State of California
Department of Transportation
Division of Engineering Services

Trenching and Shoring Manual

Issued by
Structure Construction

June 2011

Revision 2
July 2025

Visit our website for updated manual sections:
Trenching and Shoring Manual

Preface

The California Department of Transportation Trenching & Shoring Manual was originally developed by Structure Construction in 1977. Its purpose then, which continues now, was to provide technical guidance for Structure Construction field engineers analyzing designs of trenching & shoring systems used in the California Highway Construction program. Beginning with the initial edition, this manual was well received by both the Department and the construction industry, and was distributed nationwide as well as to foreign countries.

This 2025 manual revision remains to be devoted to the analysis of trench and excavation earth support (shoring) systems needed for the construction of the Department's infrastructure. An additional very important objective is to inform the Engineer of California's legal requirements regarding worker protection. The Engineer should bear in mind that this manual is a book of reference and instruction to be used with respect to the administration and engineering of excavation shoring. In cases of conflict, the contract documents will prevail.

This revision retains the enhancements made in 2011 through significant contribution from Anoosh Shamsabadi PhD, PE, and those of Kenneth J Burkle, PE, both of whom have continued to support this manual and the current Caltrans Trenching and Shoring Check Program. While the 2011 edition emphasized the AASHTO, this 2025 revision of the manual reintroduces the more rigorous and classical analysis of cantilevered shoring systems.

The first two chapters remain devoted to the legal requirements and the responsibilities of the various parties involved. Excavation safety begins with a clear understanding of the responsibilities of one's role in the planned work. Not only must construction personnel be aware of the various legal requirements, but they must also thoroughly understand the risks excavations pose to worker safety.

The engineering objective of a shoring system is to be both safe and practical. The design of a shoring system requires two distinct efforts. First is the classification of the soil to be supported, determination of inherent soil strength, calculation of lateral loads, and distribution of lateral pressures. This is the soil mechanics or geotechnical engineering effort. The second is the structural design or analysis of members comprising the shoring system. The first part, the practical application of soil mechanics, is the more difficult. The behavior and interaction of soils with earth support systems is a complex and often controversial subject. Books, papers, and "experts" do not always concur even on basic theory or assumptions. Consequently, there are no absolute answers or exact numerical solutions. A flexible, yet conservative approach is justified. This manual presents methodologies that will be adequate for most situations. The Engineer, on their part, must recognize situations that affect the use of the procedures discussed in the manual and utilize sound engineering judgment as to which methods are appropriate.

There are many texts and publications of value, however, be cautious with dated and historical material that may not be current with soil engineering theories. There are other satisfactory methods of approaching an engineering problem. The subject of Geotechnical Engineering, which is used in shoring design, is recognized as an engineering art. The need for good judgment cannot be over emphasized. Do not lose sight of the primary objective: a safe and practical means of doing the work.

There are two major reasons why the Department considers shoring and earth retaining systems a subject apart from other temporary works such as falsework. First, an accident in a trench or excavation is more likely to have a greater potential for the maximum penalty, that is, the death of a workman. Cave-ins or shoring failures can happen suddenly, with little or no warning and with little opportunity for workers to take evasive action. Second, earth support systems design involves the complex interaction of soil types plus engineering factors that are often debatable and highly empirical.

Trenching or shoring is generally considered temporary work. However, a shoring system's "temporary" status may extend 90 days or longer for complicated structures; this time can be extended even longer due to delays caused by weather, material shortages, labor disputes, or other disruptions. Thus consideration of a shoring system's "temporary" status, should be evaluated and monitored against the initial assumptions made during the design and installation.

The Department's goal in maintaining this manual is to provide practical guidance for commonly used temporary systems. This manual is the result of merging the Structure Construction (SC) experience with continued research and study by engineering staff from the Division of Engineering Services (DES) and industry. This manual represents hundreds of years of combined experience.

It is impossible to acknowledge each and every individual who contributed to the development of the manual. However, recognition is due to the major contributors as follows:

Significant contributions

G. W. Thompson – SC Falsework Engineer and original contributor in 1977 (Retired)
T. E. De Rosia – SC Falsework Engineer and Editor for 1990 (Retired)
Anoosh Shamsabadi, PhD, PE, Senior Bridge Engineer – DES Office of Earthquake Engineering for the revisions of 2011 (Retired)
Kenneth J Burkle, PE, SC Senior Bridge Engineer (Retired)

Winter Training 2022 Instruction Team

Ken Haas, PE, Senior Bridge Engineer
David Toci, PE, Senior Bridge Engineer
Jim Nicholls, PE, Senior Bridge Engineer (Retired)
Alejandro Lara, PE, Senior Bridge Engineer
Andy Chen, PE, Senior Bridge Engineer

Structure Construction Earth Retaining Systems Technical Team (ERSTT)

Jeff Abercrombie, PE, Supervising Bridge Engineer– ERSTT Sponsor (Retired)
Andy Gill, PE, Supervising Bridge Engineer– ERSTT Sponsor
Anthony R. English, PE, Supervising Bridge Engineer (Former Chair)

Senior Bridge Engineers

Michael Trinh, PE
Clifford Law, PE
Alejandro Lara, PE
Parvin Sebti, PE (Retired)

SC Structure Representatives

Sokheng Thun
Ann Meyer, PE (Separated)

Structure Construction Headquarters Staff

Cheryl Poulin, PE, Senior Bridge Engineer (Retired)
James Nicholls, PE, Senior Bridge Engineer – SC Falsework Engineer (Retired)
Roy Auer, PE, Senior Bridge Engineer – Technical Manual Manager
Barbora Rejmanek, ADA Specialist/ Editor
Tor Amble, PE (Retired)

Cartoons by George W. Thompson, PE, included in memoriam.

