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A Manual for the Design of Temporary Earth Retention Systems (TERS) for the Michigan Dept. of Transportation

Center for Structural Durability at Michigan Tech – a MDOT Center of Excellence

FINAL REPORT

Part 1 of 2

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Engineering Manual Preamble

This manual provides guidance to administrative, engineering, and technical staff. Engineering practice requires that professionals use a combination of technical skills and judgment in decision making. Engineering judgment is necessary to allow decisions to account for unique site-specific conditions and considerations to provide high-quality products, within budget, and to protect the public health, safety, and welfare. This manual provides the general operational guidelines; however, it is understood that adaptation, adjustments, and deviations are sometimes necessary. Innovation is a key foundational element to advance the state of engineering practice and develop more effective and efficient engineering solutions and materials. As such, it is essential that our engineering manuals provide a vehicle to promote, pilot, or implement technologies or practices that provide efficiencies and quality products, while maintaining the safety, health, and welfare of the public. When making significant or impactful deviations from the technical information from these guidance materials, it is expected that reasonable consultations with experts, technical committees, and/or policy setting bodies occur prior to actions within the time frames allowed. It is also expected that these consultations will eliminate any potential conflicts of interest, perceived or otherwise. Michigan Department of Transportation Leadership is committed to a culture of innovation to optimize engineering solutions.

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2. Perform Services only in areas of their competence.
3. Issue public statement only in an objective and truthful manner.
4. Act for each employer or client as faithful agents or trustees.
5. Avoid deceptive acts.
6. Conduct themselves honorably, reasonably, ethically, and lawfully so as to enhance the honor, reputation, and usefulness of the profession.

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The purpose of this manual is to provide basic guidance on the design of temporary earth retaining systems (TERS) used on Michigan Department of Transportation (MDOT) projects. Temporary works are designed as short-term systems during construction and are either removed after construction or, in some cases, left in place. MDOT must approve all TERS with a design height of six feet or greater before construction. This manual covers the basic design elements for a TERS by providing nine detailed design examples. The structural steel design of typical internal bracing of TERS is provided in the manual's final chapter.</td></tr>
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1 Introduction

1.1 Scope

The purpose of this manual is to provide basic guidance on the design of temporary earth retaining systems (TERS) used on the Michigan Department of Transportation (MDOT) projects. Temporary works are designed as short-term support systems and are either removed after construction or, in some cases, are left in place. MDOT must approve all TERS designs that are six feet in height above the dredge line or higher *prior to construction*. This manual discusses the basic design concepts for cantilevered sheet piles, anchored sheet piles, internally braced cofferdams, and soldier pile and lagging for temporary works. In general, cofferdam construction is common for bridge piers over waterways, while sheet piling is more common for staged construction or when temporary earth retention is required.

A summary of the design concepts is provided, followed by examples of the design steps for each type of TERS. There are several software programs available for the design of TERS such as SPW 911 by PileBuck International, Inc., Support IT by GTSoft Ltd., or CivilTech Software Shoring Suite. This manual uses the software program SupportIT (www.GTSoft.org) for the design examples. MDOT must approve the use of other software not listed above for the design of TERS.

General considerations for the design of sheet pile retaining wall systems during all stages of construction include the following:

a) Evaluation of the earth and hydrostatic pressures that act on the TERS
b) Determination of the required depth of piling penetration
c) Calculation of the maximum bending moments in the piling
d) Calculation of the stresses and deflections in the wall and selection of the appropriate piling section
e) Design of the anchorage and waling systems

It is important to note that it is the designer's responsibility to understand these essential elements for the design and installation of TERS. Also, designers and contractors must follow the project documents for specific design and construction requirements.

1.2 Manual's organization

This manual is divided into six chapters providing essential design information for temporary earth retaining systems. Following this introductory chapter, Chapter Two provides a general description of typical TERS used on MDOT projects. Chapter Three provides a review sheet piling design systems. Chapter Four discusses soil properties used in the design of TERS. Chapter Five offers geotechnical design examples of cantilever walls, anchored walls, internally braced cofferdams, and soldier pile walls. Aspects of the structural design of TERS are provided in Chapter 6, the final chapter.

1.3 Terms and Definitions

The following terms and definitions have been adapted from the US Army Corps of Engineers Design of Sheet Pile Walls (US Army Corps of Engineers, 1994) for use in this manual.

1. **Active Pressure**: The limiting pressure between the wall and soil produced when the relative wall/soil motion tends to allow the soil to expand horizontally.
2. **Anchor:** A device or structure which, by interacting with the soil or rock, generates the required anchor force.
3. **Anchor Force**: The reaction force (usually expressed per foot of wall), which the anchor must provide to the wall.
4. **Anchorage:** A mechanical assemblage consisting of wales, tie rods, and anchors that supplement soil support for an anchored wall.
    a. *Single anchored wall*: Anchors are attached to the wall at only one elevation.
    b. *Multiple anchored walls*: Anchors are attached to the wall at more than one elevation.
5. **Anchored Wall**: A sheet pile wall that derives its support from a combination of interaction with the surrounding soil and one (or more) mechanical devices which inhibit motion at an isolated point(s).
6. **At-rest Pressure**: The horizontal in situ earth pressure when no horizontal deformation of the soil occurs.
7. **Backfill:** A generic term applied to the material on the retained (Active) side of the wall.
8. **Braced Wall**: A braced wall is a wall that is supported by braces or struts that transfers the lateral earth pressures (and water pressures) between opposing walls through compressive struts.
9. **Cantilever Wall:** A sheet pile wall that derives its support solely through interaction with the surrounding soil.
10. **Dredge Line**: A generic term applied to the soil surface on the dredge side of a retaining wall system.
11. **Dredge Side**: A generic term referring to the side of a retaining wall with the lower soil and/or water surface elevation. In some software, this is also known as the "Passive Side".
12. **Factor of safety**:
    a. Factor of safety for the rotational failure of the entire wall/soil system (mass overturning) is the ratio of available resisting effort to driving effort.
    b. Factor of safety (strength reduction factor) applied to soil strength parameters for assessing limiting soil pressures in classical design procedures.
    c. Structural material factor of safety is the ratio of limiting stress (usually yield stress) for the material to the calculated stress.
13. **Foundation**: A generic term applied to the soil on either side of the wall below the elevation of the dredge line.
14. **Passive Pressure**: The limiting pressure between the wall and soil produced when the relative wall/soil motion tends to compress the soil horizontally.
15. **Penetration**: The depth to which the sheet piling is driven below the dredge line.

16. **Retained Side**: A generic term referring to the side of a retaining wall with the higher soil surface elevation. In some software and manuals, the retained side is referred to as the "Active Side."
17. **Retaining Wall**: A sheet pile wall (cantilever or anchored) that sustains a difference in soil and/or water surface elevations from one side to the other. The change in soil surface elevations or water elevation may be produced by excavation, dredging, backfilling, or a combination.
18. **Sheet Pile Toe:** The base of the sheet pile that is driven into the ground.
19. **Sheet Pile Upstand**: The height of the sheet pile above the ground surface on the retained or active side of the sheet pile wall.
20. **Sheet Pile Wall**: A row of interlocking, vertical pile segments driven to form an essentially straight wall whose plan dimension is sufficiently large that its behavior may be based on a typical unit (usually 1 foot) vertical slice.
21. **Soil-structure Interaction**: A process for analyzing wall/soil systems in which compatibility of soil pressures and structural displacements are enforced.
22. **Tie Rods**: Parallel bars or tendons which transfer the anchor force from the anchor to the wales.
23. **Top of Sheet Pile:** The top of the sheet pile and the point where the sheet pile is hammered into the soil.
24. **Wales**: Horizontal beam(s) attached to the wall to transfer the anchor force from the tie rods to the sheet piling and in braced sections of sheeting the wales transfer loads to the struts. Also referred to as walers.
25. **Wall Height**: The distance measured from the ground surface on the dredge side to the ground surface on the retained side of the sheet pile.

2 Temporary Earth Retaining Structures used on MDOT Projects

Temporary Earth Retaining Systems (TERS) used on MDOT projects generally include the following systems: (1) cantilever sheet pile walls, (2) anchored sheet pile walls, (3) internally braced sheet pile walls, and (4) soldier pile and lagging walls. Each of these systems is briefly reviewed in this chapter. Combinations of these systems can also be used on projects. More specialized walls, such as column piles and concrete diaphragm walls, are not be covered in this manual.

2.1 Cantilever sheet pile walls

Cantilever sheet pile walls, according to Head and Wynne (1985), account for approximately 75% of all sheet pile walls constructed in the 1980s in the US and UK. In terms of design, however, they are considered a subset of anchored sheet pile walls. Tschebotarioff's (1972) well-known textbook on foundation design includes a section on anchored bulkheads (in his chapter on waterfront structures) but does not include the design cantilever walls. Interestingly, the term "cantilever" is not included in the book's index. A possible reason for this omission is that cantilever walls are generally limited to sand, gravelly soils, or stiff clays and are usually not applicable in soft clay. A significant limitation to cantilever sheet pile walls is that they can experience large deflections. Also, they tend to be more susceptible to erosion and scour in front of the wall. While MDOT does not restrict the height of cantilever walls, in general, cantilever sheet piles are constructed to a maximum height of 12 to 15 ft.

Figure 2-1 illustrates a simple illustration of a cantilever wall. Cantilever sheet pile walls are driven to a depth where the wall becomes stable while supporting the lateral stresses acting on the wall that develop during and after excavation and without the use of additional supports such as anchors, bracing, or other structural elements.

The design of cantilever sheet piling walls must follow the MDOT 2012 Standard Specification for Construction (MDOT 2012) and the AASHTO Standard Specifications for Highway Bridges, 17th edition (AASHTO 2002).

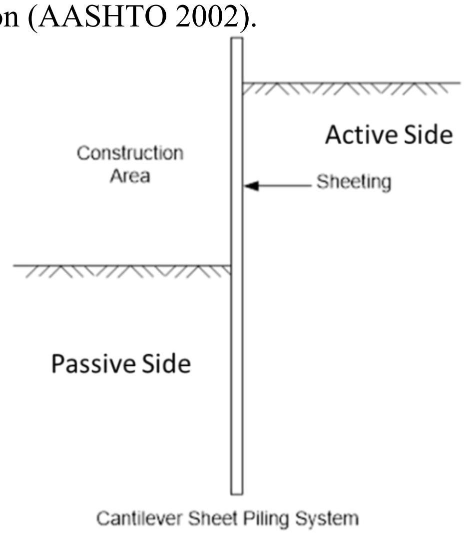


Figure 2-1 Cantilever sheet pile retaining wall system.

2.2 Anchored sheet pile walls

Additional support is generally required for cantilever walls higher than 12 to 15 feet to limit wall deflections and higher bending moments in the sheet pile wall. To minimize wall deflections, anchors are placed near the top of the sheet pile wall. The lateral support for anchored earth retaining wall systems comes from the lateral passive earth pressure on the embedded portion of the wall and anchor near the top of the piling. The additional support anchors allow walls to be constructed more than 35 ft. Figure 2-2 shows typical single and multiple anchored sheet piling systems.

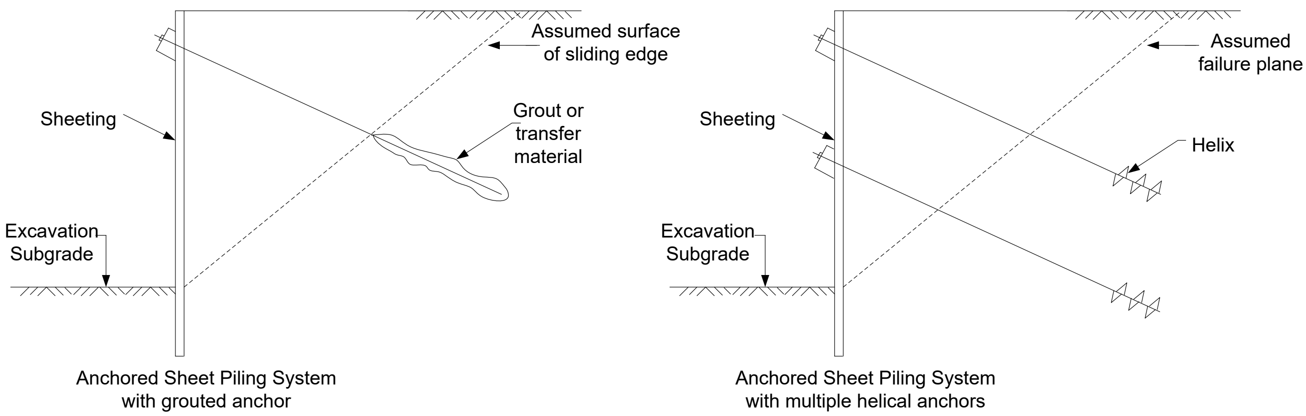


Figure 2-2 Anchored sheet pile earth retaining wall systems.

It is important to note that anchored walls are used for "top-down" construction and, therefore, are constructed in stages because each phase of construction affects the lateral earth pressure on the steel sheeting. Consequently, each phase of construction must be designed, analyzed, and submitted as part of the design documents for review. For example, different phases of construction for multiple anchored system designs include cantilever design (excavation to install the first anchor), anchored analysis (excavation below the first anchor to install the second anchor) and multiple anchored analysis (when all the anchors are installed). Anchored steel sheet piling walls must follow MDOT 2012 Standard Specification for Construction (MDOT 2012) and the AASHTO Standard Specifications for Highway Bridges, 17th edition (AASHTO 2002).

2.3 Internally braced cofferdams

Internally braced cofferdams are temporary earth and water retention systems used to support the sides of deep excavation and for the construction of foundations in water and soft clay. Cofferdams can be partially or wholly enclosed structures. Cofferdam structures consist of vertical steel sheet piling and are often internally braced (supported) by a system of wales and struts. Figure 2.3 shows a schematic of internally braced cofferdams indicating the various parts of the braced cofferdam. Wide-flange beams for wales and stringers, transmit the lateral earth and water pressure forces on the sheet piling to the internally braced struts. For enclosed cofferdams, the wales and stringers can also be subjected to axial loads at the corners of the structure.

For typical braced cofferdams, sheet piles must be driven deeper than the excavation to develop bottom anchorage. During construction, walers are placed horizontally along the length of the excavation and supported by horizontal struts. In some construction projects, a tremie concrete seal is constructed at the base of the excavation to provide bottom bracing, a base to work on, and to control water uplift pressure. In this case, the tremie concrete seal must be designed for combined axial and bending. The axial loading is from earth pressure, and bending is from the uplift pressure. The thickness of the tremie concrete seal must be provided in the plans. Soil movement and the subsequent pattern of deformation associated with each type of support must be considered at all stages of construction. Design of steel sheet piling and cofferdams must follow MDOT 2012 Standard Specification for Construction (MDOT 2012) and the AASHTO Standard Specifications for Highway Bridges, 17th edition (AASHTO 2012).

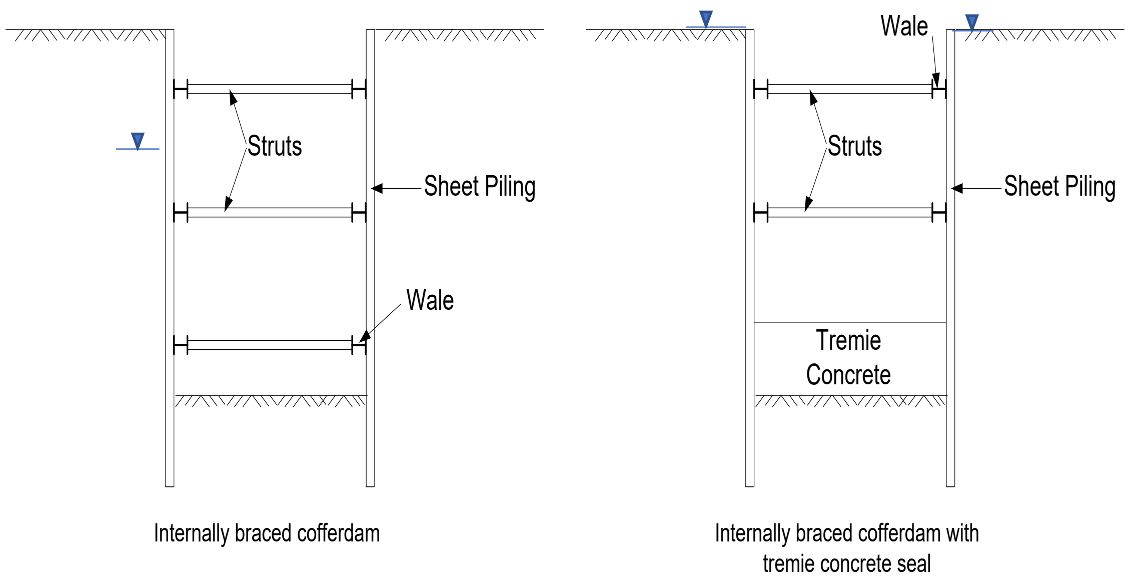


Figure 2-3 Internally braced cofferdams for an earth excavation.

The following steps are generally followed when constructing an internally braced cofferdam with high groundwater.

1. Drive steel sheet pile to build the enclosure.
2. Excavate to one foot below the top strut/wale elevation.
3. Install top wales/struts.
4. Continue excavation.
5. Once groundwater is encountered, it is essential to keep the water in the cofferdam at the same elevation as the water outside the cofferdam to maintain static equilibrium.
6. Continue excavation to the bottom of the tremie seal elevation or excavation limit.
7. Drive piles if required.

8. Pour tremie concrete if required. Keep water in the cofferdam at or slightly above the water elevation outside the cofferdam during the tremie pour and cure period. Do not allow the water to overtop the cofferdam.
9. After the tremie concrete has reached at least 50% of the design strength, the cofferdam can be dewatered to install subsequent bracing levels.
10. Dewater to tremie seal: form and cast footing.
11. The abutment wall can be formed and poured. Struts may need to be removed and adjusted.
12. Backfill.
13. Remove walers/struts.
14. Remove or cut-off steel sheet piling.

The pier foundation sequence is similar, except it may need to be built in open water.

2.4 Cantilevered soldier pile walls

Cantilevered soldier pile walls consist of vertical steel or concrete structural members with lagging placed between the vertical members. The vertical members, generally steel beams, are placed in pre-drilled holes and grouted at a spacing of between six to ten feet on center. The vertical spacing, however, is designed by the contractor's engineer. It is important to note that soldier pile walls differ from sheet pile walls in that the passive support below the dredge line is not continuous, i.e., only the soldier pile beam's width below the dredge line can develop passive resistance to the active soil and water pressures.

After the vertical beam or "soldier pile" has been set, excavation begins in cuts of about five feet depending on soil's ability to stand-up. A soil's "stand-up" time is considered the length of time that the soil can maintain an unsupported vertical face without sloughing. After excavation, horizontal sheeting, commonly called lagging, is placed between the installed soldier piles. Lagging is frequently made of wood planks, but may also consist of light steel, sheeting, corrugated guardrail sections, or precast concrete. For soils with short stand-up times, such as dry sand, lagging must be installed immediately after excavation. In soils that maintain a vertical face for more extended periods, lagging can be placed when the complete lift has been excavated.

Soldier piles are either installed with pile driving equipment, but on MDOT projects are usually set in pre-excavated holes and then concreted in place. The most common soldier piles (vertical sections) are made from rolled steel sections, usually wide flange or bearing pile. Deeper wide flange sections are used where higher stiffness and flexural strength is required in the soldier pile. Cantilevered soldier piles depend on passive resistance of the foundation material and the moment-resisting capacity of the vertical structural members for stability. Therefore its maximum height is limited by the competence of the foundation material and the moment-resisting capacity of the vertical structural members. The unanchored economic height of this type of wall is generally limited to a height of 18 feet (FHWA 1999).

Soldier pile walls can also be constructed with anchor tie-backs or struts to allow significantly higher wall heights. Although soldier pile walls of 100 feet have been built, wall heights on the order of 35 ft or less are typical on MDOT projects. Figure 2-4 illustrates a temporary soldier pile wall.

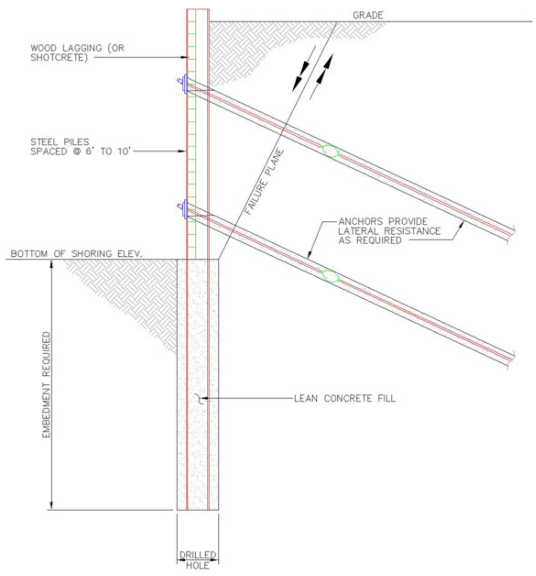


Figure 2-4 Example of a temporary soldier pile construction.

A significant design element of large soldier pile walls with anchor tie backs is the analysis of the "overall" or "global" stability of the wall. All sheet pile walls, including soldier pile walls, must be checked for adequate overall or global stability. Figure 2-2-5 illustrates a case where the anchors from a soldier pile wall do not extend beyond a potential slip surface, resulting in a potentially unstable wall.

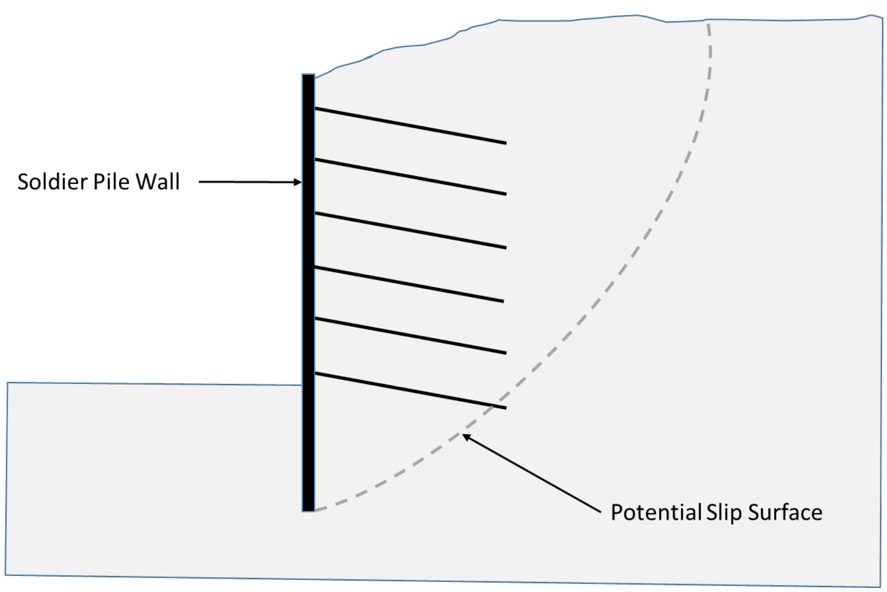


Figure 2-2-5 Potential slip surface for a soldier pile wall.

2.5 Shallow trench excavations

Per MDOT Standard Specifications for Construction (MDOT 2012), excavations six feet in depth or less do not require design. These excavations, however, must be constructed following MiOSHA regulations for trench excavations (MiOSHA 2013). It should be noted, however, that MiOSHA trenching rules and regulations apply to any trench excavation over five feet in depth. The MiOSHA regulations require that site soils be inspected by a designated competent person who can require the trenches to be sloped to a stable angle and be braced with an approved trench jack or box.

2.6 Wall deformations induced by excavation

Important design consideration of a TERS is estimating the magnitude of wall movement, which must be limited to minimize the ground movement outside of the excavation. In this manual and several software programs, wall movement is referred to as "deformation." The magnitude of wall deformations is a function primarily of the TERS's stiffness and lateral loads from soil and water. Essential factors in the magnitude of deformation for cantilever and anchored walls include wall penetration depth and stiffness, as well as the number of anchor levels and the anchor spacing. For braced excavations, the excavation's width and depth along with the strut spacing, stiffness, and preload will be factored into the amount of wall deformation and resulting ground movement that will develop outside the excavation.

2.7 Construction Defects

An equally important issue is construction defects. According to Ou (2006)'s Deep Excavation – Theory and Practice book

> "*Construction defects can cause, in less serious situations, extra wall deflection, greater ground settlement and excavation, bottom movement or, in serious conditions, the collapse of excavation and damage to adjacent buildings and public facilities. The magnitude of stress and deformation due to construction defects cannot be predicted through theoretical simulation or empirical formula. Such conditions can only be prevented by the improvement of construction quality.*"

Common construction defects include (Ou 2006, Gaba et al. 2003):

- inadequate support due to insufficient embedment,
- buckling of the struts providing lateral wall support,
- structural inadequacy of the connection between the strut and the wall,
- inadequate foundation for raking struts,
- leakage through the retaining wall in which soils can flow out of the wall causing a void to form behind the wall and surface settlement,
- dewatering inside cofferdams during excavation resulting in unbalanced hydrostatic pressure before bracing is installed,
- pulling out used sheet piles and creating a void at a depth that is difficult to fix,

- over-excavation in which the contractor excavates below the designed depth, which can result in large amounts of deformation and possible collapse of the system,
- inadequate workmanship and poor construction control,
- installation of bracing with incorrect orientation, and
- field substitution of lesser size bracing section without approval.

3 Sheet Pile Analysis Methods

There are two analysis methods commonly used for sheet pile design, the fixed-earth method and the free-earth method. The difference between the two methods is the assumption of how the active and passive stresses develop along the sheet piling and the flexibility of the sheet piling. The fixed-earth method assumes flexible sheet piling that develops a "*point of fixture*" at a depth below the dredge line where the pile does not move. Below the "*point of fixture*," the active and passive stresses reverse. The free-earth method, on the other hand, assumes that the piling is ridged and free to move to allow the active and passive pressures to fully develop along the sheet piling.

In general, the fixed-earth method is used in the design of cantilever and cantilevered soldier pile walls while the free-earth method is used in the design of anchored and braced walls. The following sections discuss the various design aspects of fixed and free-earth design methods.

3.1 Gross and net pressure diagrams

Traditionally, there have been two ways to illustrate the earth pressure against a sheet pile wall, a gross-pressure diagram, and a net-pressure diagram. Figure 3-1, from the US Sheet Pile Manual example number 1, illustrates the two methods for a cantilever wall in sand. The gross-pressure diagram is developed by calculating the active and passive pressures acting on the wall. The net-pressure diagram is developed by subtracting the passive pressure from the active pressure. Both pressure diagrams can be used in the design of fixed and free-earth designs. The US Steel Pile Manual uses a net-pressure diagram for a design example of cantilever walls while Peck et. al. (1974) use a gross-pressure diagram for a design example of cantilever walls. For simple sheet pile problems, the net-pressure diagram is commonly used. However, for multiple loading and soil layers, the gross-pressure diagram is commonly used. The gross-pressure diagram will be used in this manual for the design examples in Chapter 5.

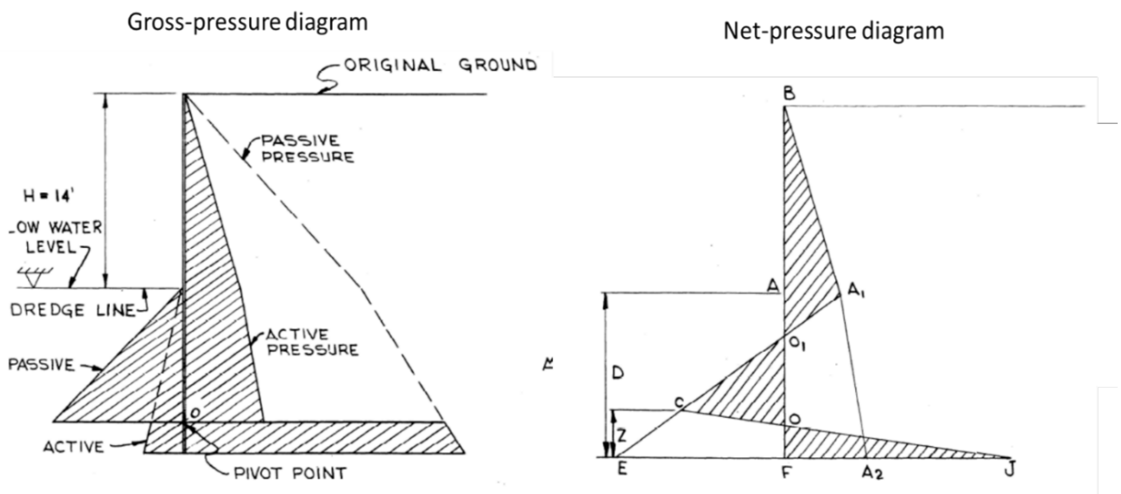


Figure 3-1 Gross-pressure and net-pressure diagrams from the US Steel Sheet Pile Manual (Design example number 1).

3.2 Fixed-Earth Method

A key assumption in the fixed-earth method is that the sheet piling is driven deep enough so that the piling becomes fixed at a location below the dredge line known as the "*point of fixture*" or the pivot point "O" as shown in Figure 3-2. The portion of the pile below the point "O," however, is assumed to rotate in the opposite direction, reversing the active and passive stresses. For this condition to develop, the soil must be strong enough to prevent pile movement at point "O."

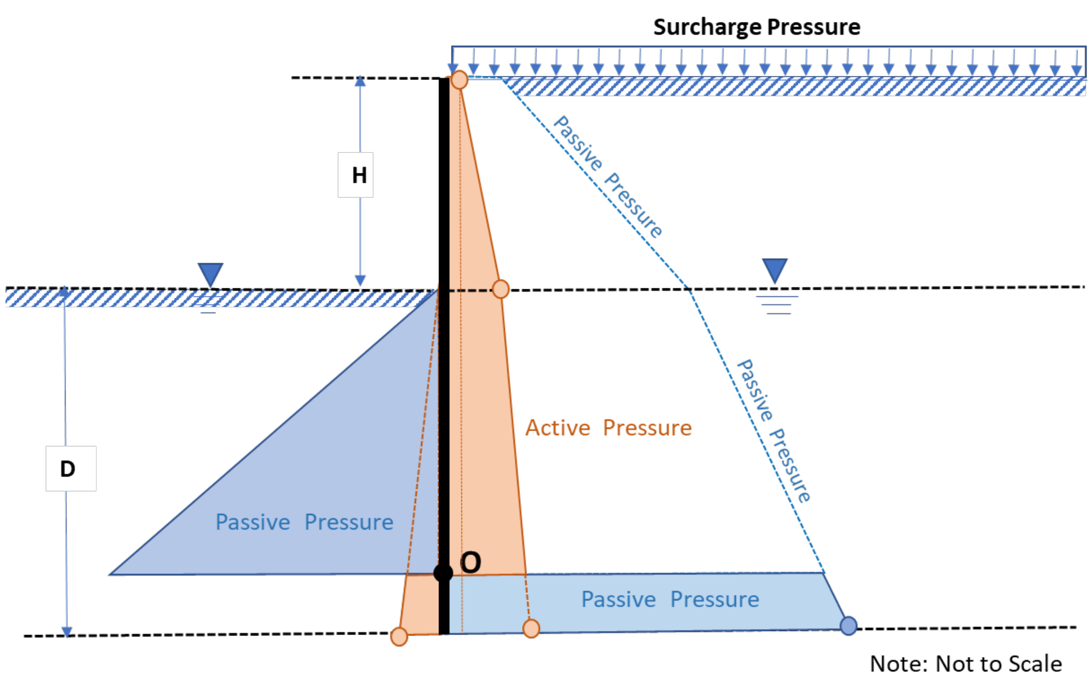


Figure 3-2 Fixed-earth sheet pile assumed pressures.

In the fixed-earth method, the requirements for static equilibrium are satisfied by taking the sum of the forces in the horizontal direction equal to zero and the sum of the moments about any point along the pile being equal to zero. In the US Steel Sheet Pile Manual, the depth of the wall is determined through an iterative trial-and-error process by solving for horizontal force and moment equilibrium involving the two following unknowns, "D," the sheet pile embedment depth, and the "z," the length of sheet pile below the point of fixture "**O**" and the end of the pile (see Figure 3-1). Because "D" and "z" are unknown, two equilibrium equations are required to solve for "D" and "z," which can be solved directly by hand or using software such as MathCad or MatLab. This method, however, becomes difficult if not impossible to solve with more than one soil layer or additional loads, e.g., surcharge load, line load, or layered soils, are included in the analysis.

Before the development of computers, an "*approximate*" or "*simplified*" method was developed in Germany by Blum (1930) and Krey (1936) to simplify the fixed-earth analysis method. The US Steel Sheet Piling Design Manual (USS 1984) and Teng's (1962), for example, refers to this method as the "*simplified method*." The simplified method uses a gross-pressure diagram and replaces the soil pressures acting on the wall below the pivot point by a single force "R" acting at the pivot point "**O**," as shown in Figure 3-3. Taking the summation of moments at the pivot point eliminates the force "R" resulting in only one unknown, D_o, the depth from the dredge line to the pivot point, **O**. Once the depth D_o is determined, the simplified method requires that the depth, D_o, be increased by 20% to compensate for the stresses acting on the wall below the pivot point in calculating the sheet piling's embedment depth, D_u, as shown in Figure 3-4. The subscript "u" referrers to this depth as being "*unfactored*" since the analysis assumes that the sheet piling is at its limiting condition, i.e., at a factor of safety at unity or FOS = 1.0. A factor of safety can then be assigned to the unfactored depth D_u to arrive at a final depth D_f. For example, a standard method used in the US Sheet Pile Manual is to add 20 to 40% on to the D_u length, as shown in Figure 3-4.

The benefit of the simplified method is that it allows the designer to analyze any number of soil layers or loads on the wall, including cohesionless and cohesive soil layers. Further, the simplified method is widely used today and forms the basis for many sheet pile software programs, including the SupportIT and SPW911 software programs. *The simplified method will be used in the design examples in Chapter 5 for cantilever and cantilever soldier pile walls.*

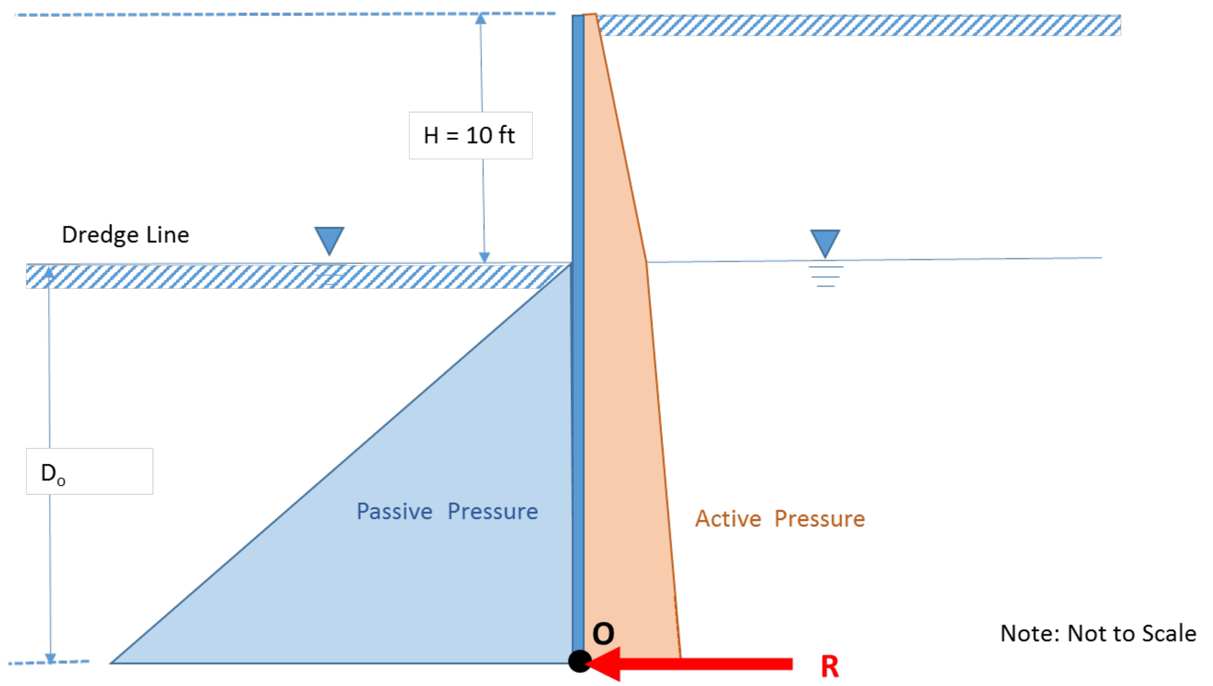


Figure 3-3 Case 1 Simplified method stress distribution.

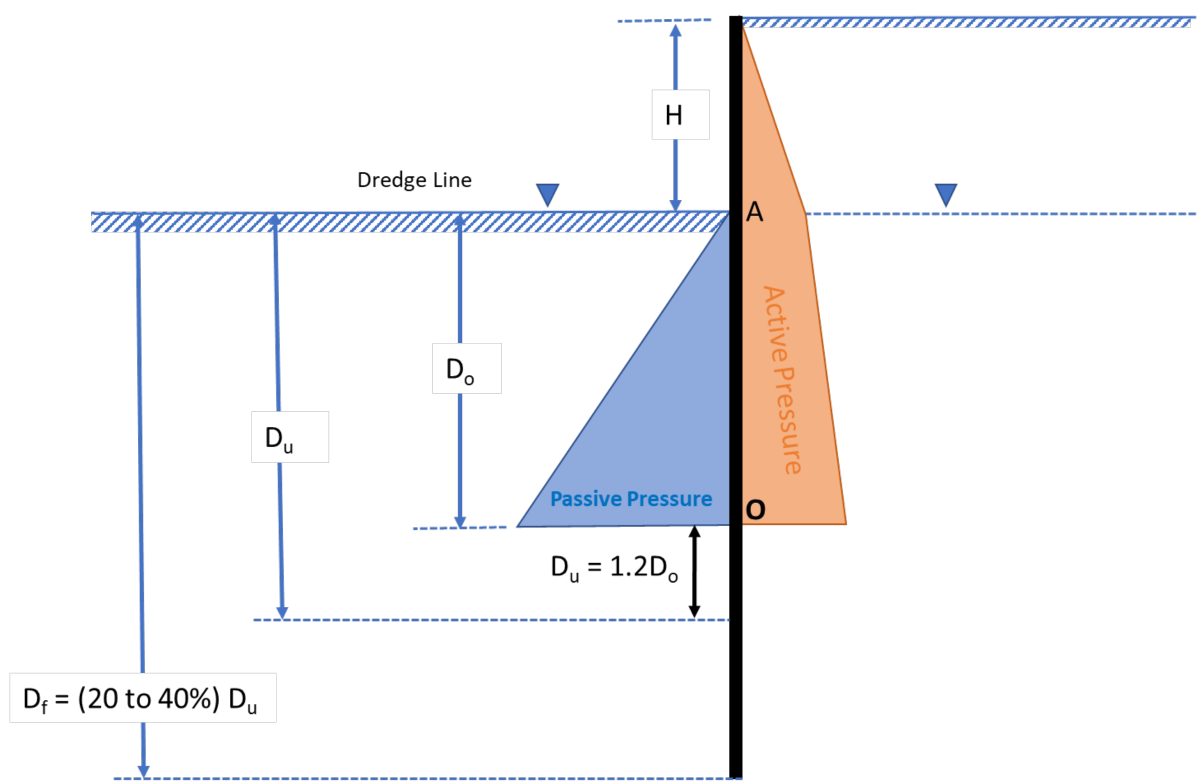


Figure 3-4 Case 1 Simplified method increases D_o by 1.2

The general procedures used in the manual for fixed-earth design is as follows:

1. Determine the soil strength parameters for each soil layer in the analysis.
2. Determine the soil's active and passive earth pressure coefficients, K_a, and K_p.
3. Determine the gross earth pressures acting on the sheet piling. The water pressure acting on the wall must be included, even if it is at the same elevation on both sides of the sheet pile. If there is a lag between the groundwater level in the retained soil and front of the wall, this difference must also be included.
4. Calculate the forces acting on the wall with the depth, D_o, as a variable.
5. Solve for D_o assuming a FOS = 1.0.
6. Multiply the depth, D_o, by 20% to determine the unfactored depth, D_u.
7. After calculating the depth, D_u, which is at a factor of safety equal to one, the location of zero (horizontal) shear forces, at a depth of D_s, is calculated. The zero shear location is the location of the maximum bending moment acting on the sheet piling.
8. Calculate the sheet pile's maximum bending moment.
9. Finally, determine the final embedment depth, D_f, by applying a factor of safety (FOS) to the unfactored embedment depth, D_u.

3.3 Free-Earth Method

According to Tschebotarioff (1973), the free-earth method is considered the oldest and conservative of the design methods for *anchored sheet pile design*. While the free-earth design method often provides economical designs with shorter embedment depth, the method also results in calculating more substantial bending stresses than the fixed-earth method. A critical assumption in the free-earth method is that the sheet piling is assumed to be rigid, rotating about the anchor where wall support is provided by an unyielding anchor. The pile's embedment depth is calculated by taking the moment equilibrium at the anchor, as shown in Figure 3-5. The load acting on the anchor is then calculated by taking the summation of horizontal forces acting on the sheet pile. The location of the maximum bending moment is determined as in the fixed-earth method at the location of shear zero. Since the sheet piling is assumed to be rigid, the method tends to overestimate the maximum bending moment. To reduce the maximum bending moment, Rowe (1953) developed a moment reduction procedure known as the "Rowe Moment Reduction Theory." This reduction method is based on the relative flexibility of the sheet piling and the soil-structure interaction. The more flexible sheet piling and the softer soils, the more significant the reduction will be. The Rowe Moment Reduction procedure is widely used today when determining the maximum bending stresses in a sheet piling design.

The free-earth method also assumes that the active and passive earth pressures fully develop along the sheet pile wall, as shown in Figure 3-5. Tschebotarioff notes, however, that this is a questionable assumption since this assumes that the soil below the dredge line has fully reached its limit shearing strength throughout the depth of the sheet pile's embedded depth. If the full passive resistance is not developed, then the soil is not capable of producing effective restraint to the sheet piling at the extent necessary to induce negative bending moments. To account for this underdevelopment of the passive forces, Tschebotarioff recommended that a factor of safety be applied to the passive pressures.

The effective stress and water pressures assumed in the free-earth method are also shown in Figure 3-5. This figure illustrates that both the effective stress as well as the water pressure that acts on the wall. Therefore, the water pressures must be included in the analysis for both fixed and free-earth methods, even when the water elevation is at the same on both sides of the wall.

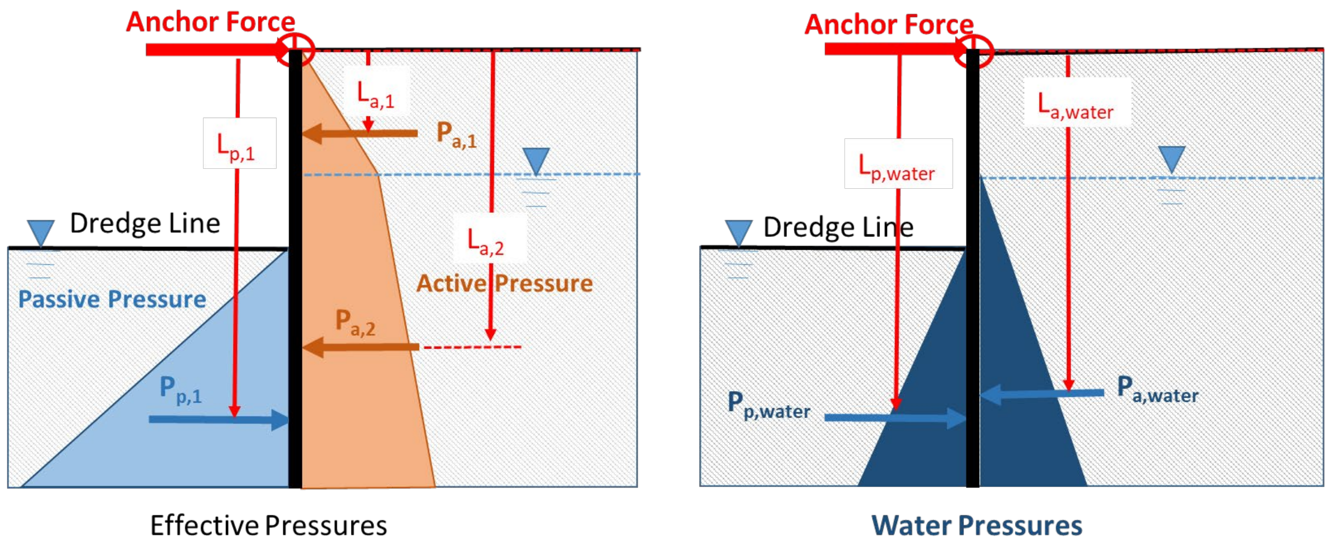


Figure 3-5 Free-earth design method for a cohesionless soil.

The general procedures used for free-earth design are as follows:

1. Determine the soil strength parameters.
2. Determine the soil's active and passive earth pressure coefficients.
3. Determine gross earth pressures acting on the sheet piling. This includes the water pressure acting on the wall below the groundwater table and if there is a lag between the groundwater level in the retained soil and in front of the wall, which must be accounted for in the design.
4. Determine the forces and forces moment arm acting on the wall with the embedment depth, D, as a variable. The moment arm is between the force and the anchor location.
5. Determine moment equilibrium at the anchor level with the embedment unfactored depth, D_u at a factor of safety equal to one.
6. After calculating the unfactored depth, D_u, calculate the point of zero shear (horizontal) forces acting on the sheet piling. This is the location of the maximum bending moment in the sheet piling.
7. After determining the location of zero shear, D_s, on the sheet piling, calculate the piling's maximum bending moment at this location.
8. Finally, determine the final embedment depth by applying a factor of safety to the embedment depth D_f.

3.4 Wall Friction

Wall friction will develop between the sheet piling and the soil with the net effect to assist in the stability of the pile, thus reducing the embedment's depth. The magnitude of the friction is a function of the soil friction strength ϕ. There have been numerous recommendations for determining wall friction. MDOT, however, limits wall friction to 10% of the soil's friction angle, ϕ.

3.5 Wall adhesion

Similar to wall friction, wall adhesion is calculated as a portion of the soil's undrained shear strength. Due to cohesive soil's remolding during pile driving, however, the strength of the cohesive soil is generally considered to be zero. Therefore, adhesion for temporary sheet pile installations is not generally included in the analysis.

3.6 Factor of Safety (FOS)

As discussed above, both the fixed-earth and free-earth methods determine the sheet piling's maximum bending moment using an unfactored embedment depth, D_u. A safety factor is then applied to the D_u to calculate the final embedment depth, D_f. There are, however, a number of safety factors used today. These factor of safety (FOS) include the following:

Method 1 – USS Sheet Piling Manual - Increase, D_u, by 20 to 40%
Method 2 – UK Code of Practice, Gross pressure CP2
Method 3 – Net Pressure
Method 4 – Burland-Potts
Method 5 – Soil Factor Values

The SupportIT Software allows the user to apply four of the five FOS methods. The *Gross Pressure (CP2), Net Pressure (BSPH)* and the *Burland-Potts* are applied in the "*Setup*" tab as shown in the left box in Figure 3-6 or the "*Wall*" tab as shown in the right box in Figure 3-6. The fourth method, "*Soil Factors*", is located at the bottom of the "*Wall*" tab.

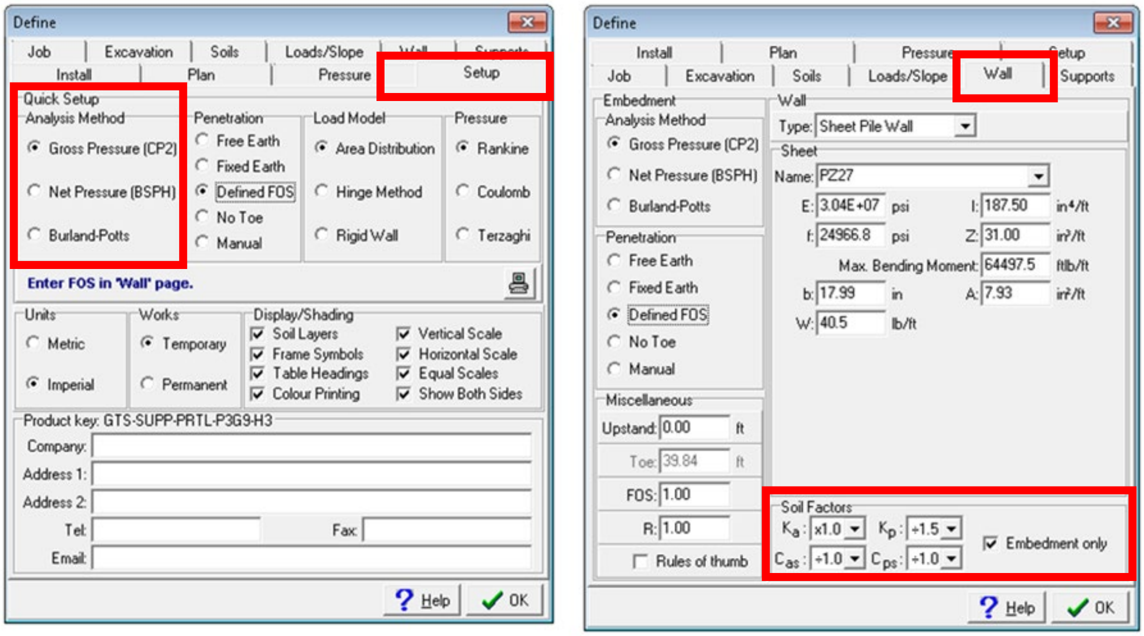


Figure 3-6 SupportIT Software's factor of safety methods.

3.6.1 USS Sheet Piling Manual

The simplest method to apply a factor of safety is to increase the unfactored embedment depth, D_u, by a certain percent. Terzaghi (1955), in his Norman Medal address, recommended that after adding appropriate factors of safety to the sheet pile design, the final embedment length should be increased at a minimum of 20%. Later, the USS Sheet Piling Manual (1969, 1984) recommended that the unfactored depth, D_u, be increased by 20 to 40%.

3.6.2 Civil engineering code of practice number 2, CP2

In 1951, the UK established the "Civil engineering code of practice no. 2, known as CP2". A factor of safety was introduced by applying a single factor of safety of at least 2.0 to the passive earth pressure. Tschebotarioff (1973) notes, however, that using a factor of safety of 2.00 is approximately equivalent to an increase in embedment depth of 70% compared to the method used in the USS Sheet Piling Manual that increases the embedment depth between 20 and 40%. Other foundation manuals have recommended factors of safety of between 1.5 and 2.0, e.g., Teng (1962).

3.6.3 Net Pressure Method

The Net Pressure method uses the net pressure distribution (active minus passive pressure) and defines the factor of safety as the restoring moments (net passive) divided by the overturning moments (net active). This is shown graphically in Figure 3-7.

$$\frac{\sum M_{net\ passive}}{\sum M_{net\ active}} = FOS$$

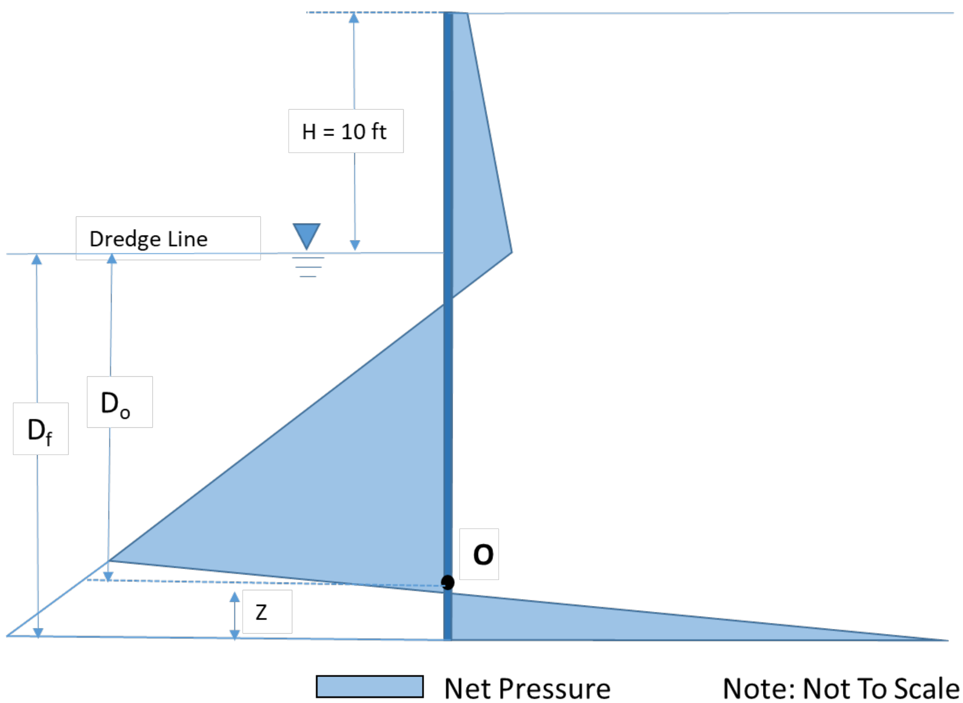


Figure 3-7 Net pressure factor of safety method.

3.6.4 Burland-Potts Method

According to the SupportIT manual, the Burland-Potts Method is defined as follows:

> "*Also referred to as the Revised Method, this method eliminates some of the balancing loads from the moment equilibrium equation. It consists of applying the FOS to the moment of the net available passive resistance, which is the difference between the gross passive pressure and the components of the active pressure, which result from the weight of soil below the dredge level, as shown by the unshaded area*".

$$\frac{\sum M_{Moment\ of\ net\ available\ passive\ pressure}}{\sum M_{Moment\ of\ retained\ material}} = FOS$$

The Burland-Potts method is shown graphically in Figure 3-8.

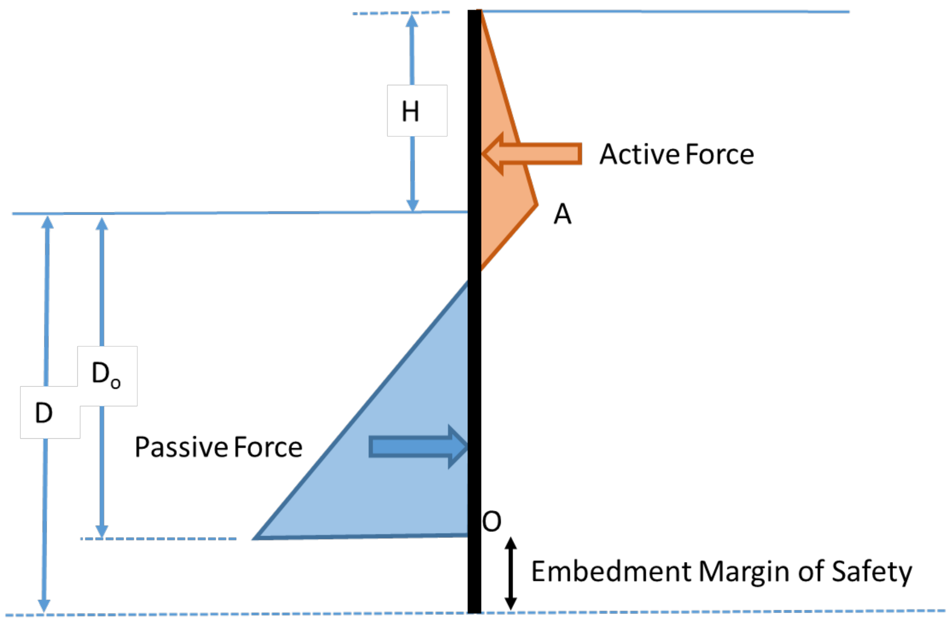


Figure 3-8 Burland-Potts factor of safety method.

3.6.5 Soil Factor Values

The British BS8002 "Code of Practice for Earth-retaining structures" (1994) eliminated the various safety factors and applied the FOS to the strength parameters instead. According to Bolton (1996) and Powrie and Simpson (2001), they recommend "*that safety factors can be most generally and most usefully be applied to soil strength, rather than to passive resistance, rotational moments, or structural load effects such as bending moments and prop forces*." By adding the factor of safety to the soil strength, this accounts for uncertainty in the soil parameters as well as the degree to which the soil strength is mobilized. For temporary structures, BS8002 (Circa 104, 2003) recommends that following values:

Effective Stress Parameters: c' and ϕ' $F_s = 1.2$ (but lower values when $\phi \geq 30°$)
Total Stress Parameters: $c_u = 1.5$

The strength reduction factors are applied as follows:

$$\phi_{reduced} = \frac{\phi\prime}{FOS} \qquad c'_{reduced} = \frac{c\prime}{FOS}$$

In the SupportIT software, the reduction factors can be added in the "Soil Factors" section in the "Wall" tab as shown in Figure 3-6.

4 Soil Properties and Lateral Earth Pressures

Construction of Temporary Earth Retention Systems (TERS) by their nature carry elevated risk due mainly to the difficulty in assessing the soil's strength and groundwater elevation, and their respective variabilities. An important factor in designing TERS, therefore, is in properly assessing the soils that will be excavated and supported by the TERS. Lateral earth pressures that act against the TERS, in turn, are a function of soil properties and groundwater conditions. Additional loading will result from the construction sequence utilized as well as from surcharge loads. This chapter will, therefore, review the following topics:

- Field Site Assessment
- Basic Soil Properties
- Soil Strength
- Lateral Earth Pressure
- Surcharge Loads

4.1 Field Site Assessment

4.1.1 Field Site Reconnaissance

Construction of TERS, by nature, can be a high-risk activity and, therefore, must be designed accordingly. The design must take into account the of potential risks observed during the field site assessment. The following list of items should be considered when assessing a site for a TERS:

- River water elevation (normal, low, high)
- Flood plain extent
- Groundwater elevation
- Historical use of the property
- Potential environmental contamination
- Potential for archeologically significant artifacts
- Distances to relevant structures
- Nearby foundation systems, e.g., spread footings
- Overhead utility lines
- Underground utilities and structures
- Blasting at or near the surface
- Potential levels of pile driving vibrations and airborne sound
- Unstable ground, e.g., karst, existing sinkholes, liquefiable soils
- Driving obstructions such as bedrock, hardpan clays, cobbles and boulders, and rip-rap.

4.1.2 Site Geotechnical Exploration

Site soils and rock investigations are conducted by MDOT personnel and/or consultants. The information from the investigation, which will generally include soil boring data, field testing and laboratory testing results, is provided to the contractor and their engineers for assessment via design plans and Reference Information Documents (RID).

A site investigation consists of conducting drilling operations (soil borings) to obtain soil samples and to identify the soil stratigraphy and groundwater levels. In addition, most drilling operations conduct Standard Penetration Testing (SPT) (ASTM D1586-79, 1984), which is used to assess the strength of soil as well as collect a disturbed sample of soil for visual identification and description of the soil's engineering properties. In the absence of higher quality laboratory test results, SPT data can be used by the TERS designer to estimate soil coefficients for soil strength from the relative density of coarse-grained soils (such as sand and gravels), or the consistency of fine-grained soils (such as silts and clays). Generally, correlations from SPT testing and laboratory unconfined compression test data are available to aid in this determination.

The cone penetration test (CPT (ASTM D3441-79, 1986)) is also becoming a standard soil test but has to be used with caution due to the presence of cobbles and boulders in the glacial soils, which covers the vast majority of Michigan.

The depth of bedrock, if within the general depth of sheet pile penetration depth, must also be determined. In most cases, drilling operations can establish the depth of bedrock. In some cases, however, geophysics methods, such as seismic refraction or electrical resistivity methods, can be used. The nature and condition of the bedrock in terms of rock strength and weathering should also be assessed.

