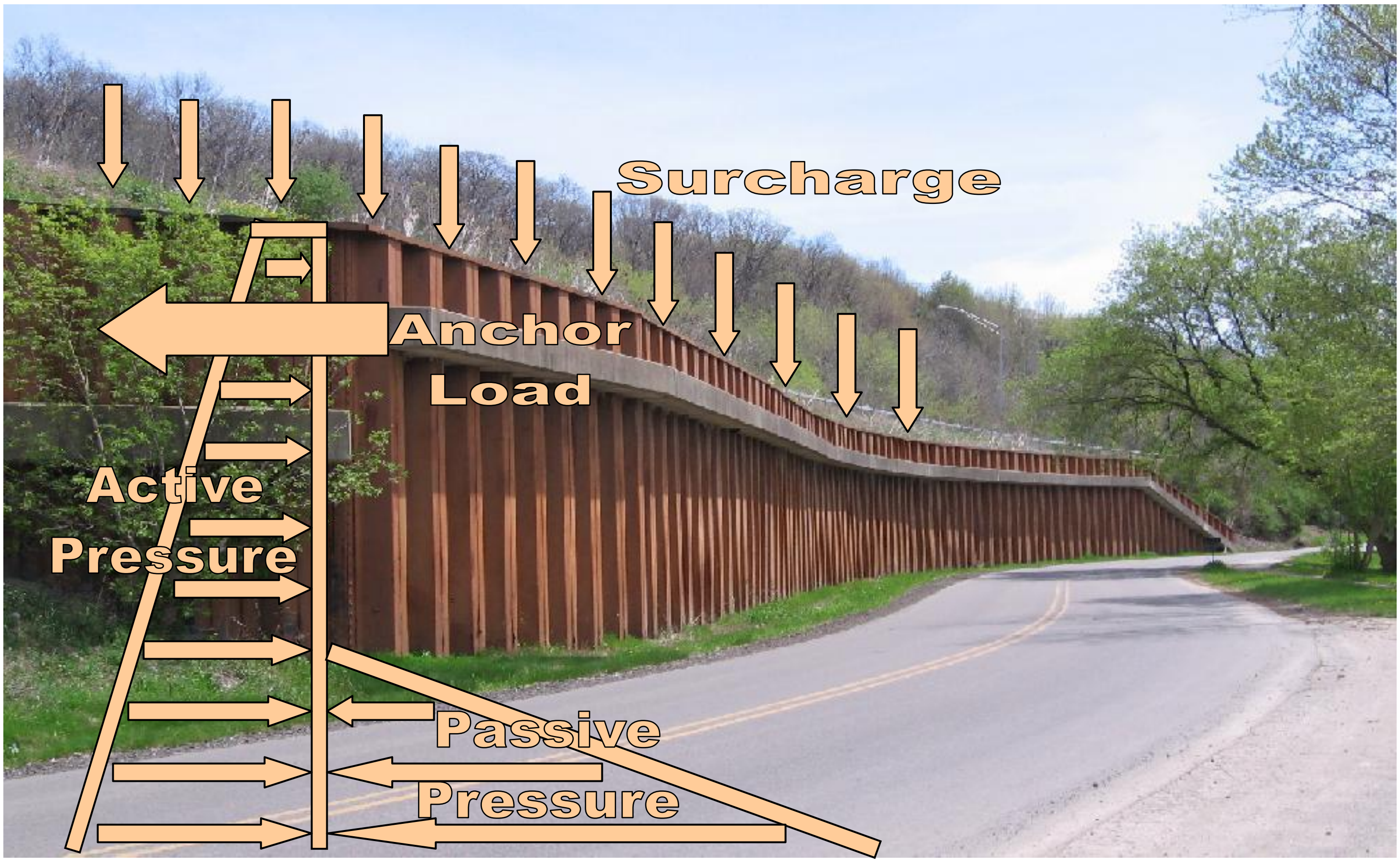
GDP-11

GEOTECHNICAL DESIGN PROCEDURE FOR FLEXIBLE WALL SYSTEMS



GEOTECHNICAL DESIGN PROCEDURE
GDP-11
Revision #4

AUGUST 2015

NEW YORK STATE OF OPPORTUNITY. Department of Transportation | Office of Technical Services | Geotechnical Engineering Bureau

GEOTECHNICAL DESIGN PROCEDURE:
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STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION

GEOTECHNICAL ENGINEERING BUREAU

AUGUST 2015

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I. INTRODUCTION

A. Purpose

The purpose of this document is to provide an acceptable design method and theory for the geotechnical design of flexible cantilevered or anchored retaining walls to be constructed on New York State Department of Transportation projects.

The following text provides a general discussion and design guidelines for these flexible wall systems. This document provides any designer with a framework for progressing a design and an understanding of the criteria which can be used during a geotechnical review. All structural aspects of these wall systems shall be performed in accordance with the Department's accepted procedures.

B. General Discussion

Flexible cantilevered or anchored retaining walls are defined in this document to include temporary or permanent flexible wall systems, or shoring systems, comprised of sheeting or soldier piles and lagging. An anchored system may include the aforementioned shoring systems supported by grouted tieback anchors, anchors to a deadman, rakers to a foundation block or braces or struts to an equivalent or existing wall system or structural element.

Sheeting members of a shoring system are structural units which, when connected one to another, will form a continuous wall. The wall continuity is usually obtained by interlocking devices formed as part of the manufactured product. In New York State, the majority of the sheeting used is made of steel, with timber, vinyl, and concrete used less often.

Soldier piles used as part of a shoring system are structural units, or members, which are spaced at set intervals. A lagging material is placed between the soldier piles to complete the shoring system. In New York State, the majority of the soldier piles used are made of steel, with concrete and timber used less often. The lagging material is usually dependent upon the design life of the wall. A temporary wall will usually incorporate timber lagging, with steel sheeting as lagging used less often. A permanent wall will usually incorporate concrete lagging with an architectural finish.

C. Soil Parameters

Soil parameters are the design assumptions which characterize the soil type. Typically, designs are progressed using effective stress parameters to account for long-term stability of the flexible wall system. For projects in design, the wall designer will be provided the soil parameters to use in the design of the flexible wall system. For projects in construction, the soil and loading parameters for the design of the detailed wall are as indicated in the contract plans. If a flexible wall system is proposed in an area which soil parameters are not listed, the Contractor shall contact the Engineer, who shall relay the request to the D.C.E.S.

II. DESIGN PREMISE

A. Lateral Earth Pressures

A flexible wall system design is required to resist the anticipated lateral pressures without undergoing significant or excessive lateral deflections. The following list provides an acceptable geotechnical theory for the development of the lateral earth pressures and potential external loads and soil backfill configurations which must be accounted for in design:

1. Earth Pressure Theory:

Use the Rankine Theory for the development of earth pressures on a flexible wall system. This theory assumes that wall friction (δ) equals zero.

2. Surcharge Loads:

The term "surcharge" refers to an additional loading on the proposed wall system. This term usually refers to traffic loading that is in proximity to the wall system. Use the Spangler Method of analysis (area load of finite length) or Boussinesq Method of analysis to determine the lateral pressure caused by the surcharge loading. The uniform surcharge is usually given a value of 250 psf (12 kPa) or an equivalent height of fill. If the designer knows that heavier construction equipment will be in the vicinity of the wall, the surcharge loading shall be increased accordingly. A uniform surcharge of at least 250 psf (12 kPa) is always assumed at the top of a wall that has a level backfill. See Appendix Page A-1.

For analysis of railroad loadings, refer to "6. *Railroad Loading*" of this Section.

3. Hydrostatic Pressure:

The identification of the existing groundwater table is necessary to design for sufficient support against all possible loadings. Since the locks of sheeting are more or less water tight when installed and become more watertight as soil is drawn in, water can be trapped behind the wall causing a head imbalance and greatly increasing the total load. Therefore, the elevation, or head difference, shall be accounted for in design of the wall system. The hydrostatic head is the difference between the groundwater elevation and the bottom of dewatered excavation. See Appendix Page A-1.

4. *Inclined Backfill:*

An inclined backfill will induce an additional load on the wall. See Appendix Page A-2. This situation shall be analyzed by the following:

Infinite Slope

If the backfill slope remains inclined beyond the limits of the active wedge, the backfill slope shall be assumed to extend infinitely away from the wall at an angle β. Using this condition, the Rankine earth pressure is a function of the angle β. To compute horizontal earth pressures, the resulting earth pressure must be adjusted by the backslope angle. Subsequent active earth forces are found using these adjusted earth pressures.

Finite Slope

If the backfill slope changes to horizontal within the limits of the active wedge of failure, the slope may be analyzed in two ways:

A The broken back slope design (A.R.E.A.) method may be used. This method is described in Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Association of State Highway and Transportation Officials (A.A.S.H.T.O.), Seventeenth Edition.

B The sloping backfill may be assumed to be equivalent to a horizontal surcharge loading, located an offset of one-half the distance from the wall to the slope break. The surcharge loading shall be equivalent to the full height of the slope.

5. Inclined Foreslope:

An inclined foreslope, or slope in front of the wall system, will reduce the amount of passive resistance available to resist loadings. See Appendix Page A-3. This situation shall be analyzed by the following:

Infinite Slope

If the foreslope extends beyond the passive wedge, the foreslope shall be assumed to extend infinitely away from the wall at an angle β. Using this condition, the Rankine earth pressure is a function of the angle β. To compute horizontal earth pressures, the resulting earth pressure must be adjusted by the foreslope angle. Subsequent passive earth forces are found using these adjusted earth pressures.

Finite Slope

If the foreslope changes to horizontal within the limits of the passive wedge of failure, the slope shall be assumed to be finite. In this case, the slope may be analyzed in two ways:

A. Infinite slope as noted above.

B. An excavation to the bottom of the slope.

Engineering judgment shall then be applied when determining which solution to use.

Note in both the infinite and finite slope cases, if the angle β is equal to or greater than the internal angle of friction of the soil, the excavation shall be assumed to extend down to the bottom of the slope.

6. *Railroad Loading:*

When the proposed excavation requires the support of railroad loads, the designer shall follow all current applicable railroad requirements. Embankment Zones and Excavation Restrictions are described in Chapter 23 of the Highway Design Manual. See Appendix Page A-4.

The system shall be designed to carry E-80 live load consisting of 80 kips axles spaced 5 ft. on centers (356 kN axles spaced 1.5 m on centers). A lower value load can be used if the railroad indicates, in writing, that the lower value is acceptable for the specific site. Use the Spangler Method of analysis (area load of infinite length) or the Boussinesq Method of analysis to determine the lateral pressure caused by the railroad loading. The load on the track shall be taken as a strip load with a width equal to the length of the ties (8 ft. 6 in.) (2.6 m). The vertical surcharge caused by each axle shall be equal to the axle weight divided by the tie length and the axle spacing.

7. *Cohesive Soil:*

Due to the variability of the length of time a shoring system is in place, cohesive soils shall be modeled in the drained condition. These soils shall be modeled as cohesiveless soils using the drained internal angle of friction. Typically, drained internal angles of friction for New York State clays range from 22E to 26E (undrained shear strength=0).

B. <u>Factor of Safety</u>

A factor of safety (F.S.) shall be applied to the coefficient of passive earth pressure (Kp). The value for the factor of safety is dependent on the design life of the wall (temporary or permanent). The passive pressure coefficients (Kp’) used in the design calculations shall be reduced as follows:

1. *Temporary Retaining Wall:*

   The factor of safety (F.S.) for a temporary wall is 1.25.
   Kp’ = Kp / 1.25.

2. *Permanent Retaining Wall:*

   The factor of safety (F.S.) for a permanent wall is 1.50.
   Kp’ = Kp / 1.50.

III. FLEXIBLE CANTILEVERED WALLS

A. General

Sheeting is driven to a depth sufficient for the passive pressure exerted on the embedded portion to resist the lateral active earth pressures acting on the cantilevered section. To achieve the required passive earth pressure resistance, embedment depths can often be quite high. Therefore, due to limitations on the availability of certain section modulus and its associated costs, cantilevered sheeting walls are usually practical to a maximum height of approximately 15 ft. (4.6 m).

Soldier piles of a soldier pile and lagging wall system are vertical structural elements spaced at set intervals, typically 6 ft. to 10 ft. (1.8 m to 3.0 m). A soldier pile and lagging wall also derives its resistance from the embedded portion of the wall but, because of the higher available section modulus, greater excavation depths can be supported as compared to those supported by sheeting. Cantilevered soldier piles are usually practical for excavations up to approximately 20 ft. (6 m) in height.

The minimum timber lagging thickness for a soldier pile and lagging wall should be determined from the table in Appendix B, taken from Lateral Support Systems and Underpinning, Vol. 1. Design and Construction, FHWA-RD-75128, April 1976.