Signed,

JOHN LAMMERS
Deputy Division Chief
Structure Construction
Division of Engineering Services

Nomenclature

c	=	Cohesive intercept: Component of soil shear strength which is independent of the force pushing the particles together
C	=	Calculated cohesive force (lbs)
c_a	=	Adhesive value between a cohesive soil and another surface (wall)
E	=	Modulus of elasticity (psi)
GW	=	Groundwater surface
I	=	Moment of inertia (in^4)
K_a	=	Lateral earth pressure coefficient for active pressure condition
K_o	=	Lateral earth pressure coefficient for at-rest condition
K_p	=	Lateral earth pressure coefficient for passive pressure condition
K_w	=	Equivalent fluid soil pressure (also known as EFP) (pcf)
K_{ph}	=	Horizontal component of lateral earth pressure coefficient for passive pressure condition
K_{pv}	=	Vertical component of lateral earth pressure coefficient for passive pressure condition
N	=	Standard penetration resistance
N_c	=	Bearing capacity factor
N_0	=	Stability number
Q	=	Level surcharge loading (pcf)
q_u	=	Unconfined compressive strength (psf)
S	=	Section modulus (in^3)
S_b	=	Bond strength (psf); frictional force between soil and ground anchor
SF	=	Safety factor
S_u	=	Undrained shear strength
α - Alpha	=	Angle from vertical to center of surcharge strip
β - Beta	=	Angle of soil slope
γ - Gamma	=	Unit weight of soil (pcf)
δ - Delta	=	Wall friction angle
ε - Epsilon	=	Linear strain
θ - Theta	=	Angle of repose
μ - Mu	=	Angle of tieback with horizontal
ρ - Rho	=	Degree of flexibility of an anchored bulkhead (Rowe's Moment Reduction theory)
σ - Sigma	=	Normal stress
Σ - Sigma	=	Sum
τ - Tau	=	Soil shear stress
υ - Upsilon	=	Poisson's ratio
ϕ - Phi	=	Angle of internal friction of soil
ψ - Psi	=	Failure wedge or slip angle
ω - Omega	=	Angle of the wall with respect to vertical

AISC = American Institute of Steel Construction
AISC Manual = AISC Steel Construction Manual
AREA = American Railway Engineering Association
AREMA = American Railway Engineering and Maintenance-of-Way Association
ASTM = American Society for Testing and Materials
Cal/OSHA = California Department of Industrial Relations, Division of Occupational Safety and Health
CT = California Department of Transportation (Caltrans)
DES = Caltrans Division of Engineering Services
FHWA = Federal Highway Administration
FS = Factor of safety
HQ = Headquarters in Sacramento, California
METS = Materials Engineering and Testing Services (within DES)
SC = Structure Construction (a subdivision of DES)

Collapse = Term used when the intended supporting system cannot resist the loads imposed or provide sufficient rigidity to prevent unacceptable distortion of the system.

Drag Coefficient (C_d) = Used for calculating pressure values of fluid flowing around the restraining system.

NDS = 2018 National Design Specification for Wood Construction by the American Wood Council.

Pad = Timber or concrete members used to distribute loads to the soil. (Falsework Manual Sections 8-2, *Timber Pads*, and 8-3, *Concrete Pads*)

Racker = A brace used to apply a restraining force to the shoring system placed at an angle from the ground to a point above on the retaining system.

Rough Sawn = Lumber which is untreated and rough cut, creating dimensions that are similar to the nominal dimension (e.g., a 2x4 is actually 2 inches wide and 4 inches deep)

S4S = Surfaced on four sides; this creates dimensions which are typically ½ inch less than nominal (e.g ., a 2x4 is actually 1.5 inches wide and 3.5 inches deep)

TABLE OF CONTENTS

Preface

Nomenclature

Chapter 1	Legal Requirements	Page
	1-1 Legal Requirements and Responsibilities	1-2
	1-2 Labor Code	1-3
	1-3 Cal/OSHA	1-4
	1-4 State Statutes	1-6
	1-5 Federal Highway Administration (FHWA)	1-6
	1-6 Railroad Relations and Requirements	1-6
	1-7 Excavation Shop drawings	1-7
	1-8 Summary	1-8

Chapter 2	Cal/OSHA Overview	Page
	2-1 Introduction	2-2
	2-2 Some Important Cal/OSHA Definitions	2-5
	2-3 Some Important Cal/OSHA Requirements	2-6
	2-4 Manufactured Products	2-14
	2-5 Information About Text Formatting in the Construction Safety Orders	2-15

Chapter 3	Soils	Page
	3-1 Introduction	3-2
	3-2 Soil Identification, Classification, Description, and Presentation	3-2
	3-3 Soil Properties and Strength	3-4
	3-4 Standard Penetration Test (SPT)	3-4
	3-5 Cone Penetration Test (CPT)	3-7
	3-6 Field and Laboratory Tests	3-9
	3-7 Shear Strength	3-10
	3-8 Contractor Soil Investigations	3-11
	3-9 Special Ground Conditions	3-11

Chapter 4 Earth Pressure Theory and Application — Page

4-1	Introduction	4-2
4-2	Shoring Types	4-2
4-3	Developing Earth Pressures for Granular Soil	4-5
4-4	Developing Earth Pressures for Cohesive Soil	4-16
4-5	Estimating Maximum Allowable Embankment Slope Angle	4-22
4-6	Log-Spiral Passive Earth Pressure	4-28

Chapter 5 Surcharges — Page

5-1	Surcharge Loads	5-2

Chapter 6 Structural Design of Shoring Systems — Page

6-1	Introduction	6-2
6-2	Allowable Working Stresses	6-2
6-3	Mechanics of Stress Analysis	6-4
6-4	Overstress	6-4
6-5	Lagging	6-5

Chapter 7 Unrestrained Shoring Systems — Page

7-1	Types of Unrestrained Shoring Systems	7-2
7-2	Effective Pile Width	7-3
7-3	System Deflection	7-5
7-4	Soil Pressure Distribution for Layered Soil	7-7
7-5	Lateral Earth Pressures for Unrestrained Shoring Systems	7-8

Chapter 8 Restrained Shoring Systems — Page

8-1	Lateral Earth Pressures for Restrained Shoring Systems	8-2
8-2	Cohesionless Soils	8-2
8-3	Cohesive Soils	8-5
8-4	Calculation Procedures	8-7