It is essential that the groundwater conditions at the site be known. Drilling operations can generally confirm the groundwater table in coarse-grained soils. The designer, however, must be careful in fine-grain soils where the true level of the groundwater can be challenging to establish due to the low permeability of fine-grained soils. An essential source for groundwater level information is the Michigan Department of Environmental Quality (MDEQ) "Well Record Retrieval System." This web site can be accessed at http://www.deq.state.mi.us/well-logs/. In some sites, water level fluctuations can occur. The level and timing of the groundwater fluctuations must be known and accounted for in the design of the TERS.

4.2 Soil Properties

4.2.1 Field Soil Classification

MDOT uses a field soil classification system for describing the soils from the site investigation. The classification is the "Uniform Field Soil Classification System (Modified Unified Description)" and is provided in Appendix A. The design engineer must be familiar with this classification system since this classification is used to assess the strength parameters used in the design of TERS.

A key component of the system is identifying the primary soil constituents as being one of the following:

1. **Coarse-grained**, e.g., gravels and sands,
2. **Fine-grained soils**, silts, and clays.
3. **Organic soils**, e.g., peat and marl.

It is important to note that while the unified soil classification system provides a laboratory test to establish the type of soil, this modified classification is assessed in the field using visual and field tests. The field classification system further notes:

> *It should be understood that the soil descriptions are based upon the judgment of the individual making the description. Laboratory classification tests are not intended to be used to verify the description, but to further determine the engineering behavior for geotechnical design and analysis and construction.*

Secondary soil constituents represent one or more soil types other than the primary constituent that appears in the soil in significant percentages sufficient to readily affect the appearance or engineering behavior of the soil. An example would be a sandy soil with a fair amount of silt would be referred to as "silty sand." Note that the silty term modifies the primary constituent, which in this example is sand.

Tertiary soil constituents represent one or more soil types that are present in the soil in quantities sufficient that can be identify, but NOT in sufficient quantities to significantly affect the engineering behavior of the soil. The percent of tertiary soils in soil vary for coarse and fine-grained soils. For coarse-grained soils, tertiary soils can represent between 15 and 29%, while for fine-grained soil they can represent between 5 and 12%.

The following soil descriptors can also be added to the soil description.

1. **Color**: Brown, Gray, Yellow, Red, Black, Light-, Dark-, Pale-, etc.
2. **Moisture Content**: Dry, Moist, Saturated. Note that the descriptor is judged by the appearance of the sample before manipulating.
3. **Structure:** Fissured, Friable, Blocky, Varved, Laminated, Lenses, Layers.

An example of a soil description with soil descriptors would be a "*red laminated clay with sand*."

4.2.2 Selecting Soil Parameters

Soil properties vary both vertically and laterally and will have a distribution of values. Figure 4-1, from the British Embedded Retaining Wall Manual (CIRIA 2003), illustrates a typical normal distribution of design parameters. Parameter "C" in Figure 4-1 represents the mean (average) or the "most probable" soil parameter value. Value C, however, would only have a 50% probability (1 in 2) chance of being accurate. Value "B," on the other hand, would represent a worst-case value or a 1 in 1000 chance of being accurate. It is typical in this situation to select a design parameter that is at least one standard deviation from the mean or point “C” to the lower values. This is represented by a region “A” termed moderately conservative. In this way, the uncertainty of the strength of the soil can be addressed to some degree. Later, a factor of safety will be used to further minimize the probability of a failure of a TERS.

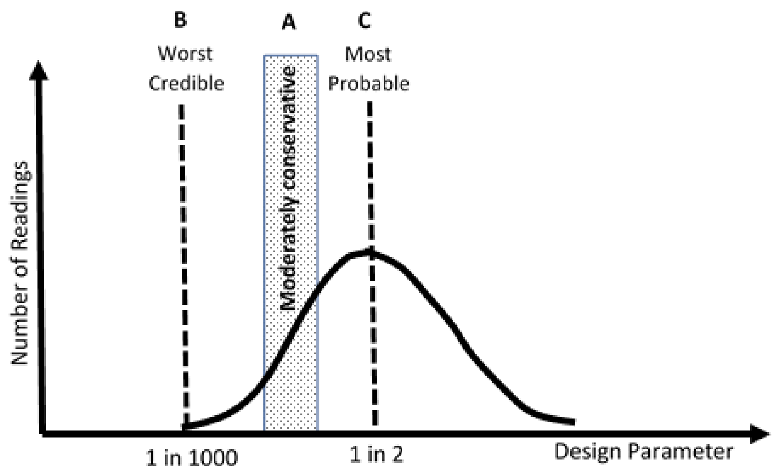


Figure 4-1 Normal distribution of soil values showing worst to most probable soil values.

4.2.3 Soil and Water Design Parameters

The Limit Equilibrium Method (LEM) of analysis will be used for the design of TERS. Essential design parameters used in LEM are as follows:

1. soil shear strength,
2. unit weight, and
3. groundwater condition.

Therefore, selecting appropriate parameters is essential for design. These topics will be covered in the following sections.

4.2.3.1 Soil Shear Strength

Soil strength for sheet pile design for LEM design will use the Mohr-Coulomb failure criteria, which is composed of two soil parameters, the angle of internal frictional φ' and a soil cohesion c'. The standard form of strength criteria is as follows:

$$\tau_f = c' + \sigma' \tan \varphi'$$

Where τ_f = soil’s shear strength at failure,
c' = cohesion,
σ' = effective stress,
φ' = angle of internal friction.

It is common to separate soils into two basic soil groups, coarse-grain soils, and fine-grained soils, to simplify the analysis for sheet pile wall designs. Soils will be assigned as being either cohesionless or cohesive. While this is a significant simplification, experience has shown that this assumption can be used effectively as long as appropriate soil strength values are selected, and a basic understanding of soil behavior is used to modify the strength values for various situations.

It is very important to note that the strength parameters φ' and c' are not intrinsic material properties but are parameters that depend on the applied stresses, the degree of consolidation under those stresses, and drainage conditions during shear, among others.

A short introduction to the various aspects of cohesionless and cohesive soils is presented along with methods to assess soil based on index values.

4.2.3.2 Effective Stress

Soil shear strength and deformation are a function of the effective stress in the soil. Therefore, when the soil is saturated, it is important to use effective stress to estimate the soil's strength and effective unit weight. Effective stress, σ', is the total stress, σ_t, minus the pore water, u, pressure and the effective weight, γ' is the saturated unit weight, γ_{sat}, minus the unit weight of water, γ_w, as shown below:

$$\sigma' = \sigma_t - u$$

$$\gamma' = \gamma_{sat} - \gamma_w$$

4.2.3.3 Standard Penetration Test, SPT

The Standard Penetration Test (SPT) obtains a disturbed sample of soil for visual identification and description, and for laboratory testing, e.g., particle size analysis and Atterberg Limits. MDOT allows the results from the SPT to be used to assign the shear strength values for both cohesionless and cohesive soils. To assess the soil strength, the SPT system drops a 130-pound hammer onto an 18-inch soil split spoon sampler. The number of hammer blows required to drive the sampler the last 12 inches is referred to as N value.

There are two important corrections that are made when obtaining SPT N values made by geotechnical firms and shown on contract plans. The first correction is to account for the "efficiency" of the hammer. Older manual SPT rope pull systems have a lower efficiency than do the newer hydraulic SPT systems. The standard correction is to adjust the N values to an efficiency of 60% to be consistent with the older rope pull systems and then report the N value as N_{60}. The second correction is used to correct the N_{60} value for overburden depth. This correction is needed because the N value of soil will increase with overburden stress (depth). The adjustment is made by adjusting the values to a nominal overburden pressure of 2,000 psf, which is a depth of about 15 to 16 feet. In general, this correction is made by the designer, and only for non-cohesive soils.

4.2.3.4 Coarse-Grained Soil Shear Strength

Coarse-grain materials are non-cohesive and consist of gravels, sands, and in some cases, coarse-grained silts. An essential and key property of coarse-grained soils is that they have relatively high permeability and can be considered to have "*drained conditions*." This is an essential consideration because it is assumed that during the construction of a TERS, "excess" pore water pressures do not develop in the soils and that the groundwater table can move freely in the soil. As noted above, the shear strength is characterized by the angle of internal friction (φ). For most coarse-grained soils, the cohesive strength is considered to be zero, which is $c = 0$. In addition, because only minimal excess pore water will develop during shear loading, the friction angle φ is the same for both total and effective stresses, that is $\varphi' = \varphi$. The value of φ for coarse-grain soils varies depending on the particle shape, gradation, and relative density.

4.2.3.5 Fine-Grained Soil Shear Strength

Fine-grained soils consist of low permeability soils such as silts and clays and are cohesive. For short term construction, the "*undrained shear strength*," S_u, is used to design the TERS. The undrained strength assumes excess pore water pressure fully develops when the soil deforms. This is known as a "*total stress*" condition because it assumes that during construction and utilization of the TERS, only minimal excess pore water will dissipate over the temporary life of the TERS. Therefore, tests such as the STP, pocket penetrometer, and hand shear vane can be used to assess the undrained shear strength of the soil. Assuming that the excess pore water has fully developed in the soil, the shear strength of the soil S_u is equal to the cohesion, c, of the soil or

$$S_u = \tau_f = c$$

The angle of internal friction is assumed to be 0.

4.2.3.6 Overconsolidated Fine-grain Soils

Because TERS are temporary structures, the long-term nature of fine-grain soils is not considered. It is important to emphasize, however, that silts and clays can have very different stress histories, and many fine-grained soils tend to be over-consolidated near the surface, which tends to make the soil stiff to very stiff. Fine-grain soils that are normally consolidated can be soft to stiff. Over time, though, over-consolidated soils start to soften, reducing the strength of the soil. In cases where softening might be an issue, a more detailed analysis of the strength of the soil needs to be conducted.

4.2.3.7 Soil Unit Weight

The lateral earth pressures that develop against a TERS are a function of the unit weight of the soil. Because higher unit weights will result in higher lateral loads, it is essential that more conservative values be used.

4.2.3.8 Soil Permeability

While the soil's permeability is an important parameter, it is not directly used in the LEM design. Casagrande, 1936, provided a range of soil permeability values with the following terms and ranges:

Good Drainage:	1 to 10^{-4} cm/sec	(2,820 feet/day to 0.28 feet/day)
Poor Drainage:	10^{-4} to 10^{-7} cm/sec	(0.28 feet/day to 0.00028 feet/day)
Impermeable:	$<10^{-7}$ cm/sec	(<0.00028 feet/day)

In general, soils considered to have good drainage can be assumed to have drained conditions, while soils with poor drainage would be considered to have "undrained conditions." The design engineer will need to determine whether the soil strength can be modeled using a "drained condition" or an "undrained" condition when designing a TERS.

4.3 Earth Pressure Theories

Determining the lateral loads that act against a TERS is an important aspect of designing a TERS and, therefore, must be carefully determined. There are, however, a number of methods available for selecting earth pressures. In addition, there are some common methods that should not be used in certain situations. The following sections present the lateral earth pressure theories commonly used for the design of TERS on MDOT projects.

4.3.1 At-Rest, Active and Passive Earth Pressures

The lateral or horizontal stresses that develop in soil are functions of the vertical stress, strength, and boundary conditions of the soil, including its deformation and stress history. The vertical stresses, σ_v, can be determined with a fair degree of accuracy by knowing the depth of the soil, z, and the soil's unit weight, γ, as follows:

$$\sigma_v = z\,\gamma$$

The horizontal stresses, on the other hand, is much more difficult to determine. Instead, the horizontal stress is determined using a coefficient, K, which is the ratio of the vertical stress to the horizontal stress, as shown in the following equation:

$$K = \frac{\sigma_h}{\sigma_v}$$

There are three conditions (or states) soil can be in with respect to the horizontal stresses, K_o the at-rest condition, K_a the active condition, and K_p the passive condition. The at-rest condition is when the soil's horizontal stresses develop as the vertical stresses increase, and no movement of soil occurs. The active condition develops when the existing horizontal stresses in the soil are reduced, such as when a wall rotates outward, relieving the horizontal stresses in the soil but not the vertical stresses. Passive conditions develop when the horizontal stresses increase while the vertical stresses remain constant such as when forces are placed on a wall increasing the horizontal stresses behind the wall. This is a rather simplification of these conditions, but in general, when designing a wall to handle vertical or lateral stresses, it is generally adequate.

Typical ranges of the three coefficients are as follows:

1. At-rest, K_o, (Typical range: 0.2 to 0.5)
2. Active, K_a, (Typical range: (0.15 to 1.0)
3. Passive, K_p, (Typical range: 1.0 to 10.0).

Note that the active coefficient is always less than one, and the passive coefficient is greater than one but less than 10.

The most common equations used to assess the at-rest coefficient, $K_{o,}$ is the Jaky Equation shown below which also includes Poisson's Ratio for the soil, μ

$$K_o = 1 - sin\phi' = \frac{\mu}{1-\mu}$$

For overconsolidated soils (OCR) the following equation can be used

$$K_o = (1 - sin\ \phi')(OCR)^{\sin \varphi'}$$

Note, however, that K_o is not used in the LEM analysis, but is used in more complex modeling systems when the soil-structure interaction is included in the analysis of TERS.

The active condition occurs when a retaining wall moves outward into the excavation allowing the horizontal stresses to be reduced behind the wall. At some point the soil fails behind the wall, limiting the horizontal stresses, σ_h, to the "Active Limit" or

$$\sigma_h = K_a\ \sigma_v$$

The passive condition occurs when the soil is "pushed" against the wall, and the horizontal stresses increase until the soil fails. At this point, the soil is at its passive limit. The lateral earth pressure or horizontal stress, σ_h, can be determined by

$$\sigma_h = K_p\ \sigma_v$$

A simplified illustration of the active and passive stresses action on a wall is shown in Figure 4-2.

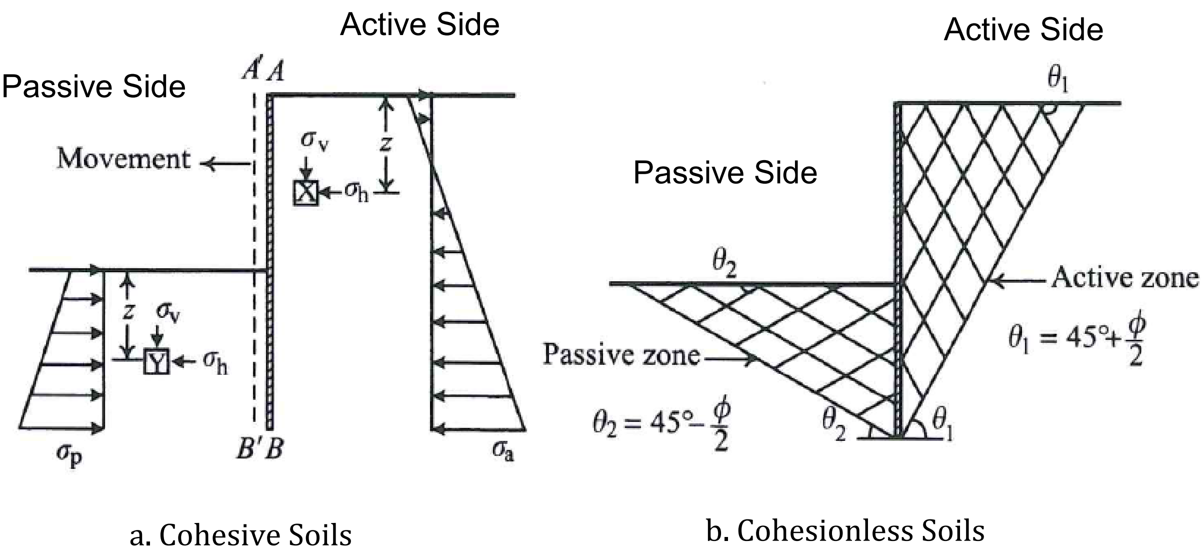


Figure 4-2 Simplified wall is showing active and passive stresses for cohesive and cohesionless soils.

4.3.2 Calculating the Active and Passive Earth Pressure Coefficients

For MDOT TERS, the following three methods are commonly employed for determining active and passive earth pressure coefficients:

1. Rankine (1857),
2. Coulomb (1776),
3. Caquot and Kérisel (1948).

The following important guidelines should be followed when using these three methods:

1. For level back slopes, all three methods give the same results, whether for coarse-grained or fine-grained soils.
2. For sloped back slopes, friction and adhesion can develop between the wall and the soil as soil deformation develops. This friction affects the direction of the active and passive stresses acting on the wall. The Coulomb and Caquot-Kérisel methods should be used to determine the active pressures acting on the wall when wall friction is to be included in the analysis.
3. To calculate the passive stresses acting on a wall with a sloped backfill, ONLY the Caquot-Kérisel method should be used. The reason is that the Coulomb method assumes the failure surface in passive failure is a plane failure when in fact it is a curved surface as shown in Figure 3-3. The curved failure surface results in much lower K_p values then are calculated using the Coulomb equations for K_p.

4. Neither Rankine nor Coulomb consider the mode of wall deformation. That is, these methods assume the wall will deform as a rigid member without redistribution of stresses.
5. Lastly, the Caquot-Kérisel method was developed to account for such realities as wall friction, sloping ground, and more complicated patterns of deformation.

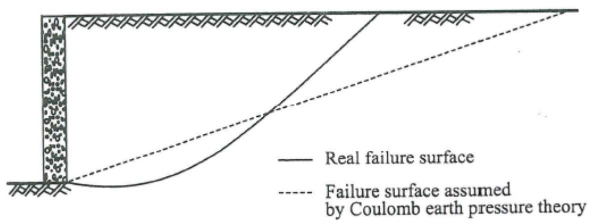


Figure 4-3 Comparison of the failure surfaces behind a wall.

4.3.3 Rankine Earth Pressure

Rankine theory is based on the concept of plastic equilibrium within the soil mass and assumes that the earth pressures increase with depth. Consequently, the method does not include wall friction (δ = 0) nor shearing stresses at the surface of contact between the wall and the soil. It also assumed that the ground and failure surfaces are a plane surface while the resultant force acts parallel to the backfill slope.

Rankine's coefficients of active and passive pressures are defined as follows:

$$K_a = \cos\beta \; [\frac{\cos\beta - [\cos^2\beta - \cos^2\phi]^{\frac{1}{2}}}{\cos\beta + [\cos^2\beta - \cos^2\phi]^{\frac{1}{2}}}]$$

$$K_p = \cos\beta \; [\frac{\cos\beta + [\cos^2\beta - \cos^2\phi]^{\frac{1}{2}}}{\cos\beta - [\cos^2\beta - \cos^2\phi]^{\frac{1}{2}}}]$$

where,

ϕ = Angle of internal friction
β = Angle of the backfill slope
K_p = Coefficient of passive earth pressure (1 to 10)
K_a = Coefficient of active earth pressure (0.17 to 1)

If the backfill is level (β = 0), the equations are simplified as:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45° - \frac{\phi}{2})$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2(45° + \frac{\phi}{2})$$

4.3.4 Coulomb Earth Pressure

Coulomb's (1776) earth pressure theory assumes the soil behind the wall acts as a rigid block as the active or passive limits are reached. Because the soil is assumed to act as a rigid block, friction can be added to the stability analysis.

According to the US Steel Sheet Pile Manual:

> *An inherent assumption of the Rankine Theory is that the presence of the wall does not affect the shearing stresses at the surface of wall contact. However, since the friction between the retaining wall and the soil has a significant effect on the vertical shear stresses in the soil, the lateral stresses on the wall are actually different than those assumed by the Rankine Theory. Most of this error can be avoided by using the Coulomb Theory, which considers the changes in tangential stress along the contact surface due to wall friction.*

The Coulomb method uses the following assumptions*:*

1. The wall is rough, so that friction develops between the wall and soil as wall movement occurs.
2. The failure wedge is a plane surface and is a function of the soil internal friction angle ϕ,
3. Lateral earth pressure varies linearly with depth.
4. The direction of the lateral earth pressure acts at an angle β □with a line normal to the wall.
5. The resultant earth pressure acts at a distance equal to one-third of the wall height from the base.

The Coulomb theory provides a method of analysis to give a resultant horizontal force on a retaining wall system for any wall slope, wall friction and slope of backfill ($\beta \leq \varphi$) as shown in Figure 4-4. The following equations are used to calculate the coefficient of active and passive earth pressure for a vertical wall.

$$K_a = \frac{\cos^2\phi}{\cos\delta\left[1 + \sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos\delta\cos\beta}}\right]^2}$$

$$K_p = \frac{\cos^2\phi}{\cos\delta\left[1 - \sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos\delta\cos\beta}}\right]^2}$$

Where

ϕ = angle of internal friction of soil

δ = angle of wall friction

β = angle of the backfill with respect to the horizontal plane.

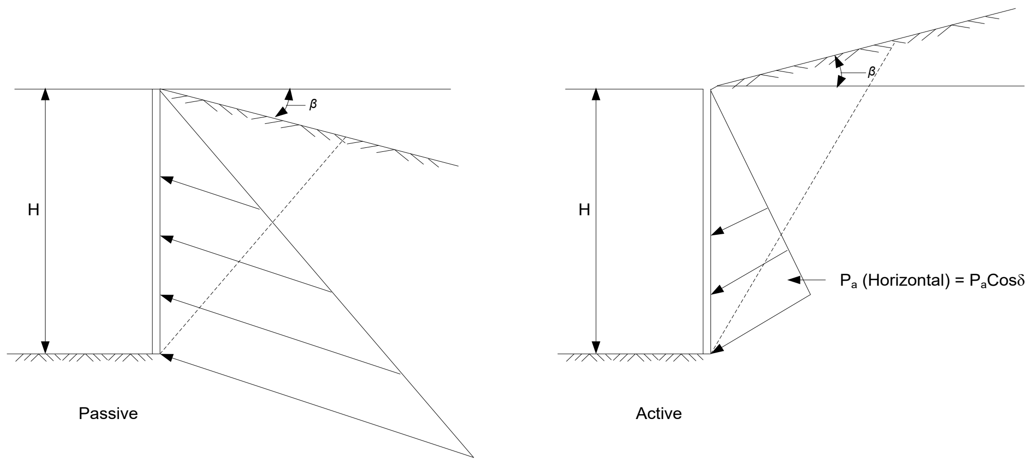


Figure 4-4 Coulomb Earth Pressure Theory.

In general, the wall friction angle, δ, falls between 0° to 22° and is always less than the soil's friction angle, ϕ. MDOT limits the amount of wall friction to 1/3 of the friction angle, ϕ. The NAVFAC manual (1986) provides a series of soil-wall interface friction angles in Table 3.1.

4.3.5 Log-Spiral Theory - Caquot and Kérisel Chart

For sloped backfills, however, the Coulomb method overestimates the passive resistance. According to the US Steel Sheet Pile Manual,

> *"The Coulomb Theory of earth pressure assumes that the surface of sliding or failure is a plane. This assumption deviates somewhat from reality. For the active case the error introduced is small. However, for the passive case, the error can be large and is always on the unsafe side. If the angle of wall friction, δ, is low, the failure surface is almost plane. However, if δ is high, the passive failure plane deviates considerably from Coulomb's assumption, which predicts unrealistically high passive pressures. Large angles of wall friction that cause a downward tangential shearing force will increase the vertical pressures in the soil close to the wall, thus causing a curved failure surface, as shown in Figure 4(a). The soil fails on this curved surface of least resistance and not on the Coulomb plane, which would require a greater lateral driving force. Figure 4(b) shows the reduction in the passive earth pressure coefficient, K_p, for increasing values of wall friction for the actual curved surface of failure."*

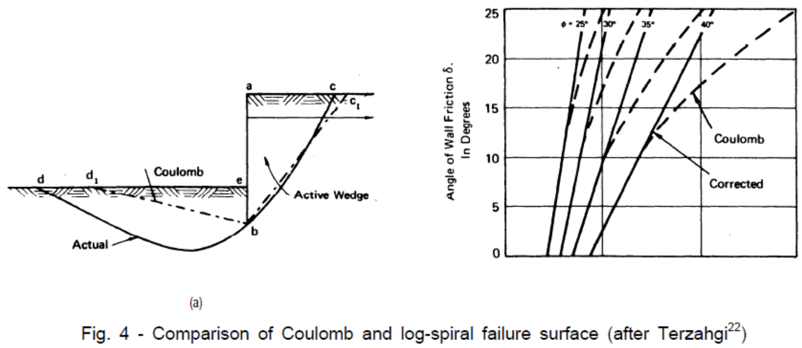


Figure 4-5 Figure 4 from Terzaghi (1954).

The Caquot and Kérisel method is used to estimate the lateral earth pressures acting on sheet piles using a log-spiral failure surface that takes into account both positive and negative back slopes and wall friction. This method is presented in a chart in the United States Steel Corporation's "Steel Sheet Piling Design Manual" (USS 1984), the Navy Facilities Engineering Command "Foundations and Earth Structures" manual (NAFVAC 1986) as well as several other manuals.

The SupportIT software provides two methods for estimating the earth pressures for sloped backfills, the "modify K values" and the "BSPH (British Sheet Piling Handbook) approximation." Figure 4-6 illustrates the software tab where these two methods are selected in the SupportIT Software. The first method, "Modify K values," earth pressure values are calculated using the given angle of the backslope. The second method, the BSPH method, assumes that soil pressure changes by 5% for each 5 degrees of slope. The SupportIT Manual, however, does not provide the details of how each method determines the lateral earth pressures. Nonetheless, the "K values" method will be used in this manual for Case 2.

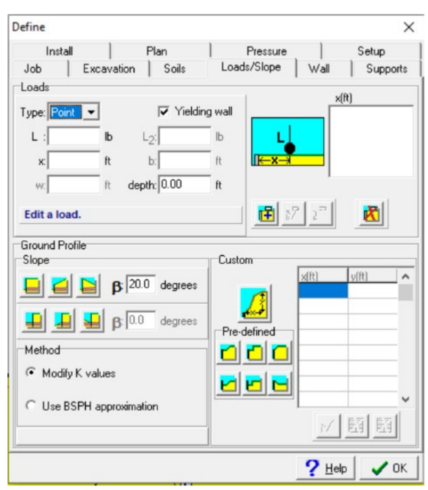


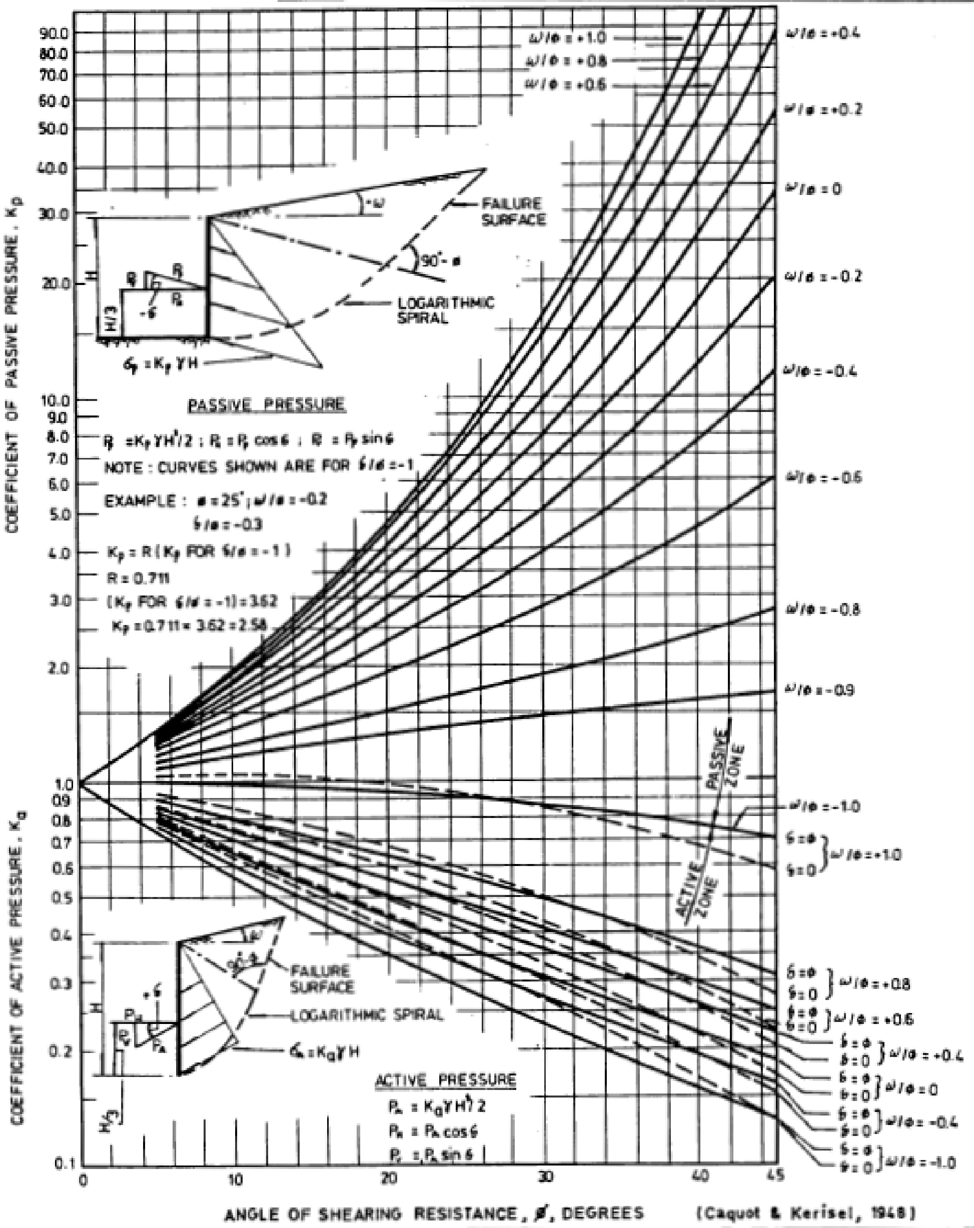
Figure 4-6 SupportIT software section for selecting the method for estimating the earth pressure coefficients for a sloped backfill.

Table 4-1 Ultimate friction angles for dissimilar materials, adapted from (NAVFAC 1986)

Interface Materials	Friction Factor, tan δ	Friction Angle, δ degrees
Mass concrete on the following foundation materials:		
Clean sound rock………………………………………	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand………...	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel…………………………...	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand………………………………………………...	0.35 to 0.45	19 to 24
Fine sandy silt, non-plastic silt…………………………	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay…………………………………………………...	0.40 to0.50	22 to 26
Medium stiff and stiff lay and silty clay…………………	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean grave, gravel-sand mixtures, well-graded rock fill with spalls…………………………………	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill………………………………………..	0.30	17
Silty sand, gravel or sand mixed with silt or clay.............	0.25	14
Fine sandy silt, non-plastic silt…………………………	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls…………………………………	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size Hard rock fill……………………………………....	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay………..	0.30	17
Fine sandy silt, non-plastic silt…………………………	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock………………..	0.70	35
Dressed hard rock on dressed soft rock…………….....	0.65	33
Dressed hard rock on dressed hard rock……...........	0.55	29
Masonry on wood (cross grain) ………………………....	0.50	26
Steel on steel at sheet pile interlock……………………...	0.30	17

REDUCTION FACTOR, R, OF Kp FOR VARIOUS RATIOS OF δ/ø

ø \ δ/ø	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



EARTH PRESSURE COEFFICENTS — SLOPING GROUND | FIGURE 4

Figure 4-7 Caquot and Kérisel chart (NAVFAC 1986)

4.3.6 Fine-grain Cohesive Soils

The earth pressure methods discussed above were originally developed for coarse-grained cohesionless soils, not cohesive soils such as clays. Over time they have been adapted for cohesive soils. For example, Bell (1952) modified Rankine's solution to include the effect of backfill with cohesion.

Clay soils can be difficult to evaluate. As discussed in the design of TERS, the undrained shear strength S_u is used. S_u, however, is *not* a fundamental property of the soil and can change with time due to changes in moisture, stress, and other factors. According to the Caltrans Trenching and Shoring Manual (2011):

> "*Extreme caution is advised when using cohesive soil to evaluate soil stresses. The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrinkage-swell, wet-dry, and degree of saturation. Tension cracks (gaps) can form, which may considerably alter the assumptions for the estimation of stress*."

Excavation in medium to stiff clays can result in vertically unsupported cuts up to some height. The height of an unsupported excavation, known as the critical height, H_c, can be estimated as follows

$$H_c = \frac{2c - q_{surcharge}}{\gamma}$$

Where $c = S_u$ is the undrained shear strength of the soil, and $q_{surcharge}$ is the surcharge load assumed for design. An illustration of the critical height of a vertical unsupported excavation in clay is shown in Figure 4-8. In vertical excavations, tension cracks can also develop behind the excavation. It is commonly assumed that tension cracks can develop to a depth of the critical height of the clay. A serious issue occurs when cracks fill with water adding hydraulic pressures to the unsupported wall or in the case of sheet piling when water fills the area between the wall and the sheet piling created by soil shrinkage.

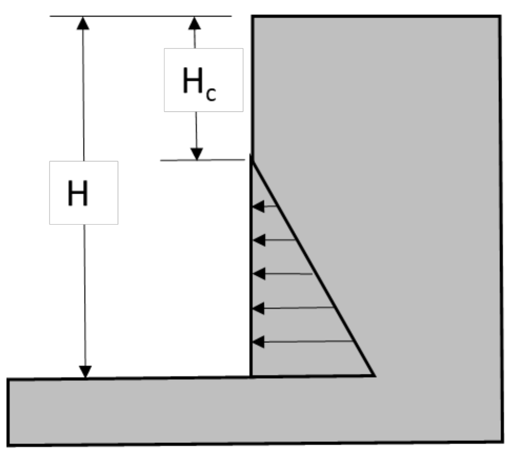


Figure 4-8 Illustration of the critical height of an excavation in clay

When calculating the active earth pressure envelope, net pressure can have a negative value for cohesive soils with high undrained shear strength, S_u. In this case, the lateral earth pressure is then assumed to be zero. To account for potential water pressure in the clay layer, however, a "minimum effective fluid pressure" is assigned to the clay layer. Software programs such as the SupportIT software use a "minimum equivalent fluid density" of about 31.4 pcf (5 kN/m^3) is used as a default pressure. The pressure developed by the "minimum equivalent fluid" is then compared to the combined active soil pressure plus the water pressure. The highest pressure is then used in the design.

To account for the uncertainty and changing conditions in cohesive soils, the following earth pressure coefficients are used $K_a = K_p = 1$. In addition, water pressure can develop behind the sheet pile wall. To account for this possibility, the Caltrans Manual (2011) suggests the active pressure is applied over the length of the wall when a potential tension crack is filled with water. A more conservative approach is to assume the water pressure also acts along the length of the sheet pile.

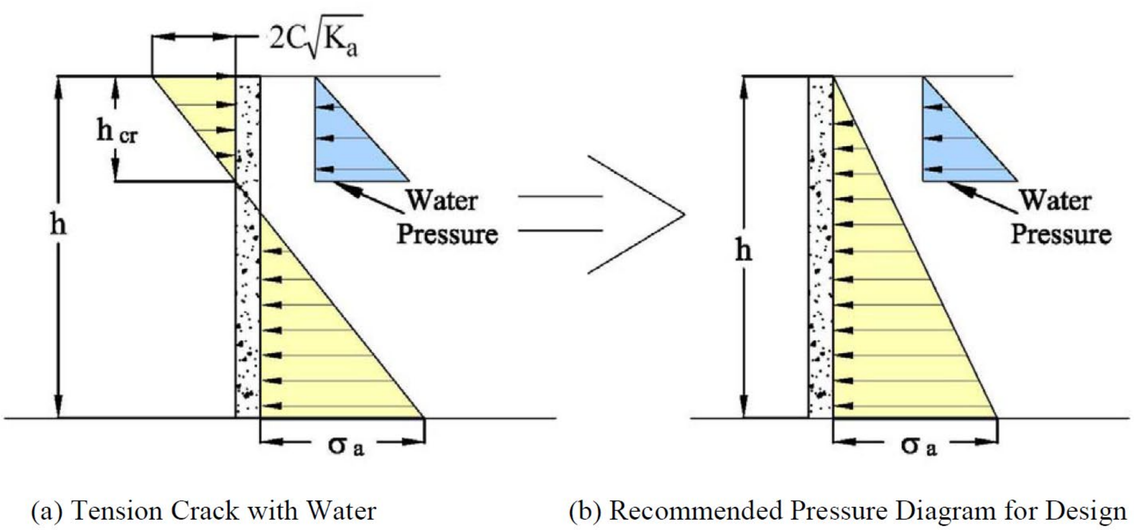


Figure 4-9 Assumptions used for tension cracks in clay soils (Caltrans 2011).

4.4 Determination of Groundwater Pressures

The assessment of groundwater pressure acting on the wall is important and must be based on actual field conditions. As noted above, the assessment of fine-grained soils assumes that the soils are in an undrained condition. In most cases, the water pressures acting on the wall are included in the saturated unit weight of the soils. Therefore, it is very important that an accurate assessment of the saturated unit weight of the soil be conducted. The assumption for coarse-grained soils generally assumes an *undrained* condition.

4.5 Surcharge Loads

Temporary retaining wall systems are generally designed to retain various surface loadings as well as the earth pressure. These surface loads are generally imposed close enough to the excavation site to generate lateral pressure on the structure. Loading cases for surcharge loads include a uniform surcharge, point loads, line loads parallel to the wall, and strip loads parallel to the wall.

4.5.1 Uniform Surcharge and Traffic Loads

When a uniformly distributed surcharge is applied at the surface, the vertical pressure at all depths increase equally, and the intensity of surcharge (q) will be added to the vertical earth pressure (γh) at the depth h. The lateral pressure due to the uniform surcharge load (σ_h) can be computed by the following equation.

$$\sigma_h = q\,K$$

Where, q is the uniform surcharge intensity (force/area), and K is either the active coefficient (K_a) or passive coefficient (K_p), depending on the direction of the wall movement. The uniform lateral pressure will then be added to the earth pressure on the wall. Figure 4-10 shows the lateral pressure on the wall due to the uniform loading on the surface.

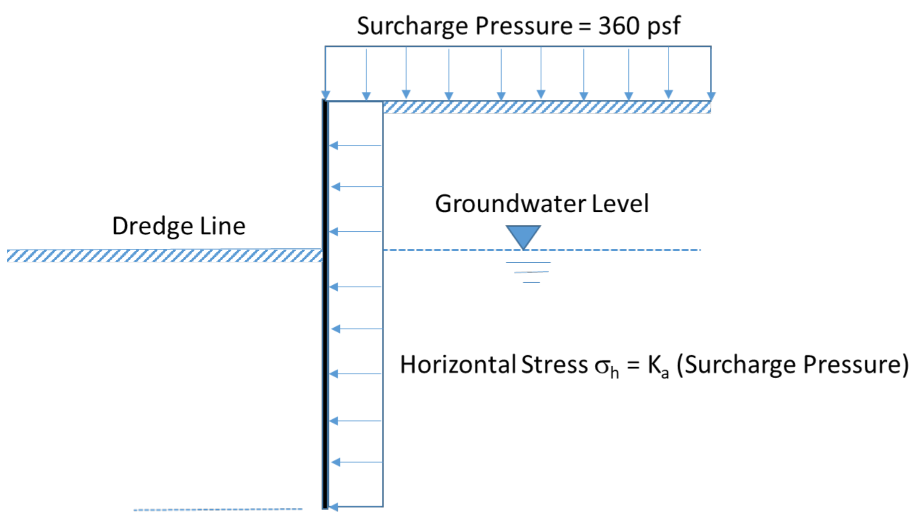
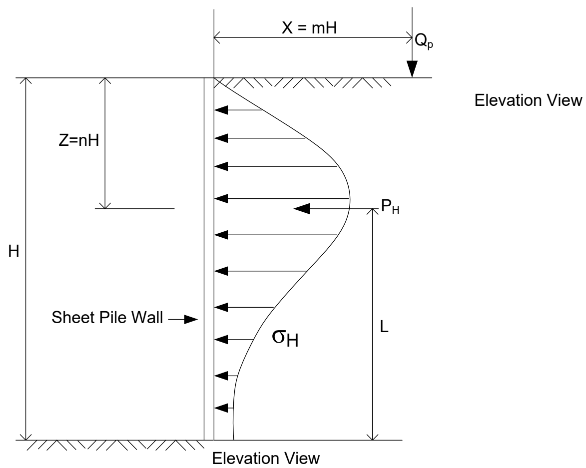


Figure 4-10 Lateral pressure due to uniform loading.

To account for traffic loads and other possible point type loadings, MDOT requires a minimum surcharge of 360 psf be used for the design of TERS (MDOT 2012).

4.5.2 Point Loads

Figure 4-11 and associated equations show the stress distribution and lateral pressure on the wall due to a point surcharge load (Q_p).



$$\sigma_H = 0.28\frac{Q_p}{H^2}\cdot\frac{n^2}{(0.16+n^2)^3}\ (\text{for } m \le 0.4)$$

$$P_H = 0.78\frac{Q_p}{H}$$

$$\sigma_H = 1.77\frac{Q_p}{H^2}\cdot\frac{m^2n^2}{(m^2+n^2)^3}\ (\text{for } m > 0.4)$$

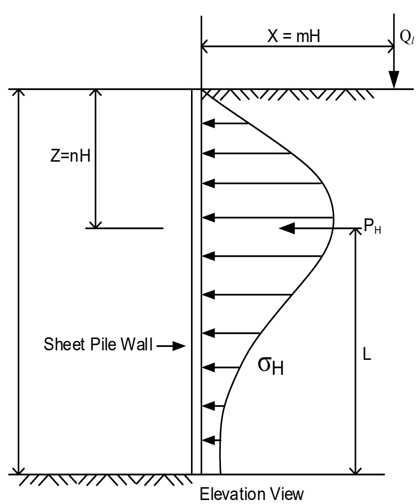
$$P_H = 0.45\frac{Q_p}{H}$$

$$n = \frac{Z}{H} \qquad m = \frac{X}{H}$$

Figure 4-11 Lateral pressure due to a point load.

4.5.3 Line Loads

A narrow width wall footing or similar parallel loads to the retaining wall may be taken as a line load. In this case, lateral pressure increases from zero at the ground level to a maximum at a certain depth and then reduces. Figure 4-12 and associated equations show the stress distribution and lateral pressure on the wall due to a line surcharge load (Q_l).

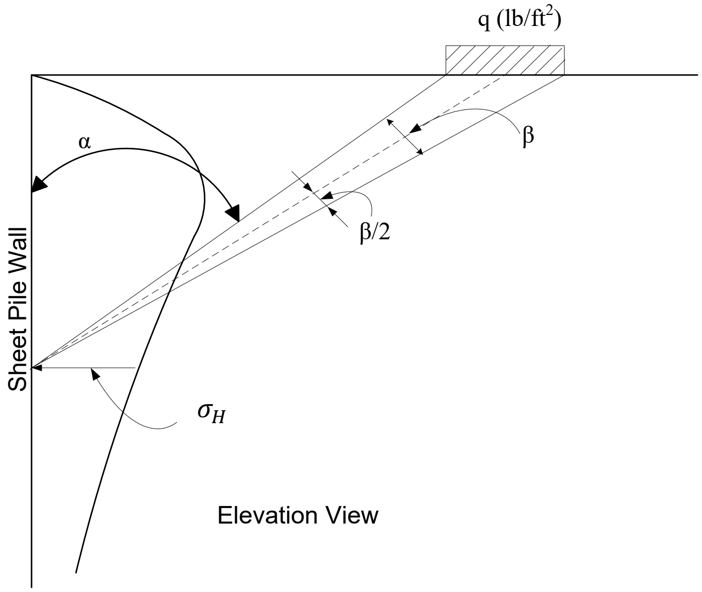


$$\sigma_H = 0.20\frac{Q_l}{H}\cdot\frac{n}{(0.16+n^2)^2}\ (\text{for } m \le 0.4)$$
$$P_H = 0.55Q_l, resultant\ force$$
$$\sigma_H = 1.28\frac{Q_l}{H}\cdot\frac{m^2n}{(m^2+n^2)^2}\ (\text{for } m > 0.4)$$
$$P_H = \frac{0.64Q_l}{(m^2+1)}\ resultant\ force$$
$$n = \frac{Z}{H} \qquad m = \frac{X}{H}$$

Figure 4-12 Lateral pressure due to line load.

4.5.4 Strip Loads

Traffic loads (highway or railroad) adjacent to the structure is an example of strip loads parallel to the earth retaining wall. Figure 4-13 and associated equation shows the stress distribution and lateral pressure on the wall due to a strip surcharge load (q).



$$\sigma_H = \frac{2q}{\pi}[\beta - sin\beta cos2\alpha]$$

Note: α and β in radians

Figure 4-13 Lateral pressure due to strip loads.

Examples of common surcharge loads on temporary earth retaining structures that must be considered in the design include soil embankment adjacent to the structure, construction loads due to material and equipment, traffic loads due to an adjacent railroad or highway, ice flows, and loads induced on the wall due to the pile driving.

4.5.5 Compaction Induced Earth Pressure

Where heavy static and dynamic compaction equipment is used within a distance of one-half the wall height behind the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account. The load factor for compact induced earth pressure shall be the same as lateral earth pressures (1.3 γ) (AASHTO 2002).

4.6 Ground Movements

A prediction of ground movements outside of the excavation cannot be conducted using Limit Equilibrium methods. All TERS will cause ground movements, and it is important they be limited to an acceptable level. The prediction of accurate ground movement is not possible but can be estimated from either empirical method based on field measurements or from analytical methods based on numerical methods. It is assumed in this manual that ground movements associated with the case examples below will be within acceptable limits when the TERS is properly constructed.

Gaba et al. (2003) provide the following construction controls to minimize ground movements associated with excavation and installation of a TERS:

- Good workmanship is essential
- Supports should be installed tight to the wall. The bracing and any packing between the bracing and the waling should not rely on friction or adhesion between the brace end and the waling to hold it in place
- The wall should have adequate embedment in stiff strata for vertical and lateral stability
- Minimize the first-stage of excavation and install the first anchor and brace as early as possible in the construction sequence
- Minimize the amount of excavation beyond the first proposed support levels
- Minimize delays to the wall's construction and support system
- Avoid over-excavation
- If a clay berm is being used for lateral support, the berm should be covered or protected from changes in moisture content and possible weakening due to an increase in moisture content
- Minimize the removal of fines during dewatering
- Minimize groundwater drawdown outside the excavation.

5 Temporary Earth Retaining Systems (TERS) Design Examples

5.1 General

Today, computer software programs are commonly used for the design of temporary earth retaining systems (TERS), allowing sheet piling designs to be accomplished in a minimal amount of time, while also allowing designers to optimize the design as well as investigate more complex designs. The software, however, should be used with caution along with some level of skepticism (Gaba et al. 2003). The software programs require a basic understanding of the assumptions used in the software. To assist designers in understanding some of the software design methods, nine hand-calculated design examples are provided in Chapter 5. The software program SupportIT version 2.37 is used to compare the hand-calculated designs with the SupportIT software output. The SupportIT design examples are provided in Appendix B.

Every effort has been made to match the hand calculations to the computer solutions. Hand-calculations, however, are only approximate solutions, whereas computer software can utilize higher level approximation methods in producing its results. Therefore, it was not possible to precisely match the computer solutions. Further, the design examples in this chapter are provided as examples and are not meant for design purposes.

5.2 Design Examples

The purpose of this section is to present the following eight sheet pile design problems to highlight the basic calculations used in the design of a sheet pile wall.

Case 1 – Cantilever TERS in Cohesionless Soil with Level Backfill

Case 2 – Cantilever TERS in Cohesionless Soil with Sloped Backfill

Case 3 – Cantilever TERS in Stiff Cohesive with Level Back Slope

Case 4 – Anchored Cantilever TERS in Cohesionless Soil

Case 5 – Anchored Cantilever TERS in Stiff Cohesive Soil

Case 6 – Braced Cofferdam TERS in Soft and Stiff Cohesive Soil

Case 7 – Braced Cofferdam TERS in Cohesionless Soil

Case 8 – Braced Cofferdam TERS in Soft Cohesive Soil

Case 9 – Cantilevered Soldier Pile TERS in Coarse-grained Soil

5.2.1 Case 1 – Cantilever TERS in Cohesionless Soil with Level Backfill

As discussed previously, cantilever walls rely entirely on their depth of penetration for their stability; there is no additional support provided. Further, cantilever walls are not recommended in projects constructed in soft clay soils. The calculation of the pile's penetration depth is, therefore critical in cantilever walls since the depth of penetration needs to be designed deep enough to prevent translation or rotation of the toe. The calculations in Case 1 use a fixed-earth method to estimate the bending moments in the sheet piling and the final pile embedment depth, D_f. The calculations assume a one-foot wide sheet piling.

Case 1, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall

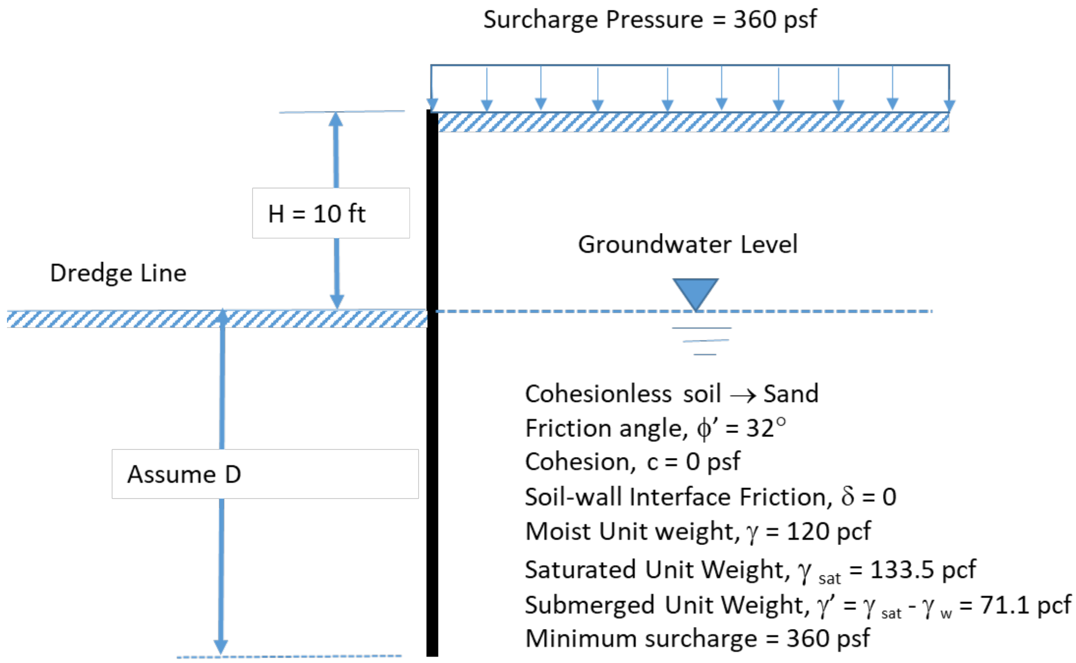


Figure 5-1 Case 1 Cantilever wall in cohesionless soil with level backfill.

Case 1, Step 2: Calculate Active and Passive Earth Pressures and Forces Acting on the Wall

Case 1 assumes a level backfill with no wall-soil interface friction, δ, i.e., δ = 0. Using the Rankine Method, the coefficients of active and passive pressures are calculated as follows:

$$K_a = tan^2\left(45^\circ - \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ - \frac{32^\circ}{2}\right) = \mathbf{0.31}$$

$$K_p = tan^2\left(45^\circ + \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ + \frac{32^\circ}{2}\right) = \mathbf{3.25}$$

Vertical and horizontal (lateral) pressures are calculated as follows:

Vertical and horizontal soil stress <u>above</u> the groundwater table:

Vertical Soil pressure: $\sigma_v = z\gamma_{soil}$
Active Lateral Pressure: $\sigma_h = \sigma_v K_a$
Passive Lateral Pressure: $\sigma_h = \sigma_v K_p$
Water Pressure: $\sigma_w = z\gamma_w$

Vertical and horizontal soil stress <u>below</u> the groundwater table:

Effective Vertical Soil Pressure: $\sigma'_v = z\gamma'_{soil}$
Active Lateral Pressure: $\sigma'_h = \sigma'_v K_a$
Passive Lateral Pressure: $\sigma'_h = \sigma'_v K_p$

where z = the vertical depth of the soil
γ_{soil} = unit weight of the soil above the groundwater table
γ_w = unit weight of water
γ'_{soil} = effective unit weight of the soil above the groundwater table = γ'_{sat} - γ_w
γ'_{sat} = the saturated unit weight of the soil below the groundwater table.

Surcharge Pressure: $\sigma_{a1} = K_a\ \sigma_v = (0.31)(360) = \underline{111.6\text{ psf}}$

(Note that the surcharge pressure is assumed to act along the entire length of the sheet pile.)

<u>Active pressure</u> to the groundwater table: $\sigma_{a2'} = K_a\gamma H = (0.31)(120)(10) = \underline{372.0\text{ psf}}$
$\sigma_{a2} = \sigma_{a1} + \sigma_{a2'} = 111.6 + 372.0 = \underline{483.6\text{ psf}}$

<u>Water pressure</u> at a depth of 16 feet below the groundwater table: $u_{a,w} = z\gamma_w$

<u>Case 1, Step 3: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0</u>

The following calculations are used to determine the sheet pile depth, D_u, for Case 1. The forces and the location of the forces used in the moment calculations are shown in Figure 5-2. The water pressure diagram is not shown but the resulting water forces are.

<u>Calculation of soil pressures:</u>

<u>Active Stress Distributions:</u>
Pressure 1: Surcharge – rectangular pressure distribution along the total length of the wall
Pressure 2: Sand above dredge line – triangular pressure distribution
Pressure 3: Sand below the dredge line – rectangular distribution
Pressure 4: Sand below the dredge line – triangular distribution
Pressure 5: Water pressure below the groundwater table – triangular distribution

Passive Stress Distributions:
Pressure 1: Sand below the dredge line – triangular distribution
Pressure 2: Water pressure below the groundwater table – triangular distribution

Forces and Force Locations in Terms of D_o (summing moments about O)

Active Forces, lbs, and Force Locations, ft

Active Force, P_{a1}:	$P_{a1} = [0.31(360)(10 + D_o)](1\ ft) = 1{,}116 + 111.6D_o$
Active Force P_{a2}:	$P_{a2} = 0.5[(0.31)(120)(10)](10)(1\ ft) = 1{,}860$
Active Force P_{a3}:	$P_{a3} = [0.31(120)(10](D_o)(1\ ft) = 372D_o$
Active Force P_{a4}:	$P_{a4} = 0.5[(0.31)(71.1)(D_o)]D_0(1\ ft) = 11.02D_o^2$
Active Water P_{aw}:	$P_{aw} = 0.5(62.4D_o^2)(1\ ft) = 31.2\ D_o^2$
Active Force Location, L_{a1}	$L_{a1} = (10+D_o)(0.5) = 5 + 0.5D_o$
Active Force Location, L_{a2}	$L_{a2} = 0.33(10) + D_0 = 3.33 + D_o$
Active Force Location, L_{a3}	$L_{a3} = 0.5D_0$
Active Force Location, L_{a4}	$L_{a4} = 0.33D_o$
Active Force Location, L_{aw}	$L_{aw} = 0.33D_o$

Passive Force, lbs, and Location, ft

Passive Force P_{p1}:	$P_{p1} = 0.5(3.25)(71.1)(D_o)D_0(1\ ft) = 115.54D_o^2$
Passive Water P_{pw}:	$P_{pw} = 0.5(62.4D_o^2) = 31.2\ D_o^2$
Passive Force Location, L_{p1}	$L_{p1} = 0.33D_o$
Passive Force Location, L_{aw}	$L_{pw} = 0.33D_o$

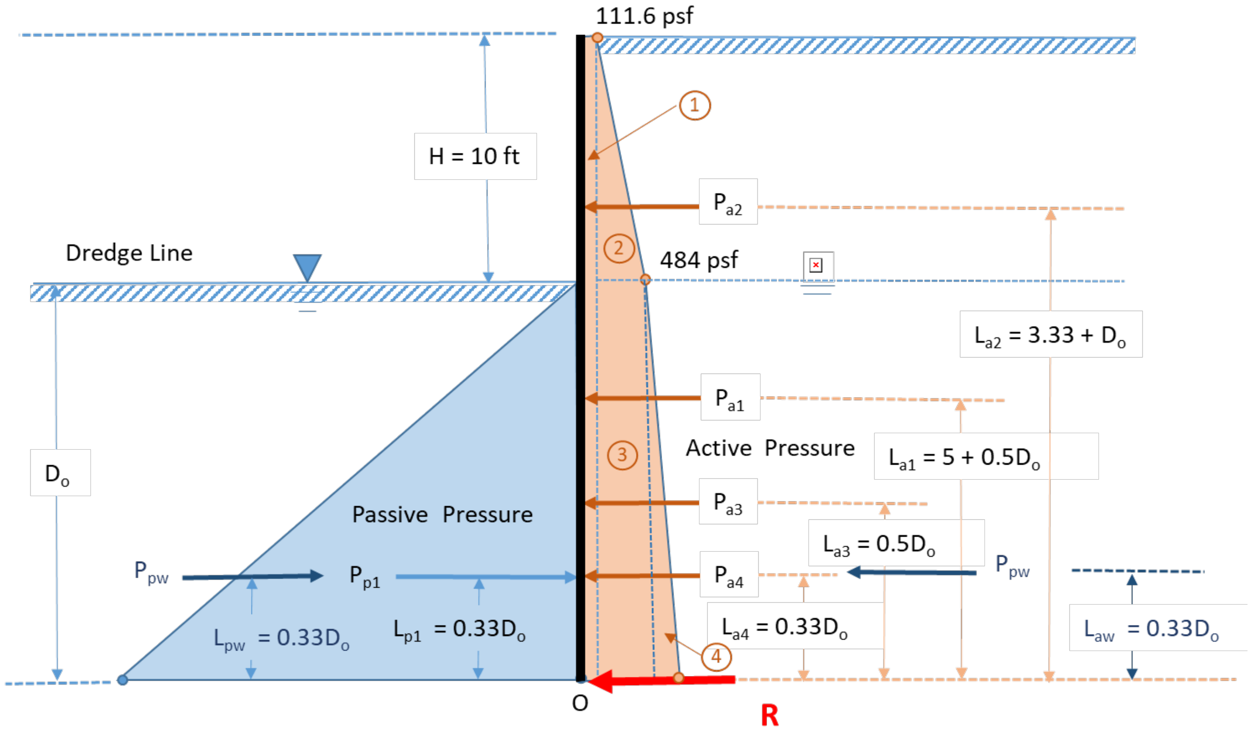


Figure 5-2 Case 1 Gross pressures, forces, and force locations acting on the sheet pile wall.

To determine the pivot point embedment depth, D_o, the summation of moments about the pivot point, O, is conducted with a factor of safety, *FOS*, equal to one, as shown below.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS = 1.0$$

where

$$\Sigma M_{disturbing} = P_{a1}L_{a1} + P_{a2}L_{2a} + P_{a3}L_{a3} + P_{a4}L_{a4} + P_{aw}L_{aw}$$

$$\Sigma M_{restoring} = P_{p1}L_{p1} + P_{pw}L_{pw}$$

$$\Sigma M_{disturbing} = [(1{,}116 + 111.6D_o)(5 + 0.5D_o)] + [(1{,}860)(3.33 + D_o)] + [(372D_o)(0.5D_0)] + [(11.0D_o^2)(0.33D_o)] + [(31.2\ D_o^2)(0.33D_o)]$$

$$\Sigma M_{restoring} = [(P_{p1})(L_{p1})] = [(115.5D_o^2)(0.33D_o)] + [(31.2\ D_o^2)(0.33D_o)]$$

D_o is calculated by substituting depth values in for D_o until a *FOS* is equal to one. This operation can easily be complished using an EXCEL spreadsheet. The embedment depth, D_o, was determined to be 14.55 feet, as shown in a portion of an EXCEL sheet provided in Table 5-1.