Additional design guidance for sheeting and soldier pile and lagging walls is provided and/or referenced in Appendix D.

B. Analysis

Use either the Simplified Method or the Conventional Method for the design of a cantilevered sheeting wall. To account for the differences between the two methods, the calculated depth of embedment, obtained using the Simplified Method, shall be increased by 20%. This increase is not a factor of safety. The factor of safety shall be applied to the passive pressure coefficient as stated in "II. Design Premise: B. Factor of Safety".

Use either the Simplified Method or the Conventional Method of analysis for the development of the lateral pressures on a soldier pile and lagging wall. However, as opposed to a sheeting wall which is analyzed per foot (meter) of wall, the calculations for the design of a soldier pile and lagging wall must account for the spacing of the individual soldier piles. To determine the active pressures above the dredgeline, include a factor equivalent to the spacing in the calculations. To determine the active pressures below the dredgeline, include a factor equivalent to the width of the soldier pile (for driven piles), or diameter of the hole (for piles installed in excavated holes) in the calculations. To determine the passive resistance of a soldier pile embedded in soil, assume that the net passive resistance is mobilized across a maximum of three times the soldier pile width (for driven piles), or three times the diameter of the hole (for piles installed in excavated holes).

Both the Simplified and Conventional Method of analyses are outlined in USS Steel Sheet Piling Manual. The Simplified Method is also described in Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Association of State Highway and Transportation Officials (A.A.S.H.T.O.), Seventeenth Edition. The Conventional Method can also be found in such references as: Foundation Analysis and Design, Fourth Edition by Joseph E. Bowles and Foundations and Earth Structures by the Department of the Navy, Naval Facilities Engineering Command, Design Manual 7.2.

C. Constructability

Prior to the analysis, the designer shall evaluate the site conditions and subsurface profile to determine which type of flexible wall system is appropriate. Subsurface profiles which include cobbles, boulders and/or very compact material are sites where sheeting is not recommended and the designer should investigate alternate wall systems such as soldier piles and lagging. The designer should also focus on the type and size of equipment that will be needed to install the wall members. The designer should contemplate the limits of the wall with respect to the existing site conditions and include the design of any necessary connections. These considerations are valid for both cantilevered and anchored wall systems.

IV. FLEXIBLE ANCHORED WALLS

A. General

When the height of excavation increases over 15 ft. (4.6 m), or if the embedment depth is limited (for example, the presence of boulders or bedrock), it becomes necessary to investigate the use of additional support for the wall system. An anchored wall derives its support by the passive pressure on the front of the embedded portion of the wall and the anchor tie rod near the top of the wall. Anchored walls are suitable for heights up to approximately 35 ft. (10.5 m).

An additional factor of safety of 1.5 shall be applied to all anchor and brace loads.

Each phase of construction of an anchored wall shall be analyzed. Each phase of construction affects the lateral earth pressures on the sheeting or soldier piles and therefore, the embedment and section modulus requirements. Ex.: Phase I: cantilever analysis (excavation to install first anchor), Phase II: anchored analysis (excavation below first anchor to install second anchor), Phase III: multiple anchor analysis (excavation below second anchor to install third anchor), etc...Final Phase: multiple anchor analysis.

Additional design guidance for grouted tiebacks and steel ties is provided and/or referenced in Appendix D.

B. Analysis

1. Single Row of Anchors:

Use the Free Earth Support Method for the design of an anchored sheeting or soldier pile and lagging wall. The Free Earth Support Method assumes the wall is rigid and may rotate at the anchor level.

For the design of an anchored soldier pile and lagging wall system, the design must account for the spacing of the individual soldier piles as stated in "III. Flexible Cantilevered Walls: B. Analysis".

The designer shall analyze the effect of any additional vertical or horizontal loads imposed on the soldier piles or sheeting by the angle (orientation with respect to the wall) of the anchor. The embedment of sheeting or H-piles (or other sections used as soldier piles) below the bottom of the excavation should be checked to ensure that it is sufficient to support the weight of the wall and the vertical component of the tieback force. The factor of safety should be at least 1.5 based on the design load, assuming resistance to the vertical load below the bottom of excavation only. Pile and sheeting bearing capacity should be calculated as shown in the manual on Design and Construction of Driven Pile Foundations, FHWA-HI-97-013, Rev. November 1998 with P_d and P_D equal to the values on the excavation side of the wall.

2. Multiple Row of Anchors:

Use the method of analysis for a braced excavation, based on a rectangular (Terzaghi & Peck, 1967) or trapezoidal (Terzaghi & Peck, 1948) pressure distribution. The rectangular pressure distribution is outlined in such references as: Foundation Analysis and Design, Fourth Edition by Joseph E. Bowles, Principles of Foundation Engineering, Second Edition by Braja M. Das and in Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Association of State Highway and Transportation Officials (A.A.S.H.T.O.), Seventeenth Edition. See Appendix Page C-1.

When a rectangular or trapezoidal pressure distribution is used, all of this pressure has to be resisted by the anchors and by the bending resistance of the sheeting or H-piles. Do not consider active or passive earth pressure below the bottom of the excavation when calculating the required anchor loads, unless groundwater level is above the bottom of excavation. In that case, passive pressure may be used to help resist active earth pressure and excess hydrostatic pressure. Due consideration should be given to the effect of uplift on the passive pressure and to the amount of movement required to mobilize full passive pressure.

For the design of an anchored soldier pile and lagging wall system, the calculations shall account for the spacing of the individual soldier piles as stated in "III. Flexible Cantilevered Walls: B. Analysis".

The designer shall analyze the effect of any additional vertical or horizontal loads imposed on the soldier piles or sheeting by the angle (orientation with respect to the wall) of the anchor. The embedment of sheeting or H-piles (or other sections used as soldier piles) below the bottom of the excavation should be checked to ensure that it is sufficient to support the weight of the wall and the vertical component of the tieback force. The factor of safety should be at least 1.5 based on the design load, assuming resistance to the vertical load below the bottom of excavation only. Pile and sheeting bearing capacity should be calculated as shown in the manual on Design and Construction of Driven Pile Foundations, FHWA-HI-97-013, Rev. November 1998 with P_d and P_D equal to the values on the excavation side of the wall.

C. Anchor Types

The following are possible types of anchor support systems:

1. Grouted Tiebacks:

A grouted tieback is a system used to transfer tensile loads from the flexible wall to soil or rock. It consists of all prestressing steel, or tendons, the anchorage, grout, coatings, sheathings, couplers and encapsulation (if applicable).

2. Deadman:

A deadman may consist of large masses of precast or cast-in-place concrete, driven soldier piles or a continuous sheeting wall. The required depth of the deadman shall be analyzed based on the active and passive earth pressures exerted on the deadman. See Appendix Page C-2.

Deadman anchors must be located a distance from the anchored wall such that they can fully mobilize their passive pressure resistance outside of the anchored wall's active zone. This is described in such references as: USS Steel Sheet Piling Manual and Foundation Analysis and Design, Fourth Edition by Joseph E. Bowles. See Appendix Page C-2.

3. Struts or Braces / Rakers:

Struts or braces are structural members designed to resist pressure in the direction of their length. Struts are usually installed to extend from the flexible wall to an adjacent parallel structure. Rakers are struts that are positioned at an angle extending from the flexible wall to a foundation block or supporting substructure.

D. <u>Constructability</u>

Constructability concerns are outlined in "III. <u>Flexible Cantilevered Walls: C. Constructability</u>". The following are additional considerations which must be addressed:

1. *General:*

The mass stability of the earth-tieback-wall system will be checked by the Geotechnical Engineering Bureau unless the consultant agreement states that the consultant will do all the geotechnical design work for the project. The designer will be notified of any special requirements that have to be included in the contract to ensure mass stability.

Sheeting Walls:

In the case of permanent anchored sheeting walls (not H-pile and lagging walls with drainage zones) without special features that would permit water to drain from behind the wall (weep holes alone are ineffective), the effects of an unanticipated rise in groundwater level during periods of heavy precipitation should be considered. Unless detailed groundwater level analyses indicate otherwise, the final anchor design should be based on a 10 ft. (3 m) rise in the groundwater level compared to the highest groundwater level determined from subsurface explorations. To account for possible perched water conditions, multiply by 1.25 the calculated anchor loads above the groundwater level (after adding the 10 ft. (3 m) rise).

Soldier Pile and Lagging Walls:

H-pile (or other type of solider pile) and lagging walls should not be used in excavations below groundwater level unless the design includes appropriate positive methods to control seepage.

2. *Grouted Tiebacks:*

The presence of existing structures and utilities should be taken into account when deciding upon the location and inclination of anchors. The installation of the grouted tieback, location and inclination, should be surveyed against these existing site constraints. The design shall meet the requirements for minimum ground cover for the grouted tieback (Recommendations for Prestressed Rock and Soil Anchors, Post-Tensioning Institute, Fourth Edition: 2004).

The minimum anchor free length is:

a. 15 ft. (4.6 m) or
b. the length of the tieback from the face of the wall to the theoretical failure plane plus H/5, whichever is greater.

The theoretical failure plane is inclined at an angle of 45E- φ/2 with the vertical, where φ is the friction angle of the soil, if the backslope is horizontal. For cases where the backslope is not horizontal, the inclination of the failure plane should be determined from Foundations and Earth Structures, Design Manual 7-2, NAVFAC DM-7.2, May 1982, p.7.2-65, or by means of a trial wedge analysis. The point of intersection of the theoretical failure plane with the face of the wall for walls in non-plastic soils can be determined as follows:

a. <u>H-pile and lagging wall with single level of anchors:</u> H/10 below the bottom of the excavation, Fig. 1(a).
b. <u>Sheeting wall with single level of anchors:</u> Level below bottom of excavation where moment in sheet pile is zero. Fig. 1(a).
c. <u>H-pile and lagging wall with more than one level of anchors:</u> Bottom of excavation, Fig. 1(b).
d. <u>Sheeting wall with more than one level of anchors and groundwater level below bottom of excavation:</u> Bottom of excavation, Fig. 1(b).
e. <u>Sheeting wall with more than one level of anchors and groundwater level above bottom of excavation:</u> Level below bottom of excavation where moment in sheeting is zero.

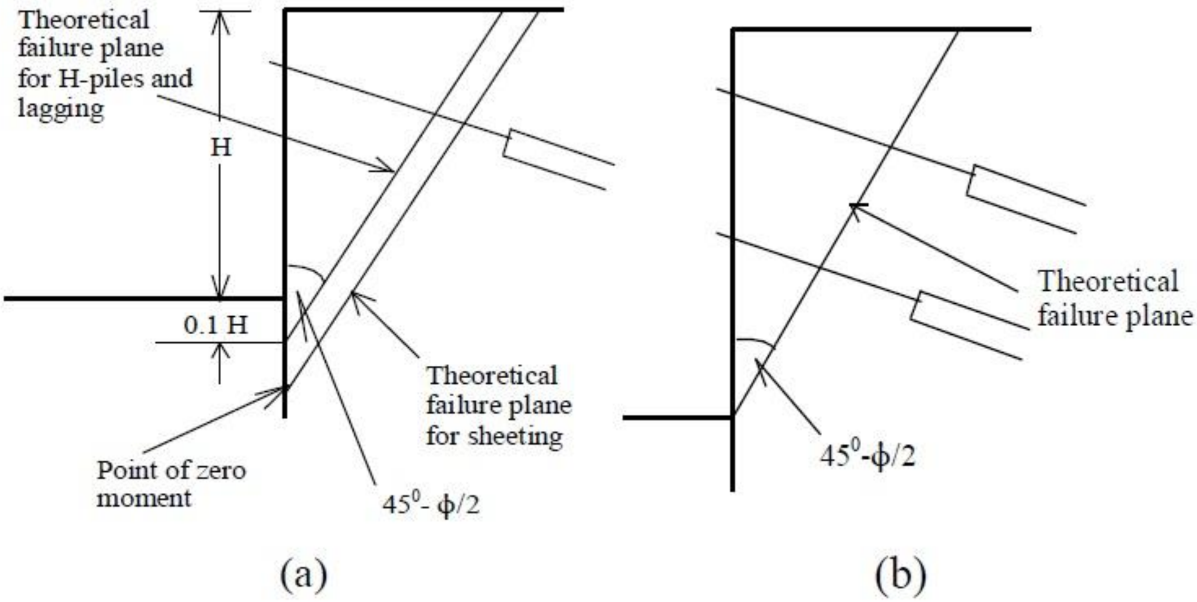

Figure 1 - Location of Theoretical Failure Plane

3. Deadman:

Both the proposed maintenance and protection of traffic scheme and the construction sequencing should be evaluated to ensure that there is no interference with the method and sequence of tie rod installation and its subsequent functioning.