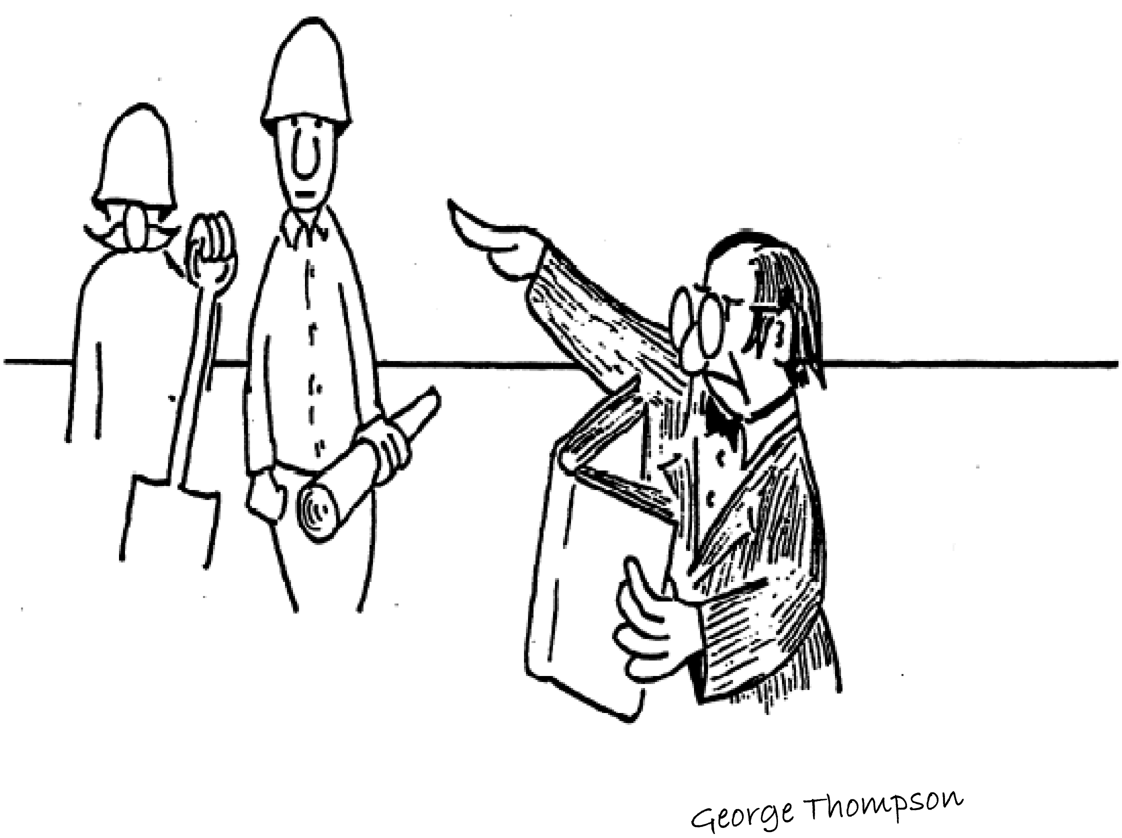
Chapter 9	**Railroads**	**Page**
	9-1 Introduction	9-2
	9-2 Summary of Typical Railroad Requirements from the Guidelines	9-4
	9-3 Track Monitoring Requirements for Railroads	9-6
	9-4 Example 9-1 (Railroad Example)	9-8
Chapter 10	**Special Conditions**	**Page**
	10-1 Special Conditions	10-2
	10-2 Use of Anchor Blocks	10-3
	10-3 Heave	10-16
	10-4 Piping	10-21
	10-5 Slope Stability	10-25
Chapter 11	**Construction Considerations and Final Summary**	**Page**
	11-1 Construction	11-2
	11-2 Encroachment Permit Projects	11-4
	11-3 Ground Anchor Restrained Shoring Systems	11-7
	11-4 Summary	11-21
Appendix A	**Additional Theory for Inquiring Minds**	**Page**
	A-1 Introduction	A-2
	A-2 Log-Spiral Passive Earth Pressure	A-2
	A-3 Culmann's Graphical Solution for Active Earth Pressure	A-18
Appendix B	**Example Problems**	**Page**
Appendix C	**Surcharges - Tabular Values**	**Page**
	C-1 Tabular Values for Strip Loads	C-2
	C-2 Additional Notes: Boussinesq versus Wayne C. Teng Equations	C-16

Appendix D Sheet Piles Page

D-1 Steel Sheet Piling Products D-2
D-2 Z Sheet Pile D-2
D-3 L.B. Foster Piling Products D-9

CHAPTER 1

LEGAL REQUIREMENTS



Chapter 1: Legal Requirements

Table of Contents

Chapter 1: Legal Requirements 1
Table of Contents 1
1-1 Legal Requirements and Responsibilities 2
1-2 Labor Code 3
1-3 Cal/OSHA 4
1-4 State Statutes 6
1-5 Federal Highway Administration (FHWA) 6
1-6 Railroad Relations and Requirements 6
1-7 Excavation Shop drawings 7
1-8 Summary 8

1-1 Legal Requirements and Responsibilities

The State of California provides for the planning and design of permanent work to be prepared by the Department. During construction, the Contractor is responsible for the design, construction, and removal of temporary works needed to build the planned infrastructure. This chapter deals with the responsibilities of the Contractor and the Department as related to trench and excavation work performed in completing the contract work.

The *Contact Specifications (CS)*, Section 7-1.02*, Legal Relations and Responsibility to the Public – Laws*, requires the Contractor to adhere to all existing and future laws. The *CS*, Section 7-1.02K, *Legal Relations and Responsibility to the Public – Laws – Labor Code*, directs the Contractor to adhere to the California Labor Code, which is discussed in more detail below. Within here, subsection (6)(b), *Occupational Safety and Health Standards – Excavation Safety*, requires the protection of workers and public in trench and excavation operations as part of the California Labor Code, Section (§) 6705, while excavating on the project. For any excavation 5 feet or more in depth, the Contractor must submit shop drawings for a protective system.

