DESIGN PRACTICE FOR TIEBACK EXCAVATIONS IN THE U.S.

Richard P. Ray, Ph.D. P.E. M.ASCE

Assoc. Professor, Dept. of Civil & Environmental Engineering
University of South Carolina USA
Visiting Professor, Szechenyi Istvan University
Gyor, Hungary

1. INTRODUCTION

As a first consideration, one must acknowledge that any excavation, no matter how complex, is a means to an end. It is *not* the object of a final design. This humbling thought makes for a very interesting engineering problem: get to the bottom quickly with a minimum of expense. The design of deep excavations now considers geotechnical data collected before, during, and after construction. The approach has become more holistic in that changes in design parameters are expected to come from a variety of sources. Soil and groundwater conditions are primary concerns, however, the condition and movement of surrounding structures, excavation sequencing, and performance of the tiebacks themselves all contribute to design. The increasing dependence on field instrumentation and sophisticated analyses has made the design process more similar to tunneling projects than classical foundation designs. This paper discusses some of the features of the design process as it has developed in the U.S. As with most geotechnical design methods, tie-back excavations are strongly influenced by past experience and field observation. Several projects are discussed to illustrate the design process.

2. PAST STATE OF THE ART CONTRIBUTIONS

The best starting point for excavations is Peck's 1969 State-of-the-Art report (Peck, 1969). This work, with later elaborations in Peck, Hansen, and Thornburn (1974) gave three important tools to the geotechnical engineer: Well-documented and categorized performance data, theory to support generalizations of the data, and a methodology with which to improve the data and theory. The observational method is now so widely used in so many different applications that it can hardly be recognized as such (eg. Sara 1993). Earth pressure data for braced excavations from these works became the accepted standard for design. The shape of earth pressure diagrams described in Terzaghi and Peck (1967) and previously mentioned references were a departure from earlier diagrams. They represented something different as well, ie, *design envelopes* representing *possible* earth pressures the wall may experience in its lifetime. The idea of limiting lateral and vertical movement of the wall and soil behind the wall became elements of the design as well. The concept of volumetric movement was also a valuable contribution. D'Appalonia (1971) presented data on vertical and horizontal movement of braced and tied-back walls, categorizing them with respect to soil type and wall system. Goldberg et al (1975) summarized this data, and others, and compared empirical data to finite element analyses of similar designs.

Most structural design of tieback walls assumed loads based on a series of simply-supported beams or continuous only over part of the wall. While structural engineers contended that

geotechs could not analyze anything more complex than a simply-supported beam, the reasons for this simplification are more subtle. It acknowledges soil exploration and construction processes are imperfect. A simply-supported beam assumption would often be a more representative model of field conditions than a continuous beam. When problems occur, there is little need to quibble over the finer points of moment continuity (figure 1). Other influences, such as construction sequencing, bracing flexibility, and other non-geotechnical conditions meant that this assumption was not a bad one. In summary, design, analysis, and observation were well-matched for accuracy, time and effort, and constructability.

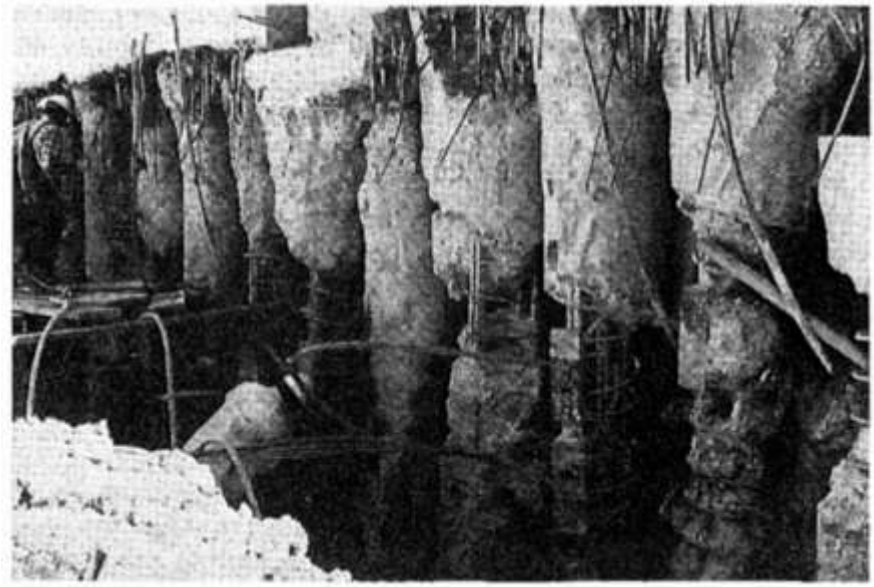

Figure 1. Example of a Non-Continuous Tie-back Wall (Ulrich, 1991)

Newly developed (1970's) numerical tools soon out-stripped the engineer's ability to explore, test, and monitor field performance. Fortunately, cool reasoning prevailed and the role of numerical modeling was kept in perspective. Clough et al (1989), Clough and O'Rourke (1990), offered several excellent perspectives on integrating numerical tools with design, construction and monitoring. Unfortunately for design engineers, construction methods were changing faster than field data could be critically appraised. One such method involved the use of earth-anchored tiebacks to brace the excavation wall. The problem for engineers was not the anchor itself, but how to incorporate its behavior into design. Since it was a more efficient system, builders could go deeper, faster, with less cost. (Recall that the excavation was a means to an end). This lead to some difficulties with analysis since there were few verifiable models to choose from and most required many weeks to implement. The problems for a designer were even greater since he had no template or code to work from. Responsibility for the design of such systems was handed back and forth between the project structural engineer, project geotechnical consultant, general contractor, specialty contractor, and owner. Since there was no prescribed process, every project was different. The trend to deeper and deeper excavations meant that many types of soils, previously in a "safe" region of stress, were now in a region of undrained failure or near-failure. Strain levels could become very high on a local basis and large volumes of soil were being strained to produce significant deflections at the surface as well as below neighboring buildings.

Also with increasing depth came increasing structural flexibility. Walls were now membranes "pinned" to the soil behind them and brute force with high factors of safety became a very expensive approach. Structural considerations and movements were also more complex since the wall profile could reverse its curvature several times.

A series of studies by the Federal Highway Administration (Long et al, 1998, Weatherby et al, 1998, Weatherby, 1998, Sabatini et al, 1999) helped to standardize the study of tieback wall behavior, their design, and how to specify their construction. The studies included demonstration

projects, detailed field and model studies, and a review of analysis methodologies. These studies clarified the design process and the role of the geotechnical engineer in that process. Further work by the U.S. Army Corps of Engineers (Ebeling et al, 2002, Strom and Ebeling, 2001, 2002a, 2002b) helped to clarify approaches to tieback wall design and introduced simplified methods for projects where design parameters were well within the limits of a growing body of experience.

More recently, incorporation of sophisticated analysis methods to verify design assumptions have been used to predict wall movements, likelihood of neighboring building damage, and application of reduction-factor types of design (Finno, 2007, Hsiao, 2007, Hashash, 2006). The analysis methods (and their authors) still acknowledge the degree of uncertainty inherent in such a venture.

3. DESIGN FEATURES TO CONSIDER

Before any calculations can be made the engineer must consider the final product, its intended function, and construction sequences. Those items are:

1. Soil Conditions. Strength, stiffness, effective stress parameters, degree of consolidation, shrink/swell potential, hydraulic conductivity.
2. Groundwater. Height of groundwater table, temporary and permanent dewatering program, inter-connectedness of multiple aquifers, basal heave potential
3. Temporary wall or permanent foundation. Will the wall be part of the building structure or only a temporary restraint?
4. Placement prior to excavation or top-down. This will have a very pronounced effect on wall rigidity and toe stability.
5. Location and levels of tieback support. This will impact wall rigidity and the ability to control lateral movement.
6. Angle and penetration distance of tieback. Sometimes influenced by interfering utilities, tiebacks from across street, tunnels, soft strata, construction limitations.
7. Tieback pre-stressing level and sequence.
8. Proximity to other structures and their lawyers.

Some of these considerations are shown in figure 2. The main goal of the figure is to illustrate the interconnectedness of many disparate (and conflicting) design requirements. This is perhaps the reason why this type of construction is performed by specialty firms who have experience with balancing these requirements and are capable of responding quickly (within the same day) to small and large problems.

4. CONCEPTS COMMON TO ALL DESIGN APPROACHES

Several concepts are necessary to understand before any design approach is applied. Among them are: wall stiffness, global stability, and earth pressure calculation methods.

Wall stiffness is an important quantity to understand since stiff walls behave differently than flexible ones. Stiffness depends on wall material, section properties and support spacing. A useful quantity for estimating wall stiffness of unit thickness (plane strain) is given by

$$k = \frac{EI}{\gamma_w h_{ave}^4} \qquad \textbf{Eq. 1}$$

where k = Dimensionless wall stiffness
E = Modulus of wall system
I = Moment of inertia of wall system
h_{ave} = average distance between tiebacks
γ_w = unit weight of water to make equation dimensionless

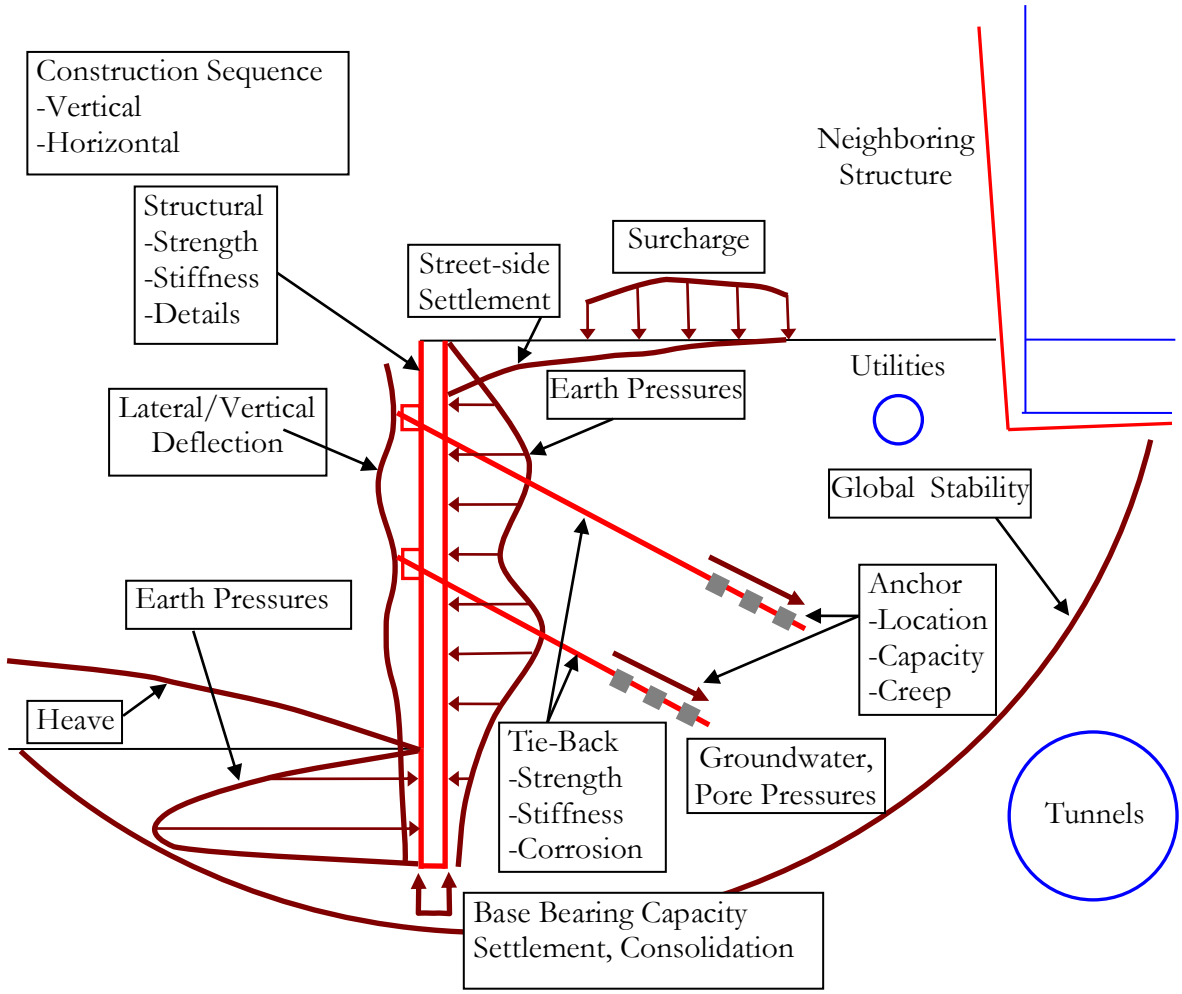

Figure 2. Design Considerations for Tie-back Excavation

The more experienced (older) reader will recognize this factor as the inverse of Rowe's flexibility number for sheet pile walls (Rowe, 1952). For non-continuous walls, the wall properties are averaged over the width of the wall. Differences in behavior between stiff and flexible walls are shown in Table 1.

