EARTH RETENTION SYSTEMS
TEMPORARY AND PERMANENT

by
Thomas C. Anderson

MEETING REPRINT
EARTH-RETENTION SYSTEMS--TEMPORARY AND PERMANENT

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I. Evolution of Earth-retention Systems

Where space does not permit an open excavation with unsupported slopes, earth-retention systems are necessary in order to make the sides of an excavation vertical or near vertical. There has been a tremendous evolution of earth-retention systems in the past few decades. The systems have progressed from internally-braced sheeting to temporary tiedback sheeting to permanently tiedback wall systems.

In the 1950s, contractors began using tiebacks to temporarily support the sides of deep excavations. The use of tiebacks to support sheeting walls results in excavations uncluttered by braces and, in many cases, a faster or less costly project. A comparison between raker and braced sheeting and tiedback sheeting is shown in Figure 1.

FIGURE 1. EXCAVATION SUPPORT

1a. Raker and Braced Sheet  ing

1b. Tiedback Sheet  ing

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Braces interfere with and increase the cost of the construction which follows. We also have found that, as the height of the wall increases, the cost of the bracing increases. For heights above thirty feet, tiebacks are less expensive than braces. Using tiebacks in lieu of braces reduces the excavation cost, the concrete cost, and frequently, the actual cost of the excavation-support system. The cost-effectiveness of tiedback sheeting is proven by its wide-spread use.

Permanent tiebacks have been used to support a variety of structures in Europe since the mid-1960s and in the United States since the early 1970s. Permanently tiedback walls are especially applicable and cost-effective for situations where space is a problem and in cuts where temporary sheeting is required to construct a conventional wall system.

II. Temporary Tiedback Sheetung

The history of tiedback sheeting in the Metropolitan Minneapolis-St. Paul area dates back to the late 1960s and early 1970s when the Federal Reserve Bank, Midwest Federal Parking Ramp, and IDS Building were constructed. Recent projects in the late 1970s and early 1980s have included the Pillsbury Corporate Office Building, the Block 40 Lowertown in St. Paul, and expansions of the University of Minnesota Hospital and Civil/Mineral Engineering Building.

The typical tiedback sheeting system used on these projects is a soldier beam and wood lagging system (see Figure 2). Other systems include steel sheet piling and slurry walls, but the soldier beam and wood lagging system is the most economical. Typically, soldier beams are either driven or installed in predrilled holes at six to eight feet on center, depending upon soil conditions, depth of cut, and design. H-piles are employed if the soldier beams are driven, whereas wide flanges or other structural shapes can be utilized if beams are placed in predrilled holes. Pre-drilling is used to reduce noise and vibrations, to penetrate hard or boulder layers to ensure adequate toe penetration, and to set soldier piles at precise locations for wall-line projects.

Wood lagging retains the earth between the soldier beams. Three-inch-thick mixed hardwoods normally are used for the lagging and they either are placed behind the front flange of the soldier beams or attached to the front face with studs and plates or with clips for the contact sheeting method. Louvers of one-to-two-inch vertical spaces between the lagging boards are considered good practice so as to allow for seepage and prevent building of water pressures. Straw is placed as necessary to act as a filter to prevent loss of material from seepage.

In the Metropolitan Minneapolis-St. Paul area, the principal type of tieback is the pressure-injected tieback which develops its capacity in the dense sand and gravel deposits. Rock tiebacks have been used on a few projects, such as the Midwest Federal Parking Ramp. Typically, a double-channel wale connects the tiebacks to the soldier beams.
III. Permanently Tiedback Wall Applications

Permanent tiedback systems offer the engineer a technique for solving a variety of structural problems. The following text and figures describe applications for permanently tiedback walls and the advantages they offer over conventional wall construction. Other applications of permanent tiebacks include basement slab, tower, and dam tiedowns.

A. Retaining Walls

In comparison to conventional retaining walls, permanently tiedback walls completely eliminate foundation piles, large footings, and backfill (see Figure 3). In addition, the quantity of excavation and concrete are reduced when a tiedback wall is used. The savings resulting from elimination or reduction of these items far exceeds the cost of installing the permanent tiebacks, in many situations. Generally, in cut situations where temporary sheeting is required for conventional wall construction, permanently tiedback walls will be the most economical solution.
FIGURE 3.

COMPARISON BETWEEN CONVENTIONAL RETAINING WALL AND PERMANENTLY TIEBACK WALL

3a. Conventional Retaining Wall

3b. Permanently Tiedback Wall
A similar application is that of depressed roadway construction where retaining walls are required on both sides of the roadway. As seen in Figure 4, tiedback walls do not require wide construction easements. Only a narrow temporary easement is necessary for the construction of permanently tiedback walls, since footings are not required.