The drawings must show the design and details for providing worker protection from caving ground during excavation. The review time allowed for the shop drawings will vary depending on the design of the protective system and additional time will be allowed if the shoring must also be reviewed by a railroad representative.

The Department has the responsibility for administering the contract. This means that interpretation of contract requirements, including acceptance of materials, is done by the Department, not by any other agency such as the Department of Industrial Relations, Division of Occupational Safety and Health (also known as Cal/OSHA). Although the work must be performed in compliance with the California Code of Regulations (CCR), Title 8, Industrial Relations, there may be situations or conditions where the regulations are not adequate or applicable; under these circumstances, the Engineer makes an interpretation and informs the Contractor accordingly of what is required. See *CS,* Section 5-1.03, *Control of Work – Engineer's Authority*.

The shop drawings for protective systems for excavations must comply with the following:

1. Special provisions
2. Project plans
3. Revised standard specifications, then standard specifications
4. Revised standard plans, then standard plans
5. Change orders

6. California Code of Regulations (CCR), Title 8
7. California Streets and Highways Code
8. California Labor Code
9. All existing and future state and federal laws and county and municipal ordinances and regulations of other governmental bodies or agencies, such as railroads, having jurisdiction within the project.

Please note that shop drawings for worker protection in excavations are often referred to simply as “excavation plans.”

1-2 Labor Code

The California Labor Code is the document of enacted law to which all employers and employees must conform.

Division 5, Safety in Employment, Part 1 – Occupational Safety and Health, Sections 6300 to 6725, pertain to the subject of trenching and shoring. Section 6300 establishes the California Occupational Safety and Health Act. This authorizes the enforcement of effective standards for safety at work sites. Section 6307 gives Cal/OSHA the power, jurisdiction, and supervision over every place of employment to enforce and administer the various safety orders found within the CCR, Title 8. Section 6706 pertains to the permit requirements for trench or excavation construction.

Every employer in California is required by law (Labor Code Section) to provide a safe and healthful workplace for their employees. Title 8 of the CCR requires every California employer to have an effective Injury and Illness Prevention Program in writing that must be in accord with CCR, Title 8, General Industry Safety Orders, § 3203, *Injury and Illness Prevention Program.* This requirement is also in the CCR, Title 8, Construction Safety Orders, § 1509, *Injury and Illness Prevention Program*. Effective safety programs rely not only on inspection for compliance with the Construction Safety Orders but also include education and training activities and taking positive actions regarding conduct of the work.

Under the Department of Transportation *Contract Specifications (CS)*, the Contractor is responsible for performing the work in accordance with the contract. This responsibility includes compliance with all state and federal laws, applicable county or municipal ordinances and regulations, and the California Occupational Safety and Health Regulations. These safety regulations are contained within the larger CCR, Title 8, Industrial Relations (CCR Title 8). This manual will refer to the CCR Title 8 when referencing general safety regulations, while other references to the more specific subset of Construction Safety Orders will be noted as such. Note that the California Code of Regulations (CCR), Title 8, Chapter 4, subchapter 4 contains the Construction

Safety Orders (hereafter referenced as Cal/OSHA CSO). The hierarchy of the CCR is outlined below to help the reader understand these external requirements:

California Code of Regulations (CCR)

Title 1. General Provisions

Title 8. Industrial Relations

Division 1. Department of Industrial Relations

Chapter 3.2. California Occupational Safety and Health Regulations (Cal/OSHA)

Subchapter 2. Regulations of the Division of Occupational Safety and Health (Sections 340 - 344.90)

Chapter 4. Division of Industrial Safety

Subchapter 4. Construction Safety Orders (Sections 1500 - 1962)

Article 6. *Excavations* (Sections 1539-1547)

1-3 Cal/OSHA

Cal/OSHA enforces the safety regulations within the CCR Title 8 in every place of employment by means of inspections and investigations. Citations are issued for violations, and penalties may be assessed. In the event of an "imminent hazard," entry to the area in violation is prohibited.

Cal/OSHA operates from several district offices dispersed geographically throughout the state, and can be found at this link, Cal/OSHA offices.

Cal/OSHA CSO establish minimum safety standards whenever employment exists in connection with the construction, alteration, painting, repairing, construction maintenance, renovation, removal, or wrecking of any fixed structure or its parts. They also apply to all excavations not covered by other safety orders for a specific industry or operation. At construction projects, the Cal/OSHA CSO take precedence over any other general orders that are inconsistent with them, except for Compressed Air Safety Orders or Tunnel Safety Orders, Subchapters 3 and 20 of Chapter 4 of Division 1 of Title 8.

Cal/OSHA CSO, § 1541, *General Requirements,* states that no work in or adjacent to an excavation will be performed until conditions have been examined and found to be safe by a competent person. Refer to Chapter 2, *Cal/OSHA Overview*, Section 2-1, *Introduction*, of this manual for information on the competent person. Also, all excavation work must have daily and other periodic inspections by the competent person.

Cal/OSHA § 341, *Permit Requirements,* Subsection (d)(5)(A) (from CCR Title 8, Chapter 3.2, Subchapter 2, Article 2), requires a permit prior to the start of any excavation work 5 feet or deeper into which a person is required to descend. The employer must hold either an Annual or a Project Permit. Note: For purposes of this subsection, "descend" means to enter any part of the trench or excavation once the excavation has attained a depth of 5 feet or more.

A Cal/OSHA permit is not an approval of any worker protection plan for excavations. The Contractor submits an application to Cal/OSHA to procure an excavation permit. This application will describe the work, its location, and when it is to be performed. Cal/OSHA may request the Contractor to furnish additional details for unusual work, perhaps even a set of plans. These plans are not necessarily the detailed plans that are submitted to the Engineer for review and authorization.

The objective of a Cal/OSHA permit is to put Cal/OSHA on notice that potentially hazardous work is scheduled at a specific location. Cal/OSHA may then arrange to inspect the work.

Cal/OSHA issues permits for various conditions. A single permit can cover work of a similar nature on different contracts. It can be for a specific type of work within a Cal/OSHA regional area. In this case, the permit will have a time limit and the user is obligated to inform the appropriate Cal/OSHA office of the schedule for work covered by the permit. A copy of the permit is to be posted at the work site. It is the responsibility of the Structure Representative to verify that the Contractor has secured a proper permit before allowing any trenching or excavation work to begin.