4. Struts or Braces / Rakers:

The location and spacing of struts or rakers should be critiqued with respect to the allotted working space and proposed construction. Consideration should be given to access by workers, supplies and equipment.

The installation of the raker block should be evaluated with respect to the support of the wall system. The wall should be analyzed for any additional excavation or other construction impacts necessary to install the raker block.

V. REVIEW REQUIREMENTS

A. General

All designs will be reviewed using the analyses and theories stated in this document. All designs that are part of a construction submittal shall be stamped by a currently registered New York State Professional Engineer and shall follow the methods described or yield comparable results.

All designs shall be detailed in accordance with the current Departmental guidelines for the applicable item(s). Copies of these guidelines are available from the Geotechnical Engineering Bureau.

B. Flexible Cantilevered Walls

For review of the design of a flexible cantilevered wall, the following information is required:

1. All design assumptions.

2. Cite all reference material. Provide copies of relevant pages of any reference material that is used in the design and that is not included in the reference list on page 14.

3. Design elevations, including top and toe of sheeting or soldier pile, bottom of excavation, site specific soil layering and parameters. Cross sections are preferred.

4. Calculations or a computer design for the sheeting or soldier pile and lagging wall design. If a computer program is used, provide documentation of the assumptions used in writing the program.

5. Summary of constructability aspects of the proposed design as described in "III. Flexible Cantilevered Walls: C: Constructability".

An example design calculation is shown on Appendix Pages E-1 & 2 (US Customary Units) or Pages F-1 & 2 (International System of Units).

C. Flexible Anchored Walls

For review of the design of a flexible anchored wall, the following information is required:

1. All design assumptions.

2. Cite all reference material. Provide copies of relevant pages of any reference material that is used in the design and that is not included in the reference list on page 14.

3. Design elevations, including top and toe of sheeting or soldier pile, bottom of excavation, location of wales or bracing, deadman/raker block location(s), site specific soil layering and parameters. Cross sections are preferred.

4. Calculations or a computer design for the anchored sheeting or soldier pile and lagging wall design. These calculations shall include each phase of construction. If a computer program is used, provide documentation of the assumptions used in writing the program. The design loads for the anchors/braces shall account for the proposed inclination (if applicable).

5. Calculations for the deadman or raker block design (if applicable).

6. Calculations for the waler design(s) showing connections.

7. For grouted tiebacks, specify proposed free length, inclination and corrosion protection (if applicable).

8. Summary of constructability aspects of the proposed design as described in "IV. <u>Flexible Anchored Walls: D. Constructability</u>".

An example design calculation is shown on Appendix Pages E-3, 4 & 5 (US Customary Units) or Pages F-3, 4, & 5 (International System of Units).

REFERENCES

1. USS Steel Sheet Piling Design Manual, Updated and reprinted by US Department of Transportation / FHWA with permission: July, 1984.

2. Foundations and Earth Structures by the Department of the Navy, Naval Facilities Engineering Command, Design Manual 7.2: May, 1982.

3. Foundation Analysis and Design, Fourth Edition by Joseph E. Bowles.

4. Principles of Foundation Engineering, Second Edition by Braja M. Das.

5. Section 5: Retaining Walls in the Standard Specifications for Highway Bridges, Adopted by the American Association of State Highway and Transportation Officials (A.A.S.H.T.O.), Seventeenth Edition, 2002.

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7. Recommendations for Prestressed Rock and Soil Anchors, Post-Tensioning Institute, Fourth Edition: 2004.

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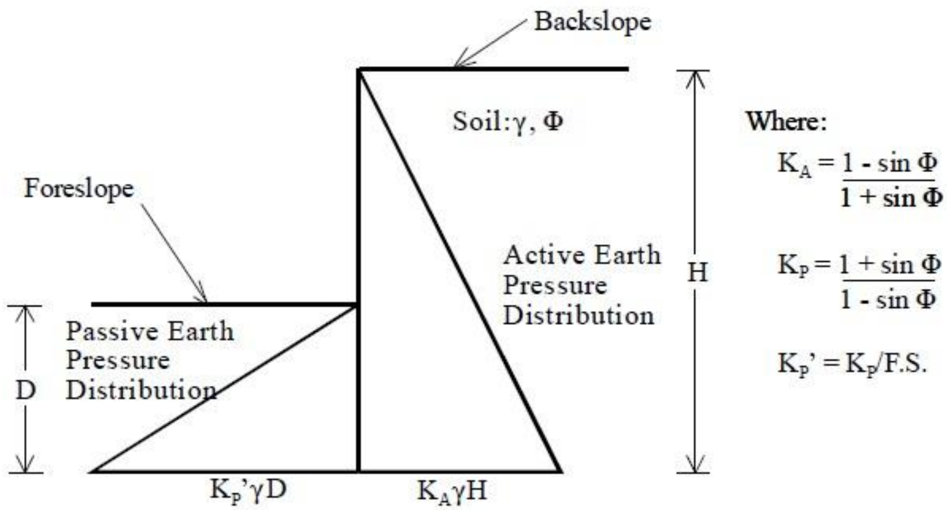
9. Soil Mechanics in Engineering Practice, 2nd ed., K. Terzaghi and R. B. Peck, 1967, John Wiley and Sons, New York. The first edition was published in 1948.

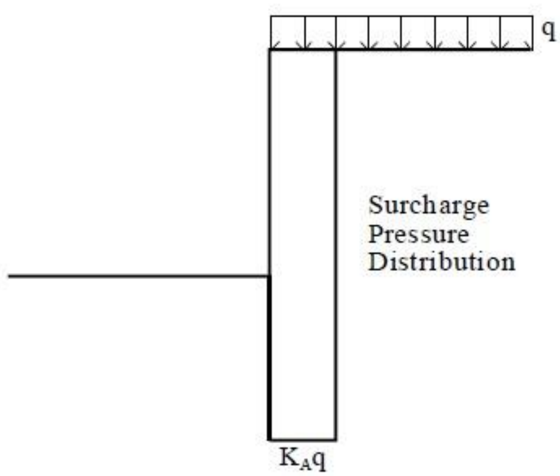
10. Design and Construction of Driven Pile Foundations, FHWA-HI-97-013 Revised November, 1998.

APPENDICIES

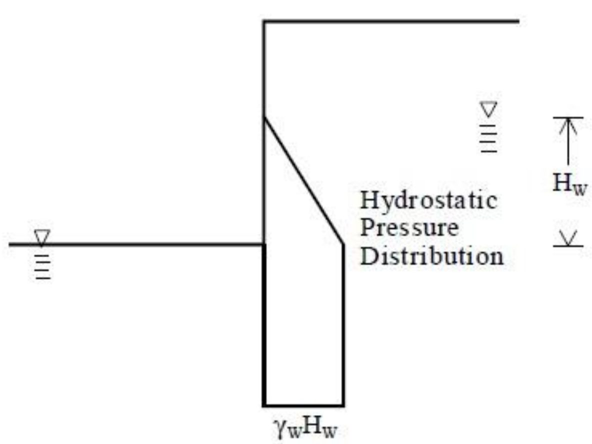
Earth Pressures



Surcharge Loads



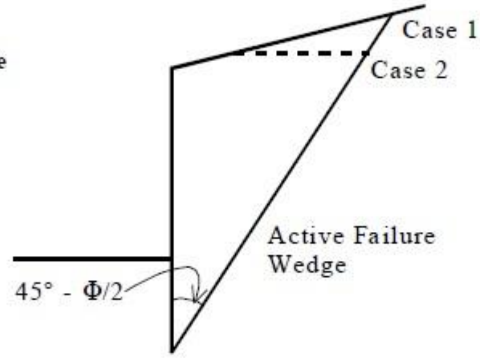
Hydrostatic Loads



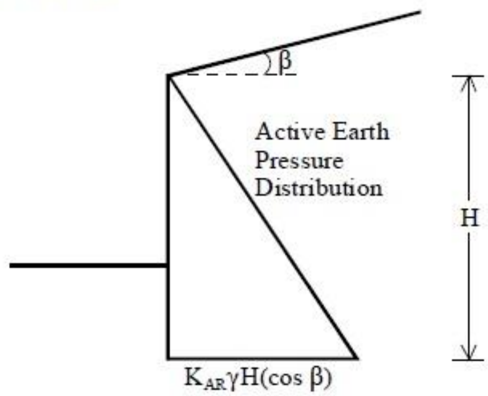
Inclined Backfill

Plot the anticipated active failure wedge line against the slope line. If the slope line intersects with the active failure wedge line, the slope can be considered infinite (Case 1), otherwise the slope can be modeled by using an equivalent surcharge (Case 2).

Active Failure Wedge

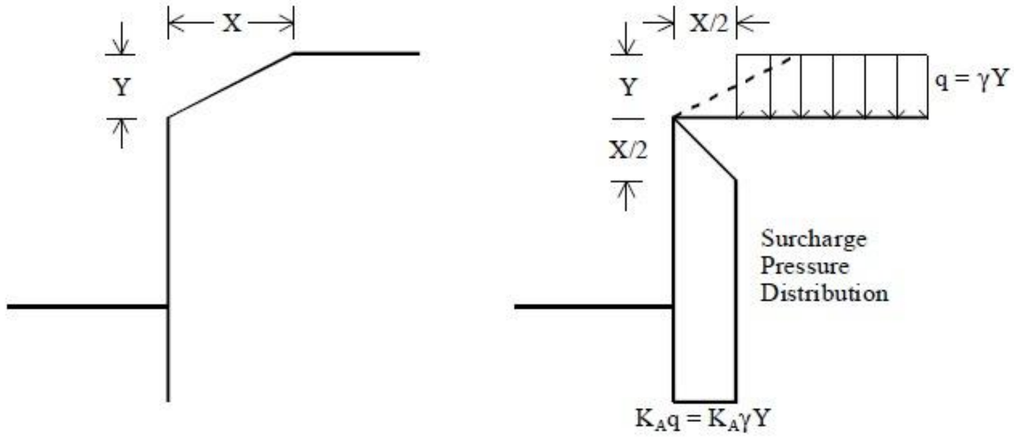


Case 1 - Infinite Slope Analysis



Where : $K_{AR} = (\cos\beta)\dfrac{[\cos\beta - (\cos^2\beta - \cos^2\Phi)^{0.5}]}{[\cos\beta + (\cos^2\beta - \cos^2\Phi)^{0.5}]}$

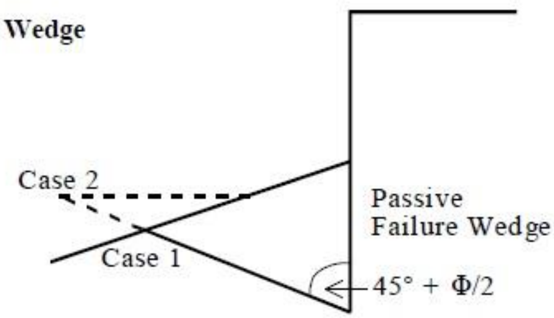
Case 2 - Equivalent Surcharge Method



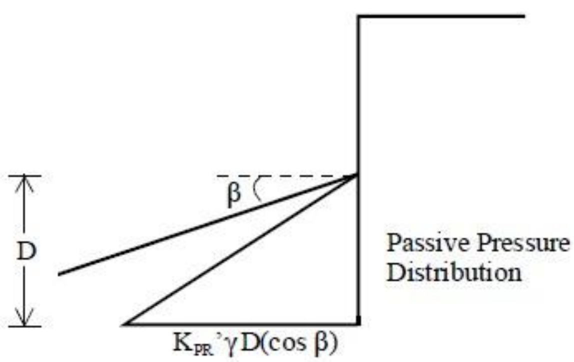
Inclined Foreslope

Plot the anticipated passive failure wedge line against the slope line. If the slope line intersects with the passive failure wedge line, the slope can be considered infinite (Case 1), otherwise the slope can be accounted for by increasing the depth of excavation (Case 2). In the latter case, both methods should be analyzed and engineering judgment used to determine the solution.

