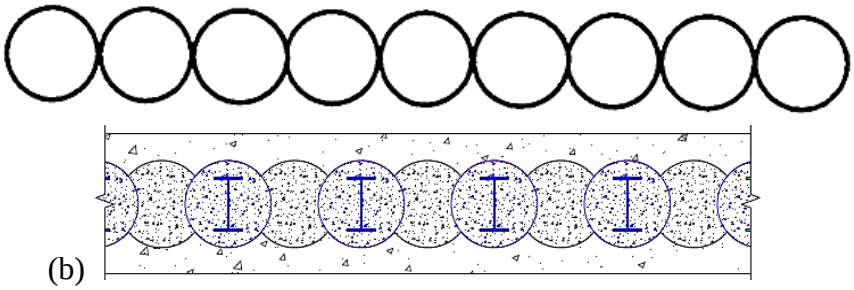
In granular soils, soil nail walls are installed above the groundwater table, as the (typical) facing of the wall (i.e., about 100 mm [4 in] thick for temporary conditions, and up to about 300 mm [12 in] for permanent) is usually not designed robust enough to withstand hydrostatic and/or seepage pressures. Large cost overruns can occur from overbreak (i.e., over excavation) due to poor excavation practice and due to removal of obstructions since the space(s) must be filled with shotcrete and not left void or open. Since the purpose of soil nailing is to create a composite earth mass, these wall systems are not applicable in areas with nearby utilities or where there is a possibility that an excavation may need to be performed close to the back of the wall. Unfavorable or difficult ground conditions for soil nailing include dry, poorly graded cohesionless soils, soils with high groundwater, soils with numerous cobbles and boulders, soft to very soft fine-grained soils, organic soils, highly aggressive/corrosive soil and/or groundwater, and weathered rock with unfavorable weakness planes (Tanyu et al, 2008).

2.4 TANGENT/SECANT BORED PILE WALLS

A tangent or secant pile wall is constructed in a top-down manner using a series of bored piles (i.e., drilled shafts) where adjacent piles abut each other (i.e., tangent pile wall) or overlap or

interlock with each other (i.e., secant pile wall), as shown in Fig. 7. For both tangent and secant pile walls, primary vertical piles are constructed at prescribed intervals by drilling a borehole to the desired depth and then filling the borehole with concrete or cement/bentonite grout (with or without steel reinforcement).

(a)



(b)

Fig. 7. Plan view schematic of a linear (a) tangent pile wall and (b) secant pile wall

For tangent pile walls, the secondary vertical piles are then installed in between and abutting the primary piles, wherein reinforcing steel is inserted, and the secondary piles are filled with the appropriate concrete mix. For a secant pile wall, the secondary vertical piles are then installed in between and overlapping the primary piles ensuring a closed joint or intersection between piles (Fig. 8a), then reinforcing steel is inserted and the secondary piles are filled with the appropriate concrete mix. The reinforcement for both types of walls may be composed of either a steel reinforcement cage or a steel beam section (Fig. 8b). Typically, the primary and secondary piles are of the same diameter. Both wall types are relatively stiff and provide a continuous wall along the alignment. When the ground conditions are favorable (e.g., in stable, medium stiff-to-still clays and dense sands) and when groundwater or seepage is not present, the individual piles can be spaced slightly apart (i.e., typically about 1 to 2 pile diameters), where the soil between the piles is excavated and replaced with shotcrete/wire mesh or cast-in-place reinforced concrete facing to provide a continuous wall and to prevent ground loss between the piles.

(a)

(b)

Fig. 8. Photographs of (a) an externally braced secant pile wall and (b) plan view of the top of a secant pile wall with reinforcement in both primary and secondary piles (courtesy of Hayward Baker, Inc.)

Tangent and secant pile wall systems can be used for temporary and/or or permanent ground support, and can serve as a permanent load bearing wall, and can be designed to provide resistance to both axial and lateral loading (e.g., from superstructures, earth pressures, water pressures, and surcharge). A key benefit of tangent and secant pile walls is that they can be constructed using

conventional drilled shaft excavation equipment and procedures, as compared to diaphragm wall and soil mixed wall systems. In addition, since comparatively smaller equipment is required with tangent or secant pile walls and smaller openings are made at any one location, less physical space is required for access, maneuvering, slurry preparation and processing, fabrication and handling of rebar cages, and spoil material. The practical depth of these walls is dictated by the capabilities of the drilling equipment, so wall or pile depths can be varied, as needed, to accommodate the loading and wall geometry.

When constructed properly, tangent and secant pile walls can be watertight so that a dewatering operation is not required within the excavation. If watertightness is a performance requirement, the quality of construction, especially with regards to verticality and positioning, is of upmost importance; even a minor deviation from the verticality tolerance can result in open spaces between piles. It is important to establish comprehensive yet realistic/fair tolerances within the specifications for the contractor to execute the work. Tolerances that should be established along with common tolerance values for each item include:

- Pile diameter - Allowable tolerance for the minimum and maximum diameter of the pile is 25 mm (1 inch) smaller (-25 mm or -1") and 150 mm (6 inch) larger (+150 mm or +6"), respectively, than the diameter shown on the plans.
- Verticality - Maximum permitted deviation of the completed pile from the vertical at any level is 1-2 in 100 (1 to 2%). Note, a verticality tolerance of 1% is a very restrictive tolerance, whereas a vertical tolerance of 2% is a more practically achievable tolerance.
- Horizontal positioning - At cut-off level, the maximum permitted deviation from the center of pile shown on the drawings shall be 25 mm (1 inch) in any direction.
- Allowable thru-wall seepage - Groundwater seepage through the joints or exposed face of the secant/tangent pile wall shall be limited to be less than visible and measurable flow, or as specified by the Engineer. Slight moisture on the exposed face of the secant pile wall, as evidenced by damp and darker colored concrete, shall be acceptable. Unacceptable seepage is defined as a regular steady drip or accumulated wetness observed running down the exposed face of the secant pile wall.

To ensure proper alignment, a reinforced concrete guidewall is formed in the ground prior to the start of the construction of the secant or tangent pile walls (Fig. 9a). The guidewall provides restraint to the positioning and verticality of the secant or tangent piles, and aids in the support of the reinforcing steel, casing extractors, etc. during the placement of concrete (Fig. 9b).

Some of the drawbacks associated with tangent/secant pile walls include

- Concerns with aesthetics, as the finished facing of the individual piles will likely be relatively rough and irregular (especially dependent on whether temporary casing was used during installation)
- The volume, temporary storage, and costs associated with excavated spoils and waste materials that are generated during construction, especially if a drilling support fluid is used and if contaminated materials are present
- The individual piles and overall wall system take time to construct, especially for large diameters and deep piles. Proper sequencing is also a concern as many well written

(a)  (b) 

Fig. 9. Photographs of (a) guidewall form for secant piles and (b) primary (unreinforced) and secondary (reinforced) piles installed (courtesy of Weeks Marine)

- specifications mandate that a new pile cannot be constructed within about 3 to 4 pile diameters from a freshly poured pile (i.e., prior to concrete set) to avoid communication between holes and to avoid compromising the integrity of the newly cast pile.
- The unit costs associated with constructing a secant pile wall are about 25% to 35% greater than for a tangent pile wall due to the tighter tolerances required to ensure proper positioning, verticality, overlap, etc. of the secant pile wall.

2.5 DEEP SOIL MIXED WALLS

Deep soil mixed walls are constructed by using specially designed equipment and tooling to blend the in-situ soil with a binder (e.g., cement, lime, etc.) to create overlapping columns of a soil-cement composite mass with improved properties compared to the untreated condition. Earth retaining walls typically contain reinforcing steel elements used to resist lateral earth, surcharge, and water pressures in deep excavations (Fig. 10). Hydraulic cutoff (i.e., seepage control) walls are used to prevent water movement through or under the retaining structures and into excavations below the groundwater table.

The primary construction goals are to provide a homogenized soil-cement mixture and to ensure an even distribution of binder throughout treated soil volume with a uniform moisture content and without significant unmixed portions of native soil or binder. The shear strength of the soil-cement composite mass is much less than typical strengths for concrete (i.e., typically less than about 2.07 to 3.45 MPa (300 to 500 psi) for soil mixing compared to 27.6 MPa (4,000 psi) or greater for tangent/secant pile and diaphragm walls). There are different methods and types of mixing equipment used to penetrate the soil and blend the binder and in-situ soil, including vertical axis with one or more shafts, cutter type on two wheels, and vertical cutter (i.e., chainsaw type) soil mixing method (Fig. 11). Typically, the binder injection ports are located at or near the cutting & mixing blades or teeth.

For vertical axis mixing, which is the most commonly used method in the U.S., slurry is injected during penetration of the tooling, with the nozzles located near bottom of the shaft or along the bottom level of cutting blades. The slurry is mixed with soil on both the downstroke and upstroke of the tooling, which increases the thoroughness of the mixing. Good construction practice often involves double-stroking (i.e., moving the tooling up and down within the same depth multiple

Fig. 10. Photograph of an anchored soil mixed wall
(courtesy of Schnabel Foundation Company)

(a) (b) (c)

Fig. 11. Photographs of (a) multi-axis vertical mixing method (courtesy of Nicholson Construction Co.), (b) cutter soil mixing (CSM) method (courtesy of Bauer Maschinen), and (c) vertical cutter soil mixing method (courtesy of Geosystems, L.P.)

times) and dwell time at the bottom of the element to achieve thorough mixing at a location that would not otherwise receive a full complement of mixing blade passes. For cutter soil mixing (CSM), the soil is mixed with water during penetration of the tooling/cutter to homogenize and increase the fluidity of the soil. During withdrawal, a slurry with a relatively low water-to-cement (w/c) ratio is injected into the soil. Practical depth limitation of both the vertical axis method and the CSM method for onshore earth retaining applications is about 40 m (130 ft).

With the vertical cutter soil mixing method, a specialized vertically-mounted cutter (chainsaw-type tool) is inserted into ground with simultaneous slurry injection and blends the soil and slurry to the target depth. The slurry injection and mixing continue as the crawler machine advances horizontally along the wall alignment to create a continuous wall of mixed soil. The vertical mixing action blends the entire soil profile, which eliminates any stratification and creates a soil mix wall with a high degree of homogeneity and extremely low permeability. The practical depth limitation for this method is about 30 m (100 ft).

Important to understand the relationship between the different mixing methods, mixing energy, binder type and amount, and binder(s) and soil. The values for an engineering property that realistically can be achieved for specific soils at a project site can range significantly. Lab testing is performed as part of the design process to verify feasibility of using soil mixing at the site, assess a reasonable range of property values for design, and may be performed again by the contractor as part of construction process. The engineering properties are influenced by the physico-chemical properties of the soil (e.g., pH, soil type, organic content, etc.); degree of blades, rotational speed, rate of mixing, binder content, and mixing water. Correspondingly, the quality of the mixed soil-cement material depends on various installation parameters, including injection method, tool rotation and penetration speeds, and geometry of the mixing tools.

Ultimately, one needs to ask oneself a straightforward geotechnical engineering question: are the soils at this site feasible and/or suitable for mixing? First, soil mixing is feasible/suitable in locations with soils that can be stabilized with cement, lime, slag, or other binders: (a) cohesive soils with high moisture contents (e.g., very soft to medium stiff clays), (b) saturated cohesionless soils (e.g., loose-to-medium dense sands), and (c) very soft-to-medium stiff organic soils and peat. However, soil mixing is not feasible/suitable in very stiff or very dense soils, or in geologic conditions with large cobbles or boulders. Dense gravels, cobbles, boulders, logs, and other obstructions can make penetration difficult or impossible.

The next set of questions pertain to construction and environmental considerations. Are construction materials readily available (e.g., constant and adequate supply of binder that can be ensured throughout the project)? Is there relatively unrestricted overhead clearance available along the alignment and across the project site? The equipment used to construct soil mixed walls are relatively heavy, and typically require the use of a working platform, timber mats, and/or steel plates if near surface ground conditions are too soft to support equipment. Are there environmental constraints associated with the project? The overall soil mixing process is relatively vibration- or noise-free, with the exception of the engines required to power the various pieces of equipment. A significant amount of spoils from wet mixing will likely be generated; do the spoils need to be disposed offsite or can they be used productively elsewhere on the project? A benefit of soil mixing is that the binder may immobilize contaminants in-situ and/or prevent their transport elsewhere (due to the seepage barrier wall).

Additional advantages and benefits of soil mixing for excavation support walls include structural elements can be inserted into the fluid mix; no need for other lagging between vertical structural elements; spoils from wet method may be used as site fill material; can achieve high production rates in highly suitable ground conditions; can uniformly treat variable and layered heterogeneous soils; increases the shear strength and decreases compressibility of soft silts, clays, organics soils, and peat; and a dewatering system is not necessary if the wall is designed as a cutoff barrier wall.

Conversely, disadvantages and limitations of soil mixing for excavation support walls include the requirement for experienced and specialized design engineers, contractors, and skilled labor; the need for sophisticated and specialized construction equipment; the need for a large working space with no overhead restrictions; freeze/thaw degradation may occur of the exposed wall face; the method is not applicable in very dense or very stiff soils (or if boulders are present); significant variability in treated soil strength may occur; utilities, cobbles, boulders, dense sand deposits,

buried logs, and other obstructions can interfere with penetration and efficiency of mixing equipment; and the high unit and mobilization costs.

2.6 DIAPHRAGM WALLS

A diaphragm wall is a structural foundation element that is constructed in discrete sections (i.e., panels) in a top-down manner using sophisticated and specialized equipment (Fig. 12) to excavate vertical trenches below the ground surface that are filled with steel reinforcement (e.g., a reinforcement cage or steel beam sections) and structural concrete. A drilling support fluid is commonly used to stabilize the trench and to counteract the stresses induced by earth, surcharge, and water pressures. Adjacent panels are joined at their ends by formed keyed joints that are created using specially fabricated stop end shapes or by milling along a vertical plane at the cast end of the previously constructed panel.