Cal/OSHA CSO, § 1540, *Excavations*, defines a Trench (Trench excavation) as:

> “A narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is not greater than 15 feet. If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 feet or less, (measured at the bottom of the excavation), the excavation is also considered to be a trench.”

Excavations, which are more than 15 feet wide at the bottom, or shafts, tunnels, and mines, are excavations by Cal/OSHA definition. Thus, an excavation permit and excavation plan are still required for these “non-trench” conditions. Box culvert and bridge foundations are examples of excavations. Bridge abutments and retaining wall will often present a trench condition at the time that vertical rebar or back wall form panels are erected. The solution is to either provide a shoring system to retain the earth, or lay the slope back at an acceptable angle.

1-4 State Statutes

The Professional Engineers Act referenced in California Streets and Highways Code, § 137.6 of Article 3 in Chapter 1 of Division 1 of the Statutes, requires that the review and authorization of Contractor's plans for temporary structures in connection with the construction of State highways must be done by a registered professional engineer. Note that the Professional Engineers Act is found in Chapter 7 (commencing with § 6700), Division 3, of the Business and Professions Code. The Engineer has the responsibility to see that appropriate shop drawings are submitted and properly reviewed for work to be performed within the State right-of-way.

1-5 Federal Highway Administration (FHWA)

The *Contract Specifications,* Section 7, *Legal Relations and Responsibility to the Public,* contains the federal requirements for the project. These include provisions for safety and accident prevention. The Contractor is required to comply with all applicable federal, state, and local laws governing safety, health, and sanitation. Conformance with current Cal/OSHA standards will satisfy federal requirements, including Federal OSHA.

1-6 Railroad Relations and Requirements

If the project is on or adjacent to railroad property, the contract will contain a railroad agreement with the Department as referenced in *CS,* Section 5-1.20C, *Control of Work – Coordination with Other Entities – Railroad Relations*. This agreement is located in the *Information Handout* for the project and requires the Contractor to cooperate with the railroad where work is over, under, or adjacent to tracks, or within railroad property, and that all rules and regulations of the affected railroad must be complied with. The agreement also requires that the Contractor and subcontractors have authorized railroad insurance. The *Contract Specifications,* Section 5-1.36B, *Control of Work – Property and Facility Preservation – Railroad Property*, requires submission of shop drawings for an excavation on or affecting railroad property to the railroad for review and approval.

The Department of Transportation has established an administrative procedure for handling protective system shop drawings for excavations on or affecting railroad property. This procedure is detailed in Bridge Construction Memo (BCM) C-11, *Shop Drawing Review of Temporary Structures*.

The *Contract Specifications*, Section 5-1.36, *Control of Work – Property and Facility Preservation*, includes a provision detailing a timeline for the submission of shop drawings of the protective systems for excavations on or affecting railroad property. Note that the railroad communicates directly with the Structure Construction (SC)

Falsework Engineer, not with the Engineer on the job site. Adequate time should be allowed for the review procedure. *Contract Specifications,* Section 5-1.36B, *Railroad Property*, and Section 7-1.02K(6)(b), *Excavation Safety*, allows 65 days for the review of shop drawings for excavations on or affecting railroad properties. Alert the Contractor of the procedure and review duration at the preconstruction conference.

The Structure Representative on the project will handle the review and authorization of excavation shop drawings that involve railroads. When there is no SC involvement or items on a project requiring a review of excavation shop drawings, the District should request technical assistance from SC by contacting the Area Construction Manager, or SC Headquarters[1] in Sacramento.

1-7 Excavation Shop drawings

The Contractor must submit worker protection (excavation) shop drawings for any excavation 5 feet or deeper, as noted previously in *CS*, Section 7-1.02K(6)(b), *Excavation Safety*, to the Engineer for review and authorization. Such plans are to be submitted in a timely manner before the Contractor begins excavating. The Engineer must authorize the excavation shop drawings before work begins.

A copy of all excavation shop drawings authorized by the Engineer should be sent by email to the SC Falsework Engineer the same day authorization is sent to the Contractor. Follow the procedure detailed in BCM C-11, *Shop Drawing Review of Temporary Structures*. Briefly, the procedure is to retain one copy of the following documents in the job file and send one copy to SC Headquarters for retention in VISION and for emergency response:

1. Authorized shop drawings
2. Temporary structure analysis report
3. Engineering analysis calculations
4. Contractor's calculations
5. Manufacturer's catalog data for manufactured assemblies.

If the Contractor elects to use the standard options in the Cal/OSHA CSO, it is not required that a professional engineer prepare the plan. However, a worker protection (excavation) shop drawing is still required. This plan can be a letter to the Engineer containing the information outlined in Section 2-1.08, *Introduction - Worker Protection System Shop Drawing Submittal*, in Chapter 2, *Cal/OSHA Overview,* of this manual.

The details in the Cal/OSHA CSO consist of sloping, benching, and tables of minimum member sizes for timber and aluminum hydraulic shoring with member spacings related

[1] Caltrans internal use only

to the three general types of soil, along with various restrictions on use of materials and construction methods.

The Engineer is cautioned that conditions may be such that the Cal/OSHA CSO will not apply. For example, when a surcharge load exceeds the two feet of spoils (lateral pressure of approximately 72 psf) that the Cal/OSHA CSO reference, an engineered system is required. The proposed plan must provide a system at least as effective as the Cal/OSHA CSO, and the plan must be prepared and signed by a California registered professional engineer. The Contractor's engineered plan should include the following items in addition to the information listed in the Cal/OSHA CSO:

1. A detailed engineering drawing showing sizes, spacing, connections, etc. of materials.
2. Appropriate additional soils data.
3. A geotechnical engineer or a civil engineer specializing in soils must prepare soils reports and supplemental data.
4. Supporting data, such as design calculations or material tests.

The Contractor's engineer must provide a structural review of any plan that deviates from the Cal/OSHA CSO.