Table 1 Values for Stiff an d Flexible Wall Systems (Strom and Ebeling, 2002)

Wall Stiffness	Wall System	EI (k-ft/ftx10^4)	EI/h^4
Flexible	Vertical sheet piles	0.3 to 5.0	3.7 (1)
	Soldier beam	0.1 to 4.0	1.5 (2)
Stiff	Secant Cylinder Pile	8.0 to 250.0	239.8 (3)
	Continuous RC wall	30.0 to 150.0	123.1 (4)
	Discrete RC wall	35.0 to 160.0	92.3 (5)

(1) Relative stiffness based on PZ 27 sheetpiling. Per Olmsted Prototype Wall.
(2) Relative stiffness based on HP12×53 soldier beams spaced at 8.0 ft on center (OC). Per FHWA-RD-97-130 design example.
(3) Relative stiffness based on 5.0-ft-diam caisson piles spaced at 7.0 feet OC. Per Monongahela River Locks and Dams 2 Project.
(4) Relative stiffness based on 3.0-ft-thick continuous slurry trench wall. Bonneville Navigation Lock Temporary Tieback Wall.
(5) Relative stiffness based on W36×393 soldier beams at 6.0 ft OC with concrete lagging. Per Bonneville Navigation Lock Upstream Wall

Flexible walls are typically soldier pile and lagging or sheet pile walls. Secant pile and slurry walls tend to be rigid wall systems.

For examining the deep-seated failure of a wall, a stability number is often computed. This is based on Taylor's approach to slope stability in computing a number based on the height differences, soil weight, and soil strength. Once a stability number is calculated, one could easily determine the likelihood of deep-seated failure. Additionally, stability numbers are used to evaluate performance of deep tie-back systems. The stability approach was clearly outlined by Clough and O'Rourke and has been used by many authors to indicate the relative stability of the wall tieback soil system. It can also be used to categorize various wall systems to enable comparison of performance.

Calculation of earth pressure is normally performed via use of lateral earth pressure coefficients. The coefficient relates effective vertical stress to effective horizontal stress. Typical methods to compute earth pressure coefficients are shown below. Pore pressures exert lateral forces on tie-back walls if they are present. However, most excavations are de-watered and pore pressures behind the wall are reduced greatly. Stress relief during excavation may also cause negative pore pressures to generate. Cohesive soils may exhibit very different behavior when pore pressures are significantly negative. With time, pore pressures would equilibrate and so the question becomes "how much time?" This will be a function of the conductivity of the soil in question and the availability of drainage paths, both natural (stratigraphic or preferential paths) and man-made (dewatering, piezometers, open cuts, fracturing due to stress relief).

5. CALCULATION OF APPARENT EARTH PRESSURES

Earth pressure diagrams for sand, stiff clay and firm clay follow the general form presented by Terzaghi and Peck (1967). Some alterations to those diagrams, based on field data, model tests and numerical analyses were presented by Sabatini et al (1999) and were further examined by Strom and Ebeling (2001; 2002a,b) Diagrams by these authors form the basis for design today in the U.S.

5.1. Earth Pressures in Sand

Shown in figure 3 are earth pressure diagrams from Terzaghi and Peck and Sabatini et al. The changes by the latter authors acknowledge the role of deformations and wall interaction that occur especially during the "cantilever stage" of construction, before installation of the first level of tiebacks. Sabatini's diagrams equate pressure to total computed load in case another method is used to compute earth load (as opposed to earth pressure). Diagrams for single tieback and multiple tieback geometries are shown. Note that the locations of tiebacks must be known before the Sabatini diagrams can be constructed.

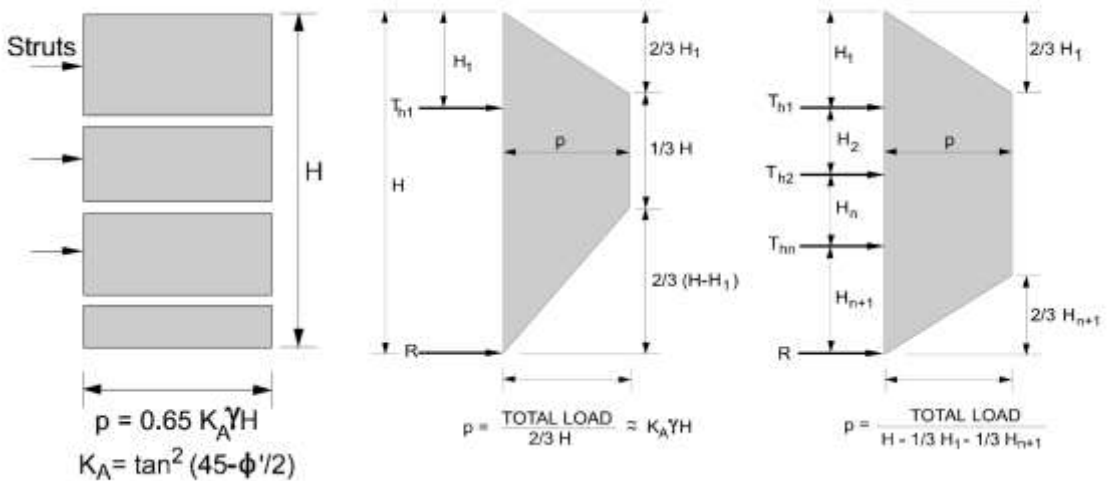

Figure 3. Earth pressure diagram for sand: Terzaghi & Peck, Sabatini et al

5.2. Earth Pressure in Stiff to Hard Fissured Clays

The design pressures for fissured clays may be very dependent on local conditions and are best left to local experts for choice of appropriate strength values. Past studies, summarized in figure 4 and table 2 show the possible ranges.

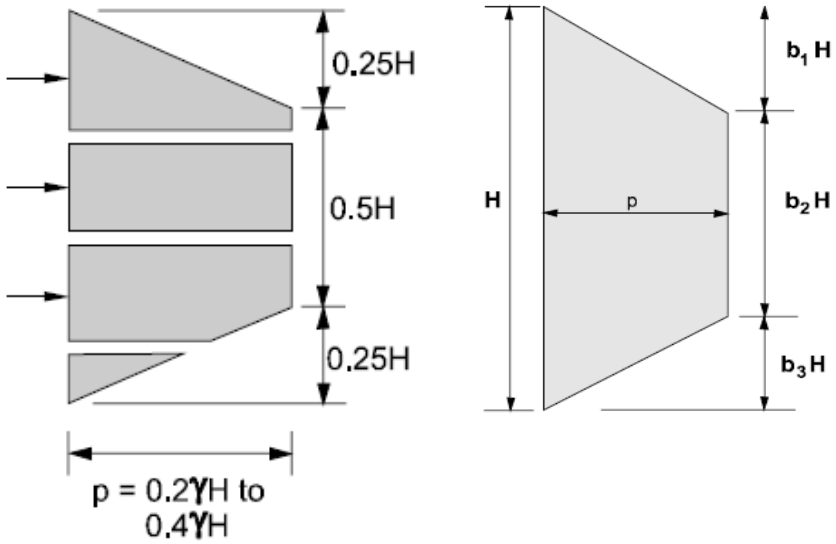

Figure 4. Apparent Earth Pressure Diagrams; Terzaghi & Peck , Sabatini et al

Table 2Ranges of values for apparent earth presures in stiff fissured clay

Reference	b1	b2	b3	Range of maximum pressure, p	Total load
Terzaghi & Peck (1967)	0.25	0.50	0.25	(0.2 to 0.4)γH	(0.15 to 0.30)γH^2
Schnabel (1982)	0.20	0.60	0.20	(0.2)γH	(0.16)γH^2
Winter (1990)	0.20	0.60	0.20	(0.2 to 0.32)γH	(0.15 to 0.26)γH^2
Ulrich (1989)	0.25	0.50	0.25	(0.2 to 0.4)γH	(0.15 to 0.30)γH^2
FHWA-RD-75-130 (1976)	0.0	1.0	0.0	(0.15 to 0.30)γH	(0.15 to 0.30)γH^2
Sabatini et al (1999)	0.17	0.66	0.17	(0.2 to 0.4)γH	(0.17 to 0.33)γH^2

Ulrich (1989) reported on seven tieback excavations in overconsolidated, fissured Beaumont Clay in Houston Texas. Measured pressures from these projects corresponded to the lower side of the Terzaghi and Peck diagram (figure 5). Wall systems were classified as stiff (drilled piers) with stability numbers N_s , less than 4. Other issues may have clouded some of the data, but in general, the interpreted pressures, based on measured tieback loads, were consistent.

Based on the results in table 3 and model tests, Sabatini et al modified the pressure diagrams as shown in figure 6. These diagrams are for stiff to hard fissured clays with stability numbers $N_s \leq 4$. Maximum pressures, p, would correspond to a total load between $3H^2$ and $6H^2$.

For permanent wall systems, the use of lower bound values would be problematic. Negative pore pressures in clays can be quite high and dissipate in a very irregular pattern, due to stress relief along fissures and the presence of silt and sand lenses. Temporary values can be compared to long-term total loads using $K_A\gamma H^2$, where K_A is based on the drained friction angle of the clay soil. For most anchored wall applications, the drained friction angle should correspond to the fully softened friction angle. The larger of the resultant forces from the two diagrams should be

used for design. For example, Beaumont Clay (Houston, Texas) exhibits a drained friction angle of approximately 17° resulting in an equivalent total force to the Terzaghi and Peck envelope using a maximum pressure ordinate of 0.47γH. Long-term stability problems have been common in the Texas Gulf Coast.