In the right ground conditions, hydromill trench cutters (Fig. 12a) are typically more economical than hydraulic grabs (Fig. 12b) when the depth of panels is greater than about 24 m (80 ft). A hydraulic grab (or clamshell) can only excavate soil and weak rock, but a hydromill trench cutter can be used to excavate weak soil through very hard rock. However, the wheels of a trench cutter may become jammed or "stuck" due to obstructions (e.g., debris, old piling, etc.) and some ground conditions (e.g., some clays and highly weathered rock materials). The selection of the equipment and drilling support fluid is based on a "right tool for the right conditions" philosophy as well as experience working in a local geology.

(a)  (b) 

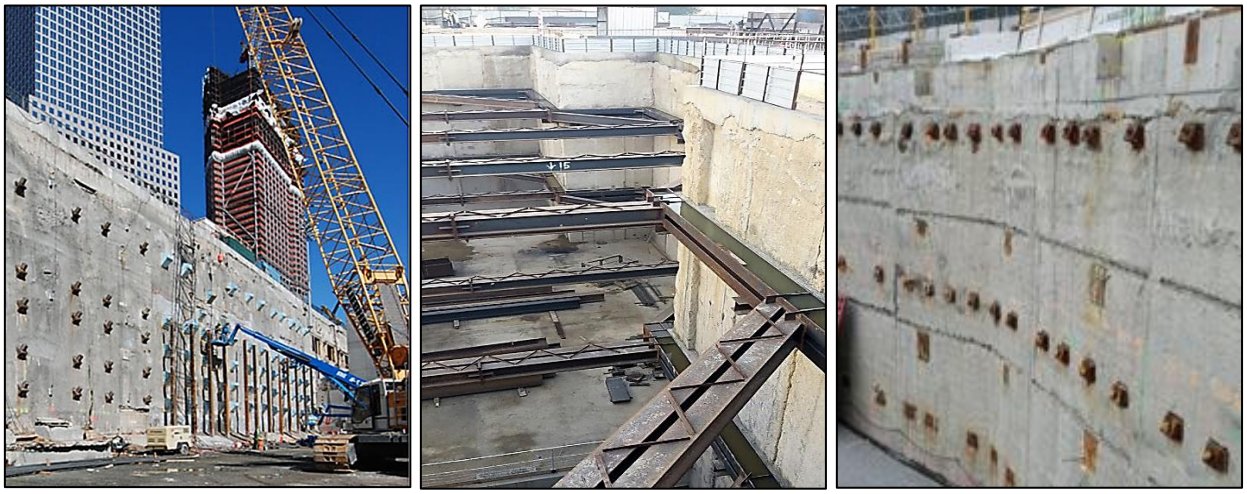
Fig. 12. Photographs of diaphragm wall equipment: (a) a hydromill trench cutter (courtesy of Hub Foundation Co.), and (b) a hydraulic grab (courtesy of Cementation Skanska)

A reinforced concrete guidewall is formed in the ground prior to the start of the construction of the diaphragm wall to ensure proper alignment during the initial portions of the excavation and to aid during the placement of concrete. Similar concerns with regard to verticality and watertightness discussed for tangent/secant pile walls are applicable to diaphragm walls.

In addition to technical considerations, panel geometry is controlled by the capability (and availability) of the equipment to be used by the contractor to perform the work. The dimensions of typical panels range from about 2.1 m to 7.6 m (7 ft to 25 ft) in length by about 610 mm to 1219

mm (24 inch to 48 inch) in width, which can require multiple passes or slots to be made for each panel. For typical earth retaining applications, diaphragm walls have been installed in a linear alignment to about 30 m to 40 m (100 ft to 130 ft) deep; however, for some utility and water/sewer projects, diaphragm walls installed in the shape of large diameter shafts have been installed as deep as 110 m (360 ft). Shorter panel widths are commonly used in weak and potentially unstable soils, in areas adjacent to sensitive structures to ground movements, and in conditions where large horizontal pressures are exerted due to surcharge pressures (e.g., from adjacent structures). When ground conditions are amenable (e.g., stable ground and no structures within zone of influence), longer panels may be used.

Diaphragm walls are commonly used on projects where movements of the AER wall and the retained soil need to be limited (e.g., adjacent to structures, sensitive equipment, etc.); therefore, this wall system is designed to be relatively rigid. Furthermore, diaphragm walls can be designed to withstand vertical loading in addition to earth, surcharge, and water pressures, and can function as temporary and/or permanent structures but are typically incorporated into the permanent work due to the costs associated with this wall system. Lateral support can be provided by ground anchors (Fig. 13a) or by internal bracing (Fig. 13b). If a permanent diaphragm wall is to be constructed and incorporated into the final structure, lateral bracing may also be provided by the floor(s) of the structure. In such cases, the reinforcement to be installed in the panel may include blockouts, dowels, and/or sleeves to facilitate later connection to the structure and its floor system.



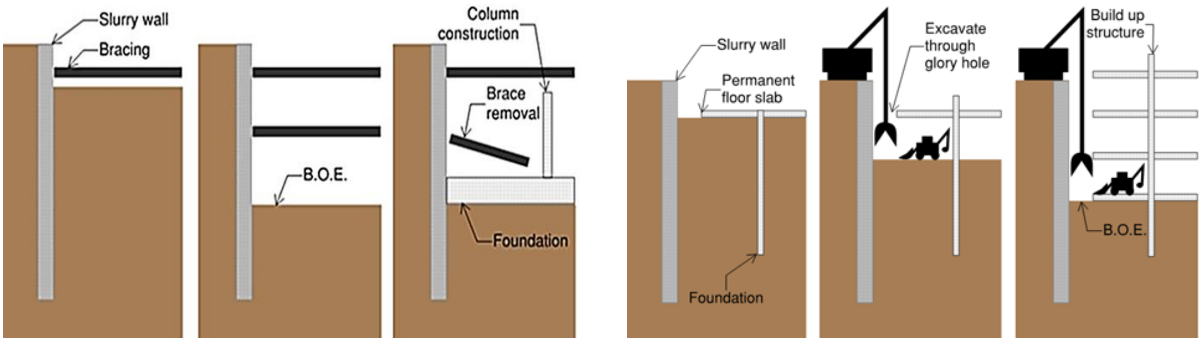
(a) (b) (c)

Fig. 13. Photographs of (a) anchored diaphragm wall with exposed multiple levels of ground anchors (courtesy of Nicholson Construction Co.), (b) diaphragm wall that is supported lateral by multiple levels of steel bracing (courtesy of Trevi S.p.A.), and (c) an anchored diaphragm wall where soil mixing was performed first to prevent/minimize against cave-ins of the in-situ soil (courtesy of W. Carswell)

Once the diaphragm walls have been installed, the soil can then be excavated using one of two methods: conventional excavation (Fig. 14a) or top-down excavation (Fig. 14b). For the conventional method, as the soil is excavated, temporary lateral support is provided to the diaphragm walls using bracing and/or ground anchors (Fig. 13), which is installed, as required, as the excavation proceeds to the desired depth (i.e., bottom of excavation or BOE). Then, the permanent structure is constructed in a bottom-up manner, removing the bracing or de-tensioning

the ground anchors as the permanent support system is installed (e.g., from floor slabs in buildings and floor and roof slabs in tunnels).

For top-down construction, components of the permanent structure (e.g., floor and/or roof slabs) are installed, which serve as lateral support to the wall, as the excavation of the soil proceeds to the bottom of excavation. Long-arm excavators and/or cranes equipped with clamshell buckets (similar to Fig. 12b) are used to remove the soil from temporary openings (i.e., gloryholes) in the floor and/or roof slabs. In general, due to the nature of the construction and excavation, the top-down method, which results in a significantly stiffer lateral bracing system, has resulted in smaller excavation-induced movements of the diaphragm wall and ground surface than those observed and measured due to the conventional method (Carswell et al, 2019).



(a) (b)

Fig. 14. (a) Conventional and (b) top-down methods of excavation and construction for diaphragm walls (Carswell et al, 2019)

If constructed properly, diaphragm walls are water tight and can be installed deep enough (i.e., embedded an appropriate depth into an impermeable layer) to create a cutoff barrier to groundwater flow, and a dewatering system would be required to extract the groundwater present within and to ensure stability of the soil at the base of the excavation. If the diaphragm wall is constructed with a bottom seal or a mat slab below the base of the excavation to withstand hydrostatic uplift, a temporary or permanent dewatering system would not be required.

Some of the drawbacks associated with diaphragm walls include:

- Concerns with aesthetics, as the finished facing of the panels may be relatively rough and irregular (especially if a vertical face of the soil was not maintained throughout excavation due to cave in or removal of an obstruction). However, a relatively smooth facing can be achieved, as shown in Fig. 13.
- The volume, temporary storage, and costs associated with excavated spoils and waste materials that are generated during construction, especially if the soils are contaminated
- The need for a large space for the equipment (i.e., slurry plant) needed for the preparation and processing of the drilling support fluid.
- The individual panels and overall wall system take time to construct, especially for deep panels, long panels, and in areas with restricted overhead clearance.

- Proper sequencing is also a concern as adjacent panels cannot be constructed in a continuous sequence to avoid communication between panels and to avoid compromising the integrity of the newly cast panel.
- The size of the excavation equipment and support equipment can be quite large and typically occupy a large amount of space and require stable working platform and/or the use of support mats below the equipment.
- This wall system has associated high mobilization and operational costs.

3. SELECTION CONSIDERATIONS

As project sites have become smaller in size, more congested, and are located in areas with oftentimes poor or unsuitable soils, AER wall systems become the method of choice to achieve a project's needs and performance criteria. However, one must evaluate many factors, at times iteratively, when selecting the type of wall system appropriate for a given project. Not all of the considerations and concerns are technical in nature; economics/finances, politics, and environmental concerns are also part of the decision-making process, and sometimes govern over even the best technical solution.

At the onset of the project, an owner and/or engineer must identify project and environmental constraints (e.g., accessibility, right of way and easements, headroom or overhead limitations, spatial limitations on the site, disposal concerns, existing utilities, adjacent structures, and possible contaminated soil conditions) to define the applicability of methods and scope of work. With regard to the wall system itself, will the wall system be temporary (e.g., less than about 36 months) or permanent (e.g., service life as long as the main structure, up to 75 to 100 years or more)? What are the desired function and performance needs for the wall system: support of the adjacent ground, grade separation, groundwater/seepage barrier, and/or provide support to vertical loading in addition to lateral loading? Since each project has a finite amount of funding available, consideration is also given to the following:

- Corrosion protection of all susceptible components and members
- Availability of experienced contractors, equipment, skilled labor, and materials
- Proximity to sensitive structures, utilities, and/or roadways
- Costs for excavation and disposal (can be quite high if the soils are contaminated)
- Wall aesthetics and overall appearance (i.e., for a permanent wall that will be exposed)
- Special drainage or dewatering requirements
- Speed of construction (i.e., scheduling), and
- Life cycle costs (e.g., maintenance and corrosion susceptibility).

4. ASSESSMENT OF SITE CONDITIONS AND DESIGN PARAMETERS

Properly performing AER systems requires the designer and contractor to use a combination of theory, practice, and practicality. That is, engineers must utilize the correct design theories with appropriate parameters that reflect the methodology and techniques used in practice to construct the AER systems within the physical conditions present at the site. Each project should require an investigation of the above ground features and characteristics as well as subsurface site

characterization and sampling. A review of available documents and records is also important for understanding the history and prior use of the site.

Ultimately, it is necessary for the designer to match the site investigation and laboratory testing with the needs of the project, in this case, in terms of AER wall systems (i.e., the vertical wall and its lateral support system). That is, the more variable the site and the subsurface profile and soil characteristics, the more comprehensive the site investigation should be. Correspondingly, the more complex, higher profile, and higher risk the project, the more evaluation and testing should be performed. That is, performing more testing (i.e., quantity) and different types of testing provides additional data as well as informs the designer when developing and analyzing more complex models, which is what is really needed when attempting to provide practical solutions and answers to questions for the more complex and higher profile projects. However, each test that is to be performed should be attempting to answer a question; the test should not be performed to just obtain an additional data point. In the end, the risk exposure (and acceptability of the risk) needs to be balanced with the project scope and desired performance as well as with the resources available to construct the work.

For AER wall systems, what information and data should be collected and analyzed? From where and how many samples should be collected? What pre-design activities should be performed: surface and subsurface characterization, in-situ testing, and laboratory testing? What physical constraints and limitations exist at the project site (e.g., overhead, utilities, limited right of way or easements)? How variable is the subsurface? What will be the design height of the wall, and will it be for temporary or permanent use? Are there structures located behind the wall within the zone of influence, what type of structures are they, and how sensitive are the structures to ground movements?

During a site investigation, information of interest to be collected includes surface topography, evidence of rock outcrop, assessment of site access conditions, determination if the site was disturbed or used for some other purpose previously, availability and limits of right of way or easements, presence and location of utilities, evidence of potential instability (e.g., scarp from an old landslide, excessive settlement of the ground and neighboring structures due to weak or organic soils, etc.), and presence/location of groundwater and surface drainage patterns. Based on a preliminary or cursory investigation, it may be readily evident that some wall types and/or support method (e.g., ground anchors versus internal bracing) are not applicable for a given project site. To properly design the AER wall system, the site investigation program should define any potentially problematic or unsuitable soil conditions that could affect the construction and performance of the AER wall system:

- Cohesionless sands and silts that tend to run or cave-in and/or that may be susceptible to liquefaction or vibration-induced densification;
- Highly compressible soils (e.g., high plasticity clays and organic soils) that have time dependent strength properties and that exhibit creep behavior;
- Aggressive or corrosive ground conditions, which are classified according to the electrochemical properties (i.e., pH [acidic or alkaline], salt content, and resistivity) of the soil, presence or potential of stray electrical currents, type of soil (e.g., organic and calcareous), and presence of debris or industrial waste (e.g., fly ash, cinder, and slag).