**FIGURE 4.**

**DEPRESSED ROADWAYS**

4a. Conventional Retaining Wall

4b. Permanently Tiedback Wall
Permanent tiebacks also are used to stabilize or repair existing retaining walls which are overturning or founded on soil or rock which is part of a slide. Figure 5 shows a permanent tieback and pile system used to save a failing wall. Normally, an existing wall can be stabilized with permanent tiebacks at a fraction of the cost of constructing a new wall and without disturbing the rock or soil behind the wall.

**FIGURE 5.**
**WALL REPAIR**

5a. Unstable Wall Would Be Replaced

5b. Permanent Tiebacks Used to Stabilize Existing Wall
B. Bridge Abutments

Permanent tiebacks enable bridge abutments to be built from the top down. Figures 6 and 7 show the comparison between conventional and tiedback bridge abutments, along with the sequence used to construct tiedback bridge abutments.

**FIGURE 6.**
BRIDGE ABUTMENTS

6a. Conventional Bridge Abutment

6b. Permanently Tiedback Bridge Abutment
FIGURE 7. CONSTRUCTION PROCEDURE
FOR A TIEBACK
BRIDGE ABUTMENT

7a. Plan of Completed Bridge

7b. Installation of Abutment
Soldier Piles

7c. Installation of Beam
Seats and Deck
First, drilled piers are constructed from existing grade. Drilled pier walls cause only minimal disturbance and they are economical to install with readily available drilling equipment. After piers are installed, beam seats are poured, beams are placed, and the deck is constructed. Upon completion of the deck, traffic can be restored without restrictions. Once the deck is built, excavation proceeds without interruptions.

Tiebacks are then installed, tested, and locked off as the excavation is completed to final subgrade. The final step is the pouring of a cast-in-place wall against the tiedback pier system.

Permanently tiedback bridge abutments offer several advantages:

- the bridge can be built in segments while traffic is maintained;
- the bridge abutments do not require footings for bending resistance (footing construction would disrupt traffic on nearby roadways that run perpendicular to the bridge);
- the tiedback abutment wall functions as the temporary earth support;
- the quantity of excavation is reduced; and
- the backfill is eliminated.

Permanently tiedback bridge piers must bear on soil or rock capable of carrying the traffic and bridge loads. This can be accomplished by extending the piers to a bearing stratum located below subgrade.
Permanently tiedback walls also can be used to widen roadways under existing bridges. Figure 8 shows a comparison between conventional and tiedback wall construction for this application. Depending on the existing abutment construction, the permanently tiedback wall can be designed to support the fill behind the abutment or designed to underpin the abutment and support the fill behind it.

**FIGURE 8.**

**WIDENING ROADWAY UNDER EXISTING BRIDGE**

8a. Conventional Method

8b. Method Using Permanently Tiedback Wall
Many old bridge abutments are deteriorating or becoming unstable under increased traffic or loads. In addition, the clearance under a bridge may need to be increased to accommodate greater vehicle height. Permanent tiebacks can be used to increase the overturning and sliding resistance of gravity walls and abutments and allow the height of the wall to be increased. If tiebacks are not used, the abutments would have to be replaced. Figure 9 shows a situation where permanent tiebacks were used to strengthen a bridge abutment to allow the railroad tracks to be lowered.

**FIGURE 9.**
**ABUTMENT REPAIR**

9a. Old Bridge Abutment Would Require Replacement

9b. Permanent Tiebacks Used to Strengthen Bridge Abutment
C. Unbalanced Lateral Pressure

Figure 10 shows permanent tiebacks used to support unbalanced earth pressures which result when a building is constructed on a sloping site or into a hillside. A normal building foundation is not designed to resist these forces. When designing tiedback walls of this type, care must be taken to ensure that the wall and the building can accommodate relative movements. If the wall is rigidly connected to the structure, relative movements could cause damage. A separate retaining wall may be built in order to prevent wall deformations from affecting the building, or the building can be designed to accommodate the movements. This application is illustrated in the permanently tiedback wall constructed along the cliff for the Adult Detention Center for Ramsey County in Downtown St. Paul.

**FIGURE 10.**

**UNBALANCED LATERAL PRESSURES**

10a. Structure Designed to Resist Unbalanced Lateral Pressures

10b. Permanent Tiebacks Used to Resist Unbalanced Lateral Pressures
D. Landslide Stabilization

Permanent tiebacks have been used effectively to stabilize or to prevent landslides. Permanently tiedback walls can be constructed to stabilize cut and fill slides associated with highway and railroad construction. Often these walls can be built without disrupting traffic on the existing roadway or railroad. Permanently tiedback walls also are used to prevent slides from occurring, as well as allow structures to be built into a potential slide area. As shown in Figures 11 and 12, a landslide may develop when an excavation is made into a hillside. Tiedback walls will enable the maximum development of these types of sites. In landslide applications, the permanent tiebacks extend below the failure surface and provide the force required for equilibrium.