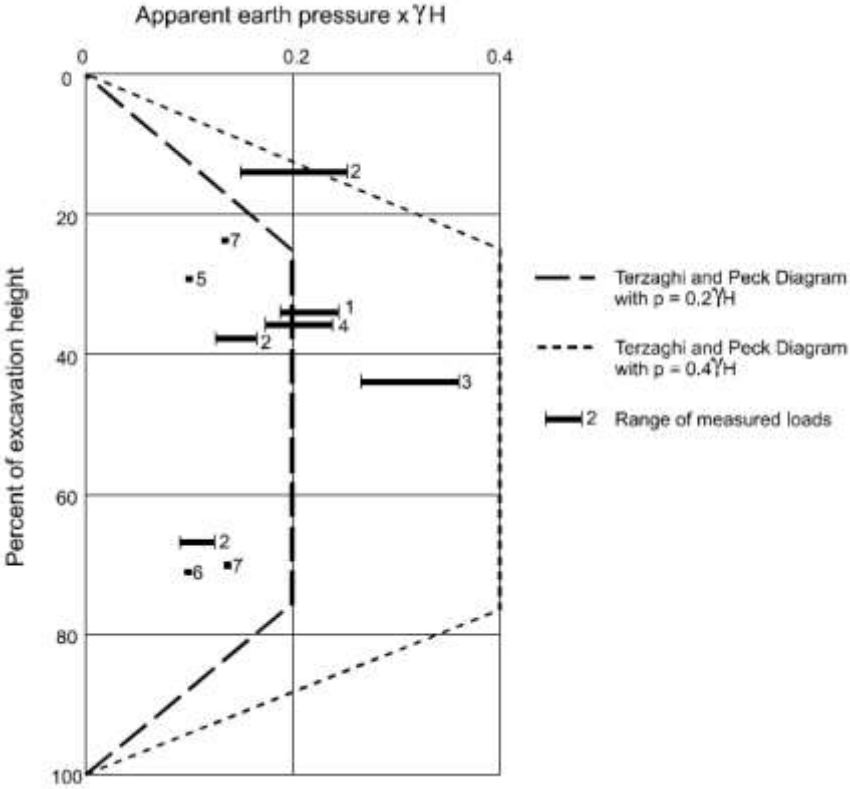

Figure 5.Measured earth pressures in stiff, fissured Beaumont Clay (Ulrich, 1989)

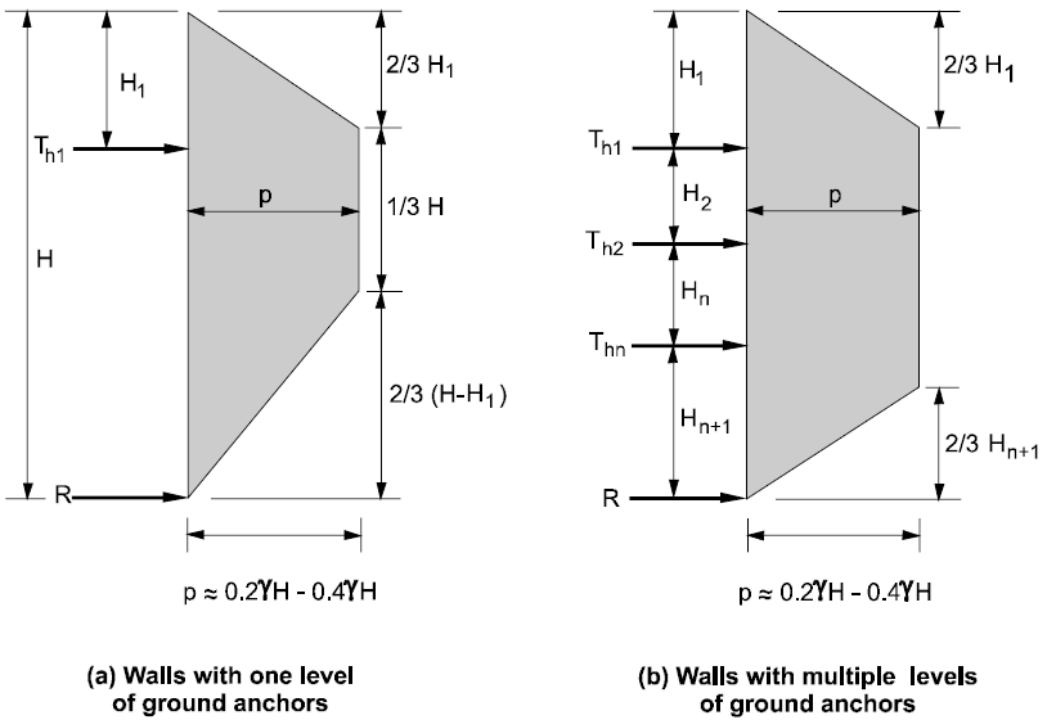

Figure 6. Apparent Earth Pressure Diagrams for Stiff, Fissured Clay (Sabatini et al, 1999)

5.3. Earth Pressure in Soft to Firm Clay

Temporary and permanent tieback walls can be constructed in soft to firm clays if a suitable zone for anchors can be found. Reaching far for a suitable anchor zone will increase flexibility of the wall system. The formula for Terzaghi and Peck's total stress active earth pressure diagram for soft-firm clays (N_S>4) uses a formula for maximum pressure based on undrained strength.

$$K_A = 1 - m\frac{4S_u}{\gamma H} \qquad \textbf{Eq. 2}$$

where

K_A	= active earth pressure coefficient
m	= empirical factor that accounts for the possibility of basal instability
	= 0.4 if site is underlain by soft clays and N_S>6
	= 1.0 otherwise
S_u	= undrained shear strength
H	= depth of excavation
γ	= total unit weight

An improvement to this approach was formulated by Henkel (1971). He showed the possible kinematic mechanism for basal heave and the associated active earth pressure coefficient. The formula for active earth pressure coefficient can then be written as

$$K_A = 1 - \frac{4S_u}{\gamma H} + 2\sqrt{2}\,\frac{d}{H}\left(1 - \frac{5.14S_{ub}}{\gamma H}\right) \qquad \textbf{Eq. 3}$$

where

S_u	= undrained shear strength of soil behind wall
S_{ub}	= undrained shear strength of soil below wall
d	= distance from base of excavation to "strong" layer
H	= depth of excavation
γ	= total unit weight

With increasing stability number, Henkel's plastic mechanism will control, below N_S~5.0, using K_A=0.22 and the Terzaghi and Peck diagram would be more appropriate. Figure 7 shows the relationship between Henkel's and Terzaghi and Peck's earth pressure values.

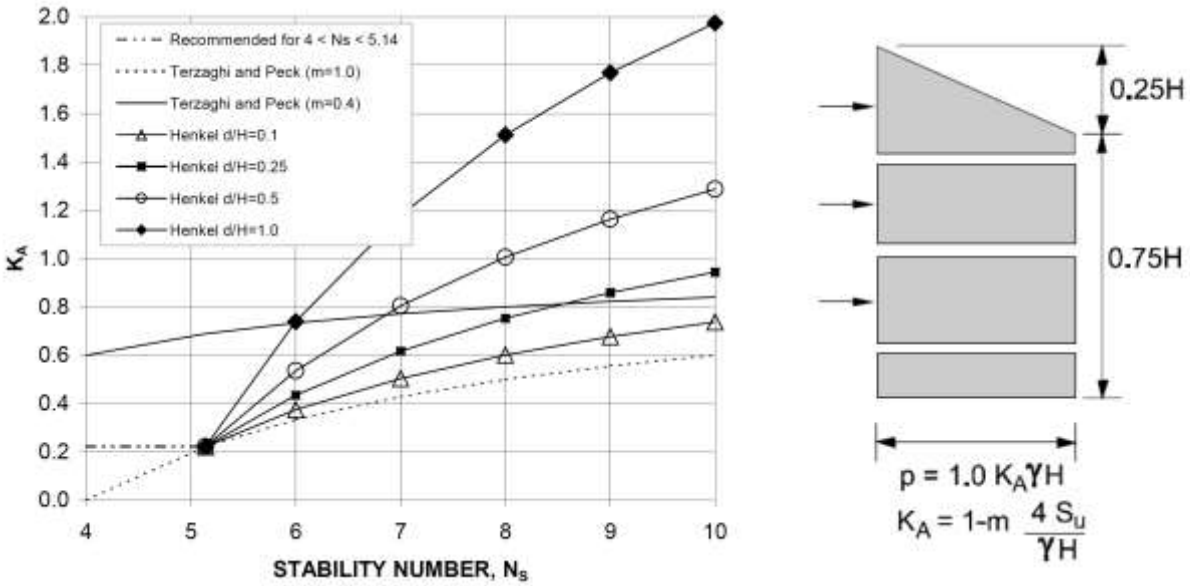

Figure 7. Henkel's and Terzaghi and Peck's computed earth pressures for increasing stability number and apparent earth pressure diagram for soft to medium clay (Sabatini, 1999)

For conditions where there are layered soils and Henkel's approach is difficult to implement, limit equilibrium computations can serve the same purpose in determining a total load. The apparent earth pressure diagram is shown in figure 7.

6. MODELS AND DESIGN METHODS

The U.S. Army Corps of Engineers produced a series of reports about the design of earth retaining systems. Work by Strom and Ebeling (2001, 2002a, 2002b) and Ebeling et al (2002) discuss assumptions of different models, design methodologies, required data, and performance of tieback walls and their components.

Methods in approximate increasing sophistication include:

1. Rigid model. The soil is modeled as trapezoidal pressure distribution, tieback points are rigid, the wall is a continuous beam
2. Winkler model. Soil is modeled as a series of springs (very closely spaced) that can be nonlinear. Simulation of construction sequencing is possible.
3. Finite Element Method. Soil can be nonlinear, anchor and wall can have interface elements, wall and structural elements modeled as elastic. Construction sequence can be included to study unload/reload process.

Each approach has it's set of assumptions and procedures. Winkler models can become more sophisticated, but yield little information about the performance of backfill. Finite element models can give a false sense of security by the very nature of their sophistication. Checks on whether the forces calculated make sense, the deflections are appropriate, and overall behavior is in keeping with similar configurations give more credence to the analysis. However, finite element analysis results, like other aspects of geotechnics, are best understood over time.

6.1. Rigid Method

The rigid method of design is based on several simplifying assumptions. It is applied in a different manner, depending on whether the wall system can be considered stiff or flexible. The most important aspects of the analysis/design are:

1. The wall is assumed to be fixed at a point where there is zero net earth pressure. Any structural support below that point is ignored.
2. Tiebacks are assumed to be rigid. Movement prior to tieback installation is neglected.
3. Loading diagrams can be trapezoidal or triangular.
4. Loading is not dependent on wall movement, it will follow the wall with the same loading intensity
5. The wall is modeled as an elastic beam on rigid supports.

For flexible systems, apparent pressure diagrams are used and tieback forces are computed by tributary area or hinge method. A summary of these two methods are shown in figure 8. Note also that the passive resistance force is assumed to act at the base of the excavation. If another method to determine earth force is used, it is distributed in accordance with the diagrams shown previously and similar computations are performed. Once tieback loads are calculated, the wall is modeled as a beam with the earth pressures acting on it and tiebacks as rigid supports. Axial and shear loads as well as moments can be calculated in order to size structural elements.

For stiffer systems, especially where the toe of the wall is supported in very stiff soil or rock, the pressure diagram is assumed to be more triangular, or a slightly modified triangular diagram. Earth pressure loads are based on at-rest or nearly at-rest conditions. Sometimes a combination of both trapezoid and triangular distributions is used. Most importantly is to establish the amount of movement anticipated in the stiff wall system. Less movement means earth pressures will be closer to at-rest conditions.