Passive Failure Wedge



Case 1 - Infinite Slope Analysis



Where : $K_{PR} = (\cos\beta)\dfrac{[\cos\beta + (\cos^2\beta - \cos^2\Phi)^{0.5}]}{[\cos\beta - (\cos^2\beta - \cos^2\Phi)^{0.5}]}$ $\qquad K_{PR}' = K_{PR}/F.S.$

Case 2 - Increase Excavation Depth to Compensate for Slope

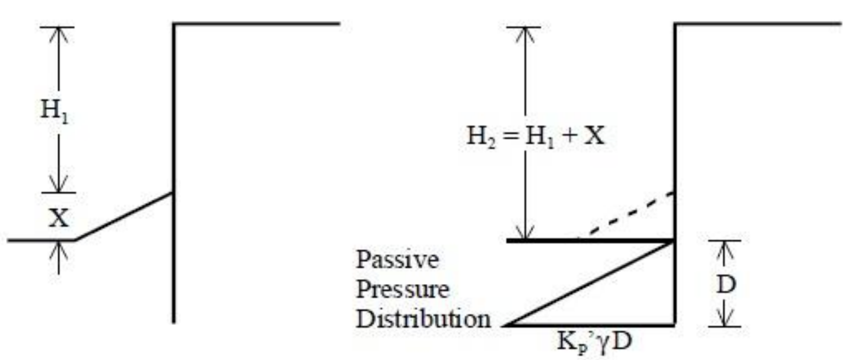
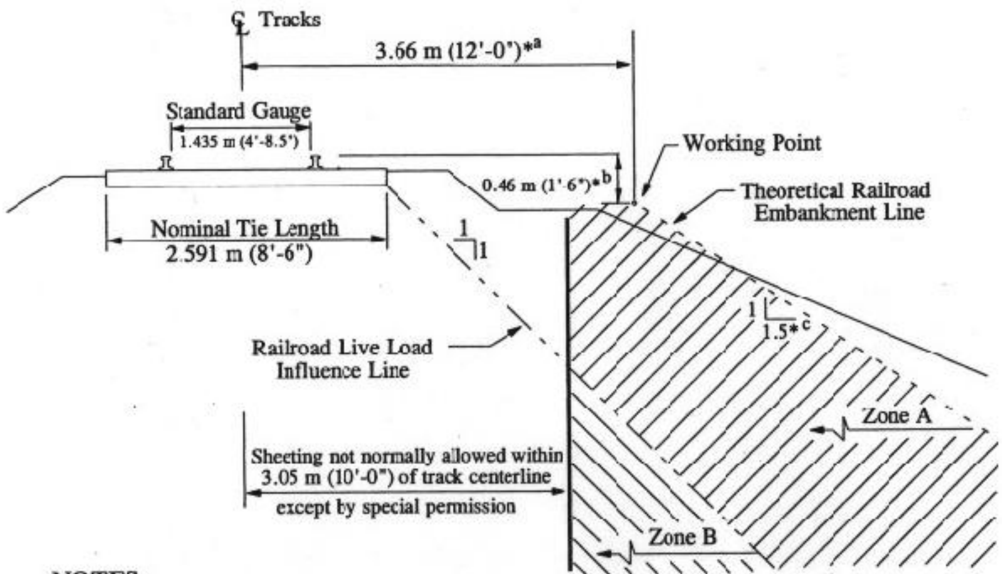


Figure 23-4 Embankment Zones and Excavation Restrictions

NOTE: Consult Geotechnical Engineering Bureau when consideration is first given to excavating in proximity to railroads.



NOTES:

1. Sheeting for the support of railroad tracks is not permitted closer than 3.05 m (10'-0") from the centerline of the nearest track.
2. Excavations located** outside of the theoretical railroad embankment do not require sheeting for the support of railroad tracks.
3. Sheeting, installed prior to excavation, is required for excavations located** within the theoretical railroad embankment (Zones A & B). The following conditions apply to the design and construction of this sheeting.
   A. A Cooper's E-80 live load, applied as a Boussinesq distribution, must be used to account for the railroad surcharge. Contact the Geotechnical Engineering Bureau for assistance with the application and calculation of the railroad surcharge.
   B. Sheeting shall be interlocking steel sheet piles, unless penetration cannot be obtained or dry, non-running, stable material will be encountered, in which cases the designer may consider soldier pile and lagging walls subject to railroad approval.
   C. During construction, the sheeting shall not extend above the top of rail elevation. After construction, sheeting for excavations located** in Zone A may be removed after the excavation is backfilled. (There are situations where this sheeting should be left in place, such as, to prevent settlement of spread footings, etc. Contact GEB.) Sheeting for excavations located** in Zone B must be left in place. All sheeting left in place ("Interim Steel Sheeting") must be cut off 0.46 m (18") below existing ground line.

*For Norfolk Southern tracks, increase indicated quantities as follows:
a. Distance from centerline of track to working point: 4.27 m (14'-0")
b. Top of Rail down to working point: 1.10 m (3'-7.25")
c. Slope of theoretical Railroad embankment line: 1(V) ON 2(H)

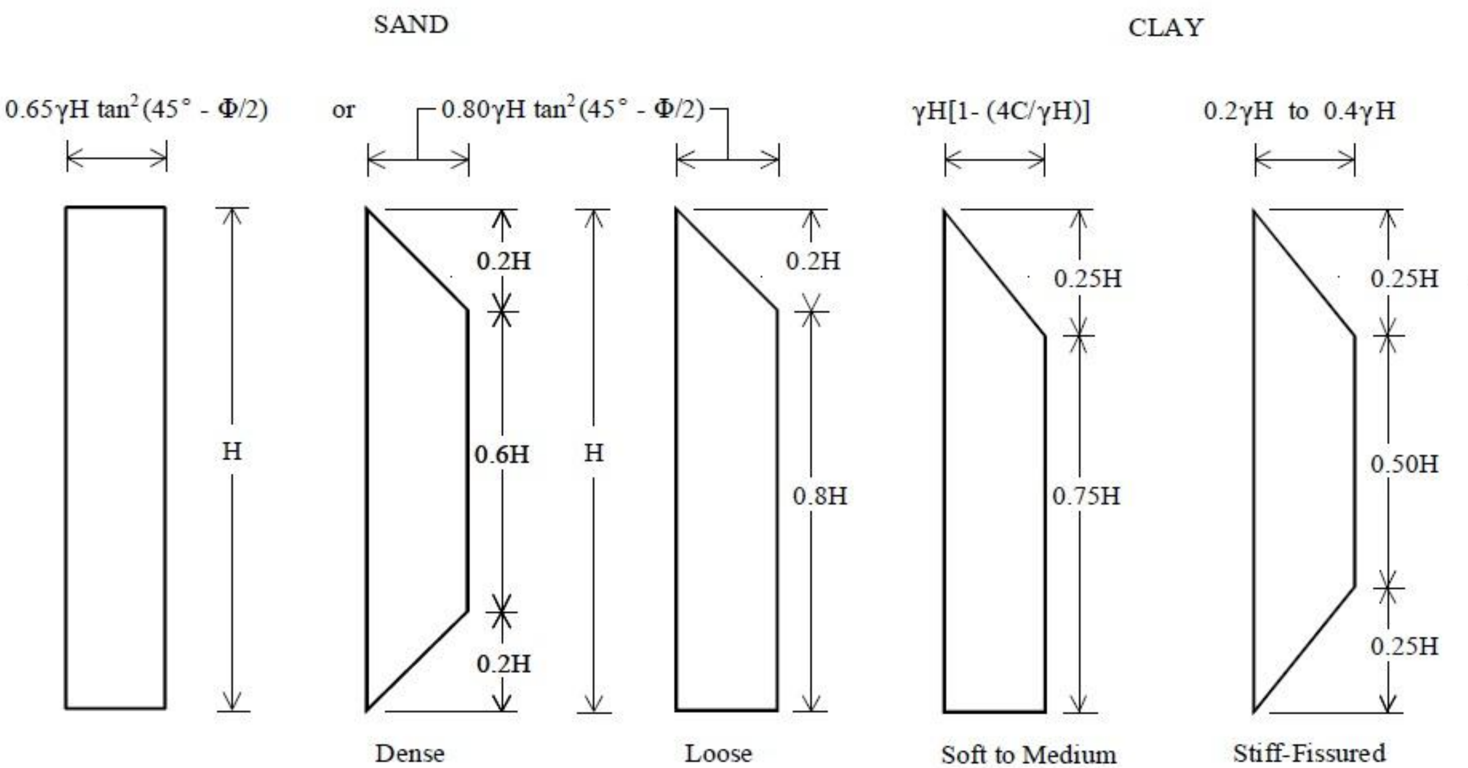
**The excavation location is defined as the point of intersection between the bottom of the excavation and the proposed line of sheeting at its closest offset from the railroad.

RECOMMENDED THICKNESSES OF WOOD LAGGING

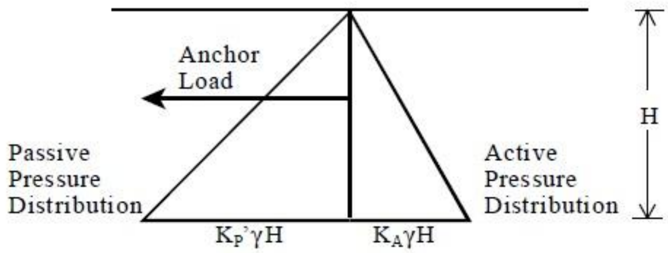
Soil Competence	Soil Description	Unified Classification	Depth (ft.)	Recommended Thickness (inches) of Lagging (rough-cut) for Clear Spans of:					
				5 ft.	6 ft.	7 ft.	8 ft.	9 ft.	10 ft.
Competent Soils	Silts or fine sand and silt above the water table.	ML SM-ML							
	Sands and gravels (medium dense to dense).	GW, GP, GM GC, SW, SP, SM	0 to 25	2	3	3	3	4	4
	Clays (stiff to very stiff); non-fissured.	CL, CH	25 to 60	3	3	3	4	4	5
	Clays, medium consistency and $\gamma H/S_u < 5$.	CL, CH							
Difficult Soils	Sands and silty sands, (loose).	SW, SP, SM							
	Clayey sands (medium dense to dense) below water table.	SC	0 to 25	3	3	3	4	4	5
	Clays, heavily overconsolidated, fissured.	CL, CH	25 to 60	3	3	4	4	5	5
	Cohesionless silt or fine sand and silt below water table.	ML; SM-ML							
Potentially Dangerous Soils	Soft clays $\gamma H/S_u > 5$.	CL, CH	0 to 25	3	3	4	5	--	--
	Slightly plastic silts below water table.	ML	15 to 25	3	4	5	6	--	--
	Clayey sands (loose), below water table.	SC	25 to 35	4	5	6	--	--	--

NOTE: In the category of "Potentially Dangerous Soils", use of lagging is questionable.
Reference: FHWA Report No. FHWA-RD-75-128 Lateral Support Systems and Underpinning, Vol. I.

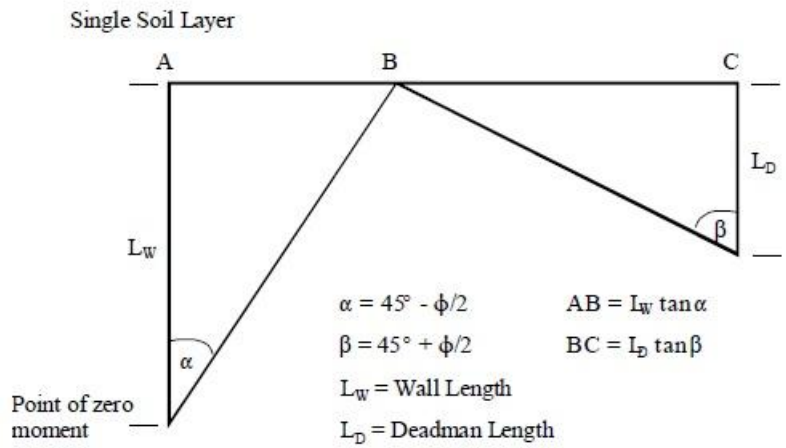
Earth Pressures For Braced Excavations



Deadman Pressure Distribution



Location Requirements for a Continuous Deadman



$\alpha = 45° - \phi/2$ $AB = L_W \tan\alpha$

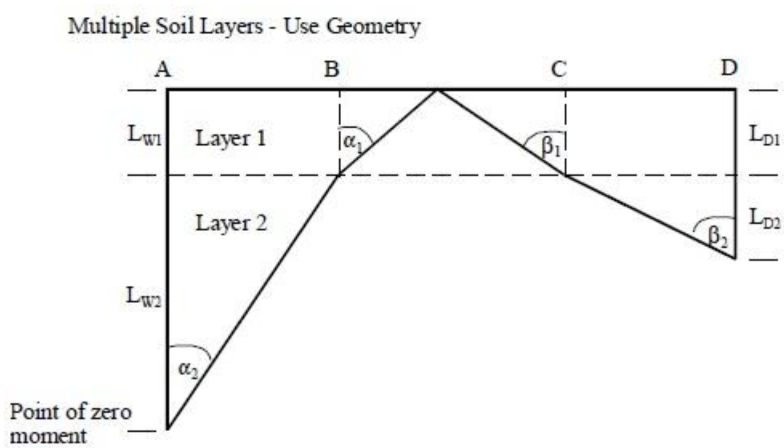
$\beta = 45° + \phi/2$ $BC = L_D \tan\beta$

L_W = Wall Length

L_D = Deadman Length

Minimum Distance between Wall and Deadman $= ABC = L_W \tan\alpha + L_D \tan\beta$

$= L_W \tan(45° - \phi/2) + L_D (\tan 45° + \phi/2)$



Minimum Distance between Wall and Deadman $= ABCD$

$= L_{W2} \tan\alpha_2 + L_{W1} \tan\alpha_1 + L_{D2} \tan\beta_2 + L_{D1} \tan\beta_1$

DESIGN GUIDELINES FOR THE USE OF
THE SOLIDER PILE AND LAGGING WALL SPECIFICATIONS

The specifications are adaptable and require the wall designer to show site specific details on the Contract Plans. The main purpose of these guidelines is to provide a checklist of the necessary information that must be placed on the Contract Plans by the wall designer. In addition to serving as a checklist, these guidelines also include some information on design issues that relate to wall constructability. These guidelines are intended to be used with the Bridge Detail (BD) Sheet BD-EE11E Excavation and Embankment Soldier Pile and Lagging Wall Sample Details, available at https://www.dot.ny.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc/ee-excavation-and-embankment-usc

The item numbers for these specifications are serialized to allow for separate bidding when multiple walls are on a single project. For projects with only one wall, the "nn" will be "01".