Metallic components that are buried or in direct contact with these soils must be protected against the potential for corrosion;

- Weak soil or rock layers, especially when jointed, fractured, and/or inclined, which could be susceptible to sliding instability; and
- Man-made or natural obstructions, boulders, cemented layers, etc. that could adversely affect the construction of the wall elements and drilling and grouting of the ground anchors.

The subsurface site characterization is focused on obtaining the necessary information and data to adequately evaluate the properties and characteristics of the soil and rock to determine the appropriate parameters to design the AER wall system (i.e., the vertical wall and its lateral support system). The extent of the site investigation should be consistent with the scope of the project (i.e., size, critical nature/risk exposure, available budget, etc.), performance requirements (i.e., lateral movement), temporary or permanent wall system, and other project constraints.

The questions always asked, among many others, are how many borings, etc. should be made, where should they be located, how deep do they need to go, and how many samples should be taken? The answer is "it depends" on the project scope, constraints, wall function, site variability, etc. The types of in-situ and laboratory tests and the determination of strength properties are beyond the scope of this paper; however, extensive discussion on in-situ site characterization, laboratory testing, and determination of shear strength parameters is provided in *Evaluation of Soil and Rock Properties* (Sabatini et al, 2002) and *Geotechnical Site Characterization* (Loehr et al, 2016).

Guideline recommendations for the locations of in-situ sampling and testing for a permanent AER wall system are illustrated in Fig. 15 (Sabatini et al, 1999). Sampling and testing need to be performed in front and behind, when possible and if needed, the proposed face of the wall system as well as at multiple locations along the wall alignment. The sampling and/or in-situ testing must be performed to an adequate depth to define the subsurface profile (e.g., layering, inclination and thickness of the layers, and spatial variability) and to identify potential weak layers, rock and its quality, and strength parameters needed for stability.

For sampling and testing in front of the wall, it is important to determine the soil shear strength that is available and needed for the passive resistance to address stability concerns. Along the wall alignment, it is important to determine the soil/rock stratigraphy through which the wall will be constructed. The depth of the boring/sampling should extend below the base of the proposed wall a distance of at least the height of the wall to identify the bearing stratum of the wall as well as any potentially weak or unsuitable layers. The sampling and testing behind the walls are to define the soils/rock within and through which the ground anchors will be installed, if required, and should extend to a depth that encompasses the proposed anchor bond zone (i.e., where the anchor load is resisted).

5. EXTERNALLY SUPPORTED STRUCTURAL WALL SYSTEMS

As many sources of technical design guidance, design methodologies, and step-by-step design procedures are available (e.g., AASHTO, 2010; PTI, 2014; Sabatini et al, 1999; and Tanyu et al, 2008), the following sections will present some of the important items for consideration when

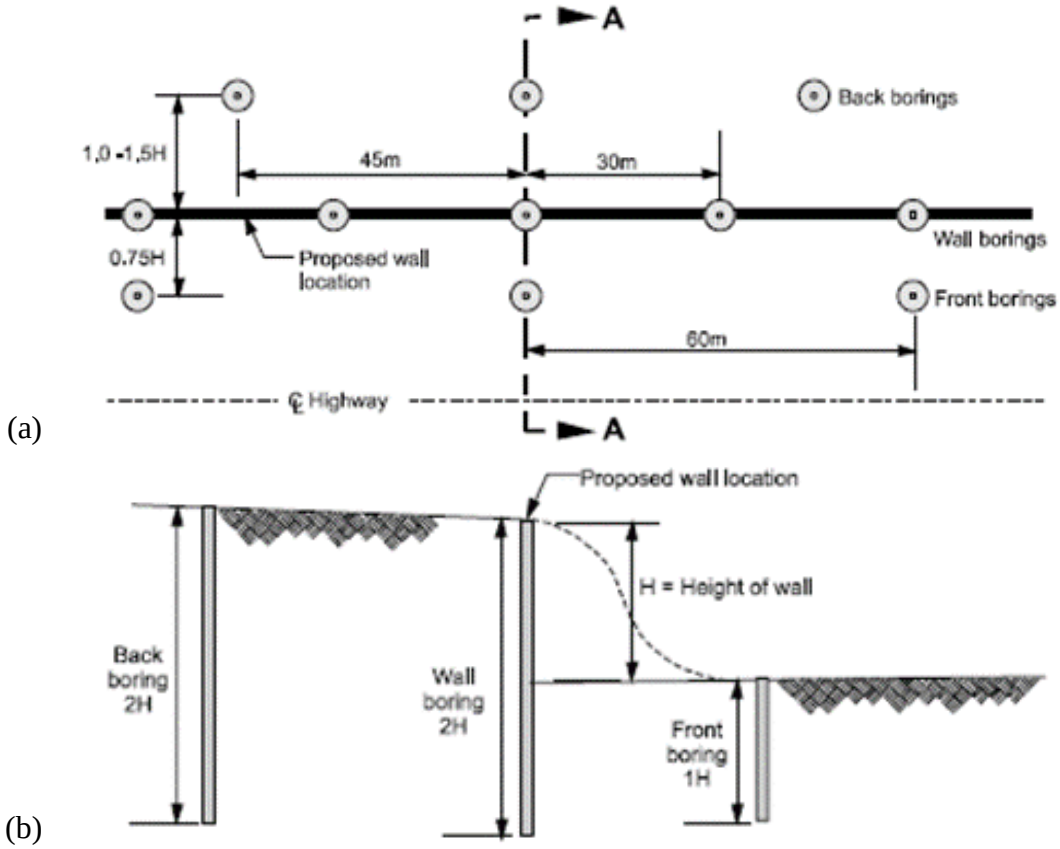


Fig. 15. Geotechnical boring layout for a permanent AER wall system:
(a) plan view and (b) section A-A (after Sabatini et al, 1999)

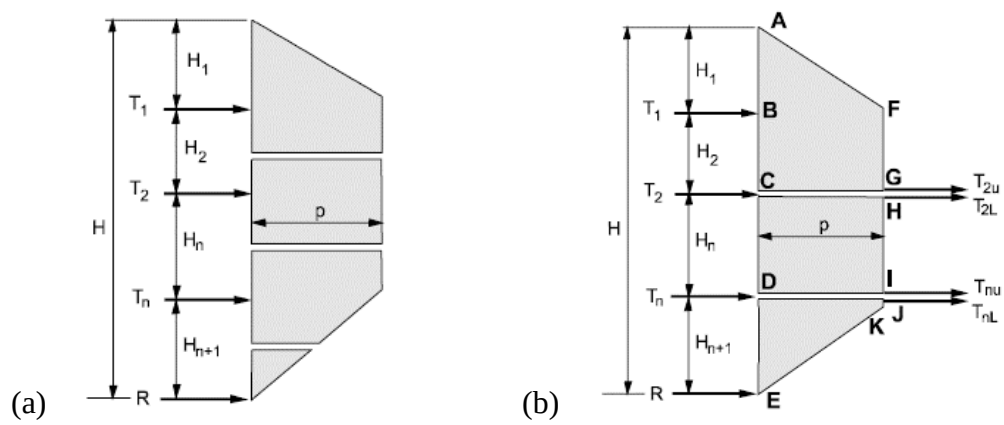
designing AER walls systems. In general, externally supported structural walls rely primarily on the bending resistance of its vertical structural elements to resist the applied lateral and axial loads.

5.1 DESIGN PROCEDURE

The general steps for the design of externally supported AER wall systems include:

1. Establish the project requirements and criteria for the AER system, which includes the wall geometry (i.e., alignment and wall heights), loading (i.e., earth pressures, surcharges, water pressures, and vertical loading), temporary and/or permanent wall, performance criteria (e.g., settlements and movements), and constraints (i.e., project and environmental).
2. Perform the site investigation and characterization and any in-situ and laboratory testing to evaluate the subsurface conditions and determine the geotechnical properties of the soil and rock for design.
3. Establish the required factors of safety (for Allowable Stress Design platform) and/or load and resistance factors (for Load & Resistance Factor Design platform).
4. Define the appropriate level of corrosion protection for the ground anchors (and any metallic components in direct contact with the ground) based on site conditions, aggressiveness of the ground conditions, design life of the AER wall system, and tolerance to risk exposure.

5. Select appropriate axial and lateral loads and pressures that will be acting on the AER wall system within its zone of influence (e.g., earth pressure distribution, water pressure, surcharge pressures, and extreme event loading (e.g., seismic pressures)). The computations may be performed manually using the tributary area method or hinge method of analysis (Fig. 16) and using Apparent Earth Pressure (AEP) diagrams (Fig's 17 to 19) or may be developed based on experience and/or site-specific data. The Terzaghi and Peck AEP diagrams were developed based on an envelope of maximum pressures, incorporates variability inherent in construction, and do not include water pressure or surcharge(s) pressures. Alternatively, the computations may be performed using commercially available computer software systems such as DeepEX, PYWALL, Shoring Suite, Plaxis, and WALLAP.
6. Calculate the loads in the ground anchor loads and the bending moments in the vertical structural members/wall system. Note, adjust the locations (i.e., vertically along the wall height) of the ground anchors to determine the optimum moment distribution by balancing the moments as practical as possible.



T_1 = load acting over the length $H_1 + H_2/2$

T_2 = load acting over the length $H_2/2 + H_n/2$

T_n = load acting over the length $H_n/2 + H_{n+1}/2$

R = load acting over the length $H_{n+1}/2$

T_1 = calculated by $\Sigma(M_C) = 0$

T_{2u} = Total earth pressure (ABCGF) $- T_1$

T_{2l} = calculated by $\Sigma(M_D) = 0$

T_{nu} = Total earth pressure (CDIH) $- T_{2l}$

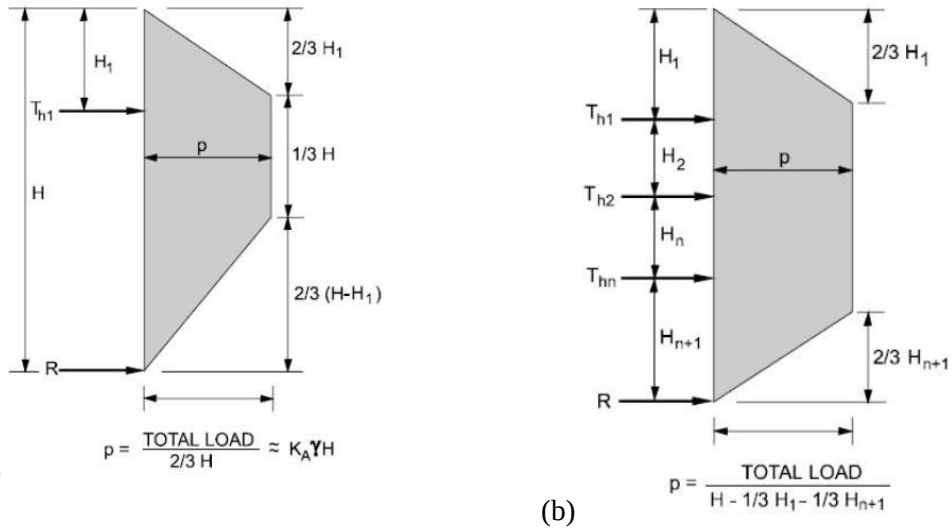
T_{nl} = calculated by $\Sigma(M_E) = 0$

$T_2 = T_{2u} + T_{2l}$

$T_n = T_{nu} + T_{nl}$

R = Total earth pressure - $T_1 - T_2 - T_n$

Fig. 16. Manual computation methods of analysis: (a) tributary area method and (b) hinge method of analysis for multiple levels of ground anchors/bracing (after Tanyu et al, 2008)



where: H_1 = Distance from the ground surface to the uppermost ground anchor

H_{n+1} = Distance from the base of excavation to the lowermost ground anchor

T_{hi} = Horizontal load in ground anchor i

R = Reaction force to be resisted by the subgrade (i.e., below base of excavation)

p = Maximum ordinate of the diagram

Total Load = $0.65K_A\gamma H^2$

Fig. 17. Recommended AEP diagrams for sand for walls with (a) one level or (b) multiple levels of ground anchors/bracing (Sabatini et al, 1999)

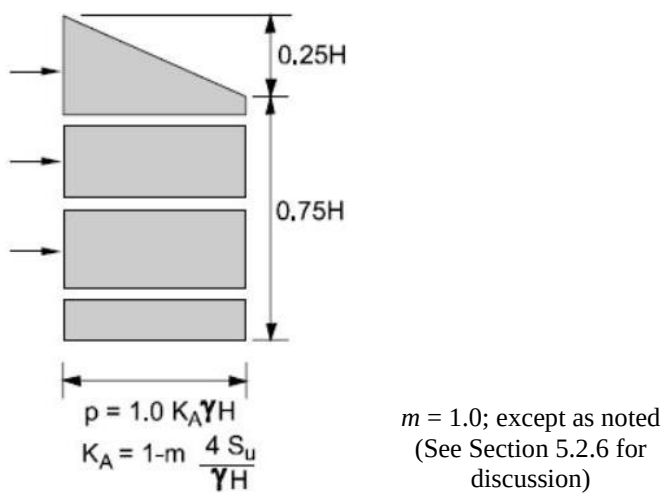
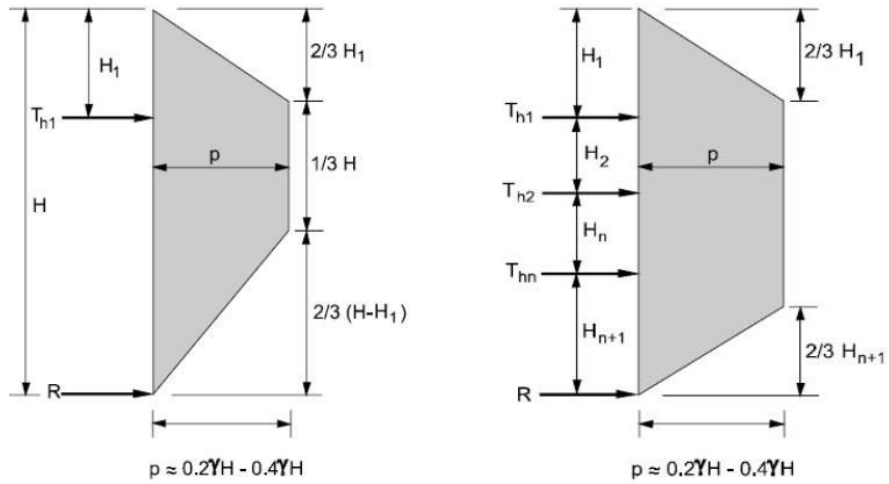