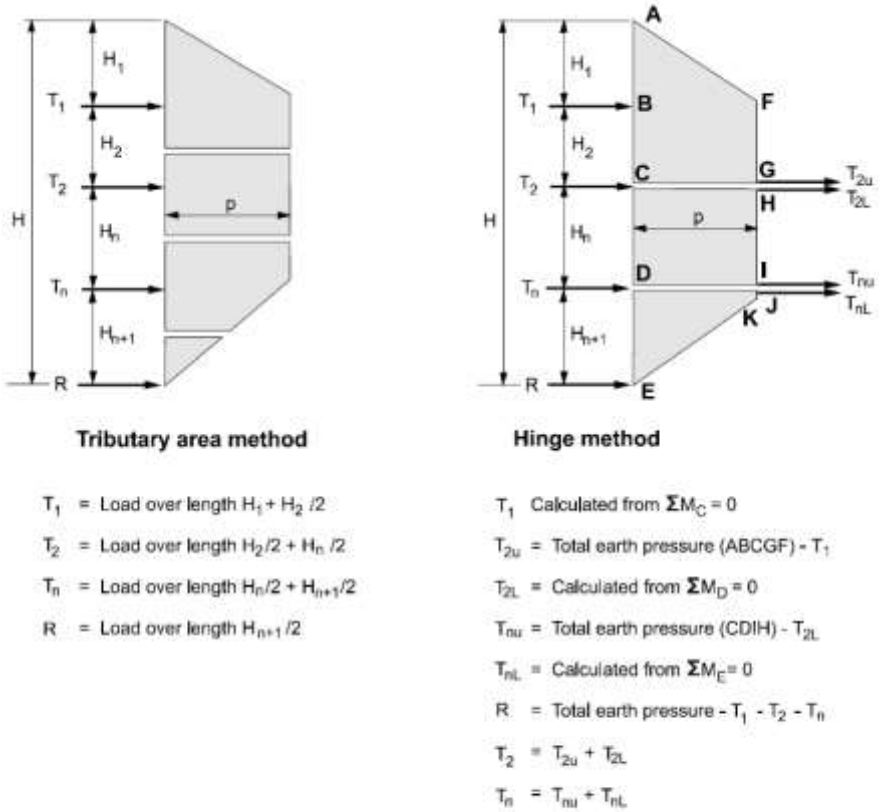

Figure 8. Tributary area and hinge method for computing tieback loads

6.2. Staged analysis for stiff wall systems

Analysis of construction stages can be performed using the rigid method and stiff walls. The wall is analyzed in stages as the excavation proceeds downward. The segment of wall included in the excavation starts at the top and continues to the point of contraflexure; assumed to be where net earth pressures are zero. Driving side pressures are assumed to be near at-rest. Full passive pressure, or a mobilized passive pressure (using $\varphi_{mob}=\varphi/F.S.$) are applied to the resisting side. A hinge support a the point of zero net pressure is assumed and the cantilever stage is easily solved. For successive stages, the wall is assumed simply supported at the anchor levels and at the point of zero net pressure. Loading, shear, and moments are computed as a beam analysis problem. The problem will become statically indeterminate but easily within reach of numerical solution.

There are also methods to analyze a continuous beam on non-rigid supports. One method is to use the lateral deflection of the beam just prior to the placement of the tieback. The point is then fixed at that deflection for the remainder of the analysis. Such an approach is easily modeled with standard structural analysis software. Another method is to place the tieback onto the wall as an elastic member. The member is pre-loaded to hold the beam with the same deflection just before the tieback was placed.

7. WINKLER ANALYSIS PROCEDURE

The Winkler analysis method has enjoyed success in the design and analysis of deep foundations subject to vertical and lateral loads. While little is learned of the behavior of neighboring soil (especially settlement or heave behind the wall), estimation of tieback loadings and wall moments can be performed very efficiently. Nonlinear behavior and soil layering are easily considered since soil properties are applied at discrete locations. Subgrade methods can mimic staged construction and tie-back tensioning as well. Formulations for the Winkler medium are based on lateral response of pile foundations and can be viewed as p-y curves (figure 9). For the sake of this

presentation, terms consistent with FHwA and USACE methods are used. The soil response for a wall is termed R-y curve and the tieback response T-y curve. Neither R-y nor T-y curves are symmetric about the zero loading axis since the soil is still governed by active/passive earth pressure principles and the tieback is a slender element.

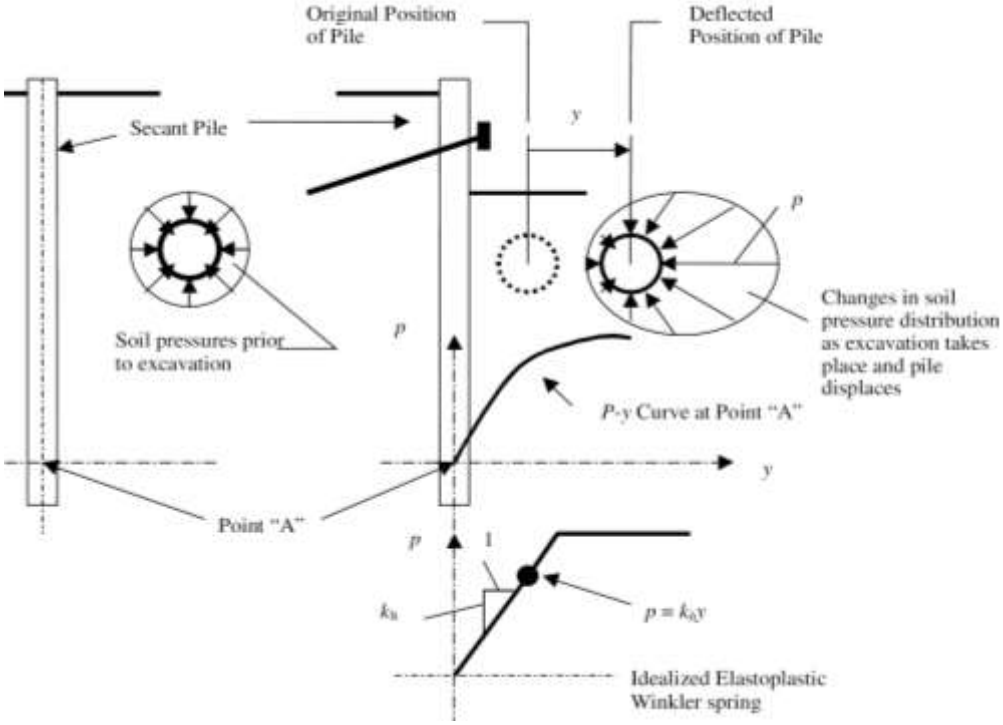

Figure 9. Winkler (p-y) concept.

Methods to compute appropriate coefficients of subgrade reaction range from assuming constant values, to values varying linearly with depth, to nonlinear curves. Most curves consider active or passive conditions, or both (as a net pressure curve) R-y curves depend on soil properties as well as staged construction considerations. Typical curves are elastic-plastic and are scaled according to at-rest, pre-construction stress conditions (figure 10). Limits of the R-y curve are based on the fully active or passive conditions and contributing area for the soil spring. While these limits are rarely reached overall, local segments may experience maximum active or passive forces. This will simply generate a redistribution of additional loads to neighboring springs. Defining a yield deflection point for the springs is more difficult and usually based on field or full-scale test performance. Alternately, stiffness can be estimated using modulus of subgrade reaction, reference deflection method or Pfister (1982) method. Springs for both active and passive condition can be manipulated throughout the analysis to match non-linear or special conditions. Winkler analyses suffer from several weaknesses.

1. Load-deformation behavior is nonlinear and not easily represented by an elastic-plastic response curve
2. Soil stiffness will vary with respect to confining stress; a parameter not considered directly by the method.
3. Neighboring stress conditions are not considered, so arching or other movements are ignored.

Prestressed anchor springs (T-y curves) are modeled in a similar fashion. The pre-yield portion of the response curved is based on the yield displacement, tendon yield strength, and tendon elastic stiffness. The yield displacement is

$$y_y = \frac{-(f_y - 0.60 f_y)L_u}{E_s}\cos\alpha \qquad \textbf{Eq. 4}$$

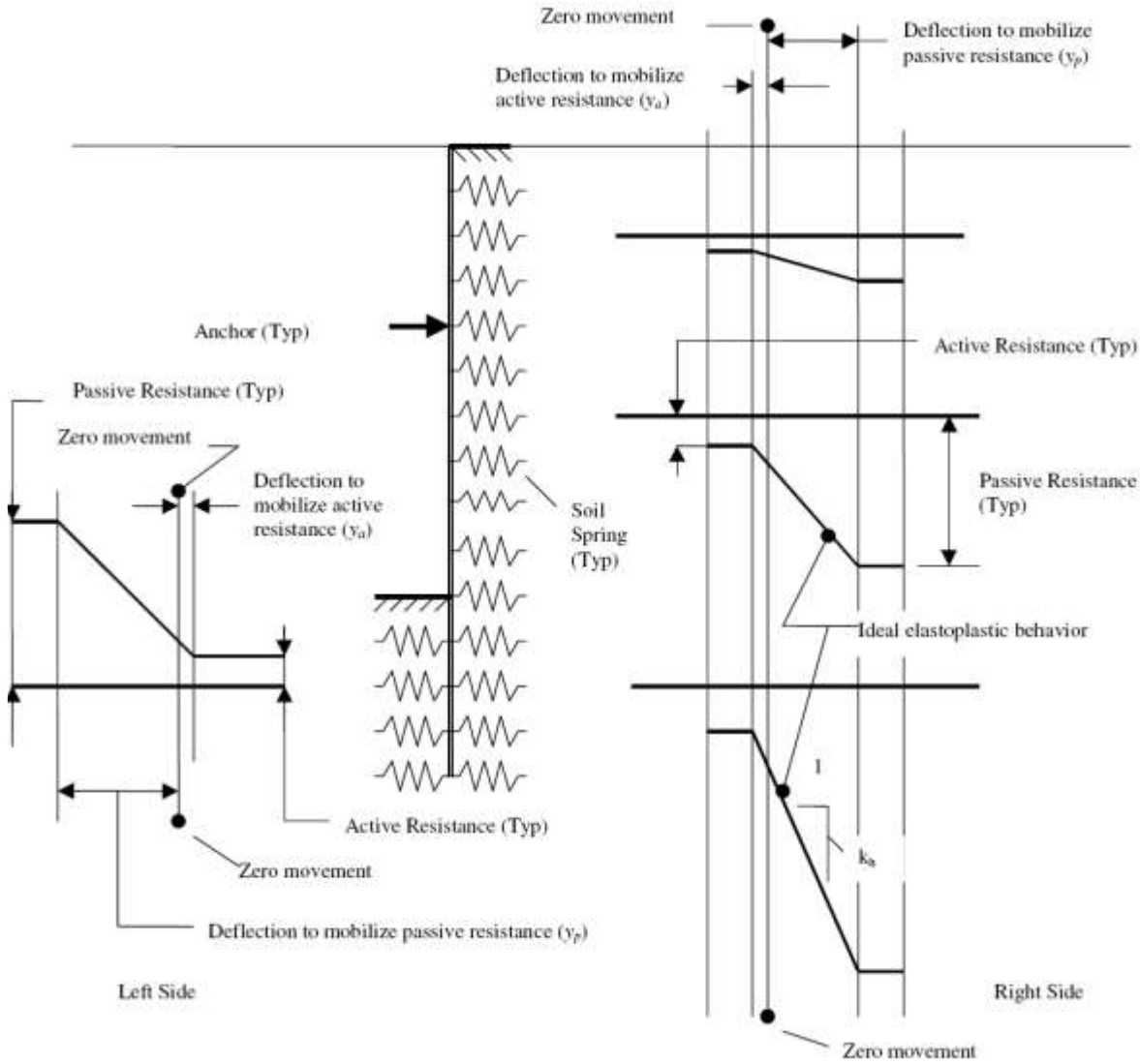

Figure 8. Tieback wall springs for Winkler analysis (Strom and Ebeling, 2001)

The tendon yield strength (T_y) is

$$T_y = A_S f_y \cos\alpha \qquad \textbf{Eq. 5}$$

and the tendon elastic stiffness (k_T) is

$$k_T = \frac{A_S E_S}{L_u} \cos\alpha \qquad \textbf{Eq. 6}$$

where f_y = yield stress of strand
L_u = effective elastic length of the anchor (unbonded length + ½ bonded length)
α = anchor inclination with horizontal
E_S = Young's modulus for anchor tendon
A_S = Area of anchor tendon

A ground anchor T-y curve is shown in figure 11. The load region beyond yield can assumed flat (plastic) or interpolated between the yield point and ultimate capacity with f_u replacing the first occurrence of f_y in equation 4 and in equation 5 to generate y_u and T_u.