Table 5-1 Case 1 Embedment depth, D_o, for *FOS* = 1.0

Case 1: Calculation of Sheet Pile Depth, D_o			
Depth of Sheet Pile, D_o (ft)		14.55	
	FOS	1.00	
Restoring Moment:		Disturbing Moment:	
P_{p1}	24,466.0	P_{a1}	2,740.0
L_{p1}	4.8	L_{a1}	12.3
P_{pw}	6,606.7	P_{a2}	1,860.0
L_{pw}	4.8	L_{a2}	17.9
M_r	149,214	P_{a3}	5,413.3
		L_{a3}	7.3
		P_{a4}	2,333.5
		L_{a4}	4.8
		P_{aw}	6,606.7
		L_{aw}	4.8
		M_d	149,214

To account for the stresses below the pivot point, the *simplified method* multiplies the D_o by 1.2 as follows:

D_o = 14.55 ft

$D_u = 1.2D_o = 1.2(14.55) =$ **17.46 ft**

$D_{SupportITu}$ = **17.30 ft**

Notes:

1. In using the *simplified method* it is recommended that a check be conducted to make sure the horizontal force below the pivot point is greater than the resultant "R" shown in Figure 5-2. This step, however, is generally not conducted because it has been found that the additional length generally always concludes with acceptable results.
2. The embedment depth D_u is unfactored; that is, there is no safety factor applied to the embedment depth. This embedment depth is used to calculate the bending moments on the pile. Once this is accomplished, a factor of safety (FOS) is applied to calculate the final embedment depth, D_f.

Case 1, Step 4: Calculate Maximum Bending Moment in Sheet Pile Wall

The maximum bending stress applied to the sheet pile is calculated with the embedment depth, D_u with a FOS = 1.0. The maximum bending moment occurs at the point of zero shear (horizontal forces) in the sheet piling. For this example, the zero-shear location on the sheet piling is calculated using the "*gross pressure*" diagram shown in Figure 5-3.

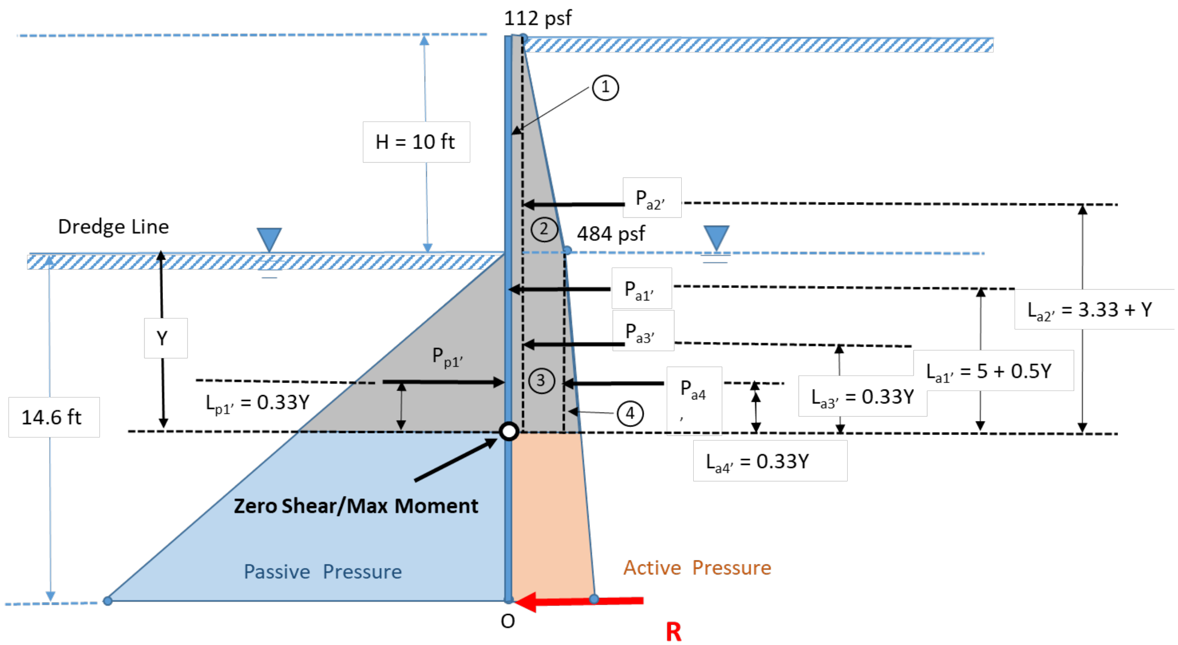


Figure 5-3 Case 1 Gross pressure diagram to locate the maximum sheet pile moment.

Note: To simplify the calculations, the water forces are not included in the following calculations. If there is a water imbalance, however, the water forces must be included.

The steps below are used to determine the maximum moment acting on the sheet pile:

Step 4A: Assume a depth "Y" where the active forces equal the passive forces (zero shear), and the maximum bending stress occurs. The maximum moment will be located between the dredge line and the pivot point "O" at a depth "Y."

Determine forces acting on sheet piling above the point of zero shear in terms of the depth "Y."

Active Forces:

$P_{a1'} = 111.6(10 + Y) = 1{,}116 + 111.6Y$

$P_{a2'} = 0.5(372)(10) = 1{,}860$

$P_{a3'} = 0.31(120)(10)(Y) = 372Y$

$P_{a4'} = 0.5(0.31)(71.1)(Y)(Y) = 11.0Y^2$

Passive Force:

$P_{p1'} = 0.5(3.25)(71.1)Y^2 = 115.5Y^2$

Solve for the depth "Y" by $\Sigma P_{p'} = \Sigma P_{a'}$ or

$P_{p1'} = P_{a1'} + P_{a2'} + P_{a3'} + P_{a4'}$
$115.5Y^2 = (1{,}116 + 111.6Y) + (1{,}860) + (372Y) + (11.02Y^2)$
$104.48Y^2 - 483.6Y - 2{,}976.0 = 0$

Solution for Y:
Y = **8.13 ft**
$Y_{SuppportIT}$ **= 8.13 ft** (maximum moment is located at a distance H+Y, or **18.13 ft)**

Step 4B: Calculate the maximum bending moment at depth "Y":

Active Force Moment Arms, Y, above the Zero Shear/Max Moment Point:

$L_{a1'} = 0.5(10 + Y) = 5 + 0.5Y$
$L_{a2'} = 0.333(10) + Y = 3.33 + Y$
$L_{a3'} = 0.5(Y) = 0.5Y$
$L_{a4'} = 0.333(Y) = 0.333Y$

Passive Force Moment Arms, Y, above the Zero Shear/Max Moment Point:
$L_{p1'} = 0.333(Y) = 0.333Y$
Max Bending Moment = $[\Sigma P_{a'} - \Sigma P_{p'}]$

Max Bending Moment = $[(P_{a1'}\, L_{a1'}) + (P_{a2'}\, L_{a2'}) + (P_{a3'}\, L_{a3'}) + (P_{a4'}\, L_{a4'})] - [P_{p1'} L_{p1'}]$

Maximum Moment - Hand = 33,148 ft-lbs/ft

Maximum Moment – SupportIT = 33,109 ft-lbs/ft

Case 1, Step 5: Sheet Pile Selection

The maximum moment at point O is equal to 33,148 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 50 ksi, a required section modulus, *Z*, is determined as follows:

Required section modulus, $Z = M/f_s$ = [33,148 ft-lbs/ft x 12 in/ft] / 50,000 psi = 8.0 in^3/ft

The section modulus of US Steel's PZ22 is 18.1 in^3/ft; therefore, a PZ22 sheet pile wall can meet the section modulus requirement.

MDOT also limits the maximum deflection at the top of the sheet pile wall to 2.0-in. A deflection analysis is conducted by solving the second order differential equation

$$\frac{d^2y}{d^2x} = \frac{M}{EI}$$

where M = maximum bending moment
y = pile deflection
x = location along the sheet pile
E = Young's modulus for the sheet pile
I = Moment of inertia for the sheet pile

The calculation of deflection is beyond the scope of this case study. According to the SupportIT solution provided in Appendix B.1, while a PZ22 meets the maximum moment, it does not meet the maximum two-inch deflection limit. Therefore, a PZ27 pile is required to achieve the two-inch deflection limit. The estimated deflection of a PZ27 sheet pile is 1.9 inches.

Case 1, Step 6: Add a Factor of Safety (FOS) to Increase the Length of the Sheet Pile

US Steel Pile Manual: Increase the embedment length, D_u, by 20 to 40%

USS Sheet Piling Manual: The unfactored depth, D_u, is increased by 20 to 40%, as shown in Figure 5-4 to determine the final embedment depth, D_f.

20% Increase: $D_f = 1.20D_u = 1.2(17.46) =$ 20.95 ft

Total Sheet Pile Length = H + D_f = 10 + 20.9 = 30.95 ft say 31 feet

40% Increase: $D_f = 1.40D_u = 1.4(17.46) =$ 24.44 ft

Total Sheet Pile Length = H + D_f = 10 + 24.4 = 34.4 ft say 35 feet

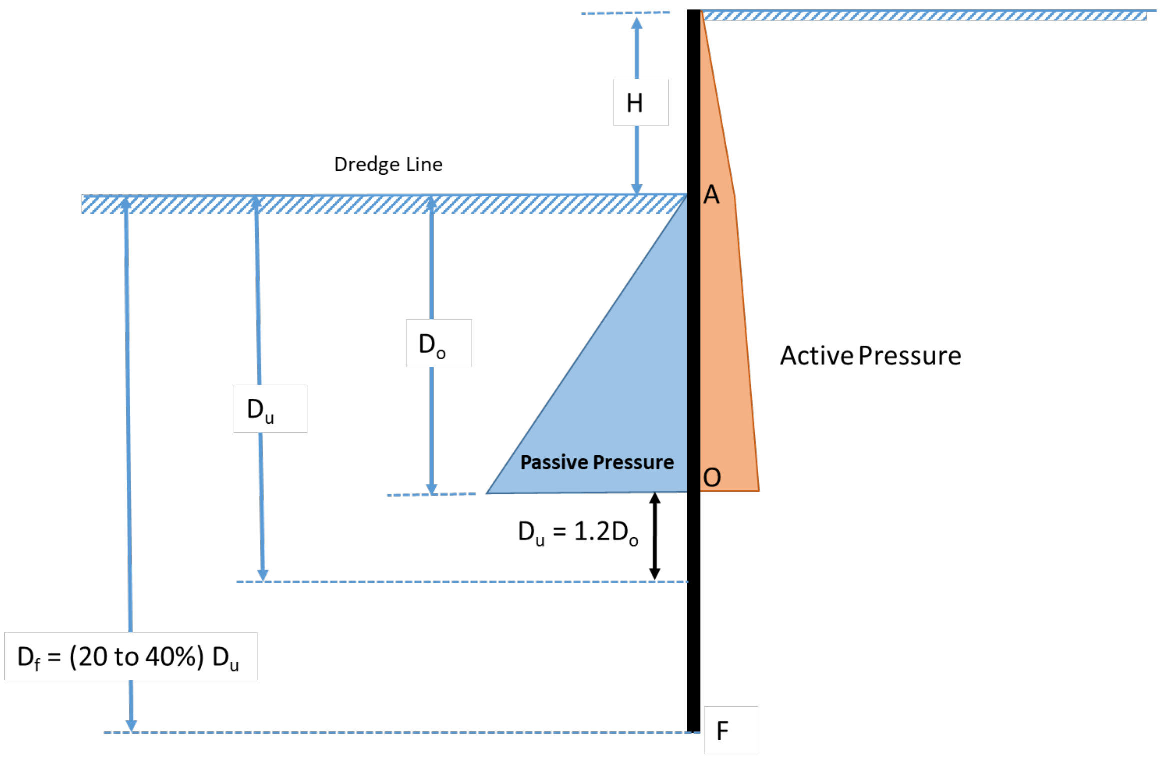


Figure 5-4 Case 1 US Steel Sheet Pile Manual 20 to 40% embedment increase method.

Gross Pressure, CP2

The traditional CP2 method to calculate the final embedment depth, D_f, reduces the passive resistance by a factor of safety (FOS) as follows:

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS = 1.50$$

The same EXCEL spreadsheet used in Step 3 is used to calculate the embedment depth with a factor of safety by increasing the embedment depth until the FOS = 1.5. This is equivalent to dividing the passive earth pressure by a factor of safety and then reformulating the equations in Step 3. As in Case 1 a factor of safety = 1.50 is used. The resulting EXCEL sheet calculation is provided in Table 5-2.

Table 5-2 Case 1 Gross Pressure CP2 (Method 2) determination of embedment depth, D_f, for FOS = 1.5

Case 1: Calculation of Sheet Pile Depth, D_o			
Depth of Sheet Pile, D_o (ft)		21.91	
	FOS	1.50	
Restoring Moment:		Disturbing Moment:	
P_{p1}	55,483.96	P_{a1}	3,561.6
L_{p1}	7.23	L_{a1}	16.0
P_{pw}	14,982.69	P_{a2}	1,860.0
L_{pw}	7.23	L_{a2}	25.2
M_r	509,583	P_{a3}	8,151.9
		L_{a3}	11.0
		P_{a4}	5,292.0
		L_{a4}	7.2
		P_{aw}	14,982.7
		L_{aw}	7.2
		M_d	339,722

To compensate for using the simplified method, the depth D_o must be increased by 20% to obtain D_f, the final embedment depth

<u>For FOS = 1.50:</u>

$D_f = 1.2(D_o) = 1.2(21.91) = \underline{26.29\text{ ft}}$

$D_f = \underline{26.29\text{ ft}}$

$D_{SupportIT,f} = \underline{26.16\text{ ft}}$

Comparison to the SupportIT software results (Appendix B.1):

Table 5-3 Case 1 Comparison of hand calculations to SupportIT calculations.

	SupportIT (Total pile length, ft)	Hand Calculations (Total pile length, ft)
Maximum soil pressure at dredge line, (psf/ft)	483.3	483.6
Maximum Bending Moment Location, (ft), FOS = 1.0	18.14	18.13
Maximum Bending Moment, (ft-lbs/ft), FOS =1.0	33,109	33,148
Sheet Pile Embedment Length, D_o (ft) FOS = 1.0	17.30	17.46
Sheet Pile Embedment Length, D_u (ft) FOS = 1.0	20.8	20.9
USS 20% FOS Embedment Length, D_f (ft)	25.0 (35)	25.1 (35)
USS 40% FOS Embedment Length, D_f (ft)	29.1 (39)	29.3 (39)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	26.16 (36)	26.29 (36)

Note that for Case 1, the final embedment depth, D_f, was calculated assuming no friction between the soil and the sheet pile, i.e., $\delta = 0°$. Wall friction has a significant effect on the computed embedment depth. For example, by adding 10° wall friction at a FOS = 1.5, the embedment depth is reduced from 26 feet to 19 feet. A SupportIT output for Case 1 with 10° friction is included in Appendix B1.

5.2.2 Case 2 – Cantilever TERS in Cohesionless Soil with Sloped Backfill

Case 2 assumes the same soil parameters as in Case 1 but with a positive back slope of ß = 20°, as illustrated in Figure 5-5. No surcharge loading, however, is assumed to act on the back slope. An essential assumption in the analysis of sloped backfills is that the active forces act parallel to the slope. The calculations in Case 2 uses a fixed-earth method to estimate the bending moments in the sheet piling and the final pile embedment depth, D_f. The calculations assume a one-foot wide sheet piling.

<u>Case 2, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall</u>

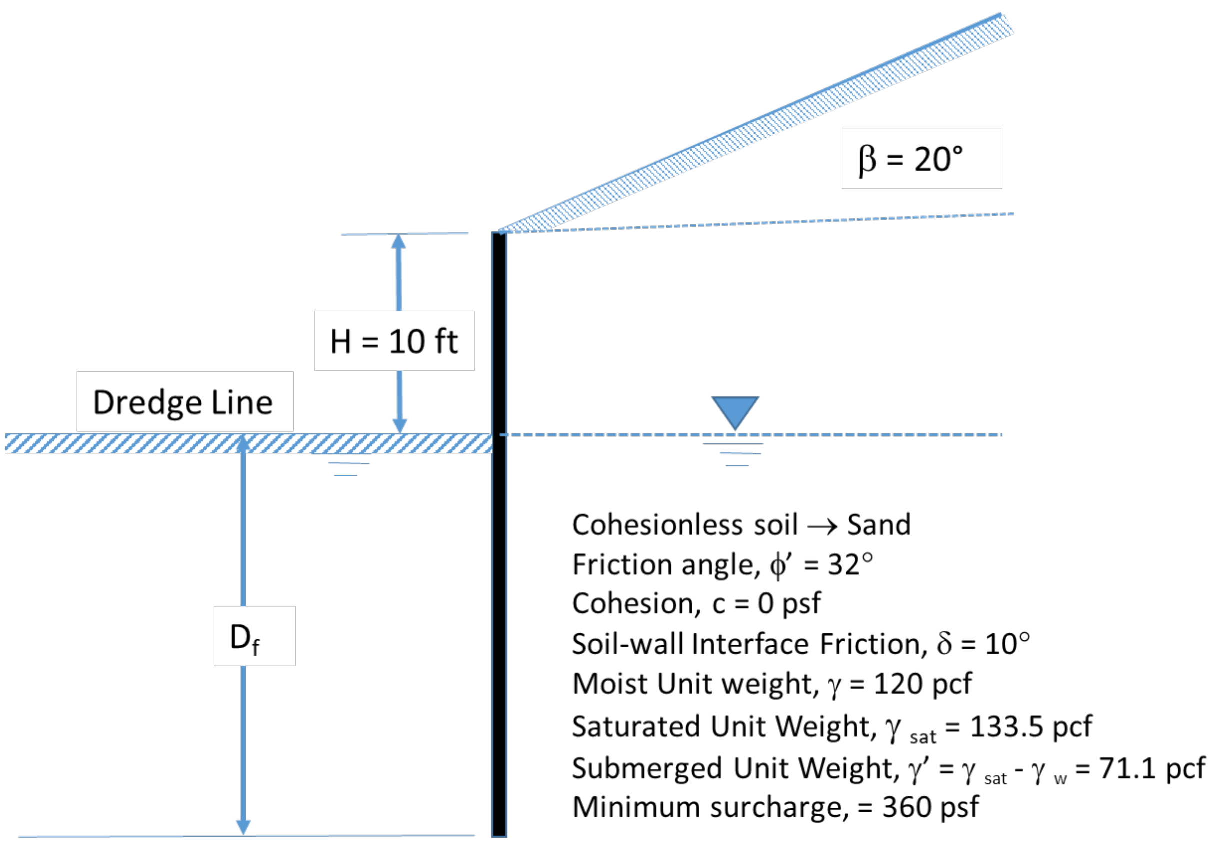


Figure 5-5 Case 2 Cantilevered TERS in cohesionless soil with sloped backfill.

Case 2, Step 2: Determination of Active and Passive Earth Pressures and Forces

MDOT limits the amount of soil/wall friction used in the analysis of the sheet pile embedment to one-third of the friction angle ϕor $\delta = 0.33\phi$.

MDOT Sheet pile wall friction limit: $\delta = \phi'/3 = 32°/3 = 10.7°$, use $\delta = 10°$.

$$\phi' = 32°$$
$$\beta = 20°$$
$$\delta = 10°$$

The gross active and passive pressures acting on the sheet pile wall have the same distribution as in Case 1 but with adjusted earth pressures to account for the sloped backfill. The gross soil pressures acting on the sheet pile wall with an assumed embedment depth of 16 feet are shown in Figure 5-6. It should be noted that the 16 ft depth is arbitrary and is only used to indicate the calculation of active and passive pressures.

The US Steel Sheet Pile Manual, NACFAC manual, and other manual provide the Caquot-Kerisel chart (Figure 4-7) to modify the earth pressure coefficients. As discussed in Chapter 4, the SupportIT software offers two methods to modify the earth pressure coefficients to account for a slopes backfile. Case 2 will use the "K values" modification values, which are provided below.

Coulomb	SupportIT Adjusted K Values
$K_a = 0.38$	$K_a = 0.38$
$K_p = 12.15$	$K_p = 4.57$

Active Pressures:

Lateral Pressure above Groundwater: $\sigma_{a1'} = K_a\gamma H = 0.38(120)(10) = \underline{456.0\text{ psf}}$

Lateral Pressure below Groundwater: $\sigma'_{a2} = 456.0 + K_a\gamma' D = 456.0 + (0.38)(71.1)(16) = \underline{888.3\text{ psf}}$

Water Pressure: $u = z\gamma_w = 62.4(16) = \underline{998.4\text{ psf}}$

Passive Pressures:

Lateral Pressure below Groundwater: $\sigma'_{p1} = K_p\gamma H = 4.57(71.1)(16) = \underline{5,198.8\text{ psf}}$

Water Pressure: $u = z\gamma_w = 62.4(16) = \underline{998.4\text{ psf}}$

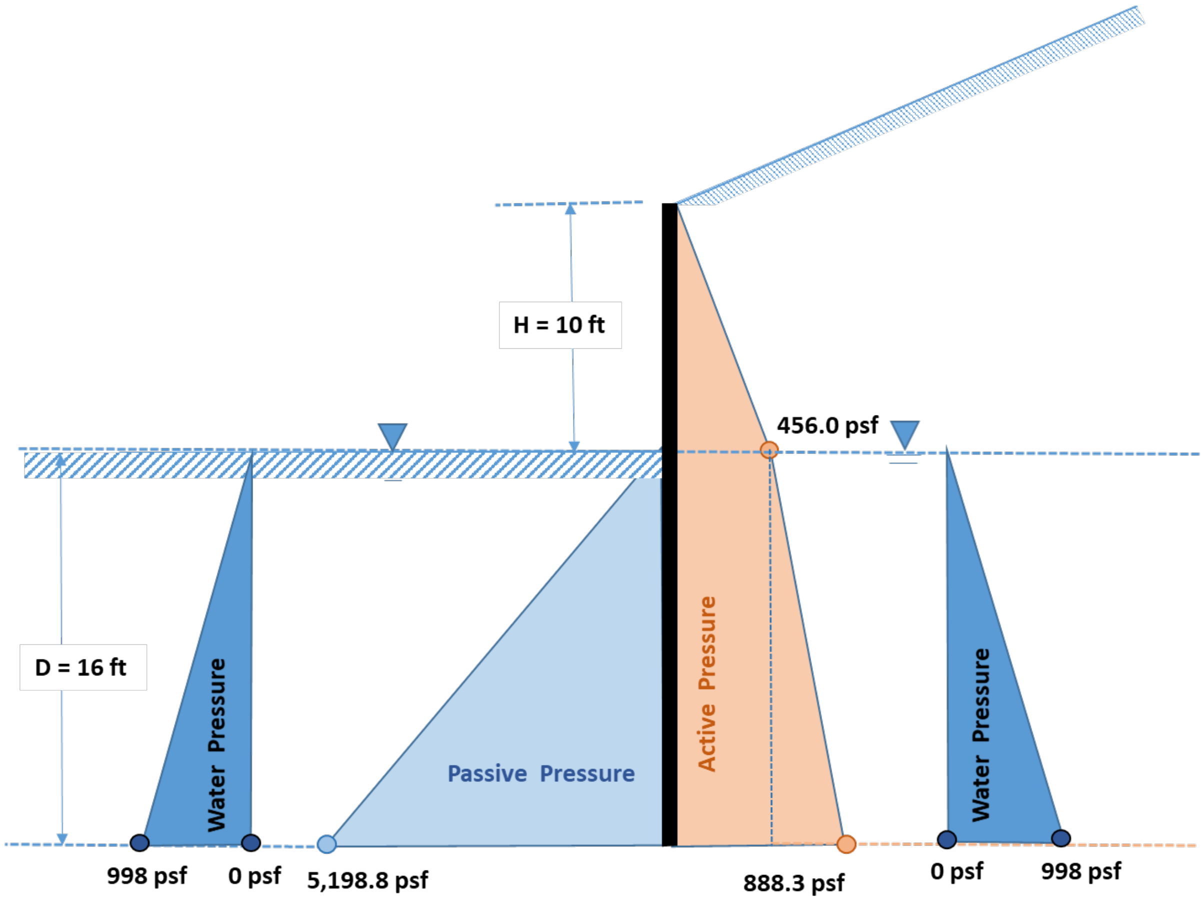


Figure 5-6 Case 2 Gross pressure distribution for a sloped backfill.

<u>Case 2, Step 3: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0</u>

Using the simplified method of analysis, equations for the forces acting on the sheet piling and their locations are developed based on a depth D_o below the dredge line to point "O," the pivot point where the wall rotates, and the active and passive stresses reverse. The forces are determined by multiplying the soil pressures distribution times a one-foot width of the sheet piling. The calculations are provided below, while the location of the forces are shown in Figure 5-7.

Forces and Force Locations in Terms of D_o

Active Forces, lbs

Active Force P_{a1}:	$P_{a1} = 0.5(0.38)(120)(10)(10)(1\text{ ft}) = 2{,}280.0$
Active Force P_{a2}:	$P_{a2} = 0.38(120)(10)(D_o)(1\text{ ft}) = 456.0D_o$
Active Force P_{a3}:	$P_{a3} = 0.5(0.38)(71.1)(D_o)(D_o)(1\text{ ft}) = 13.51D_o^2$
Water Force P_{aw}:	$P_{aw} = 0.5(62.4)(D_o)^2(1\text{ ft}) = 31.2\ D_o^2$
Active Force Location, L_{a1}	$L_{a1} = 0.33(10) + D_0 = 3.33 + D_o$
Active Force Location, L_{a2}	$L_{a2} = 0.5D_o$
Active Force Location, L_{a3}	$L_{a3} = 0.33D_o$
Water Force Location, L_{aw}	$L_{aw} = 0.33D_o$

Passive Force, lbs

Passive Force P_{p1}:	$P_{p1} = 0.5(4.57)(71.1)(D_o)(D_o)(1\text{ ft}) = 162.46D_o^2$
Water Force P_{aw}:	$P_{pw} = 0.5(62.4)(D_o)^2(1\text{ ft}) = 31.2\ D_o^2$
Passive Force Location, L_{p1}	$L_{p1} = 0.33(D_o) = 0.33D_o$
Water Force Location, L_{aw}	$L_{pw} = 0.33D_o$

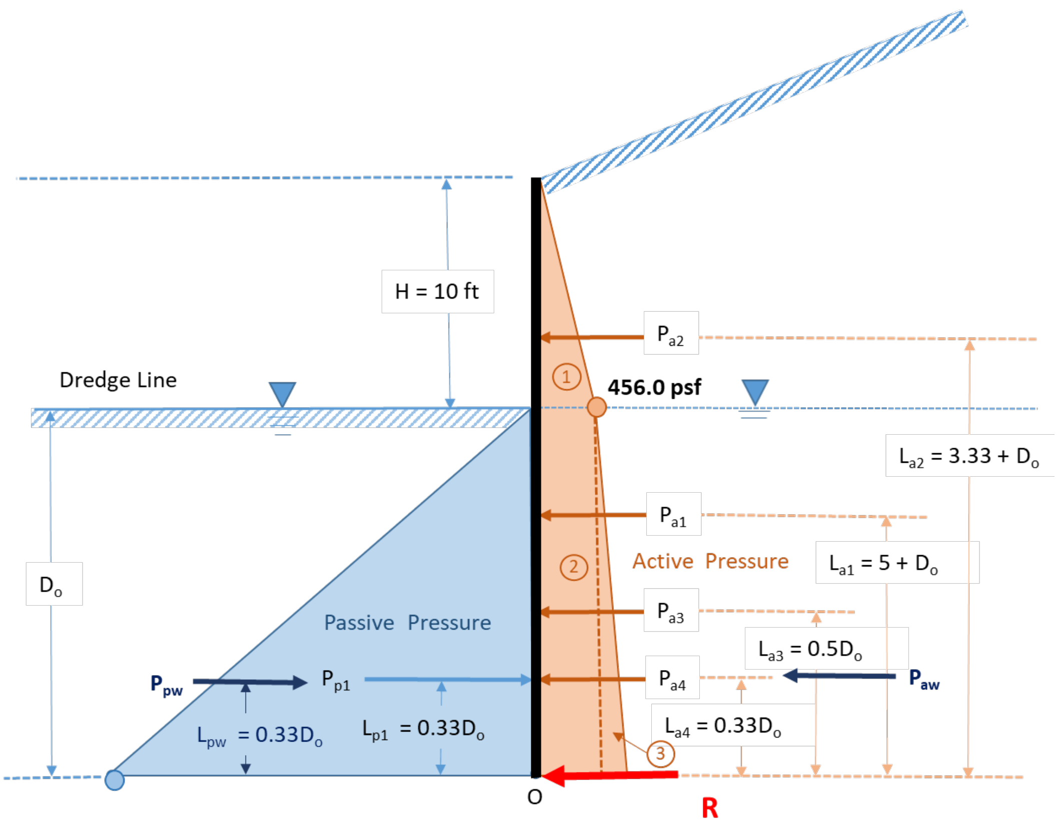


Figure 5-7 Case 2 Location of forces acting on the sheet pile wall, backfill slope = 20°.

As in Case 1, the initial sheet pile embedment depth is determined based on an "*un-factored*" analysis, i.e., FOS = 1.00. To determine the embedment depth, the summation of the moment about the pivot point "O" is conducted and equated to FOS = 1, as shown below.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS$$

Where

$\Sigma M_{disturbing}$ = $P_{a1}L_{a1} + P_{a2}L_{2a} + P_{a3}L_{a3} + P_{aw}L_{aw}$

$\Sigma M_{restoring}$ = $P_{p1}L_{p1} + + P_{pw}L_{pw}$

$\Sigma M_{disturbing}$ = $[2{,}280.0(3.33 + D_o)] + [(456.0D_o)(0.5D_o)] + [(13.51D_o^2)(0.33D_o)] + [(31.2\ D_o^2)(0.33D_o)$

$\Sigma M_{restoring}$ = $[(162.46D_o^2)(0.33D_o)] + [(31.2\ D_o^2)(0.33D_o)$

D_o was determined by placing the equations for the forces and moment arms into an EXCEL sheet. The spreadsheet is set up to input a depth, D_o, which then calculates the *FOS*. For a *FOS* = 1.0, the embedment depth D_o is calculated to be 10.47 feet, as shown in Table 5-4.

Table 5-4 Case 2 Embedment depth, D_o, for *FOS* = 1.0.

Case 2 -Calculation of Sheet Pile Depth, D_o (FOS = 1.0)			
Depth of Sheet Pile, D_o (ft)		10.47	
FOS		1.00	
Restoring Moment:		Disturbing Moment:	
P_{p1}	17,825	P_{a1}	2,280
L_{p1}	3.46	L_{a1}	13.8
P_{pw}	3423	P_{a2}	4,776
L_{pw}	3.46	L_{a2}	5.2
M_r	73,447	P_{a3}	1,482
		L_{a3}	3.5
		P_{aw}	3423
		L_{aw}	3.46
		M_d	73,447

To account for using the simplistic method D_o must be multiplied by a factor 1.2 as follows:

$D_u = 1.2D_o = 1.2(10.47) =$ **12.56 ft**

$D_{u,SupportIT}$ = **12.52 ft**

In using the *simplified method*, it is recommended that a check be conducted to make sure the horizontal force below the pivot point is greater than the resultant "R" shown in Figure 5-7. This step, however, is generally not conducted because it has been found that the additional length generally always concludes with acceptable results.

Case 2, Step 4: Calculate Maximum Bending Moment to Determine the Required Size of Sheet Pile

A hand calculation is not provided for the maximum bending moment for Case 2. The same method to calculate the maximum bending moment in Case 1 can be used in Case 2. According to the SupportIT calculations (Appendix B.2), the maximum bending moment for Case 2 (sloped backfill) is 30,765 ft-lbs/ft versus 33,235 ft-lbs/ft for Case 1 (with a level backfill and a 360 psf surcharge).

Case 2, Step 5: Sheet Pile Selection

The maximum moment at point O is equal to 18,753 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 50 ksi, a required section modulus, *Z*, is determined as follows:

Required section modulus, $Z = M/f_s$ = [18,753 ft-lbs/ft x 12 in/ft] / 50,000 psi = 4.5 in^3/ft

The section modulus of US Steel's PZ22 is 18.1 in^3/ft; therefore, a PZ22 sheet pile wall can meet the section modulus requirement.

Case 2, Step 6: Calculate Sheet Pile Total Length, D_f, Using the CP2 Method at FOS = 1.50

To apply a factor of safety of 1.5 using the Gross Pressure CP2 method, the same procedure used in Case 1 – Method 2 is used. The EXCEL output is provided in Table 5-5.

Case 2 -Calculation of Sheet Pile Depth, D_o (FOS = 1.0)			
Depth of Sheet Pile, D_o (ft)		14.86	
FOS		1.50	
Restoring Moment:		Disturbing Moment:	
P_{p1}	35,874	P_{a1}	2,280
L_{p1}	4.9	L_{a1}	18.2
P_{p1}	6890	P_{a2}	6,776
L_{p1}	4.9	L_{a2}	7.4
M_r	209,706	P_{a3}	2,983
		L_{a3}	4.9
		P_{a3}	6890
		L_{a3}	4.9
		M_d	140,235

Table 5-5 Case 2 Embedment depth, D_o, for *FOS* = 1.5.

To compensate for the *simplified method,* the depth D_o must be increased by 20% as follows:

$D_f = 1.2D_0 = 1.2(14.86) =$ **<u>17.83 ft</u>**

$D_{f,SupportIT} =$ **<u>17.72 ft</u>**

Total Sheet Pile Length = H + D_f = 10 + 17.83 = **<u>27.83 ft Say 28 ft</u>**

<u>Comparison to the SupportIT software results (Appendix B.2):</u>

Table 5-6 Case 2 Comparison of hand calculations to SupportIT calculations.

	SupportIT **(Total pile length, ft)**	**Hand Calculations** **(Total pile length, ft)**
Maximum soil pressure at dredge line, (psf/ft)	456.0	456.0
Maximum Bending Moment Location, (ft), FOS = 1.0	15.73	*
Maximum Bending Moment, (ft-lbs/ft), FOS =1.0	18,754	*
Sheet Pile Embedment Length, D_o (ft) FOS = 1.0	12.52	12.56
Sheet Pile Embedment Length, D_u (ft) FOS = 1.0	22.52	12.56
USS 20% FOS Embedment Length, D_f (ft)	15.02 (15)	15.07 (15)
USS 40% FOS Embedment Length, D_f (ft)	17.53 (28)	17.58 (28)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	27.72 (28)	27.83 (28)

* not calculated

5.2.3 Case 3 – Cantilever TERS in Firm Clay with Level Back Slope

The soils in Cases 1 and 2 are cohesionless soils, which are assumed to have "drained conditions" and therefore use an *effective stress analysis* as well as assuming the soil has no cohesive strength or c = 0. Sheet pile wall design in cohesive (clay) soils for temporary sheet pile walls, on the other hand, are designed for "undrained conditions" using a *total stress analysis* and assume the soil has no frictional strength or $\phi = 0$.

There are at least three important issues in designing sheet pile walls in cohesive soils. First, over time the strength of the soil can change, especially for overconsolidated clays, resulting in changing lateral earth pressures acting on the wall. The earth pressures that develop immediately after installation of the sheet pile wall are calculated based on the assumption that undrained shear strength, *c*, (also known as S_u) exists in the soils. Assuming that the frictional strength of the soil is zero ($\phi = 0$) results in the coefficients of earth pressure being equal to one, i.e., for a level backfill, $K_a = \cos^2(45 - \phi/2) = \cos^2(45 - 0/2) = 1.0$; $K_p = \cos^2(45 + \phi/2) = \cos^2(45 + 0/2) = 1.0$. In heavily overconsolidated clays, however, the horizontal stresses from previous geological loading, such as from glaciers, can still be locked in, resulting in the *in-situ* earth pressure coefficient K_o being greater than one.

Second, it is common to assume the strength of cohesive soils remains constant with depth. This is a significant problem in using a "*limit equilibrium method*" in determining the required embedment depth of the sheet pile wall because the geostatic pressures increase with depth (assuming a constant unit weight for the cohesive soil). Further assuming a $K_a = K_p$ equal to one increases the active pressure at the same rate as the passive pressures. Assuming a constant cohesive strength thus requires a significant embedment depth to reach equilibrium between the active and passive pressures.

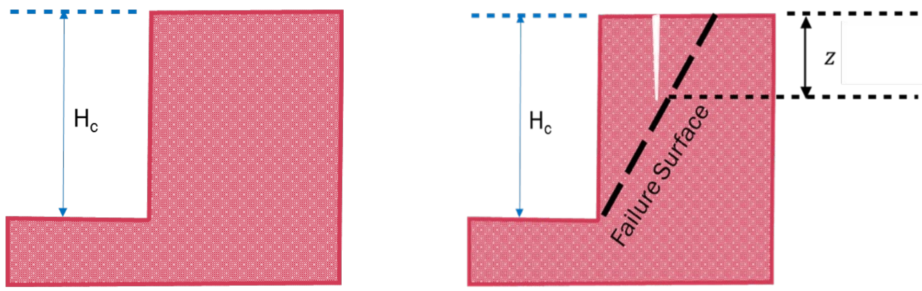
Third, cohesive soils can have unsupported vertical excavations due to its cohesive strength, S_u, the undrained shear strength of clay soil. In this manual, the S_u is given by the term "c." The height of the unsupported slope is commonly referred to as its "critical height," which, according to Terzaghi (1943), "*is the maximum height which the slope can have before the state of tension is relieved by the formation of vertical cracks*." According to Terzaghi, the equation for the critical height of a slope is $H_c = 4c/\gamma$ assuming no tension cracks develop in the slope. When tension cracks do develop, however, the equation reduces to $H'_c = 2.67c/\gamma$ to account for the tension cracks that intersect a potential failure plane, as shown in Figure 5-8. According to Terzaghi (1943), H'_c represents the maximum height of a vertical slope that has been weakened by tension cracks. Many designers, including the SupportIT software, however, use a more conservative assumption of

$$H_c = \frac{2c}{\gamma}$$

To account for loads placed on top of the cohesive soil, the SupportIT software further reduces the critical height as follows:

$$H'_c = \frac{2(c - q_{surcharge})}{\gamma}$$

where c = undrained shear strength,
$q_{surcharge}$ = surcharge,
γ = soil unit weight.



$H_c = \mathbf{4}\,\frac{S_u}{\gamma}$ Terzaghi (1943) $H_c = \mathbf{2.67}\,\frac{S_u}{\gamma}$ Terzaghi (1943)

Figure 5-8 Critical height relationships for unsupported cohesive soils (Terzaghi 1943).

The undrained shear strength of cohesive soil is determined by conducting undrained uniaxial compression tests, or in the field, a pocket penetrometer can be used. The pocket penetrometer provides the unconfined compressive strength of the soil, q_u. It is important to note that the relationship between the unconfined compressive strength and undrained shear strength is as follows:

$$q_u = 2c$$

The active and passive pressures acting on a sheet pile wall in cohesive soils ($\phi = 0$) with a level backfill are as follows:

$$P_a = \gamma z \, tan^2\left(45° - \frac{\phi}{2}\right) - 2c \, tan\left(45° + \frac{\phi}{2}\right)$$

$$P_p = \gamma z \, tan^2\left(45° + \frac{\phi}{2}\right) - 2c \, tan\left(45° + \frac{\phi}{2}\right)$$

For cohesive soils, where $\phi = 0$, these equations reduce to

$$P_a = \gamma z - 2c$$

$$P_p = \gamma z + 2c$$

The above equations have been modified in the British *Piling Handbook* 8th edition (2005) to account for wall adhesion, $S_{w\,max}$, as

$$P_a = \gamma z - cK_{ac}$$

$$P_p = \gamma z + cK_{pc}$$

Where

$$K_{ac} = K_p = 2\sqrt{\left(1 + \frac{S_{w\,max}}{c}\right)}$$

Soil adhesion, S_w, is assumed to be zero for this case, i.e., $S_{w\,max} = 0$. SupportIT software, however, does not include soil adhesion in the calculation of K_{ac} or K_{pc}.

Case 3, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall

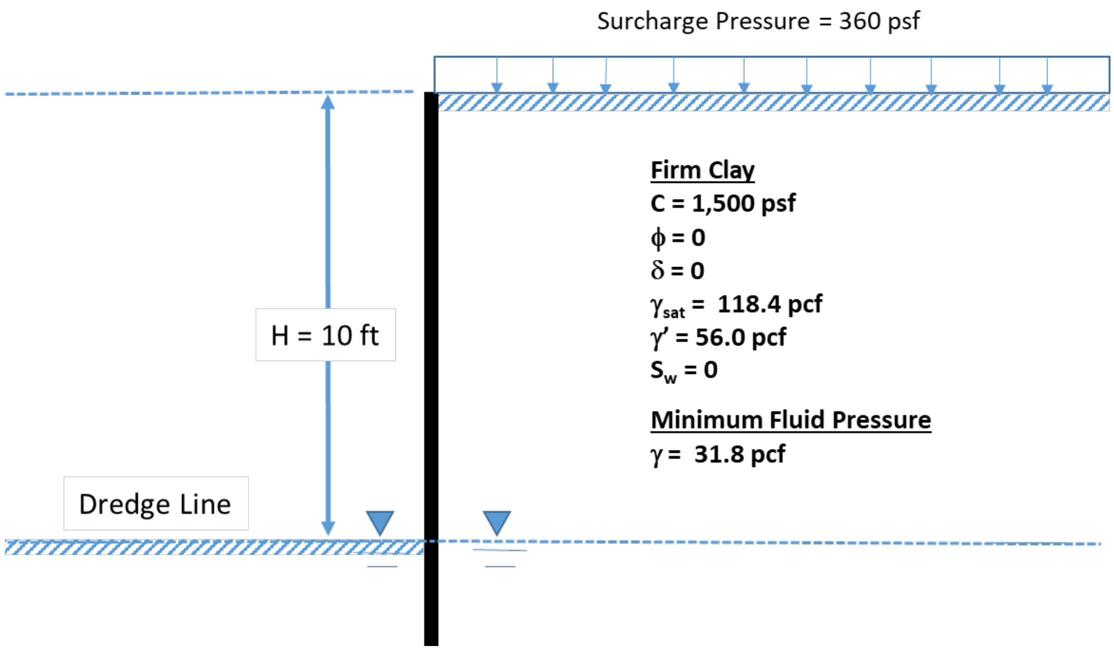


Figure 5-9 Case 3 Cantilevered sheet pile wall in firm clay.

Case 3, Step 2: Determination of Active and Passive Pressure Coefficients

As discussed above, the active and passive earth pressure coefficients will be equal to one for firm clay.

$K_a = K_p = 1.0$

Case 3, Step 3: Calculate the critical height for the soil

Cohesion: c = 1,500 psf

Calculate the soil's critical height: $H_c = \frac{2(c-q)}{\gamma} = \frac{2(1{,}500-360)}{118.4} \approx 19.2\ feet > 10\ ft\ (H)$

Due to the cohesive strength of the clay, there is no "*active*" pressure acting on the sheet pile to a depth of 19 feet. In this situation, designers can apply a number of approaches to "add" an active pressure to the sheet pile. One approach is to apply a "*Minimum Fluid Pressure (MFP)*." The SupportIT software recommends that a minimum fluid density of 5 kN/m^3 or 31.8 pcf be applied to the design calculations. An MFP of 31.8 pcf is applied in Case 3.

Case 3, Step 4: Calculate Active and Passive Earth Pressures and Forces

Active pressure: Minimum Fluid Pressure: $\sigma_{a1} = \gamma z = 31.8(z)$

Passive Pressure: Cohesive soil passive resistance: $\sigma_{p1} = 2c = 2(1{,}500) = 3{,}000$ psf

Increase in soil pressure: $\sigma_{p2} = z\gamma = 118.4(z)$

Note: Because the analysis is a total stress analysis, the soil's saturated unit weight is used, not the effective unit weight, γ'. Therefore, the water pressure is included in the soil's unit weight.

Active Pressures to a depth of 15 feet:

Minimum Fluid Pressure: $\sigma_{a10'} = K_a\gamma H = 1.0(31.8)(10) = \underline{318\text{ psf}}$

$\sigma_{a15'} = K_a\gamma H = 1.0(31.8)(15) = \underline{477\text{ psf}}$

Passive Pressures to a depth of 15 feet:

Lateral Pressure below Groundwater: $\sigma_{p1} = 2c = 2(1{,}500) = \underline{3{,}000\text{ psf}}$ (constant with depth)

$\sigma_{p2} = K_p\gamma H = 1.0(118.4)(5) = \underline{592\text{ psf}}$

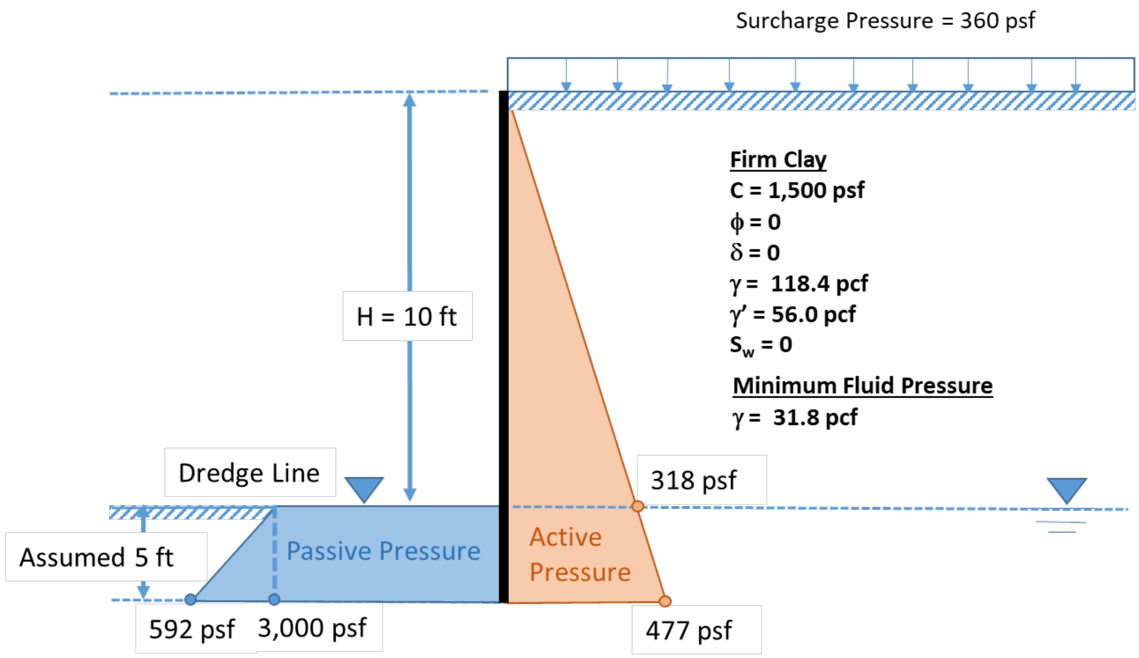


Figure 5-10 Case 3 Gross pressure distribution for a 5-ft embedment depth.

Case 3, Step 5: Calculate Sheet Pile Wall Embedment Depth for *FOS* = 1.0

The *simplified method* is used in Case 3 as it was in Case 1 and 2. This assumes the sheet pile wall is embedded to a depth where it becomes "fixed," i.e., fixed-earth method, and a pivot point where the stress on the wall reverses at a depth, D_o, below the dredge line.

Pressure Diagram - Forces and Force Locations in Terms of D_o

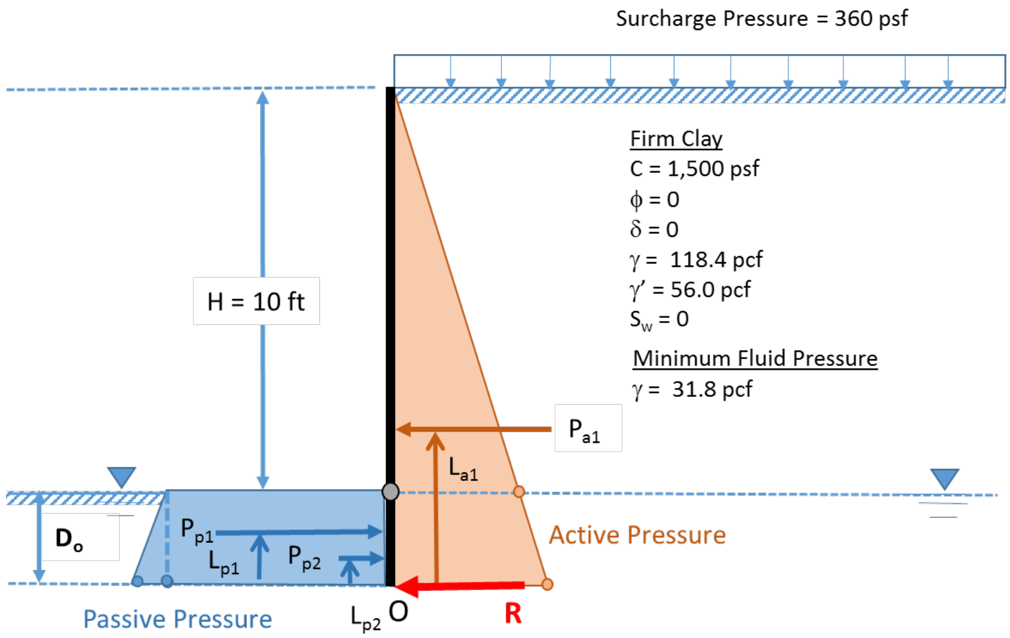


Figure 5-11 Case 3 Gross pressure distribution for an embedment depth D_o.

Active Forces, lbs

Active Force, P_{a1}:	$P_{a1} = 0.5(31.8)\ (10 + D_o)(10 + D_o)(1) = 15.9(10 + D_o)^2$
Active Force Location, L_{a1}	$L_{a1} = 0.33(10 + D_o) = 3.33 + 0.33D_o$

Passive Force, lbs

Passive Force P_{p1}:	$P_{p1} = 3{,}000(D_o)(1) = 3{,}000D_o$
Passive Force P_{p2}:	$P_{p2} = 0.5(118.4)(D_o^2)(1) = 59.18D_o^2$
Passive Force Location, L_{p1}	$L_{p1} = 0.5D_o$
Passive Force Location, L_{p2}	$L_{p2} = 0.33D_o$

To determine the embedment depth, the summation of moment at the dredge line is conducted and equated to *FOS* = 1, as shown below.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS$$

where

$\Sigma M_{disturbing}$ = $P_{a1}L_{a1}$

$\Sigma M_{restoring}$ = $P_{p1}L_{p1} + P_{p2}L_{p2}$

D_o is determined by placing the equations into an EXCEL sheet. The spreadsheet is set up to input a depth, D_o, which then calculates the *FOS*. For a *FOS* = 1.0, the embedment depth D_o is calculated at 2.62 feet as shown in Table 5-7.

Table 5-7 Case 3 Embedment depth, D_o, for *FOS* = 1.0

Case 3 -Calculation of Sheet Pile Depth, D_0			
Depth of Sheet Pile, D_o		2.62	
FOS		1.00	
Restoring Moment:		Disturbing Moment:	
Pp_1	7,846	Pa_1	2,530
Lp_1	1.31	La_1	4.2
Pp_2	405	M_a	10,611
Lp_2	0.9		
M_p	10,611		

The simplified method requires a 20% increase in D_o to obtain the unfactored embedment length, D_u, as follows:

D_o = **2.62 ft**

$D_u = 1.2D_o = 1.2(2.62) =$ **3.14 ft**

$D_{u,SupportIT}$ = **3.16 ft**

Case 3, Step 6: Calculate Maximum Bending Moment to Determine the Required Size of Sheet Pile

A depth “Y” is assumed where the active forces equal the passive forces (zero shear), and the maximum bending stress occurs in the sheet piling. This point occurs between the dredge line and the pivot point “O” at a depth “Y” as shown in Figure 5-12.

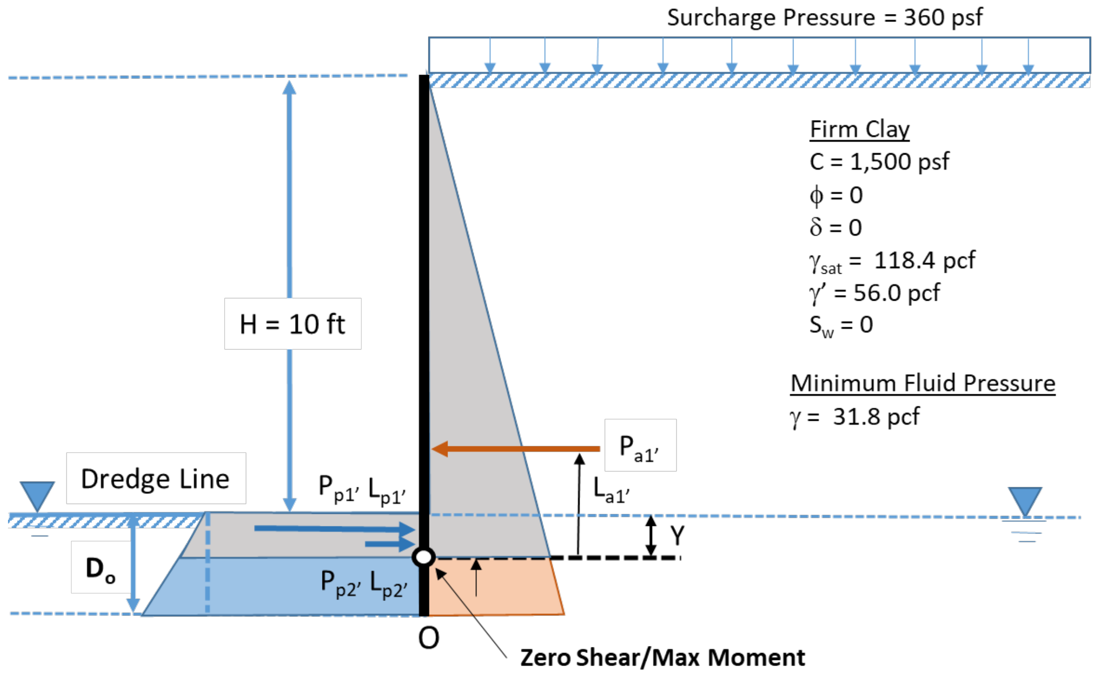


Figure 5-12 Case 3 Gross pressure diagram indicating the location of net zero-shear and maximum moment.

Step 6A:

Determine forces acting on sheet pile above the point of zero shear in terms of the depth "Y."

Active Forces:

$P_{a1'} = 0.5(31.8)(10 + Y)^2 = 15.9Y^2 + 318Y + 1{,}590$

Passive Force:

$P_{p1'} = 3{,}000Y$

$P_{p2'} = 0.5(118.4)Y^2 = 59.2Y^2$

Solve for the depth "Y" by $\Sigma P_{a'} = \Sigma P_{p'}$ or $P_{a1'} = P_{p1'} + P_{p2'}$

$15.9Y^2 + 318Y + 1{,}590 = 3{,}000Y + 59.2Y^2$

Solution: **Y = 0.59 ft**

Solution: **$Y_{SupportIT}$ = 0.60 ft**

Step 6B:

Calculate the moment arm of the forces in terms of the depth "Y":

Active Force Moment Arms:
$L_{a1'} = 0.333(10 + Y) = 0.333(10.587) = 3.53$
$P_{a1'} = 15.9Y^2 + 318Y + 1,590 = 1,782$

Passive Force Moment Arms:
$L_{p1'} = 0.5Y$
$L_{p2'} = 0.333(Y) = 0.333Y$

Max Bending Moment = $\Sigma P_{a'} - \Sigma P_{p'}$

$P_{p1'} = 3,000Y = 1,770$
$P_{p2'} = 59.2Y^2 = 20$
$L_{a1'} = 3.52$
$L_{p1'} = 0.5Y = 0.29$
$L_{p2'} = 0.333Y = 0.20$

Max Bending Moment $= (P_{a1'} L_{a1'}) – [(P_{p1'}L_{p1'}) + (P_{p2'} L_{p2'})]$
$= (1,782)(3.53) - [(1,761)(0.29) + (20)(0.20)]$
$= 5,776$ ft-lbs/ft

Maximum Moment = **5,780 ft-lbs/ft**

Maximum Moment$_{SupportIT}$ **= 5,796 ft-lbs/ft**

Case 3, Step 7: Sheet Pile Selection

The maximum moment at point O is equal to 5,780 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 50 ksi, a required section modulus, *Z*, is determined as follows:

Required section modulus, $Z = M/f_s$ = [5,780 ft-lbs/ft x 12 in/ft] / 50,000 psi = 1.4 in^3/ft

The section modulus of US Steel's PZ22 is 18.1 in^3/ft; therefore, a PZ22 sheet pile wall can meet the section modulus requirement.

Case 3, Step 8: Calculate Sheet Pile Total Length with a Factor of Safety

The Gross Pressure CP2 method is used to determine the embedment depth D_f. This procedure is the same as Method 2 discussed in Case 1. The depth D_f is then recalculated as shown in Table 5-8 using a FOS = 1.50.

Table 5-8 Case 3 Embedment depth, D_o, with a *FOS* = 1.5.

Case 3 - Calculation of Sheet Pile Depth, D_o			
Depth of Sheet Pile, D_o		3.54	
FOS		1.50	
Restoring Moment:		Disturbing Moment:	
Pp_1	10,622	Pa_1	2,915
Lp_1	1.77	La_1	4.5
Pp_2	742	M_a	13,114
Lp_2	1.17		
M_p	19,671		

To compensate for the *simplified method*, the depth D_o is increased by 20% as follows:

$D_f = (1.2)D_o = 1.2(3.54) =$ **4.25 ft**

$D_{f,SupportIT} =$ **4.29 ft**

Total Sheet Pile Length = H + D_f = 10 + 4.25 = **14.25 ft**

Comparison to the SupportIT software results (see Appendix B.3):

Table 5-9 Case 3 Comparison of hand calculations to SupportIT calculations.

	SupportIT (Total pile length, ft)	**Hand Calculations (Total pile length, ft)**
Maximum soil pressure at dredge line, (psf/ft)	318	318
Maximum Bending Moment Location, (ft), FOS = 1.0	0.60	0.59
Maximum Bending Moment, (ft-lbs/ft), FOS =1.0	5,795	5,776
Sheet Pile Embedment Length, D_o (ft) FOS = 1.0	2.63	2.62
Sheet Pile Embedment Length, D_u (ft) FOS = 1.0	3.16	3.14
USS 20% FOS Embedment Length, D_f (ft)	3.8 (14)	3.8 (14)
USS 40% FOS Embedment Length, D_f (ft)	4.4 (15)	4.4 (15)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	4.3 (15)	4.3 (15)

5.2.4 Case 4 – Anchored Cantilever TERS in Cohesionless Soil

Case 4 will use the same soil conditions as in Case 1 but will extend the excavation to a 20-foot depth. The 20-foot will require anchored support for stability. To make the example problem comparable to Case 1, the water table is lowered to the dredge line at a depth of 20 feet, as shown in Figure 5-13.

The free-earth method is used to analyze the sheet pile wall stability in Case 4. The free earth support condition ensures that the penetration of the piles is sufficient to prevent forward movement of the toe, but not adequate to prevent rotation. The pile embedment depth is calculated by taking the summation of moments at the anchor location. The anchor force, T, is determined by the summation of horizontal forces acting on the wall. In Case 4, the anchor is installed at a depth of two feet.