Fig. 18. Terzaghi and Peck AEP diagram for soft-to-medium clays (Sabatini et al, 1999)



where: H_1 = Distance from the ground surface to the uppermost ground anchor

H_{n+1} = Distance from the base of excavation to the lowermost ground anchor

T_{hi} = Horizontal load in ground anchor i

R = Reaction force to be resisted by the subgrade (i.e., below base of excavation)

p = Maximum ordinate of the diagram

Total Load (kN/m/m of wall)= $3H^2$ to $6H^2$ (where, H = wall height in meters)

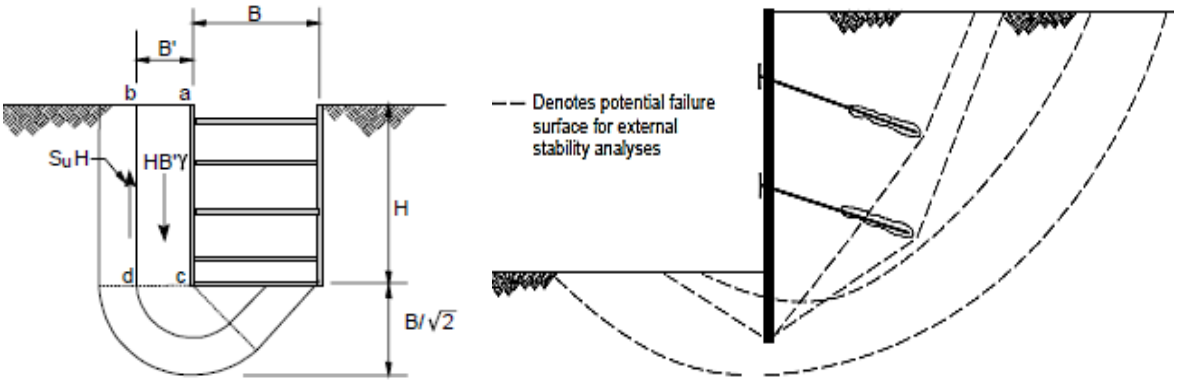
Fig. 19. Recommended AEP diagrams for stiff-to-hard clay for walls with (a) one level or (b) multiple levels of ground anchors/bracing (Sabatini et al, 1999)

7. Evaluate the forces, bond stress, and inclination of the ground anchors. The ground anchors should be installed as flat as possible (i.e., installed subhorizontally typically between angles of about 15° and 30°). Ground anchors installed at a steep angle subhorizontally (i.e., greater than about 40°) will apply large downward axial loads on the wall; in such cases, the bearing capacity should also be evaluated. If the bearing capacity is insufficient, large lateral movements of the wall may result due to the settling of the wall. The designer also needs to ensure the entire ground anchor will be located within the available easement or right of way and will avoid known obstructions and utilities. The type (i.e., bar or strand tendons), dimensioning (e.g., tendon size, length, components, and drill hole), and center-to-center spacing (i.e., horizontally along the wall alignment) of the ground anchor must be determined according to the anticipated and computed forces without failing geotechnically and structurally and to what can realistically be constructed at the site.

   The length of the ground anchor is dependent on the wall height, and must extend beyond the unstable zone (i.e., active failure wedge and critical failure plane) and metastable zone (i.e., typically a distance of about 1.5 m [5 ft] measured perpendicular to the active failure wedge) and through the bond zone (i.e., the portion resisting the tensile forces in the ground anchor). The free length in a ground anchor (i.e., length of anchor along which no tensile force should be resisted) is typically a minimum of 3 m (10 ft) for bar tendons and 4.6 m (15 ft) for strand tendons. The bond zone should be located a minimum of 1.5 m (5 ft) or 20% of the wall height beyond the critical failure plane. The length ground anchor within the bond zone should be the maximum of the development length of the tendon or the required length for load transfer

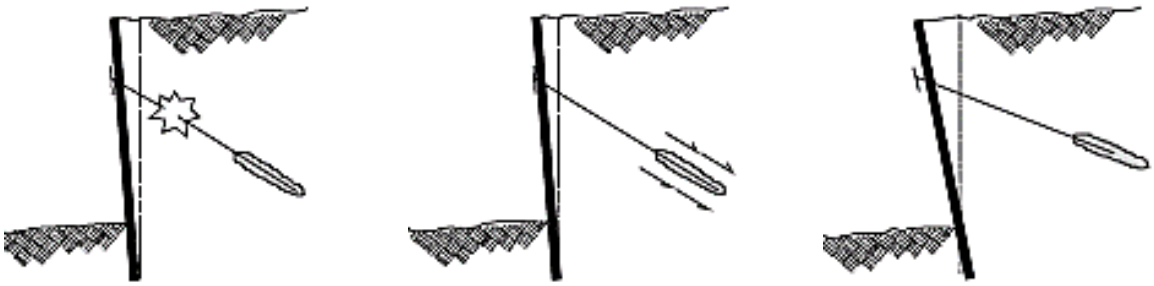
(grout-to-ground bond). Presumptive average ultimate bond stress values for the ground-to-grout interface along the anchor bond zone in different soil and rock materials are provided in Table 7 of GEC-4 (Sabatini et al, 1999) and in Table 5-3 in the FHWA Micropile Design and Construction reference manual (Sabatini et al, 2005).

8. Once the preliminary geometry of the wall and its supports has been determined, the external stability (Fig. 20) and internal stability (Fig. 21) of the wall system must be evaluated. In addition, the bearing capacity at the base of the wall must also be evaluated. If necessary, revise the wall and anchor/bracing geometry until the external and internal stability are acceptable.



(a) (b)

Fig. 20. Examples of external stability evaluations: (a) basal stability and (b) various potential failure surfaces (after Tanyu et al, 2008)

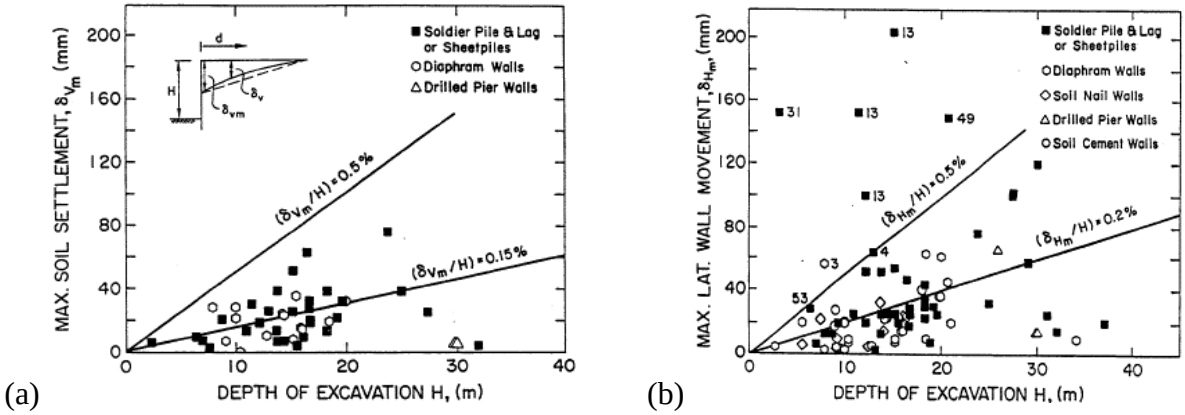


(a) (b) (c)

Fig. 21. Examples of internal stability evaluations: (a) tensile failure of the tendon, (b) pullout failure of the grout-to-ground bond, and (c) pullout failure of the tendon-to-grout bond (after Sabatini et al, 1999)

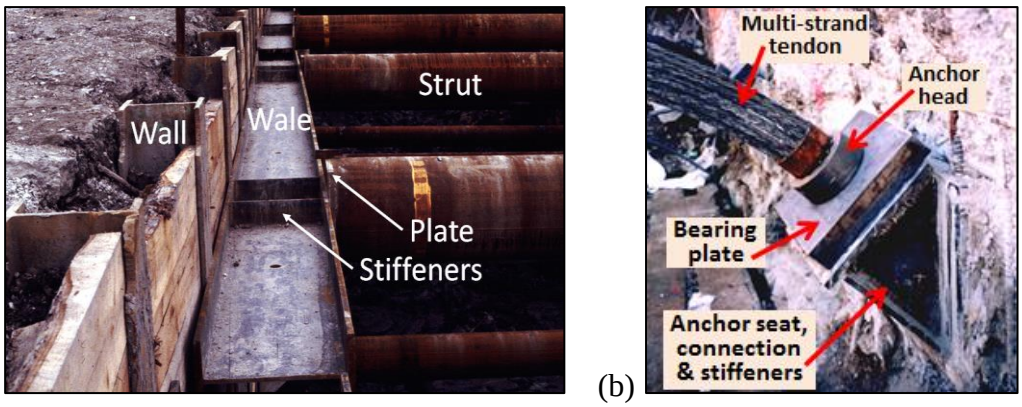
9. The lateral wall movements and possible settlement of the ground surface must be evaluated and estimated using correlations (Fig. 22) published in the technical literature (e.g., Clough and O'Rourke, 1990) or using commercially available software programs to analyze the complex soil structure interaction problem.

10. Once external and internal stability of the wall system have been satisfied and the estimated settlement of the ground surface and maximum displacement of the wall have been determined to be within acceptable limits, the components of the wall system (Fig. 23) must be designed structurally (e.g., internal steel reinforcement, concrete mix design, connections of the bracing and/or ground anchors to the wall, and permanent facing, if required).



(a) (b)

Fig. 22. Estimation of (a) settlement of the ground surface and (b) lateral displacement of the wall based on depth of excavation and wall type (Clough and O'Rourke, 1990)



(a) (b)

Fig. 23. Examples of structural components that need to be designed: (a) external bracing components (Finno, 2016), and (b) connection components for a multi-strand ground anchor (courtesy of Nicholson Construction Co.)

6. GROUND ANCHORS

Ground anchors can be grouped into three basic categories: temporary, permanent, and removable anchors. A temporary anchor is applicable to projects where the design service life is typically less than about 36 months. Under normal circumstances (e.g., no aggressive ground conditions), short-term protection of the anchor tendon is provided only by the cement grout encapsulating the tendon (i.e., no protection). If the duration the ground anchor may be in service is somewhat uncertain or if there is a potential for aggressive ground conditions, then an epoxy coating (i.e., Class II, single corrosion protection) may be applied to the tendon by the manufacturer to provide protection to the tendon. A permanent ground anchor comprises a double corrosion protection provided by the cement grout encapsulating the tendon, similar to a temporary anchor, as well as an outer encapsulation provided by a corrugated plastic sheathing (Class I, double corrosion protected). If designed in accordance with the applicable codes or standards of practice, a permanent ground anchor should provide a design life of typically up to 100 to 120 years.

In aggressive ground conditions and/or where the consequences of failure are deemed high or critical, temporary ground anchors should be manufactured in accordance with double corrosion protection requirements. Aggressive ground conditions (i.e., potentially corrosive) are denoted or classified according to the electrochemical properties (i.e., pH [acidic or alkaline], salt content,

and resistivity) of the soil, presence or potential of stray electrical currents, type of soil (e.g., organic and calcareous), and presence of debris or industrial waste (e.g., fly ash, cinder, and slag). As with all buried metallic elements, corrosion protection should be provided in all instances where the steel tendon can be exposed to the natural elements, in-situ ground, and groundwater, especially should be provided near or at the anchor head. Trumpets are used to ensure the grout encapsulates the upper portion of the hole. Grease and grout can be used to protect the head, wedges/nuts, and tendon at the outer surface of the anchor head.

6.1.1 CONVENTIONAL GROUND ANCHORS

Barley (1997) reported that, at the test load, a conventional ground anchor with a 10 m (33 ft) long fixed length (i.e., bond length) will need to extend some 30 mm (1.2 in) at the proximal end of the fixed length before any load will be transferred to the distal end of the tendon. Since the elastic behavior of the tendon is different to that of the grouted soil around it (i.e., strain incompatibility), the resulting differential strains induced during loading cause debonding at the weakest interface (i.e., along the ground-to-grout interface). A highly non-linear profile of bond stress results as the resistance in the anchor approaches the geotechnical strength limit state (i.e., ultimate ground-to-grout resistance). For clarity, *resistance* commonly refers to the properties of the ground-to-grout interface, whereas *capacity* generally refers to the strength properties of the tendon.

Progressive debonding, as the phenomenon is commonly referred, generally results in an inefficient use of the in-situ ground-to-grout bond strength. Under these circumstances as the tendon undergoes elongational deformation, more of the resistance towards the distal end of the fixed length of the anchor is being utilized since the peak bond strength toward the proximal end has been exceeded. That is, during elongational deformation, the available resistance increases to a peak state; however, once the peak bond strength has been exceeded, the available resistance reduces to a softened strength state and then to a residual strength state with continued deformation (Fig. 24).

6.1.2 SINGLE BORE MULTIPLE ANCHORS (SBMAS)

Single bore multiple anchors (SMBAs), as opposed to conventional ground anchors (Fig. 25a), utilize multiple individual unit tendons of varying lengths that are installed within a single borehole (Fig. 25b). Various research studies and practical applications have demonstrated that this SBMA configuration facilitates greater efficiency of load transfer to the surrounding ground. Unlike conventional ground anchors, especially in heterogeneous and variable ground conditions, the design of SBMAs can be optimized because the bond length of an individual unit anchor can be designed individually, which maximizes the inherent strength of the ground.