**FIGURE 11.**

**SITE DEVELOPMENT SLOPE STABILIZATION**

11a. Excavation Could Cause a Landslide and Site Normally Would not be Developed

11b. Permanent Tiebacks Allowed this Site to be Developed
FIGURE 12.
SLOPE STABILIZATION
MERCY HOSPITAL
SCRANTON, PENNSYLVANIA

12a. Excavation Could Cause a Landslide and Site Normally Would not be Developed

12b. Permanent Tiebacks Allowed this Site to be Developed
E. Waterfront Walls

Permanent tiebacks are used to support walls constructed along rivers, lakes, and streams (see Figure 13). Using permanent tiebacks to anchor the wall offers advantages over conventional deadmen anchors, e.g., deadmen, excavating, tie-rods, and backfilling all are eliminated when permanent tiebacks are used. Permanent tiebacks are installed without disturbing the existing ground surface, thus allowing normal operations behind the wall to be maintained. If pile-supported deadmen anchors or uninterrupted access are required, permanent tiebacks normally will be an economical support system for waterfront walls.

**FIGURE 13.**
WATERFRONT WALLS (BULKHEADS)

13a. Deadman-anchored Bulkhead

13b. Tiedback Bulkheads
IV. Design of Tiedback Earth-retention Systems

The various aspects of the design of tiedback earth-retention systems will be discussed in the following paragraphs. Due to space limitations, the design of landslide control walls will not be considered, although the design considerations (with the exception of the landslide restraint force) basically are the same as for other permanently tiedback walls.

A. Loading Diagrams and Behavior

1. Loading Diagram

The first step in the design process is the selection of the earth pressure. The tiedback wall systems discussed thus far all seem to meet the Terzaghi and Peck criteria for use of the "apparent earth pressure". These walls and tiebacks are installed from top to bottom as excavation proceeds. This is the condition (explained by Terzaghi and Peck) which caused the pressure redistribution against the wall.

Schnabel Foundation Company uses the earth pressure envelope (see Figure 14) for the design of most tiedback walls. It is very similar to the 1948 recommendations of Terzaghi and Peck. One difference is that this diagram is used for certain sand, clay, or mixed soils without distinguishing between them. This is a direct result of supporting instrumentation data and the quality of the workmanship. For reasonable soil values, there is close agreement regarding the maximum unit pressure and total pressure against the wall between Schnabel's diagram and Terzaghi and Peck's diagram (see Figure 14 for comparison). Schnabel's diagram is an empirical earth pressure distribution, based on measuring actual jobs, and it is used to design similar projects in similar soils. This diagram is not used blindly in softer soil or soils which may exert greater pressures. In addition, if unusual surcharges or water exert pressure on the wall, these must be added to the earth pressure.

FIGURE 14. COMPARISON OF SEVERAL "APPEARANT EARTH-PRESSURE" ENVELOPES

14a. Schnabel

14b. Terzaghi and Peck sand

14c. Terzaghi and Peck clay
Virtually, all of these pressure diagrams are based on strut or braced-load measurements and actually are envelopes to cover the scatter of data due largely to variations in construction techniques. Measured-brace loads varied from the average measured load by as much as 30 percent, and this seemed to be the result of workmanship and other factors over which the designers had no control. The total force calculated using these envelopes is about 30 percent greater than the active earth pressure, because the envelope was increased to provide for this random variation of individual brace loads above the average. Therefore, an envelope with small pressures may seem reasonable when tiebacks are used instead of braces for the reasons listed below. This concept deserves more study.

a. Tiebacks usually are tested to at least 120 percent of design load; then the load is reduced and locked off at about 75 percent or more of the design load. This process removes some of the anomalies causing load variations that are inherent with braces, which may or may not be preloaded. Therefore, it is believed that the tieback installation procedure leads to better load control and load distribution.

b. Typically, observational data on tiebacks do not show a gain in load as the excavation is deepened; rather, tiebacks generally hold the load constant or decrease slightly. Thus, in contrast to bracing which usually shows a gain in load as the excavation is advanced further, the lateral force is predetermined by tieback preload and lock-off.

2. Behavior

Excavation always involves removal of material and, consequently, causes a change in the state of stress in the soil or rock beneath and beside the excavation. Since no material can experience stress change without corresponding deformations, excavation is always associated with settlement of the adjacent ground surface. On the other hand, a properly designed and carefully constructed earth-retention system can materially reduce the change in lateral pressure in the material adjacent to the excavation and thus is capable of reducing the settlement to a value that may be regarded as a practicable minimum for a given job.

Regarding behavior of earth-retention systems, a well-constructed tiedback wall usually will exhibit less lateral displacement than an internally-braced wall. Figure 15 (after O'Rourke and Cording, 1974) shows a summary of settlements adjacent to two braced cuts and one tiedback excavation in Washington, DC. The indicated settlements are representative for competent soils and show that the tiedback excavation exhibited less movement than the braced cuts. There are several reasons for this:

- Each tieback is tested to an overload and then locked off at 75 percent to 100 percent of the design load. Thus, tiebacks tend to remove more "slack" from the support system than bracing;
tiebacks extend a considerable distance behind the wall and
prestress a block of soil. In such a process, the earth mass
engaged by the tieback tends to deform as a unit. The composite
soil-wall system is less deformable than a conventional braced
wall acting only on the face of the cut;

excavators tend to dig farther below a brace level than a tie-
back level to facilitate removal of earth;

temperature strains have more of an effect on bracing; and

movement from brace removal and re-strutting is eliminated.