Case 4, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall

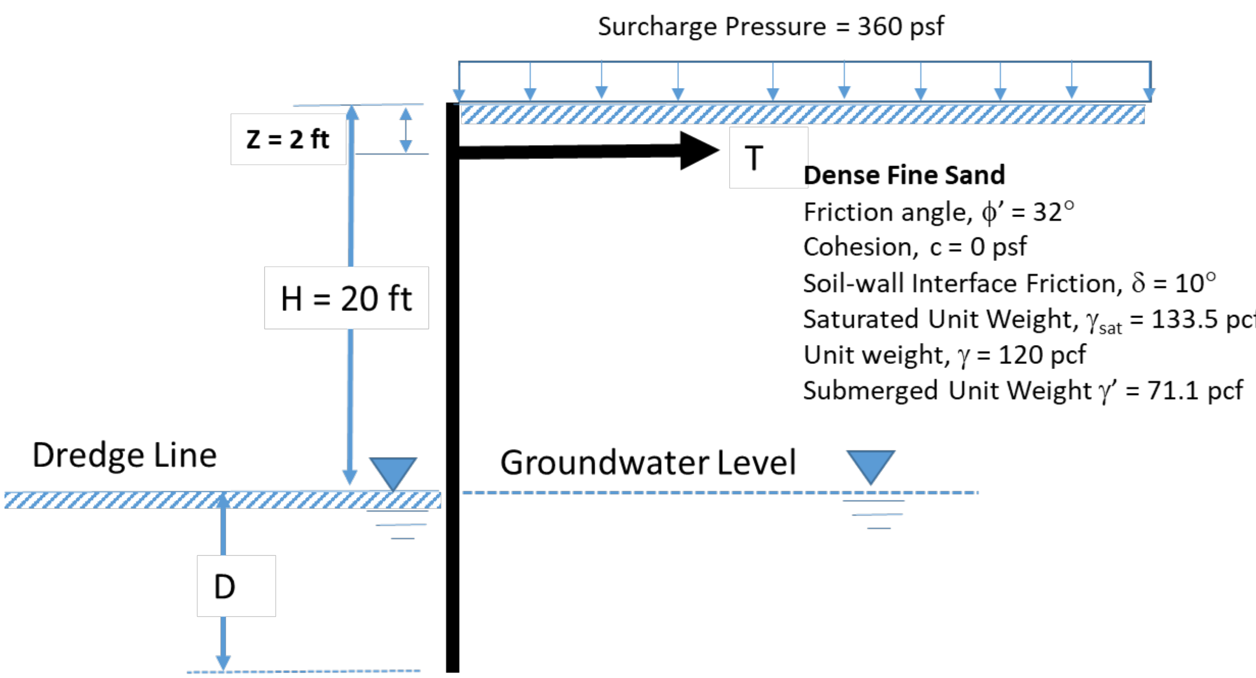


Figure 5-13 Case 4 Anchored cantilever wall in cohesionless soil.

Case 4, Step 2: Determination of Active and Passive Earth Pressures and Forces to a Depth of 30-ft

Using the Rankine Method, the coefficients of active and passive lateral pressures are determined as follows and shown in Figure 5-14.

$$K_a = tan^2\left(45^\circ - \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ - \frac{32^\circ}{2}\right) = \mathbf{0.31}$$

$$K_p = tan^2\left(45^\circ + \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ + \frac{32^\circ}{2}\right) = \mathbf{3.25}$$

Active Pressures: $\sigma_{a1} = K_a\sigma_v = 0.31(360) = \underline{111.6 \text{ psf}}$

$\sigma_{a20} = 112 + K_a\sigma_v = 112 + 0.31(20)(120) = \underline{855.6 \text{ psf}}$

$\sigma'_{a30} = 856 + K_a\sigma_v = 855.6 + 0.31(10)(71.1) = \underline{1{,}076.0 \text{ psf}}$

Passive Pressures: $\sigma_{p10} = K_a\sigma_v = 3.25(10)(71.1) = \underline{2{,}311 \text{ psf}}$

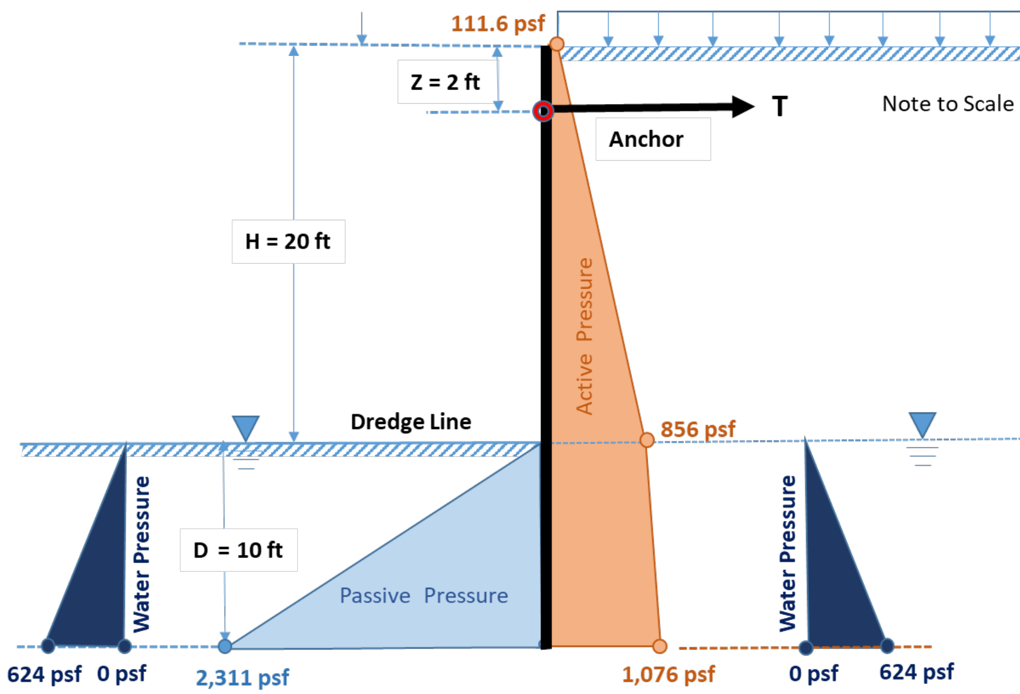


Figure 5-14 Case 4 Lateral earth pressures to a depth of 30 feet.

Case 4, Step 3: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0

The anchor is located at a depth two feet below the top of the wall, as shown in Figure 5-14. To estimate the wall's maximum bending moment and embedment length D_u, the summation of moments is taken at the anchor's location using a FOS = 1.00. For this calculation, it is assumed that the anchor acts like a plastic hinge with no capacity to resist moments, i.e., $\Sigma M = 0$. The lateral active and passive forces and their locations based on the anchor position are shown in Figure 5-15.

The active and passive pressures acting on the wall are shown in Figure 5-15. The active pressures are divided into seven regions and the passive pressure into one region. For clarity, the water pressure distribution is not shown in Figure 5-15, but its resultant forces are shown. The equations for the lateral forces acting on the wall for the seven active forces and one passive force are provided.

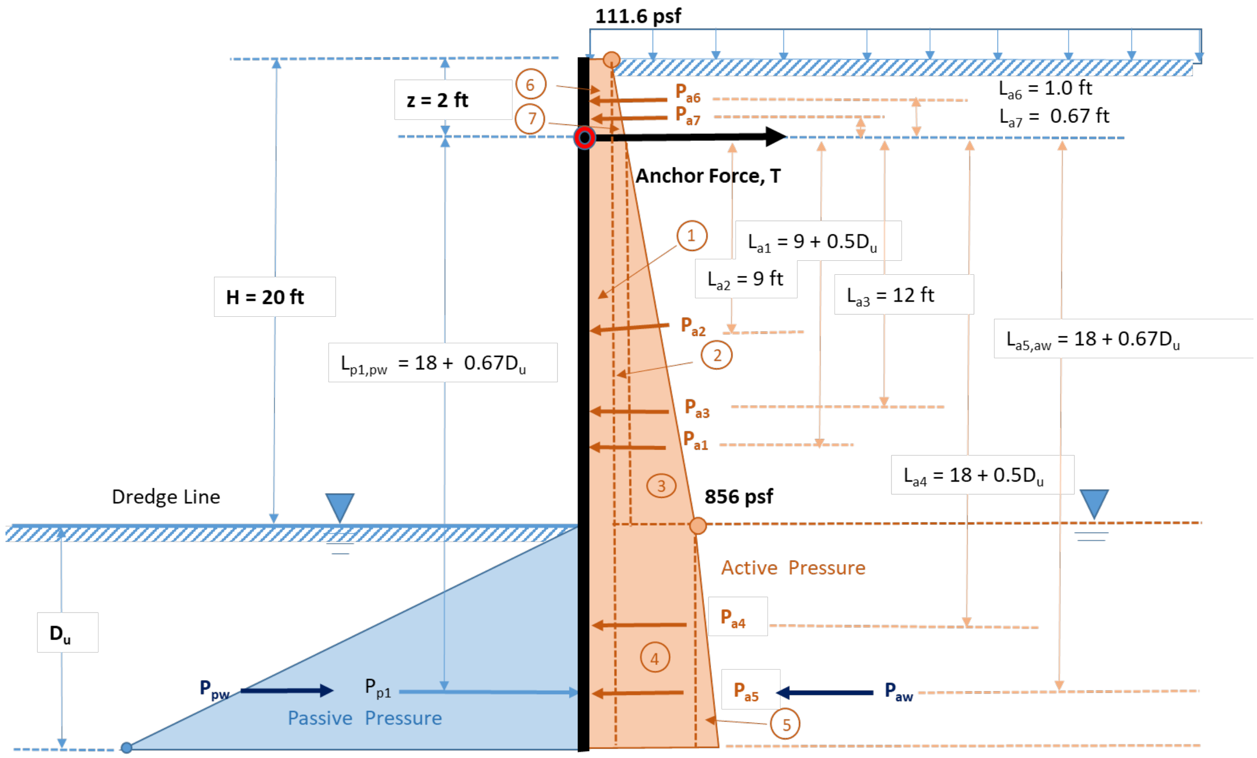


Figure 5-15 Case 4 Active and passive forces and locations acting on the sheet pile wall as a function of D_u.

<u>Active Forces, lbs</u>

Active Force P_{a1}:	$P_{a1} = [(0.31)(360)(18 + D_u)](1) = \underline{2{,}008.8 + 111.6D_u}$
Active Force P_{a2}:	$P_{a2} = (0.31)(2)(120)(18)(1) = \underline{1{,}339.2}$
Active Force P_{a3}:	$P_{a3} = (0.31)(0.5)(120)(18)(18)(1) = \underline{6{,}026.4}$
Active Force P_{a4}:	$P_{a4} = (0.31)(20)(120)D(1) = \underline{744D_u}$
Active Force P_{a5}:	$P_{a5} = [(0.31)(0.5)(71.1)(D)(D)(1) = \underline{11.02D_u^2}$
Active Force P_{a6}:	$P_{a6} = (0.31)(360)(2) = \underline{223.2}$
Active Force P_{a7}:	$P_{a7} = (0.31)(0.5)(2)(120)(2) = \underline{74.4}$
Water Force P_{aw}:	$P_{aw} = (0.5)(62.4)(D_u)^2 = \underline{31.2D_u^2}$

<u>Moment Arm from Anchor, ft</u>

Active Force Location, L_{a1}	$L_{a1} = 0.5(18+D_u) = \underline{9 + 0.5D_u}$
Active Force Location, L_{a2}	$L_{a2} = \underline{9}$
Active Force Location, L_{a3}	$L_{a3} = \underline{12}$
Active Force Location, L_{a4}	$L_{a4} = 18 + 0.5D_u = \underline{18 + 0.5D_u}$
Active Force Location, L_{a5}	$L_{a5} = \underline{18 + 0.67D_u}$
Active Force Location, L_{a6}	$L_{a6} = 0.5(2) = \underline{1}$
Active Force Location, L_{a7}	$L_{a7} = 0.33(2) = \underline{0.67}$
Water Force Location, L_{a7}	$L_{aw} = 18 + 0.67D_u = \underline{18 + 0.67D_u}$

Passive Force, lbs

Passive Force P_{p1}: $P_{p1} = 0.5(3.25)(71.1)(D)(D)(1\ ft) = \underline{115.5D_u^2}$

Water Force P_{pw}: $P_{pw} = (0.5)(62.4)(D_u)^2 = \underline{31.2D_u^2}$

Moment Arm from Anchor, ft

Passive Force Location, L_{p1} $L_{p1} = \underline{18 + 0.67D_u}$

Water Force Location, L_{pw} $L_{pw} = \underline{18 + 0.67D_u}$

For stability, the moments about the anchor should be in equilibrium for an embedment depth, D_u, the summation of the moment about the anchor is conducted and equated to FOS = 1 as follows.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = 1.0$$

where

$\Sigma M_{disturbing} = P_{a1}L_{a1} + P_{a2}L_{2a} + P_{a3}L_{a3} + P_{a4}L_{a4} + P_{a5}L_{a5} + P_{aw}L_{aw}$

$\Sigma M_{restoring} = P_{p1}L_{p1} + P_{a6}L_{a6} + P_{a7}L_{a7} + P_{pw}L_{pw}$

D_u is determined using an EXCEL spreadsheet as shown in Table 5-10. The spreadsheet is set up to input a depth, D_u, which then calculates the *FOS*. For a *FOS* equal to one, the embedment depth D is 11.06 as shown in Table 5-10.

Table 5-10 Case 4 Embedment depth, D, for *FOS* = 1.0.

Case 4 - Calculation of Sheet Pile Depth, D_u			
Depth of Sheet Pile, D_u (ft)		11.06	
FOS		1.00	
Restoring Moment:		Disturbing Moment:	
P_{p1}	14,134	P_{a1}	3,243
L_{p1}	25.4	L_{a1}	14.5
P_{a6}	223	P_{a2}	1,339
L_{a6}	1.0	L_{a2}	9.0
P_{a7}	74.4	P_{a3}	6,026
L_{a7}	0.7	L_{a3}	12.0
P_{pw}	3,818	P_{a4}	8,230
L_{pw}	25.4	L_{a4}	23.5
M_r	456,449	P_{a5}	1,349
		L_{a5}	25.4
		P_{aw}	3,818
		L_{aw}	25.4
		M_d	456,449

Unfactored: FOS = 1.0:

D_u = <u>11.06 ft</u>
$D_{u,SupportIT}$ = <u>11.12 ft</u>

<u>Case 4, Step 4(a): Determine Anchor Load</u>

The anchor load is determined as follows:

T = Active forces – Passive Forces = $(P_{a1} + P_{a2} + P_{a3} + P_{a4} + P_{a5} + P_{a6} + P_{a7} + P_{aw}) - (P_{p1} + P_{pw})$

T = (3,243 + 1,339 + 6,026 + 8,229 + 1,346 + 223 + 74 + 3,817) – (14,131 + 3,817) = <u>6,352 lbs/ft</u>

T = <u>6,352 lbs/ft</u>
$T_{SupportIT}$ <u>= 6,278 lbs/ft</u>

<u>Case 4, Step 4(b): Determine Shear Force in Sheet Pile</u>

The shear force at a depth of 2 ft, however, does not include the shear force above the anchor. Therefore, the sheet pile shear force is calculated as follows:

$\tau_{at\ anchor}$ = (3,243 + 1,339 + 6,026 + 8,229 + 1,346) – 14,131 = <u>6,052 lbs/ft</u>

$\tau_{at\ anchor}$ = <u>6,052 lbs/ft</u>
$\tau_{SupportIT}$ = <u>5,977 lbs/ft</u>

<u>Case 4, Step 5: Determine sheet pile maximum bending moment</u>

Determination of the location of zero shear:

The maximum bending moment in the sheet piling occurs at the location of the zero shear force at a distance D_s below the anchor as shown in Figure 5-16. To locate the "zero shear force location", the shear forces are summed below the anchor until they equal the anchor load of 6,052.

$\tau' = \Sigma P_a$ (to a depth of D_s)
$6{,}052 = 0.31[(360) + (2)(120)]D_s + [0.31(0.5)(120)D_s]D_s = 186.0D_s + 18.6D_s^2$

$18.6D_s^2 + 186.0D_s - 6{,}053 = 0$

D_s = <u>13.72 ft (15.72 ft along sheet pile)</u>
$D_{s,SupportIT}$ <u>= 13.61 ft (15.61 ft along sheet pile)</u>

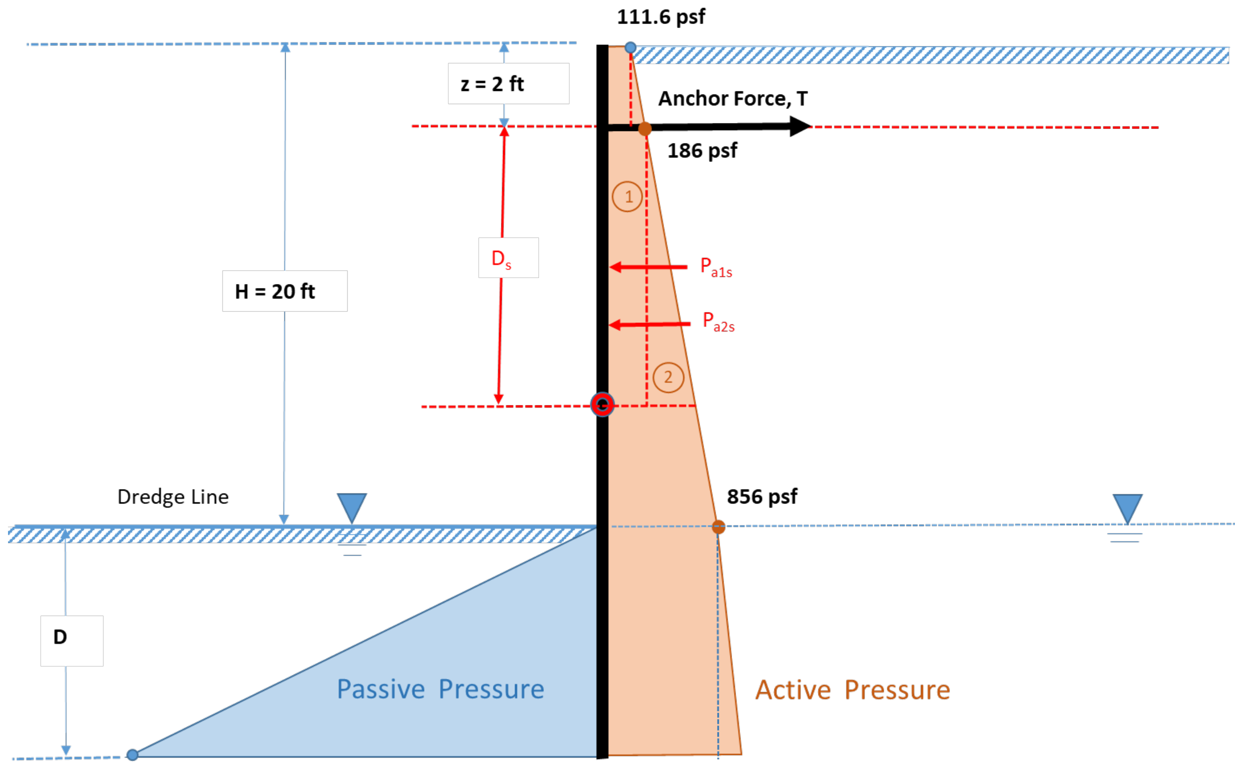


Figure 5-16 Case 4 Location of shear stress point at D_s.

Calculate the maximum moment at depth D_s for the forces acting above point D_s to the anchor. In this calculation, the shear force at the anchor, 6,052 lb/ft is used.

Anchor Moment:

$M = +(\tau_{anchor})(D_s) = 6{,}052\ (13.72) = \underline{+83{,}033\text{ ft-lbs/ft}}$

$P_{a1s} = 0.31[360 + (2)(120)]D_s = 186.0(13.72) = 2{,}551.9\text{ lbs/ft}$

$L_{a1,s} = D_s/2 = 13.72/2 = 6.86\text{ ft}$

$M_{a1'} = -(P_{a1s})(L_{a1,s}) = -(2{,}551.9)(6.86) = -\underline{17{,}506.2}$

$P_{a2s} = [0.5(0.31)(120)(13.72)^2] = 3{,}501.2\text{ lbs/ft}$

$L_{a2,s} = 13.72/3 = 4.57\text{ ft}$

$M_{a2,s'} = -(3{,}501.2)(4.57) = -16{,}012\text{ ft-lbs/ft}$

$M_{a2,s'} = -16{,}012\text{ ft-lbs/ft}$

Total Moment:

M = +83,047 – [17,506 + 16,012] = +49,529 ft-lbs/ft

M = <u>+49,529 ft-lbs/ft</u>

$M_{SupportIT}$ <u>= +48,262 ft-lbs/ft</u>

<u>Case 4, Step 6: Sheet Pile Selection</u>

The maximum moment was estimated to be 49,529 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 25 ksi, the required section modulus is determined as follows

Required section modulus = M/f_s = [49,529 ft-lbs/ft x 12 in/ft]/25,000 psi = 23.8 in^3/ft

The section modulus of US Steel's PZ27 is 30.2 in^3/ft. Therefore a PZ27 would meet the section modulus requirement. According to the SupportIT solution provided in Appendix B.4, it was determined from the SupportIT software that a PZ27 sheet pile would have 1.2 inches of deflection, which meets the deflection limit.

<u>Case 4, Step 7: Calculate Sheet Pile Total Length with a Factor of Safety</u>

The Gross Pressure CP2 method is used to determine the embedment depth D_f using a FOS = 1.50. The depth D_f is then recalculated, as shown in Table 5-11.

Table 5-11 Case 4 Embedment depth, D, for *FOS* = 1.5.

Case 4 - Calculation of Sheet Pile Depth, D_f			
Depth of Sheet Pile, D_u (ft)		17.42	
FOS		1.50	
Restoring Moment:		Disturbing Moment:	
P_{p1}	35,042	P_{a1}	3,953
L_{p1}	29.7	L_{a1}	17.7
P_{a6}	223	P_{a2}	1,339
L_{a6}	1.0	L_{a2}	9.0
P_{a7}	74.4	P_{a3}	6,026
L_{a7}	0.7	L_{a3}	12.0
P_{pw}	9,466	P_{a4}	12,959
L_{pw}	29.7	L_{a4}	26.7
M_r	1,320,810	P_{a5}	3,343
		L_{a5}	29.7
		P_{aw}	9,466
		L_{aw}	29.7
		M_d	880,540

Note that Case 4 is based on the free earth method using the Gross Pressure CP2 Method.

D_f = <u>17.42 ft</u>

$D_{f,SupportIT}$ = <u>17.49 ft</u>

Two sets of SupportIT Software output are provided in Appendix B.4. One set is for the FOS = 1.00 case and the second set is for the FOS = 1.50 case.

Comparison to the SupportIT software results:

Table 5-12 Case 4 Comparison of hand calculations to SupportIT output (Appendix B.4).

	SupportIT (Total pile length, ft)	Hand Calculations (Total pile length, ft)
Maximum soil pressure at dredge line, (psf/ft)	856	856
Anchor Load, (lbs/ft)	6,278	6,352
Sheet Pile Shear Force, (lbs/ft)	5,977	6.052
Zero Shear location along sheet pile, (ft)	15.61	15.72
Maximum Moment, (ft-lbs/ft), FOS = 1.0	48,262	49,529
Sheet Pile Embedment, FOS = 1.00, D_u (ft)	11.12	11.06
USS 20% FOS Embedment Length, D_f (ft)	13.3 (33)	13.2 (33)
USS 40% FOS Embedment Length, D_f (ft)	15.6 (37)	15.6 (37)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	17.49 (38)	17.42 (38)

5.2.5 Case 5 – Anchored Cantilever TERS in Firm Cohesive Soil

Case 5 uses the same soil conditions as Case 3 but extends the excavation to a depth of 20 feet, thus requiring an anchor for stability. Developing anchor pullout strength in clay is difficult, so a five-foot sand backfill is placed over the clay soil for anchor installation. A *free earth support method* is conducted using the sheet pile selected in Case 1, a US Steel PZ22. To make the example problem comparable to Case 3, the water table is lowered to the dredge line at a depth of 25 feet below the top of the sheet pile wall, as shown in Figure 5-17.

The *free earth method* has no point of fixture and requires the following two unknowns be determined, the sheet pile embedment length, D, and the anchor force, T. The depth of the embedment is calculated by taking the summation of moments at the anchor location, while the anchor force, T, is determined by the summation of horizontal forces acting on the wall. The anchor is installed at a depth of three feet in the sand backfill.

<u>Case 5, Step 1: Define the Dimensions and Soil Properties for Sheet Pile Wall</u>

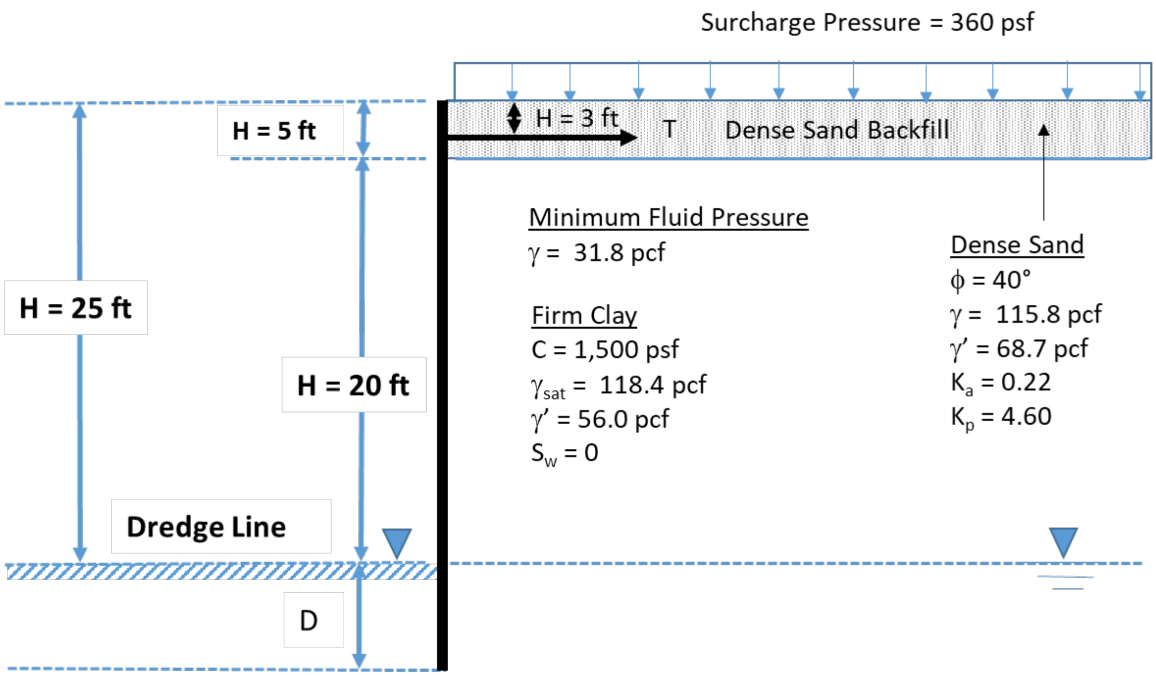


Figure 5-17 Case 5 Anchored cantilever TERS in firm clay.

<u>Case 5, Step 2: Determination of Active and Passive Pressure Coefficients</u>

Sand:

$$K_a = tan^2\left(45^\circ - \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ - \frac{40^\circ}{2}\right) = \mathbf{0.22}$$

$$K_p = tan^2\left(45^\circ + \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ + \frac{40^\circ}{2}\right) = \mathbf{4.60}$$

Clay: $K_a = K_p = 1.0$

Case 5, Step 3: Calculate the Critical Height for the Soil

The clay layer's critical height, H_c, will include the surcharge pressure (360 psf). Therefore, the total surcharge is 360 psf + (5 ft x 115.8 lb/ft^3) or 939 psf. The calculation of the critical height, H_c is as follows:

Cohesion: c = 1,500 psf

$$H_c = \frac{2c - q_{surcharge}}{\gamma} = \frac{2(1{,}500) - 939\,psf}{118.4\,pcf} = 17.4\,feet < 20\,ft\,(H)$$

Thus, the top 17.4 ft of the clay layer will be self-supporting, while the bottom 2.6 ft will have an active pressure applied to the sheet pile wall.

The SupportIT software, however, will apply a "*minimum fluid pressure*" (MFP) to the top 17.4 ft of the sheet pile wall. The default MFP recommended by SupportIT is 31.8 psf. The clay's active pressure is applied to the wall starting at 17.4 ft to the base of the sheet pile. However, the clay's active pressure is lower than the MFP at this depth. Therefore, the SupportIT software uses the larger MFP along the sheet pile wall for the stability calculation for the sheet pile.

Case 5, Step 4: Calculate Active and Passive Earth Pressures and Forces

The active and passive pressures acting on the sheet pile wall are shown in Figure 5-18.

Active pressure:	Minimum Fluid Pressure:	$\sigma_{a1} = \gamma z = 31.8(z)$
Passive Pressure:	Cohesive soil passive resistance:	$\sigma_{p1} = 2c = 2(1{,}500) = 3000$ psf
	Increase in soil pressure:	$\sigma_{p2} = z\gamma = 118.4(z)$

Note: Since the analysis is a total stress analysis, the bulk unit weight of the soil γ is used, not the effective unit weight, γ'. Therefore, the water below the water table is included in the unit weight.

Pressures Acting on Sheet Pile Wall:

Active Pressures:

Surcharge Pressure in sand backfill:	$\sigma_{a1} = K_a\sigma_v = (0.22)(360\text{ psf}) = \underline{79.2}$ psf
Sand backfill:	$\sigma_{a2} = K_a\sigma_v = (0.22)(5\text{ ft})(115.8\text{ pcf}) = \underline{127.4}$ psf
	$\sigma_{a5'} = \sigma_{a1} + \sigma_{a2} = 79.2 + 127.4 = \underline{206.6}$ psf
Minimum Fluid Pressure:	$\sigma_{a5'} = K_a\gamma H = (1.0)(31.8)(5) = \underline{159\text{ psf}}$
	The active pressure will exceed the MFP at 23.8 ft
	$\sigma_{a23.8'} = 31.80(23.79) = 756.5$ psf, MFP
	$\sigma_{a23.8'} = (23.79 - 17.4)(118.4) = 756.6$ psf, (MFP)
	$\sigma_{a30'} = K_a\gamma H = (1)(30 - 17.4)(118.4) = \underline{1{,}491.8\text{ psf (Active)}}$

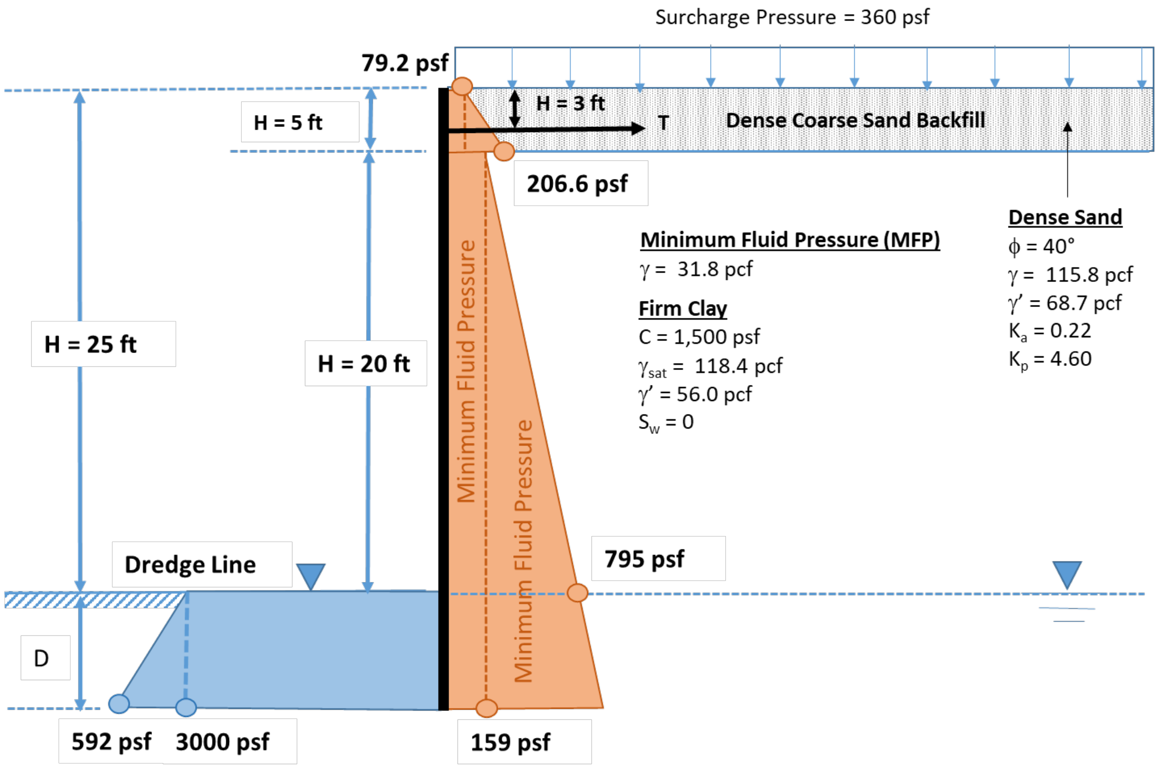


Figure 5-18 Case 5 Lateral earth pressures.

The lateral active and passive forces acting on the sheet pile wall, and their respective locations, are shown in Figure 5-19. A more detailed view of the sand backfill section is provided in Figure 5-20.

Passive Pressures to a depth of 5 feet below the dredge line:

Lateral Pressure below Groundwater: $\sigma_{p1} = 2c = (2)(1{,}500) = \underline{3{,}000\text{ psf}}$ (constant with depth)

$\sigma_{p2} = K_p\gamma H = (1)(118.4)(5) = \underline{592\text{ psf}}$

Forces and Force Locations in Terms of D as Shown Below

Active Forces, lbs/ft

$P_{a1} = 0.22[(3)(360)](1) = 237.6$

$P_{a2} = 0.22[(0.5)(3)(115.8)(3)](1) = 114.6$

$P_{a3} = 0.22[(2)(360)](1) = 158.4$

$P_{a4} = 0.22[(3)(115.8)(2)](1) = 152.9$

$P_{a5} = 0.22[(0.5)(2)(115.8)(2)](1) = 51.0$

$P_{a6} = 1.0[(5)(31.8)(20 + D_u)](1) = 3{,}182 + 159.1D_u$

$P_{a7} = 1.0[(0.5)[(31.8)(20 + D)(20 + D)](1) = 15.9D_u^2 + 636.4D_u + 6{,}360.4$

Active Forces Moment Arm, (ft), referenced to the anchor force location

$L_{a1} = (0.5)(3) = 1.5$
$L_{a2} = (2/3)(3) = 1.0$
$L_{a3} = (0.5)(2) = 1.0$
$L_{a4} = (0.5)(2) = 1.0$
$L_{a5} = (2/3)(2) = 1.33$
$L_{a6} = 2 + 0.5(20 + D_u) = 12 + 0.5D_u$
$L_{a7} = 2 + 0.67(20 + D_u) = 15.3 + 0.67D_u$

Passive Force, lbs/ft

Passive Force P_{p1}: $P_{p1} = (1)[(2)(1{,}500)D](1) = 3{,}000D_u$
Passive Force P_{p1}: $P_{p2} = (1)[0.5(118.4)(D)(D)](1) = 59.3D_u^2$

Passive Force Moment Arm (ft)

Passive Force Location, L_{p1} $L_{p1} = 22 + (0.5)(D_u) = 22 + 0.5D_u$
Passive Force Location, L_{p2} $L_{p2} = 22 + (0.67)(D_u) = 22 + 0.67D_u$

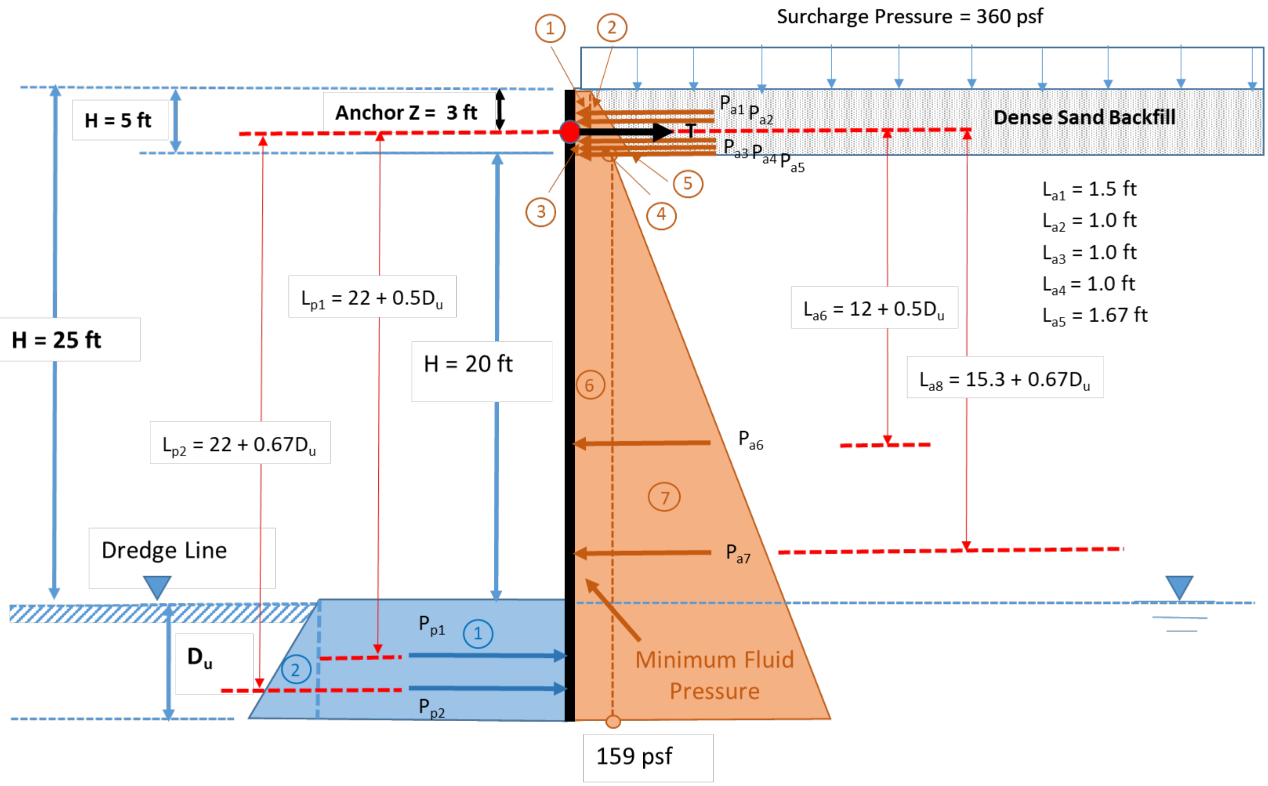


Figure 5-19 Case 5 Lateral active and passive forces acting on the wall.

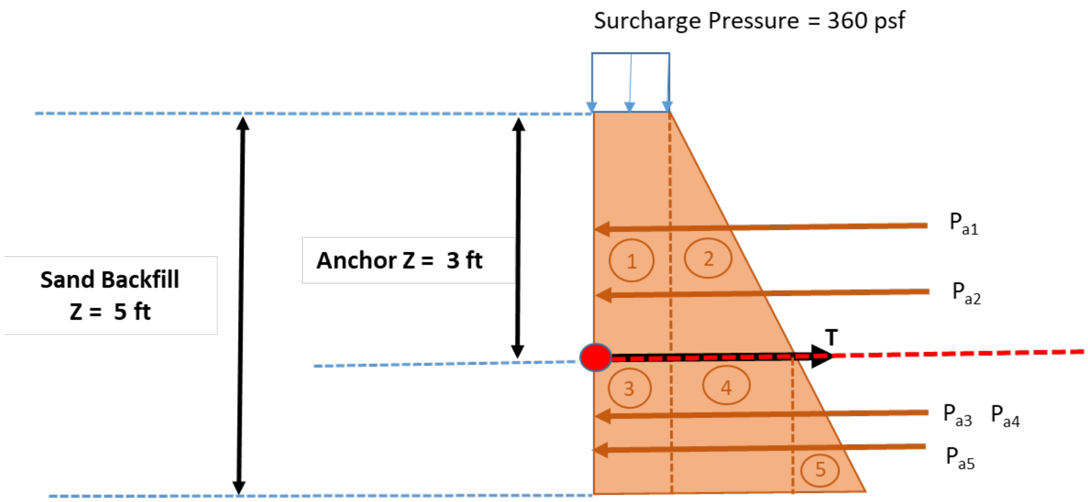


Figure 5-20 Case 5 Active pressures and forces acting on the sheet pile wall from the backfill.

Case 5, Step 5: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0

Note: Moments at the anchor rod are assumed to be zero. Therefore, to determine the sheet pile's embedment depth, the summation of moments are taken at the anchor location as shown in Figure 5-19.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS$$

where

$\Sigma M_{restoring} = P_{p1}L_{a3} + P_{a1}L_{p1} + P_{p2}L_{p2}$

$\Sigma M_{disturbing} = P_{a3}L_{a3} + P_{a4}L_{a4} + P_{a5}L_{a5} + P_{a6}L_{a6}$

FOS = 1.0 or $\Sigma M_{restoring} = \Sigma M_{disturbing}$

The depth, D_u, is determined by placing the equations into an EXCEL sheet as shown in Table 5-13. The spreadsheet is set up to input a depth, D_u, which then calculates the *FOS*. For a *FOS* = 1.0, the embedment depth D_u is 2.52 feet.

D_u = 2.52 ft

$D_{u,SupportIT}$ = 2.54 ft

Table 5-13 Case 5 Embedment depth, D_u, for *FOS* = 1.0.

Case 5 Calculation of Sheet Pile Depth, D_u			
Depth of Sheet Pile, D_u (ft)		2.52	
FOS		1.00	
Restoring Moment:		Disturbing Moment:	
P_{a1}	237.9	P_{a3}	158.4
L_{a1}	1.5	L_{a3}	1.0
P_{a2}	114.6	P_{a4}	152.9
L_{a2}	1.0	L_{a4}	1.0
P_{P1}	7,547.4	P_{a5}	51.0
L_{P1}	23.3	L_{a5}	1.3
P_{p2}	374.6	P_{a6}	3582.3
L_{p2}	23.7	L_{a6}	13.3
M_r	184,880	P_{a7}	8066.1
		L_{a7}	17.0
		M_d	184,880

Case 5, Step 6: Determine Anchor Tension

The anchor load, T, is determined as follows:

T = Active forces – Passive Forces = $(P_{a1} + P_{a2} + P_{a3} + P_{a4}\ P_{a5} + P_{a6} + P_{a7}) – (P_{p1} + P_{p2})$

$T = (237.9 + 114.6 + 158.4 + 152.9 + 51.0 + 3{,}582.3 + 8{,}066.1) – (7{,}547.4 + 374.7)$

T = 4,441 lbs/ft (per foot of sheet pile wall)

$T_{SupportIT}$ = 4,503 lbs/ft

Case 5, Step 7: Determine the Maximum Moment in the Sheet Pile

Location of zero-shear:

The maximum bending moment in the sheet pile wall is located at a depth D_s, where the shear stresses. The depth D_s is shown in Figure 5-21. An enlarged view near the anchor location is shown in Figure 5-20. The zero shear force location is calculated by summing the forces acting on the sheet pile wall below the anchor to where they equal the shear force acting on the anchor. This calculation excludes the shear forces acting above the anchor.

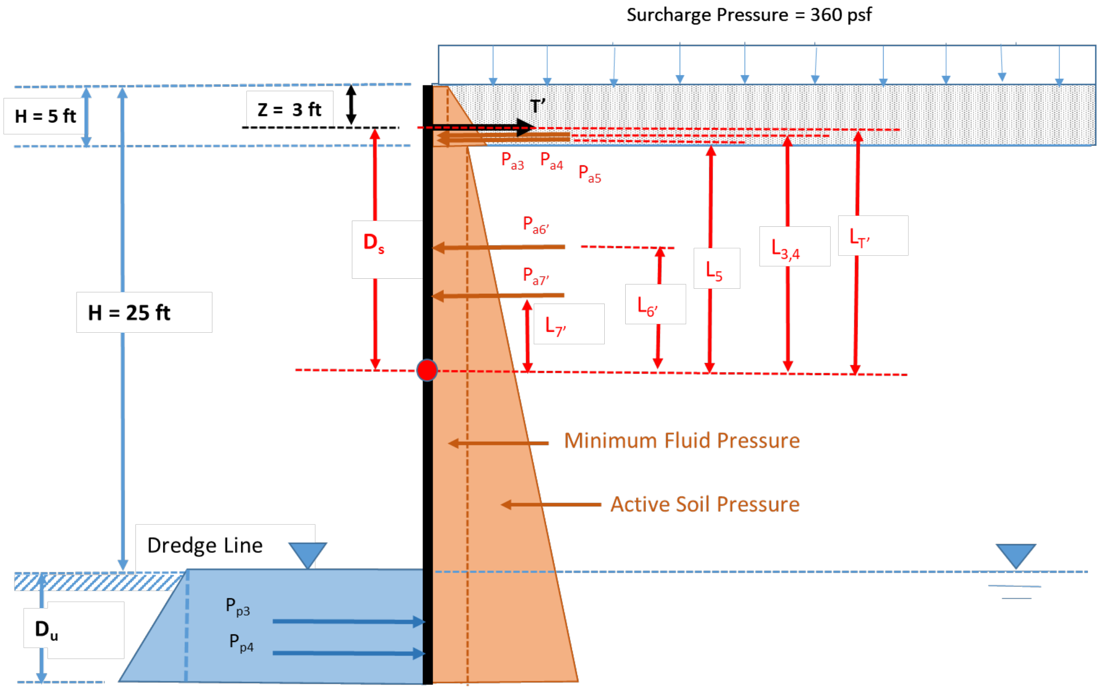


Figure 5-21 Case 5 Location of zero-shear stress in the pile.

The shear force, τ', acting on the anchor is recalculated as follows:

$$\tau' = P_{a3} + P_{a4}\ P_{a5} + P_{a6} + P_{a7} - (P_{p1} + P_{p2})$$

$$\tau' = [(158.4 + 152.9 + 51.0 + 3{,}582.3 + 8{,}066.3) - (7{,}548.0 + 374.7)] = \underline{4{,}088 \text{ lbs}}$$

The forces (lbs/ft) and moment arms (ft), in terms of D_s, are as follows:

$\tau' = 4{,}088$
$P_{a3} = 158.4$
$P_{a4} = 152.9$
$P_{a5} = 51.0$
$P_{a6'} = (5)(31.8)(D_s - 2) = 159D_s - 318.0$
$P_{a7'} = (0.5)(31.8)(D_s - 2)^2 = 15.91D_s^2 - 63.64D_s + 63.64$

$$\tau' = 4{,}088 = 158.2 + 152.9 + 51.0 + 159D_s - 318.0 + 15.91D_s^2 - 63.64D_s + 63.64$$

Therefore $15.91D_s^2 + 95.36D_s - 3{,}980 = 0$

Solving for D_s:

D_s = $\underline{13.10 \text{ ft}}$

$D_{s,\ SupportIT}$ = $\underline{13.19 \text{ ft}}$

Maximum Bending Moment Location: 13.1 + 3 = 16.1 ft

Calculate the maximum bending moment at 16.1 ft

Forces and moment arms in terms of D_s (ft)

$T' = 4,088$
$P_{a3} = 158.4$
$P_{a4} = 152.9$
$P_{a5} = 51.0$
$P_{a6'} = (5)(31.8)(D_s - 2) = 159D_s - 318 = 159(13.1) - 318.0 = 1,764.9$
$P_{a7'} = 15.91D_s^2 - 63.64D_s + 63.64 = 15.91(13.1)^2 - 63.64(13.1) + 63.6 = 1,960.1$

$L_{sT'} = D_s = 13.1$
$L_{s3} = (D_s - 1) = 12.1$
$L_{s4} = (D_s - 1) = 12.1$
$L_{s5} = (D_s - 1.67) = 11.77$
$L_{6'} = (0.5)(D_s - 2) = (0.5)(13.1 - 2) = 5.55$
$L_{7'} = (0.33)(D_s - 2) = (0.33)(13.1 - 2) = 3.70$

The maximum moment is calculated by taking the summation of moments at D_s (similar to Case 4 as shown in Figure 5-16) as follows:

Moment: $M = [T' L_{sT'}] - [P_{a3} L_{s3} + P_{a4} L_{s4} + P_{a5} L_{s5} + P_{a6'} L_{6'} + P_{a7'} L_{7'}]$

Moment: $M = [4,088(13.1)] - [158.4(12.1) + 152.9(12.1) + 51.0(11.77) + 1,764.9(5.55) + 1,959.0(3.7)]$

M = + 32,142 ft-lbs/ft

$M_{SupportIT}$ = +32,461 ft-lbs/ft

Case 5, Step 8: Sheet Pile Selection

The maximum moment was estimated to be 32,142 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 25 ksi, the required section modulus is determined as follows

Required section modulus = M/f_s = [32,142 ft-lbs/ft x 12 in/ft]/25,000 psi = 15.4 in^3/ft

The section modulus of US Steel's PZ22 is 18.40 in^3/ft., therefore a PZ22 would meet the section modulus requirement.

According to the SupportIT solution provided in Appendix B.5, it was determined that a PZ22 sheet pile would have 1.3 inches of deflection, which meets the minimum deflection limit.

Case 5, Step 9: Calculate Sheet Pile Total Length with *FOS* = 1.5

The Gross Pressure CP2 method is used to determine the embedment depth D_f. The depth D_f is then recalculated, as shown in Table 5-14 with a FOS = 1.5.

Table 5-14 Case 5 Pile embedment depth, D_f, for a *FOS* = 1.5.

Case 5 Calculation of Sheet Pile Depth, D_f			
Depth of Sheet Pile, D_f (ft)		4.30	
FOS		1.50	
Restoring Moment:		Disturbing Moment:	
P_{a1}	237.9	P_{a3}	158.4
L_{a1}	1.5	L_{a3}	1.0
P_{a2}	114.6	P_{a4}	152.9
L_{a2}	1.0	L_{a4}	1.0
P_{P1}	12,900.0	P_{a5}	51.0
L_{P1}	24.2	L_{a5}	1.3
P_{p2}	1,094.4	P_{a6}	3866.1
L_{p2}	24.9	L_{a6}	14.15
M_r	339,237	P_{a7}	9395.1
		L_{a7}	18.2
		M_d	225,897

D_f = <u>4.30 ft</u>

$D_{f,SupportIT}$ = <u>4.30 ft</u>

Comparison to SupportIT software results (Appendix B.5):

Table 5-15 Case 5 Comparison of hand calculations to SupportIT calculations.

	SupportIT (Total pile length, ft)	Hand Calculations (Total pile length, ft)
Maximum soil pressure at dredge line, (psf/ft)	797	797
Anchor Load, (lbs/ft)	4,503	4,441
Zero Shear location along sheet pile, (ft)	13.19	13.10
Maximum Moment, (ft-lbs/ft), FOS = 1.0	32,461	32,142
Sheet Pile Embedment, FOS = 1.00, D_u (ft)	2.52	2.54
USS 20% FOS Embedment Length, D_f (ft)	3.0 (28)	3.0 (28)
USS 40% FOS Embedment Length, D_f (ft)	3.5 (29)	3.6 (29)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	4.3 (30)	4.3 (30)

5.2.6 Case 6 – Braced Cofferdam TERS in Soft and Firm Cohesive Soils

Case 6, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall

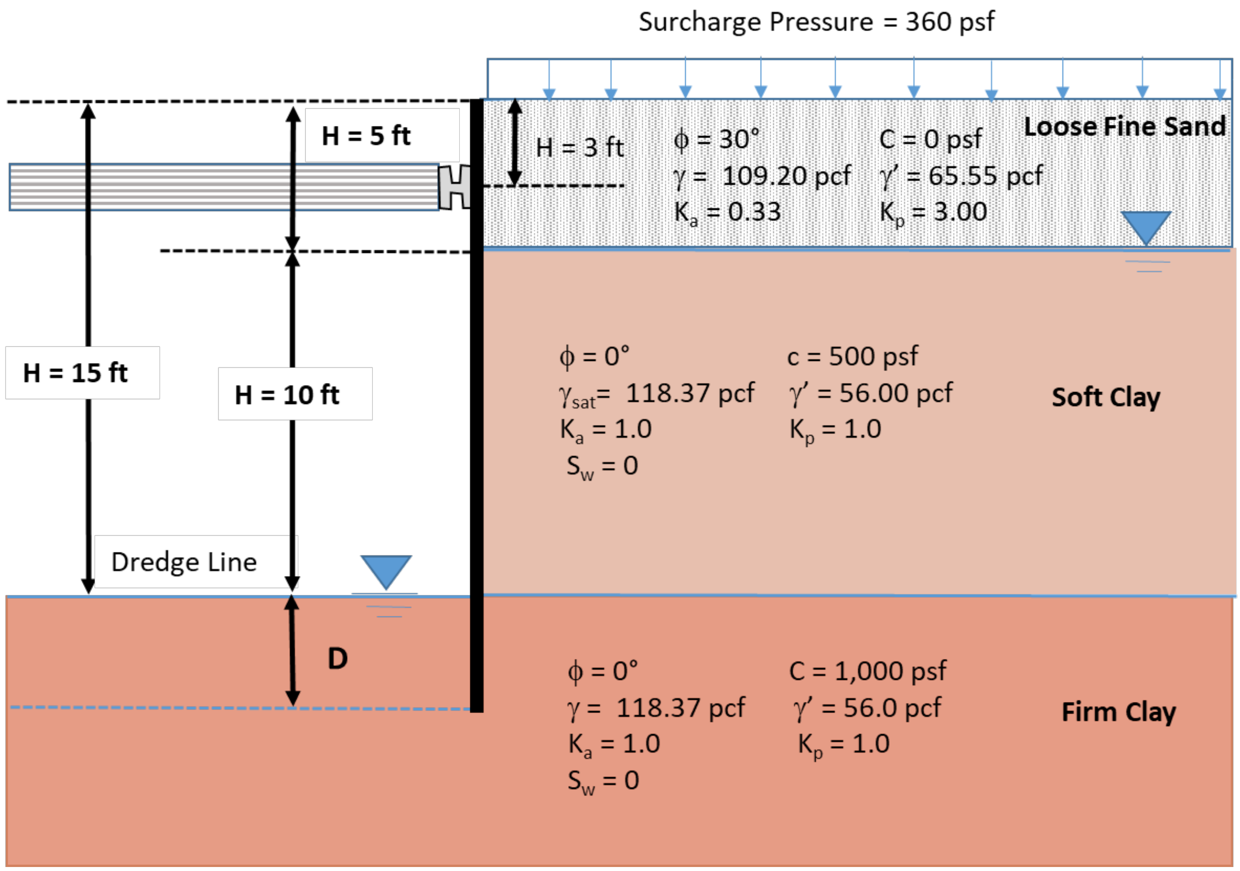


Figure 5-22 Case 6 Braced TERS in soft and firm clay.

Case 6, Step 2: Determination of Active and Passive Pressure Coefficients

Sand:

$$K_a = tan^2\left(45^\circ - \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ - \frac{30^\circ}{2}\right) = \mathbf{0.33}$$

$$K_p = tan^2\left(45^\circ + \frac{\emptyset'}{2}\right) = tan^2\left(45^\circ + \frac{30^\circ}{2}\right) = \mathbf{3.00}$$

Firm clay and soft clay:

$$\mathbf{K_a = K_p = 1.0}$$

Case 6, Step 3: Calculate the Critical Height for Each Clay Layer

Soft Clay Cohesion: c = 500 psf

Firm Clay Cohesion: c = 1000 psf

Surcharge: $q_{surcharge}$ = 360 + (5)(109.2) = 906 psf

Soft Clay Layer, Critical Height: $H_c = \frac{2c - q_{surcharge}}{\gamma} = \frac{2(500) - 906}{118.4} \approx\ 0.79\ feet$

Firm Clay Layer, Critical Height:

$$H_c = \frac{2c - q_{surcharge}}{\gamma} = \frac{2(1{,}000) - [360 + (5)(109.2) + (10)(118.37)]}{118.4} \approx\ -0.76\ feet$$

Soft Clay Compressive Strength: S_u = 2c = 2(500) = 1,000 psf

Firm Clay Compressive Strength: S_u = 2c = 2(1,500) = 3,000 psf

Case 6, Step 4: Calculate Active and Passive Pressures Acting on Sheet Pile Wall:

The following assumptions are used in Case 6:

1. The top layer, "*fine loose sand,*" applies an active lateral pressure to the wall in the upper five feet of the wall. This pressure includes the active pressure from the 360 psf surcharge.
2. A "minimum fluid pressure," MFP = 31.8 psf, is applied to the sheet pile wall from the top of the wall downward. For example, the MFP at a depth of five feet is 5 ft x 31.8 pcf = 159.0 psf. The primary purpose of the MFP is to add a factor of safety to the calculation by applying fluid pressure to the wall when the clay is self-supporting.
3. The unsupported height of the soft clay layer is 0.79 ft, depth of 5.79 ft. Note that this estimate includes the weigh of the loose sand layer and the surcharge of 360 psf.
4. At a depth of 5.79 ft, the clay's active soil pressure (K_a = 1) starts to act on the wall. However, between 5.79 and 7.88 ft, the "minimum fluid pressure" (MFP) exceeds the active soil pressure. SupportIT takes the greater of the calculated active pressures or the MFP. Therefore, the MFP is applied between 5.0 ft and 7.88 ft.
5. Between 7.88 and 15 feet, the active clay pressure is applied to the wall.
6. The firm clay at a depth of 15 feet has a critical height of – 0.76 ft. This means that the active pressure from the firm clay will start at a depth of 15 – 0.76 = 14.24 ft.
7. At the dredge line, the passive pressure is equal to the compressive strength of the clay.
8. At the dredge line, the passive resistance of the clay (K_p = 1) is applied.
9. At the dredge line, the active water pressure exceeds the clay's active pressure and is applied to the wall.

The assumptions used in the SupportIT software are discussed below for Case 6.

At some depth along the sheet pile, the MFP will equal the active pressure. The following calculation is used to determine the depth where the MFP equals the clay's active pressure.

MFP = 31.8(z)

Clay active pressure = (z – 5.79)(118.37) = 118.37(z) – 685.36

31.8(z) = 118.37(z) – 685.36

<u>z = 7.92 ft</u>

<u>Active Pressures:</u>

Surcharge Pressure in sand backfill: $\boldsymbol{\sigma_{a1}} = K_a\sigma_v$ = (0.33)(360 psf) = <u>**118.8 psf**</u>

Sand backfill: $\sigma_{a2} = K_a\sigma_v$ = (0.33)(5 ft)(109.2 pcf) = <u>180.2</u> psf

$\boldsymbol{\sigma_{a5}} = \sigma_{a1} + \sigma_{a2}$ = 118.8 + 180.2 = <u>**299 psf**</u>

Minimum Fluid Pressure: $\boldsymbol{\sigma_{a5'}} = K_a\gamma H$ = (1.0)(31.8)(5) = <u>**159.0 psf**</u>

$\boldsymbol{\sigma_{a7.92'}} = K_a\gamma H$ = (1)(31.8)(7.92) = <u>**251.8 psf**</u>

Active Soil, Soft Clay Layer: *The active pressure starts at a depth of 5.79 ft,*

That is, $\sigma_{a5.79'}$ = 0 psf

$\sigma_{a7.92'} = K_a\gamma H$ = (1.0)(118.37)(7.92-5.79) = <u>252.1 psf</u>

$\boldsymbol{\sigma_{a15'}} = K_a\gamma H$ = (1.0)(118.37)(15.0 - 5.79) = <u>**1,090.2 psf**</u>

Active Soil, Firm Clay Layer: *Note: The critical height of the firm clay is -0.76 ft at a depth of 15 feet. That is, SupportIT will use the active pressure from a depth of 15-0.76 = 14.24 ft.*

$\sigma_{a15.08'} = K_a\gamma H$ = (1.0)(118.37)(15.08 – 14.24) = <u>99.4 psf</u>

$\sigma_{a17.41'} = K_a\gamma H$ = (1.0)(118.37)(17.41 – 14.24) = <u>375.2 psf</u>

$\sigma_{a20'} = K_a\gamma H$ = (1.0)(118.37)(20.0 – 14.24) = <u>681.8 psf</u>

Active Water Pressure: $\boldsymbol{\sigma_{a15'}} = K_a\gamma H$ = (10)(62.4) = <u>**624.0 psf** > 99.4 psf (active)</u>

$\boldsymbol{\sigma_{a20'}} = K_a\gamma H$ = (15)(62.4) = <u>**936 psf** > 681.8 psf (active)</u>

Note: The water pressure exceeds the clay active pressure, therefore, the higher water pressure governs in the firm clay layer between 15-20 ft.