Compared to conventional ground anchors (Fig. 26a), SBMAs provide an optimized anchor system that can transfer the applied loading simultaneously to multiple separate short unit lengths within the fixed length of the anchor (Fig. 26b) with a significantly reduced progressive debonding. Therefore, the in-situ ground-to-grout bond strength is mobilized efficiently, resulting in a considerable increase in the capacity of the anchor. By effectively distributing the load along the fixed length portion of the anchor, SBMAs efficiently mobilize and maximize ground strength and, in appropriate ground conditions, can double the capacity provided by conventional anchors.

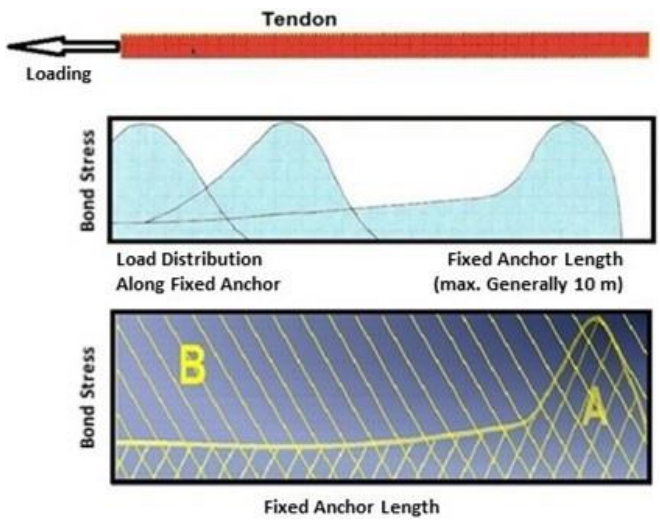


Fig. 24. Progressive debonding along a conventional fixed anchor length (after Barley, 1997)

(a) (b)

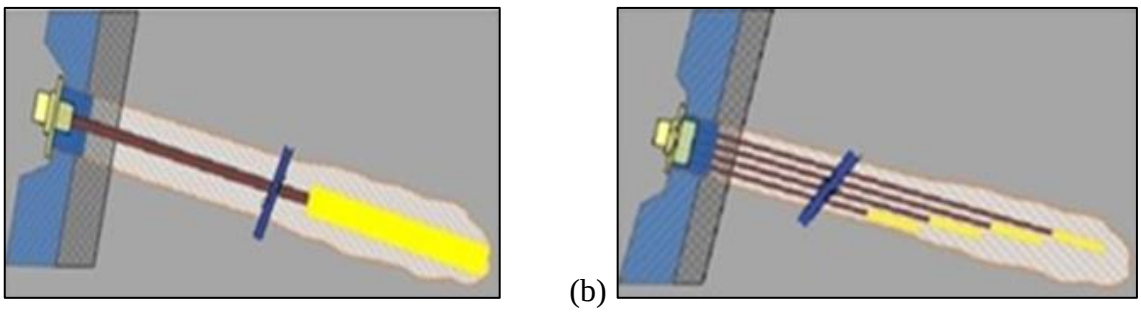


Fig. 25. Schematic of (a) a conventional ground anchor and (b) a SBMA

(a) (b)

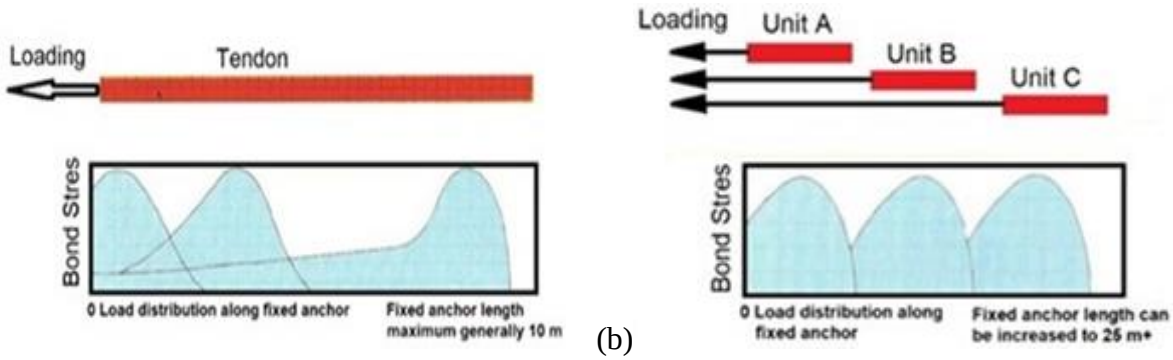


Fig. 26. Load distribution in (a) conventional ground anchors and (b) SBMAs

Ostermeyer and Barley (2003) provide a detailed discussion of the process for the design of SBMAs, and the basic principles are similar to the design of conventional anchors (Eq. 1) with the exception that an efficiency factor (f_{eff}) is introduced in the equation to compute the ultimate anchor load (Eq. 2). The efficiency factor accounts for the non-linear bond stress profile already described (Fig. 25).

$$T_{f\_conv} = \pi \cdot d_a \cdot L_f \cdot \tau_{ult} \tag{1}$$

$$T_{f\_SBMA} = \pi \cdot d_a \cdot L_f \cdot \tau_{ult} \cdot f_{eff} \tag{2}$$

where, T_{f_conv} is the ultimate capacity of a conventional ground anchor; T_{f_SBMA} is the ultimate capacity of a unit anchor; d_a is the diameter of the borehole; L_f is the fixed length of a unit anchor; and τ_{ult} is the ultimate ground-to-grout bond stress. A relationship for the efficiency factor, f_{eff}, was back-calculated from the ratio of actual bond stress to idealized bond stress area determined from the analysis of multiple anchors with different fixed lengths, installed in mixed soils, and tested to failure (Barley, 1997). Each computed ratio was plotted versus its respective fixed length (where L_f is in meters), as shown by the data points in Fig. 27, and a relationship for f_{eff} with respect to fixed length (L_f) was established using curve fitting (Eq. 3).

$$f_{eff} = 1.6 \cdot L^{-0.57} \tag{3}$$

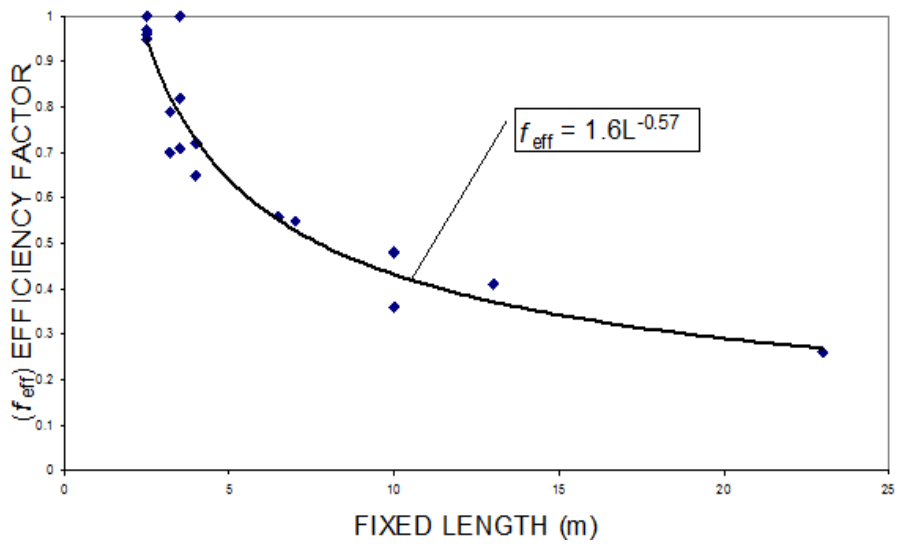


Fig. 27. Efficiency factor versus anchor's fixed length (after Barley, 1997)

The fixed length of conventional ground anchors ranges considerably (L_f = 6 to 15 m [20 to 50 ft] or longer) depending on the ground conditions and design loading; however, the fixed lengths of the unit anchors are consistently less than those for conventional anchors and typically ranges from 2.5 to 4.5 m (8 to 15 ft). As an example, the efficiency factor ranges from 0.34 to 0.58 for conventional anchors (with L_f = 6 to 15 m) and ranges from 0.95 to 0.68 for SBMAs (with L_f = 2.5 to 4.5 m). Moreover, since SBMAs can accommodate greater loads than conventional anchors, designers can reduce the overall quantity of anchors required to support the structure.

The SBMA system can accommodate variable ground conditions directly and simply by adjusting the fixed lengths of the unit anchors based on the bond strength available. If the soil is weaker within the upper portion of the fixed length, then the proximal unit anchors will have longer unit fixed lengths compared to those at greater depth. For the SBMA system, there is no theoretical limit to the total (or overall) fixed length, whereas there is little or no practical increase in load capacity for conventional anchors with fixed lengths greater than about 10 m (33 ft). Regardless of the unit length, equal load is applied to each unit anchor such that each unit anchor is mobilizing the same percentage of the ultimate ground-to-grout bond capacity.

The construction and installation of SBMAs follows the general methodologies adopted in most conventional grouted anchor construction (i.e., tendon fabrication, drilling, installation, and

grouting). Irrespective of the type of anchor - conventional or SBMA, the free length (i.e., unbonded zone) is the same because this length is based on the retained height of the wall and/or the geometry of the unstable soil/rock mass. However, the arrangement and length of the fixed length (i.e., bond zone) for SBMAs is significantly different from conventional anchors, which results in fewer and longer SBMAs compared to more and shorter conventional anchors.

The stressing of SBMAs require a slightly different test set up than conventional ground anchors, whereby separate hydraulic jacks are hydraulically synchronized to stress each individual tendon simultaneously so that each unit anchor receives the same load. SBMA systems involve the installation of multiple unit anchors into a single borehole, whereby each unit anchor has its own individual tendon, its own unit fixed length of borehole, and is loaded with its own unit stressing jack. The loading of all of the unit anchors is performed simultaneously by using multiple hydraulically synchronized jacks (Fig. 28a) or a single hollow ram jack (Fig. 28b).

(a) 

(b) 

Fig. 28. Testing of SBMA anchors: (a) using multiple jacks (Doha metro, Qatar), and (b) a single hollow ram jack simultaneously stresses a 3-unit system (Izmir, Turkey)

7. INTERNALLY STABILIZED WALLS (SOIL NAIL WALLS)

The most current technical design guidance and state-of-the-practice document available on the design and construction of soil nail walls is FHWA GEC-7 (Lazarte et al, 2015), which includes the implementation of the Load and Resistance Factor Design (LRFD) platform. The 2015 manual is an update of the previous version of GEC-7 (Lazarte et al, 2003), which implemented the Allowable Stress Design (ASD) platform. The design and analysis are typically performed using commercially available computer software systems such as SLIDE, SNAP-2, DeepEX, and Snail-Plus. The general sequencing of the construction of soil nail walls is illustrated in Fig. 29. The following sections will present an overview and discussion of the pertinent design and construction guidance for soil nail walls (Fig. 30) presented in GEC-7 (Lazarte et al, 2015); however, seismic and drainage design and considerations will not be discussed hereinafter.

7.1 DESIGN PROCEDURE

The general steps for the design of a soil nail wall include:

1. Establish the project requirements and criteria for the soil nail wall, which includes the wall geometry (i.e., alignment and wall heights), function and design life (i.e., temporary and/or permanent wall), performance criteria, and constraints (i.e., project and environmental).

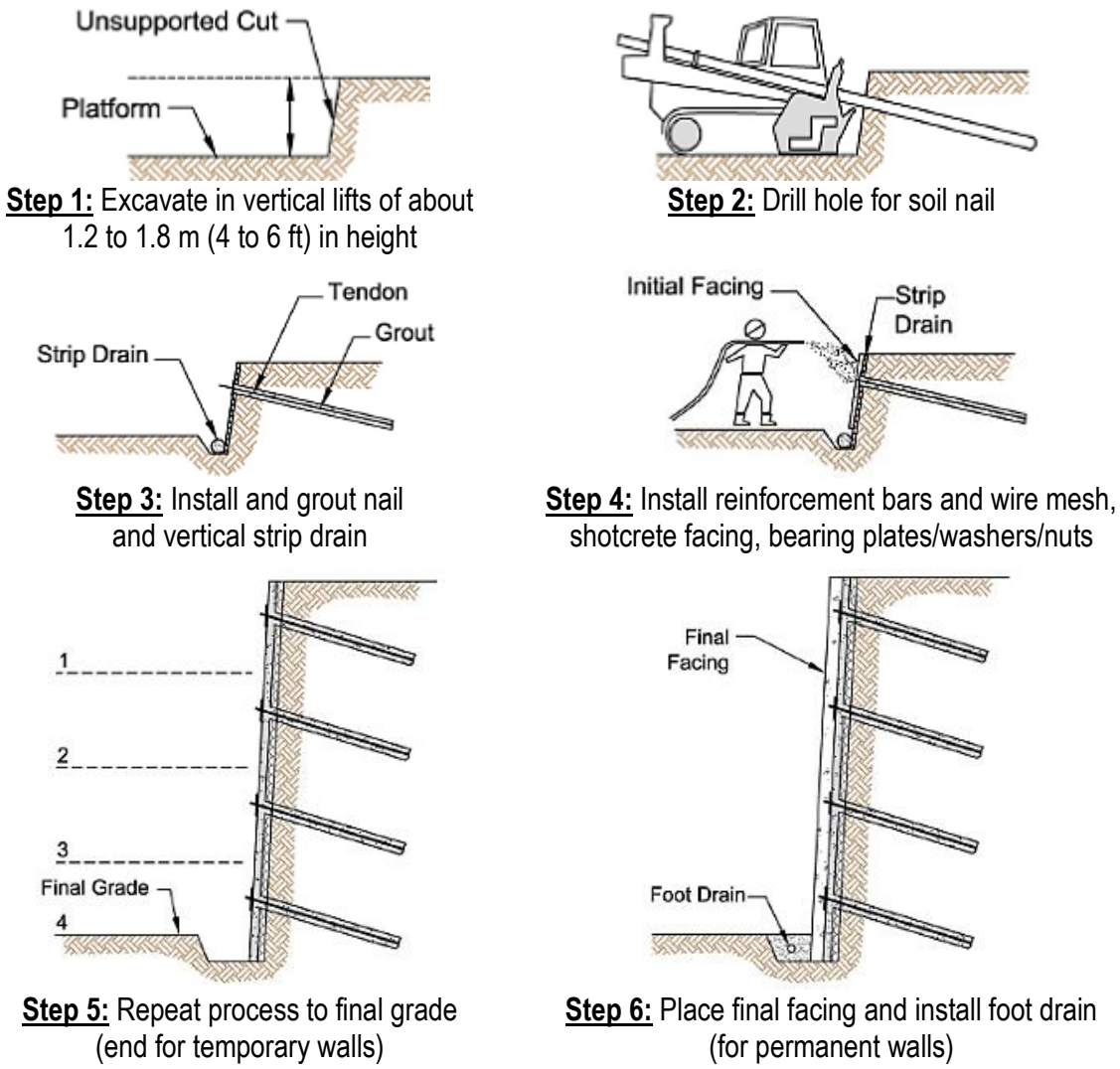


Fig. 29. General sequencing of the construction of soil nail walls (mod. Lazarte et al, 2014)