**FIGURE 75.**

**SUMMARY OF SETTLEMENTS ADJACENT**

**TO SUPPORT CUTS IN COMPETENT SOILS**

With regard to the difference in behavior between flexible soldier
pile walls and stiffer slurry walls, experience, as well as finite-
element studies, has demonstrated convincingly that the lateral
movements of the walls and the corresponding settlement of the
ground behind them depend far more on the spacing and stiffness of
the supports and on the depths of excavation relative to the
placement of support than on the inherent stiffness of the wall.

**B. Stability**

Two aspects that must be considered in the design of tiedback wall
systems are wall stability and mass stability.
1. Wall Stability

In order to be stable, the wall must be strong enough to confine the excavation when acted upon by the appropriate tieback forces. The unique feature of tiedback walls is that the tiebacks act to resist all or most of the earth pressure. The walls are designed to support the earth pressure between tiebacks. The usual design procedure is to consider each span between tiebacks (except the top) as a simple span. Even though most walls penetrate below subgrade, it is assumed that no pressure acts on the wall below subgrade when using the envelopes shown in Figure 14. As far as the wall facing is concerned, it could be H-beams and lagging, a slurry wall, reinforced concrete, tangent piles, masonry, steel sheet piling, or anything strong enough to resist the forces. As demonstrated in Figure 16, the distance between the tieback anchor and structure is an important part of the design.

FIGURE 16.
EXAMPLE OF SOIL IN WHICH ANCHOR SHOULD NOT BE PLACED
ANCHOR SHOULD NOT BE MADE IN SOIL REPRESENTED BY THE SHADED AREA

Tiebacks should not develop any of their capacity in the soil between the critical failure surface and the wall. Clearly, any capacity the anchor developed in this soil would be reduced if the wall moved. This is prevented by specifying the length of tieback which must be unbonded.

Tiebacks usually apply a downward force on the wall. Walls have failed as a result of not considering this condition; hence, this force should not be ignored in the design of the wall. Wall friction acts to decrease the downward component of the tieback force. Normally, in competent soils, axial loading of the wall is not considered until the angle of the tieback from the horizontal exceeds $\phi$ (angle of internal friction). When steeper tiebacks are necessary, the wall should be designed to develop resisting capacity below subgrade and carry this downward component to a depth where suitable bearing capacity can be achieved.
2. Mass Stability

Tiebacks tie the wall to a wedge of soil which must be internally and externally stable. If the wall is properly designed and the tiebacks have the capacity for which they are designed, the wall pressures and tiebacks create internal forces in the wedge. The wedge also must be checked for external forces to verify its stability. The shape of this mass is fixed by the location of the tieback anchors, so a stable mass can be assured by properly locating the anchors. As shown in Figure 17, the tiebacks are an internal, unifying force in the wedge. The four sides of this wedge are the wall, the ground surface, a vertical plane through the ends of the tiebacks, and the weakest surface through the soil between the wall and the ends of the tiebacks. Obviously, the easiest way to increase mass stability is by increasing the length of tiebacks.

FIGURE 17.
MASS ANALYZED FOR STABILITY
C. Tiebacks

A tieback is a structural element which uses a grouted anchor in the ground to secure a tendon which applies a force to a structure. Figure 18 shows the components of a tieback. The anchor length is the portion of the tieback where the force is transmitted to the ground. The portion of the tendon between the anchor and the structure is not bonded to the ground, and it is free to elongate elastically. Force is applied to the tieback by post-tensioning.

**Figure 18.**
**Components of a Tieback**

![Diagram of a Tieback](image)

\[ l_a = \text{Anchor Length} \]
\[ l_t = \text{Total Length} \]
\[ l_{fb} = \text{Unbonded Length} \]
\[ d = \text{Anchor Diameter} \]

1. Preliminary Considerations

First, it is necessary to determine whether or not tiebacks can be used at a particular site. This decision is based on the following considerations:

a. Feasibility of Installing Tiebacks

The presence of utilities, subways, or other underground structures may make it impossible for tiebacks to be installed. Normally, tiebacks can be installed at angles between horizontal and 45° from horizontal. If tiebacks can be installed, and they are permanent, it will be necessary for the owner to obtain permanent easements for the tiebacks or purchase the land above them.
b. Feasibility of Developing Tieback Capacity

Permanent tiebacks can be anchored in any rock, coarse-grained soils with SPT ≥ 10 blows/ft, and fine-grained soils with a liquidity index ≤ 0.2 and an unconfined compressive strength greater than 1.0 ksf. Permanent tiebacks are not recommended in fill material or soft clay, which may tend to creep with time.