<u>Passive Soil Pressures to a depth of 20 feet:</u>

Lateral Pressure below Groundwater: $\boldsymbol{\sigma_{p15}}$ = 2c = 2(1,000) = **-<u>2,000 psf</u>** (constant with depth)

$\sigma_{p15.08}$ = 2000 + (15.08 – 15.00)(118.37) = -<u>2,009.5 psf</u>

$\sigma_{p17.41}$ = 2000 + (17.41 – 15.00)(118.37) = -<u>2,009.5 psf</u>

$\boldsymbol{\sigma_{p20}}$ = 2000 + (20 – 15.00)(118.37) = <u>**2,591.9 psf**</u>

Figure 5-23 illustrates the active and passive pressures acting on a sheet pile to a depth of 20 feet corresponding to the bolded results above. Table 5-16 provides the pressure values used by the SupportIT software to determine the forces acting on the sheet pile wall.

Table 5-16 Case 6 SupportIT pressures acting on the sheet piling.

Soil Depth (ft)	Active Soil Pressure γ = 118.37 pcf	Passive Soil Pressure σ_{p1} = 2c	Minimum Fluid Pressure γ = 31.8 pcf	Active Water Pressure γ = 62.4 pcf	Net Pressure Acting on Sheet Pile
0	118.8	-	-	-	118.8
3.27	237.0	-	-	-	237.0
5.00	299.0	-	-	-	299.0
5.03	0	-	159.8	1.9	159.8
5.73	0	-	182.4	45.6	182.4
5.79	0	-	184.3	49.3	184.3
6.66	106.53	-	212.0	103.6	212.0
7.83	241.5	-	249.4	176.6	249.4
7.95	255.4	-	253.1	184.1	255.4
10.17	518.2	-	-	322.6	518.2
15.0	1090.2	0.0	-	624.0	1,090.2
15.08	99.4	-2,009.5	-	629.0	-1,380.5
17.41	375.2	-2,285.8	-	774.4	-1,509.9
20.0	681.8	-2,591.9	-	936.0	-

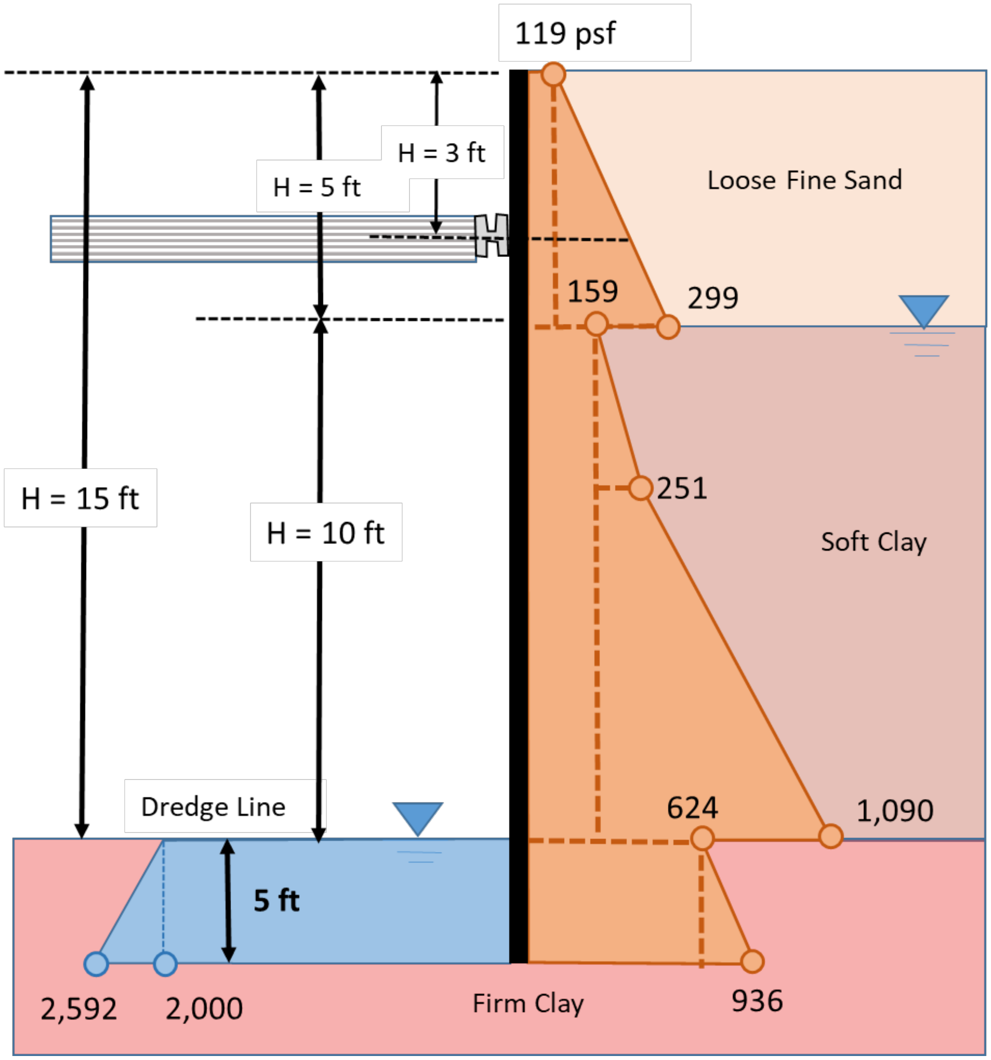


Figure 5-23 Case 6 Gross active and passive pressures are acting on a sheet pile at 20 feet.

<u>Case 6, Step 5: Calculate Resultant Forces Acting on Sheet Pile Embedment Depth</u>

The location of the brace is at a depth three feet below the top of the wall, as shown in Figure 5-22. To estimate the wall embedment length, D_u, the summation of moments, is taken at the location of the brace. The calculations for a brace are essentially the same as for the anchor in Case 5. It is also assumed that the brace location acts like a plastic hinge with no capacity to resist moments, i.e., $\Sigma M = 0$. The lateral active and passive forces and their locations based on the anchor position are shown in Figure 5-23. It should be noted that by taking the summation of moments at the anchor does not guarantee a point of sheet pile fixture.

The equations for the lateral earth pressures acting on the wall are as follows.

Active Forces (lbs) and Locations (ft) (from brace location):

$P_{a1} = 0.33(360)(3)(1) = 356.4$

$P_{a2} = 0.33(0.5)(3)(109.2)(3)(1) = 162.2$

$P_{a3} = 0.33(360)(2)(1) = 237.6$

$P_{a4} = 0.33(3)(109.2)(2)(1) = 216.2$

$P_{a5} = 0.33(0.5)(2)(109.2)(2)(1) = 72.1$

$P_{a6} = 1.0(5)(31.8)(10)(1) = 1{,}590.0$

$P_{a7} = 1.0(0.5)(2.92)(31.8)(2.92)(1) = 135.6$

$P_{a8} = 1.0(2.92)(31.8)(7.08)(1) = 657.4$

$P_{a9} = 1.0(0.5)(7.08)(118.37)(7.08)(1) = 2{,}966.7$

$P_{a10} = 1.0(10)(62.4)D(1) = 624D_u$

$P_{a11} = 1.0(0.5)(D)(62.4)D\ (1) = 31.2D_u^2$

$L_{a1} = 0.5(3') = 1.5'$

$L_{a2} = 1.0$

$L_{a3} = 1.0$

$L_{a4} = 1.0$

$L_{a5} = (2/3)(2') = 1.33$

$L_{a6} = 5 + 2 = 7$

$L_{a7} = (2/3)(2.92) + 2 = 3.95$

$L_{a8} = (7.08)/2 + 2 + 2.92 = 8.46$

$L_{a9} = (2/3)(7.08) + 2 + 2.92 = 9.63$

$L_{a10} = 12 + 0.5D_u$

$L_{a11} = 12 + 0.67D_u$

Passive Force (lbs) and Locations (from brace location):

$P_{p1} = 1.0(2000)D = 2{,}000D_u$

$P_{p2} = 1.0(0.5)(118.37)D^2 = 59.1D_u^2$

$L_{p1} = 12 + 0.5D_u$

$L_{p2} = 12 + 0.67D_u$

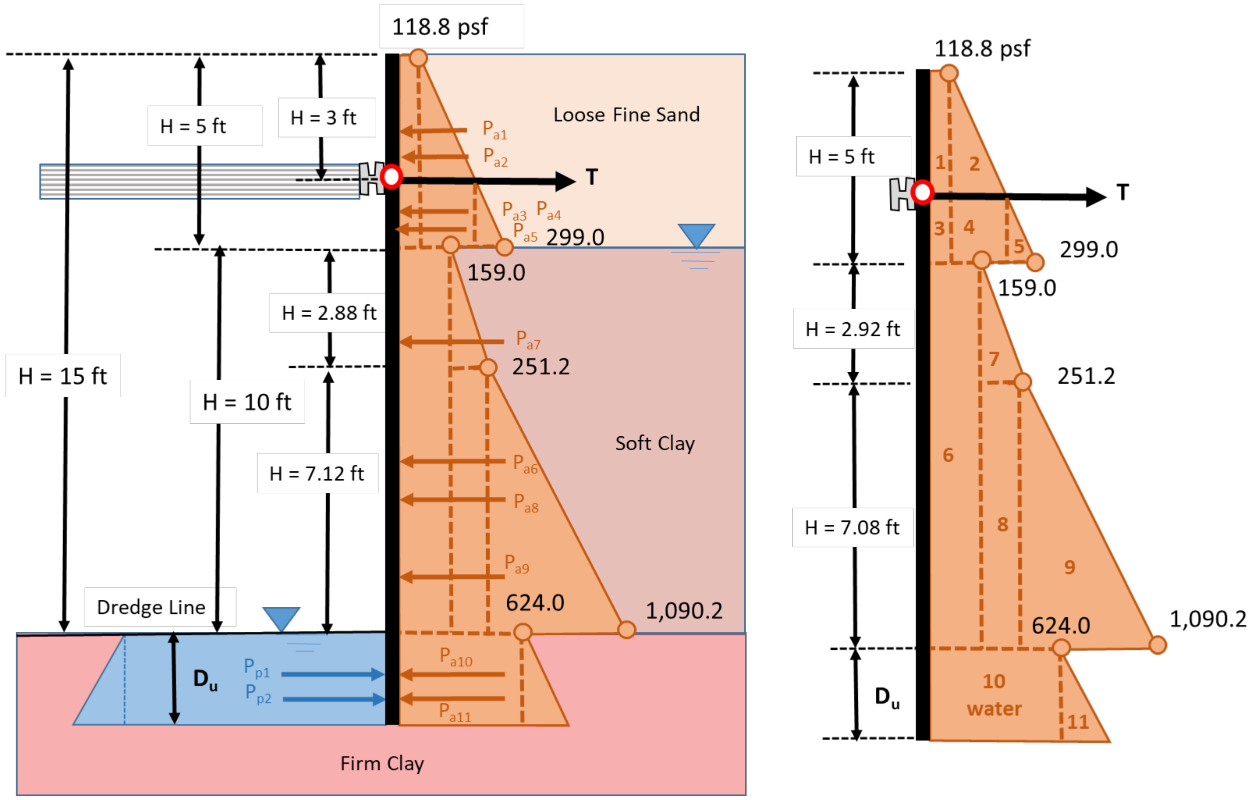


Figure 5-24 Case 6 Lateral and passive forces acting on the wall.

<u>Case 6, Step 6: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0</u>

Moments at the braced location are assumed to be zero. Therefore, to determine the embedment depth, D_u, the summation of moments, is taken at the braced location, as shown in Figure 5-24.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS$$

where

$$\Sigma M_{Restoring} = P_{a1}L_{a1} + P_{a2}L_{a2} + P_{p1}L_{p1} + P_{p2}L_{p2}$$

$$\Sigma M_{Disturbing} = P_{a3}L_{a3} + P_{a4}L_{a4} + P_{a5}L_{a5} + P_{a6}L_{a6} + P_{a7}L_{a7} + P_{a8}L_{a8} + P_{a9}L_{a9} + P_{a10}L_{a10} + P_{a11}L_{a11}$$

FOS = 1.0 or $\Sigma M_{Restoring} = \Sigma M_{Disturbing}$

D_u can be determined by placing the equations into an EXCEL sheet as shown in Table 5-17. The spreadsheet is set up to input a depth, D_u, which then calculates the *FOS*. For a *FOS* = 1.0, the embedment depth D_u is 2.43 feet.

D_u = <u>2.42 ft</u>

$D_{u,SupportIT}$ = <u>2.42 ft</u>

Table 5-17 Case 6 Embedment depth, D, for *FOS* = 1.0.

Calculation of Sheet Pile Depth, D_u (FOS = 1.0)			
Depth of Sheet Pile, D_u (ft)	2.42		
FOS	1.00		
Restoring Moment		Disturbing Moment:	
P_{a1}	356.4	P_{a3}	237.6
L_{a1}	1.5	L_{a3}	1.0
P_{a2}	162.2	P_{a4}	216.2
L_{a2}	1.0	L_{a4}	1.0
P_{P1}	4,838.8	P_{a5}	72.1
L_{P1}	13.2	L_{a5}	1.3
P_{P2}	345.9	P_{a6}	1590.0
L_{P2}	13.6	L_{a6}	7.0
M_r	68,794	P_{a7}	135.6
		L_{a7}	3.9
		P_{a8}	657.4
		L_{a8}	8.4
		P_{a9}	2966.7
		L_{a9}	9.6
		P_{a10}	1509.7
		L_{a10}	13.2
		P_{a11}	182.6
		L_{a11}	13.6
		M_d	68,759

<u>Case 6, Step 7: Determine Brace Load (lbs/ft)</u>

The anchor load is determined as follows:

T = Active forces – Passive Forces

$T = [(P_{a1} + P_{a2} + P_{a3} + P_{a4} + P_{a5} + P_{a6} + P_{a7} + P_{a8} + P_{a9} + P_{a10} + P_{a11}) - (P_{p1} + P_{p2})]$

$T = [(356.4 + 162.2 + 237.6 + 216.2 + 72.1 + 1{,}590.0 + 135.6 + 657.4 + 2{,}966.7 + 1{,}509.7 + 182.6) - (4{,}838.8 + 345.9)]$

T = <u>2,901 lbs/ft</u>

$T_{SupportIT}$ = <u>2,948 lbs/ft</u>

Case 6, Step 8: Determine the Maximum Moment in the Sheet Pile

Location of zero-shear load:

The maximum bending moment in the sheet pile wall is located at a depth where the shear stresses are zero at D_s as shown in Figure 5-25. The zero shear force location is calculated by summing the forces acting on the sheet pile wall below the anchor to where they are equal to the shear force acting on anchor. This calculation excludes the shear forces acting above the anchor and below the location of zero shear.

The shear force acting on the anchor is recalculated as follows:

$T' = \Sigma P_a = [P_{a3} + P_{a4} + P_{a5} + P_{a6} + P_{a7} + P_{a8} + P_{a9}] - [P_{p1} + P_{p2}]$

$T' = [237.6 + 216.2 + 72.1 + 1{,}590.0 + 135.6 + 657.4 + 2{,}966.7 + 1{,}509.7 + 182.6] - [4{,}838.8 + 345.9]$

$\underline{T' = 2{,}383.2}$

Forces in terms of D_s:

$P_{a3} = 237.6$
$P_{a4} = 216.2$
$P_{a5} = 72.1$
$P_{a6'} = (1)(5)(31.8)(D_s - 2)(1) = 159.0D_s - 318.0$
$P_{a7} = 135.6$
$P_{a8'} = (2.92)(31.8)(D_s - 4.92)(1) = 92.86D_s - 456.85$
$P_{a9'} = (1)(0.5)(D_s - 4.92)(118.37)(D_s - 4.92) = 59.1D_s^2 - 582.4D_s + 1{,}432.99$

$T' = 2{,}383.2 = 237.6 + 216.2 + 72.1 + 159.15D_s - 318.0 + 135.6 + 92.86D_s - 456.85 + 59.15D_s^2 - 582.4D_s + 1{,}432.99$

$59.15D_s^2 - 330.4D_s - 1{,}063.6 = 0$

$\underline{D_s = 7.87 \text{ ft}}$
$\underline{D_{SupportIT} = 7.95 \text{ ft}}$

Zero-shear Depth: z = $\underline{3 + 7.87 = 10.87 \text{ ft}}$

Zero-shear Depth$_{SupportIT}$: z = $\underline{10.94 \text{ ft}}$

Therefore, the maximum moment will be located at a distance of 10.87 ft from the top of the sheet pile. To *calculate the maximum moment at point D_s*, the forces and moment arms are first calculated as follows:

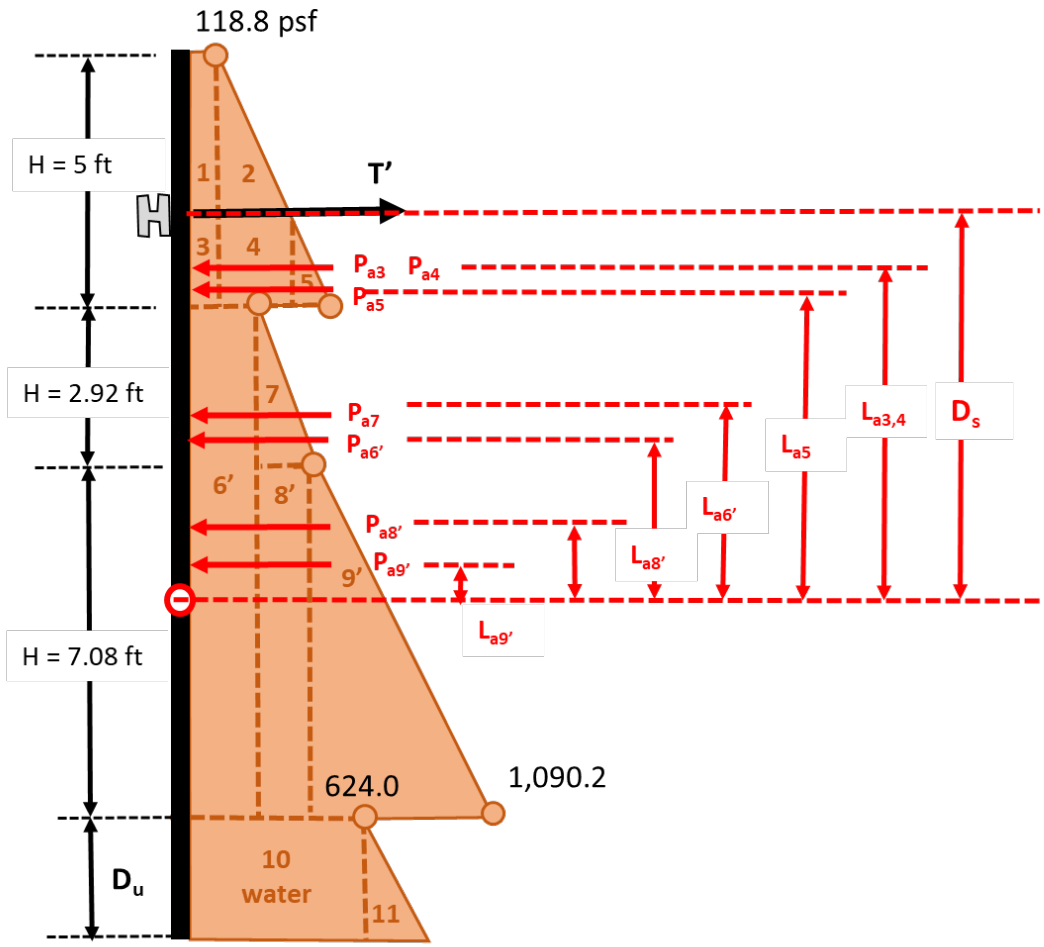


Figure 5-25 Determination of zero-shear location on sheet pile wall.

<u>Forces (lbs/ft)</u>

T' = 2,383.2

P_{a3} = 237.6
P_{a4} = 216.2
P_{a5} = 72.1
$P_{a6'}$ = 159.0(7.92 - 2) = 941.3
P_{a7} = 135.6
$P_{a8'} = 91.68D_s - 447.35 = 91.68(7.87) - 447.35 = 274.2$
$P_{a9'} = 59.15D_s^2 - 577.3D_s + 1{,}408.36 = 59.15(7.87)^2 - 577.3(7.87) + 1{,}408.36 = 528.6$

Moment Arms

$L_{ts} = D_s = 7.87$
$L_{a3'} = D_s - 1 = 7.87 – 1 = 6.87$
$L_{a4'} = D_s - 1 = 7.87 – 1 = 6.87$
$L_{a5'} = D_s – (0.67)(2)= 7.87 - (0.67)(2) = 6.53$
$L_{a6'} = 0.5(D_s – 2) = 0.5(7.87 – 2) = 2.94$
$L_{a7'} = D_s - 2 - (0.67)(2.88) = 7.87 – 3.92 = 3.94$
$L_{a8'} = 0.5(D_s – 2 – 2.88) = 0.5(7.87 – 2 - 2.88) = 1.50$
$L_{a9'} = 0.33(D_s – 2 – 2.88) = 0.33(7.87 – 4.88) = 1.00$
The maximum moment, M, is calculated as follows:

Moment: $M = [T\ L_{sT}] - [P_{a3}\ L_{a3'} + P_{a4}\ L_{a4} + P_{a5}\ L_{a5} + P_{a6'}\ L_{a6'} + P_{a7}\ L_{a7'} + P_{a8'}\ L_{a8'} + P_{a9'}\ L_{a9'}]$

Moment: M = [2,383.2(7.87)] - [237.6(6.87) + 216.2(6.87) + 72.1(6.53) + 941.3(2.94) +

(135.6)(3.94) + (274.2)(1.5) + (528.6)(1.0)] = 11,030

M = + 10,926 ft-lbs/ft

$M_{SupportIT}$ = +10,650 lbs/ft

Case 6, Step 9: Sheet Pile Selection

The maximum moment was estimated to be 11,030 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 25 ksi, the required section modulus is determined as follows

Required section modulus = M/f_s = [11,030 ft-lbs/ft x 12 in/ft] / 25,000 psi = 5.3 in^3/ft

The section modulus of US Steel's PZ22 is 18.40 in^3/ft. Therefore a PZ22 would meet the section modulus requirement.

According to the SupportIT solution provided in Appendix B.6, it was determined that a PZ22 sheet pile would have 1.3 inches of deflection, which meets the minimum deflection limit.

Case 6, Step 10: Calculate Sheet Pile Total Length with a Factor of Safety

The Gross Pressure CP2 method with a FOS = 1.5 is used to determine the embedment depth D_f. The depth D_f is then recalculated as shown in Table 5-18.

Table 5-18 Case 6 Embedment depth, D_f, for *FOS* = 1.5.

Calculation of Sheet Pile Depth, D_f			
Depth of Sheet Pile, D_f (ft)		4.38	
FOS		1.50	
Restoring Moment		Disturbing Moment:	
P_{a1}	356.4	P_{a3}	237.6
L_{a1}	1.5	L_{a3}	1.0
P_{a2}	162.2	P_{a4}	216.2
L_{a1}	1.0	L_{a4}	1.0
P_{P1}	8,760.0	P_{a5}	72.1
L_{P1}	14.2	L_{a5}	1.3
P_{P2}	1,133.8	P_{a6}	1590.0
L_{P2}	14.9	L_{a6}	7.0
M_r	141,399	P_{a7}	132.0
		L_{a7}	3.9
		P_{a8}	657.4
		L_{a8}	8.4
		P_{a9}	2966.7
		L_{a9}	9.6
		P_{a10}	2733.1
		L_{a10}	14.2
		P_{a11}	598.6
		L_{a11}	14.9
		M_d	94,037

Case 6 is based on the *free earth method*, not the simplified method used in Cases 1-3. Therefore, no adjustment to the embedment depth, D_f, is required after the application of the *FOS*.

D_f = 4.38 ft

$D_{fSupportIT}$ = 4.38 ft

Total Sheet Pile Length = H + D_f = 15 + 4.37 = **19.4ft**

Comparison of SupportIT and Hand Calculations:

Table 5-19 Case 6 Comparison of hand calculations to SupportIT calculations.

	SupportIT (Total pile length, ft)	Hand Calculations (Total pile length, ft)
Maximum soil pressure at dredge line, (psf/ft)	1,089.7	1,090.2
Anchor Load, (lbs/ft)	2,948	2,901
Zero Shear location along sheet pile, (ft)	10.94	10.87
Maximum Moment, (ft-lbs/ft), FOS = 1.0	10,650	10,674
Sheet Pile Embedment, FOS = 1.00, D_u (ft)	2.42	2.42
USS 20% FOS Embedment Length, D_f (ft)	2.9 (18)	32.9 (18)
USS 40% FOS Embedment Length, D_f (ft)	3.3 (18.5)	3.4 (18.5)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	4.38 (19)	4.37 (19)

5.2.7 Case 7 – Braced Wall TERS in Cohesionless Soil

Braced walls are constructed with anchors or struts to increase the sheet piling's stability. The two main failure modes braced walls are (1) excessive anchor or strut loads and (2) bottom heave. The interaction of the wall and soil is highly complex and, therefore, difficult to analyze without simplifying assumptions. Also, the flexibility of the wall plays a significant role in the transfer of earth pressures to the braced wall struts. When using conventional limit equilibrium methods for design, there is a need for considerable engineering judgment.

The two design elements for braced walls covered are (1) the wall's equilibrium stability, which includes embedment depth (toe depth), and determining the stresses and bending moments in the sheet pile and (2) determining the anchor or strut loads. It is important to note that these two design elements require separate steps. In braced excavations, the design of the piling must be checked at each stage of construction, which is known as "staged construction."

A common design method for the stability of a braced wall is the "*hinge*" method that assumes a "*pin-joint*" at each of the strut locations. The sections between the supports are then considered to be "simply-supported beams," while the lowest section is analyzed as a "cantilevered wall." These design assumptions are shown in Figure 5-26. The software programs SupportIT and SPW911 can utilize this concept.

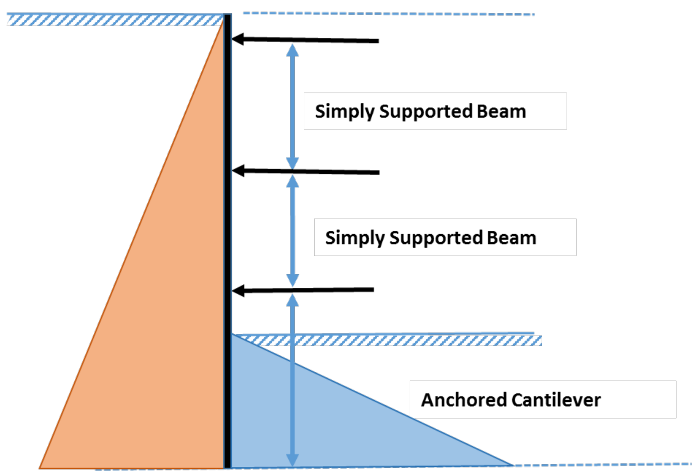


Figure 5-26 Stability analysis method used for braced walls.

Case 7 illustrates the design of a 30-foot deep excavation in cohesionless soils with the groundwater elevation at 30 feet, i.e., at the dredge line. The same soil parameters used in Cases 1 and 4 are used in Case 7. Case 4 (H = 20 ft cantilever wall with one anchor) determined that a PZ22 sheet piling can handle the maximum bending stresses using one anchor. Case 7, therefore, will be designed with PZ22 sheet piling but will require three struts. The US Steel Pile Manual recommends a minimum eight-foot spacing between struts for constructability. <u>The analysis and design of the struts, however, will also require a "staged construction" design</u>. That is, for each construction stage, the stability of the sheet piling must be analyzed so that the piling's bending moments and deflection limits are not exceeded. To optimize the strut loads, spacing, and sheet pile section, an iterative process would be required.

Case 7 is analyzed in four stages. These stages are listed below and illustrated in Figure 5-28.

Stage 1. Install sheet piles to final depth; excavate to 8 ft.
Stage 2. Install the first strut at 3 ft and excavate to 18 ft.
Stage 3. Install the second strut at 13 ft and excavate to 25 ft.
Stage 4. Install the third strut at 22 ft and excavate to the final depth at 30 feet.

The following sections discuss the design for each stage 1 through 4.

Stage 1: Install sheet piles, excavate eight feet:

Stage 1 requires that the sheet piling be driven to the final designed depth, which is determined in Case 4 at 50 feet. Stage 1 is analyzed as a cantilever wall similar to Case 1, which had a 10-foot excavation and required a 20-ft embedment depth but required PZ27 sheet piling. To utilize PZ22 sheet piling, however, the excavation must be limited to a depth of eight feet as shown in **Error! Reference source not found.**.

The hand calculations for Stage 1 are similar to those performed in Case 1 and are not provided here. Instead, the SupportIT output for this case is provided in Appendix B.7.

Based on the SupportIT calculations, an eight-foot excavation would result in a maximum bending moment of 14,955 ft-lbs/ft and a deflection of 0.9 inches with a PZ22 sheet pile. If the excavation is increased to ten feet, however, the bending moment would increase to 25,709 ft-lbs/ft, but the deflection to 2.3 inches exceeds the specification, thus requiring PZ27 sheet piling. The 8 and 10-foot analyses are illustrated in Figure 5-27.

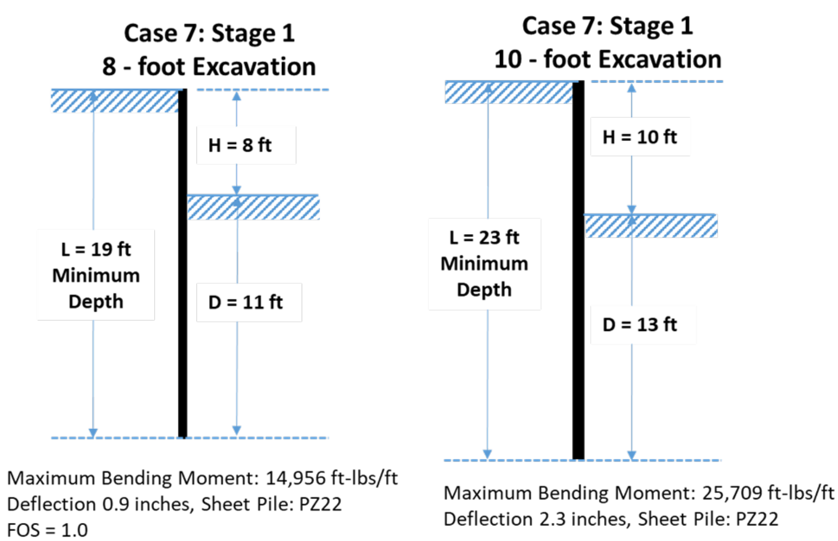


Figure 5-27 Comparison of an eight-foot excavation and a ten-foot excavation.

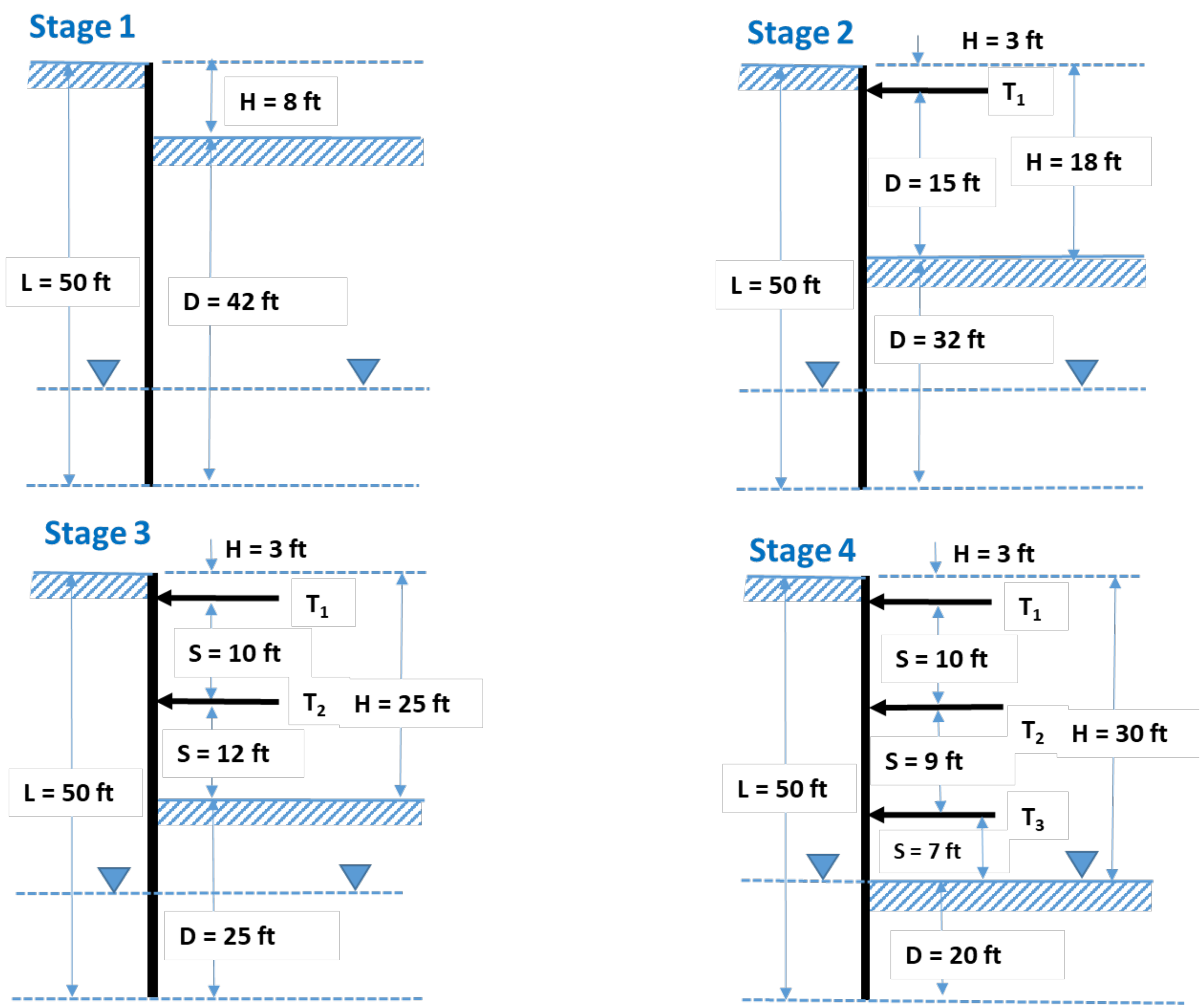


Figure 5-28 Case 7 Four stages for braced wall construction.

Stage 2: Install the first strut at 3 ft and excavate to 18 ft

In Stage 2, the first anchor is installed at a depth of 3 ft, and then excavation advanced to a depth of 18 ft, as shown in Figure 5-29. Stage 2 is similar to Case 4 where one anchor is installed, and excavation then advanced to the final depth. At a depth of 18 feet, the wall's maximum bending moment reaches 26,669 ft-lbs/ft, but the maximum deflection of only 0.9 inches, which allows a PZ22 sheet pile to be used. It is important to note again that excavation control is essential in limiting the bending moments in the sheet pile.

Since the hand calculations for Stage 2 are similar to those that would be performed in Case 4 they are not repeated here. Instead, the SupportIT output for Stage 2 is provided in Appendix B.7.

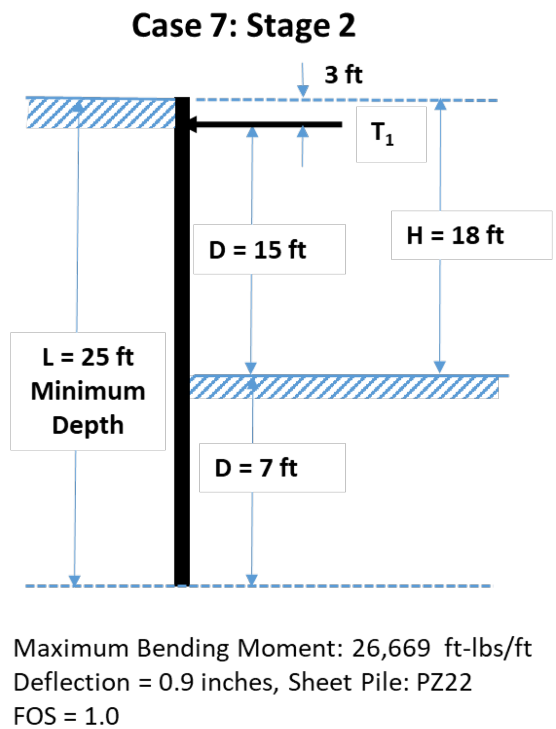


Figure 5-29 Case 7: Stage 2 construction.

Stage 3: Install strut 2 at a depth of 13 feet and excavate to 25 feet

In Stage 3, the second anchor is installed at a depth of 13 ft, and then the excavation advanced to a depth of 25 ft as shown in Figure 5-30. Stage 3 is also similar to Case 4 where one anchor is installed, and excavation then advanced to the final depth assuming that the first anchor acts as a hinge point where no moments can develop. At a depth of 25 feet, the wall's maximum bending moment reaches 29903 ft-lbs/ft allowing PZ22 sheet piling to be used. It is important to note again that excavation control is essential in limiting the bending moments in the sheet pile.

Since the hand calculations for Stage 3 are similar to those that would be performed in Case 4 they are not repeated here. Instead, the SupportIT output for Stage 3 is provided in Appendix B.7.

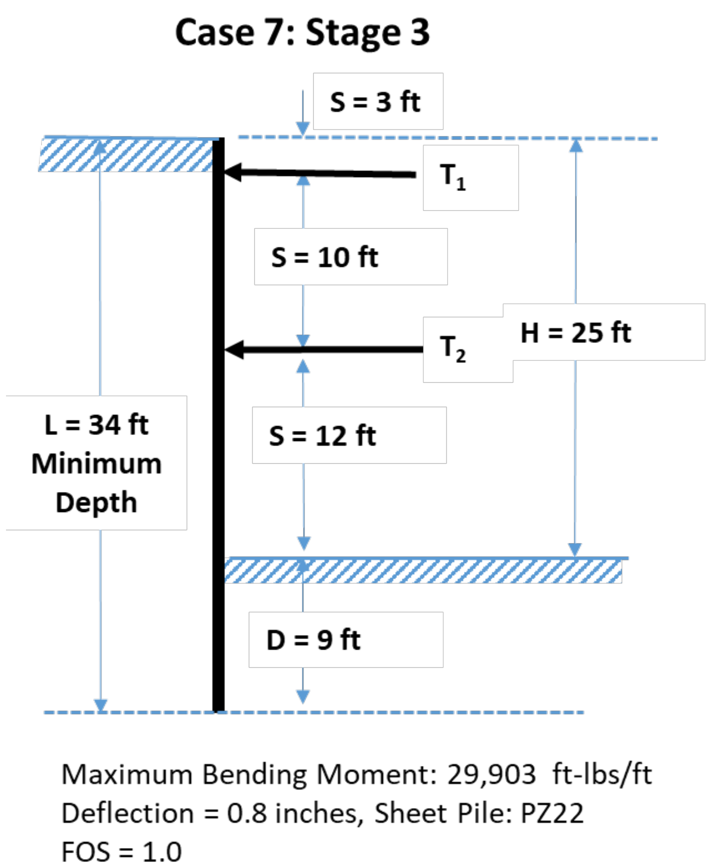


Figure 5-30 Case 7 – Stage 3 construction.

Stage 4: Install strut at a depth of 22 ft and excavate to a final depth of 30 feet

Stage 4 construction applies the third strut at a depth of 22 ft, and excavation continued to the final depth of 30 feet as shown in Figure 5-31. The stability analysis for Stage 4 consists of analyzing the third strut as a single anchored cantilever wall similar to that conducted in Case 4. The calculations for Stage 4 are provided below.

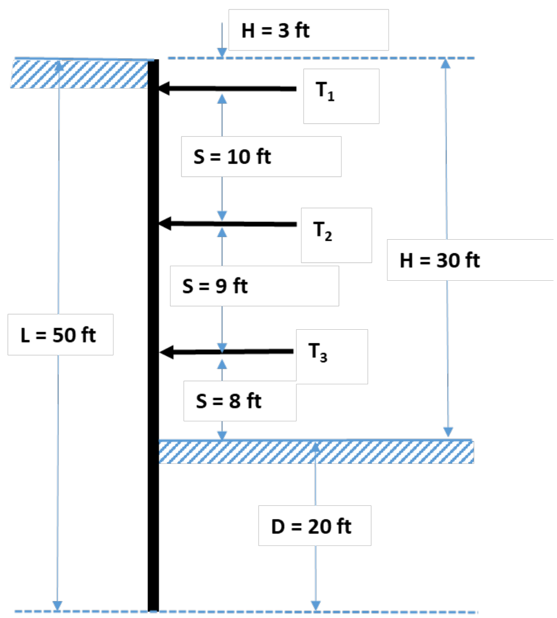


Figure 5-31 Case 7 – Stage 4 construction.

<u>Case 7, Step 1: Define the Dimensions and Soil Properties to be Analyzed for the Cantilever Wall</u>

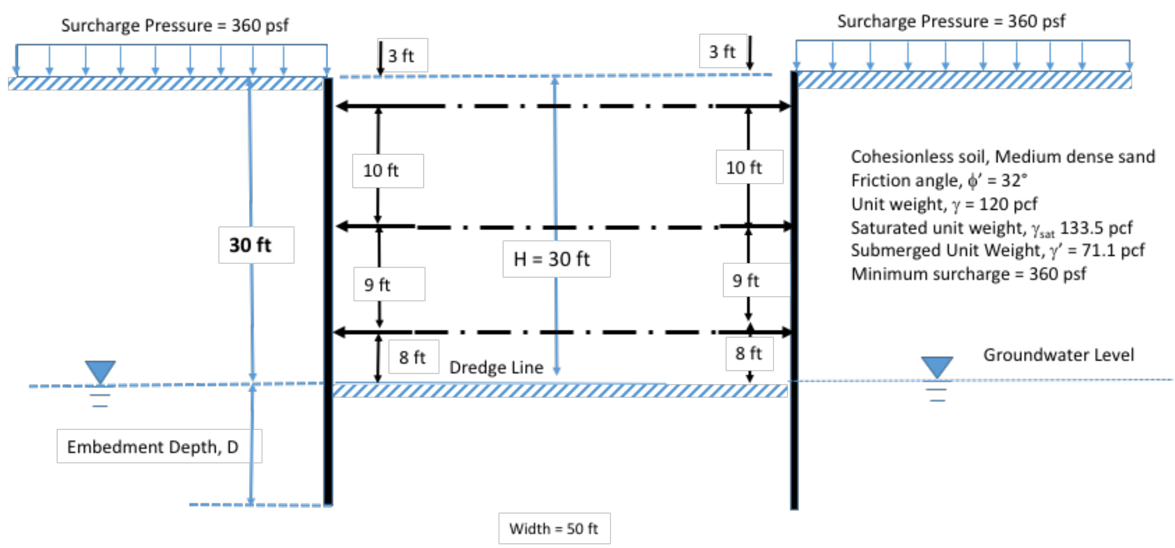


Figure 5-32 Case 7 Braced wall design parameters in cohesionless soil.

Case 7, Step 2: Calculate Sheet Pile Embedment Depth for *FOS* = 1.0 below the third Strut.

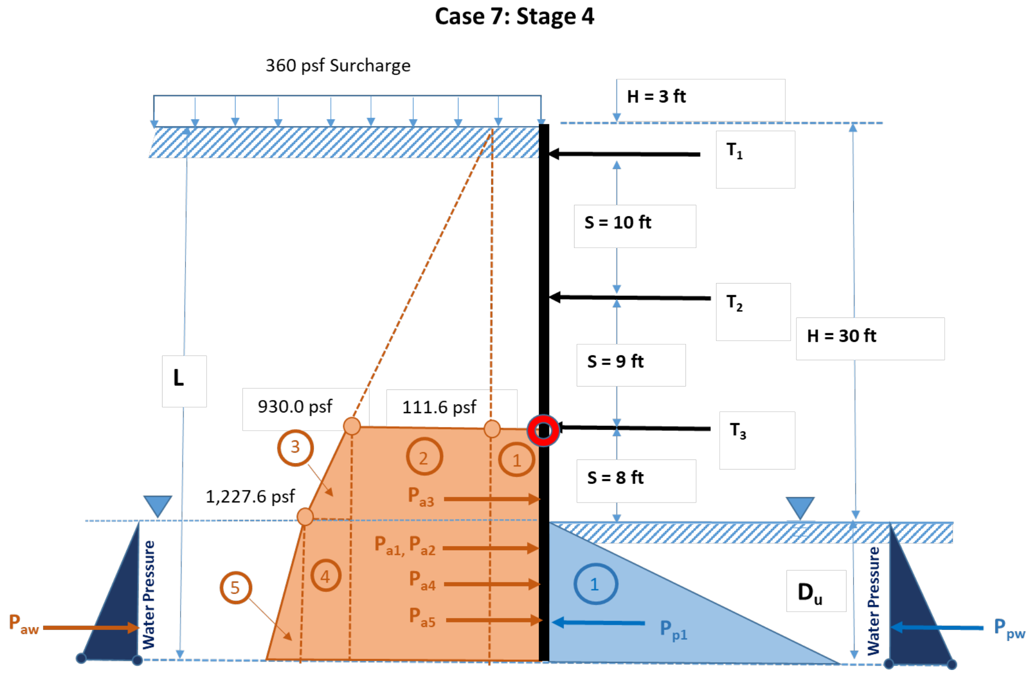


Figure 5-33 Case 7 Stage 4 analysis of anchor-cantilever wall section below Strut 3.

The strut-cantilever analysis will be conducted as a free-earth design by taking the summation of moments at the location of the third strut at a depth of 23 feet using the embedment depth, D_u, as the unknown and solving for D.

Active Forces, lbs

Active Force, P_{a1}: $P_{a1} = 112(8 + D_u)(1) = 896 + 112D$

Active Force P_{a2}: $P_{a2} = 0.31(22)(120)(8 + D_u)(1) = 6{,}547.2 + 818.4D$

Active Force P_{a3}: $P_{a3} = 0.31(0.5)(8)(120)(8)(1) = 1{,}190.4$

Active Force P_{a4}: $P_{a4} = 0.31(8)(120)(D_u)(1) = 297.6D_u$

Active Force P_{a5}: $P_{a5} = 0.31(0.5)(D_u)(71.1)(D)](1) = 11.02D_u^2$

Active Force P_{aw}: $P_{aw} = (0.5)(62.4)(D_u)(D_u)](1) = 31.2D_u^2$

Active Force Moment Arms to Strut 3 (ft)

Active Force Location, L_{a1} $L_{a1} = 0.5(8 + D) = 4 + 0.5D_u$

Active Force Location, L_{a2} $L_{a2} = 0.5(8 + D) = 4 + 0.5D_u$

Active Force Location, L_{a3} $L_{a3} = 0.67(8) = 5.36$

Active Force Location, L_{a4} $L_{a4} = 8 + 0.50D_u$

Active Force Location, L_{a5} $L_{a5} = 8 + 0.67D_u$

Active Force Location, L_{aw} $L_{aw} = 8 + 0.67D_u$

Passive Force, lbs, and Locations

Passive Force P_{p1}: $P_{p1} = 0.5(3.25)(71.1)(D)D(1) = 115.54D_u^2$

Passive Force P_{pw}: $P_{pw} = 0.5(3.25)(71.1)(D)D(1) = 115.54D_u^2$

Passive Force Moment Arms to Strut 3 (ft)

Passive Force Location, L_{p1} $L_{p1} = 8 + 0.67D_u = 8 + 0.67D_u$

Passive Force Location, L_{pw} $L_{pw} = 8 + 0.67D_u$

The embedment depth, D is determined by taking the summation of the moment about the anchor with *FOS* = 1 as follows.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS = 1.0$$

where

$$\Sigma M_{disturbing} = P_{a1}L_{a1} + P_{a2}L_{2a} + P_{a3}L_{a3} + P_{a4}L_{a4} + P_{a5}L_{a5} + P_{aw}L_{aw}$$

$$\Sigma M_{restoring} = P_{p1}L_{p1} + P_{pw}L_{pw}$$

D_u is determined using and EXCEL spreadsheet as shown in Table 5-20. The embedment (toe) depth is 12.04 ft. The SupportIT output provided in Appendix B.7.

Table 5-20 Case 7 Embedment depth *FOS* = 1.

Calculation of Sheet Pile Depth, D_u (FOS = 1.0)			
Depth of Sheet Pile, D_u		12.04	
FOS		1.00	
Passive Moment:		Active Moment:	
P_{p1}	16,741.3	P_{a1}	2236.4
L_{p1}	16.1	L_{a1}	10.0
P_{pw}	4,522.3	P_{a2}	16400.2
L_{pw}	16.1	L_{a2}	10.0
M_r	341,629	P_{a3}	1190.4
		L_{a3}	5.3
		P_{a4}	3582.9
		L_{a4}	14.0
		P_{a5}	1597.3
		L_{a5}	16.1
		P_{aw}	4522.3
		L_{aw}	16.1
		M_d	341,629

Unfactored: FOS = 1.0:

D_u = 12.04 ft

$D_{u,SupportIT}$ = 11.96 ft

Case 7, Step 3: Determine Zero-Shear Load and Location to determine the Maximum Bending Moment

The maximum bending moment in the sheet pile wall is located at the depth, D_s, where the shear forces are zero. The zero-shear force location is calculated by summing the forces acting on the sheet pile wall below the anchor to where they equal the shear force acting on the anchor. Since the water pressure is equal on both sides of the sheet pile wall, the water loads are not included in the following calculations. If the water pressures are not equally acting on the wall, they must be included in the analysis.

The zero-shear force location is calculated by first calculating the difference between the active and passive forces acting as follows:

T = Active forces – Passive Forces = (P_{a1} + P_{a2} + P_{a3} + P_{a4} + P_{a5}) – (P_{p1}) (water load not included)

T = (2,236 + 16,399 + 1,190 + 3,583 + 1,597) – 16,739 = 8,266 lbs

T = 8,266 lbs (per ft of sheet pile wall)

$T_{SupportIT}$ = 8,200 lbs/ft

Case 7, Step 4: Determine of the maximum bending moment:

Determination of the location of zero-shear:

The zero-shear stress location, D_s, is located near the dredge line where the active and passive forces equal the strut load. The distance D_s is shown in Figure 5-34. D_s is calculated as follows:

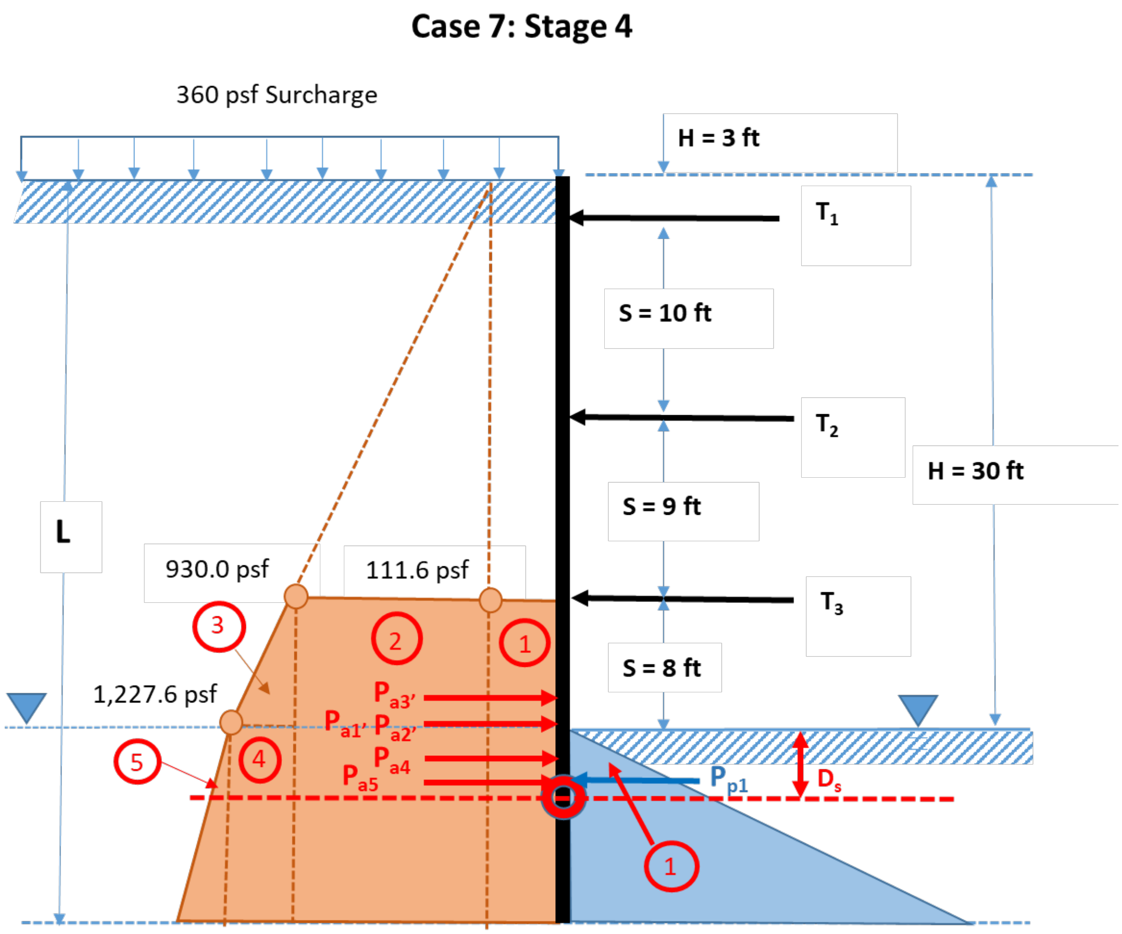


Figure 5-34 Case 7 Stage 4 Determination of zero shear location and maximum bending moment.

<u>Active & Passive Forces, lbs in terms of D_s:</u>

Active Force, P_{a1}:	$P_{a1'} = 0.31(360)(8 + D_s)(1) = 892.8 + 111.6D_s$
Active Force P_{a2}:	$P_{a2'} = 0.31(22)(120)(8 + D_s)(1) = 6{,}547.2 + 818.4D_s$
Active Force P_{a3}:	$P_{a3'} = 0.5(0.31)(8)(120)(8)(1) = 1{,}190.4$
Active Force P_{a4}:	$P_{a4'} = 0.31(8)(120)(D_s)(1) = 297.6D_s$
Active Force P_{a5}:	$P_{a5'} = 0.5(0.31)(D_s)(71.1)(D_s)](1) = 11.02D_s^2$
Passive Force $P_{p1'}$:	$P_{p1'} = 0.5(3.25)(71.1)(D_s)D_s(1) = 115.54D_s^2$

$T = 8{,}266 = [P_{a1'} + P_{a2'} + P_{a3'} + P_{a4'} + P_{a5'}] – [P_{p1'}]$

$8{,}266 = [892.8 + 111.6D_s + 6{,}547.2 + 818.4D_s + 1{,}190.4 + 297.6D_s + 11.02D_s^2] – [115.54D_s^2]$

$104.48D_s^2 – 1{,}227.6D_s - 364.4 = 0$

<u>$D_s = -0.29$ ft</u>

Maximum Moment Location: 30.00 - 0.29 = <u>29.71 ft</u>

Maximum Moment Location$_{\mathbf{SuppotrtIT}}$: <u>29.76 ft</u>

Note that the bending moment is located above the dredge line.

The maximum bending moment is calculated by taking the summation of moments at 29.71 feet for the forces acting above the location D_s. With multiple struts, the lowest strut is analyzed as a cantilever wall. Therefore, the shear force at strut 3 is not included in the calculation.

<u>Shear Forces (lbs):</u>

$P_{a1'}$ = 892.8 + 111.6D_s = 892.8 + 111.6(-0.29) = 860.4

$P_{a2'}$ = 6,547.2 + 818.4D_s = 6,547.2 + 818.4(-0.29) = 6,310.0

$P_{a3'}$ = 1,190.4

<u>Moment Arm (ft):</u>

$L_{a1'}$ = 0.5(8 + D_s) = 4 + 0.5(-0.29) = 3.86

$L_{a2'}$ = 0.5(8 + D_s) = 4 + 0.5(-0.29) = 3.86

$L_{a3'}$ = 0.33(8) + D_s = 5.36 -0.29 = 2.38

Moment: M = [$P_{a1'}$ $L_{a1'}$ + $P_{a2'}$ $L_{a2'}$ + $P_{a3'}$ $L_{a3'}$]

Moment: M = [(860.4)(3.86) + (6,310.0)(3.86) + (1,109.4)(2.38)] = 30,473 ft-lbs/ft

<u>Max Moment = +30,318 ft-lbs/ft</u>

<u>Max Moment$_{\mathbf{SupportIT}}$ = +30,295 ft-lbs/ft</u>

<u>Case 7, Step 5: Sheet Pile Selection</u>

The maximum moment was estimated to be 30,318 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 25 ksi, the required section modulus is determined as follows

Required section modulus = M/f_s = [30,318 ft-lbs/ft x 12 in/ft] / 25,000 psi = 14.6 in^3/ft

The section modulus of US Steel's PZ22 is 18.4 in^3/ft. Therefore a PZ22 will meet the section modulus requirement. According to the SupportIT solution provided in Appendix B.7, it was determined that a PZ22 sheet pile would have 0.6 inches of deflection, which meets the minimum MDOT deflection limit requirement of 2.0 in.

Case 7, Step 6: Calculate Sheet Pile Total Length with a Factor of Safety

The Gross Pressure CP2 method with a FOS = 1.5 is used to determine the embedment depth D_f. The depth D_f is then recalculated as shown in Table 5-21.

Table 5-21 Case 7 Embedment Depth using a free-earth method and *FOS* = 1.5

Calculation of Sheet Pile Depth, D_f (FOS = 1.5)			
Depth of Sheet Pile, D_f		20.08	
FOS		1.50	
Passive Moment:		Active Moment:	
P_{p1}	46,572.8	P_{a1}	3133.8
L_{p1}	21.5	L_{a1}	14.0
P_{pw}	12,580.7	P_{a2}	22981.1
L_{pw}	21.5	L_{a2}	14.0
M_r	1,269,077	P_{a3}	1190.4
		L_{a3}	5.3
		P_{a4}	5976.0
		L_{a4}	18.0
		P_{a5}	4443.6
		L_{a5}	21.5
		P_{aw}	12580.7
		L_{aw}	21.5
		M_d	846,051

Toe Depth = 20.08 ft

Toe Depth$_{SupportIT}$ = 20.09 ft

Case 7 is based on the *free earth method*, not the simplified method used in Cases 1-3. Therefore, no adjustment to the embedment depth, D_f, is required after the application of the *FOS*.

Embedment Depth, D_f = 20.1 ft

Total Sheet Pile Length = H + D_f = 30 + 20.1 = 50.1ft

L = 51.1 ft

$L_{SupportIT}$ = 51.1 ft

Comparison to the SupportIT software results:

Table 5-22 Case 7 Comparison of SupportIT and Hand Calculations

	SupportIT (Total pile length, ft, ≅ L)	**Hand Calculations** (Total pile length, ft, ≅ L)
Maximum soil pressure at dredge line, (psf/ft)	1,227	1,227
Max Shear Load, Lbs/ft	8,200	8,266
Zero Shear location along sheet pile, (ft)	29.76	29.71
Maximum Moment, (ft-lbs/ft), FOS = 1.0	30,295	30,318
Sheet Pile Embedment, FOS = 1.00, D_u (ft)	11.96	12.04
USS 20% FOS Embedment Length, D_f (ft)	14.4 (45)	14.4 (45)
USS 40% FOS Embedment Length, D_f (ft)	16.7 (47)	16.7 (47)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5	20.09 (50)	20.08 (50)

Case 7, Step 7: Calculation of Strut Loads Using the Terzaghi-Peck Apparent Pressure Diagram

The sheet piling's bending stresses and embedment depth were determined using a "standard" Rankine analysis. That is, the shear forces acting on the sheet piling were based on the soil stresses calculated by the Rankine method. In the United States, the Terzaghi and Peck's (TP) *apparent pressure diagrams* (TP), however, are commonly used to determine strut loads. The British Code Standards 8002:1994 allows the use of TP pressure diagrams (*Piling Handbook* 8th ed., 2005) but also suggests other methods that have been shown to provide more accurate load estimates than the TP pressure diagrams. Case 7 will use the Terzaghi-Peck (TP) *apparent pressure diagrams* to estimate the strut loads. The TP diagram for cohesionless soils is shown in Figure 5-35.