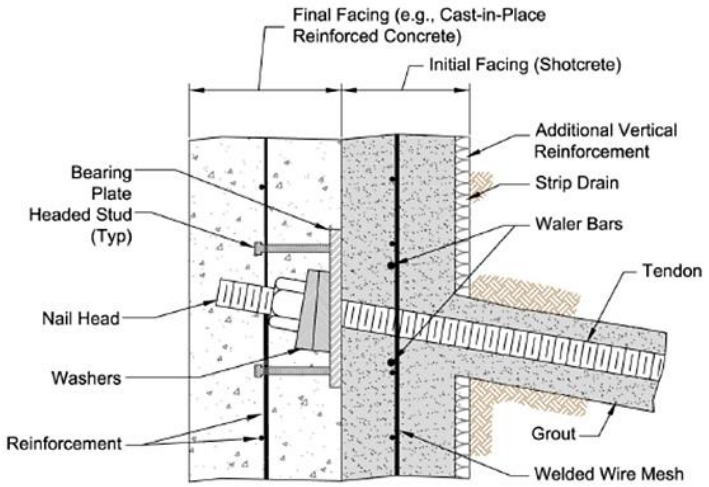
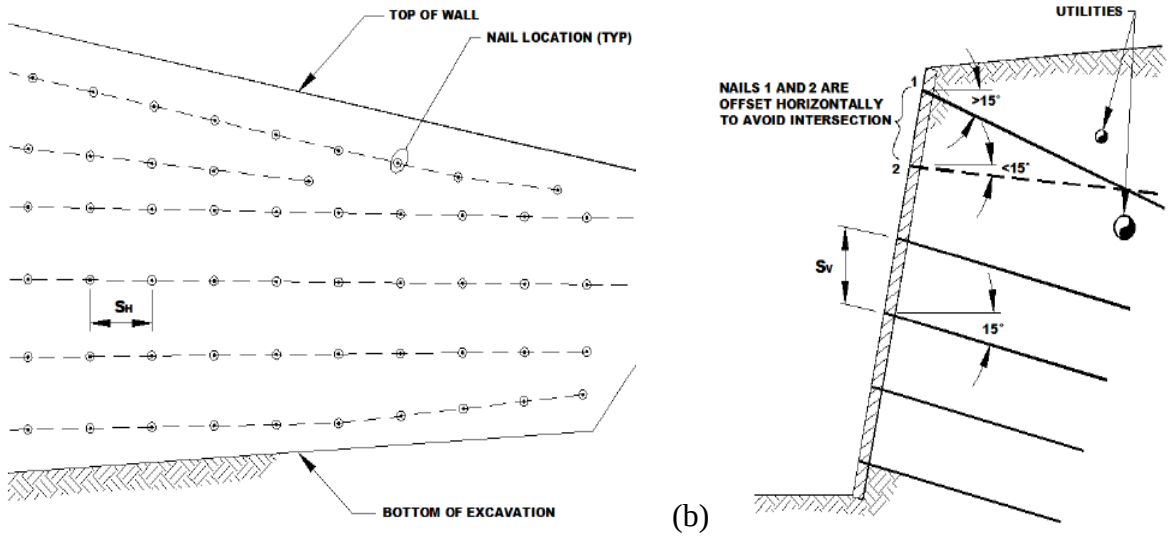


Fig. 30. Cross-section of typical components of a soil nail wall (Lazarte et al, 2015)

2. Perform the site investigation and characterization and any in-situ and laboratory testing to evaluate the subsurface conditions and determine the geotechnical properties of the soil and rock for design. Define the appropriate level of corrosion protection for the soil nails based on site conditions, aggressiveness of the ground conditions, design life, and tolerance to risk exposure.
3. Establish the appropriate loads acting on the soil nail wall (i.e., earth and surcharge pressures) due to the applicable load combinations.
4. Establish soil nail configuration and cross-sections (Fig. 31). The designer must define the arrangement of the soil nails vertically and horizontally such that the center-to-center spacing between soil nails in either direction is about 1.2 to 1.8 m (4 to 6 ft). The soil nails can be arranged in a square or staggered (i.e., adjacent rows are offset by ½ the horizontal spacing between soils). Known obstructions and utilities must be avoided by steepening the inclination of and/or splaying horizontally the soil nails. Typically, soil nails are installed subhorizontally at an angle between 10° and 20° degrees. The designer must select the proper corrosion protection in accordance with the specifications and/or according to the aggressiveness of the ground conditions at the project site. ASTM A615 Grade 420 (Gr. 60) and Grade 517 (Gr. 75) (ASTM A615) have been used for soil nails; however, Grade 517 (Gr. 75) is the more commonly used material type.



(a) (b)

Fig. 31. Examples of (a) soil nail patterns for non-horizontal ground and (b) cross-section of a soil nail wall with nearby utilities (Lazarte et al, 2015)

5. Select the appropriate resistance factors to be used in the design of the soil nail wall. Resistance factors for the design of soil nail walls for various service and strength limit states (e.g., overall stability, pullout resistance, lateral sliding, and nail tensile resistance) are provided in Table 6.3 of Lazarte et al (2015).

6/7. Once the preliminary geometry of the wall and the arrangement and spacing of the soil nails has been determined, the soil nail wall needs to be evaluated for the various stability concerns for each excavation phase of the wall height (Fig. 32). In addition, the structural and geotechnical strength limit states must also be evaluated. The stability evaluations are typically performed automatically (or by selection) when using commercially available software:

external stability (Fig. 33), internal stability (Fig. 34), and potential facing failure modes (Fig. 35). The limit states that need to be evaluated during these analyses include pullout resistance, sliding stability (if applicable), nail tensile resistance, facing bending/flexural resistance, facing punching shear resistance, and facing headed stud resistance. If necessary, revise the soil nail dimensioning and sizing until the external and internal stability are acceptable.

8. Evaluate the service limit states (i.e., lateral deformation and vertical displacement of the soil nail wall) and estimate the settlement of the ground surface behind the wall.
9. If located in a seismically active area, perform a seismic design and analysis in accordance with the requirements applicable for the local seismicity zone.
10. Evaluate the internal drainage and surface water runoff and develop details to ensure hydrostatic conditions and pressures do not act on the soil nail wall.

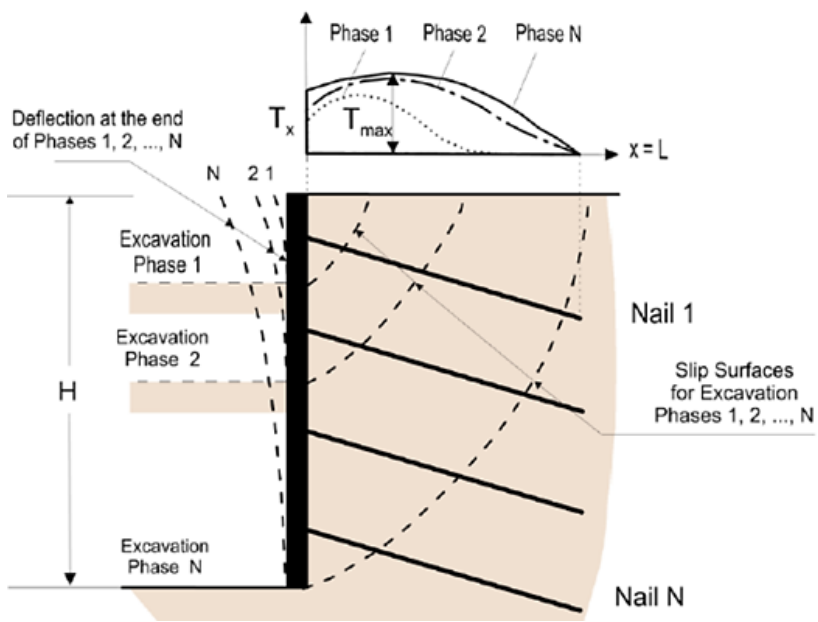
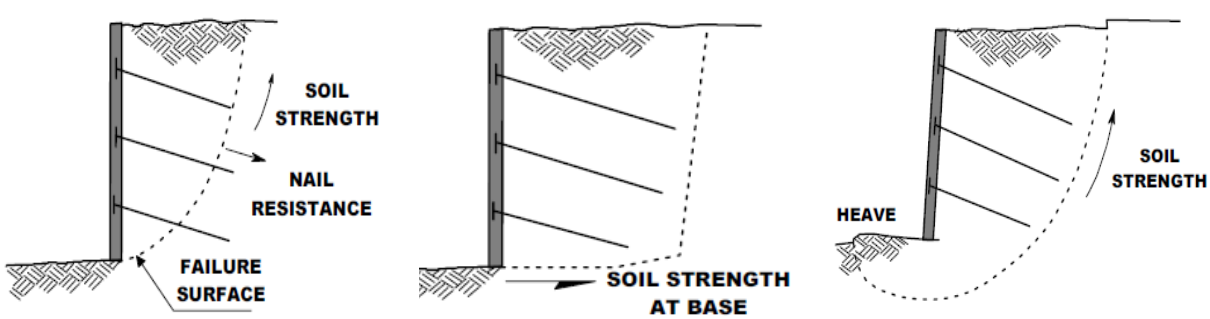


Fig. 32. Conceptual soil nail behavior - potential slip surfaces and soil nail tensile forces (after Lazarte et al, 2015)



(a) (b) (c)

Fig. 33. Examples of external stability concerns: (a) global stability, (b) sliding stability, and (c) bearing capacity / basal heave (after Lazarte et al, 2003)

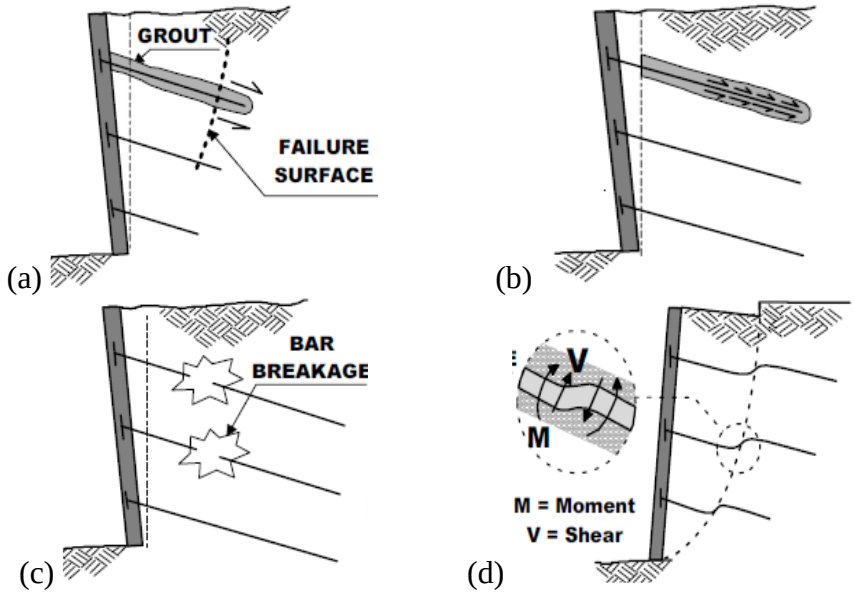


Fig. 34. Examples of internal stability concerns: (a) nail-soil pullout, (b) bar-grout pullout, (c) nail tensile resistance, and (d) nail bending and/or shear (after Lazarte et al, 2003)

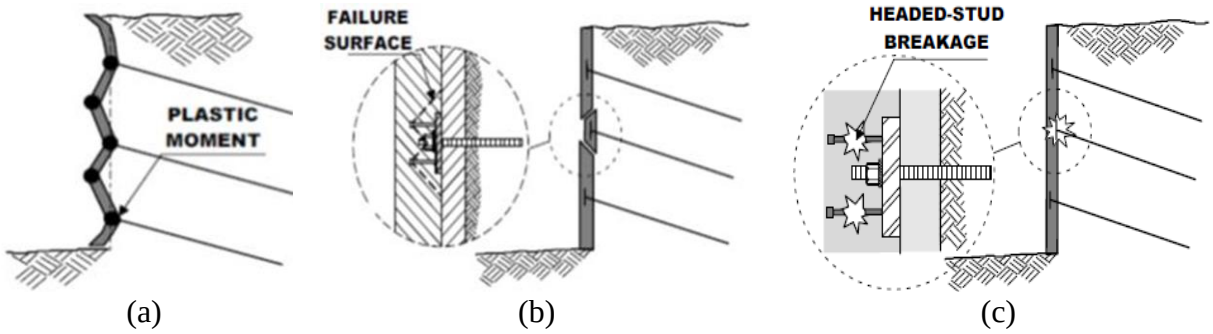


Fig. 35. Examples of facing stability concerns: (a) facing flexural resistance, (b) facing punching shear, and (c) headed-stud in tension (after Lazarte et al, 2003)

8. PERFORMANCE MONITORING

As projects have become more complex with increasingly greater risk, the safeguard and management of an owner's hard and soft assets (e.g., financial resources, real estate, structures, roadways, etc.) has garnered greater attention. Historically, it has been common that owners and contractors had visual surveys performed to ensure conformance to the requirements mandated in the project specifications, mainly for payment purposes. Nowadays, engineers and contractors are utilizing more sophisticated methods and instruments to observe, measure, and monitor the structures, utilities, ground conditions, etc. at a project site to ensure conformance with the specifications as well as to monitor the performance of the constructed facility to validate and/or improve predictions and understand risk exposure for future projects.