Modeling the wall toe (passive response) is usually done in a manner similar to lateral response of deep foundations. Formulations for lateral resistance near the (newly excavated) surface are accomplished using the approach by Wang and Reese (1986). Passive failure wedges for soldier beams in sand and clay are shown in figure 12. Once active and passive values are known, the R-y curve is constructed in a similar manner.

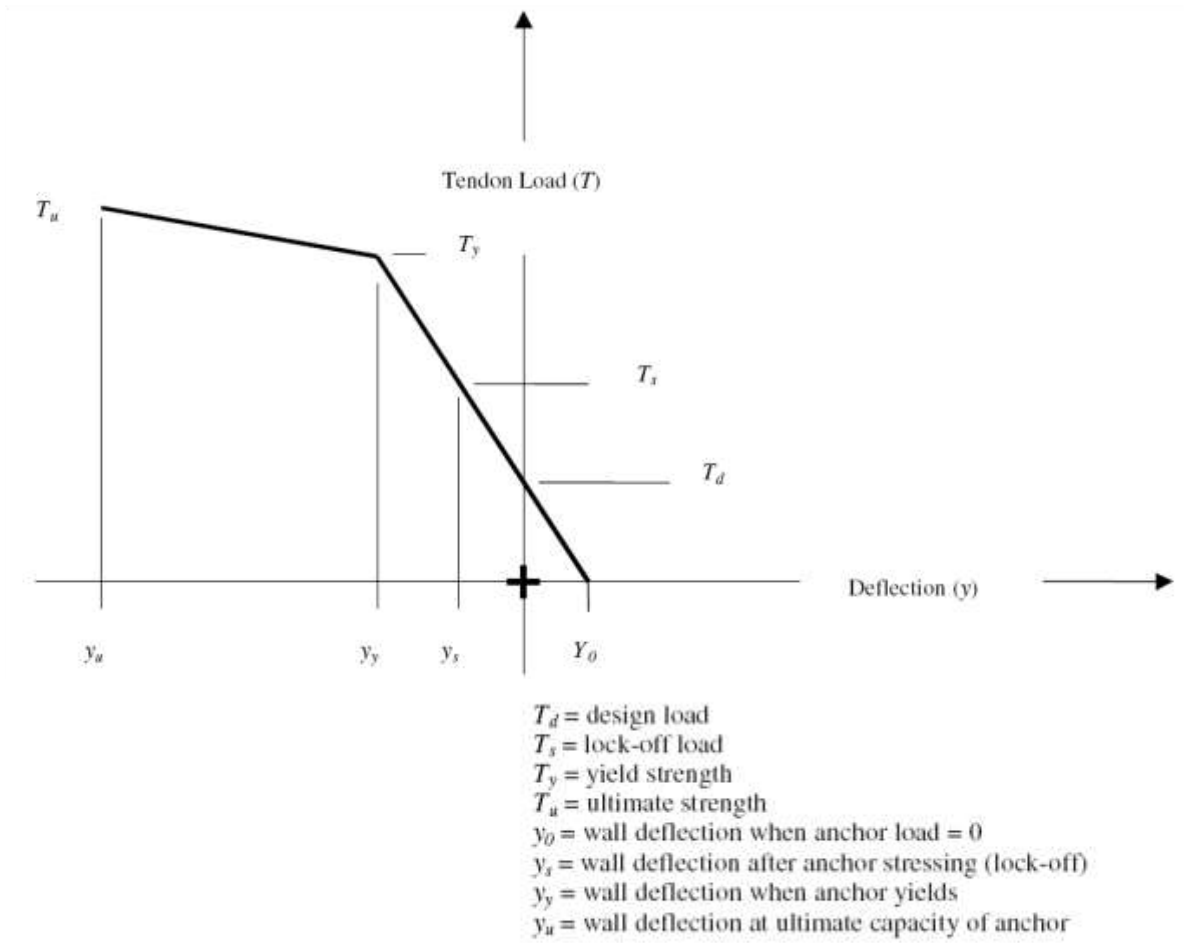

Figure 9. Ground anchor T-y curve (Strom and Ebeling, 2001)

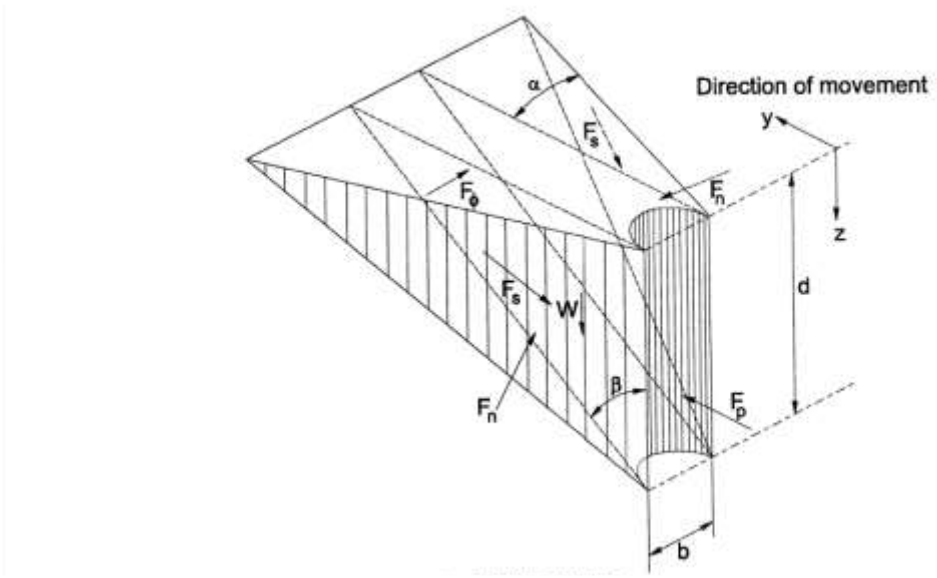

Figure 10. Passive failure wedge of soldier beam in sand (Wang and Reese, 1986)

Modeling construction stages with Winkler models is accomplished in two ways: shifting the R-y curve with each step, or modeling the active side as a static load. The shifted R-y method (Weatherby et al, 1998) adjusts the R-y curves that are driven to yield during a construction stage. The new curve (used to compute resistance to the newly installed tieback) is shifted to the present value of the deflected wall. The hysteretic behavior is shown in figure 13.

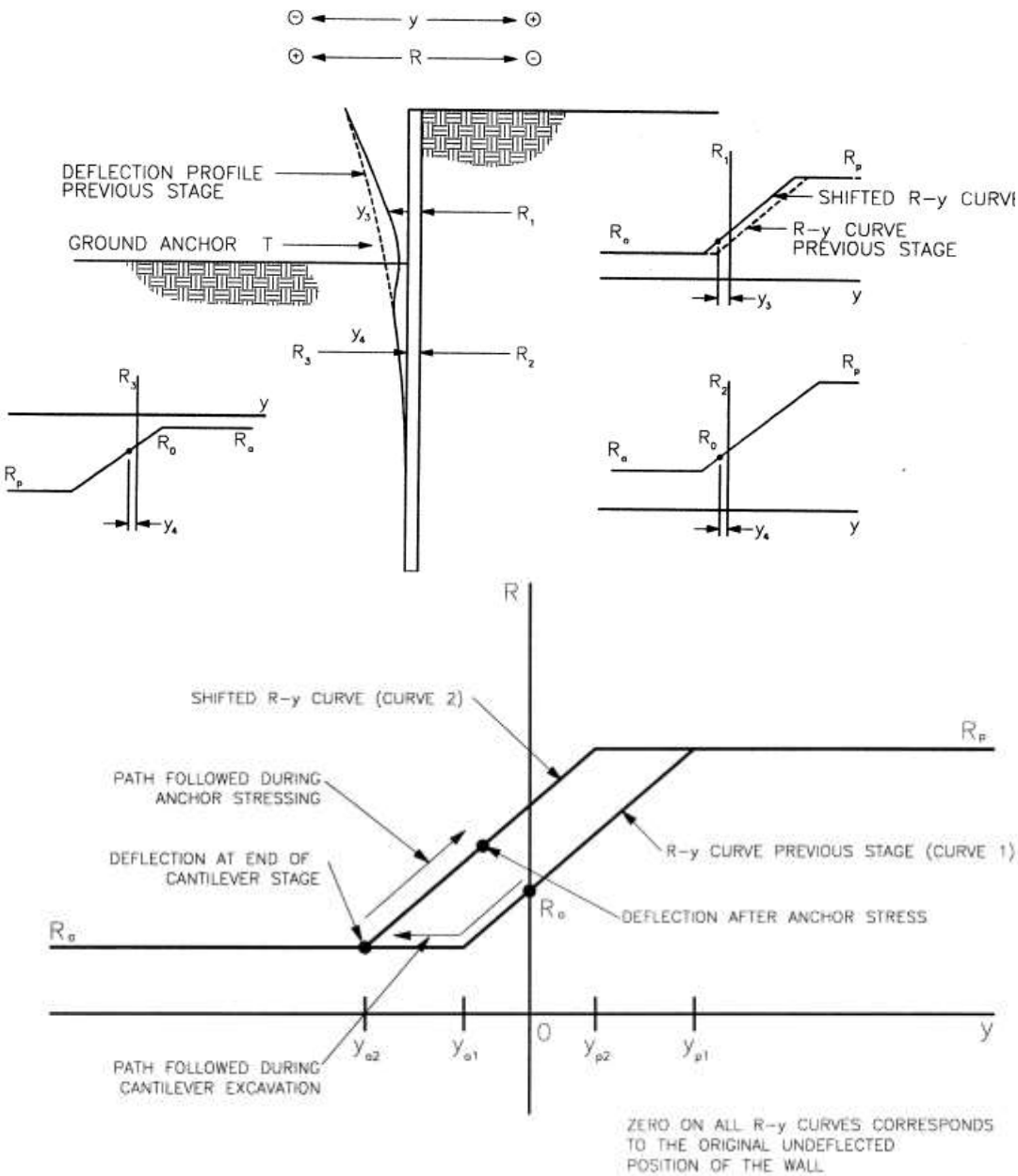

Figure 11. Shifted R-y method to model construction stages (Weatherby et al, 1998)

An alternate method involves loading the wall with a suitable active (or near at-rest) pressure. Then, at the excavation and tieback stage, applying a penalty load to maintain deflection at that point and applying the pre-stressed anchor. Passive-side reactions are either elasto-plastic or are elastic and monitored for maximum load. When the load is reached in a passive spring, that spring is replaced with a force equal to the maximum load and the analysis repeated. Such a method is easily implemented in most structural analysis software and can be extended to 3D.