I. Function of the Wall - The specifications do not differentiate between permanent walls and temporary walls. The specifications may be used for either situation, with the appropriate details placed on the Contract Plans.

A. Short Term Structural Function (temporary walls) - Used material is acceptable for structural steel and untreated wood is acceptable for lagging. Walls outside the roadbed limits may either be completely removed or cut off and left in place. If, however, the wall is located within the roadbed limits, total removal is not allowed, thereby eliminating long-term settlement concerns. The specifications allow used material for structural steel, so no Special Note is necessary. Untreated wood lagging, however, must be specified on the Contract Plans. Refer to Section VIII B. for the appropriate Special Notes for temporary walls.

B. Long Term Structural Function (permanent walls) - These walls remain in place and require new materials. Special Notes are required for this situation. Refer to Section VIII A. for the appropriate Special Notes for permanent walls.

C. Support of Railroad Tracks- In general, soldier pile and lagging walls are not permitted for the support of railroad tracks. Contact the Regional Geotechnical Engineer or refer to the Geotechnical Engineering Bureau Design Manual for exceptions and limitations.

II. Payment Lines and Limits

A. Indicate Payment Lines for those items that will be computed from the Contract Plans:

Lagging - top of lagging to bottom of lagging as shown in the elevation view on the BD Sheet.

B. Indicate Payment Limits for those items that will be measured in the field:

Soldier Piles - top of pile to tip of pile, as shown in Sections X-X and Y-Y on the BD Sheet.

Holes in Earth - existing ground surface to bottom of hole in earth. The bottom of the hole in earth can be either the top of rock when a rock socket is present (as shown in Section Y-Y on the example drawing) or the tip of the pile when there is no rock socket.

Note that the upper payment limit of holes in earth may not always be the same as the upper payment limit of soldier piles.

Rock Sockets - top of rock to bottom of socket as shown in Section Y-Y on the BD Sheet.

III. Holes in Earth and Rock Sockets - The specifications allow soldier piles to be either driven or placed in a hole. Holes are necessary for one (or more) of the following reasons:

- Rock sockets are required.
- Possibility of encountering obstructions or very compact material.
- To minimize vibrations.

If holes are specified, the minimum diameter of the hole should be selected to provide a 3 in. (75 mm) minimum clear space around the soldier pile (i.e. soldier pile diagonal dimension plus 6 in. (150 mm)).

IV. Soldier Piles - Provide only the relevant soldier pile information outlined in the table on the BD Sheet. Factors to consider in selection of a soldier pile section are:

A. Pile Section - Select either an HP section or a relatively square W section because:

For Driven Piles

- The section modulus is nearly the same in the x and y directions and, therefore, small rotations during placement will not result in a deficient section.
- The pile section will fit a standard drive head.
- The reinforced shoes for driving are standard.
- The driving stresses will be evenly distributed.

For Piles Placed in Holes

- The required hole diameter is minimized.

B. Flange Width - Select a soldier pile section with a minimum flange width of 11 in. (275 mm). This will provide a minimum bearing area for the lagging of 3 in. (75 mm), plus 1 ½ in. (38 mm) of clearance between the end of the lagging and the web (necessary for concrete lagging).

C. Availability - Select soldier pile sections which are available domestically. The Manual of Steel Construction indicates the availability of sections.

D. Yield Stress - Unless otherwise indicated on the Contract Plans, ASTM A36 steel will be furnished. If a higher yield stress is necessary, it must be indicated by a Special Note (see Section VIII C.).

V. Backfilling of Holes in Earth and Rock Sockets - Two types of backfill are allowed in the specifications: concrete and (excavatable) grout. The designer must specify the backfill type(s) and limit(s) on the Contract Plans (refer to Section Y-Y on the BD Sheet). Some things to consider when selecting type(s) and limit(s) of backfill are:

- Concrete must be used in rock sockets (a requirement of the specifications).
- The backfill between the bottom of the hole in earth (or the top of rock socket, if any) and the dredgeline may be either grout or concrete. Concrete would be appropriate for permanent walls.
- Grout must be used above the dredgeline, since it can be excavated.

VI. Lagging - Show the lagging type. For the type of lagging chosen, show the following:

- Treated wood (permanent walls) - indicate full dimension thickness.
- Untreated wood (temporary walls) - indicate full dimension thickness.
- Precast concrete panels - show the panel design using Detail A on the BD Sheet. The Geotechnical Engineering Bureau can, if requested, provide the maximum moment. The design of the panels is to be provided by the Regional Design Group, the Office of Structures or the Design Consultant.
- Steel sheeting - indicate minimum section modulus.

VII. Waling and Bracing - Show the following information, if applicable:

- Elevation of walers and braces.
- Spacing of braces.
- Section modulus of walers.
- Design section of braces.

VIII.Examples of Special Notes to be Placed on the Contract Plans

A. Long Term Structural Function (permanent walls)
   The soldier pile and lagging wall shown will be left in place. Used material is not permitted in the Item for Installing Soldier Piles for Soldier Pile and Lagging Wall.

B. Short Term Structural Function (temporary walls)
   1. Located inside of roadbed limits (Refer to NYSDOT Standard Specifications, page 1-2, for the definition of roadbed limits).
      When no longer necessary for excavation support, remove lagging to a minimum of 2 ft. (0.6 m) below subgrade surface or 4 ft. (1.2 m) below final ground surface. Cut off and remove soldier piles to a minimum of 2 ft. (0.6 m) below subgrade surface or 4 ft. (1.2 m) below final ground surface.

   2. Located outside of roadbed limits.
      a. For walls where no adjacent structure or utility is present, no Special Notes are necessary because the specifications address this situation.
      b. For walls where adjacent structures or utilities might be damaged by removal operations:
         When no longer necessary for excavation support, remove lagging to a minimum of 2 ft. (0.6 m) below subgrade surface or 4 ft. (1.2 m) below final ground surface. Cut off and remove soldier piles to a minimum of 2 ft. (0.6 m) below subgrade surface or 4 ft. (1.2 m) below final ground surface.

C. Other Special Notes
   1. If a higher yield stress is necessary for the soldier piles:
      Provide soldier pile sections meeting the requirements of ASTM A572 Grade 50 Steel.
   2. For situations where a casing will be necessary for installing holes in earth (i.e. loose cohesionless soil, high groundwater, adjacent structures, utilities, etc.):
      Temporary sleeves or casings are required for the Item for Holes in Earth for Soldier Pile and Lagging Walls. No extra payment will be made for the casing.
   3. For situations where it appears installing holes in earth will be very difficult and possibly require special equipment and/or procedures:
      Due to the presence of ______________________, progressing the Holes in Earth for Soldier Pile and Lagging Wall may require special equipment and/or procedures. No extra payment will be made for special equipment and/or procedures.

SELECTING A SOLDIER PILE SECTION FOR A SOLDIER PILE AND LAGGING WALL WITH ROCK SOCKETS

The bending moment in a soldier pile varies with depth and the material in which it is embedded. The maximum bending moment ($\mathbf{M_{max}}$) expected to occur in the soldier pile is used to size the pile. The $\mathbf{M_{max}}$ for a cantilevered soldier pile wall with some or all of its embedment in rock (i.e., rock sockets) is traditionally dependant on the elevation of the rock surface assumed during design.

During construction, it is likely that the actual rock surface will vary somewhat from the elevation(s) assumed in design. This occurs because the amount of subsurface information available to the designer is generally insufficient to precisely define the rock profile. During construction, if the actual rock elevation is found to be <u>lower</u> than the assumed rock elevation, the soldier pile section specified on the Contract Plans is often no longer adequate. When this occurs, the first step is for the wall designer to review the assumptions from the original analysis and compare them to the actual site conditions (i.e. soil conditions, ground water elevation, surcharge loads, etc.). A re-analysis with the revised assumptions may prove the soldier pile section shown on the Contract Plans is still adequate. If a re-analysis indicates the section is insufficient, possible remedies and their associated consequences, are as follows:

Remedy- Increase the section modulus of the soldier pile by either ordering new steel or welding steel cover plates to the flanges of the existing soldier piles.
Consequence- Delays, orders-on-contract, claims.

Remedy- Reduce the factors of safety in the original design.
Consequence- Not acceptable for permanent walls or critical temporary walls.

Remedy- Change the wall design by adding anchors.
Consequence- R.O.W. restrictions, additional design, delays, requires specialty Contractor and equipment, orders-on-contract, claims.

Remedy- Change the wall design by reducing the soldier pile spacing.
Consequence- If decreasing the pile spacing is an option, payment will be required for the additional quantities of drilling and soldier piles.

The most effective way to ensure safety and avoid costly delays and orders-on-contract is to specify a soldier pile section that is able to accommodate likely variations in rock elevation encountered during construction.

The following design recommendations provide a rational approach to sizing soldier piles for walls with rock sockets:

I. Determine the likelihood of a varying rock surface at the project site.

A. The most obvious indication of this condition is differing rock elevations in the available subsurface explorations. In cases where there are few drill holes near the proposed wall location, consider requesting additional drill holes or a seismic refraction survey to better define the rock profile.

B. Discuss the probable rock profile with the Area and Regional Geotechnical Engineer.

C. At the time the rock socket design request is made (refer to Section III.), consult with an Engineering Geologist on the variability of the rock surface at the project site. In general, the Regions with the most variable rock surfaces are 1, 2, 7, 8 and 11.

D. Based on the results of A, B, and C above, decide if the rock is likely to differ by more than 2 ft. (0.6 m) from the assumed rock elevation. The specification allows for a 2 ft. (0.6 m) difference between the rock elevation shown on the Contract Plans and the actual rock elevation before the E.I.C. is obligated to contact the Geotechnical Engineering Bureau for recommendations.

II. Sizing a soldier pile section.

A. General

For a typical cantilevered soldier pile wall with all pile embedment in soil, $\mathbf{M_{max}}$ occurs at a depth of between 0.5H* (for compact soils) and 1.0H* (for loose or soft soils) below the dredge line, where H is the exposed height of the wall (refer to Figure 1).

* These ratios are the result of a study performed by the Structures Foundation Section of the Geotechnical Engineering Bureau. The study was based on a "typical" cantilevered soldier pile wall constructed in New York State. The assumptions used in the study are as follows:

- Temporary wall with a factor of safety of 1.25 on $K_{p.}$
- A 2 ft. (0.6 m) traffic surcharge: 250 psf (12.0 kPa).
- No slope above or in front of the wall.
- One soil layer with a unit weight of 120 pcf (18.8 kN/m^3) and friction angles ranging from 22° to 37°.
- Wall friction (δ) equals zero.
- Groundwater at the dredge line.
- A predrilled hole with a diameter of 24 in. (600 mm).
- Soldier pile spacing of 6 ft. (1.8 m) and 8 ft. (2.4 m).
- An exposed wall height ranging from 8 ft. (2.4 m) to 16 ft. (5.0 m).