Various types of instrumentation are utilized to measure loads/pressures (e.g., load cells, tactile pressure cells, soil pressure cells, and fiber optic sensors, Fig. 36), movements and deformations (e.g., optical/visual surveys, strain gauges, inclinometers, and tiltmeters, Fig. 37), and groundwater conditions (e.g., stand pipes and piezometers). The ground response has typically been monitored

using optical survey points, inclinometers, extensometers, and piezometers, whereas the AER wall and adjacent structures have been monitored using optical surveys, strain gages, load cells, and tiltmeter.

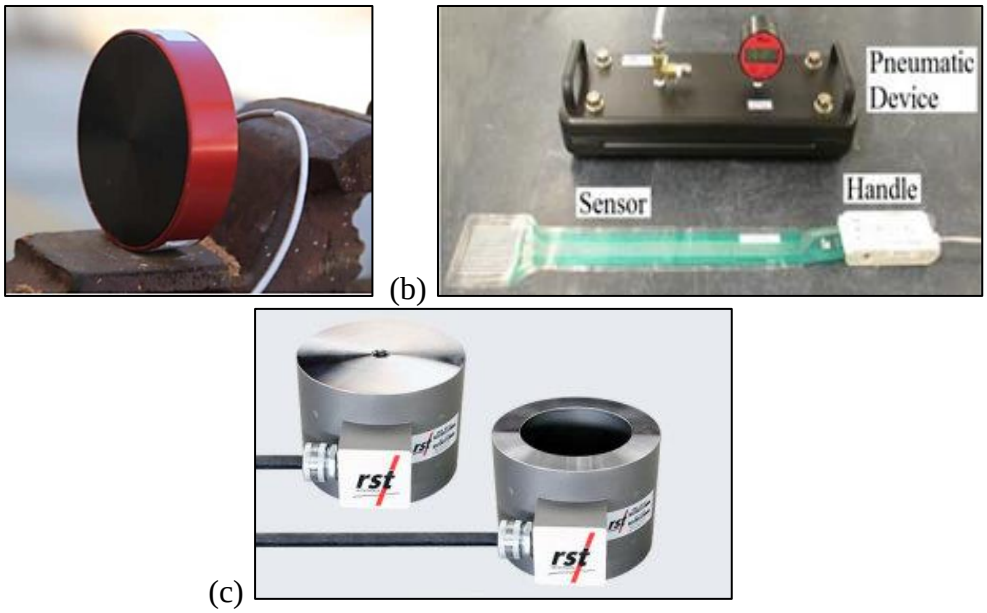


Fig. 36. Instrumentation for measurement of loads and pressures: (a) soil pressure cell (Keykhosropour et al, 2018), (b) a tactile pressure sensor (Gillis et al, 2015), and (c) two different types of load cells (www.geonor.com)

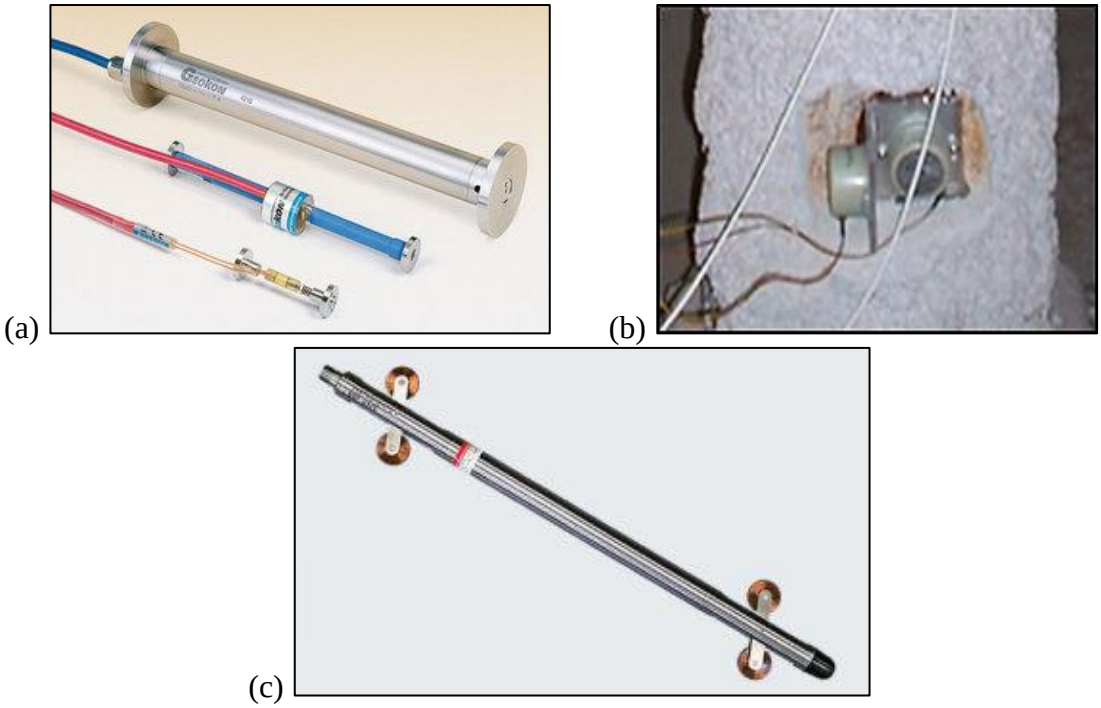


Fig. 37. Instrumentation for deformation and movement monitoring: (a) concrete embedment strain gauges (www.geokon.com), (b) tiltmeter attached to a column (Finno, 2016), and (c) an inclinometer (www.geonor.com)

Understanding the distribution and magnitude of earth pressures resulting from self-weight and applied loading is critical to the design and performance of AER wall systems. Earth pressures develop or vary due to natural, environmental, or man-made conditions (e.g., increases in vertical and horizontal stresses due to equipment loading and decreases in vertical stress due to excavation of soil). The use of instrumentation, especially with automated and wireless capabilities, can provide real-time monitoring and evaluation, data to use with the observational method, and as a means of forewarning in the event the system or a component is not functioning as intended. In addition to the instrumentation integrated into the construction equipment performing the work, the elegance of modern technology is that different devices are available to monitoring just about anything of concern on or around a project site, and it can all be done safely and remotely.

The mini-case history presented hereinafter provides an example where instrumentation was used to monitor performance and to evaluate the accuracy of predictive models.

8.1 CASE 1 - BRACED SECANT PILE WALL

Maniscalco and Ieronymaki (2018) present a case study of the performance of a reinforced concrete secant pile wall in New York City, N.Y. The rigid reinforced concrete secant pile wall was used to provide support to the in-situ soil and adjacent structure (Fig. 38), limit settlement of the ground surface and structure, and provide a groundwater cutoff within the excavation during the 13.1 m (43 ft) deep excavation. The heterogeneous subsurface consists of fill, sand, clay, decomposed rock, and rock with the groundwater table located at approximately 3.7 m (12 ft) from the ground surface (Fig. 39). The depth to the top of rock ranged from about 5.8 m to 20.1 m (19 ft to 66 ft) from the ground surface.

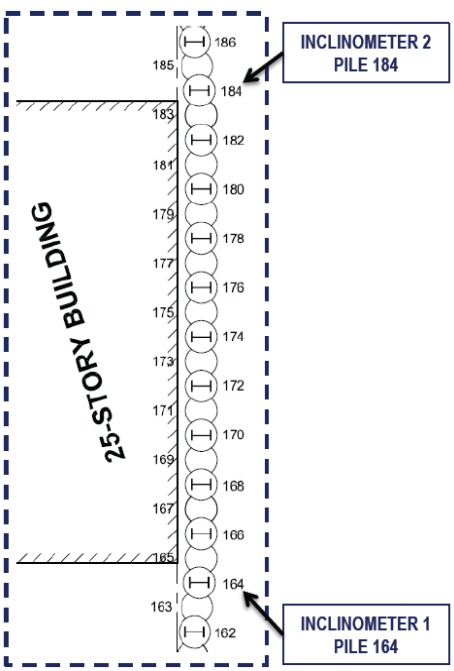


Fig. 38. Partial plan view of the west secant pile wall and inclinometers in near the existing 25-story building (Maniscalco and Ieronymaki, 2018)

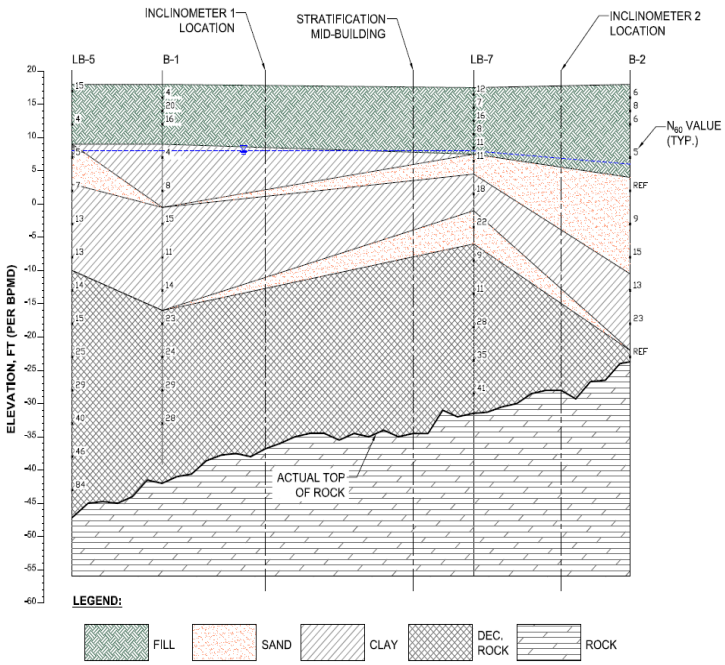


Fig. 39. Subsurface profile inferred from soil boring data at the west secant pile wall (Maniscalco and Ieronymaki, 2018)

A total of 192 secant piles were installed around the perimeter of the site, measuring about (38.1 m by 37.5 m [125 ft by 123 ft]), and were supported by three levels of steel bracing at the corners of the box-shaped excavation (Fig. 40). Each secant pile was about 1,000 mm (39.375 in) and 864 mm (34 in) in diameter in the overburden and rock, respectively, and was reinforced with a W610x195 (W24x131) steel beam and structural concrete with a (design) minimum 28-day compressive strength of 27.6 MPa (4,000 psi). However, results from compression tests performed on the concrete indicated that the compressive strength was greater than 51.7 MPa (7,400 psi).

The secant piles were constructed using temporary sectional steel casing (Fig. 41) through the overburden and seated into rock, and the soil was excavated using soil and rock augers and drilling buckets. Once the casing was seated into the rock, the tooling was changed and the rock socket was drilled using rock augers, core barrels, and a down-the-hole (DTH) hammer. The overlap between adjacent secant piles was 229 mm (9 in) and 92 mm (3.625 in) in the overburden and rock, respectively. The project requirements mandated that the secant piles be socketed a minimum of 600 mm (2 ft) into rock or 600 mm (2 ft) below the bottom of the excavation, resulting in the length of the rock sockets to vary from 600 mm to 7.6 m (2 ft to 25 ft) across the site. A reinforced concrete guidewall was constructed prior to any drilling.

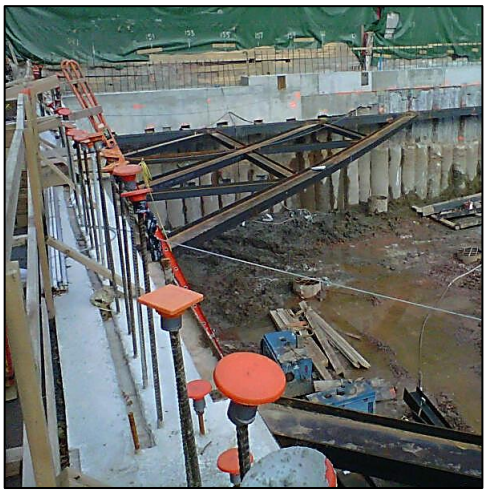


Fig. 40. View of the west secant wall at an excavated depth of 5.8 m (19 ft) with the first level of bracing installed (Maniscalco and Ieronymaki, 2018)

Fig. 41. Use of temporary sectional steel casing to install each secant pile (Maniscalco and Ieronymaki, 2018)

During the construction of the secant piles, inclinometers were installed in piles 164 and 184 in the west secant pile wall (Fig. 38). Once the inclinometers were operational, continuous measurements of the west wall were performed to monitor the lateral movement throughout the excavation and construction process (Fig. 42). The three levels of steel bracing were installed at depths, measured from the top of wall, of about 2.1 m (7 ft), 4.6 m (15 ft), and 8.5 m (28 ft).