8. FINITE ELEMENT MODELING

One of the first extensively studied field problem using finite elements was made on Port Allen and Old River Locks by Clough and Duncan (1969). Use of a simple hyperbolic soil model by Duncan and Clough (1971) showed the promise, and difficulties, in numerical implementation of nonlinear behavior. Initial and boundary conditions were sometimes elusive, complex non-linear behavior was difficult or impossible, stress concentrations were not well tolerated, and construction staging was either very simplistic or very tedious. More recent soil-structure

interaction problems point out the complexities of interface behavior and excavation and construction sequencing. Figure 14a shows interface stresses near the top of the wall before the start of excavation. Immediately following excavation, the wall bends inward, reducing normal stress and generating downward shear stress on the wall (figure 14b). After tensioning, normal stresses increase and shear stress may reduce or reverse direction (figure 14c). Subsequent excavation starts the cycle over again. Therefore, interface elements are subjected to very complex stress paths and should be capable of modeling such nuances.

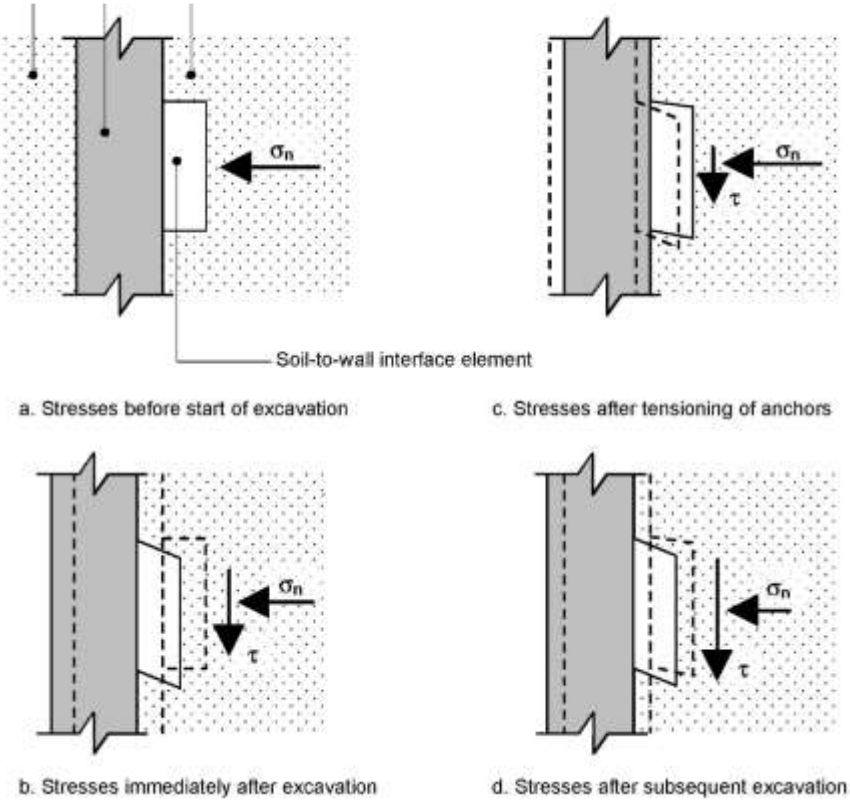

Figure 12. Interface stresses during excavation and tensioning (Strom and Ebeling 2001)

Finite element methods are rarely used as purely a design tool, but rather an analysis tool to verify and modify designs on an iterative basis. The process begins with a design generated from previous work or a simpler method discussed earlier. The design is "checked" by the finite element analysis to verify assumptions concerning wall movement, anchor placement and pre-stress levels. Structural adequacy of the wall can be estimated as well, however this may be accomplished in unison with the structural engineer. Global stability can be assessed with respect to mobilized strength values. Various strategies for using factors of safety include mobilized strength (c,φ) or increased load conditions. Detailed studies of the design process using nonlinear finite elements in concert with subgrade and rigid methods are discussed by Mosher and Knowles (1990). A discussion of 2D vs. 3D analysis for soilder pile systems is given by Hong et al (2003), and a good post-construction analysis is presented by Konstantakos et al (2004).

9. TIEBACK DESIGN

Designing a good tieback system requires proper placement of the anchoring system, adequate capacity of the grouted anchor, a method to proof test and lock off the proper tieback force, and a means to prevent corrosion for weakening the system over time. Since the tiebacks are not rigid, their contribution to the stability of the wall system is dependent on wall movement and pre-stressing. An additional consideration is the contribution of vertical downward force due to the orientation of the tiebacks.

9.1. Anchor Capacities

Capacities are determined by the length and depth of bonding between grout and soil behind (at least) the Coulomb active earth pressure line. This line may extend even further behind the line if at-rest conditions are assumed for the wall system. Typically the depth of anchorage is restricted only by the capacity of the machine performing the installation and the location of neighboring tieback systems, utilities, or tunnels. The most problematic level of tiebacks is the topmost. It has the greatest distance to cover, the weakest soil in which to anchor, and the greatest likelihood of interfering elements. This top-most level will also have the greatest amount of deflection associated with a given load and will have the greatest influence on the settlement of neighboring structures and soil behind the wall. Estimated ultimate transfer capacities for various soil and rock conditions can be found in Sabatini et al (1999) and Strom and Ebeling (2001).

9.2. Capacity Testing and Lock-off

Methods to test for anchor capacity are usually dictated by construction restraints. The tests are fairly quick, however creep evaluations are often made. Rarely are tests performed prior to excavation. Loading sequences may include a proof test to a level well above design load, a performance test where the load is cycled in steps to different loading levels, or a creep test to determine any long-term movement under constant loading. Maximum loads during testing may be as high as 150% of design load. Excursions to such high values insure that the proper location of the anchor resistance is mobilized and the zone nearer to the wall has already yielded (see figure15). Lock off values will vary from 50 to 100% of design load, depending again on construction conditions and contractor/engineer experience. Measurements of tieback extension during capacity testing allow for some evaluation of elastic properties of the tiebacks as well as creep potential.

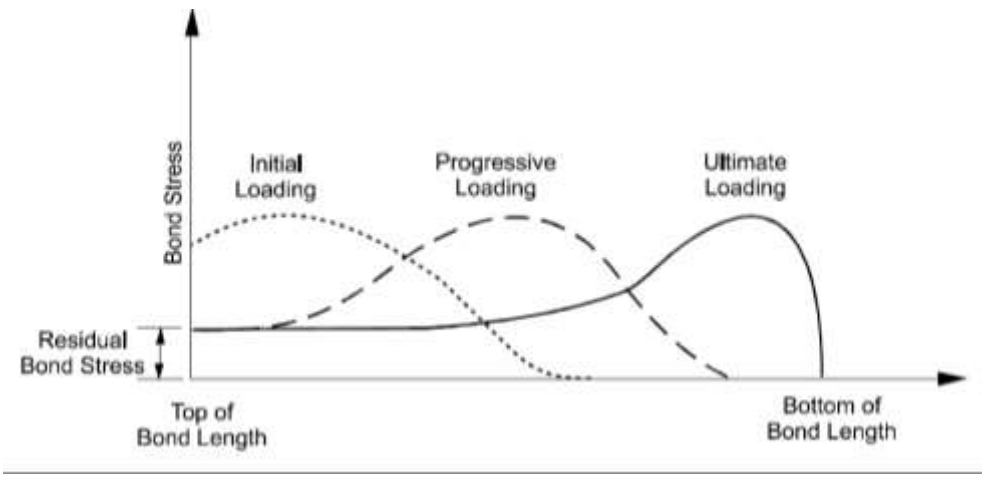

Figure 13. Mobilization of bond stress for a tension anchor.

9.3. Corrosion Protection

Knowledge of the soil (and backfill) environment is foremost in assessment of corrosion potential. Primarily soil and groundwater pH and resistivity will influence likelihood of corrosion. Most corrosion prevention is in the form of coatings or barrier layers. Galvanic corrosion due to stray currents may occur in urban settings, hydrogen embrittlement may occur under high stresses. Manufacturers are aware of these dangers and provide some guidance to reducing the negative effects of corrosion.

10. COMPUTING MOMENTS IN WALL

Computations of structural moments in the wall can be accomplished by simplified methods shown in figure 16, or more sophisticated structural analysis methods. Keep in mind that the greatest variability is in the combinations of earth pressures and anchor loads, so many combinations of loads should be checked. Constructability issues must be considered, especially when designing reinforcing steel and specifying concrete properties. Figure 1 was the result of high steel percentages and low slump concrete. For an above-ground structure they were typical; for a drilled pier wall, they were unacceptable.

Suggestions for wale design can be found in Strom and Ebeling (2002a, b), Sabatini et al (1999), structural steel manuals, and general wall design manuals (FHWA-RD-97-130)

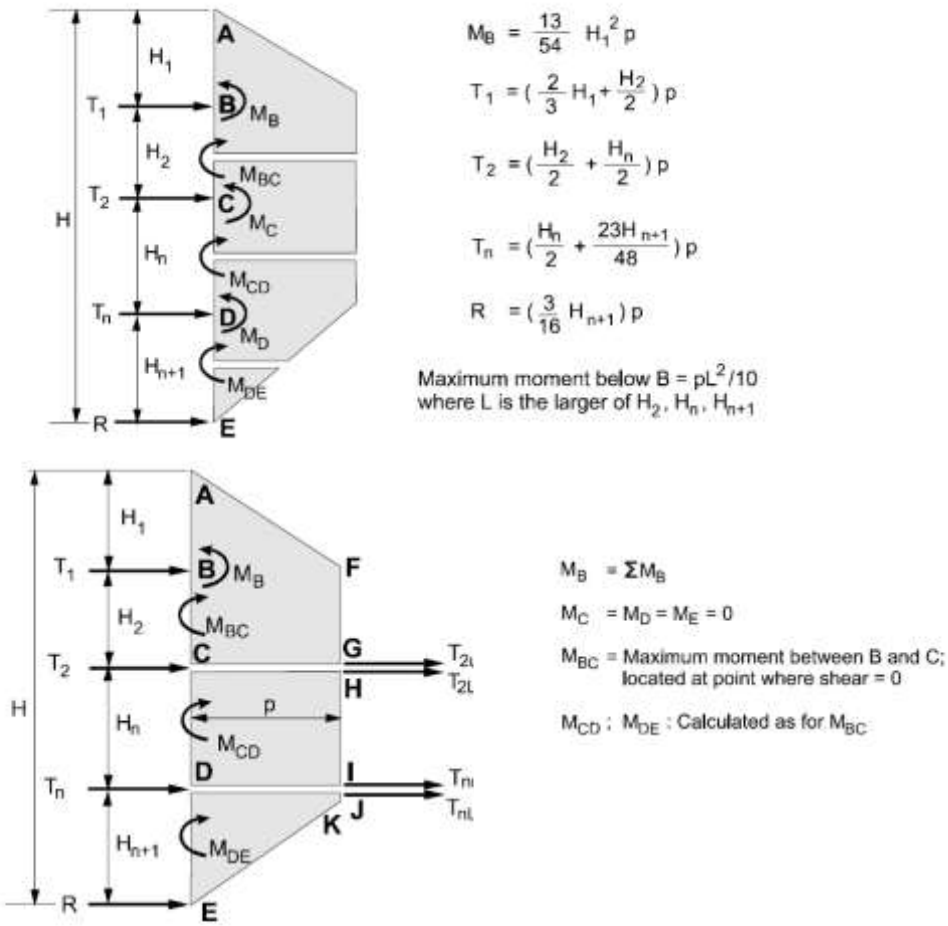

Figure 14. Computation of moments in wall by tributary and hinge methods

11. DEEP STABILITY AND VERTICAL CAPACITY

11.1. Stability in Soft Clays

Perhaps the most vulnerable condition for a deep excavation is the possibility of deep-seated movement around the wall and support system. Such a failure scenario is sometimes difficult to analyze since it is not a "clean" stability problem as one may see in slopes or retaining walls. The projected failure surface may be littered with tiebacks, structural elements, unusual pore pressures due to dewatering programs and the excavation process itself. The fact that this is all going on while construction is taking place everywhere leads to very complicated analysis. O'Rourke and McGinn (2006) offer a detailed discussion of evaluating deep stability of an anchored wall in soft clay. Their paper emphasizes the process of selecting appropriate soil properties as input to the

analyses. Additional details of the stability analyses undertaken are in O'Rourke and O'Donnell (1997) and other issues with effective stress analysis in discussions of the paper by Whittle and Ladd (1998). The system stiffness chart (figure 17), developed by Mana and Clough (1981) is often used to estimate lateral wall movements. It can be used to estimate system stability as well. What it also shows is the inevitability of wall movement regardless of wall stiffness when basal heave becomes a factor. This chart may undergo revision since analysis methods did not have the advantage of small strain nonlinear soil models as they do today (O'Rourke and McGinn, 2006).