It is very important that a comprehensive soils or geologic investigation be conducted during the design phase of the project in order to determine the in situ soil or rock properties. Borings should be taken where the tieback anchors will be made. Standard penetration resistances, Atterberg limits, and unconfined compression tests should be made when soils are encountered. When rock is encountered, continuous rock cores should be taken and RQDs recorded. Jar samples, rock cores, and drillers logs also should be made available to the tieback contractor.

c. Corrosiveness of the Soil

Since permanent tiebacks are designed to function for an extended period of time, they should not be installed in extremely aggressive environments without proper corrosion protection. Prestressing steels (either bars or high-strength strands) that are used for tieback tendons normally are subjected to high strains which make them susceptible to corrosion. As a result, corrosion protection should be provided for all permanent tiebacks; the selection should be based on resistivity, soil and groundwater pH, soluble sulfate content, and sulfide content tests.

d. Relative Economy of This System

The cost of the system must be determined in order to decide whether or not a particular type of tieback is cost-effective.

2. Types

Schnabel Foundation Company installs the following types of permanent tiebacks:

- Pressure-injected: used in relatively clean granular soils;
- Hollow-stem-augered: used in stiffer clays and mixed soils;
- Regroutable: used in softer soils; and
- Rock: used in any kind of rock.
3. Installation Procedures

a. Pressure-injected tiebacks are installed by driving and/or drilling a closed-end casing to the desired length. The tendon is then inserted into the casing. Next, the casing is extracted a short distance using centerhole hydraulic cylinders, and the closure point is driven free from the end of the casing. Grout is pumped down the casing while the casing is extracted. Grout pressures in excess of ≥ 150 psi are maintained until the entire anchor length has been grouted. See Figure 19.

FIGURE 19.
STEPS FOR PRESSURE-INJECTED TIEBACK INSTALLATION

19a. Drive Casing

19b. Insert Tendon and Disengage Point

19c. High-pressure-grout Anchor Length and Low-pressure-grout Unbonded Length While Extracting the Casing
b. Hollow-stem-augered tiebacks are constructed by first inserting the tieback tendon in the auger. A slip fit point is attached to the tendon and the point is inserted in the end of the auger. Next, the tieback hole is drilled to the desired depth, and upon completion, grout is pumped down the auger as the auger is extracted. See Figure 20.

**FIGURE 20.**
HOLLOW-STEM-AUGERED TIEBACK INSTALLATION PROCEDURE

20a. Insert Tendon in Auger

20b. Drill

20c. Pump Grout While Extracting the Auger
c. Regroutable tiebacks use postgrouting to enlarge the anchor grout by delayed multiple grout injections. Each injection is separated by about one day and attempts to fracture the grout already in place and wedge it outward into the soil. A special grout tube with valves located along the anchor length enables postgrouting to be performed. The grout tube is designed so a double packer can be used inside the tube to selectively grout each valve. Figure 21 shows one type of regroutable tieback.

**Figure 21.**
TMD Tieback Installation Sequence

21a. Insert deformed metal tube in grout-filled drill hole

21b. Inflated seal

21c. First-phase grouting of each valve

21d. Multi-phase grouting of each valve

21e. Fill tube with grout and insert tendon
d. Rock tiebacks are constructed by drilling a three-to-six-inch hole into rock. Rotary, percussive, or a combination of both drilling methods are used to advance the borehole. Casing may have to be used to maintain the borehole open in overburden or in fractured rock zones. After the hole has been drilled, a grout tube and tendon is inserted; then, grout is pumped down the grout tube until clean grout emerges at the surface. See Figure 22.

**FIGURE 22.**
CONSTRUCTION STEPS FOR A TREMIE-GROUTED STRAIGHT-SHAFTED ROCK TIEBACK

22a. Drill and Clean Hole

22b. Insert Tendon and Grout

22c. Tendon Grouted in Place

4. Design

The design of tiebacks is interrelated with the design of the structures they support. When designing a tiedback structure, the tieback system is selected first. This involves choosing the tieback tendon, the corrosion protection for the tendon, the method of installation, and the estimation of the tieback capacity.
The ultimate capacity of a tieback is actually determined by the load-transfer mechanism between the grouted anchor and the rock or soil. The rate of load transfer varies depending on the tieback type, the method of installation, and the soil or rock properties. In general, the rate of load transfer is not uniform along the anchor length; instead it is greatest near the front of the anchor and decreases towards the back of the anchor. This non-uniform load-transfer rate is a result of the varying displacements along the anchor length. The ultimate capacity for each tieback type can be estimated by using the following empirical data and equations.

The load-transfer rate for pressure-injected tiebacks is estimated from empirical data since no theoretical relationship has been developed to accurately estimate their ultimate capacity. Figure 23 gives the equation for the ultimate capacity of pressure-injected tiebacks along with the load-transfer rate for various soils.