1-8 Summary

This manual presents the technical engineering information that can be used by the Engineer in making a review of excavation shop drawings.

The design or engineering analysis of a shoring system is accomplished in the following sequence:

1. The soil or earth that is to be retained and its engineering properties are determined.
2. Soil properties are then used in geotechnical mechanics or procedures to determine the horizontal earth pressure acting on the shoring system.
3. The design lateral force is then distributed in the form of a pressure diagram. The distribution or shape of the diagram is a function of the type of shoring system and the soil interaction with that system.
4. Lateral loads due to surcharges and from sources other than basic soil pressure (e.g., groundwater) are determined and combined with the basic soil pressure diagram. The resulting combined lateral pressures become the design lateral pressure diagram.

5. The design lateral pressure diagram is applied to the system, and a structural analysis is made. Again, there is a range from simplified to complex procedures that can be used.

Remember to use a proper balance of engineering effort. If the soil data is not detailed or is not available, it is not proper to use complex or sophisticated analyses. With good soils data, it is satisfactory to first use simplified analysis procedures which lead to a conservative check; then, if the system appears inadequate, a more detailed and refined procedure may be appropriate.

The extent of engineering analysis required is a function of the size of the project and how unusual or unique it is. A simplified analysis procedure can be used for the majority of trenching and shoring systems seen on projects. For complex systems, the Engineer may be presented with methods that are not discussed in this manual. The Engineer should be prepared to do some research. A procedure should not be rejected simply because it is not covered in this manual. This manual presents standard engineering procedures and some tools for more complex soil geometrics. Additional design information or copies of text material confirming the design theory needed to support the Contractor's calculations should be requested. Geotechnical Services of the Division of Engineering Services (DES) is available for consultation on soil properties.

It is recognized that the installation of the authorized excavation shop drawings is of equal importance. Construction activities include verifying quality of workmanship, inspection, and taking appropriate timely action with regard to changing conditions. See Chapter 11, *Construction Considerations and Final Summary,* of this manual for more information.

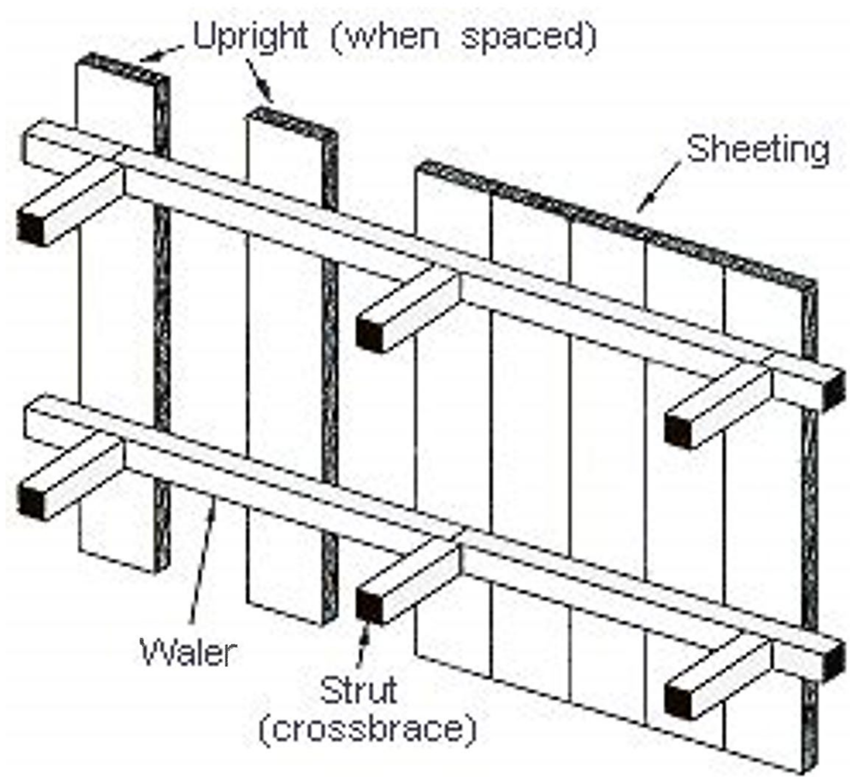
When excavation shop drawings are being reviewed, the following procedure is recommended: Perform an initial review of the shoring in conformance with the procedures in the *Falsework Manual*, Chapter 2, *Review of Shop Drawings*, Section 2-4, *Shop Drawing Review*. As with any set of shop drawings, if the submitted material is incomplete, the Contractor should be notified immediately. It will be necessary for the Contractor to submit all additional information needed to perform a review, which may include a more thorough description of design procedures, assumptions, and additional calculations. If the review indicates discrepancies in the design, it will be necessary to review the criteria and assumptions used by the designer. Note: there is no requirement that the design methodology used be in conformance with that outlined in this manual. If warranted, request from the Contractor additional information to support the alternative earth pressure theory used in their analysis. In case of a dispute, contact the SC Falsework Engineer[1] in Sacramento.

[1] Caltrans internal use only

JULY 2025

CHAPTER 2

Cal/OSHA OVERVIEW



Chapter 2: Cal/OSHA Overview

Table of Contents

Chapter 2: Cal/OSHA Overview 1
Table of Contents 1
2-1 Introduction 2
2-1.01 Excavations 20' Deep or Less without Deviations 2
2-1.02 Excavations Over 20' Deep or with Deviations 3
2-1.03 Maintain Design Plan at the Jobsite 3
2-1.04 Registered Professional Engineer 3
2-1.05 Competent Person 3
2-1.06 Surcharges 4
2-1.07 Tabulated Data 4
2-1.08 Worker Protection System Shop Drawing Submittal 4
2-2 Some Important Cal/OSHA Definitions 5
2-3 Some Important Cal/OSHA Requirements 6
2-3.01 General Requirements Section 1541 6
2-3.02 Protective System Selection 7
2-3.03 Soil Classification 9
2-3.04 Sloping or Benching Systems 10
2-3.05 Timber Shoring for Trenches 11
2-3.06 Aluminum Hydraulic Shoring for Trenches 12
2-3.07 Shield Systems 13
2-4 Manufactured Products 14
2-5 Information About Text Formatting in the Construction Safety Orders 15

2-1 Introduction

The California Division of Occupational Safety and Health, better known as Cal/OSHA, reports that more construction deaths occur during work in trenches than in any other form of construction work. This is despite a number of trench and excavation failures that go unreported. It is evident from this that continued diligence must be given to the planning, construction, monitoring, and supervisory aspects of excavations and trenching.