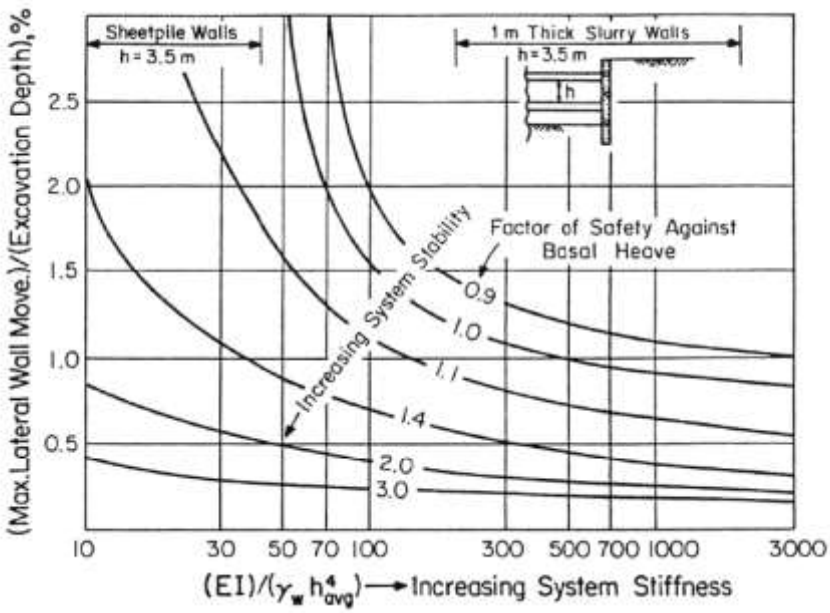

Figure 15. System stiffness chart for excavation (Mana and Clough 1981)

Global stability can be assessed using typical limit equilibrium (slope stability) methods. However, the effects of the tieback system must be accounted for. This is done in its simplest for by applying the stabilizing forces of the tiebacks to the wall as a horizontal surcharge load if the wall penetrates the failure surface, or as an inclined load if the wall does not (figure 18). A more complex method to analyze the contribution of anchors is to include its contribution directly into the force or moment equilibrium calculation. The maximum contribution will be a function of anchor capacity, similar to modeling soil-nailed slope.

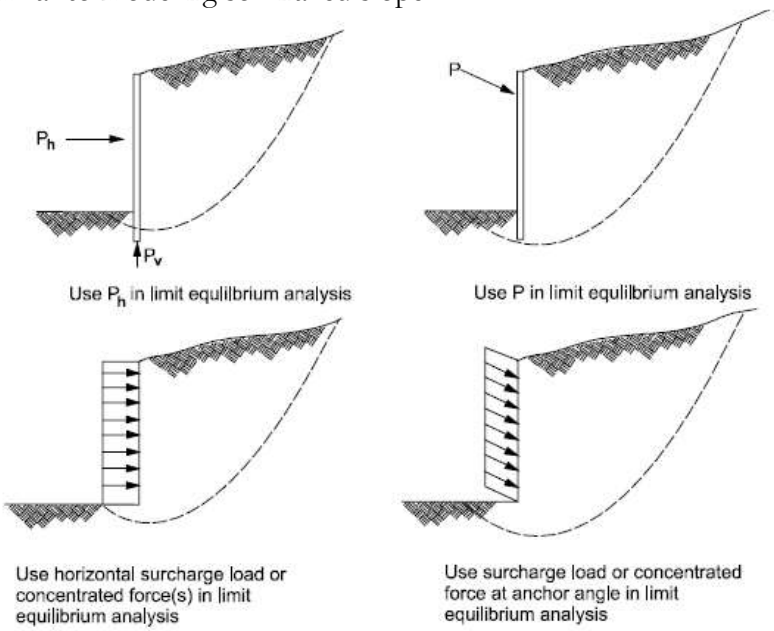

Figure 16. Global stability and contribution of ground anchors

Vertical axial capacity must be checked when the wall is founded in a weaker bearing material. The vertical component of tieback forces must be carried in one form or another, preferably by the wall itself in vertical capacity. Guerra et al (2004) list 20 case studies of loss of vertical stability of concrete soldier piles in clay. They offer possible failure mechanisms and methods to calculate instability.

12. SOIL MOVEMENT BEHIND WALL

As a starting point, Peck's 1969 State of the Art paper addresses magnitude of movement behind walls with different stiffnesses and soil strengths. One of the figures from that paper is shown in figure 19.

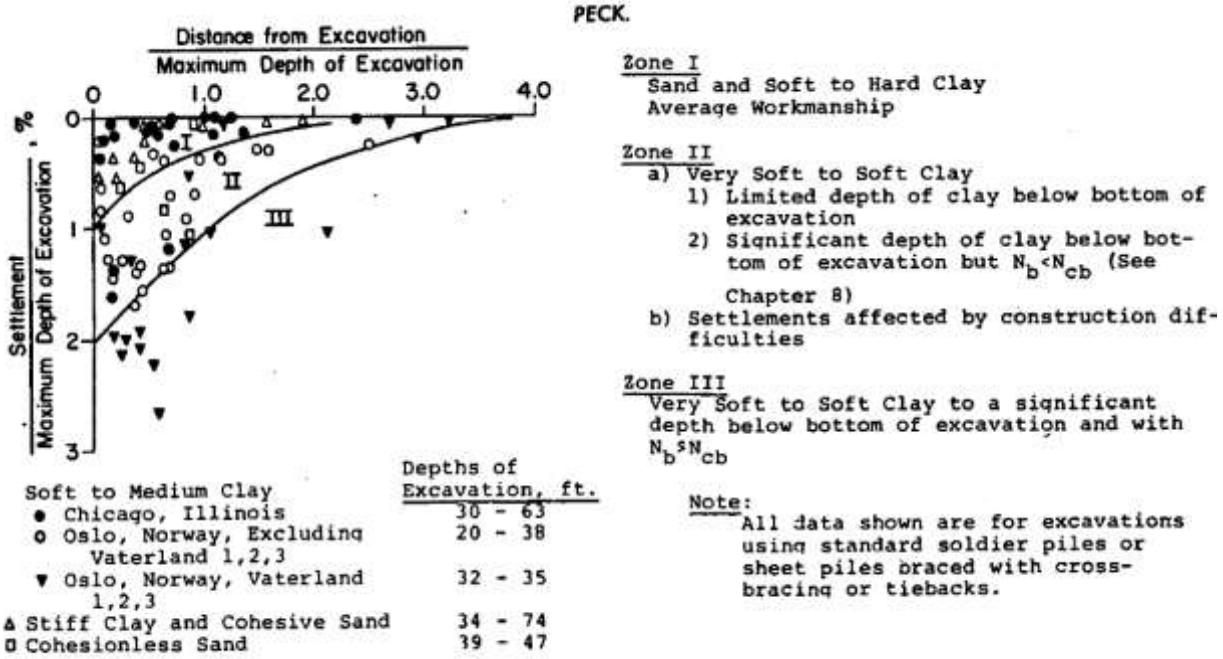

Figure 19. Settlement profile measurements and estimates (Peck, 1967)

A further study by Golberg et al 1975 collected data from 63 case studies of excavation displacements. Later papers by Clough and O'Rourke systematized the methodology based on Rowe's dimensionless wall stiffness parameter (see figure 18 above). These profiles have been become more generalized and are now presented as limits to movement (figure 20).

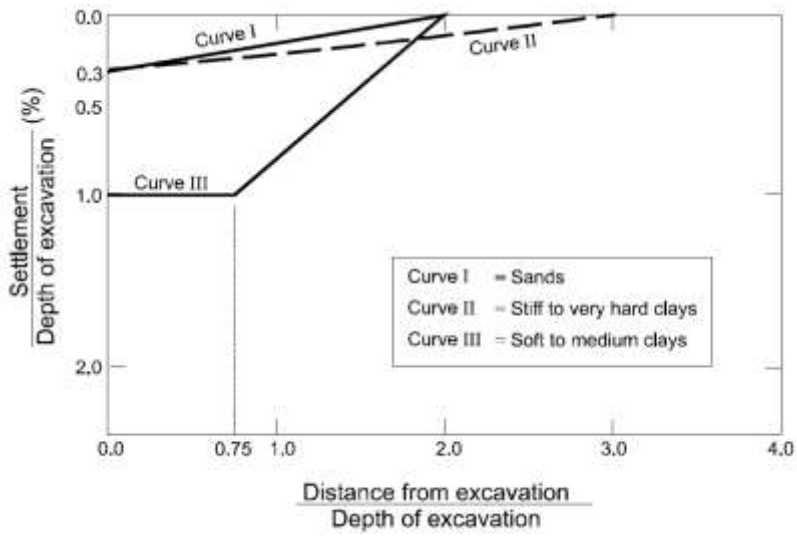

Figure 17. Settlement estimates for various soils and distances

12.1. Parallel to an Excavation Wall

Roboski and Finno (2006) presented expressions for distributions of the lateral and vertical movements and maximum distortion, respectively, in terms of excavation geometry and maximum displacement at a point (δ_{max}). The distribution of settlement is important for determining the likelihood of building damage nearby.

$$\delta(x) = \delta_{\max}\left(1 - \frac{1}{2} * erfc\left(\frac{2.8\,(x + L_w\,[0.015 + 0.035\,\ln\frac{H_e}{L_w}])}{0.5L_w - L_w\,[\,0.015 + 0.035\,\ln\frac{H_e}{L_w}]}\right)\right) \qquad \textbf{Eq.7}$$

$$Maximum\ \ Distortion = \frac{2.8\delta_{\max}}{0.5L_w\left(0.97 + 0.069\,\ln\frac{H_e}{L_w}\right)\sqrt{\pi}} \qquad \textbf{Eq. 8}$$

where $\delta(x)$ = settlement or lateral movement at distance x from the corner of the wall,
δmax = the maximum movement, and
H_e/L_w = ratio of the depth of the excavation to the length of the wall.

Equations 7 is an empirically-based complementary error function which can be used when the excavation-induced ground movements can develop with little restraint. Use of any of these empirically-based methods to estimate a movement distribution requires an independent evaluation of the maximum movement. This can be estimated from previous graphs and tables or pther analyses. Son and Cording (2005) demonstrated the usefulness of using quantities of lateral strain and angular distortion to estimate the likelihood of building damage. Their findings are shown in figure 21.