**FIGURE 23.**

**ULTIMATE CAPACITY AND LOAD-TRANSFER RATE FOR PRESSURE-INJECTED TIEBACKS**

\[ P = I_a T \]

where:

- \( P \): ultimate tieback capacity
- \( I_a \): anchor length
- \( T \): rate of load transfer

23a. Ultimate Capacity of Pressure-injected Tiebacks

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Rate of Load Transfer (kips/ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean sands and gravels</td>
<td>10.50</td>
</tr>
<tr>
<td>Clean coarse sands</td>
<td>10.20</td>
</tr>
<tr>
<td>Clean medium and fine sands</td>
<td>8.20</td>
</tr>
<tr>
<td>Silty sands</td>
<td>4.8</td>
</tr>
</tbody>
</table>

(Diameter of anchor = 4 in, minimum of 12 ft of overburden)

23b. Rate of Load Transfer for Pressure-injected Tiebacks
Hollow-stem-augered tiebacks are shaft tiebacks. Figure 24 shows the ultimate capacity equation and soil-grout bond stresses for hollow-stem-augered tiebacks in various soils.

**FIGURE 24.**

**ULTIMATE CAPACITY AND SOIL-GROUT BOND STRESSES FOR HOLLOW-STEM-AUGERED TIEBACKS**

\[ P = \pi d l a f' g \]

where

- \( P \) = ultimate tieback capacity
- \( d \) = tieback diameter
- \( l a \) = anchor length
- \( f' g \) = soil grout bond stress

24a. Ultimate Capacity of a Hollow-stem-augered Tieback

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Std. Penetration Resistances (blows/ft)</th>
<th>Bond Stress between Grout and Soil (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft Clay</td>
<td>2.4</td>
<td>50-75</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>3.6</td>
<td>50-100</td>
</tr>
<tr>
<td>Sandy Clay</td>
<td>3.6</td>
<td>75-100</td>
</tr>
<tr>
<td>Medium Clay</td>
<td>4.8</td>
<td>75-125</td>
</tr>
<tr>
<td>Firm Clay</td>
<td>6.12</td>
<td>100-150</td>
</tr>
<tr>
<td>Stiff Clay</td>
<td>8.15</td>
<td>1,000-2,000</td>
</tr>
<tr>
<td>Very Stiff Clay</td>
<td>15.30</td>
<td>1,500-2,500</td>
</tr>
<tr>
<td>Hard Clay</td>
<td>Over 30</td>
<td>1,500-4,000</td>
</tr>
</tbody>
</table>

24b. Soil-grout Bond Stresses for Hollow-stem-augered Tiebacks

The mechanism by which a regroutable tieback develops its capacity is not well understood. Available data shows that postgrouting improves the capacity of tiebacks in cohesive soils. Depending on the soil and the postgrouting system, increases ranging from 25 percent to more than 300 percent are common.
Rock tiebacks also are shaft tiebacks and their ultimate capacity can be estimated from the equation shown in Figure 25. The table contains typical values of bond stresses for various rock types.

**FIGURE 25.**

**ULTIMATE CAPACITY AND ROCK-GROUT BOND STRESSES FOR ROCK TIEBACKS**

\[ P = \pi d l f_t \]

where:
- \( P \) = ultimate tieback capacity
- \( \pi \) = 3.14
- \( d \) = tieback diameter
- \( l_a \) = anchor length
- \( f_t \) = rock-grout bond stress

**25a. Ultimate Capacity of Permanent Rock Tiebacks**

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Bond Stress between Grout and Rock (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>120-250</td>
</tr>
<tr>
<td>Sch Shales</td>
<td>30-120</td>
</tr>
<tr>
<td>Slate and Hard Shales</td>
<td>120-200</td>
</tr>
<tr>
<td>Sch Limestone</td>
<td>150-220</td>
</tr>
<tr>
<td>Hard Limestone</td>
<td>300-400</td>
</tr>
<tr>
<td>Granite and Basalt</td>
<td>250-800</td>
</tr>
</tbody>
</table>

Data was summarized by Goldberg et al. (1976) from:
1. Inland-Ryerson (1974, ACI Ad-Hoc Committee)
2. Littlejohn (1970)
3. Littlejohn et al. (1975)

**25b. Rock-grout Bond Stresses for Rock Tiebacks**

**5. Corrosion Protection**

Permanent tiebacks have been installed routinely since the mid-1960s in Europe and since the early 1970s in the United States. They are performing well in a variety of environments. Most tiebacks use cement grout for protection over their anchor length. Available reports indicate that there is no evidence of a corrosion failure where the tieback tendon was encased in grout. Corrosion failures have occurred along the unbonded length of unprotected tendons, with most of them located within 6.65 feet (2 m) of the anchorhead. A significant number of the tieback corrosion failures in Europe occurred in tendons fabricated using quenched and tempered pre-stressing steels. These steels do not meet ASTM specifications and they have not been used widely in the United States.
Most permanent tiebacks can be protected by Portland cement grout along the anchor length, and a grease-filled tube or heat-shrinkage sleeve over the unbonded length (see Figure 26). Grout-protected tiebacks should be electrically isolated from the structures they support, and the tendon should have a minimum of 0.50 inches of grout protection. Figure 26 shows the anchorage insulation used to isolate the tendon. Electrical isolation interrupts the long-line differential aeration corrosion cell shown in Figure 27. This cell is potentially dangerous because it does not require oxygen in the soil, and because of the relative size of the cathode and anode. If this cell develops, the tendon at the top of the anchor zone would become the anode, and the entire wall would become the cathode. Electrical isolation also would interrupt any stray-current corrosion system.