The information in this chapter is current as of the date of publication. It will be the responsibility of the reader to determine up-to-date applicable requirements. A summary of the most applicable Cal/OSHA references for excavations is located on the Caltrans Structure Construction (SC) intranet page under the “Safety” tab, and is titled *Cal/OSHA Standards for Excavations*[1].

Cal/OSHA adopted the Federal OSHA safety regulations pertaining to the protection of workers in excavations, effective September 25, 1991. These are embodied in the California Code of Regulations (CCR), Title 8; references to various sections and safety orders can be assumed to evolve from this file path, unless noted otherwise.

This chapter contains outlines of major portions of the adopted safety regulations that pertain to safety in conjunction with excavations. Major considerations, or requirements of the safety regulations in numerical order of the sections, are briefly outlined on the following pages. The text of most Cal/OSHA excavation requirements may be found in the *Cal/OSHA Standards for Excavations*, described above. This Cal/OSHA reference includes the following sections (§) of the Cal/OSHA Construction Safety Orders (CSO) found in CCR, Title 8, Chapter 4, subchapter 4 (hereafter referenced as Cal/OSHA CSO); § 1504, *Definitions;* § 1539, *Permits;* § 1540, *Excavations;* § 1541, *General Requirements*; § 1541.1, *Requirements for Protective Systems* (including appendices A - F); § 1542, *Shafts;* and § 1543, *Cofferdams*.

2-1.01 Excavations 20' Deep or Less without Deviations

Cal/OSHA CSO, § 1504 and § 1539 through § 1543 contain the excavation and shoring requirements. These sections provide a variety of excavation plans for worker protection in excavations. For excavations 20 feet or less in depth, the Contractor may use the sloping or benching of the soil, tables for timber or aluminum hydraulic shoring, or shields contained in these sections without a design from a professional engineer. Alternatively, the excavation plan may be designed by a registered professional engineer who is registered in the State.

[1] Caltrans internal use only

2-1.02 Excavations Over 20' Deep or with Deviations

A California registered professional engineer is required to design a protective system for excavations greater than 20 feet in depth, and when deviating from the Cal/OSHA excavation plans. For example:

1. Deviations from the sloping criteria.
2. Deviations not covered in the Cal/OSHA CSO from the timber or aluminum hydraulic shoring tables.
3. Shields to be used in a manner not recommended or approved by the manufacturer.
4. Surcharges that must be accounted for.
5. Alternate designs used.

The Contractor’s engineer may base the design on manufacturer's information, on a variety of tables and charts, on the use of proprietary systems, on soils information furnished by a competent person, and in accordance with accepted professional engineering practice.

2-1.03 Maintain Design Plan at the Jobsite

Cal/OSHA CSO, § 1541.1, *Requirements for Protective Systems,* requires that at least one authorized copy of the excavation plan be maintained at the jobsite during the construction of the protective system. The excavation plan includes tabulated data, manufacturer's data, or the engineer's design. For excavation utilizing shield systems, identify the CA registered professional engineer approving the use of tabulated or manufacturer's data for the specific excavation.

2-1.04 Registered Professional Engineer

For work in California, the design engineer must be a registered professional civil engineer in California pursuant to California Streets and Highways Code § 137.6.

2-1.05 Competent Person

Cal/OSHA CSO, § 1504, *Definitions,* defines a competent person as follows (<u>emphasis added</u>): "One who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous, or dangerous to employees, <u>and who has authorization</u> to take prompt corrective measures to eliminate them."

2-1.06 Surcharges

The figures and tables in the Appendices of Cal/OSHA CSO*,* § 1541.1, *Requirements for Protective Systems,* provide for a minimum surcharge equivalent to an additional soil height of 2 feet. The minimum surcharge may be considered to represent a 2-foot high soil embankment, small equipment, material storage, or other small loads adjacent to the excavation. No provision is made for nearby traffic, adjacent structure loadings, or for dynamic loadings (see Cal/OSHA CSO*,* § 1541.1, Appendix C, *Timber Shoring for Trenches*).

2-1.07 Tabulated Data

Tabulated data is defined in Cal/OSHA CSO § 1540, *Excavations*, as: "Tables and charts approved by a registered professional engineer and used to design and construct a protective system." Realize that tabulated tables or charts used for shoring boxes or engineered walls by design height, "H" are similar to retaining walls in the *Standard Plans*, for example Standard Plan Sheet B3-1A, *Retaining Wall Type 1*. One need not be a CA registered professional engineer for generating these manufactured systems, but a CA registered professional engineer must be used in selecting one for the site-specific excavation.

2-1.08 Worker Protection System Shop Drawing Submittal

The Contractor may submit worker protection system shop drawings, commonly called an excavation plan, using Cal/OSHA CSO standard details for sloping excavations or tabular data in the form of a letter stating which portions of the standard details are to apply to the plan. The letter should list:

1. Location of the work
2. Limits of the work
3. The times the work is to start and be in progress, and the sequence of the work
4. The applicable Cal/OSHA CSO standard details of figures and/or tables
5. Any other information pertaining to the progress or complexity of the work
6. Who will be in charge of the work
7. Who will be the designated, competent person responsible for safety.

If the Contractor elects to use the excavation plan details in the Cal/OSHA CSO, it is not necessary to have the excavation plan prepared by a registered engineer and the reviewing engineer does not have to perform a structural analysis. However, the reviewing engineer must ensure that the Contractor does the work in accordance with the Cal/OSHA CSO and the site conditions are such that the excavation plan is appropriate for the soil conditions encountered.