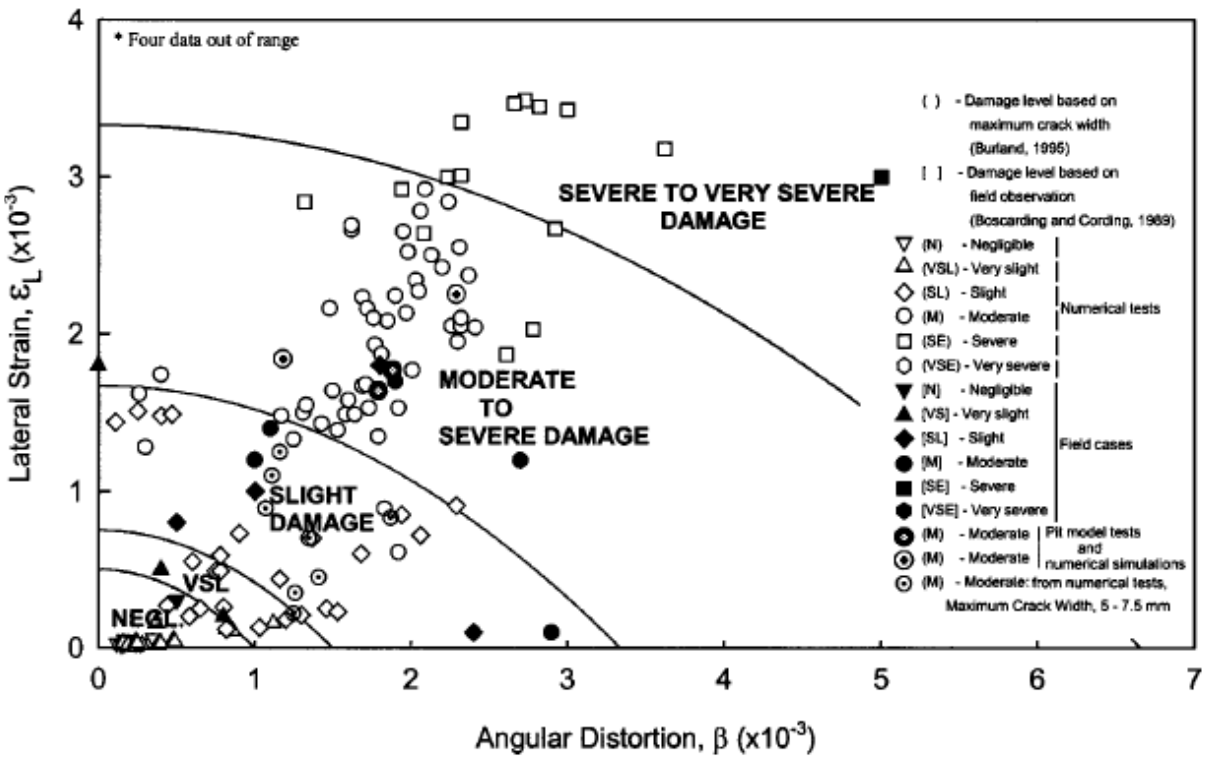

Figure 18 . Comparison of damage estimation criteria and damage levels observations from field and physical model tests and numerical analyses (Son and Cording, 2005)

13. CONCLUSIONS

Design methodologies for tieback wall design are presented with broad variety of sophistication and data requirements. All methods realize the effects of wall movement and wall stiffness on the magnitude of earth pressures considered for design. Simplified methods such as the rigid method account for these effects indirectly by making adjustments to earth pressure diagrams and assumptions about wall fixity. Winkler and finite element methods incorporate them more directly by adjusting the soil response functions themselves.

Estimating the amount of wall movement after a complex series of construction steps is still a challenge. Determining the best soil properties to place in these models is very difficult due to the highly variable states of stress generated by the excavation and tieback installation process. Iterative design by nonlinear finite element method is now possible and the design office level. Correlating results from these models to field performance still requires a great deal of engineering judgment.

REFERENCES

Clough, G.W., Smith, E.M. and Sweeney, B.P. (1989). *Movement control of excavation support systems by iterative design.* Proc., Foundation Engineering:Current Principles and Practice, ASCE, GSP 22, 869-884.

Clough, G. W. and O'Rourke, T. D. (1990). *Construction induced movements of insitu walls.* Design and Performance of Earth Retaining Structures. Proc. Specialty Conference at Cornell University, ASCE, New York, 439-470.

Ebeling, R. M., Azene, M. and Strom, R. W. (2002). *Simplified procedures used for the design of tall flexible anchored tieback walls.* ERDC/ITL, TR-02-9 U.S. Army Engineer Research and Development Center,Vicksburg, MS.

Finno, R.J. and Roboski, J.F. (2005). *Three-dimensional responses of a tied-back excavation through clay. J. Geotech. Geoenv. Engrg.*, ASCE, 131(3), 273-282.

Goldberg, D.T., Jaworski, W.E. and Gordon, M.D. (1976). *Lateral support systems and underpinning.* Report FHWA-RD-75-128, Vol. 1, FHWA, Washington, D.C.

Hsieh, P. G. and Ou, C. Y. ~1998!. *Shape of ground surface settlement profiles caused by excavation. Can. Geotech. J.*, 35, 1004–1017.

Kerr, W. C. and Tamaro G. J. (1990). *Diaphragm walls-Update on Design and Performance.* Design and Performance of Earth Retaining Structures, Proc. of a Specialty Conference at Cornell University, Geotechnical Special Publication 25. ASCE.

Long, J.H., Weatherby, D. E. and Cording, E.J. (1998) *Summary Report of Research on Permanent Ground Anchor Walls, Volume I: Current Practice and Limiting Equilibrium Analsyses*. Report FHWA-RD-98-065, Federal Highway Administration, McLean, VA.

Mana, A.I. and Clough, G.W. (1981). *Prediction of movements of braced cuts in clay.* J. Geotech. Engrg. Div., ASCE,107(GT6), 759-778.

Mueller, C.G., Long, J.H., Weatherby, D. E. and Cording, E.J., Powers III, W.F. and Briaud, J.-l. (1998). *Summary Report of Research on Permanent Ground Anchor Walls, Volume III: Model-scale Wall Tests and Ground Anchor Tests.* Report FHWA-RD-98-067, Federal Highway Administration, McLean, VA.

O'Rourke, T. D. (1998). *Closure to Deep rotational stability of tiedback excavations in clay*. J. Geotech. Geoenviron. Eng., 124(12) 1125–1228.

O'Rourke, T. D. and McGinn, A. J. (2004). *Case history of deep mixing soil stabilization for Boston Central artery.* Geotechnical Engineering for Transportation Projects, Geotechnical Special Publication No. 126, ASCE, Reston, Va., 77–136.

O'Rourke, T. D., McGinn, A. J. and Dewsnap, J. (1998). *Case history of excavation stabilized by deep mixing methods.* Geotechnical Special Publication No. 83, Design and Construction of Earth Retaining Systems, ASCE, Reston, Va., 41–61.

O'Rourke, T. D. and O'Donnell, C. J. (1997a). *Deep rotational stability of tiedback excavations in clay*. J. Geotech. Geoenviron. Eng.,123(6), 506–515.

O'Rourke, T. D. and O'Donnell, C. J. (1997b). *Field behavior of excavation stabilized by deep soil mixing.* J. Geotech. Geoenviron. Eng., 123_6_, 516–524

O'Rourke, T. D., McGinn, A. J., Dewsnap, J. and Stewart, H. E. (1997). *Performance of excavations stabilized by deep soil mixing.* Rep.Prepared for Bechtel/Parsons Brinckerhoff, the Massachusetts Highway Department, and the Federal Highway Administration at Cornell Univ., Ithaca, N.Y.

ASCE/SEL (2000). Effective analysis of diaphragm walls," Joint Technical Committee on Performance of Structures During Construciotn, ASCE/Structural Engineers Institute.

Peck, R. B. (1969). *Deep excavations and tunneling in soft ground.* State of art report. Proceedings, 7th international conference on soil mechanics and foundation engineering. Mexico City, Mexico, 225-90.

Peck, R.B., Hanson, W.E. and Thornburn, T.H. (1974). *Foundation engineering.* John Wiley and Sons, New York.

Ratay, R. T. (1996). *Handbook of temporary structures in construction*. McGraw-Hill, New York.

Rowe, P.W. (1952). *Anchored Sheet Pile Walls.* Proceedings Institute of Civil Engineers, Vol. 1, pp. 27-70.

Sabatini, P. I., Pass, D. G. and Bachus, R. C. (1999). *Ground anchors and anchored systems.* Geotechnical Engineering Circular No. 4, FHWA-SA-99-015, Federal Highway Administration, Washington, DC.

Sara, R.E (1997). *Site Characterizaiton for Design of Municipal and Hazardous Waste Landfills.* CRC press.

Strom, R. W., and Ebeling, R. M. (2001). *State of Practice in the Design of Tall, Stiff and Flexible Tieback Retaining Walls.* ERDC/ITL TR-01-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Strom, R. W. and Ebeling, R. M. (2002a). *Simplified procedures used for the design of tall stiff anchored tieback walls*. ERDC/ITL, TR-02-10, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Strom, R. W. and Ebeling, R. M. (2002b). *Methods used in tieback wall design and construction to prevent local anchor failure, progressive anchorage system failure, and ground mass stability failure.* ERDC/ITL TR-02-11, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Ulrich, E.J. Jr. (1989). *Tieback Supported Cuts in Overconsolidated Soils*. Journal of Geotechnical Engineering, ASCE, Vol 115, No. 4, pp 521-545.

Ulrich, E.J. Jr. (1991). *Closure to Tieback Supported Cuts in Overconsolidated Soils*. Journal of Geotechnical Engineering, ASCE, Vol 117, No. 1 pp 834-836.

Wang, S-T. and Reese, L. C. (1986). *Study of design methods for vertical drilled shaft retaining walls*. Texas State Department of Highways and Pubic Transportation, Austin.

Weatherby, D.E. (1998). *Design manual for permanent ground anchor walls.* Report FHWA-RD-97-130, Federal Highway Administration, Washington, DC.

Weatherby, D.E. (1998). *Summary Report of Research on Permanent Ground Anchor Walls. Volume IV: Conclusions and Recommendations.* Report FHWA-RD-98-068, Federal Highway Administration, McLean, VA.

Weatherby, D. E., Chung, M., Kim, N-K. and Briaud, J-L. (1998). *Summary report of research on permanent ground anchor walls; II, Full-scale wall tests and a soil-structure interaction model.* Report FHWA-RD-98-066, Federal Highway Administration, McLean, VA.

Xanthakos, P.P. (1991). *Ground anchors and anchored structures*. John Wiley, New York, 686 p.

Son, M. and Cording, E.J. (2005). *Estimation of Building Damage Due to Excavation-Induced Ground Movements.* Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 131, No. 2, pp 162-177.

Kostantakos, D.C., Whittle, A., Regalado, C. and Sharner, B. (2004). *Control of movements for a multi-level-anchored diaphragm wall during excavation.* Proc. Fifth Int'l Conf. on Case Histories in Geotechnical Engineering, paper 5.68, 7 pp.

Hong, S.H., Lee, F.H., Yong, K.Y. (2003). *Three-dimensional pile-sol interaction in soldier-piled excavations.* Computers and Geotechnics 30, pp. 81-107.

Mosher, R. L., and Knowles, V. R. (1990). *Finite element study of tieback wall for Bonneville Navigation Lock.* Technical Report ITL-90-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Geurra, N.M.daC., Cardoso, A.S., Fernandes, M.M., and Correia, A.G. (2004) *Vertical stability of Anchored Concrete Soldier Piles in Clay.* Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 12, pp 1259-1270.