An important design consideration in using the TP approach is selecting the earth pressure distributions below the dredge line. SupportIT and SPW911 provide two methods. First, the TP pressure diagram can be extended downward, as shown in Figure 5-36(a). The second method is to utilize the Rankine pressure diagram, as shown in Figure 5-36(b). For Case 7, the Rankine pressure distribution is used because it is similar to the method used in the preceding cases.

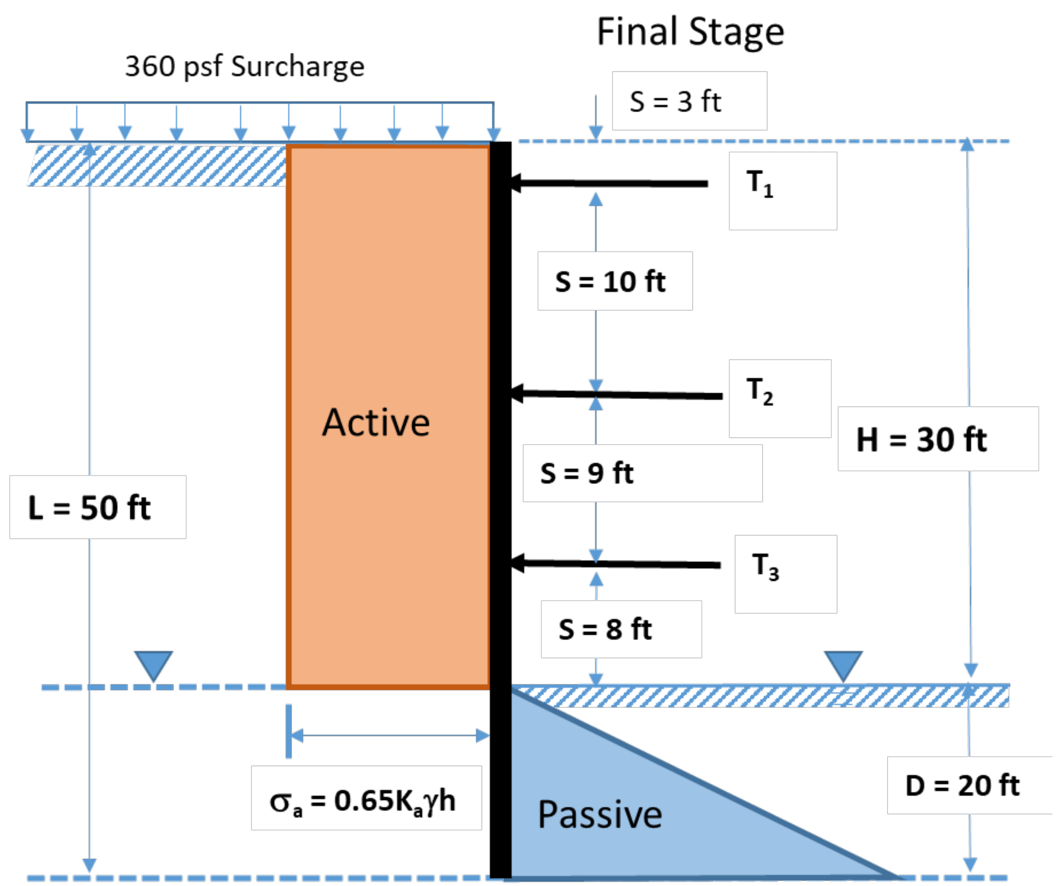


Figure 5-35 Case 7 Terzaghi-Peck (TP) apparent pressure diagram for sand.

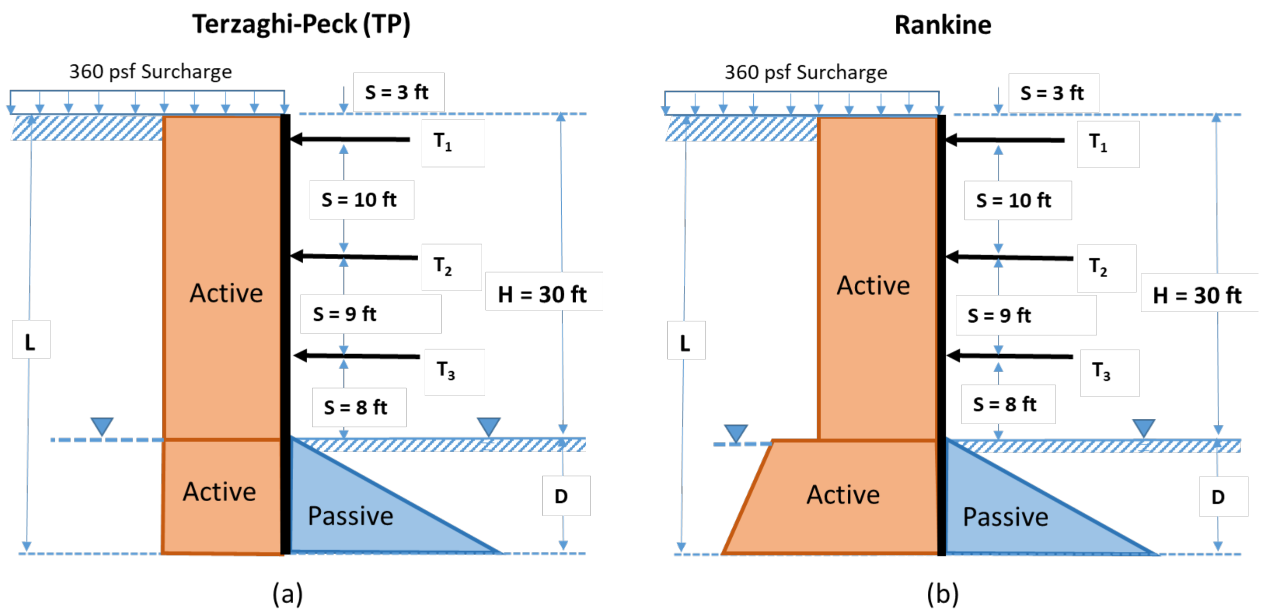


Figure 5-36 Case 7 Sheet pile toe pressure assumptions: (a) Terzaghi-Peck and (b) Rankine.

Case 7, Step 7(a): Calculate earth pressure acting on sheet pile

Terzaghi-Peck Pressures to 30 feet:

Terzaghi-Peck Pressure: $\sigma_a = 0.65K_a\gamma h = 0.65(0.31)(120)(30) = \underline{725.4\text{ psf/ft}}$

Surcharge pressure: $\sigma_{a,\,sur} = (0.31)(360) = \underline{111.6\text{ psf/ft}}$

Total pressure above dredge line: $\sigma_{a,total} = \sigma_a + \sigma_{a,\,sur} = 725.4 + 111.6 = \underline{\underline{837.0\text{ psf/ft}}}$

Rankine Active Pressure at 30 ft.

Rankine active pressure at 30 feet: $\sigma_{a,30} = K_a\gamma h = (0.31)(120)(30) = \underline{1{,}116.0\text{ psf/ft}}$

Total active Pressure at 30 feet: $\sigma_a = \sigma_{a,30} + \sigma_{a,\,sur} = 1{,}116.0 + 111.6 = \underline{\underline{1{,}227.6\text{ psf/ft}}}$

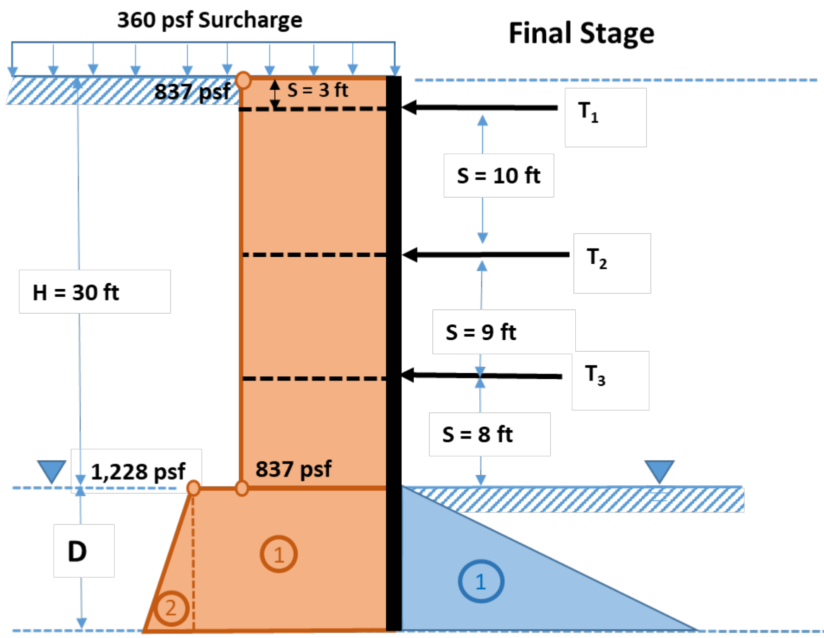


Figure 5-37 Case 7 Earth pressures and strut load distributions.

Case 7, Step 7(b): Calculate Strut Loads #1 and #2

To calculate the upper strut loads, SupportIT provides the following three methods: (1) area distribution, (2) hinge method, and (3) rigid wall method. The area distribution is computationally the most straightforward method and is used in this example. With the area distribution method, loads are calculated based on the tributary areas above and below the strut as shown in Figure 5-38.

For the third and lowest strut, in this case, T_3 (strut 3), is determined assuming it is a single anchor wall, as shown in Figure 5-39. This analysis is similar to the analysis used in Case 4.

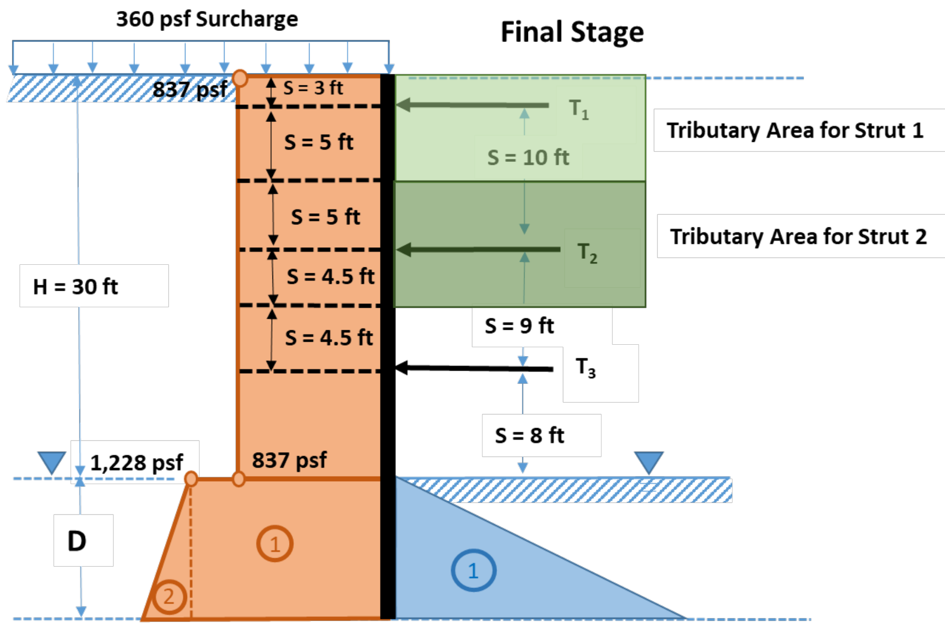


Figure 5-38 Case 7 Tributary areas for calculating strut loads #1 and #2.

Load Above T_1: T_{1above} = (3)(837.0) = 2,511 lbs/ft

Load Below T_1: T_{1below} = (5)(837.0) = 4,185 lbs/ft

Total Strut #1 load: $T_1 = T_{1above} + T_{1below}$

T_1 = 2,511.0 + 4,185.0 = <u>6,696 lbs/ft</u>

Strut Load T_1 = <u>6,696 lbs/ft</u>

SupportIT T1 = <u>6,696 lbs/ft</u>

Load Above T_2: T_{1above} = (5)(837.0) = 4,185 lbs/ft

Load Below T_2: T_{1below} = (4.5)(837.0) = 3,767 lbs/ft

Total Strut #1 load: $T_1 = T_{1above} + T_{1below}$

T_1 = 4,185 + 3,767 = <u>7,952 lbs/ft</u>

Strut Load T_2 = <u>7,952 lbs/ft</u>

SupportIT T_2 = <u>7,927 lbs/ft</u>

Case 7, Step 7(d): Calculate Strut #3 Load

To calculate the strut load T_3, the moment (M_1) is taken at T_3 by assuming the restoring moment equals the disturbing moment ($M_R = M_D$) when calculating the depth of embedment, D. The embedment depth D is used to estimate the earth pressure acting on the sheet pile below the dredge line and the load on strut #3. Note, that the calculated D is NOT the final depth of the sheet pile. The final sheet pile depth was calculated in Step 6.

The basic analysis parameters are shown in Figure 5-39.

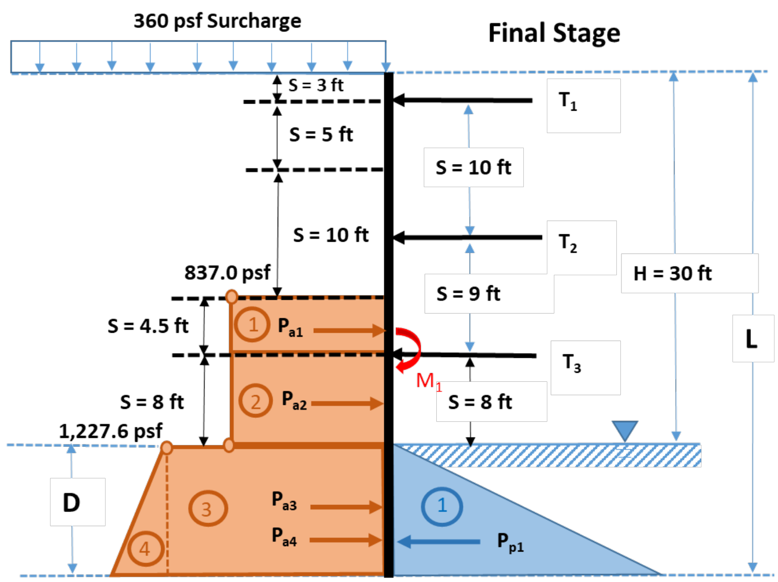


Figure 5-39 Case 7 Calculation diagrams for estimating the load on Strut #3.

Moment M_1 in terms of depth, D, for the forces below the Strut #3:

Active Forces (lbs) & Moment Arms (ft):

$P_{a1} = (4.5)(837) = 3{,}766.5$	$L_{a1} = 0.5(4.5) = 2.25$
$P_{a2} = (8.0)(837) = 6{,}696$	$L_{a2} = 0.5(8.0) = 4.0$
$P_{a3} = (1{,}227.6)D = 1{,}227.6D$	$L_{a3} = 8.0 + 0.5D$
$P_{a4} = 0.5(0.31)(71.1)D^2 = 11.02D^2$	$L_{a4} = 8.0 + 0.67D$

Passive Forces (lbs) & Moment Arms (ft):

$P_{p1} = 0.5(3.25)(71.1)D^2 = 115.54D^2$	$L_{p1} = 8.0 + 0.67D$

$M_D = (P_{a2}L_{a2} + P_{a3}L_{a3} + P_{a4}L_{a4})$

$M_R = (P_{a1}L_{a1} + P_{p1}L_{p1})$

$M_D = M_R$ Solve for D.

Solving for D can be determined using a "*Defined FOS*," with the FOS = 1.0. Again, as in calculating the bending moments in the sheet pile, there is no factoring of the soil or lateral pressure coefficients. An EXCEL is used to solve for the depth D as shown below in Table 5-23.

Table 5-23 Case 7 Calculation of D for estimating Strut #3 load.

Calculation of Sheet Pile Depth, D			
Depth of Sheet Pile, D		11.29	
Define: FOS		1.00	
Restoring Moment:		Disturbing Moment:	
P_{a1}	3766.5	P_{a2}	6,696
L_{a1}	2.25	L_{a2}	4.0
P_{p1}	14,736	P_{a3}	13,864
L_{p1}	15.6	L_{a3}	13.6
M_r	237,874	P_{a4}	1,406
		L_{a4}	15.6
		M_d	237,874

D = 11.29 ft

$D_{SupportIT}$ = 11.71 ft

Calculate forces acting on the sheet pile wall from the dredge line to D = 11.30 ft:

Strut #3 load:

$T_3 = [P_{a1} + P_{a2} + P_{a3} + P_{a4}] - P_{p1}$

$T_3 = [3,766 + 6,696 + 13,872 + 1,407] - 14,752 = 10,989$ lbs/ft

Strut Load T_3 = 10,996 lbs/ft
SupportIT: $T_{3,SupportIT}$ = 10,536 lbs/ft

Strut Load Comparison:

Table 5-24 Case 7 Comparison of hand calculations to SupportIT strut load calculations.

	Hand Calculations (lbs/ft)	**SupportIT (lbs/ft)**
Strut #1	6,696 lbs/ft	6,696 lbs/ft
Strut #2	7,952 lbs/ft	7,927 lbs/ft
Strut #3	10,996 lbs/ft	10,536 lbs/ft

Case 7, Step 8: Calculation of Potential Hydraulic Heave or Piping

An important design element in braced wall construction is the analysis of upward hydraulic gradients causing water flow into the excavation. This is known as hydraulic heave, quick conditions, or internal erosion and piping in cohesionless soils.

When the groundwater table on the active side of the sheet pile wall increases above the dredge line level, an upward hydraulic gradient develops causing water to flow into the excavation. The water flow can be controlled by dewatering wells placed outside the excavation to reduce or eliminate water flow. Other methods to reduce the flow of water into the excavation include driving the sheet piling deeper, or by placing a gravel layer on the base of the excavation. However, it is important to conduct an analysis, to verify that the water flow still does not cause hydraulic heave or piping.

Figure 5-40 illustrates Case 7 where the water level on the active side of a braced excavation increases to the surface while the water level in the excavation remains at the dredge line. This situation represents a worst-case scenario in the potential for hydraulic heave. In this situation, significant hydrostatic pressure is placed on the sheet piling and an upward water gradient causing water to flow into the excavation. A SupportIT software analysis is shown in Figure 5-41 for Case 7. In this situation, the load on strut #3 would triple, the embedment depth would increase to 53 ft, and a PZ40 sheet pile section would be required to handle the increased moments in the sheet piling.

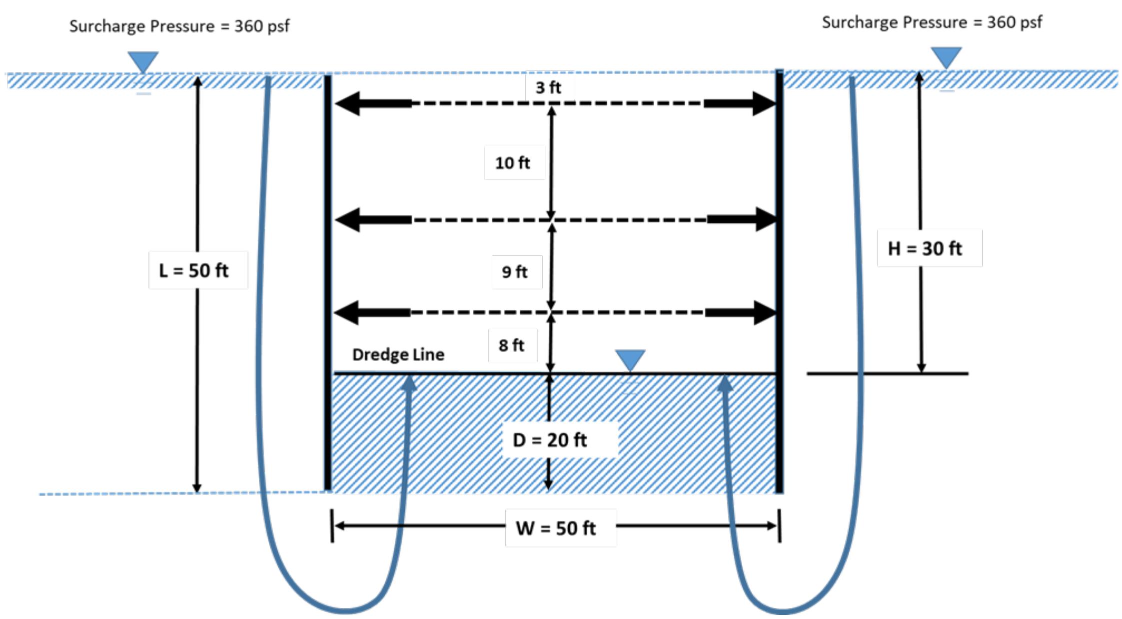


Figure 5-40 Case 7 Potential groundwater table increase to surface on the active side.

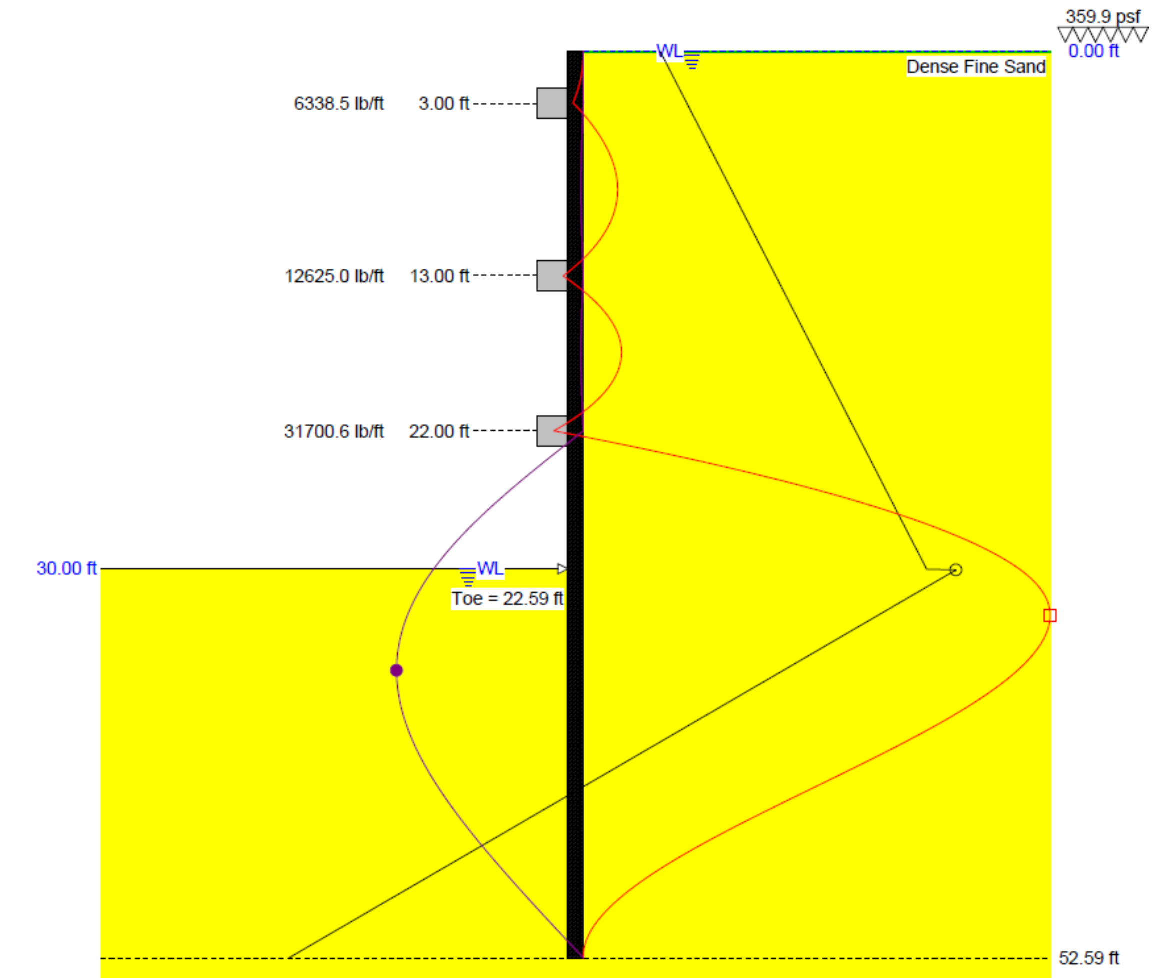


Figure 5-41 Case 7 Groundwater increase to surface on the active side, by SupportIT.

The US Steel Sheet Pile Manual provides a figure, provided in Figure 5-42, to assess the potential for hydraulic heave and piping based on embedment depth and the width of the excavation. The figure was adapted from the 1962 Navy Facilities Manual (Navdocks, 1962 and NAVFAC, 1986), which is valid for soils with buoyant unit weights $\gamma' < 75$ pcf. The soil in Case 7 has a buoyant unit weight of 71.1 pcf.

The purpose of the chart is to determine the embedment depth for the sheet piling to prevent hydraulic heave and piping. The chart has two parts. The upper section determines the factor of safety against heave in loose sands and piping in dense sands. The lower section of the figure determines the factor of safety against piping when an impervious layer is located below the excavation. Since Case 7 does not assume an impervious layer below the excavation, only the upper portion of the figure will be used.

The parameters for Case 7 with the groundwater on the active side at the surface are as follows:

Soil: Dense sand – use solid FOS lines in chart
H_u = Difference in hydraulic head = 30 ft
D = pile embedment depth ≈ 20 ft
B = Width of the excavation; assume = 50 ft (assumed)
D/H_u = 20/30 = 0.67 ≈ 0.7
B/H_u = 50/30 = 1.67 ≈ 1.7

Hydraulic heave and piping analysis:

To use the figure in Figure 5-42, a vertical line is drawn from the B/H_u = 1.7 value on the horizontal axis, and a horizontal line is drawn from the D/H_u = 0.7 value on the vertical axis. Where the two lines intersect, this point provides the factor of safety for this case. The resulting factor of safety from Figure 5-42 is **FS = 1.5.**

As noted above, the width of the excavation was assumed to be 50 feet. The chart can also be used to determine the excavation's width at a FOS = 1.0. By drawing a horizontal line from the D/H_u = 0.7 value on the vertical axis to the FOS = 1.0 line. From this intersection, a vertical line can be drawn to the horizontal (B/Hu) axis. The B/Hu value is approximately 0.35. At a sheet pile toe depth of D = 20 feet, the excavation width, B, would be 7 feet.

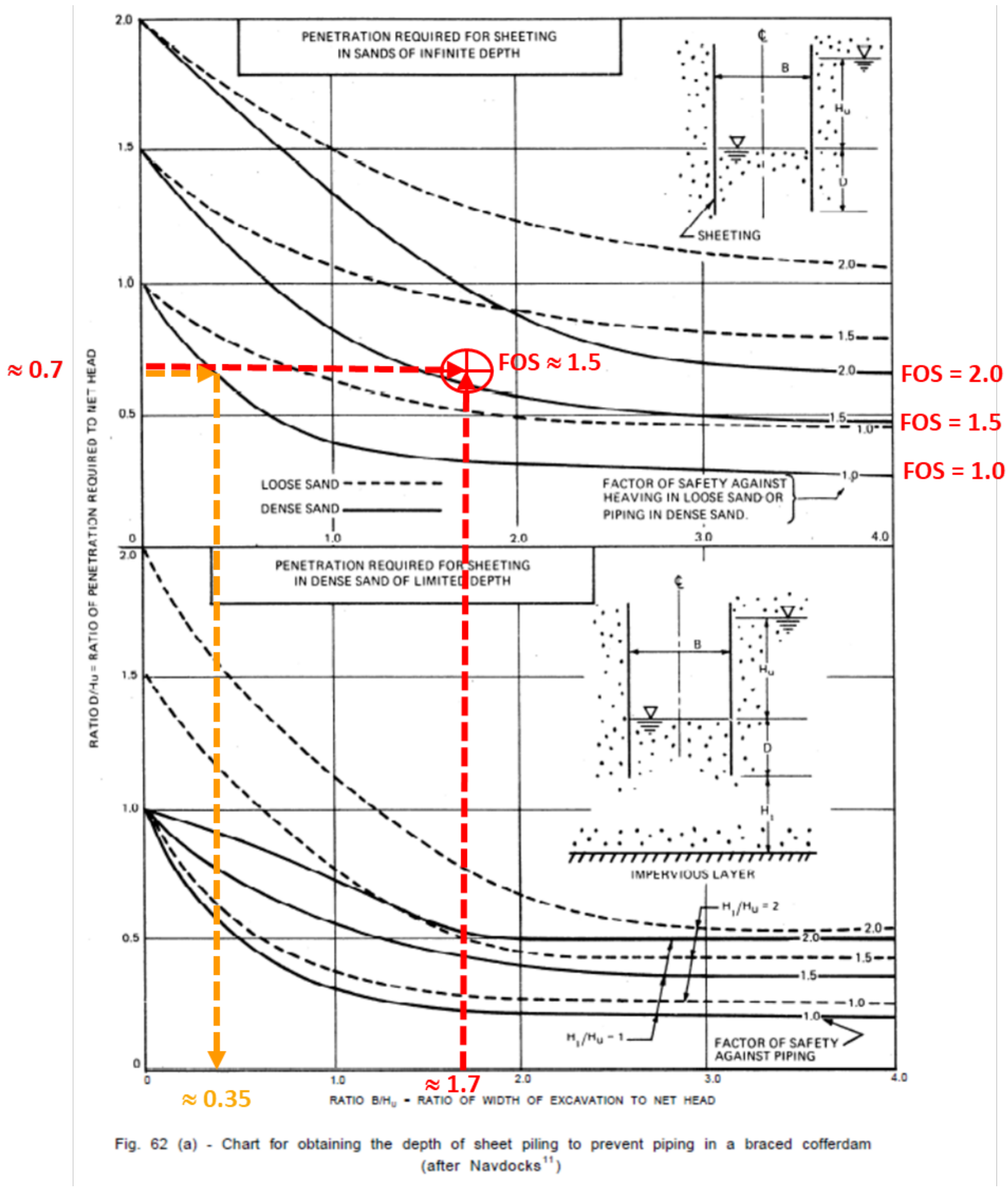


Figure 5-42 Case 7 US Steel Sheet Pile Manual chart to assess piping and required embedment penetration.

5.2.8 Case 8 – Braced Wall TERS in Soft Cohesive Soil

Case 8 illustrates the design of a 30-foot deep excavation in soft clay overlying a firm clay layer with the groundwater elevation at the dredge line, i.e., at 30 feet. Due to the clay's low strength and fluid-like quality, significantly stronger sheet piling is required in addition to much higher strut loads than were calculated in Case 7. As in Case 7, the analysis requires a "staged construction" design. That is, each construction stage the excavation must be analyzed so that its bending moments and deflection limits are not exceeded.

The "staged construction" calculations are similar to the method used in Case 7. For Case 8, the staged construction calculation will use the SupportIT software, while hand calculations will be provided for the calculation of the strut loads. To optimize the strut loads, spacing, and sheet pile section, however, an iterative process is required, which is not considered in this example.

Case 8 is analyzed in four stages as listed below and shown in Figure 5-43.

Stage 1: Install sheet piling to a final depth determined in Stage 3, excavate to 8 ft.
Stage 2: Install the first strut at 3 ft and excavate to 17 ft.
Stage 3: Install the second strut at 13 ft and excavate to 25 ft,
Stage 4: Install the third strut at 22 ft and excavate to a final depth of 30-ft.

Notes:

1. Sheet pile design using the limit equilibrium method (LEM) for soft clay soils can result in unrealistic embedment depths, e.g., soils with undrained shear strength less than 500 psf. While the active pressure increases with depth, the passive pressure from the clay's strength (two times its strength) remains constant with depth. To address this issue, a firm clay with an undrained strength of c = 1,500 psf is assumed below the dredge line. Furthermore, in using the Gross Pressure (CP2) method, there is an anomaly in undrained conditions because the calculated FOS decreases beyond a certain penetration depth. Consequently, the Net Pressure method is used by the SupportIT software to estimate the embedment depth.
2. Hand calculations are provided for determining the strut loads and compared against the SupportIT software solutions, which are provided in Appendix B.8 for Construction Stages 1 through 4.
3. A total stress analysis is utilized, so the saturated unit weight of the cohesive soil, γ_{sat}, is used and not the soil's effective unit weight, γ' in calculating the wall pressures below the groundwater table.
4. Case 8 assumes that the cohesive soil adhesion S_w is zero. As noted previously, the SupportIT software does not include soil adhesion in the calculation of K_{ac} or K_{pc}.

Case 8, Step 1: Define the Dimensions and Soil Properties for the Braced Wall Design.

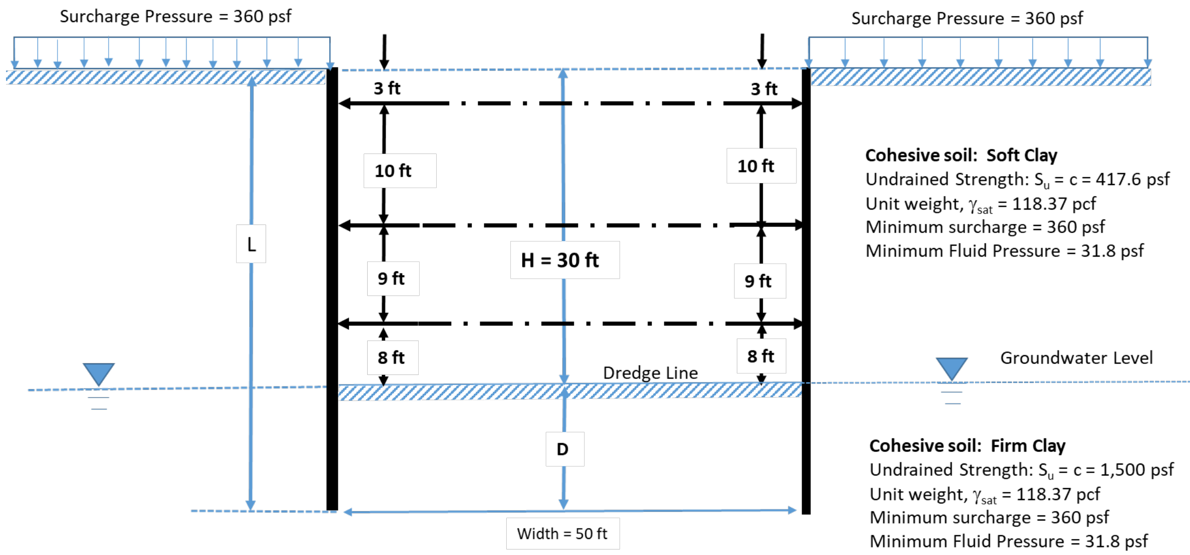


Figure 5-43 Case 8 Braced wall design parameters in cohesive soil.

Case 8, Step 2: Determination of Active and Passive Pressure Coefficients

For cohesive soils, the active and passive earth pressure coefficients are equal to one.

$$K_a = K_p = 1.0$$

Case 8, Step 3: Calculate the critical height for the soil

Soft Clay: c = 417.6 psf

Critical height: $$H_c = \frac{2c - \sigma_{surcharge}}{\gamma} = \frac{2(417.6) - 360}{118.4} \approx 4.0\ feet$$

Firm Clay: c = 1500 psf

Critical height: $$H_c = \frac{2c - \sigma_{surcharge}}{\gamma} = \frac{2(1{,}500) - 360}{118.4} \approx 22.3\ feet$$

Case 8, Step 4: Calculate Active and Passive Earth Pressures and Forces

Active pressure: Minimum Fluid Pressure: $\sigma_{a1} = \gamma z = 31.8(z)$

Cohesive soil: $\sigma_{a1} = \gamma z = 118.4(z)$

Passive Pressure: Cohesive soil passive resistance: $\sigma_{p1} = 2c = 2(1{,}500) = 3000$ psf

Increase in soil pressure: $\sigma_{p2} = z\gamma = 118.4(z)$

Pressures Acting on Sheet Pile Wall to a depth of 35 feet (Figure 5-44):

<u>Active Pressures:</u> Critical Height: H_c = 4 ft: The minimum fluid pressure (MFP) will be applied to the wall to a depth of 4 ft after wich clay's active pressure is applied. The depth can be determined by the following equation:

$MFP(z) = \gamma_{sat}(z\text{-}H_c)$
$31.82(z) = 118.37(z\text{-}4)$
z = 5.47 ft (where the MFP = active soil pressure)

Minimum Fluid Pressure: $\sigma_{a4'} = K_a\gamma H = (1.0)(31.8)(4) = \underline{127.2\text{ psf}}$

$\sigma_{a5.47'} = K_a\gamma H = (1.0)(31.8)(5.47) = \underline{174.0\text{ psf}}$

Active Soil Pressure: $\boldsymbol{\sigma_{a5.47'}} = K_a\gamma H = (1.0)(118.37)(5.47 - 4.0) = \underline{\mathbf{174.0\text{ psf}}}$

$\boldsymbol{\sigma_{a29.9'}} = K_a\gamma H = (1.0)(118.37)(30\text{-}4) = \underline{\mathbf{3{,}078\text{ psf}}}$

$\boldsymbol{\sigma_{a30'}} = K_a\gamma H = (1.0)(118.37)(30 - 22.3) = \underline{\mathbf{911\text{ psf}}}$

$\boldsymbol{\sigma_{a35'}} = K_a\gamma H = (1.0)(118.37)(30 - 22.3 + 5) = \underline{\mathbf{1{,}503\text{ psf}}}$

Passive Soil pressure: $\sigma_{p30'} = 2c = 2(1500) = \underline{3{,}000\text{ psf}}$

$\boldsymbol{\sigma_{p35'}} = 2c + K_a\gamma H = 3{,}000 + (1.0)(118.37)(5) = \underline{\mathbf{3{,}592\text{ psf}}}$

Net pressure at 30 ft: $\sigma_{NP30'} = \sigma_{a30'} - \sigma_{p30'} = 911.4 - 3{,}000 = \underline{-2{,}088.6\text{ psf}}$

$\sigma_{NP35'} = \sigma_{a35'} - \sigma_{p35'} = 1{,}503 - 3{,}592 = \underline{-2{,}088.6\text{ psf}}$

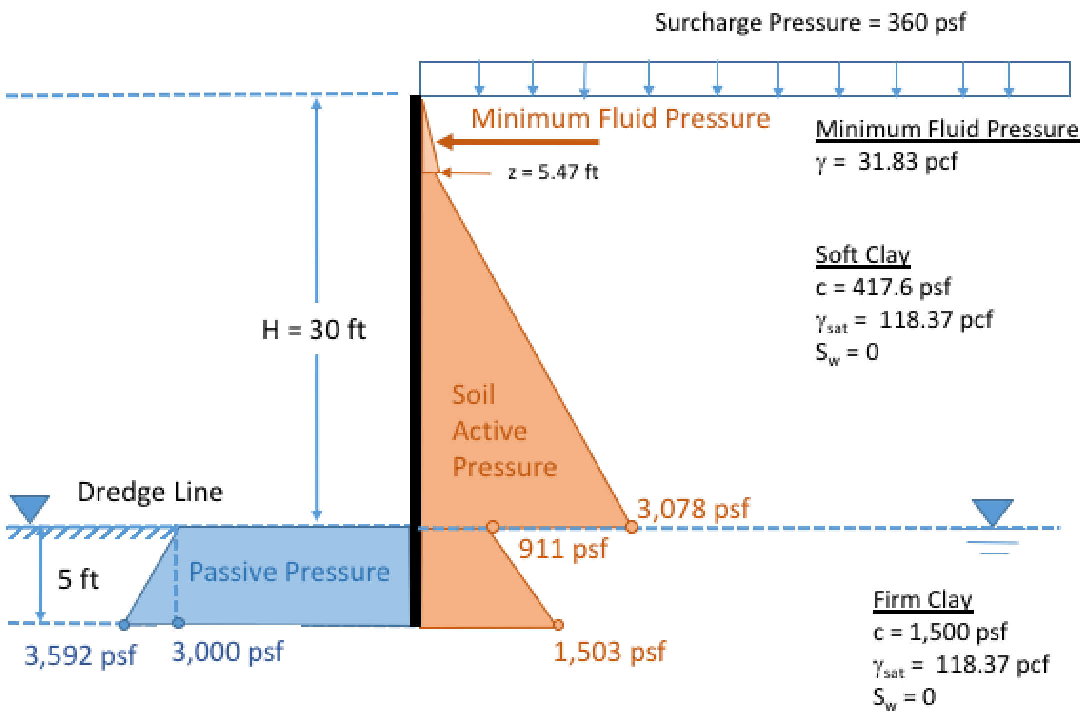


Figure 5-44 Case 8 Lateral active and passive pressure acting on the wall.

Case 8, Step 5: Staged Construction Analysis

A "staged construction analysis" is used to determine the bending moments, deflections, and required embedment depth for each stage of construction using a Rankin earth pressure model. For soft clays, however, a "net pressure" analysis is used instead of the gross pressure (CP2) method that was used in the previous cases. As noted above, the SupportIT software does not allow the CP2 method to be used for clay when $K_p = K_a = 1.0$. For stronger clays, such as firm clay, however, the difference between the CP2 analysis and the net pressure analysis is not large.

In general, the procedure for Case 8 is similar to Case 7. SupportIT solutions are provided in Appendix B.8. Hand calculations, however, are provided, illustrating the calculations to determine the strut loads. As in Case 7, the Terzaghi-Peck apparent pressure diagrams are used to calculate the strut loads.

The stages of construction are shown in Figure 5-45, while the SupportIT results for the four stages, along with the final results, are provided in Table 5-25. The final stage of construction includes a *FOS* = 1.5.

Table 5-25 Case 8 Staged construction SupportIT software results.

Construction Stage	Excavation Depth (ft)	Strut Location (ft)	FOS = 1.00 Max Moment (ft-lbs/ft)	FOS = 1.00 δ (in)	FOS = 1.50 Pile Length (ft)	Required Sheet Pile
Stage 1	8	-	5,303	0.2	23.3	PZ22
Stage 2	17	3	**84,921**	1.1	34.1	**PZ35**
Stage 3	25	13	81,633	0.6	**37.3**	**PZ35**
Stage 4	30	22	30,951	0.2	35.9	PZ27

Note that the maximum toe embedment depth is 37 ft in Stage 3 while the maximum bending moment is 84,921 ft-lb/ft in Stage 2.

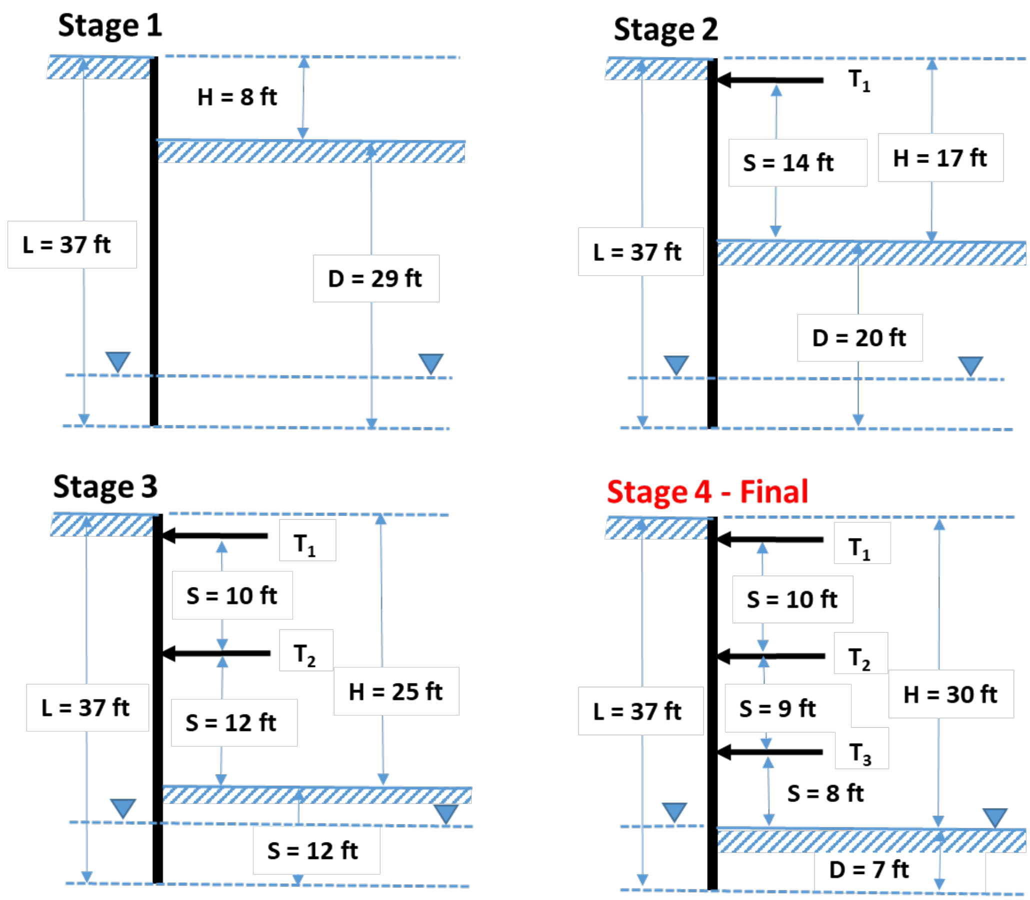


Figure 5-45 Case 8 Staged construction analysis results.

Case 8, Step 6: Calculation of Strut Loads Using the Terzaghi-Peck Apparent Pressure Diagram

As discussed in Case 7, the Terzaghi and Peck's "*apparent pressure diagrams (envelopes)*" (TP) are commonly used to estimate strut loads braced excavation in the United States. According to Peck, Hanson, and Thornburn (1974), "*an apparent pressure envelope represents a fictitious pressure distribution for estimating the maximum strut loads in a system of bracing. It does not, however, indicate the magnitude or distribution of loading on the sheet piling or wales.*" As noted above, the results provided in Table 5-25 are based on a Rankine analysis.

The Terzaghi-Peck apparent pressure diagrams are shown in Figure 5-46. The selection for the clay distribution is based on the clay's stability number, N_s, which is calculated as follows:

$$N_s = \frac{\gamma H}{c} = \frac{(118.37\ pcf)(30ft)}{417.6\ psf} = 8.5$$

For stability numbers greater than 4, Figure 5-46(c) is used to estimate the strut loads.

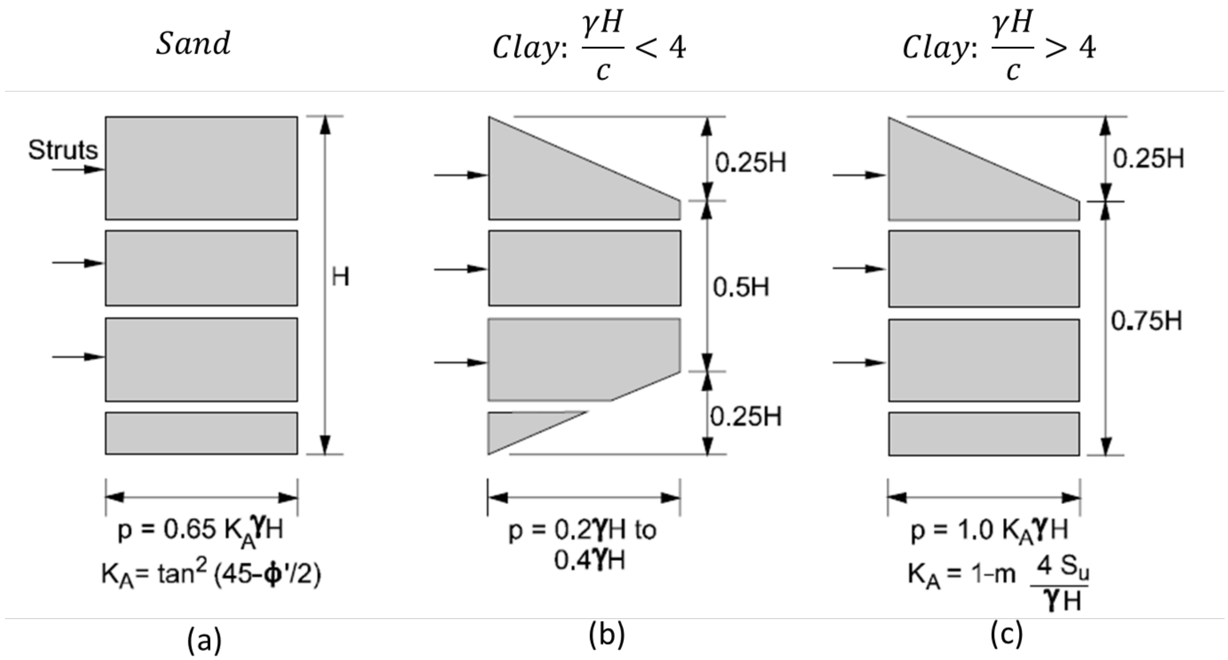


Figure 5-46 Case 8 Terzaghi-Peck apparent pressure diagrams.

The Terzaghi-Peck pressure diagram for Case 8 is shown in Figure 5-47.

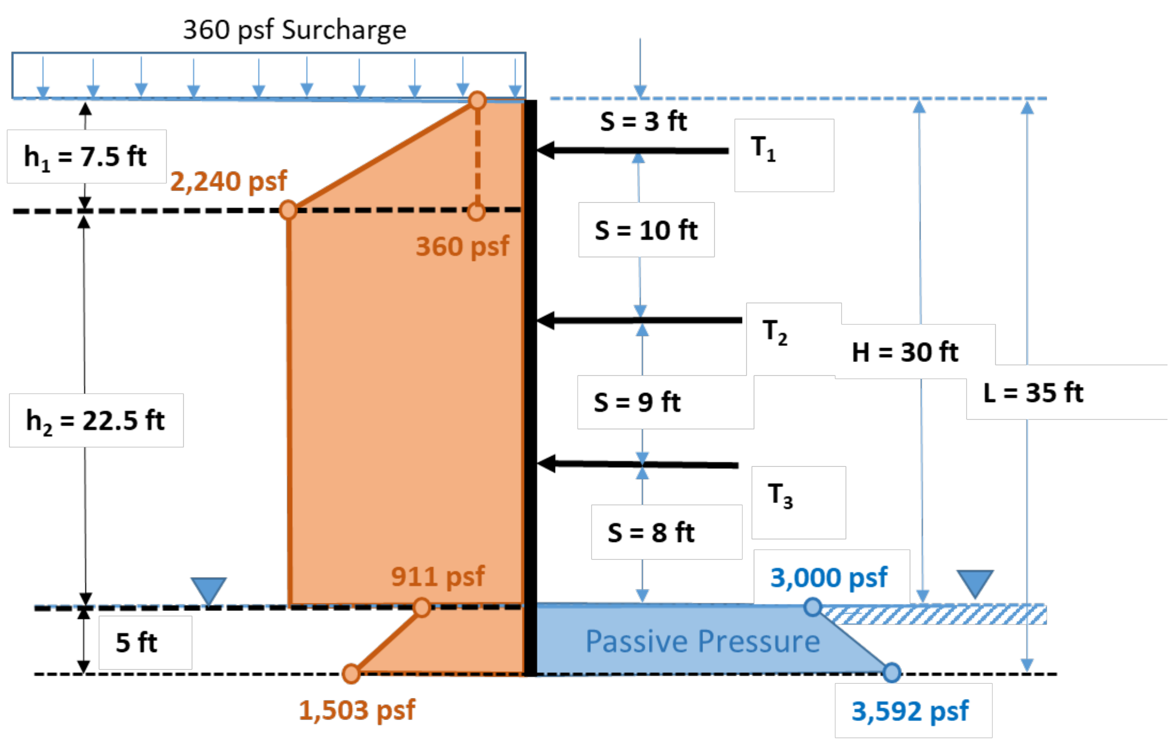


Figure 5-47 Case 8 Terzaghi-Peck pressure diagram for strut load analysis.

Case 8, Step 7(a): Calculate Terzaghi-Peck apparent earth pressure distribution

The active earth pressure coefficient, K_a, is determined as follows with m = 1 except where the excavation is underlain by soft normally consolidated clay:

$$K_a = 1 - m\frac{4S_u}{\gamma H} = 1 - (1)\frac{(4)(1417.6)}{118.4(30)} = 0.53$$

Terzaghi-Peck Pressure: $\sigma_{a,\,soil} = K_a\gamma h = 0.529(1)(118.37)(30) = 1{,}880$ psf/ft

$\sigma_{a,\,surcharge} = (360)(1) = 360$ psf/ft

Total pressure above dredge line: $\sigma_{a,\,total} = \sigma_{a,\,soil} + \sigma_{a,\,surcharge} = 1{,}880 + 360 =$ 2,240 psf/ft

Case 8, Step 7(b): Calculate Strut #1 and #2 Loads

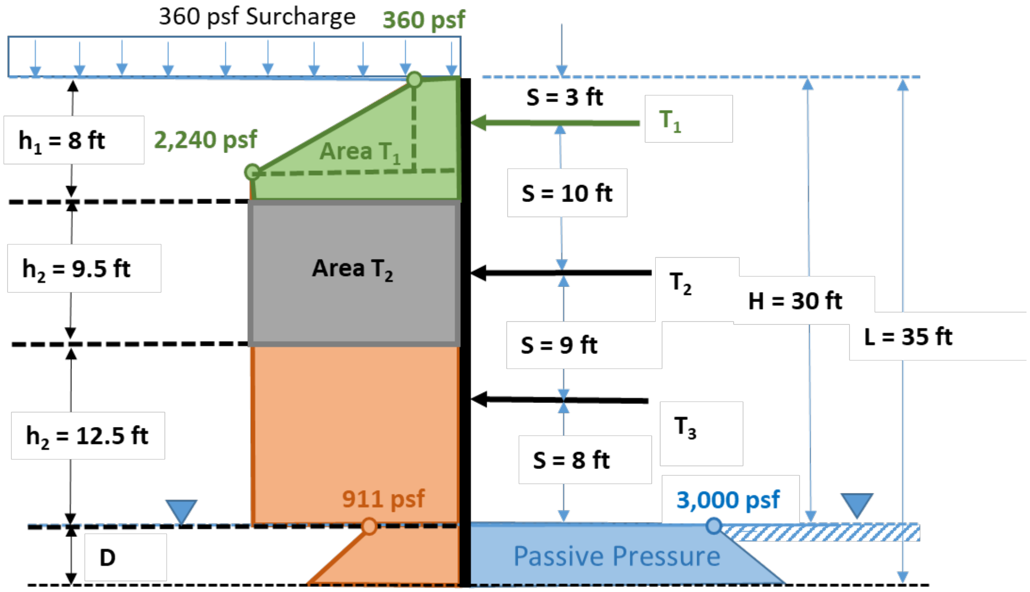


Figure 5-48 Case 8 Calculation area for Strut #1 load.

$T_1 = (0.5)(2{,}240 - 360)(7.5) + (360)(7.5) + (8 - 7.5)(2{,}240.1) =$ 10,870 lbs/ft

$T_2 = (5 + 4.5)(2{,}240) =$ 21,281 lbs/ft

T_1 = 10,870 lbs/ft

$T_{1,\,SupportIT}$ = 10,859 lbs/ft

T_2 = 21,281 lbs/ft

$T_{2,\,SupportIT}$ = 21,211 lbs/ft

Case 8, Step 7(c): Calculate Strut #3 Load

The load on strut #3 is calculated using a combination of the area distribution method and the value obtained by treating the span below the lowest strut as a single supported span similar to the previous cases. FOS = 1.5 is used in estimating the depth of the sheet pile which determines the load on strut #3. As noted above the net pressure method is used. To illustrate the difference between the gross pressure distribution and the net pressure distribution, the gross pressure distribution shown is shown in Figure 5-49, while the "net pressure" distribution is shown in Figure 5-50.

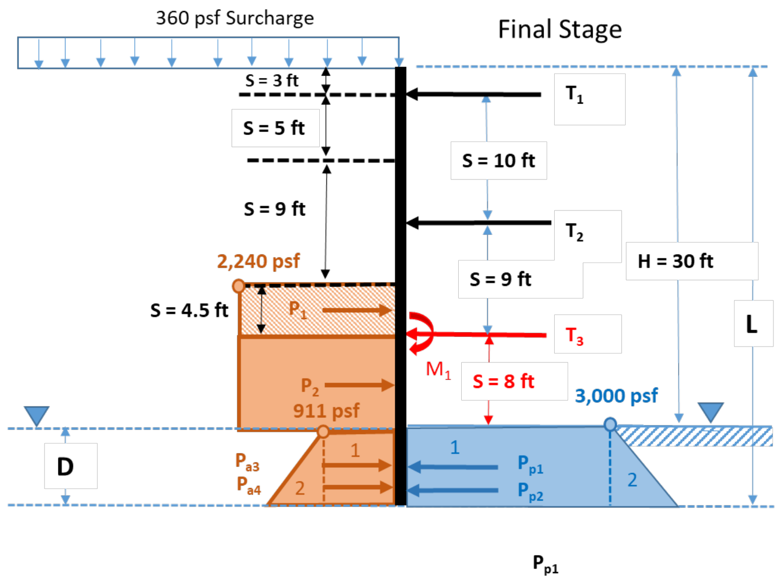


Figure 5-49 Case 8 Calculation area for Strut #3 load.

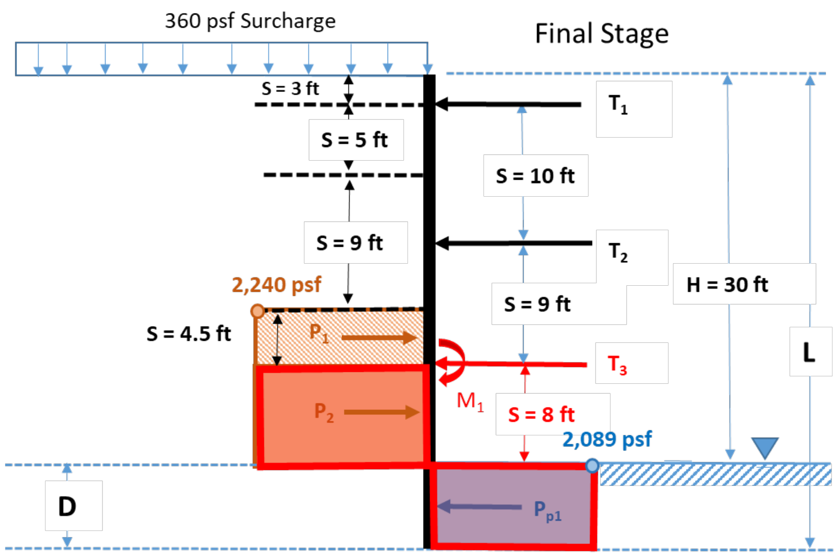


Figure 5-50 Case 8 Net Pressure distribution.

To calculate the strut load T_3, the embedment D is calculated by taking the summation of moments (M_1) at T_3 and then equating the restoring and disturbing moments, $M_R = M_D$. The embedment depth D is used to estimate the earth pressure acting on the sheet pile below the dredge line. Note, that the calculated D is NOT the final depth of the sheet pile that was determined above and shown in Table 5-26. The depth, D, is used to estimate the load on Strut #3 as shown below.

<u>Loads lbs/ft)</u>

$P_{a1} = (2{,}240)(4.5) = 10{,}080$

$P_{a2} = (2{,}240)(8) = 17{,}920$

$P_{p1} = (3000 - 911)D = 2089D$

Note, the moment is taken for the forces acting below the strut.