**FIGURE 26. GROUT-PROTECTED TIEBACKS**

**FIGURE 27. CORROSION MECHANISMS AFFECTING A TIEBACK**
If the soil surrounding the anchor length has a pH less than 4.5 or a resistivity less than 2000 ohm-cm, or if sulfides are present, a local corrosion system could develop on the tendon (see Figure 27). If these conditions exist, the tendon should be completely encapsulated in a plastic or steel tube. Figure 28 shows an encapsulated tieback. The encapsulation will interrupt any long-line and stray-current corrosion system, as well as prevent the local corrosion system from developing.

**FIGURE 28.**
ENCAPSULATED TIEBACK

![Diagram of encapsulated tieback](image)

Legend:

- 11 Anchorage cover
- 23 Anchor head and wedges
- 33 Anticorrosion grease or grout
- 43 Bearing plate
- 53 Trumpet
- 63 Anticorrosion grease or grout
- 73 PVC or polyethylene tube
- 83 Greased and sheathed strands
- 93 Spacer
- 103 Strain tendon
- 113 Corrugated polyethylene or PVC
- 123 Centralizer
- 133 Anchor grout
- 143 Grout or polyester resin
- 153 End cap

Figures 26 and 28 show two ways to provide corrosion protection for the anchorage and the tendon below the bearing plate. Care must be taken to ensure that this area is well protected since most known corrosion failures have occurred near the anchor head. The corrosion protection under the anchorage should be designed to accommodate small movement of the wall.

6. Testing

Every tieback should be tested to verify that it will carry the design load without excessive movement. Tiebacks are one of the few structural systems where each member can be tested before being placed into service. Three types of tests are recommended: performance, proof, and creep tests.
A hydraulic jack and pump are used to apply the load. The entire tieback tendon should be simultaneously loaded during testing. The movement of the tieback is measured with a dial gauge or a vernier scale supported on a reference which is independent of the tiedback structure. Movement cannot be monitored accurately by measuring the jack ram travel.

The first few tiebacks and a selected percentage of the remaining tiebacks should be performance-tested. The performance test is used to establish the load-deformation behavior of the tiebacks at a particular site. It also is used to separate and identify the causes of tieback movement, and to check that the unbonded length has been established. The movement patterns developed during the performance test are used to interpret the results of the simpler proof test.

Performance-testing is conducted by measuring the load applied to the tieback and its movement during incremental loading and unloading. Figure 29 shows a plot of a typical performance test. The upper graph in this Figure shows the total tieback movement as a function of load, while the lower graph shows the residual movement of the anchor as a function of load. The residual movement (permanent set) of the anchor is the non-elastic or unrecoverable movement of the anchor which is measured when the load is released after each loading increment. The maximum load applied during the performance test is held constant for 10 minutes, and the movements are measured and recorded at 1, 2, 3, 4, 5, 7, and 10 minutes. If the tieback is not creep-susceptible, the elongation between one minute and ten minutes normally will be less than 0.04 inches. If so, the test can be discontinued. If the movements exceed 0.04 inches, the maximum load should be held for 60 minutes so a creep curve can be plotted.

Each production tieback which is not performance-tested should be proof-tested. A proof test is a simple test which is used to measure the total movement of the tieback. Proof-testing is conducted by measuring the load applied to the tieback and its movement during incremental loading. Figure 30 shows a typical proof test plot.
The load increments are the same as those used in the performance test, except that the maximum increment is normally equal to 1.20 times the design load. The maximum load applied during a proof test is held constant for five minutes and the tieback movement is recorded. If the movement during the five-minute observation period is less than 0.03 inches, the test is discontinued. If the movement exceeds 0.03 inches, the load should be maintained until the creep rate can be determined and compared to the creep behavior observed during the performance or creep tests.

Creep tests are performed to evaluate the long-term load-carrying capacity of tiebacks installed in cohesive soils and soft shales. They normally are made on the initial two performance-tested tiebacks. During a creep test, each increment of load is held constant for a certain time period and the elongations are recorded and plotted. Figure 31 shows a typical plot of creep movement versus time on a semi-logarithmic graph, with each curve representing the creep movement at each load increment. Figure 32 shows the three characteristic types of creep curves observed during tieback testing. Curves (a) and (b) indicate acceptable behavior, provided that the creep movement estimated by projecting the creep rate over the life of the structure is not excessive. A creep rate of 0.08 inches per log cycle would produce a creep movement of approximately 0.5 inches over a period of 50 years. Curve (c) indicates that the tieback would continue to creep until it failed.