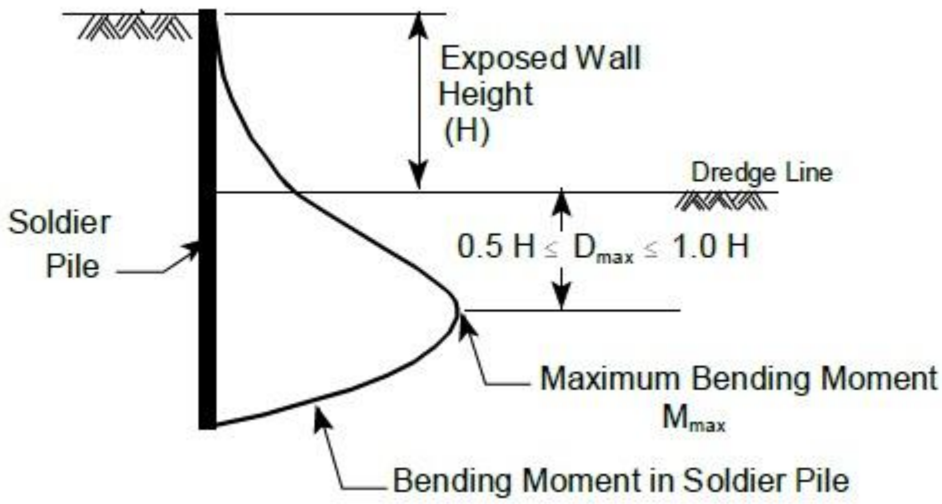


Figure 1
Bending Moment vs. Depth for a Typical
Cantilevered Soldier Pile Wall with
All Pile Embedment in Soil ("All Soil Case")

For a typical cantilevered soldier pile wall with some or all pile embedment in rock (i.e., rock sockets), the location of $\mathbf{M_{max}}$ is not straightforward and it is customary to use the larger of the following bending moments to size the pile:

1. The moment at the rock surface, or
2. The maximum moment between the dredge line and the rock surface.

Whichever case results in a larger moment depends on the distance between the dredge line and the rock surface, and is discussed in Steps C and D below.

B. Analysis

When the Geotechnical Engineering Bureau's "CASH" program is used, the output provides the bending moment values in one foot increments of pile depth. The wall designer should run the CASH program assuming a soldier pile wall with all pile embedment in soil (hereafter referred to as the "all soil" case). This output enables the wall designer to see the effect on the pile bending moment if the rock surface is lower than assumed. It also provides the depth of $\mathbf{M_{max}}$ below the dredge line for the 'all soil' case ($\mathbf{D_{max}}$), which is necessary for Steps C and D below. If CASH or a computer program that provides the moment distribution with depth is not accessible, the moments below the dredge line for the "all soil" case can be calculated by hand at several depths.

C. Rock surface at a depth of $\mathbf{D_{max}}$ or greater (Refer to Figure 2)

In general, if the rock surface is located lower than $\mathbf{D_{max}}$ below the dredge line, the $\mathbf{M_{max}}$ will occur somewhere between the dredge line and the rock surface. Regardless of the variability of the rock surface, the soldier pile should be sized using the $\mathbf{M_{max}}$ as identified in the "all soil" CASH run.

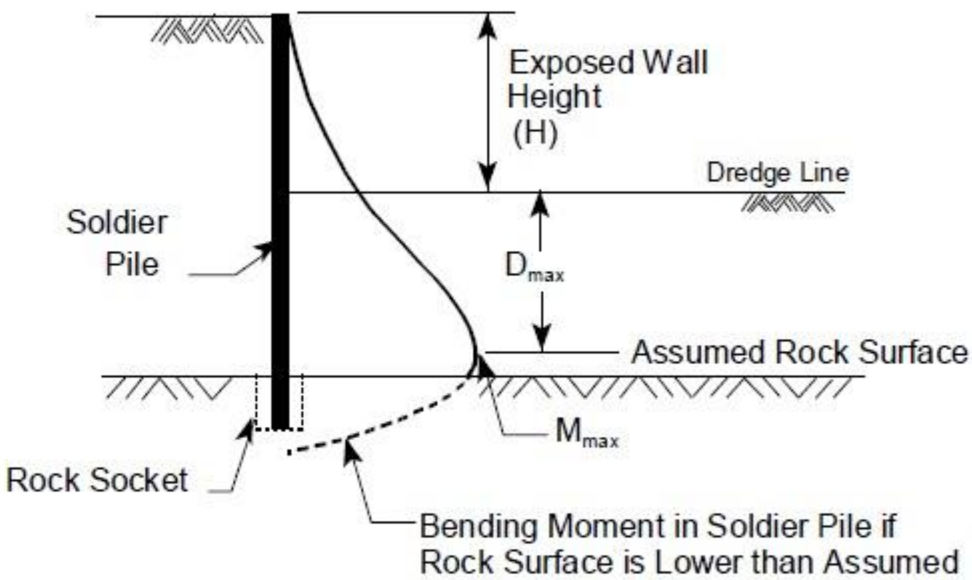


Figure 2

Bending Moment vs. Depth for a Typical Cantilevered
Soldier Pile Wall Embedded in Soil and Rock
(Rock at a Depth $\geq D_{max}$ Below Dredge line)

D. Rock surface between dredge line and $\mathbf{D_{max}}$ (Refer to Figure 3)

For a rock surface located higher than $\mathbf{D_{max}}$ below the dredge line, the $\mathbf{M_{max}}$ will occur at rock surface. It is assumed that the rock socket provides a rigid support (i.e., the pile is concreted into rock) and the soldier pile experiences no increase in bending moment below the top of the socket.

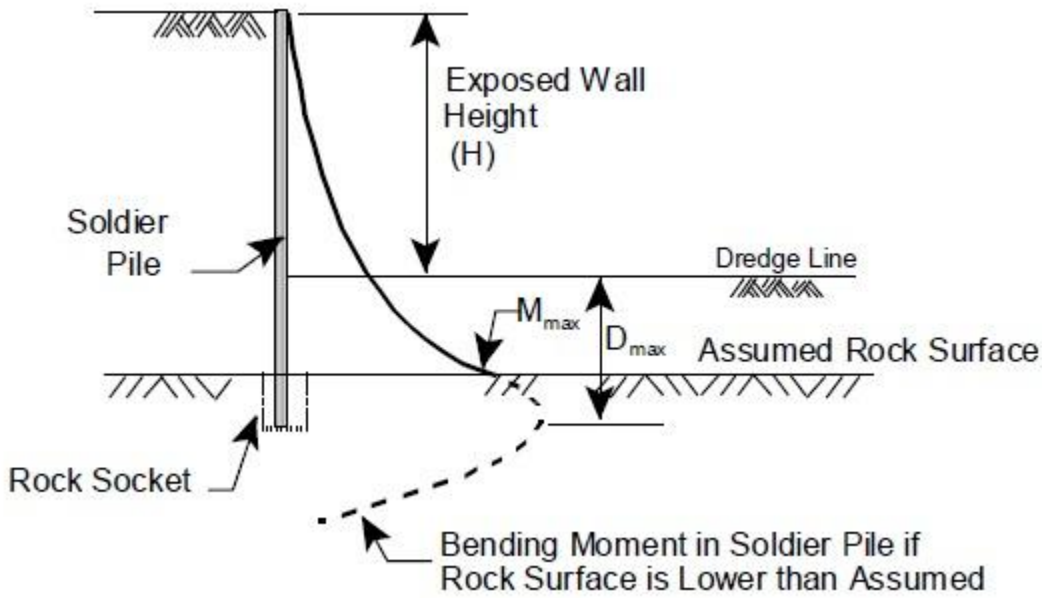


Figure 3
Bending Moment vs. Depth for a Typical Cantilevered
Soldier Pile Wall Embedded in Soil and Rock
(Rock at a Depth < D_{max} Below Dredge line)

The $\mathbf{M_{max}}$ for a rock surface located at $\mathbf{D_{max}}$ can be several times larger than the $\mathbf{M_{max}}$ for a rock surface at the dredge line. If the actual rock surface is lower than assumed in design, the applied soldier pile bending moment can be significantly greater than originally assumed, and the following cases must be considered when sizing the soldier piles:

CASE 1- Rock surface <u>IS NOT</u> likely to vary by more than 2 ft. (0.6 m) from the assumed elevation:

As an <u>absolute minimum requirement</u>, the soldier pile should be sized using the moment 2 ft. (0.6 m) below the assumed rock elevation (this moment can be found on the "all soil" Cash output).

CASE 2- Rock surface <u>IS</u> likely to vary by more than 2 ft. (0.6 m) from the assumed elevation:

Serious consideration should be given to sizing the soldier pile using the $\mathbf{M_{max}}$ identified in the "all soil" CASH run. Another approach is to design the wall with a large enough spacing so that the spacing can be reduced during construction, if necessary. If this approach is considered, the cost of the additional quantities for drilling and steel must be weighed against the cost of sizing the soldier piles using $\mathbf{M_{max}}$ as discussed above. If rock appears to be highly variable, the designer should investigate a different wall type, such as an anchored wall, in order to avoid the use of rock sockets altogether.

III. Rock Socket Design

Regardless of who designs the wall (Geotechnical Engineering Bureau, M.O. Structures Division, Regional Design Group, or Design Consultant), the rock socket designs must be provided by an Engineering Geologist of the Geotechnical Engineering Bureau. In addition to the information about the rock at the project site, the Engineering Geologist must be provided with the following information to design a rock socket:

- Soldier pile spacing,
- Flange width,
- Maximum bending moment, and
- Hole diameter.

DESIGN GUIDELINES FOR THE USE OF THE SHEETING AND EXCAVATION PROTECTION SYSTEM SPECIFICATIONS

The specifications are adaptable and require the wall designer to show site specific details on the Contract Plans. Design guidelines are outlined in the NYS Department of Transportation Bridge Manual, Section 4 Excavation, Sheeting and Cofferdams, available at:
https://www.dot.ny.gov/divisions/engineering/structures/manuals/bridge-manual-usc

These guidelines are also intended to point out how support system items are related to excavation items and to explain which excavation items include protection system provisions. These guidelines are intended to be used with the Bridge Detail (BD) Sheets:

Sheeting in Stage Construction
BD-EE16E Excavation and Embankment Sample Drawing of Stage Construction (2 of 2), available at:
https://www.dot.ny.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc/ee-excavation-and-embankment-usc

Braced Sheeting Systems
BD-EE10E Excavation and Embankment Braced Excavation Details, available at:
https://www.dot.ny.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc/ee-excavation-and-embankment-usc

DESIGN GUIDELINES FOR THE USE OF THE GROUTED TIEBACK SPECIFICATIONS

The specifications are adaptable and require the wall designer to show site specific details on the Contract Plans. The main purpose of these guidelines is to provide a checklist of the necessary information that must be placed on the Contract Plans by the wall designer. In addition to serving as a checklist, these guidelines also include some information on design issues that relate to wall constructability. These guidelines are intended to be used with the Bridge Detail (BD) Sheet BD-EE12E Excavation and Embankment Tieback Wall Details, available at:
https://www.dot.ny.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc/ee-excavation-and-embankment-usc

DESIGN GUIDELINES FOR THE USE OF THE STEEL TIES SPECIFICATIONS

The specifications are adaptable and require the wall designer to show site specific details on the Contract Plans. The main purpose of these guidelines is to provide a checklist of the necessary information that must be placed on the Contract Plans by the wall designer. In addition to serving as a checklist, these guidelines also include some information on design issues that relate to wall constructability. These guidelines are intended to be used with the Bridge Detail (BD) Sheet BD-EE12E Excavation and Embankment Tieback Wall Details, available at: https://www.dot.ny.gov/main/business-center/engineering/cadd-info/drawings/bridge-detail-sheets-usc/ee-excavation-and-embankment-usc

1. Show the position of the steel ties (plan location, elevation, etc.).

2. Show the design load of the steel ties. The design load should include a factor of safety of 1.5.

3. Indicate if the steel ties will be installed inside a carrier conduit. Carrier conduits are used only to facilitate construction of the steel ties and are not related to corrosion protection. The most common situation where the use of a carrier conduit is advantageous is when traffic lanes have to be maintained between the wall and the deadman, i.e. traffic is located over the proposed location of the steel ties. In general, it is difficult to install steel ties (whether they be threaded bars with a coupler or seven wire strand cables) in stages. Conduit, however, can be installed in stages by trenching, or in one operation by drilling (if no obstructions are present). Once all the conduit is installed, the steel ties can be pushed through while traffic is maintained above.

4. If steel ties (or steel ties with carrier conduit) will be installed in trenches, indicate excavation and backfill limits and payment items for trenches.

5. A. Walls that are constructed from the "top down" (or walls that are excavated in front): These walls are constructed by excavating downward to some depth below the proposed steel tie elevation, installing the steel tie, and then continuing the excavation.
      1. Show the maximum depth of excavation permitted below the proposed steel tie elevation. This number is usually 1 ½ ft. or 2 ft. (0.45 m or 0.6. m).
      2. Check the wall stability for the temporary condition before the steel tie is installed. For walls with more than one level of ties, multiple wall stability calculations will be required.