<u>Moment Arm (ft)</u>

$L_{a2} = 8/2 = 4$

$L_{p1} = 8 + 0.5D$

$\Sigma M_D = P_{a2}\, L_{a2} = 17{,}920(4) = 71{,}680$

$\Sigma M_R = P_{p1}\, L_{p1} = (2089D)(8 + 0.5D) = 16{,}712D + 1{,}044.5D^2$

$M_D = M_R$

$1{,}044.5D^2 + 16{,}712D - 71{,}680 = 0$

<u>D = 3.52 ft</u>

<u>$D_{SupportIT}$ = 3.48 ft</u>

<u>Strut #3 load:</u>

$T_3 = [P_{a1} + P_{a2}] - P_{p1} = [10{,}080 + 17{,}920] - [2089D]$

$T_3 = [10{,}080 + 17{,}920] - [7{,}353]$

<u>T_3 = 20,650 lbs/ft</u>

<u>$T_{3,SupportIT}$ = 20,741 lbs/ft</u>

Strut Load Comparison:

Table 5-26 Case 8 Comparison of hand calculations to SupportIT strut load calculations.

	SupportIT (lbs/ft)	Hand Calculations (lbs/ft)
Strut #1	**10,859**	**10,870**
Strut #2	**21,211**	**21,281**
Strut #3	**20,673**	**20,650**

Case 8, Step 9: Heave Analysis

Construction in soft clay requires that a heave analysis is conducted to assess the potential for heaving to occur at the excavation's bottom. According to the US Sheet Pile Manual (1984), the conventional analysis method for investigating heave, developed by Karl Terzaghi, is commonly used today. The US Steel Sheet Pile Manual Figure 60, shown in Figure 5-51, provides the parameters used in the analysis.

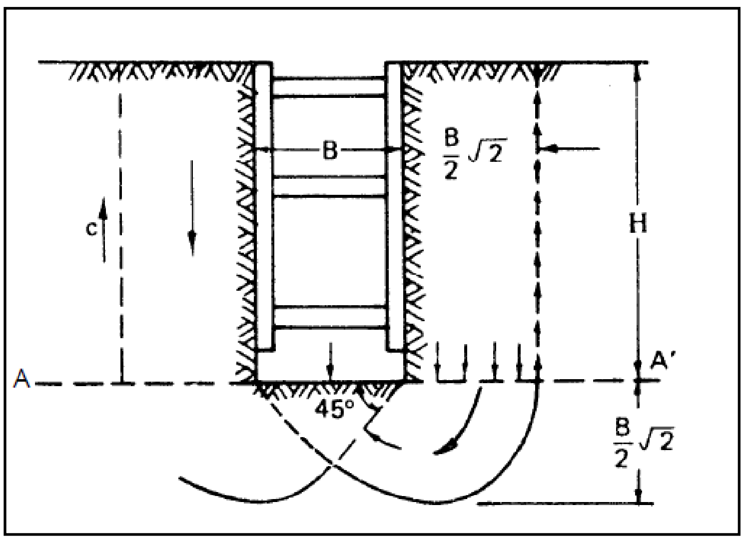


Fig. 60 - Diagram illustrating assumed mechanism for failure by heave of the bottom of a deep excavation

Figure 5-51 Case 8 Terzaghi analysis for soft clay bottom heave.

The failure model shown in Figure 5-51 assumes that a vertical column of soil along the sheet piling exerts a pressure on the horizontal plane A-A'. When the pressure exerted by this soil column exceeds the bearing capacity of the soil beneath the plane A-A', a bearing capacity failure occurs, resulting in the excavation's bottom heaving and settlement of the surrounding ground surface.

Based on this failure model, the depth of excavation at which heave will occur can be expressed by the following equation for excavations in which the height (H) is less than the excavation's width (B) or H < B;

$$H_c = \frac{5.7c}{\gamma - \sqrt{2}(\frac{c}{B})}$$

where H_c = excavation's critical height
H = height of excavation
B = excavation width
γ = soil's unit weight
c = soil's undrained shear strength

According to the US Sheet Pile Manual, "*a factor of safety of 1.5 applied to the soil cohesive shear strength is normally recommended. This method of analysis gives reliable results for excavations in which the width of the cofferdam is larger than the depth of excavation and the cofferdam is very long*".

Since the model only allows one cohesion value, three cases are considered for an excavation width of 15 ft and 50 ft with cohesion values of 417, 960, and 1,500 psf.

(a) c = 417 psf (Soft Clay)
(b) c = (417 + 1,500)/2 = 960 psf (Undrained shear strength average for soft and stiff layer)
(c) c = 1,500 psf (Firm Clay)

B = 15 ft

$$H_{c=417} = \frac{5.7(417)}{118.4 - \sqrt{2}(\frac{417}{15})} = 85\ ft$$

$$H_{c=960} = \frac{5.7(960)}{118.4 - \sqrt{2}(\frac{960}{15})} = 196\ ft$$

$$H_{c=1,500} = \frac{5.7(1,500)}{118.4 - \sqrt{2}(\frac{1,500}{15})} = 306\ ft$$

B = 50 ft

$$H_{c=417} = \frac{5.7(417)}{118.4 - \sqrt{2}(\frac{417}{50})} = 22\ ft$$

$$H_{c=960} = \frac{5.7(960)}{118.4 - \sqrt{2}(\frac{960}{50})} = 60\ ft$$

$$H_{c=1,500} = \frac{5.7(1,500)}{118.4 - \sqrt{2}(\frac{1,500}{50})} = 112\ ft$$

The factor of safety can be determined by dividing the clay's critical height by the height of the excavation. To account for the 360 psf surcharge, an additional height of three feet (360 psf/118.4 pcf = 3.04 ft) for a total of 3 feet:

$$FOS = \frac{H_c}{H}$$

B = 15 ft

$$FOS_{c=417} = \frac{85}{33} = 2.6$$

$$FOS_{c=960} = \frac{196}{33} = 5.9$$

$$FOS_{c=1,500} = \frac{306}{33} = 9.3$$

B = 50 ft

$$FOS_{c=417} = \frac{22}{33} = 0.67$$

$$FOS_{c=960} = \frac{60}{33} = 1.82$$

$$FOS_{c=1,500} = \frac{112}{33} = 3.4$$

The US Sheet Pile Manual recommends a FOS greater than 1.5. From the factor of safety calculations above, it can be seen that an excavation width of 50 feet would require the that clay soil below the excavation have a strength greater that 417 pcf. Using the above equations and back calculating this strength, a clay strength of approximately 735 psf would be required for a factor of safety equal to 1.3.

In cases where the excavations height is less than its width (H < B), the US Sheet Pile Manual provides an additional analysis developed by Bjerrum and Eide (1956) where the shape of the excavation can be considered.

5.2.9 Case 9 – Cantilevered Soldier Pile TERS in Cohesionless Soil

Cantilevered soldier pile and lagging walls are designed using similar methods that are used for cantilever walls in Cases 1 through 4. Soldier pile and lagging walls, however, are constructed with stiff members, such as steel beams, that are vertically driven and/or grouted into drilled holes at a given spacing, S, with wood lagging placed horizontally to hold the soil between the stiff supports in place. It's important to note that soldier piles are discrete support elements whereas sheet piling is continuous.

A key design parameter for soldier piles in cohesionless soils is the soldier pile's "*effective width*" (W_{eff}), which is a function of the diameter of the drill holes used for the stiff support such as a steel H or I-pile. Based on a series of experiments in 1970, it was shown that below the dredge line the passive resistance on the pile in cohesionless soil acts over a greater width than the pile's flange width "b" or the grouted drill hole diameter, "W" as shown in Figure 5-52. Consequently, the width of the soldier pile or drill hole is increased by an adjustment or "*passive arching*" factor "A." The adjustment factor "A" is based on the soil friction angle, e.g., $\phi = 35°$, as follows

$$A = 0.08\phi = 0.08(35°) = 2.80$$

$$W_{eff} = AW$$

The value of "A" can vary between 1 and 3 with a limit of the spacing between the piles "S" in which $W_{eff} < s$.

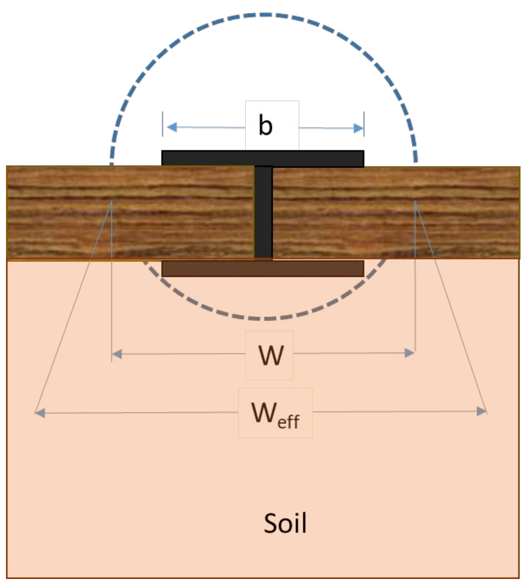


Figure 5-52 Case 9 Soldier pile effective width.

The SupportIT Manual provides the following three methods for the design of soldier pile and lagging walls: (1) the adjustment factor, A, applied to passive side only (New York Department of Transportation: Flexible Wall Systems), (2) the adjustment factor, A, applied to both the active and passive sides (California Trenching and Shoring Manual), and (3) AASHTO method (1992 publication, Standard Specifications for Highway Bridges). The AASHTO design method will be used for Case 9.

The AASHTO method recommends that the passive resistance starts at a depth 1.5 times the pile's effective width. AAHSTO further recommends that the computed pile depth be increased by 30% for temporary walls.

<u>Case 9, Step 1: Define the dimensions and soil properties to be analyzed for the cantilever wall</u>

The following design example will show the calulations for an 8-foot high cantilever soldier pile wall with a 360 psf foot surcharge, as shown in Figure 5-53. The soldier pile parameters are also provided in Figure 5-53.

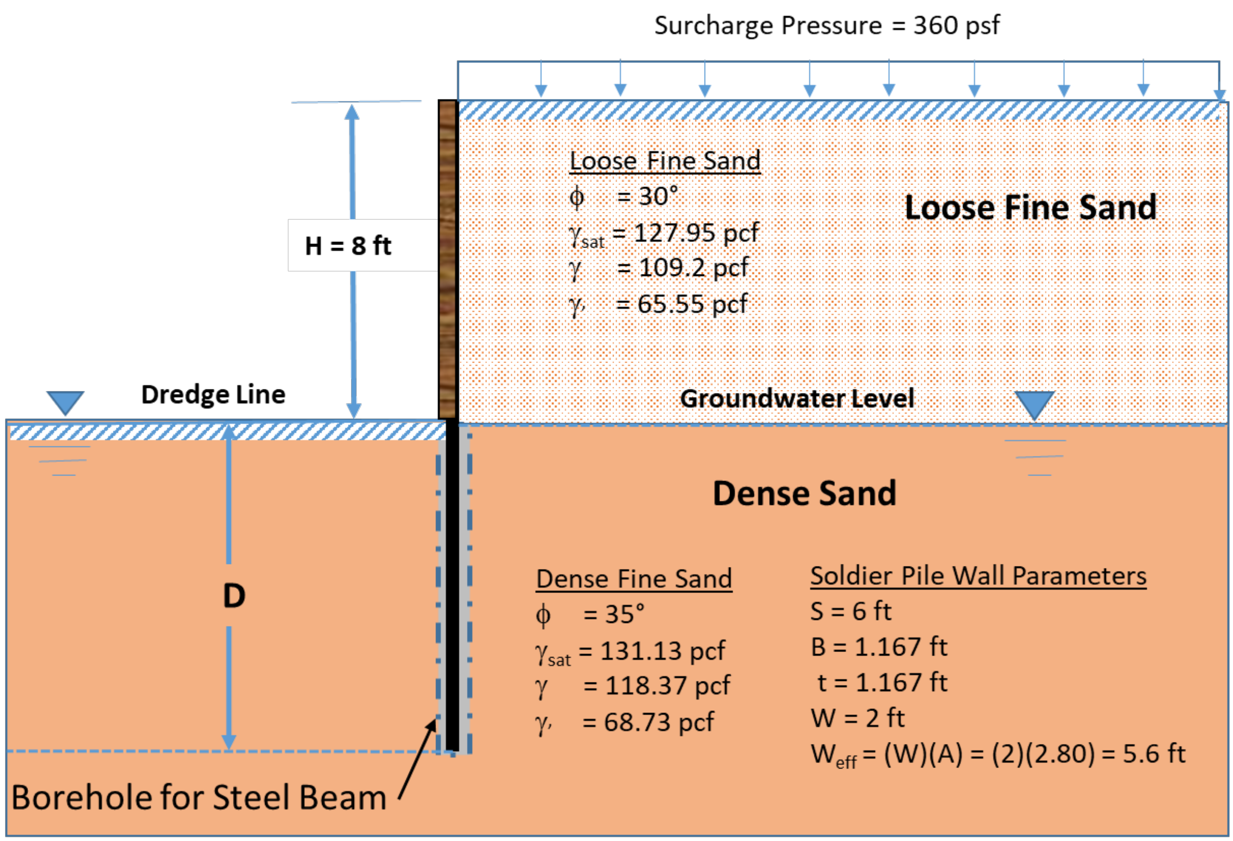


Figure 5-53 Case 9 Soldier pile parameters.

Case 9, Step 2: Calculate Active and Passive Earth Pressures and Forces

Loose Fine Sand:

$$K_a = tan^2\left(45^\circ - \frac{\phi'}{2}\right) = tan^2\left(45^\circ - \frac{30^\circ}{2}\right) = \mathbf{0.33}$$

$$K_p = tan^2\left(45^\circ + \frac{\phi'}{2}\right) = tan^2\left(45^\circ + \frac{30^\circ}{2}\right) = \mathbf{3.00}$$

Dense Fine Sand:

$$K_a = tan^2\left(45^\circ - \frac{\phi'}{2}\right) = tan^2\left(45^\circ - \frac{35^\circ}{2}\right) = \mathbf{0.27}$$

$$K_p = tan^2\left(45^\circ + \frac{\phi'}{2}\right) = tan^2\left(45^\circ + \frac{35^\circ}{2}\right) = \mathbf{3.69}$$

The "Gross" active and passive pressures acting on the soldier pile wall are shown in Figure 5-54 to an arbitrary depth of 30 feet based on the following calculations:

Active Pressures:

Surcharge pressure above Dredge Line: $\sigma_{a1'} = K_a(360) = 0.33(360) = \underline{118.8 \text{ psf}}$

Lateral Pressure above Dredge Line: $\sigma_{a2'} = 118.8 + K_a\gamma H = 0.33(109.2)(8) = \underline{407.9 \text{ psf}}$

Lateral Pressure at Dredge Line: $\sigma_{a1'} = K_a(360 + 8(109.2)) = 0.27(1{,}233.6) = \underline{333.1 \text{ psf}}$

Lateral Pressure at @ 30 ft: $\sigma_{a2'} = 333.1 + 0.27(68.73)(30) = \underline{889.8 \text{ psf}}$

Passive Pressures:

Lateral Pressure at a depth of 30 ft: $\sigma'_{p1} = K_p\gamma' H = 3.69(68.73)(30) = \underline{7{,}608 \text{ psf}}$

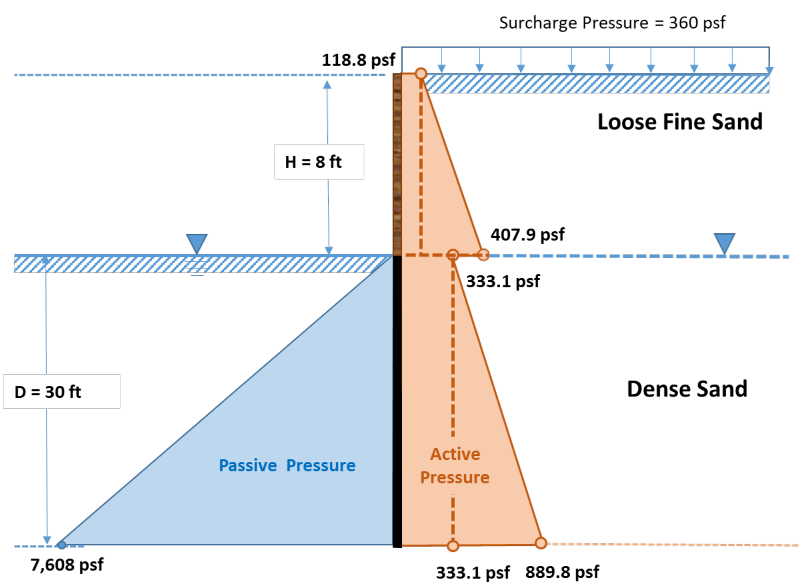


Figure 5-54 Case 9 Gross pressure acting on the soldier pile.

Since the soldier pile wall is a cantilever wall, the *simplified method* is used to determine the embedment depth and maximum bending moment on the pile. In the *simplified method*, the stresses below the pivot point "O" are not considered in the analysis, thus reducing the unknowns to one unknown, the depth D_o. Once the depth D_o is determined, the *simplified method* requires that this depth is increased by 20% ($D_u = 1.2D_o$) to compensate for the pressure acting below the pivot point, as shown in Figure 5-55.

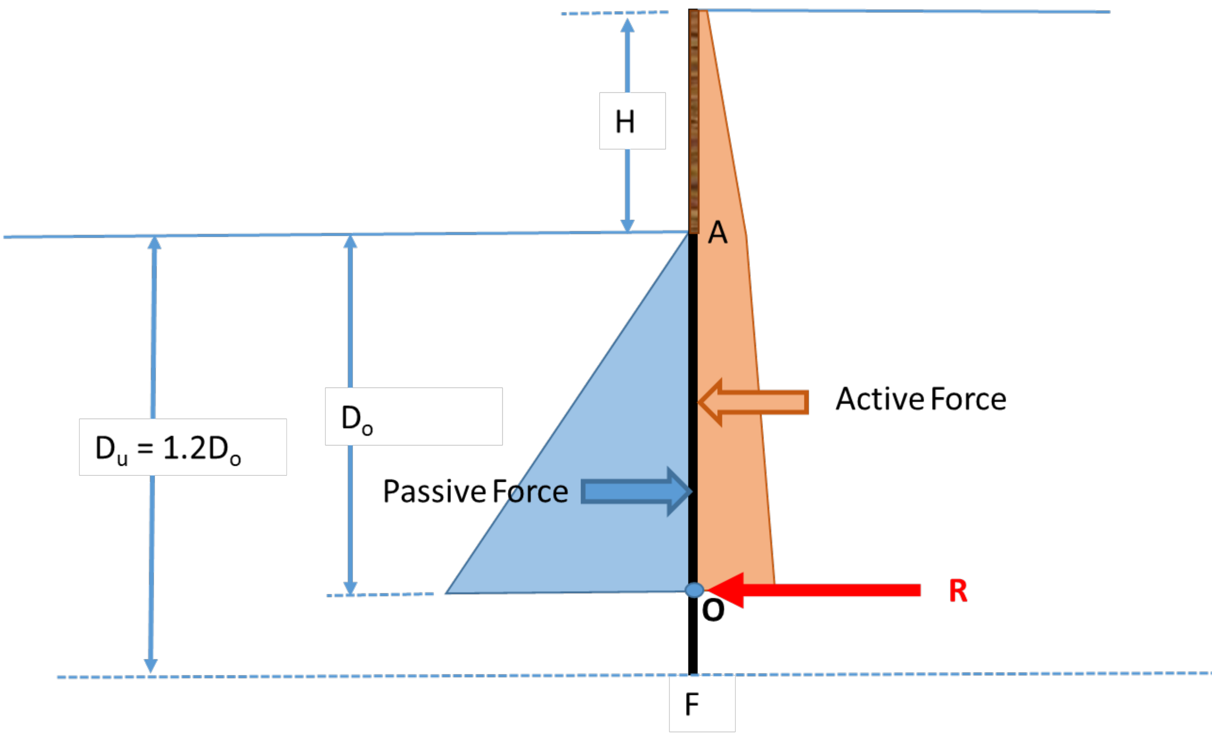


Figure 5-55 Case 9 Simplified method showing an increase of D_o by 1.2 for the embedment depth, D_u.

<u>Case 9, Step 3: Calculate Sheet Pile Unfactored Embedment Depth, D_u</u>

In the *simplified method* of analysis, equations for the forces acting on the sheet pile wall and their locations are based on the depth, D_o, below the dredge line to the pivot point, "O." The calculations are provided below based on the active and passive stresses and resultant forces shown in Figure 5-56.

Important Points:
For a soldier pile wall, the active forces acting on the wall above the dredge line are based on the distance between the soldier piles, S, and not on a "per foot" basis." For example, active force P_{a1} is calculated as follows:

$$P_{a1} = ½ [K_a\gamma H](H)(S)$$

The *active forces* below the dredge line are based on the width of the pile, “W” as follows:

$$P_{a2} = [K_a\gamma H](H)(W)$$

The *passive forces* below the dredge line, however, are based on the pile’s “effective width,” W_{eff}, as shown below:

$$P_{p1} = \tfrac{1}{2}\,[K_p\gamma' H](H)(W_{eff})$$

The AASHTO design method requires that the passive resistance starts at a depth of 1.5 times the pile’s “effective width,” W_{eff} as illustrated in Figure 5-56.

The depth below the dredge line where the passive pressure is assumed to begin is as calculated as follows:

$$W_{eff} = (0.08)(\phi)(W) = 0.08(35°)(2ft) = 5.6 \text{ ft}$$

$$\text{Passive Pressure Depth} = 1.5(W) = 1.5(2.0) = 3.0 \text{ ft}$$

$$\text{Passive Pressure at 3 ft} = (3.69)(3)(68.73) = 760.8 \text{ psf}$$

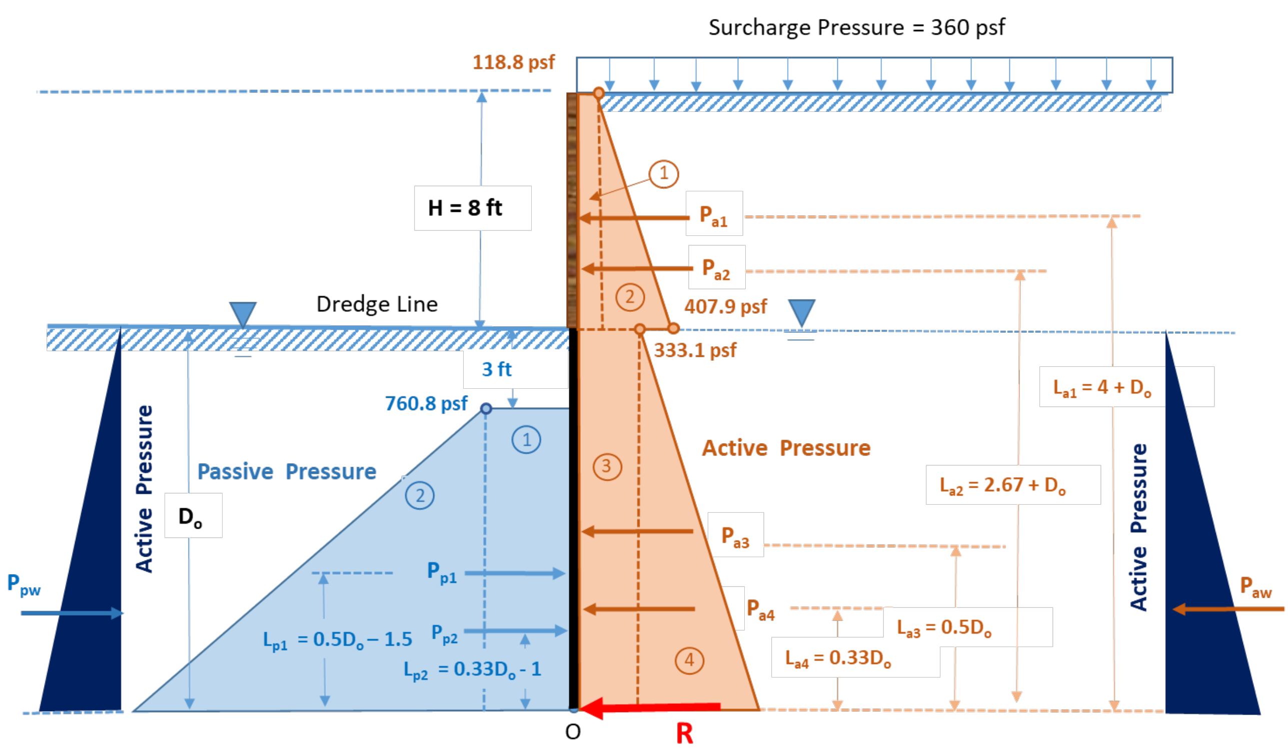


Figure 5-56 Case 9 Gross pressures, forces, and force locations acting on the sheet pile wall.

Forces and Force Locations in Terms of D_o

Active Forces, lbs, and Location

Active Force above dredge line P_{a1}:	$P_{a1} = (118.8)(8) \times (6\text{ ft}) = 5{,}702.4$ lbs
Active Force above dredge line P_{a2}:	$P_{a2} = 0.5[0.33(109.2)(8)(8)] \times (6\text{ ft}) = 6{,}918.9$ lbs
Active Force below dredge line P_{a3}:	$P_{a3} = (333.1)(D_o) \times (2\text{ ft}) = 666.3D_o$
Active Force below dredge line P_{a4}:	$P_{a4} = 0.5(0.27)(68.73)(D_o)(D_o) \times (2\text{ ft}) = 18.56D_o^2$
Active Force below dredge line P_{aw}:	$P_{aw} = 0.5(62.4)D_o^2 = 31.2D_o^2$
Active Force Location, L_{a1}	$L_{a1} = 4 + D_o$
Active Force Location, L_{a2}	$L_{a2} = 0.33(8) + D_o = 2.67 + D_o$
Active Force Location, L_{a3}	$L_{a3} = 0.5D_o$
Active Force Location, L_{a4}	$L_{a4} = 0.33D_o$
Active Force Location, L_{aw}	$L_{aw} = 0.67\ D_o$

Passive Force, lbs, and Location

Passive Force, P_{p1}:	$P_{p1} = [760.8)(D_o - 3)] \times (5.6\text{ ft})$
	$P_{p1} = 4{,}260D_o - 12{,}782.1$
Passive Force, P_{p2}:	$P_{p2} = 0.5[3.69(68.73)(D_o - 3)](D_o - 3] \times (5.6\text{ ft})$
	$P_{p2} = 710.1D_o^2 - 4{,}260.7D_o - 6{,}391.1$
Passive Water Force, P_{pw}:	$P_{pw} = 0.5(62.4)D_o^2 = 31.2D_o^2$
Passive Force Location, L_{p1}	$L_{p1} = 0.5(D_o - 3) = 0.5D_o - 1.5$
Passive Force Location, L_{p2}	$L_{p2} = 0.33(D_o - 3.0) = 0.33D_o - 1$
Passive Water Force Location, L_{pw}	$L_{pw} = 0.67\ D_o$

The determination of the sheet pile embedment depth and maximum bending moment is based on an unfactored condition, i.e., a factor of safety (*FOS*) equal to one.

To determine the embedment depth, the summation of the moment about the pivot point O is conducted and equated to *FOS* = 1, as shown below.

$$\frac{\sum M_{restoring}}{\sum M_{disturbing}} = FOS = 1.0$$

where

$$\Sigma M_{disturbing} = P_{a1}L_{a1} + P_{a2}L_{a2} + P_{a3}L_{a3} + P_{a4}L_{a4}$$

$$\Sigma M_{restoring} = P_{p1}L_{p1} + P_{p2}L_{p2}$$

D_o is determined by placing the above equations into an EXCEL sheet as shown below in

Table **5-27**. The spreadsheet is set up to input a depth, D_o, which then calculates the *FOS*. For a *FOS* = 1.0, the embedment depth D_o was determined to be 10.49 ft.

Table 5-27 Case 9 Embedment depth, D_o, for a *FOS* = 1.0

Calculation of Sheet Pile Depth, D_0 (FOS = 1.0)			
Depth of Sheet Pile, D_o		10.49	
FOS		1.00	
Restoring Moment:		Disturbing Moment:	
P_{p1}	31,898	P_{a1}	5702.4
L_{p1}	3.743265035	L_{a1}	14.49
P_{p2}	39,801	P_{a2}	6,919
L_{p2}	2.460554923	L_{a2}	13.2
P_{pw}	3430.980162	P_{a3}	6,987
L_{pw}	7.025975146	L_{a3}	5.2
M_r	241,441	P_{a4}	2,041
		L_{a4}	3.5
		P_{aw}	3430.9802
		L_{aw}	7.0259751
		M_d	241,441

To account for using the *simplified method*, D_o must be multiplied by a factor of 1.2 as follows:

D_u = 1.2 x D_o = 1.2 x 10.49 = 12.59 ft

$D_{u,\ SupportIT}$ = 12.57 ft

The *simplified method* suggests that a check is conducted to make sure that the horizontal force below the pivot point is greater than the resultant "R" shown in Figure 5-56. This step, however, is generally not conducted because it has been found that the additional length generally concludes with acceptable results.

AASHTO recommends that the piles be increased by 30% to 50% for temporary works. Therefore, the total embedment depth for a 30% increase is

D_f = 12.59 ft x 1.30 = 16.4 ft

D_f = 12.59 ft x 1.50 = 18.9 ft

Total pile length, L = 8 + 16.4 = 24.4 ft (30%)

Total pile length, L = 8 + 18.9 = 26.9 ft (50%)

Case 9, Step 4: Calculate Maximum Bending Moment to Determine Soldier Pile Size

The maximum bending moment is located at the zero shear point a distance "Y" from the dredge line as shown in Figure 5-57. The active and passive forces acting on the wall are calculated as follows:

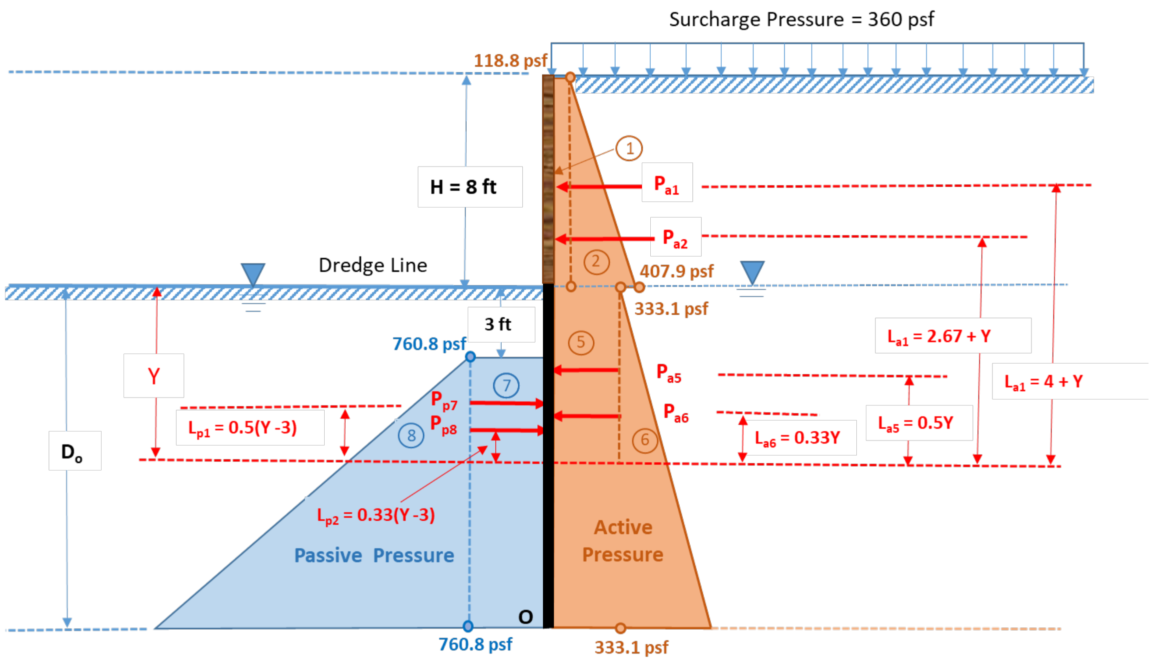


Figure 5-57 Case 9 Calculation of maximum bending moment.

Active Forces:

P_{a1} = [(118.8)(8)] x 6 ft = 5,702.4
P_{a2} = ½ [0.33(109.2)(8)](8) x 6 ft = 6,918.9
P_{a5} = [(333.1)(Y) x 2 ft = 666.2(Y)
P_{a6} = ½ [0.27(68.73)(Y)](Y) x 2 ft = 18.56(Y^2)

Passive Forces:

P_{p7} = (760.8)(Y – 3) x 5.6 ft = 4,260.7(Y) – 12,782
P_{p8} = ½ [3.69(68.73)(Y - 3)](Y – 3) x 5.6 = 710.1(Y^2) – 4,261(Y) + 6,391

Solve for Y by summing active and passive forces:

$\Sigma P_p = \Sigma P_a$

$P_{p7} + P_{p8} = P_{a1} + P_{a2} + P_{a5} + P_{a6}$
$691.6Y^2 – 666.2Y – 19{,}012 = 0$

$Y = 5.75$ ft
$Y_{SupportIT} = 5.76$ ft

Maximum Moment Depth:

Depth = 8 + 5.75 = 13.75 ft
$Depth_{SupportIT}$ = 13.76 ft

Calculate the maximum moment at point "Y":

$\Sigma M = [P_{a1}L_{a1} + P_{a2}L_{a2} + P_{a4}L_{a4} + P_{a4}L_{a4}] – [P_{p1}L_{p1} + P_{p2}L_{p2}]$

Pressures, P
$P_{a1} = 5{,}702.4$
$P_{a2} = 6{,}918.9$
$P_{a5} = 666.2(5.75) = 3{,}831$
$P_{a6} = 18.56(5.75^2) = 614$
$P_{p1} = 4{,}260.7(5.75) – 12{,}782 = 11{,}716$
$P_{p2} = 710.1(5.75^2) – 4{,}261(5.75) + 6{,}391 = 5{,}368$

Length, L
$L_{a1} = Y + 4 = 5.75 + 4 = 9.75$
$L_{a2} = Y + 2.67 = 5.75 + 2.67 = 8.42$
$L_{a5} = 0.5(Y) = (0.5)(5.75) = 2.88$
$L_{a6} = 0.33(Y) = (0.33)(5.75) = 1.90$
$L_{p1} = 0.5(Y – 3) = 0.5(5.75 – 3) = 1.38$
$L_{p2} = 0.33(Y – 3) = (0.33)(5.75 – 3) = 0.91$

Summation of moments about Y = 13.75 ft:

$\Sigma M = [P_{a1}L_{a1} + P_{a4}L_{a4} + P_{a5}L_{a5} + P_{a6}L_{a6}] – [P_{p1}L_{p1} + P_{p2}L_{p2}]$

$\Sigma M = [(5{,}702)(9.75) + (6{,}918.9)(8.42) + (3{,}831)(2.88) + (614)(1.90)] – [(11{,}716)(1.38) + (5{,}368)(0.91)]$

<u>M = 105,269 ft-lbs/ft</u>

<u>$M_{SupportIT}$ = 105,226 ft-lbs/ft</u>

<u>Case 9, Step 5: Sheet Pile Selection</u>

The maximum moment is equal to 105,269 ft-lbs/ft. Assuming a regular carbon grade steel with a yield strength f_s = 25 ksi, a required section modulus is determined as follows

Required section modulus = M/f_s = [105,269 ft-lbs/ft x 12/in/ft]/25,000 = 50.5 in^3

A section modulus of the pile will need to be greater than 50 in^3.

<u>Case 9, Step 6: Final Sheet Pile Depth Using a FOS = 1.5</u>

In Step 3, AASHTO recommends that the piles be increased by 30% to 50% for temporary works providing a final toe embedment depth, D_f as follows:

D_f = 12.59 ft x 1.30 = 16.4 ft

D_f = 12.59 ft x 1.50 = 18.9 ft

Total pile length, L = 8 + 16.4 = <u>24.4 ft</u> (30%)

Total pile length, L = 8 + 18.9 = <u>26.9 ft</u> (50%)

A FOS = 1.5 can also be calculated using the CP2 method. Using the EXCEL shown in Table 5-27, the FOS can be set to 1.5. The result is shown in Table 5-28 is D_o = 12.84 ft. Multiplying this value by 1.2 provides the D_f, as shown below.

D_f = 1.2D_o = 1.2(12.84) = <u>15.41 ft</u>

$D_{f,SupportIT}$ = <u>15.36 ft</u>

Table 5-28 Case 9 Final embedment depth, D_f, for a FOS = 1.50.

Calculation of Sheet Pile Depth, D_0 (FOS = 1.5)			
Depth of Sheet Pile, D_o		12.84	
FOS		1.50	
Restoring Moment:		Disturbing Moment:	
P_{p1}	41,933	P_{a1}	5702.4
L_{p1}	4.920842883	L_{a1}	16.84
P_{p2}	68,782	P_{a2}	6,919
L_{p2}	3.237756303	L_{a2}	15.5
P_{pw}	5145.157472	P_{a3}	8,556
L_{pw}	8.603929464	L_{a3}	6.4
M_r	473,310	P_{a4}	3,061
		L_{a4}	4.2
		P_{aw}	5145.1575
		L_{aw}	8.6039295
		M_d	315,540

Comparison to the SupportIT Software:

Table 5-29 Case 9 Comparison of hand calculations to SupportIT strut load calculations.

	SupportIT **(Total pile length, ft, ≅ L)**	**Hand Calculations** **(Total pile length, ft, ≅ L)**
Maximum soil pressure at dredge line, (psf/pile)	407.1	407.9
Zero Shear Location, ft	13.76	13.75
Maximum Moment, (ft-lbs/ft), FOS = 1.0	105,226	105,269
Soldier Pile Embedment, FOS = 1.00, D_u (ft)	12.57	12.59
AASHTO 30% Embedment Length, D_f (ft) (L)	16.3 (24.3)	16.4 (24.4)
AASHTO 50% FOS Embedment Length, D_f (ft) (L)	18.9 (26.9)	18.9 (26.9)
CP2 FOS Embedment Length, D_f (ft) FOS = 1.5 (L)	15.35 (23.3)	15.41 (23.4)

6 Structural Steel Design for Temporary Earth Retention Systems

Aside from sheet pile design (previously covered in Chapter 4 design examples), many of the structural steel design issues that arise during the design of support of excavations are associated with internally braced support systems. Internal bracing systems used in some support of excavations require the same level of design effort, thorough attention to the design of members and connections, as well as safety considerations required of a permanent structural system. A licensed professional engineer knowledgeable in structural steel analysis and design must prepare design drawings and specifications for bracing systems as well as any changes in the design that are made during the detailing, fabrication, and construction process. The professional engineer, or team of engineers, must possess both adequate geotechnical knowledge to determine design loads for the support of excavation system as well as the structural knowledge necessary to fully explore the applicable limit states and serviceability requirements for the members making up the soil load resisting system. Simply reusing members and materials from previous similar excavations is not always a safe practice due to changes in geotechnical loads, spans, and detailing that will affect the capacity and safety of the new system. Each new application *requires* stamped calculations that demonstrate the safety of all members and connections for the proposed application.

In this section of the guide, common structural engineering issues associated with support of excavations will be presented. This section is in no way intended to replace, or be capable of replacing, the conscientious judgment of a qualified professional engineer with proper training in structural steel analysis and design. Nor is it meant to limit the design approaches that may be taken by the design engineer. Its purpose is to identify structural design issues that commonly arise during the design and construction of these structures and to help the design engineer to develop a successful design to support the given loads safely and economically. While a large percentage of the failures that occur with support of excavations structural systems arise from the use of inadequate geotechnical loads (these issues will hopefully be ameliorated through careful application of the information presented in previous sections of this guide), structural design errors and omissions must also be avoided as they also lead to unsafe conditions. Highlighting the importance of analyzing changes that occur to the design during the detailing, fabrication, and construction process is another important goal as well.

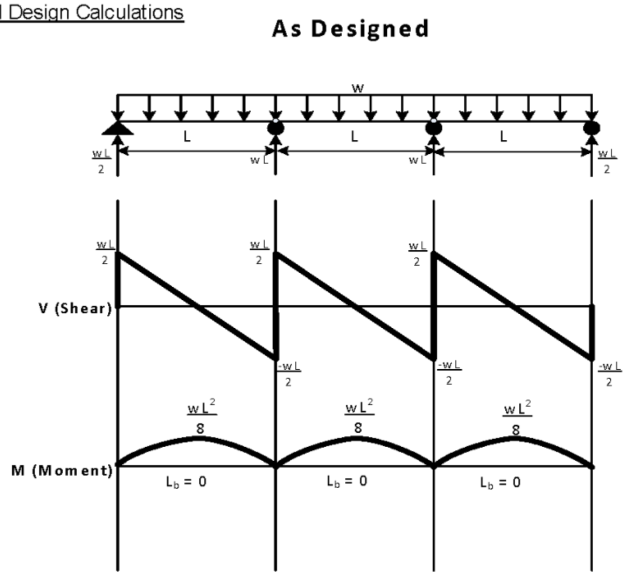
To advance these goals, several example problems are presented in this section. The first set of examples focus on the member design for an internally-braced cofferdam system including walers and struts. In these first examples, the importance of matching analysis and design assumptions and as-build conditions will be demonstrated to highlight common issues that arise during the design and construction process for these systems. Another example examining the effect of connection detailing on the suitability of members follows. Applicable member limit states (including bearing) are checked and sample calculations made to illustrate the expectations for design checks depending on the connection types that are actually installed (as opposed to assumed in the preliminary design by the engineer). Following this example, some sample connection details are presented to illustrate the use of a spacer in the strut system for both moment-restrained and non-moment-restrained connection types. Applicable limit states for each condition are presented. Finally, it is important to consider the deflection of the braced system and the contribution of waler deflections to the total deflection of the cofferdam at its top. A generic single braced frame is depicted to demonstrate this effect with guidance given to consider additional braces or support conditions.

6.1 Member Design Examples

Many of the failures experienced by support of excavation systems are associated with a failure on the part of the design engineer to properly understand and calculate the geotechnical loads on these structures. Assuming that these loads are correctly and conservatively computed, failures are still possible if structural designs and revisions are made with a cavalier attitude and insufficient care and understanding of structural analysis and design issues. In the following example, a braced frame system with two internal braces has been designed for a given geotechnical loading under the assumption that the walers will behave as three simply-supported beams. In this fictitious example, the design engineer assumes that these members remain in single-curvature and that the compression flanges will be braced against lateral-torsional buckling due to (effectively) continuous welds to the sheet piles. Owing to the difficulty of detailing a shear-only splice between the brace and the sheet pile, the detailer changes the layout of the beams to cantilever over the braces into the center span from both sides (moving the shear splices away from the braces) choosing the cantilever dimension suggested in AISC Table 3-22b that minimizes the magnitude of the design moments. It was assumed that the altered design would not create a problem owing to the fact that it reduces moment demand magnitudes. However, the redesign creates reversed curvature conditions and unbraced compression flanges in for the walers in the vicinity of the braces leading to the possibility of lateral-torsional buckling. A check of the reduced flexural capacity of the waler shows that some additional design effort is needed to prevent failure.

Steel Design Example 1: A waler supported by 2 internal braces can be modeled as a 3-span beam. The waler was initially designed as 3 simply-supported beams. However, placement of connections above the brace support was not practical, so the contractor redesigned the waler with the suggested cantilever dimensions given in AISC Table 3-22b to improve constructability under the theory that it would reduce the overall flexural demand, therefore the design change is negligable. Compare the design assumptions to the as-built conditions.

Original Design Calculations



Reference: American Institute of Steel Construction Manual (14th Edition)

$L := 20ft$ $\quad F_y := 50ksi$ $\quad w := 4klf$ (from geotechnical analysis)

Shear:

$$V_a := \frac{w \cdot L}{2} = 40 \cdot kip$$

Moment:

$$M_a := \frac{w \cdot L^2}{8} = 200\, ft \cdot kip$$

From AISC Table 3-2 Use W21x44

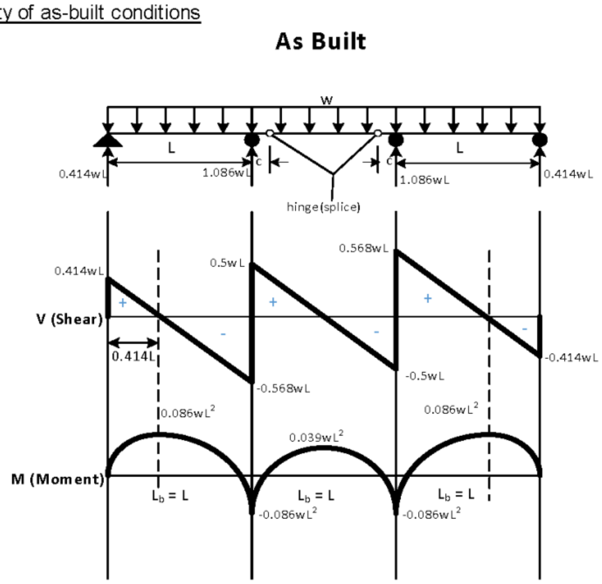
Compression flange continuously braced by sheet pile:

$L_p := 4.45ft$ > $L_b := 0ft$ Yielding Mn = Mp (AISC F2-1)

$M_{p\_\Omega b} := 238ft \cdot kip$ > $M_a = 200\, ft \cdot kip$ OK

$V_{nx\_\Omega v} := 145kip$ > $V_a = 40 \cdot kip$ OK

Capacity of as-built conditions



Given:

$L := 20ft$ $F_y := 50ksi$ $c := 0.22 \cdot L = 4.4\,ft$ (AISC Table 3-22b)

Shear:

$V_a := 0.568 \cdot w \cdot L = 45.44 \cdot kip$

Moment: Associated unbraced length

$M_{apositive} := 0.086 w \cdot L^2 = 137.6\,ft \cdot kip$ $L_{bpositive} := 0ft$

$M_{anegative} := 0.086 w \cdot L^2 = 137.6\,ft \cdot kip$ $L_{bnegative} := 20ft$

For W21x44 (AISC Table 3-2 and Table 1-1)

$L_p := 4.45ft$ $L_r := 13ft$ $c := 1.0$ $S_x := 81.6in^3$

$r_{ts} := 1.6in$ $h_0 := 20.3in$ $J := 0.770in^4$

Positive Moment

$L_p = 4.45\,ft$ > $L_{bpositive} = 0 \cdot ft$ Yielding Mn = Mp (AISC F2-1)

$M_{p\_\Omega b} := 238ft \cdot kip$ > $M_a = 200\,ft \cdot kip$ OK

Negative Moment

$L_{bnegative} = 20\,ft$ > $L_r = 13\,ft$ Elastic Buckling (Zone 3)

$x_A := 5ft$ $x_B := 10ft$ $x_C := 15ft$ $E := 29000ksi$

$$M_A := 0.414w \cdot L_{bnegative} \cdot x_A - \frac{w \cdot x_A^2}{2} = 115.6\,ft \cdot kip$$

$$M_B := 0.414w \cdot L_{bnegative} \cdot x_B - \frac{w \cdot x_B^2}{2} = 131.2\,ft \cdot kip$$

$$M_C := 0.414w \cdot L_{bnegative} \cdot x_C - \frac{w \cdot x_C^2}{2} = 46.8\,ft \cdot kip$$

$$M_{maxA} := M_{apositive} = 137.6\,ft \cdot kip$$

$$C_b := \frac{(12.5M_{maxA})}{2.5 \cdot M_{maxA} + 3 \cdot M_A + 4 \cdot M_B + 3M_C} = 1.268$$

$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 21.743 \cdot ksi$$

$\Omega_b := 1.67$ (AISC F1)

$$M_n := F_{cr} \cdot S_x = 147.852\,ft \cdot kip$$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 88.534\,ft \cdot kip$$

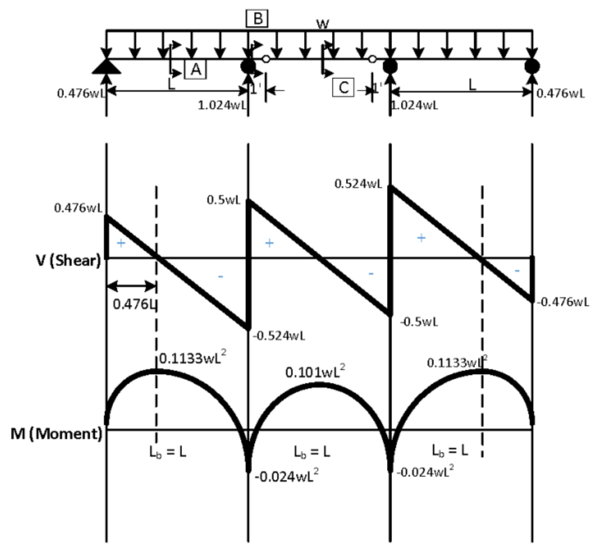
$M_{n\_\Omega b} = 88.534\,ft \cdot kip \quad < \quad M_{anegative} = 137.6\,ft \cdot kip$ **W21x44 Fails**

Redesign did not take into account the effect of unbraced length of the compression flange and the section, despite the lower magnitude of the moment demand, fails due to lateral-torsional buckling.

In the previous example, it was shown that even apparently benign changes in the design warrant additional calculations. All changes must be reviewed by a licensed structural engineer as they can lead to unconservative designs and failure. A qualified structural engineer should still be able to accommodate constructability demands such as the previously explored need to change the location of the shear hinge away from the brace connection to facilitate constructability, but still ensure adequate flexural capacity. In the following example, reduced cantilever spans are employed to reduce the magnitude of the negative moment in the vicinity of the braces below the lateral-torsional buckling limit of the original member.

Steel Design Example 2: The previous example can be redesigned to provide more practical connection locations (*i.e.*, connections not located directly at the braces. Redesign the walers from the previous problem; try 1-foot cantilevers in the internal span.

Redesigned Waler



Basic parameters:

$L := 20\text{ft}$ $\quad F_y := 50\text{ksi}$ $\quad E := 29000\text{ksi}$

$w := 4\text{klf}$

Shear:

$V_a := 0.524 \cdot w \cdot L = 41.92 \cdot \text{kip}$

Moment:

$M_{apositive} := 0.1133 \cdot w \cdot L^2 = 181.28\,\text{ft} \cdot \text{kip}$ $\quad L_{bpositive} := 0\text{ft}$

$M_{anegative} := 0.024 \cdot w \cdot L^2 = 38.4\,\text{ft} \cdot \text{kip}$ $\quad L_{bnegative} := 20\text{ft}$

For W21x44 (AISC Table 3-2 and Table 1-1)

$L_p := 4.45\text{ft}$ $\quad L_r := 13\text{ft}$ $\quad c := 1.0$

$r_{ts} := 1.6\text{in}$ $\quad h_0 := 20.3\text{in}$ $\quad J := 0.770\text{in}^4$

$S_x := 81.6\text{in}^3$ $\quad M_{px\_\Omega b} := 238\text{ft} \cdot \text{kip}$ $\quad V_{nx\_\Omega b} := 145\text{kip}$

Span A

For Positive Moment (at 0.476L)

$L_p := 4.45ft$

$L_p = 4.45\,ft$ > $L_{bpositive} = 0 \cdot ft$ Yielding Mn = Mp (AISC F2-1)

$M_{px\_\Omega b} := 238ft \cdot kip$ > $M_{apositive} = 181.28\,ft \cdot kip$ OK

OK to use W21x44

$V_{nx\_\Omega b} := 145kip$ > $V_a = 41.92 \cdot kip$ OK

For Negative Moment (at support)

$L_{bnegative} = 20\,ft$ > $L_r = 13\,ft$ Elastic Buckling

$x_A := 5ft$ $x_B := 10ft$ $x_C := 15ft$ $E := 29000ksi$

$$M_A := 0.414w \cdot L_{bnegative} \cdot x_A - \frac{w \cdot x_A^{\,2}}{2} = 115.6\,ft \cdot kip$$

$$M_B := 0.414w \cdot L_{bnegative} \cdot x_B - \frac{w \cdot x_B^{\,2}}{2} = 131.2\,ft \cdot kip$$

$$M_C := 0.414w \cdot L_{bnegative} \cdot x_C - \frac{w \cdot x_C^{\,2}}{2} = 46.8\,ft \cdot kip$$

$M_{maxA} := M_{apositive} = 181.28\,ft \cdot kip$

$$C_b := \frac{\left(12.5M_{maxA}\right)}{2.5 \cdot M_{maxA} + 3 \cdot M_A + 4 \cdot M_B + 3M_C} = 1.547$$

$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 26.51 \cdot ksi$$

$\Omega_b := 1.67$ (AISC F1)

$M_n := F_{cr} \cdot S_x = 180.269\,ft \cdot kip$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 107.945\,ft \cdot kip$$

$M_{n\_\Omega b} = 107.945\,ft \cdot kip$ > $M_{anegative} = 38.4\,ft \cdot kip$ OK

OK to use W21x44

Span B (at support)

$C_b := 1.0$

$L_{bnegative} = 20\,ft$

$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 17.141 \cdot ksi$$

$\Omega_b := 1.67$ (AISC F1)

$M_n := F_{cr} \cdot S_x = 116.562\,ft \cdot kip$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 69.798\,ft \cdot kip$$

$M_{n\_\Omega b} = 69.798\,ft \cdot kip \quad > \quad M_{anegative} = 38.4\,ft \cdot kip$ OK

OK to use W21x44

Span C (at midspan)

$M_a := 0.101\,w \cdot L^2 = 161.6\,ft \cdot kip$

$L_p := 4.45ft$

$L_p = 4.45\,ft \quad > \quad L_{bpositive} = 0 \cdot ft$ Yielding Mn = Mp (AISC F2-1)

$M_{px\_\Omega b} := 238ft \cdot kip \quad > \quad M_a = 161.6\,ft \cdot kip$ OK

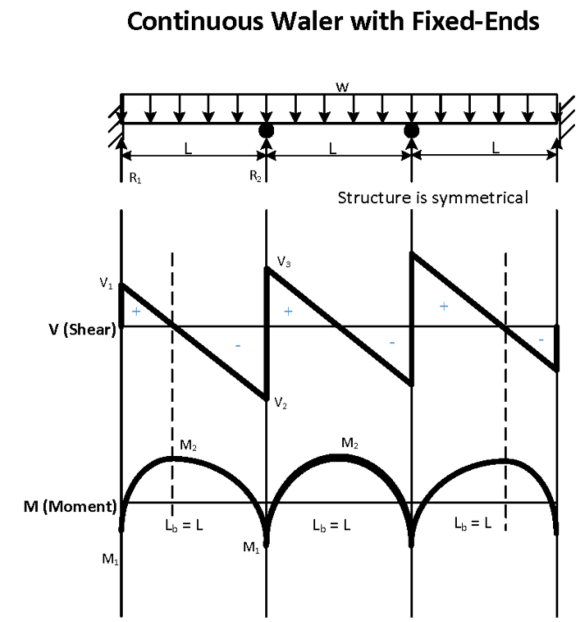
OK to use W21x44

Summary: The original beam size will be OK if 1-foot cantilevers are used.

Here, smaller cantilever spans provide the fabricator the ability to produce a simpler and cheaper connection while maintaining a safe design. Increasing the size of the member to increase its negative-moment flexural capacity would also have been an acceptable solution.

A more common configuration is a continuous waler designed with full fixed-end moment restraint. In such a case, the walers and braces will constitute an indeterminate structure, and it may be convenient to perform the analysis using a commercial structural analysis program to find the maximum shear and moments for design. In this case, similar issues exist to those illustrated in Example 2 with regard to the demonstrated need to account for the unbraced length of the compression flange of walers in both directions.

Steel Design Example 2A: A more typical configuration for the waler is a continuous beam with fixed-end connections to the transverse walers located at the ends.



From structural analysis package:

$V_1 := 40 \cdot kip$

$V_2 := V_1$

$M_1 := 66.7 \cdot kip \cdot ft$

$M_2 := 134 \cdot kip \cdot ft$

$R_1 := V_1$

$R_2 := 2 \cdot V_1$

Basic parameters:

$L := 20ft$ $\quad F_y := 50ksi$ $\quad E := 29000ksi$

$w := 4klf$

Shear:

$V_a := V_2 = 40 \cdot kip$

Moment:

$M_{apositive} := M_2 = 134\,ft \cdot kip$ $\quad L_{bpositive} := 0ft$

$M_{anegative} := M_1 = 66.7\,ft \cdot kip$ $\quad L_{bnegative} := 20ft$

A smaller section will likely be usable, try a W16x31 (AISC Table 3-2 and Table 1-1)

$L_p := 4.13ft$ $\quad L_r := 11.8ft$ $\quad c := 1.0$

$r_{ts} := 1.42in$ $\quad h_0 := 15.5in$ $\quad J := 0.461\,in^4$

$S_x := 47.2in^3$ $\quad M_{px\_\Omega b} := 135ft \cdot kip$ $\quad V_{nx\_\Omega b} := 87.5kip$

In this case, all spans have identical shear and moment distributions:

For Positive Moment

$L_p := 4.45ft$

$L_p = 4.45\,ft$ > $L_{bpositive} = 0\cdot ft$ Yielding Mn = Mp (AISC F2-1)

$M_{px\_\Omega b} := 135ft\cdot kip$ > $M_{apositive} = 134\,ft\cdot kip$ OK

OK to use W16x31

$V_{nx\_\Omega b} := 87.5kip$ > $V_a = 40\cdot kip$ OK

For Negative Moment (at supports)

$L_{bnegative} = 20\,ft$ > $L_r = 11.8\,ft$ Elastic Buckling

$x_A := 5ft$ $x_B := 10ft$ $x_C := 15ft$ $E := 29000ksi$

$C_b := 2.27$ (AISC F1 User Note: reverse curvature bending)

$$F_{cr} := \frac{\left(C_b\cdot\pi^2\cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2}\cdot\sqrt{1 + 0.078\cdot\frac{J\cdot c}{S_x\cdot h_0}\cdot\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 35.265\cdot ksi$$

$\Omega_b := 1.67$ (AISC F1)

$M_n := F_{cr}\cdot S_x = 138.709\,ft\cdot kip$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 83.06\,ft\cdot kip$$

$M_{n\_\Omega b} = 83.06\,ft\cdot kip$ > $M_{anegative} = 66.7\,ft\cdot kip$ OK

Check shear:

$\frac{V_{nx}}{\Omega_v} = 87.5\cdot kip$ (Table 3-2)

> $V_1 = 40\,kip$ OK

Web local yielding and web local crippling checks must also be perfromed at strut locations, see next example.

OK to use W16x31

In addition to the member design, the connections between walers and any splices must also be designed to carry the internal loads indicated by the analysis.

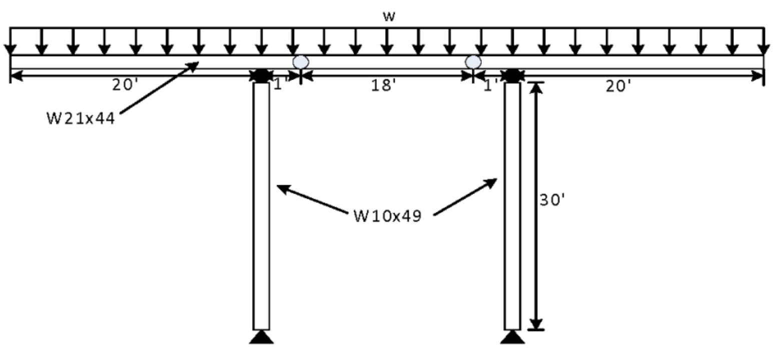
6.2 Strut Design Example

In the design of the struts used in the braced system, the agreement between the design and constructed conditions is equally important as it is in the waler design. In the following example, the effect of end condition assumptions for internal strut members is explored. The designer is free to choose moment restraining or non-moment-restraining conditions for the strut connections to the walers, but these conditions must be realistic with respect to the actual connection details constructed in the field. In the case in which fixed-end connections are assumed, it is somewhat obvious how failure to provide these connection conditions can be deleterious to the strength of the braced system, in particular within the strut. The increase in effective length (KL) of the strut associated with the connection change will significantly reduce its buckling capacity in compression, and could easily lead to failure (though it is advised to use an effective length factor of at least 1.0 for all braces). Less obvious is the converse case where a non-moment resisting (pinned) connection is assumed and the strut is installed using a fully-welded connection (flanges and web) that develops a significant portion of the strut moment capacity at its ends. In this case, the reduction of the effective length of the strut from the designed condition will be helpful, however, the behavior of the entire brace system has been altered by this connection. The strut now acts as a beam-column and must be designed and checked for a combination of compression and flexure (including 2nd-order effects). In addition, the added stiffness at the connection will cause additional flexural loads to occur in the waler near these connections, perhaps leading to lateral-torsional buckling. In addition, bearing conditions in the waler will change, requiring new checks for concentrated load effects.