FIGURE 31.
CREEP TEST

![Creep Test Graph](image-url)

1.0 ton = 8.9 kN, 1.0 inch = 25.4 mm
Tieback tests are used to identify the load-deformation behavior of each tieback, and to provide data that will enable the engineer to make a decision as to their adequacy. The total movement curve is helpful in quickly identifying any unusual behavior. However, the primary purpose of the test is to verify that the tieback will carry the load without excessive movement. The load-carrying ability of the tieback is best indicated by its behavior during the loadhold or creep test.

D. Monitoring

The long-term performance of a tieback can be evaluated by monitoring changes in the tieback load and deformation of the tiedback structure. Both load and deformations must be monitored. The tieback load can be monitored by lift-off tests or load cells. Visual checks, optical surveys, extensometers, or slope indicators can be used to observe deformations.

V. Specifications

Temporary and permanent tieback work can be obtained by three different types of specifications.

A. Open Specification

Open specifications leave the scope, design, and installation of the tiebacks up to the contractor. This method of securing bids for the work is the most commonly used method of specifying temporary tieback work. The responsibility for design is clearly placed upon the contractor, thus allowing him maximum flexibility. The competitive development of effective tiebacks engineered by specialty subcontractors is a direct result of this practice. Contractors familiar with the design and installation of earth-retention systems consistently have provided satisfactory installations. When contracting for the work, the owner's engineer must assure himself that the capabilities and resources of the contractor are appropriate for the job.
Reasons for choosing a contractor design include the following:

- the contractor is in a position to provide a firm lump sum price;
- the contractor will automatically evaluate alternative methods of performing the work to provide the most economical approach;
- patents and proprietary systems can be automatically considered, frequently providing the optimum solution; and
- change orders are minimal, thereby providing the required measure of cost control.

B. Performance Specification

Performance specifications vary the amount of design performed by the contractor and the owner's engineer. This specification method is used for temporary and permanent tieback work. It allows the owner's engineer to specify certain items which are critical to his design, while allowing the contractor the flexibility to choose the most appropriate tieback system. For example, the engineer may specify the following: the earth-pressure diagram or the horizontal loading at a particular elevation; the minimum unbonded length; the level of corrosion protection; the tieback testing procedure; and field-monitoring requirements. The owner also can prequalify the contractors in order to assure a reliable tieback installation. The prequalification can be based on experience, or a list of acceptable contractors could be included in the specifications.

When using performance specifications, the contractor is able to select the anchor types, tendons, corrosion-protection details, installation method, and tieback spacing. These items can be verified in the field and the tieback capacity can be verified by testing.

The advantages of securing the work by this method are that neither the method of forming the anchor nor the required length for developing the anchor capacity is specified; patented methods and a great variety of equipment can be considered; and materials also are adapted more readily. The major differences with open specifications are the sharing of responsibility, flexibility of the overall system design, and cost.

C. Closed Specification

Closed specifications result when the owner's engineer specifies the details of the tieback design and installation. A closed specification does not insure a better installation. When closed specifications are used, contractors not familiar with tieback work can install the tieback as specified, and if it does not perform, the owner is responsible for correcting the problem.

Closed specifications usually cause high bid prices, due to the fact that installation method and tieback type are specified incorrectly. Change orders occur more often on this type of bid work than any other.
In summary, use of an open specification places the liability for performance on the contractor, while affording maximum flexibility. This flexibility has played an important role in the evolution of competitive tieback development. It must be ensured, however, that the installation meets the job requirements and the contractor is capable of performing the work.

A performance specification can ensure the adequacy of: tieback installation; earth-pressure loadings; effects on other wall components; mass stability; minimum tie lengths; range of allowable tie angles; minimum unbonded length; level of corrosion protection; and a testing program. A prudent installation can be provided by requiring the adequacy of the above, while maintaining the flexibility of competitive methods and available materials. The design liability rests primarily with the owner's agent in this case.

Caution should be exercised when using a closed specification, since the engineer is not in the best position to select the tieback type and the installation method. Construction time and overall cost can be increased greatly when a closed specification details the incorrect tieback system.

VI. Conclusion

The use of tiebacks for temporary support of excavation walls has become commonplace. Permanent tiebacks offer many well-known advantages, in that:

- they are economical and effective tools for supporting a variety of different structures;
- they can be installed in rock and sandy soils without concern about their long-term performance;
- they can be made in cohesive soils; however, a careful testing program and a proven tieback system should be used;
- the tieback tendon can be protected easily from corrosion; and
- each tieback can be tested to verify that it will carry the design load for the service life of the structure.

Finally, engineers must be able to specify the work in such a manner that results in a cost-effective installation.
REFERENCES

1. Goldberg, D. T., "Lateral Support Systems", 1976 Geotechnical Lecture Series on Lateral Earth Pressure, Boston Society of Civil Engineers Section of ASCE.