   B. Walls that are constructed from the "bottom up" (or walls that are backfilled behind): These walls are constructed by backfilling behind the wall up to the proposed steel tie elevation, installing the steel tie, and then continuing the backfilling.
      1. Show the elevation the backfill should be placed to before the proposed steel tie is

installed.

2. Check the wall stability for the temporary condition before the steel tie is installed. For walls with more than one level of ties, multiple wall stability calculations will be required.

6. The specification allows the Contractor the option of using threaded steel bars or seven wire strand cables. If seven wire strand cables are selected, they must be loaded and locked off immediately after installation (i.e. before any further wall excavation/backfill). The lock off load serves to remove slack from the cables and set the locking wedges in the anchor head.

   The lock-off load must be shown on the Contract Plans and is dependent on the type of wall constructed as follows:

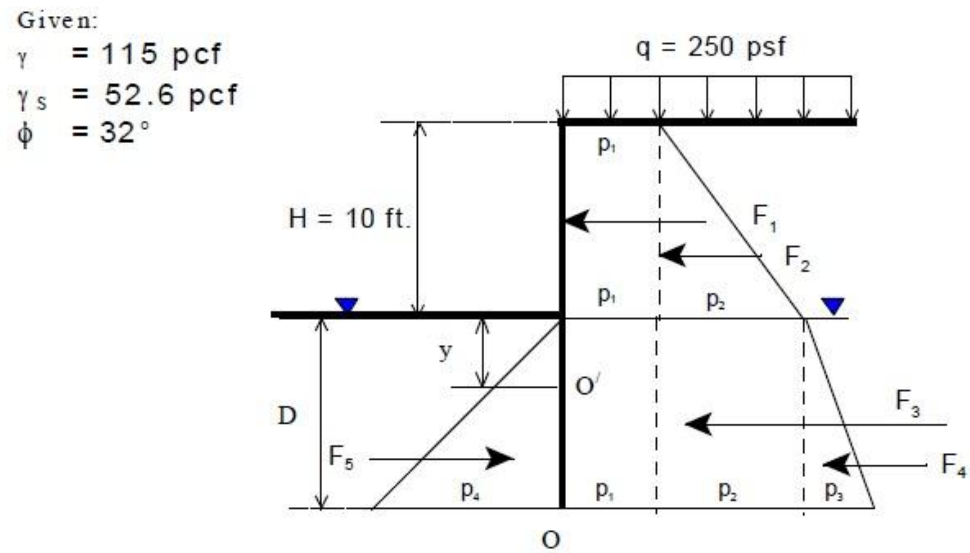
   A. Walls constructed from the "top down": The lock-off load is 80 percent of the design load.
   B. Walls constructed from the "bottom up": The lock-off load is 2 kips per strand (9 kN per strand).

   For either type of wall construction, check for adequate passive resistance behind the wall to resist the lock-off load.

7. Show corrosion protection at the anchor heads for permanent steel ties.

EXAMPLE - CANTILEVERED SHEETING WALL *(US CUSTOMARY UNITS)*

Step 1: Using the Simplified Method, determine the depth of embedment and required section modulus for the following situation (permanent sheeting).



Step 2: Rankine Theory for a level backfill:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = 0.31$$

$$K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi} = 3.25 \qquad K_p' = \frac{K_p}{1.5} = 2.18$$

Step 3: Compute the pressures:

p_1 =	$(K_a)(q)$ =	$(0.31)(250)$	= 77.5 psf
p_2 =	$(K_a)(\gamma)(H)$ =	$(0.31)(115)(10)$	= 356.5 psf
p_3 =	$(K_a)(\gamma_s)(D)$ =	$(0.31)(115\text{-}62.4)(D)$	= 16.31 D psf
p_4 =	$(K_p')((\gamma_s)(D))$ =	$(2.18)(115\text{-}62.4)(D)$	= 114.67 D psf

Compute the forces:

F_1 =	$(p_1)(H)$ =	$(0.31)(250)(10)$	= 775.0 lbs/ft.
F_2 =	$(½)(p_2)(H)$ =	$(½)(356.5)(10)$	= 1782.5 lbs/ft.
F_3 =	$(p_1+p_2)(D)$ =	$(77.5+356.5)(D)$	= 434.0 D lbs/ft.
F_4 =	$(½)(p_3)(D)$ =	$(½)(16.31\ D)(D)$	= 8.16 D^2 lbs/ft.
F_5 =	$(½)(p_4)(D)$ =	$(½)(114.67\ D)(D)$	= 57.34 D^2 lbs/ft.

Step 4: Determine depth of embedment (D). (To compute: $\Sigma M_{@o} = 0$ and solve for D).

$$\Sigma M_{@o} = (1/3)(D)(F_4) + (½)(D)(F_3) + (D + 1/3\ H)(F_2) + (D + ½H)(F_1) - (1/3)(D)(F_5) = 0$$

$$-16.39\ D^3 + 217\ D^2 + 2557.5\ D + 9816.7 = 0$$

$$D = 21.7\ \text{ft.}$$

The depth of embedment is increased by 20% to account for the differences which exist between using the Simplified vs. Conventional Method of analysis.

$$D = (D)(1.2) = 26\ \text{ft.}$$

Step 5: Find the point of zero shear (y):

$$\Sigma F_H = 0 = F_1 + F_2 + F_3 + F_4 - F_5$$

$$0 = y^2 - 8.83\ y - 52.0$$

$$y = \frac{8.83 \pm \sqrt{8.83^2 - (4)(1)(-52.0)}}{(2)(1)} \qquad \text{(quadratic equation)}$$

$$y = 12.87\ \text{ft.}$$

Step 6: Find the maximum moment which occurs at the point of zero shear:

$$\Sigma M_{@o'} = M_{max} = -16.39\ y^3 + 217\ y^2 + 2557.5\ y + 9816.7$$

$$= 43.7\ \text{kip-ft.}$$

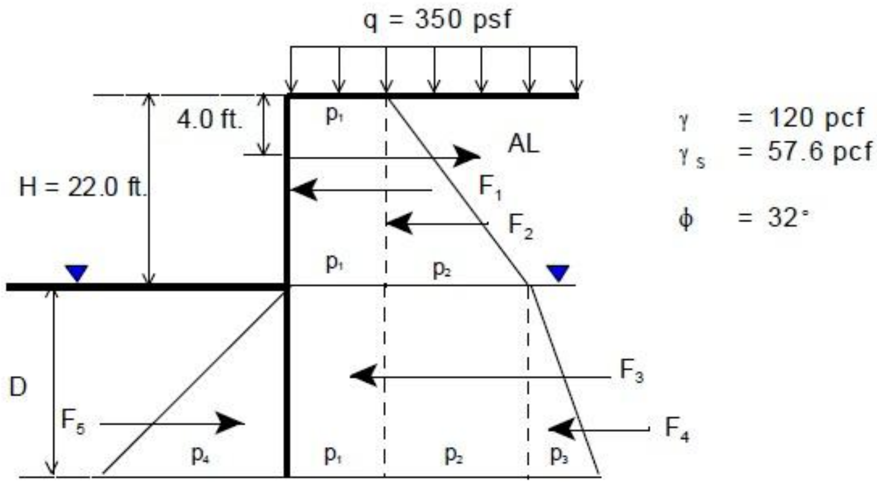
Step 7: Determine minimum section modulus:

$$S = \frac{M_{max}}{\sigma_{all}} = 21.0\ \text{in}^3 \text{ per foot of wall}$$

$(\sigma_{all} = 25\ \text{ksi})$

EXAMPLE - ANCHORED SHEETING WALL *(US CUSTOMARY UNITS)*

Step 1: Using the Free Earth Support Method, determine the depth of embedment, required section modulus and anchor design load for the following situation (temporary sheeting).



Step 2: Rankine Theory for a level backfill:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = 0.31$$

$$K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi} = 3.25 \qquad K_p' = \frac{K_p}{1.25} = 2.60$$

Step 3: Compute the pressures:

$p_1 = (K_a)(q) = (0.31)(350) = 108.5$ psf

$p_2 = (K_a)(\gamma)(H) = (0.31)(120)(22) = 818.4$ psf

$p_3 = (K_a)(\gamma_s)(D) = (0.31)(57.6)(D) = 17.86\ D$ psf

$p_4 = (K_p')(\gamma_s)(D) = (2.60)(57.6)(D) = 149.76\ D$ psf

Since the water level is at the same elevation on both sides of the wall, the net water pressure is zero.

Step 3: **(Cont.)**

Compute the forces:

								Moment Arm
F_1	=	$(p_1)(H)$	=	(108.5)(22)	=	2387.0 lbs/ft.		½ (22) – 4.0 = 7.0
F_2	=	$(½)(p_2)(H)$	=	(½)(818.4)(22)	=	9002.4 lbs/ft.		2/3 (22) – 4.0 = 10.67
F_3	=	$(p_1 + p_2)(D)$	=	(108.5+818.4)(D)	=	926.9 D lbs/ft.		(22 – 4.0) + D/2 = 18.0 + D/2
F_4	=	$(½)(p_3)(D)$	=	(½)(17.86 D)(D)	=	$8.93\ D^2$ lbs/ft.		(22 – 4.0) + 2/3 D = 18.0 + 2/3 D
F_5	=	$(½)(p_4)(D)$	=	(½)(149.76 D)(D)	=	$74.88\ D^2$ lbs/ft.		(22 – 4.0) + 2/3 D = 18.0 + 2/3 D

Step 4: Sum moments about the anchor to determine depth of embedment:

$$\Sigma M_{AL} = 0$$

$$=(7.0)(F_1) + (10.67)(F_2) + (18.0 + D/2)(F_3) + (18.0 + 2/3\ D)(F_4) - (18.0 + 2/3\ D)(F_5)$$

$$=16709.0 + 96055.61 + 16684.2\ D + 463.45\ D^2 + 160.74\ D^2 + 5.95\ D^3 – 1347.84\ D^2 – 49.92\ D^3$$

$$=112764.61 + 16684.2\ D – 723.65\ D^2 – 43.97\ D^3$$

$D \approx 16.35$ ft.

The depth of embedment is increased by 20 % to minimize lateral deflection of the sheeting at its base.

$D = 1.2(D) = 1.2(16.35) = 19.62$ ft. ≈ 20 ft.

Step 5: Determine anchor load (sum the horizontal forces):

$\Sigma F_H = 0$

$F_1 + F_2 + F_3 + F_4 - F_5 - AL = 0$

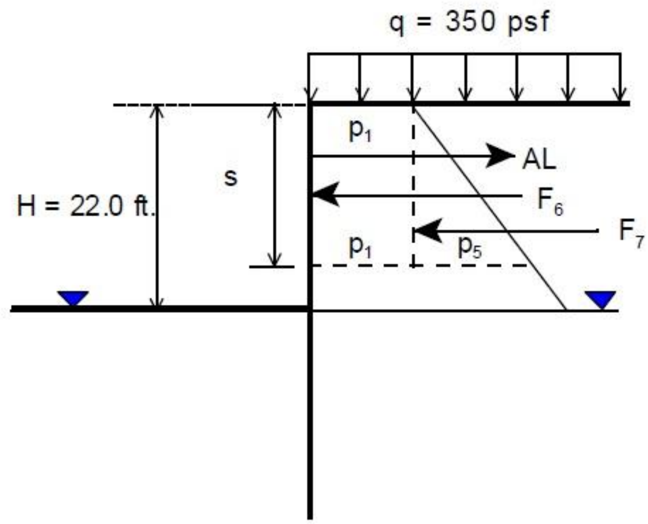
2387.0 + 9002.4 + 15154.82 + 2387.19 – 20017.11 - AL = 0

AL = 8914.3 lbs per foot of wall

A safety factor of 1.5 is applied to the anchor load to determine the anchor design load.

1.5(AL) = 13371.5 lbs per foot of wall

Step 6: Determine depth of zero shear (assume that the point of zero shear will occur a distance S from the top):



$p_5 = (K_a)(\gamma)(s) = (0.31)(120)(s) = 37.2\ s$

$F_6 = (p_1)(s) = 108.5\ s$

$F_7 = (½)(p_5)(s) = (½)(37.2\ s)(s) = 18.6\ s^2$

$\Sigma F_H = 0 \quad F_6 + F_7 - AL = 0$

$18.6\ s^2 + 108.5\ s – 8914.3 = 0$

Using the quadratic equation:

$$s = \frac{-108.5 \pm \sqrt{(108.5)^2 - (4)(18.6)(-8914.3)}}{(2)(18.6)} \qquad \text{(quadratic equation)}$$

s = 19.17 ft.