The following example depicts a comparison of the two different design approaches necessary when the strut to waler connection is designed as a pinned connection and the revised calculations necessary if pinned conditions are not provided in the field. Regardless of the connection details, the appropriate limit states must be checked, including those relating to compression (including flexural buckling and, if applicable, flexural-torsional buckling and local buckling), flexure (if applicable), and concentrated load effects on the waler flange and web. Connection details are also important and will be discussed in a later example.

Structural Steel Example 3: A waler (W21x44) supporting a sheet piling retaining wall is in turn supported by two internal braces (W10x49). The struts were designed with simply-supported end conditions but the connections were fully-welded in the field providing full moment restraint. This change redistributes loads in the framing system and creates a combined compression-bending condition in the braces. Determine whether or not the walers and braces are adequate given the change in end conditions.

AS DESIGNED



Basic Parameters:

$L := 20ft$ $\quad F_y := 50ksi$ $\quad E := 29000ksi$

$w := 4klf$

Shear:

$V_a := 0.524w \cdot L = 41.92kip$

Moment:

$M_{apositive} := 0.1133w \cdot L^2 = 181.28ft \cdot kip$ $\quad L_{bpositive} := 0ft$

$M_{anegative} := 0.024 \cdot w \cdot L^2 = 38.4\,ft \cdot kip$ $\quad L_{bnegative} := 20ft$

For W21x44 waler (AISC Table 3-2 and Table 1-1)

$L_p := 4.45ft$ $\quad L_r := 13ft$ $\quad c := 1.0$

$r_{ts} := 1.6in$ $\quad h_0 := 20.3in$ $\quad J := 0.770in^4$

$S_x := 81.6in^3$ $\quad M_{px\_\Omega b} := 238ft \cdot kip$ $\quad V_{nx\_\Omega b} := 145kip$

For Positive Moment Zone

$L_p := 4.45ft$

$L_p = 4.45\,ft$ $\quad >$ $\quad L_{bpositive} = 0 \cdot ft$ $\quad$ Yielding $\quad$ Mn = Mp (AISC F2-1)

$M_{px\_\Omega b} := 238ft \cdot kip$ $\quad >$ $\quad M_{apositive} = 181.28\,ft \cdot kip$ $\quad$ OK

OK to use W21x44

$V_{nx\_\Omega b} := 145kip$ $\quad >$ $\quad V_a = 41.92 \cdot kip$ $\quad$ OK

For Negative Moment Zone

$L_{bnegative} = 20\,ft$ > $L_r = 13\,ft$ Elatic Buckling

$x_A := 5ft$ $x_B := 10ft$ $x_C := 15ft$ $E := 29000ksi$

$$M_A := 0.414w \cdot L_{bnegative} \cdot x_A - \frac{w \cdot x_A^2}{2} = 115.6\,ft \cdot kip$$

$$M_B := 0.414w \cdot L_{bnegative} \cdot x_B - \frac{w \cdot x_B^2}{2} = 131.2\,ft \cdot kip$$

$$M_C := 0.414w \cdot L_{bnegative} \cdot x_C - \frac{w \cdot x_C^2}{2} = 46.8\,ft \cdot kip$$

$M_{maxA} := M_{apositive} = 181.28\,ft \cdot kip$

$$C_b := \frac{(12.5M_{maxA})}{2.5 \cdot M_{maxA} + 3 \cdot M_A + 4 \cdot M_B + 3M_C} = 1.547$$

$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 26.51 \cdot ksi$$

$\Omega_b := 1.67$ (AISC F1)

$M_n := F_{cr} \cdot S_x = 180.269\,ft \cdot kip$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 107.945\,ft \cdot kip$$

$M_{n\_\Omega b} = 107.945\,ft \cdot kip$ > $M_{anegative} = 38.4\,ft \cdot kip$ OK

OK to use W21x44

For W10x49 brace (AISC Table 1-1) - member is designed as a compression-only member

$A_g := 14.4in^2$ $d := 10in$ $b_f := 10.0in$ $t_{fb} := 0.560in$

$t_w := 0.340in$ $k := 1.06in$ $r_x := 4.35in$ $r_y := 2.54in$

$L_{W10x49} := 30ft$ $K := 1.0$ (AISC Table C-A-7.1)

$E := 29000ksi$ $F_y := 50ksi$

Local Buckling

Case 1 (AISC Table B4.1a)

$$b := \frac{b_f}{2} = 5 \cdot in$$

$$\lambda := \frac{b}{t_{fb}} = 8.929 \qquad \lambda_r := 0.56 \cdot \sqrt{\frac{E}{F_y}} = 13.487$$

$$\lambda = 8.929 \quad < \quad \lambda_r = 13.487$$

Therefore, there is no local buckling in flange.

Case 5 (AISC Table B4.1a)

$$h := d - 2k = 7.88 \cdot in$$

$$\lambda := \frac{h}{t_w} = 23.176 \qquad \lambda_r := 1.49 \cdot \sqrt{\frac{E}{F_y}} = 35.884$$

Therefore, there is no local buckling in web.

Flexural Buckling

$$\frac{K \cdot L_{W10x49}}{r_x} = 82.759 \quad < \quad 200 \quad \text{OK}$$

$$\frac{K \cdot L_{W10x49}}{r_y} = 141.732 \quad < \quad 200 \quad \text{OK}$$ AISC slenderness limit is met.

$$\frac{K \cdot L_{W10x49}}{r_y} > \frac{K \cdot L_{W10x49}}{r_x}$$ Therefore $\frac{K \cdot L_{W10x49}}{r_y}$ controls.

$$F_e := \frac{\left(\pi^2 \cdot E\right)}{\left(\frac{K \cdot L_{W10x49}}{r_y}\right)^2} = 14.248 \cdot ksi \qquad \text{(AISC E3-4)}$$

$$\frac{F_y}{F_e} = 3.509 \quad > \quad 2.25$$

$$F_{cr} := 0.877 F_e = 12.496 \cdot ksi \qquad \text{(AISC E3-3)}$$

$$P_n := F_{cr} \cdot A_g = 179.938 \cdot kip$$

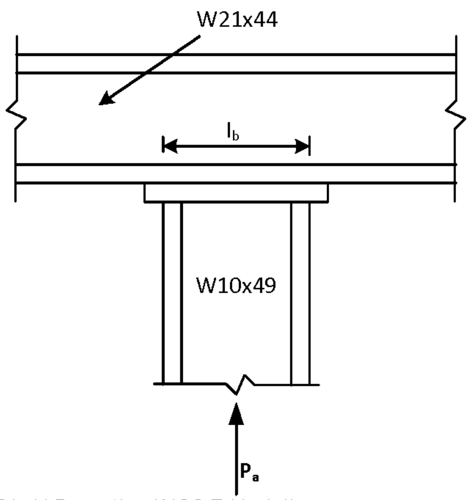
$$\Omega_c := 1.67$$

$$P_{n\_\Omega c} := \frac{P_n}{\Omega_c} = 107.747 \cdot kip$$

$P_a := 1.024 w \cdot L = 81.92 \cdot kip$

$P_{n\_\Omega c} = 107.747 \cdot kip$ > $P_a = 81.92 \cdot kip$ **OK to use W10x49**

Check the Concentrated Load (Bearing) at Braces.



W21x44 Properties (AISC Table 1-1)

$d := 20.7 in$	$t_w := 0.350 in$	$b_f := 6.50 in$	$t_f := 0.450 in$
$k := 0.950 in$	$E := 29000 ksi$	$F_y := 50 ksi$	

Web Local Yielding

Interior Load $d_{W10x49} := 10 in$

$l_b := d_{W10x49} = 10 \cdot in$ $F_{yw} := F_y = 50 \cdot ksi$

$R_a := P_a = 81.92 \cdot kip$

$\Omega := 1.50$

$R_n := F_{yw} \cdot t_w \cdot (5k + l_b) = 258.125 \cdot kip$ (AISC J10-2)

$$R_{n\_\Omega} := \frac{R_n}{\Omega} = 172.083 \cdot kip$$

$R_{n\_\Omega} = 172.083 \cdot kip$ > $R_a = 81.92 \cdot kip$ Does not need stiffeners for J10.2.

Web Local Crippling

Interior Load

$R_a := P_a = 81.92 \cdot kip \qquad l_b = 10 \cdot in$

$\Omega := 2.00$

$F_{yw} := F_y = 50 \cdot ksi$

$$R_n := 0.80 t_w^{\,2} \cdot \left[1 + 3 \cdot \left(\frac{l_b}{d}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}} = 266.828 \cdot kip \quad \text{(AISC J10-4)}$$

$$R_{n\_\Omega} := \frac{R_n}{\Omega} = 133.414 \cdot kip$$

$R_{n\_\Omega} = 133.414 \cdot kip \quad > \quad R_a = 81.92 \cdot kip$ Does not need stiffeners for J10.3.

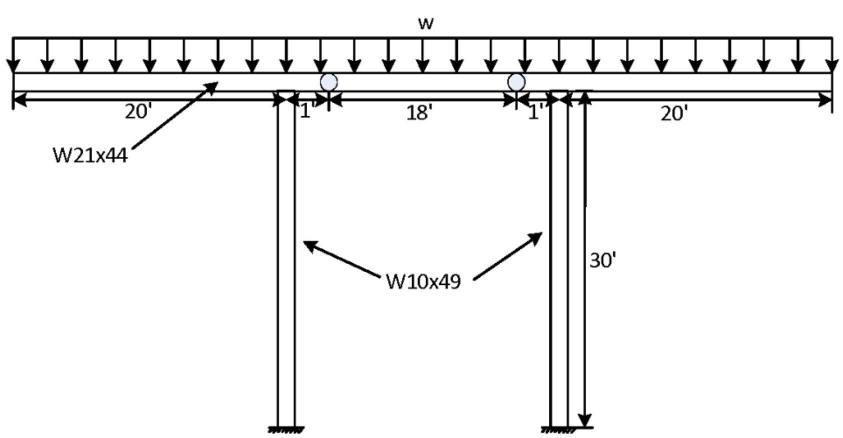
Web sidesway buckling and web compression buckling do not apply.

No concentrated tension load exists so flange local bending also does not apply

Stiffeners are not required for this design.

Check the as-built conditions to determine if the change affected the structure negatively.

AS-BUILT



Basic parameters:

$L := 20ft \qquad F_y := 50ksi \qquad E := 29000ksi \qquad w := 4klf$

Moment (from iterative 2nd-order analysis):

$M_{apositive} := 166ft \cdot kip$ $\quad L_{bpositive} := 0ft$

$M_{anegative} := 72.2ft \cdot kip$ $\quad L_{bnegative} := 20ft$

For W21x44 (AISC Table 3-2 and Table 1-1)

$L_p := 4.45ft$ $\quad L_r := 13ft$ $\quad c := 1.0$

$r_{ts} := 1.6in$ $\quad h_0 := 20.3in$ $\quad J := 0.770in^4$

$S_x := 81.6in^3$ $\quad M_{px\_\Omega b} := 238ft \cdot kip$ $\quad V_{nx\_\Omega b} := 145kip$

For Positive Moment

$L_p := 4.45ft$

$L_p = 4.45\,ft$ $\quad > \quad$ $L_{bpositive} = 0 \cdot ft$ $\quad$ Yielding $\quad$ Mn = Mp (AISC F2-1)

$M_{px\_\Omega b} := 238ft \cdot kip$ $\quad > \quad$ $M_{apositive} = 166\,ft \cdot kip$ $\quad$ OK

OK to use W21x44

For Negative Moment

$L_{bnegative} = 20\,ft$ $\quad > \quad$ $L_r = 13\,ft$ $\quad$ Elastic Buckling (Zone 3)

$C_b := 1.0$

$$F_{cr} := \frac{\left(C_b \cdot \pi^2 \cdot E\right)}{\left(\frac{L_{bnegative}}{r_{ts}}\right)^2} \cdot \sqrt{1 + 0.078 \cdot \frac{J \cdot c}{S_x \cdot h_0} \cdot \left(\frac{L_{bnegative}}{r_{ts}}\right)^2} = 17.141 \cdot ksi$$

$\Omega_b := 1.67$ $\quad$ (AISC F1)

$M_n := F_{cr} \cdot S_x = 116.562\,ft \cdot kip$

$$M_{n\_\Omega b} := \frac{M_n}{\Omega_b} = 69.798\,ft \cdot kip$$

$M_{n\_\Omega b} = 69.798\,ft \cdot kip$ $\quad < \quad$ $M_{anegative} = 72.2\,ft \cdot kip$ $\quad$ NOT OK

W21x44 fails due to increased negative moment at the brace due to increased rotational restraint provided in the connection.

For W10x49, Properties (AISC Table 1-1 and Table 3-2)

$A_g := 14.4in^2$ $\quad d := 10.0in$ $\quad b_f := 10.0in$ $\quad t_{fb} := 0.560in$

$t_w := 0.340in$ $\quad k := 1.06in$ $\quad r_x := 4.35in$ $\quad r_y := 2.54in$

$I_x := 272in^4$ $\quad M_{px\_\Omega b} := 151ft \cdot kip$ $\quad BF_{\Omega b} := 2.46kip$ $\quad L_p := 8.97ft$

$L_r := 31.6ft$

$L_{W10x49} := 30ft$ $\quad K := 1.0$ (Limited weak-axis flexural buckling support from waler)

$F_y := 50ksi$ $\quad E := 29000ksi$

$L_b := K \cdot L_{W10x49} = 30\,ft$

$P_a := 83.7kip$

$M_a := 34.2ft \cdot kip$

(Loads from iterative 2nd-order analysis using computer structural analysis package. It is assumed that sidesway is not possible.)

$P_r := P_a = 83.7 \cdot kip$

$\Omega_t := 1.67$ (AISC D2.a)

$$P_c := \frac{(F_y \cdot A_g)}{\Omega_t} = 431.138 \cdot kip$$

Combined Compression and Moment Capacity

$L_p = 8.97\,ft$ < $L_b = 30\,ft$ < $L_r = 31.6\,ft$ Inelastic buckling

$$M_{n\_\Omega b} := C_b \cdot \left[M_{px\_\Omega b} - \left[BF_{\Omega b} \cdot (L_b - L_p)\right]\right] = 99.266\,ft \cdot kip$$

$M_{n\_\Omega b} = 99.266\,ft \cdot kip$ < $M_{px\_\Omega b} = 151\,ft \cdot kip$

$M_{rx} := M_a = 34.2\,ft \cdot kip$

$M_{cx} := M_{n\_\Omega b} = 99.266\,ft \cdot kip$

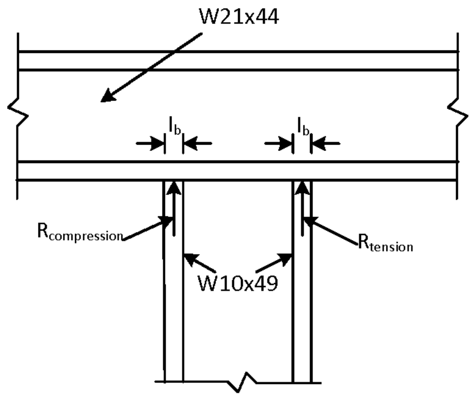
$\frac{P_r}{P_c} = 0.194$ $\quad \frac{P_r}{P_c} < 0.2 = 1$ use AISC H1-1b

$$\frac{P_r}{2P_c} + \left[\frac{8}{9} \cdot \left(\frac{M_{rx}}{M_{cx}} + 0\right)\right] = 0.403$$ (AISC H1-1b)

0.387 < 1.0 OK

W10x49 is still an adequate member (the reduction in the effective length of the brace offset the added moment demand placed on the brace).

Check the Concentrated Loads at Braces



W21x44 Properties (AISC Table 1-1)

$d := 20.7in$ $t_w := 0.350in$ $b_f := 6.50in$ $t_f := 0.450in$

$k := 0.950in$

$E := 29000ksi$ $F_y := 50ksi$

$$R_{compression} := \frac{M_a}{d - t_{fb}} + \frac{P_a}{2} = 62.227 \cdot kip$$ (Divide the axial load between the two flanges.)

(Load does not reverse)

$$R_{tension} := \frac{M_a}{d - t_{fb}} - \frac{P_a}{2} = -21.473 \cdot kip$$ No tension load exists.

Web Local Yielding

Interior Load

$R_a := R_{compression} = 62.227 \cdot kip$ $l_b := t_{fb} = 0.56 \cdot in$

$\Omega := 1.50$

$R_n := F_{yw} \cdot t_w \cdot \left(5k + l_b\right) = 92.925 \cdot kip$ (AISC J10-2)

$$R_{n\_\Omega} := \frac{R_n}{\Omega} = 61.95 \cdot kip$$

$R_{n\_\Omega} = 61.95 \cdot kip$

$R_{n\_\Omega} = 61.95 \cdot kip$ > $R_a = 62.227 \cdot kip$ Does not need stiffeners for J10.2.

Web Local Crippling

Interior Load

$l_b := t_{fb} = 0.56 \cdot in$

$\Omega := 2.00$

$R_a := R_{compression} = 62.227 \cdot kip$

$F_{yw} := F_y = 50 \cdot ksi$

$R_n := 0.80 t_w^{\,2} \cdot \left[1 + 3 \cdot \left(\frac{l_b}{d}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yw} \cdot t_f}{t_w}} = 141.257 \cdot kip$ (AISC J10-4)

$R_{n\_\Omega} := \frac{R_n}{\Omega} = 70.629 \cdot kip$

$R_{n\_\Omega} = 70.629 \cdot kip$ > $R_a = 62.227 \cdot kip$ Does not need stiffeners for J10.3.

Flange Local Bending

Axial compression in tension flange is larger than flexural tension in flange. Flange local bending does not apply.

Does not need stiffening for J10.1.

Web Panel Zone Shear

$d_b := 10.0in$

$R_a := \frac{M_a}{0.95 d_b} + P_a = 126.9 \cdot kip$ (AISC C-J10-3a)

$d_c := d = 20.7 \cdot in$ $t_{wc} := t_w = 0.35 \cdot in$

$\frac{P_r}{P_c} = 0.194$ < 0.4

$\Omega := 1.67$

$R_{n\_\Omega} := \frac{\left(0.6 \cdot F_y \cdot d_c \cdot t_{wc}\right)}{\Omega} = 130.15 \cdot kip$ (AISC J10-9)

$R_{n\_\Omega} = 130.15 \cdot kip$ > $R_a = 126.9 \cdot kip$ Does not need stiffening for J10.6.

The change in connection rotational restraint necessitated additional checks for Flange Local Bending and Web Panel Zone Shear however, transverse stiffeners or web doubler plates are not required.

It can be seen that the strut and bearing capacity were not negatively affected by the change in strut connection conditions, however, the additional negative moment in the waler lead to a need to redesign that member to prevent lateral-torsional buckling. The design engineer must submit calculations demonstrating the adequacy of the design for all applicable limit states. To ensure the safety of workers and the motoring public, the limit states checked shall be appropriate for the structure as it will be constructed in the field.

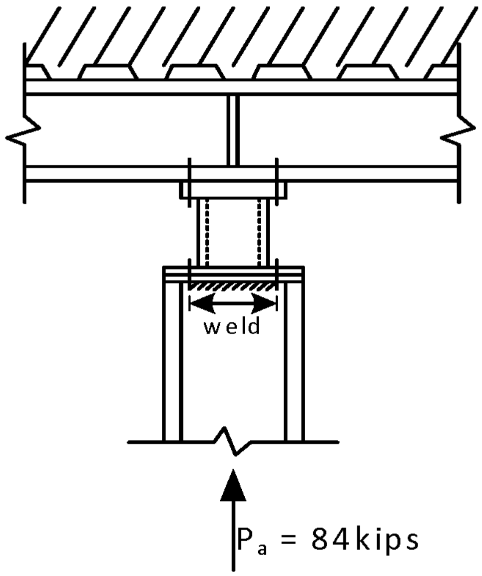
6.3 Connection Design Examples

In the previous section, it was demonstrated that it is necessary to ensure that the connection conditions assumed during the design of a frame actually match the connection conditions that are fabricated and installed in the field. To ensure that this match occurs, connections must be designed appropriately. For instance, if pinned connections are assumed at the end of I-shaped members (*e.g.*, wide-flange or bearing-pile sections), then connections should be made primarily through the web and not through the flanges to limit rotational fixidity. Likewise, moment-restrained connections must be designed with sufficient restraint in the flanges to transmit the concentrated flange loads associated with the design moments.

Within a connection, all connecting elements must be designed for the loads they will carry. While the specific geometries of any connections will vary according to the preferences of the contractor and design engineer, they must meet the design requirements of the structural steel design standard used (*e.g.*, Section J4 in the AISC Steel Construction Manual). These requirements extend to all connecting elements including webs and flanges of members, plates, angles, T-sections, spacers, bolts, and welds. Where appropriate, eccentric loads on fastener groups must also be considered. Load tables for prequalified connections may be used, but must be applied according to the appropriate design assumptions. For example, Part 10 of the AISC Steel Construction Manual contains a number of tables giving capacities on non-moment resisting shear connections that are meant to be used as framed connections that do not consider axial loads. While these geometries may be useful for shear splices in walers, it would not be appropriate to use the capacities given for shear loads as strut-to-waler connections that primarily carry axial loads and suffer a different set of failure mechanisms. Instead, Part 9 of the AISC Manual outlines the limit states and appropriate portions of the specification that need to be checked depending on the connection loading.

It will not be possible to cover all connection conditions in this manual. To attempt to do so is to limit the designer of these systems. Instead, two connection details for brace-to-waler connections utilizing spacers are presented to depict the difference between two similar connections, one with full rotational stiffness and one intended to approximate a pin. The limit states needed to be checked are listed for each connection assumption for a design based on the AISC Manual.

Connection (spacer) design example for a pined connection.
Brace flanges not provided with significant weld to ensure pin-type behavior.



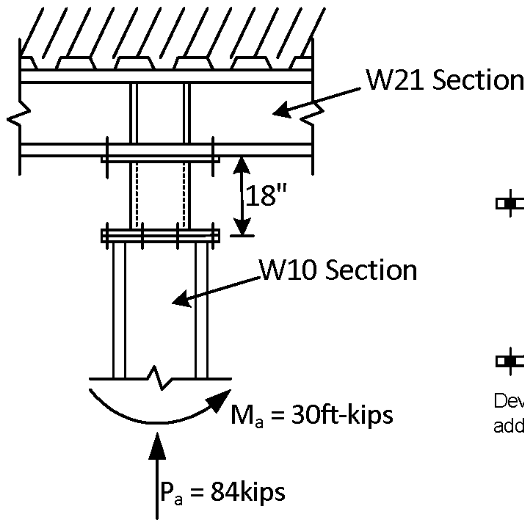
Requirements for connecting elements subject to compression yielding and buckling (AISC J4.4).

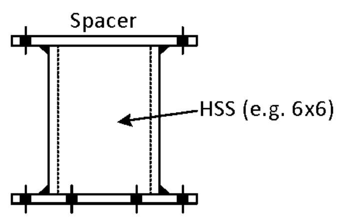
Limit States to Check (with AISC Specification section):

1. HSS in Compression - J4.4 (if KL/r < 25, otherwise - Chapter E)
   - -Flexural Buckling/Yield - E3
   - -Local Buckling - E7
   - -Tosional Buckling (if not HSS or similar) - E4.
2. Plate flexure (weak-axis yield) - F11
3. Bearing of all components in compression - J7
4. Concentrated load in waler - J10

Meet minimum stability and development requirements for bolts and welds.

Connection (spacer) design for a moment-restrained connection.





Develops both shear and moment addition to axial load.

Requirements for connecting elements subject to combined compression and flexure (AISC J4.4-5).

Limit States to Check (with AISC Specification section):

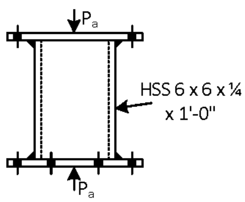
1. HSS in Combined Flexure and Compression - Chapter H
2. Plate flexure (weak-axis yield) - F11
3. Plate shear capacity - J4.2
   - Yield
   - Fracture
4. Bolt Tension - J3
5. Weld Capacity - J2
   -eccentrically loaded weld groups
   -predominantly transverse loading
6. Bearing of all components in compression - J7
7. Concentrated load in waler - J10

Extended end-plate connection design proceedure could be useful here. If guidelines are used, be sure to follow minimum width and thickness requirements as well as maximum effective width guidelines. Bear in mind that the tension flange loads will be reduced due to the axial compression in the connection, but compression flange requirements will be increased.

<u>Connection Calculation Example</u>: Buckling of Connection Elements

Analysis and desing of connecting elements should include all connecting elements to ensure that the loads are able to be transmitted effectively. The sections used have a strong influence on their suitability for these applications. For example, the axial compressive capacity of a square HSS and a single-plate spacer in compression are compared below.

<u>HSS spacer</u>



AISC J4.4: Strength of Elements in Compression

$K := 1.2$ fixed-fixed conn (trans. free)

$L := 12 \cdot in$

$r := 2.34 \cdot in$ Table 1

$\frac{K \cdot L}{r} = 6.154$ < 25, use AISC J4-6 for capacity

$A_g := 5.24 \cdot in^2$ $E := 29000 \cdot ksi$

$F_y := 46 \cdot ksi$ A500 Gr.B (Table 2-4)

$\Omega := 1.67$

$P_n := F_y \cdot A_g = 241.04 \cdot kip$

$\frac{P_n}{\Omega} = 144.335 \cdot kip$

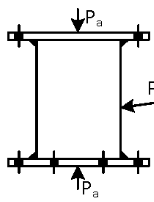
Check for slender elements

$b := 6 \cdot in$ $t := 0.25 \cdot in$ Table 4.1

$\frac{b}{t} = 24$ < $1.40 \cdot \sqrt{\frac{E}{F_y}} = 35.152$ OK

Design capacity is 144 k.

Single-plate spacer



AISC J4.4: Strength of Elements in Compression

$K := 1.2$ fixed-fixed conn (trans. free)

$L := 12 \cdot in$

$b := 5.25 \cdot in \quad d := 1 \cdot in$

$$r := \frac{d}{\sqrt{12}} = 0.289 \cdot in$$

$$\frac{K \cdot L}{r} = 49.883$$ > 25, provisions of Chapter E apply (J4.4b)

Flexural Buckling (E.3)

$E := 29000 \cdot ksi$

$F_y := 46 \cdot ksi$ (Not a usual yield stress for a plate, but we want to be consistent with HSS spacer example.)

$$\frac{K \cdot L}{r} = 49.883 \quad < \quad 4.71 \cdot \sqrt{\frac{E}{F_y}} = 118.261$$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{K \cdot L}{r}\right)^2} = 115.025 \cdot ksi \qquad \text{(E3-4)}$$

$$F_{cr} := \left(0.658^{\frac{F_y}{F_e}}\right) F_y = 38.91 \cdot ksi \qquad \text{(E3-2)}$$

$$A_g := b \cdot d = 5.25 \cdot in^2$$

$$P_n := F_{cr} \cdot A_g = 204.279 \cdot kip \qquad \text{(E3-1)}$$

$$\Omega_c := 1.67 \qquad \text{(E1)}$$

$$\frac{P_n}{\Omega_c} = 122.323 \cdot kip$$

Check for slender elements

$t := d = 1 \cdot in$ Table 4.1

$$\frac{b}{t} = 5.25 \quad < \quad 0.45 \cdot \sqrt{\frac{E}{F_y}} = 11.299 \quad \text{OK}$$

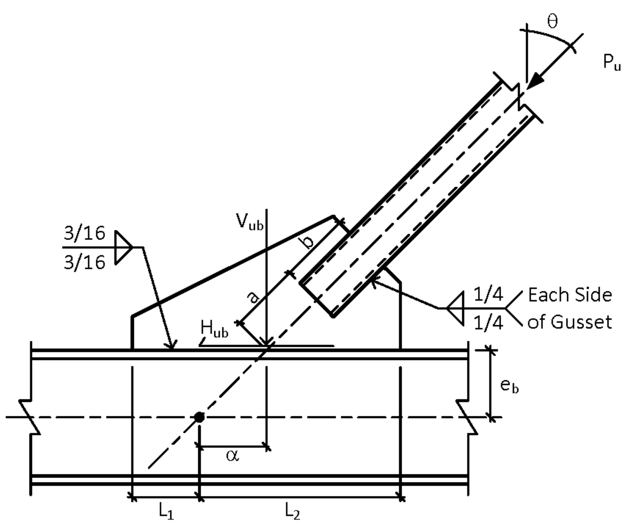
Design capacity is 122 k.

Recognizing the importance of welded connections in the design and construction of braced support systems, an example of a welded connection is also presented. In the following example, an HSS-section knee brace has been designed to support the corners of a braced cofferdam system and must be connected to the walers. In this example, it is assumed that the brace is to be installed at a 45-degree angle and that the connections at both ends of the brace are identical to one another. It is assumed that the brace will only act in compression and that load reversal will not be a concern.

Steel Design Example 4A: Welded Steel Connection Design Example

Design a welded connection for a 45-degree-angle knee brace connecting to the walers of a braced sheet pile system. Assume that the size and ultimate compressive axial load of the brace have been determined using a commercial structural analysis software package. LRFD is used for pedagogical purposes.

Reference: American Institute of Steel Construction (AISC) Manual (14th Edition)
AISC Design Examples (V14.1)



$P_u := 200 \cdot kip$ From braced system analysis

Brace properties: HSS6x6x1/2

$F_{yHSS} := 46 \cdot ksi$ $F_{uHSS} := 58 \cdot ksi$ (A500 Gr. B)

$t_b := 0.465 \cdot in$ $d_b := 6.00 \cdot in$ $E := 29000 \cdot ksi$

Waler properties: W21x44

$F_{yW} := 50 \cdot ksi$ $F_{uW} := 65 \cdot ksi$ (A992)

$d := 20.7 \cdot in$ $t_w := 0.350 \cdot in$

$b_f := 6.50 \cdot in$ $t_f := 0.450 \cdot in$

$k_{des} := 0.950 \cdot in$

Gusset plate properties

$F_{yPL} := 36 \cdot ksi$ $F_{uPL} := 58 \cdot ksi$ (A36)

$t_{PL} := 0.500 \cdot in$

All welds E70 Electrode $F_{EXX} := 70 \cdot ksi$

Connection dimensions:

$\theta := 45 \cdot deg$

$e_b := \frac{d}{2} = 10.35 \cdot in$

$\alpha := e_b \cdot \tan(\theta) = 10.35 \cdot in$

To eliminate eccentrictiy in the gusset/waler connection, choose L_1 and L_2 such that the center of the weld is located α from the working point of the connection. Let $L_1 + \alpha = L_2 - \alpha$.

$L_1 := 6 \cdot in$

$L_2 := L_1 + 2 \cdot \alpha = 26.7 \cdot in$ use $L_2 := 30.750 \cdot in$

Brace/gusset overlap and spacing to waler:

$a := 9 \cdot in$ $b := 10 \cdot in$

Connection loads:

Brace-to-gusset $P_u = 200 \cdot kip$ Compression only

Gusset-to-waler $H_{ub} := P_u \cdot \sin(\theta) = 141.421 \cdot kip$

$V_{ub} := P_u \cdot \cos(\theta) = 141.421 \cdot kip$

Design brace-to-gusset connection:

Weld strength in longitudinal shear: $R_u := P_u$

Fillet weld size: $w := 0.250 \cdot in$ (Minimum size 3/16in, Table J2.4)

$\phi := 0.75$ (Table J2.5)

$F_{nw} := 0.60 \cdot F_{EXX} = 42 \cdot ksi$

$A_w := 0.707 \cdot w \cdot b = 1.767 \cdot in^2$

per weld: $\phi R_n := \phi \cdot F_{nw} \cdot A_w = 55.676 \cdot kip$ (J2-3)

accounting for all 4 welds:

$4 \cdot \phi R_n = 222.705 \cdot kip$ > $R_u = 200 \cdot kip$ OK

Check shear yield in HSS brace at welds:

$\phi := 1.00$

$A_{gv} := t_b \cdot b = 4.65 \cdot in^2$

$\phi R_n := 4 \cdot \phi \cdot 0.60 F_{yHSS} \cdot A_{gv} = 513.36 \cdot kip$ (J4-3)

> $R_u = 200 \cdot kip$ OK

Check shear rupture in HSS brace at welds:

$\phi := 0.75$

$A_{nv} := t_b \cdot b = 4.65 \cdot in^2$

$\phi R_n := 4 \cdot \phi \cdot 0.60 F_{uHSS} \cdot A_{nv} = 485.46 \cdot kip$ (J4-4)

> $R_u = 200 \cdot kip$ OK

Check shear yield in gusset brace at welds:

$\phi := 1.00$

$A_{gv} := t_{PL} \cdot b = 5 \cdot in^2$

$\phi R_n := 2 \cdot \phi \cdot 0.60 F_{yPL} \cdot A_{gv} = 216 \cdot kip$ (J4-3)

> $R_u = 200 \cdot kip$ OK

Check shear rupture in gusset brace at welds:

$\phi := 0.75$

$A_{nv} := t_{PL} \cdot b = 5 \cdot in^2$

$\phi R_n := 2 \cdot \phi \cdot 0.60 F_{uPL} \cdot A_{nv} = 261 \cdot kip$ (J4-4)

> $R_u = 200 \cdot kip$ OK

Check compression buckling of gusset along Whitmore section:

size of Whitmore section:

length $l_w := d_b + 2 \cdot b \cdot \tan(30 \cdot deg) = 17.547 \cdot in$

area $A_w := l_w \cdot t_{PL} = 8.774 \cdot in^2$

radius of gyration $r_w := \dfrac{t_{PL}}{\sqrt{12}} = 0.144 \cdot in$

unbraced length (use average distance from Whitmore section to flange of waler)

$$c := \frac{d_b}{2} + b \cdot \tan(30 \cdot deg) = 8.774 \cdot in$$

$$l_1 := a + c \cdot \tan(\theta) = 17.774 \cdot in$$

$$l_2 := a = 9 \cdot in$$

$$l_3 := a - c \cdot \tan(\theta) = 0.226 \cdot in$$

$$l := \frac{l_1 + l_2 + l_3}{3} = 9 \cdot in$$

effective length factor for gusset plate supported on one side only $k := 1.2$

$\dfrac{k \cdot l}{r_w} = 74.825$ larger than 25, so cannot use (J4-6)

Compressive buckling strength of gusset plate based on E3:

$\phi_c := 0.90$ $\quad \frac{k \cdot l}{r_w} = 74.825 \quad < \quad 4.71 \cdot \sqrt{\frac{E}{F_{yPL}}} = 133.681$

$$F_e := \frac{\pi^2 \cdot E}{\left(\frac{k \cdot l}{r_w}\right)^2} = 51.122 \cdot ksi \qquad \text{(E3-4)}$$

$$F_{cr} := \left(0.658^{\frac{F_{yPL}}{F_e}}\right) \cdot F_{yPL} = 26.81 \cdot ksi \qquad \text{(E3-2)}$$

$$\phi P_n := \phi_c \cdot F_{cr} \cdot A_w = 211.696 \cdot kip \qquad \text{(E3-1)}$$

$> \quad R_u = 200 \cdot kip$ OK

Check gusset-to-waler connection: $V_u := H_{ub} = 141.421 \cdot kip$

$R_u := V_{ub} = 141.421 \cdot kip$

Shear yield in gusset plate:

$\phi := 1.00$

$A_{gv} := (L_1 + L_2) \cdot t_{PL} = 18.375 \cdot in^2$

$\phi R_n := \phi \cdot 0.60 \cdot F_{yPL} \cdot A_{gv} = 396.9 \cdot kip$ (J4-3)

$> \quad V_u = 141.421 \cdot kip$ OK

Weld group capacity: (Table 8-4, Angle = 45deg, a = 0.00, k = 0)

$C := 4.82 \cdot \frac{kip}{in}$

$C_1 := 1.00$

$\phi := 0.75$

$L := L_1 + L_2 = 36.75 \cdot in$ may exceed max effective length, reduce effective length in calcs.

$L := 100 \cdot \frac{3}{16} \cdot in = 18.75 \cdot in$ (but still weld full length of plates to eliminate eccentricity on connection)

$$D_{min} := \frac{P_u}{\phi \cdot C \cdot C_1 \cdot L} = 2.951$$

3/16 in fillet weld each side is sufficient.

$t_f = 0.45 \cdot in$ min. size is 3/16 in, OK

Fillet weld capacity: $w := \frac{3}{16} \cdot in$

$\phi := 0.75$ (Table J2.5)

$F_{nw} := 0.60 \cdot F_{EXX} \cdot \left[1.0 + 0.50 \cdot (\sin(\theta))^{1.5}\right] = 54.487 \cdot ksi$

$A_w := 0.707 \cdot w \cdot L \cdot 2 = 4.971 \cdot in^2$

$\phi R_n := \phi \cdot F_{nw} \cdot A_w = 203.144 \cdot kip$ (J2-3)

$> \quad P_u = 200 \cdot kip$ OK

Check shear yield in gusset brace at waler welds:

$\phi := 1.00$

$A_{gv} := t_{PL} \cdot L = 9.375 \cdot in^2$

$\phi R_n := \phi \cdot 0.60 F_{yPL} \cdot A_{gv} = 202.5 \cdot kip$ (J4-3)

$> \quad P_u = 200 \cdot kip$ OK

Check shear rupture in gusset brace at waler welds:

$\phi := 0.75$

$A_{nv} := t_{PL} \cdot L = 9.375 \cdot in^2$

$\phi R_n := \phi \cdot 0.60 F_{uPL} \cdot A_{nv} = 244.687 \cdot kip$ (J4-4)

$> \quad P_u = 200 \cdot kip$ OK

Length of beam web effective in carrying shear from gusset can found as the effective weld lenght plus 5k plus the transfer length of the flange (axial) to the web (shear):

$\phi_t := 0.90 \qquad \phi_v := 1.00$

$$L_w := \frac{2 \cdot \phi_t \cdot t_f \cdot b_f \cdot F_{yW}}{\phi_v \cdot 0.60 \cdot t_w \cdot F_{yW}} = 25.071 \cdot in$$

$L_{eff} := L + 5 \cdot k_{des} + L_w = 48.571 \cdot in$

$\phi R_n := \phi_v \cdot 0.60 \cdot F_{yW} \cdot t_w \cdot L_{eff} = 510 \cdot kip \quad > \quad R_u := H_{ub} = 141.421 \cdot kip$

OK

Check compressive concentrated load capacity of waler at connection: $R_u := V_{ub}$

Web local yielding:

$$\phi := 1.0$$

$$l_b := L$$

$$\phi R_n := \phi \cdot F_{yW} \cdot t_w \cdot \left(5 \cdot k_{des} + l_b\right) = 411.25 \cdot \text{kip} \qquad \text{(J10-2)}$$

$$> \quad R_u = 141.421 \cdot \text{kip} \quad \text{OK}$$

Web Local Crippling:

$$\phi := 0.75$$

$$\phi R_n := \phi \cdot 0.80 \cdot t_w^{\,2} \cdot \left[1 + 3 \cdot \left(\frac{l_b}{d}\right) \cdot \left(\frac{t_w}{t_f}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot F_{yW} \cdot t_f}{t_w}} = 287.415 \cdot \text{kip} \qquad \text{(J10-4)}$$

$$> \quad R_u = 141.421 \cdot \text{kip} \quad \text{OK}$$

Connection design is adequate.

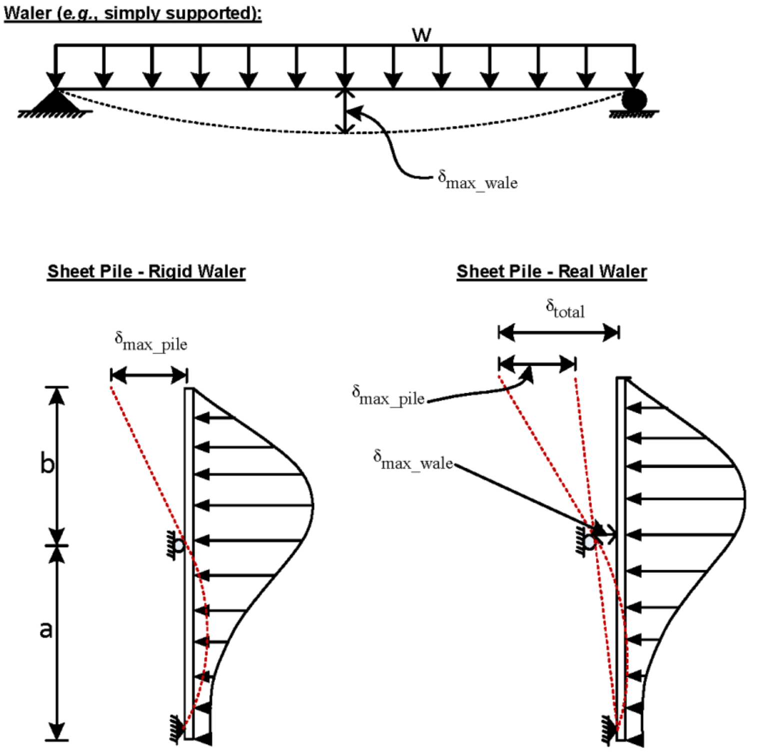
6.4 Deflection of Braced Members

While MDOT does not have a specific requirement for deflection limits for the steel members that serve as internal bracing of supported excavations, it is important to recognize their contribution to the overall deflection at the top of the sheet pile which is an important design consideration and limited to 2 inches when located adjacent to traffic. Sheet pile design methods (*e.g.*, tables or design software such as Support-IT) will usually calculate the deflection of the sheet pile of an internally braced system assuming that the braces and walers are rigid members and will assume zero deflection at these points. In reality, the struts do in fact deform only slightly under axial load, however, the flexural deformations of the walers will not be negligible and will contribute to the overall deflection at the top of the sheet pile between struts.

For determinate sheet pile designs (*i.e.*, those supported by only one level of bracing with minimal rotational support assumed at the base) a simple formula may conservatively be used to determine the total peak deflection that will occur between braces based on the sheet pile deflection calculated assuming infinitely stiff supports, the peak deflection of the walers, and their spacing.

Comments on Deflection:

-Contributions to peak deflections at the top of the sheet piles due to flexural deformations of the walers must be considered:



$\delta_{total} := \delta_{max\_pile}$

$\delta_{total} := \delta_{max\_pile} + \frac{(a+b)}{a}\delta_{max\_wale}$

Deflection limit at the top of sheet pile: $\delta_{total} \leq 2in$

For indeterminate sheet pile designs (*i.e.*, those with significant rotational restraint at the base or multiple levels of bracing), the contribution of waler deflection to the overall deflection at the top of the sheet pile can be determined using indeterminate analysis methods (*e.g.*, flexibility or direct stiffness method), conservatively assuming the waler deflection to be a support settlement for an equivalent sheet pile beam. Advanced analysis methods (*e.g.*, those based on the finite element method) may be used to determine this deflection more accurately, but such analyses must be carefully designed, validated, and documented.

6.5 Connections to Existing Structures

In some cases, it may be necessary or advantageous to make use of existing structures to anchor temporary earthworks supports (*e.g.*, anchoring braces to bridge piers). In such cases, the existing structure must be analyzed to ensure that it will have sufficient capacity to support these temporary structures during *all phases* of construction. Calculations shall be submitted demonstrating this ability along with adequate drawings that detail the connection between the existing and temporary structural elements.

Anchorage to concrete elements must be made according to the manufacturer's instructions and must also meet MDOT's specifications as described in the *Field Manual for Concrete Anchoring* (MDOT 2015) which is prepared by the MDOT Bridge Field Services section. The manual describes the types of anchorage systems that are acceptable, as well as gives guidance for design, testing, and inspection of acceptable anchoring systems using both structural adhesive and mechanical anchor systems. In designing and specifying such anchorage systems, it is necessary to review all requirements outlined in the manual. Of particular importance is a prohibition on the use of tensile anchors into concrete; supports must be configured to transfer loads in shear rather than tension. Tension anchors have proven to be problematic in such application on past projects and are no longer permitted.

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Appendix A – Michigan Department of Transportation Uniform Field Soil Classification System (Modified Unified Description)

Michigan Department of Transportation

Uniform Field Soil Classification System (Modified Unified Description)

Introduction April 6, 2009

The purpose of this system is to establish guidelines for the uniform classification of soils by inspection for MDOT Soils Engineers and Technicians. It is the intent of this system to describe only the soil constituents that have a significant influence on the visual appearance and engineering behavior of the soil. This system is intended to provide the best word description of the sample to those involved in the planning, design, construction, and maintenance processes. A method is presented for preparing a "word picture" of a sample for entering on a subsurface exploration log or other appropriate data sheet. The classification procedure involves visually and manually examining soil samples with respect to texture (grain-size), plasticity, color, structure, and moisture. In addition to classification, this system provides guidelines for assessment of soil strength (relative density for granular soils, consistency for cohesive soils), which may be included with the field classification as appropriate for engineering requirements. A glossary of terms is included at the end of this document for convenient reference.

It should be understood that the soil descriptions are based upon the judgment of the individual making the description. Laboratory classification tests are not intended to be used to verify the description, but to further determine the engineering behavior for geotechnical design and analysis, and for construction.

Primary Soil Constituents

The primary soil constituent is defined as the material fraction which has the greatest impact on the engineering behavior of the soil, and which usually represents the soil type found in the largest percentage. To determine the primary constituent, it must first be determined whether the soil is “Fine-Grained” or “Coarse-Grained” or “Organic” as defined below. The field soil classification “word picture” will be built around the primary constituent as defined by the soil types described below.

Coarse-Grained Soils: More than 50% of the soil is *RETAINED* on the (0.075 mm) #200 sieve. A good rule of thumb to determine if particles will be retained or pass the #200 sieve: If individual particles can be distinguished by the naked eye, then they will likely be retained. Also, the finest sand particles often can be identified by their sparkle or glassy quality.

Gravel Identified by particle size, gravel consists of rounded, partially angular, or angular (crushed faces) particles of rock. Gravel size particles usually occur in varying combinations with other particle sizes. Gravel is subdivided into particle size ranges as follows: (Note that particles > (75 mm) 3 inches are cobbles or boulders, as defined in the Glossary of Terms.)

Coarse -Particles passing the (75 mm) 3 inch sieve, and retained on the (19 mm) 3/4 inch sieve.

Fine -Gravel particles passing the (19 mm) 3/4 inch sieve, and retained on the (4.76 mm) #4 U.S. standard sieve.

Note: The term "gravel" in this system denotes a particle size range and should not be confused with "gravel" used to describe a type of geologic deposit or a construction material.

Sand Identified by particle size, sand consists of rock particles, usually silicate (quartz) based, ranging between gravel and silt sizes. Sand has no cohesion or plasticity. Its particles are gritty grains that can easily be seen and felt, and may be rounded (natural) or angular (usually manufactured). Sand is subdivided into particle size ranges as follows:

Coarse - Particles that will pass the (4.76 mm) #4 U.S. Standard sieve and be retained on the (2 mm) # 10 U.S. Standard sieve.

Medium - Particles that will pass the (2 mm) #10 U.S. Standard sieve and be retained on the (0.425 mm) # 40 U.S. Standard sieve.

Fine - Particles that will pass the (0.425 mm) #40 U.S. Standards sieve and be retained on the (0.075 mm) # 200 U.S. Standard sieve.

Well-Graded - Indicates relatively equal percentages of Fine, Medium, and Coarse fractions are present.

Note: The particle size of coarse-grained primary soils is important to the Soil Engineer! Always indicate the particle size or size range immediately before the primary soil constituent.

<u>Exception</u>: The use of ‘Gravel’ alone will indicate both coarse and fine gravel are present.

<u>Examples</u>: **Fine & Medium** Sand; **Coarse** Gravel. Include the particle shape (angular, partially angular, or rounded) when appropriate, such as for

aggregates or manufactured sands.

Example: **Rounded** Gravel.

Fine-Grained Soils: More than 50% of the soil PASSES the (0.075 mm) #200 sieve.

Silt Identified by behavior and particle size, silt consists of material passing the (0.075 mm) #200 sieve that is non-plastic (no cohesion) and exhibits little or no strength when dried. Silt can typically be rolled into a ball or strand, but it will easily crack and crumble. To distinguish silt from clay, place material in one hand and make 10 brisk blows with the other; if water appears on the surface, creating a glossy texture, then the primary constituent is silt.

Clay Identified by behavior and particle size, clay consists of material passing the (0.075 mm) #200 sieve AND exhibits plasticity or cohesion (ability of particles to adhere to each other, like putty) within a wide range of moisture contents. Moist clay can be rolled into a thin (3 mm) 1/8 inch thread that will not crumble. Also, clay will exhibit strength increase with decreasing moisture content, retaining considerable strength when dry.

Clay is often encountered in combination with other soil constituents such as silt and sand. If a soil exhibits plasticity, it contains clay. The amount of clay can be related to the degree of plasticity; the higher the clay content, the greater the plasticity.

Note: When applied to laboratory gradation tests, silt size is defined as that portion of the soil finer than the (0.075 mm) # 200 U.S. Standard sieve and coarser than the 0.002 mm. Clay size is that portion of soil finer than 0.002 mm. For field classification, the distinction will be strictly based upon cohesive characteristics.

<u>Organic Soils</u>:

Peat Highly organic soil, peat consists primarily of vegetable tissue in various stages of decomposition, accumulated under excessive moisture conditions, with texture ranging from fibrous to amorphous. Peat is usually black or dark brown in color, and has a distinct organic odor. Peat may have minor amounts of sand, silt, and clay in various proportions.

Fibrous Peat - Slightly or un-decomposed organic material having identifiable plant forms. Peat is relatively very light-weight and usually has spongy, compressible consistency.

Amorphous Peat (Muck) - Organic material which has undergone substantial decomposition such that recognition of plant forms is impossible. Its consistency ranges from runny paste to compact rubbery solid.

Marl Marl consists of fresh water sedimentary deposits of calcium carbonate, often with varying percentages of calcarious fine sand, silt, clay and shell fragments. These deposits are unconsolidated, so marl is usually lightweight. Marl is white or light-gray in color with consistency ranging from soft paste to spongy. It may also contain granular spheres, organic material, or inorganic soils. Note that marl will react (fizz) with weak hydrochloric acid due to the carbonate content.

Secondary Soil Constituents

Secondary soil constituents represent one or more soil types other than the primary constituent which appear in the soil in significant percentages sufficient to readily affect the appearance or engineering behavior of the soil. To correlate the field classification with laboratory classification, this definition corresponds to amounts of secondary soil constituents > 12% for fine-grained and >30% for coarse-grained secondary soil constituents. The secondary soil constituents will be added to the field classification as an adjective <u>preceding the primary constituent.</u> Two or more secondary soil constituents should be listed in <u>ascending</u> order of importance.

<u>Examples</u>: **Silty** Fine Sand; **Peaty** Marl; **Gravelly**, **<u>Silty</u>** Medium Sand; **Silty**, **<u>Sandy</u>** Clay.

Tertiary Soil Constituents

Tertiary soil constituents represent one or more soil types which are present in a soil in quantities sufficient to readily identify, but NOT in sufficient quantities to significantly affect the engineering behavior of the soil. The tertiary constituent will be added to the field classification with the phrase “with __” at the end, following the primary constituent and all other descriptors. This definition corresponds to approximately 5-12% for fine-grained and 15-29% for coarse-grained tertiary soil constituents.

<u>Example</u>: Silty Fine to Coarse Sand with **Gravel and Peat**.

Soil types which appear in the sample in percentages below tertiary levels <u>need not be included</u> in the field classification. However, the slight appearance of a soil type may be characteristic of a transition in soil constituents (more significant deposits nearby), or may be useful in identifying the soil during construction. These slight amounts can be included for descriptive purposes at the end of the field classification as “Trace of ___.”

Additional Soil Descriptors

Additional descriptors should be added as needed to adequately describe the soil for the purpose required. These descriptors should *typically* be added to the field classification before the primary and secondary constituents, in ascending order of significance (Exceptions noted below). Definitions for several descriptive terms can be found in the Glossary of Terms below. Other terms may be used as appropriate for descriptive purposes, but not for soil constituents.

Color: Brown, Gray, Yellow, Red, Black, Light-, Dark-, Pale-, etc.

Moisture Content: Dry, Moist, Saturated. Judge by appearance of sample before manipulating.

Structure: Fissured, Friable, Blocky, Varved, Laminated, Lenses, Layers, etc.

<u>Examples</u>: **Gray-Brown <u>Laminated</u>** Silty Clay; **Light-Brown <u>Saturated</u>** Fine & Medium Sand.

<u>Exceptions</u>: Certain descriptive terms such as “Fill”, may be more appropriate after the primary constituent or at the end of the field classification. Also, the description of distinct soils (inclusions) within a larger stratum should be added after the complete field classification of the predominant soil.

Examples of exceptions: Stiff Brown Sandy Clay **Fill**, with Coarse Angular Gravel and Asphalt; Gray Silty Clay with Saturated Marl, **Lenses of Saturated Fine Sand.**

Soil Strength Assessment

Soil strength refers to the degree of load-carrying capacity and resistance to deformation which a particular soil may develop. For cohesionless granular soils (sand, gravel, and silt) the relative in-place density is a measure of strength. The in-place consistency for cohesionless soils can be estimated by the Standard Penetration Test (SPT - Blow counts) and by resistance to drilling equipment or "pigtail" augers as described below. For cohesive soils, "consistency" is a measure of cohesion, or shear strength. The shear strength of clay soils can be estimated in the field using the manual methods described below, the SPT, or resistance to drilling equipment. Note that for clay soils, loss of moisture will result in increased strength; therefore, consistency of clay soils should be estimated at the natural moisture content.

The soil consistency, when appropriate and available, should be added to the field classification at the very beginning, using the terminology described below.

Examples: **Loose** Brown Rounded Fine Gravel; **Medium Stiff** Gray Moist Sandy Clay.

Cohesionless Soil

Classification	Standard Penetration, N	Relative Density, %	Resistance to Advancement of a (1.2 m) 4 ft. Long, (38 mm) 1.5 inch Diameter Spiral (Pigtail) Auger
Very Loose	< 4	0 - 15	The auger can be forced several inches into the soil, without turning, under the bodyweight of the technician.
Loose	4 - 10	15 - 35	The auger can be turned into the soil for its full length without difficulty. It can be chugged up and down after penetrating about (1/3 m) 1 ft. , so that it can be pushed down (25 mm) 1 inch into the soil.
Medium Dense	10 - 30	35 - 65	The auger cannot be advanced beyond ±(3/4 m) 2.5 ft without great difficulty. Considerable effort by chugging required to advance further.
Dense	30 - 50	65 - 85	The auger turns until tight at ±(1/3 m) 1 ft; cannot be advanced further.
Very Dense	> 50	85 - 100	The auger can be turned into the soil only to about the length of its spiral section.

Cohesive Soil

Classification	Manual Index for Consistency	Cohesion (psf)	Cohesion (kPa)	Standard Penetration, N
Very Soft	Extrudes between fingers when squeezed	0 - 250	0 - 12	< 2
Soft	Molded by light to moderate finger pressure	250 – 500	12 - 24	2 – 4
Medium Stiff	Molded by moderate to firm finger pressure	500 – 1000	24 - 48	4 – 8
Stiff	Readily indented by thumb, difficult to penetrate	1000 - 2000	48 - 96	8 – 15
Very Stiff	Readily indented by thumbnail	2000 - 4000	96 - 192	15 – 30
Hard	Indented with difficulty by thumbnail	4000 - 8000	192 - 384	> 30

Glossary of Terms

Blocky	Cohesive soil which can be broken down into small angular lumps which resist further breakdown.
Boulder	A rock fragment, usually rounded by weathering or abrasion, with average dimension of (300 mm) 12" or more.
Calcareous	Soil containing calcium carbonate, either from limestone deposits or shells. The carbonate will react (fizz) with weak hydrochloric acid.
Cemented	The adherence or bonding of coarse soil grains due to presence of a cementicious material. May be *weak* (readily fragmented), *firm* (appreciable strength), or *indurated* (very hard, water will not soften, rocklike)
Cobble	A rock fragment, usually rounded or partially angular, with an average dimension (75 to 300 mm) 3" - 12".
Dry	No appreciable moisture is apparent in the soil.
Fat Clay	Fine-Grained soil with very high plasticity and dry strength. Usually has a sticky or greasy texture due to very high affinity for water. Remains plastic at very high water contents (Liquid Limit >50).
Fill	Man-made deposits of natural soils and/or waste materials. Document the components carefully since presence and depth of fill are important engineering considerations.
Fissured	The soil breaks along definite planes of weakness with little resistance to fracturing.
Frequent	Occurring more than one per (300 mm) 1 ft thickness.
Friable	A soil which is easily crumbled or pulverized into smaller, non-uniform fragments or clumps.
Laminated	Alternating horizontal strata of different material or color, usually in increments of (6 mm) 1/4" or less.
Layer	Horizontal inclusion or stratum of sedimentary soil greater than (100 mm) 4" thick.
Lens	Inclusion of a small pocket of a sedimentary soil between (10 mm) 3/8“ and (100 mm) 4 “ thick, often with tapered edges.
Moist	Describes the condition of a soil with moderate to water content relative to the saturated condition (near optimum). Moisture is readily discernable but not in sufficient content to adversely affect the soil behavior.
Mottled	Irregularly marked soil, usually clay, with spots of different colors.
Muck	See *Amorphous Peat*, under Primary Soil Constituents heading.
Occasional	Occurring once or less per (300 mm) 1 ft thickness.
Organic	Indicates the presence of material which originated from living organisms, usually vegetative, undergoing some stage of decay. May range from microscopic size matter to fibers, stems, leaves, wood pieces, shells, etc. Usually dark brown or black in color, and accompanied by a distinct odor.
Parting	A very thin soil inclusion of up to (10 mm) 3/8“ thickness.

Saturated All of the soil voids are filled with water (zero air voids). Practically speaking, the condition where the moisture content is sufficient to substantially affect the soil behavior.

Trace Indicates appearance of a slight amount of a soil type, which may be included in the classification for descriptive or identification purposes only. The trace soil would have no effect on the soil behavior. Other modifiers such as “Slight” or “Heavy” should not be used with “Trace.”

Varved The paired arrangement of laminations in glacial sediments that reflect seasonal changes during deposition; Fine sand and silt are deposited in the glacial lake during summer, and finer particles are usually deposited in thinner laminations in winter.

See Part 2 of 2 for Appendix B

Appendix B – SupportIT Software Output

Case 1 – Cantilever TERS in Cohesionless Soil with Level Backfill
Case 2 – Cantilever TERS in Cohesionless Soil with Sloped Backfill
Case 3 – Cantilever TERS in Stiff Clay with Level Back Slope
Case 4 – Anchored Cantilever TERS in Coarse-grained Soil
Case 5 – Anchored Cantilever TERS in Stiff Clay
Case 6 – Braced Cofferdam TERS in Soft and Stiff Clay
Case 7 – Braced Cofferdam TERS in Cohesionless Soil
Case 8 – Braced Cofferdam TERS in Medium Stiff Clay
Case 9 – Cantilevered Soldier Pile TERS in Coarse-grained soil