As discussed by the authors in the paper, the objectives of the monitoring and research study were to (1) determine if the deflections of the secant pile wall could be predicted accurately using finite element analysis software (Plaxis) with the available data at the time of construction; (2) compare the predicted behavior using Plaxis with the actual measurements of wall deflection from the inclinometer data; (3) evaluate whether the complex 3D problem could be captured in a simplified

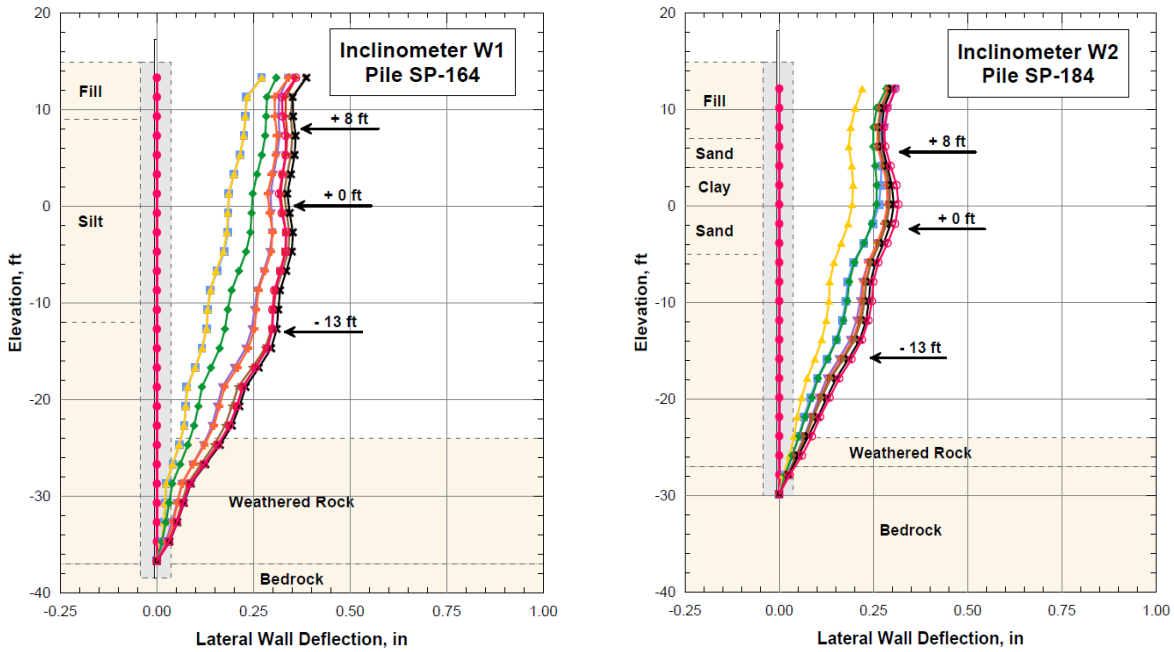


Fig. 42. Select measurement data from the two inclinometers in the west secant pile wall (Maniscalco and Ieronymaki, 2018)

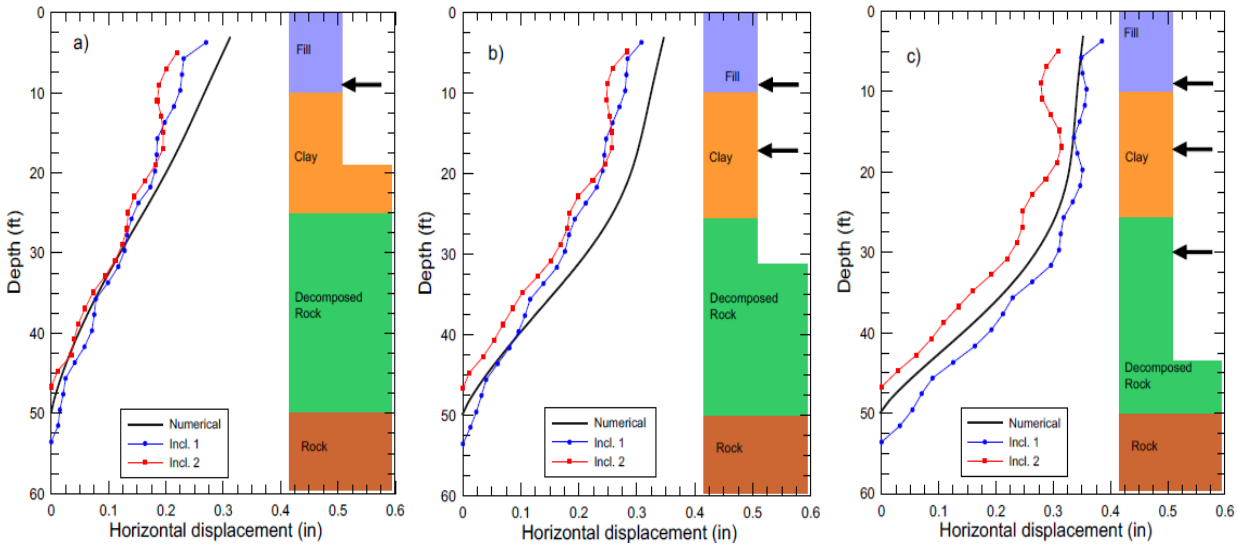
2D plane strain analysis; and (4) determine whether adequate engineering properties of the soil could be obtained using only the available SPT N-values and would they be adequate for the finite element modeling and analysis.

For the various phases/stages of the modeling, the excavation stages were considered to be 0.6 m (2 ft) below each bracing level as follows: [1] 3.4 m (11 ft) cantilever, [2] 5.8 m (19 ft) excavation depth with bracing at a depth of 2.7 m (9 ft), [3] 9.8 m (32 ft) excavation depth with bracing at depths of 2.7 m (9 ft) and 5.2 m (17 ft), and (4) 13.1 m (43 ft) excavation depth with bracing at depths of 2.7 m (9 ft), 5.2 m (17 ft), and 9.1 m (30 ft). For excavation stages below the water level within the secant wall, the groundwater table was assumed to be 0.6 m (2 ft) below the depth of excavation to account for the sequential dewatering of the excavation as construction proceeded. The results of the finite element modeling and comparison to the measurements from the inclinometers are presented in Fig. 43.

Based on the results of the retroactive finite element analysis and comparison with the measurements from the two inclinometers, a complex 3D real world problem was able to be modeled in a simplified 2D model with reasonable accuracy using SPT-derived soil properties, soil correlations and engineering judgement. Additional details can be found in Maniscalco and Ieronymaki (2018).

9. CONCLUSIONS

An anchored earth retention wall system is a system that stabilizes an unstable soil or rock mass using a systematic pattern of ground anchors or bracing in combination with a structural facing. To properly design and construct an AER wall system, the engineer needs to consider many variables, not all of which are technical in nature (i.e., political, environmental, and financial).



(a) (b) (c)

Fig. 43. Comparison of the results from the finite element modeling to the measurements from the two inclinometers in the west secant pile wall for the three levels of excavation/bracing (Maniscalco and Ieronymaki, 2018)

Project-related technical considerations include site constraints, subsurface conditions, design life and desired performance, availability and expertise of contractors, scheduling and sequencing, constructability and feasibility, and costs. However, many sources of technical design guidance and state-of-the-practice reports are available in the published literature. Even with all of the design theories, simplified design charts, and sophisticated 2D and 3D computer modeling software available, understanding the fundamental behavior, differences, applicability, and construction means of the different types of AER wall systems is critical to achieving the desired performance of the constructed work.

REFERENCES

AASHTO (2010) "AASHTO LRFD Bridge Design Specifications", Fifth Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

AnchorTest Ltd. (2015). "AnchorTest for iPad Software," London, U.K. www.anchortest.info

Barley, A.D. (1997). "The Single Bore Multiple Anchor System," *Proceedings of the International Conference, Ground Anchorages and Anchored Structures*, Institution of Civil Engineers (ICE), London, U.K., pp. 65-75.

Bruce, M.E.C., Berg, R.R., Collin, J.G., Filz, G.M., Terashi, M. and Yang, D.S. (2013). "Deep Mixing for Embankment and Foundation Support." FHWA-HRT-13-046, Federal Highway Administration, Washington D.C., 228p.

Carswell, W., Meggitt, H., and Siebert, D.R. (2019). "Excavation-induced Movement of Adjacent Foundations." *Proceedings of the DFI 44th Annual Conference on Deep Foundations*. Chicago, IL. Accepted for publication.

Clough, G. W., and O'Rourke, T. D. (1990). "Construction induced movements of in-situ walls." Proc. of Conf. on Design and Performance of Earth Retaining Structures Geotechnical Special Publication No. 25, Cornell University, Ithaca, P. C. Lambe and L.A. Hansen, eds., ASCE, New York, NY, pp. 439-470.

Dunnicliff, J. (1999). "Geotechnical Instrumentation for Monitoring Field Performance." John Wiley & Sons, Inc. New York, 579pp.

Finno, R. (2016). "Earth Retention: Geotechnical Design." Presentation delivered at the ADSC Foundation Engineering Faculty Workshop. Chattanooga, TN.

Finno, R. (2016). "Instrumentation and monitoring systems to manage risk associated with supported excavations." Presentation delivered at the ADSC Foundation Engineering Faculty Workshop. Chattanooga, TN.

Gillis, K., Dashti, S., and Hashash, Y.M.A. (2015). "Dynamic Calibration of Tactile Sensors for Measurement of Soil Pressures in Centrifuge," ASTM Geotechnical Testing Journal, Vol. 38, No. 3, pp. 261–274, doi:10.1520/GTJ20140184.

Kitazume, M., and Terashi, M. (2013). *The Deep Mixing Method*. CRC Press/Balkema, Leiden, The Netherlands.

Keykhosropour, L., Lemnitzer, A., Star, L., Marinucci, A., and Keowen, S. (2018). "Implementation of Soil Pressure Sensors in Large-Scale Soil-Structure Interaction Studies." ASTM Geotechnical Testing Journal, Vol. 41, No. 4, pp. 730–746, https://doi.org/10.1520/GTJ20170163.

Lazarte, C.A., Robinson, H., Gómez, J.G., Baxter, A., Cadden, A.W., and Berg, R.R. (2015). "Soil Nail Walls – Reference Manual." Geotechnical Engineering Circular (GEC) No. 7. Publication FHWA-NHI-14-007, Federal Highway Administration, Washington, D.C.

Lazarte, C.A., Elias, V., Espinoza, R.D., and Sabatini, P.J. (2003). "Soil Nail Walls." Geotechnical Engineering Circular (GEC) No. 7. Publication FHWA-IF-03-017, Federal Highway Administration, Washington, D.C.

Loehr, J.E., Lutenegger, A., Rosenblad, B., and Boeckmann, A. (2016). "Geotechnical Site Characterization." Geotechnical Engineering Circular No. 5. Publication FHWA-NHI-16-072, Federal Highway Administration, Washington, D.C.

Maniscalco, J.D. and Ieronymaki, E.S. (2018). "Performance and Analysis of a Braced Secant Pile Wall for a Multi-Story Building in Manhattan, N.Y." *Proceedings of the DFI-EFFC International Conference on Deep Foundations and Ground Improvement - Urbanization and Infrastructure Development: Future Challenges*. Rome, Italia. p.752-761.

Marinucci, A. and Mothersille, D. (2018). "Single Bore Multiple Anchor Systems (SBMAs) in Challenging and Variable Ground Conditions," DFI-India 2018: 8th Conference on Deep Foundation Technologies for Infrastructure Development in India, IIT Gandhinagar, Gujarat, India.

Mothersille D., (2010). "Ground Anchoring – The Current State of the Practice and Recent Developments." Presentation delivered at the ADSC Anchored Earth Retention Seminar. Boston, MA.

Mothersille, D., Düzceer, R., Gökalp, A., and Okumuşoğlu, B. (2015). "Support of 25 m Deep Excavation using Ground Anchors in Russia," *Proceedings of the Institution of Civil Engineers, Geotechnical Engineering*, Vol. 168, pp. 281-295.

Mothersille, D. and Okumusoglu, B. (2016). "Anchor Testing using Innovative Software on a Tablet." Technical Feature, *Foundation Drilling* Magazine of the ADSC-IAFD, October, pp. 23-35.

Mothersille D., Düzceer R., Gökalp A., and Adatepe, S. (2018). "Design, Construction, and Performance of Single Bore Multiple Anchored Diaphragm Wall in Izmir, Turkey."

Proceedings of the International Conference on Deep Foundations and Ground Improvement Urbanization and Infrastructure Development – Future Challenges. DFI-EFFC, Rome, June 5-8. pp. 135-148.

Ostermayer H. and Barley A.D. (2003). "Fixed anchor design - Ground Anchors." Geotechnical Engineering Handbook, vol. 2, Pub Ernst and Sohn, Berlin, Germany, 189–205.

Porterfield, J.A., Cotton, D.M., and Byrne, R.J. (1994). "Soil Nailing Field Inspectors Manual, Project Demonstration 103." Publication FHWA-SA-93-068, Federal Highway Administration, Washington D.C.

Post Tensioning Institute (PTI). (2014). Recommendations for Prestressed Rock and Soil Anchors. DC351-14. Farmington Hills, MI. 110 pp.

Sabatini, P.J., Pass, D.G., and Bachus, R.C. (1999). "Ground Anchors and Anchored Systems." Geotechnical Engineering Circular No. 4. Publication FHWA-SA-99-015, Federal Highway Administration, Washington, D.C.

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, J.A., and Zettler, T.E. (2002). "Evaluation of Soil and Rock Properties." Geotechnical Engineering Circular No. 5. Publication FHWA-IF-02-034, Federal Highway Administration, Washington, D.C.

Sabatini, P.J., Tanyu, B., Armour, T., Groneck, P., and Keeley, J. (2005). "Micropile Design and Construction Reference Manual." Publication FHWA-NHI-05-039, Federal Highway Administration, Washington, D.C.

Schaefer, V.R., Berg, R.R., Collin, J.G., Christopher, B.R., DiMaggio, J.A., Filz, G.M., Bruce, D.A., and Ayala, D. (2016). "Ground Modification Methods – Reference Manual, Volume I." Geotechnical Engineering Circular No. 13. Publication FHWA-NHI-16-027, Federal Highway Administration, Washington, D.C.

Schaefer, V.R., Berg, R.R., Collin, J.G., Christopher, B.R., DiMaggio, J.A., Filz, G.M., Bruce, D.A., and Ayala, D. (2016). "Ground Modification Methods – Reference Manual, Volume II." Geotechnical Engineering Circular No. 13. Publication FHWA-NHI-16-028, Federal Highway Administration, Washington, D.C.

Tanyu, B.F., Sabatini, P.J., and Berg, R.R. (2008). "Earth Retaining Structures." Publication FHWA-NHI-07-071, Federal Highway Administration, Washington, D.C.