Step 7: Determine maximum moment (sum moments about the point of zero shear):

$M_{max} = AL(s – 4.0) - (s/2)(F_6) - (s/3)(F_7)$

$= (8914.3)(15.17) - (9.59)(2079.95) - (6.39)(6835.29)$

$= 135229.93 – 19946.72 – 43677.5$

$= 71.6$ kip-ft.

Step 8: Determine minimum section modulus:

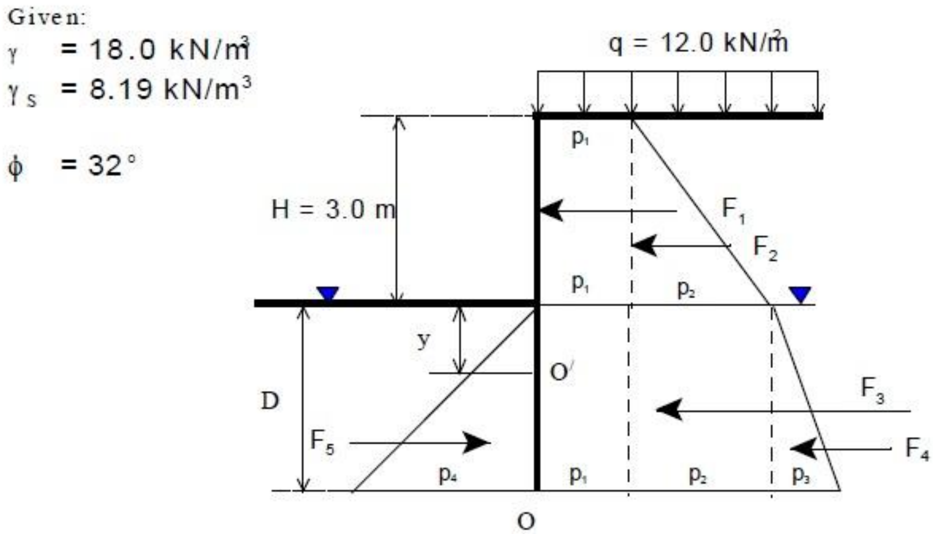
$$S = \frac{M_{max}}{\sigma_{all}} = 34.4\ in^3 \text{ per foot of wall}$$

($\sigma_{all} = 25$ ksi)

EXAMPLE-

CANTILEVERED SHEETING WALL *(INTERNATIONAL SYSTEM OF UNITS)*

Step 1: Using the Simplified Method, determine the depth of embedment and required section modulus for the following situation (permanent sheeting).



Step 2: Rankine Theory for a level backfill:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = 0.31$$

$$K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi} = 3.25 \qquad K_p' = \frac{K_p}{1.5} = 2.18$$

Step 3: Compute the pressures:

$p_1 = (K_a)(q) = (0.31)(12.0) = 3.72\ \text{kN/m}^2$

$p_2 = (K_a)(\gamma)(H) = (0.31)(18.0)(3.0) = 16.72\ \text{kN/m}^2$

$p_3 = (K_a)(\gamma_s)(D) = (0.31)(18.0\text{-}9.81)(D) = 2.54\ D\ \text{kN/m}^2$

$p_4 = (K_p')((\gamma_s)(D)) = (2.18)(18.0\text{-}9.81)(D) = 17.88\ D\ \text{kN/m}^2$

Compute the forces:

$F_1 = (p_1)(H) = (0.31)(12.0)(3.0) = 11.16\ \text{kN/m}$

$F_2 = (½)(p_2)(H) = (½)(16.72)(3.0) = 25.08\ \text{kN/m}$

$F_3 = (p_1+p_2)(D) = (3.72+16.72)(D) = 20.44\ D\ \text{kN/m}$

$F_4 = (½)(p_3)(D) = (½)(2.54\ D)(D) = 1.27\ D^2\ \text{kN/m}$

$F_5 = (½)(p_4)(D) = (½)(17.88\ D)(D) = 8.94\ D^2\ \text{kN/m}$

Step 4: Determine depth of embedment (D). (To compute: $\Sigma M_{@o} = 0$ and solve for D).

$$\Sigma M_{@o} = (1/3)(D)(F_4) + (½)(D)(F_3) + (D + 1/3\ H)(F_2) + (D + ½H)(F_1) - (1/3)(D)(F_5) = 0$$

$$-2.56\ D^3 + 10.22\ D^2 + 36.24\ D + 41.82 = 0$$

$$D = 6.5\ m$$

The depth of embedment is increased by 20% to account for the differences which exist between using the Simplified vs. Conventional Method of analysis.

$$D = (D)(1.2) = 7.8\ m$$

Step 5: Find the point of zero shear (y):

$$\Sigma F_H = 0 = F_1 + F_2 + F_3 + F_4 - F_5$$

$$0 = y^2 - 2.66\ y - 4.72$$

$$0 = \frac{2.66 \pm \pm\sqrt{2.66^2 - (4)(1)(-4.72)}}{(2)(1)} \qquad \text{(quadratic equation)}$$

$$y = 3.88\ m$$

Step 6: Find the maximum moment which occurs at the point of zero shear:

$$\Sigma M_{@o'} = M_{max} = -2.56\ y^3 + 10.22\ y^2 + 36.24\ y + 41.82$$

$$= 186.8\ kN\text{-}m$$

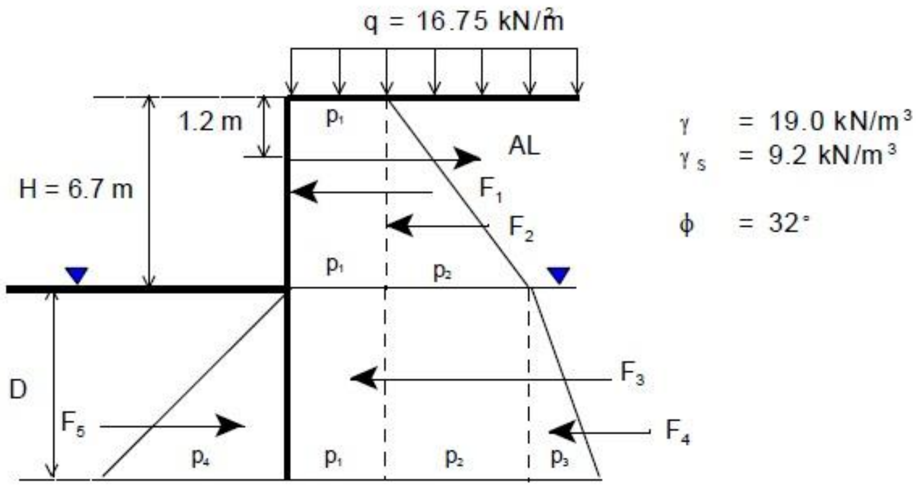
Step 7: Determine minimum section modulus:

$$S = \frac{M_{max}}{\sigma_{all}} = 1083 \times 10^3\ mm^3 \text{ per meter of wall}$$

$(\sigma_{all} = 172.5 \times 10^3\ kN/m^2)$

EXAMPLE - ANCHORED SHEETING WALL *(INTERNATIONAL SYSTEM OF UNITS)*

Step 1: Using the Free Earth Support Method, determine the depth of embedment, required section modulus and anchor design load for the following situation (temporary sheeting).



Step 2: Rankine Theory for a level backfill:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = 0.31$$

$$K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi} = 3.25 \qquad K_p' = \frac{K_p}{1.25} = 2.60$$

Step 3: Compute the pressures:

$p_1 = (K_a)(q) = (0.31)(16.75) = 5.19\ \text{kN/m}^2$

$p_2 = (K_a)(\gamma)(H) = (0.31)(19.0)(6.7) = 39.46\ \text{kN/m}^2$

$p_3 = (K_a)(\gamma_s)(D) = (0.31)(9.2)(D) = 2.85\ D\ \text{kN/m}^2$

$p_4 = (K_p')(\gamma_s)(D) = (2.60)(9.2)(D) = 23.92\ D\ \text{kN/m}^2$

Since the water level is at the same elevation on both sides of the wall, the net water pressure is zero.

Step 3: **(Cont.)**

Compute the forces:

				Moment Arm
F_1 =	$(p_1)(H)$	= (5.19)(6.7)	= 34.77 kN/m	½ (6.7) - 1.2 = 2.15
F_2 =	$(½)(p_2)(H)$	= (½)(39.46)(6.7)	= 132.19 kN/m	2/3 (6.7) - 1.2 = 3.27
F_3 =	$(p_1 + p_2)(D)$	= (5.19 + 39.46)(D)	= 44.65 D kN/m	(6.7 - 1.2) + D/2 = 5.50 + D/2
F_4 =	$(½)(p_3)(D)$	= (½)(2.85 D)(D)	= 1.43 D^2 kN/m	(6.7 - 1.2) + 2/3 D = 5.50 + 2/3 D
F_5 =	$(½)(p_4)(D)$	= (½)(23.92 D)(D)	= 11.96 D^2 kN/m	(6.7 - 1.2) + 2/3 D = 5.50 + 2/3 D

Step 4: Sum moments about the anchor to determine depth of embedment:

$\Sigma M_{AL} = 0$

$=(2.15)(F_1) + (3.27)(F_2) + (5.50 + D/2)(F_3) + (5.50 + 2/3\ D)(F_4) - (5.50 + 2/3\ D)(F_5)$

$=74.76 + 432.26 + 245.58\ D + 22.33\ D^2 + 7.87\ D^2 + 0.95\ D^3 - 65.78\ D^2 - 7.97\ D^3$

$=507.02 + 245.58\ D - 35.58\ D^2 - 7.02\ D^3$

$D \approx 4.95$ m

The depth of embedment is increased by 20 % to minimize lateral deflection of the sheeting at its base.

D = 1.2(D) = 1.2(4.95) = 5.9 m

Step 5: Determine anchor load (sum the horizontal forces):

$\Sigma F_H = 0$

$F_1 + F_2 + F_3 + F_4 - F_5 - AL = 0$

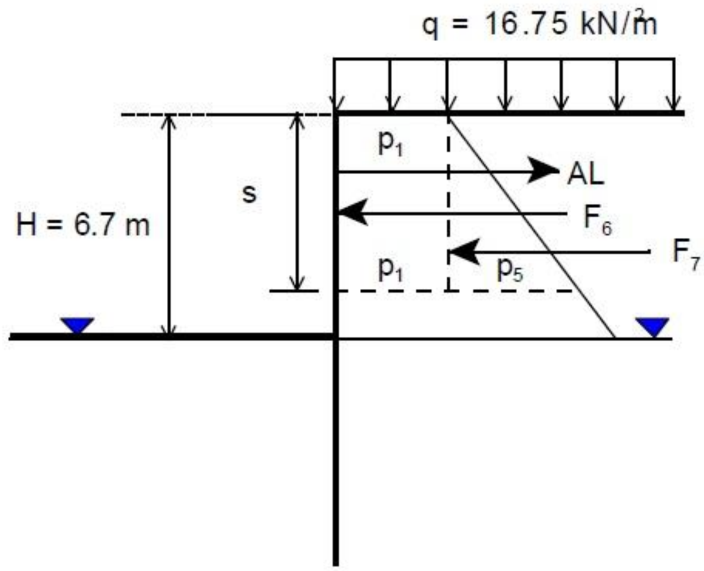
34.77 + 132.19 + 221.02 + 35.04 - 293.05 - AL = 0

AL = 129.97 kN per meter of wall

A safety factor of 1.5 is applied to the anchor load to determine the anchor design load.

1.5(AL) = 194.96 kN per meter of wall

Step 6: Determine depth of zero shear (assume that the point of zero shear will occur a distance S from the top):



p_5 = $(K_a)(\gamma)(s)$ = (0.31)(19.0)(s) = 5.89 s
F_6 = $(p_1)(s)$ = 5.19 s
F_7 = $(½)(p_5)(s)$ = (½)(5.89 s)(s) = 2.95 s^2

$\Sigma F_H = 0$ $F_6 + F_7 - AL = 0$
$2.95 s^2 + 5.19 s - 129.97 = 0$

Using the quadratic equation:

$$s = \frac{-b \pm \sqrt{(b^2 - 4ac)}}{2a}$$

s = 5.82 m

Step 7: Determine maximum moment (sum moments about the point of zero shear):

M_{max} = $AL(s - 1.2) - (s/2)(F_6) - (s/3)(F_7)$
= (129.97)(4.62) - (2.91)(30.21) - (1.94)(99.92)
= 600.46 - 87.91 - 193.84
= 318.71 kN - m

Step 8: Determine minimum section modulus:

$$S = \frac{M_{max}}{\sigma_{all}} = 1848 \times 10^3 \text{ mm}^3 \text{ per meter of wall}$$

(σ_{all} = 172.5 x 10^3 kN/m^2)