2-2 Some Important Cal/OSHA Definitions

Describing or citing primary sections can condense a lot of information about the requirements in the Cal/OSHA CSO. A few important definitions are included here, but the reader is directed to the applicable Cal/OSHA CSO for excavations included in the *Cal/OSHA Standards for Excavations*[1] reference, located on the SC intranet page.

From Cal/OSHA CSO, § 1504, *Definitions*:

2-2.01 Geotechnical Specialist (GTS): A person registered by the State as a Certified Engineering Geologist, or a Registered Civil Engineer trained in soil mechanics, or an engineering geologist or civil engineer with a minimum of 3 years applicable experience working under the direct supervision of either a Certified Engineering Geologist or Registered Civil Engineer.

From Cal/OSHA CSO, § 1540, *Excavations*:

2-2.02 Accepted Engineering Practices: Those requirements which are compatible with standards of practice required by a registered professional engineer.

2-2.03 Excavation: Any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal. *Note that excavations are defined to include trenches.*

2-2.04 Protective System: A method of protecting employees from cave-ins, from material that could fall or roll from an excavation face or into an excavation, or from collapse of adjacent structures. Protective systems include support systems, sloping and benching systems, shield systems, and other systems that provide the necessary protection.

2-2.05 Registered Professional Engineer: A person who is registered as a professional engineer in the state where the work is to be performed. However, a professional engineer, registered in any state is deemed to be a "registered professional engineer" within the meaning of this standard when approving designs for "manufactured protective systems" or "tabulated data" to be used in interstate commerce.

2-2.06 Shield (Shield System): A structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields can be permanent structures or can be designed to be portable and moved along as work progresses. Additionally, shields can be either premanufactured or job-built in accordance with Cal/OSHA CSO, § 1541.1(c)(3) or (c)(4). Shields used in trenches are usually referred to as "trench boxes" or "trench shields."

[1] Caltrans internal use only

<u>2-2.07 Shoring (Shoring System):</u> A structure, such as a metal hydraulic, mechanical or timber shoring system that supports the sides of an excavation, and which is designed to prevent cave-ins.

<u>2-2.08 Sloping (Sloping System):</u> A method of protecting employees from cave-ins by excavating to form sides of an excavation that are inclined away [from] the excavation so as to prevent cave-ins. The angle of incline required to prevent a cave-in varies with differences in such factors as the soil type, environmental conditions of exposure, and application of surcharge loads.

<u>2-2.09 Trench (Trench excavation):</u> A narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is not greater than 15 feet. If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 feet or less (measured at the bottom of the excavation), the excavation is also considered to be a trench.

2-3 Some Important Cal/OSHA Requirements

A few of the important considerations from the Cal/OSHA CSO portion of the CCR, Title 8 are listed here for quick reference. These portions of the Title 8 are summaries rather than direct quotes. The complete text of Cal/OSHA CSO, § 1541, *General Requirements*, referred to below is included in the *Cal/OSHA Standards for Excavations* reference.

2-3.01 General Requirements Section 1541

Underground utilities must be located prior to excavating. The Contractor should notify Underground Service Alert or other appropriate Regional Notification Centers a minimum of two working days prior to start of work. Excavations in the vicinity of underground utilities must be undertaken in a careful manner while supporting and protecting the utilities.

Egress provisions, which may include ladders, ramps, stairways, or other means, must be provided for excavations over 4 feet or more in depth. The maximum distance a worker must travel to exit the trench must not be more than 25 feet laterally.

Adequate protection from hazardous atmospheres must be provided. This includes testing and controls, in addition to the requirements set forth in the Cal/OSHA CSO and the General Industry Safety Orders, to prevent exposure to harmful levels of atmospheric contaminants and to ensure acceptable atmospheric conditions.

Employees must be protected from the hazards of accumulating water, from loose or falling debris, and from potentially unstable adjacent structures.

Daily inspections, inspections after rainstorms, and as otherwise required for hazardous conditions, are to be made by a competent person. Inspections must be conducted prior to the start of work and as needed throughout the shift. The competent person will need to check for potential cave-ins, indications of failure of the protective system, and for hazardous atmospheres. When the competent person finds a hazardous situation, it is important to remove the endangered employees from the area to ensure their safety, until the necessary precautions have been made.

Adequate physical barrier protection is to be provided at all excavations. This is extremely important at remotely located excavations where active construction operations are absent. All wells, pits, shafts, etc., must be barricaded and/or covered during periods when there is no active construction work. Upon completion of exploration and other similar operations, temporary shafts etc., must be backfilled.

2-3.02 Protective System Selection

Cal/OSHA CSO, § 1541.1 covers almost all of the requirements that must be considered when selecting or reviewing a shoring system. The text of this section contains general information and considerations for various shoring systems. This section describes the various shoring systems which can be used with or without the services of a registered professional engineer. Cal/OSHA classifies soils into three types: Type A, Type B, and Type C. These soil types are discussed below in Section 2-3.03, *Soil Classification*, and the Cal/OSHA definitions are found in Appendix A, *Soil Classification*, of Cal/OSHA CSO, § 1541.1. Additional information about the various shoring systems may be found in Appendix B through Appendix F of Cal/OSHA CSO, § 1541.1.

The design of a protective system for workers in an excavation may be selected from one of the possible options listed below per Cal/OSHA CSO, § 1541.1:

1. Entirely in stable rock - no shoring needed.
2. Excavation less than 5 feet deep - no shoring needed when an examination of the ground by a competent person provides no indication of a potential cave-in.
3. Sloping, benching, or shoring per Cal/OSHA CSO:
    a. Slope 1-1/2:1 as for Type C soil.
    Steeper slopes may be used for short term (1 day in Type A soil and 12 feet or less excavation).
    b. Slope using Table B-l, *Maximum Allowable Slopes*, or Figure B-l, *Slope Configurations*, of Appendix B, *Sloping and Benching*.
    Slopes dependent on soil type - see Appendix A, *Soil Classification*.

   c. Per tables or charts identified by a California registered professional engineer approving the data.
   d. Designed by a California registered professional engineer.

4. Utilizing of support systems, shield systems, or other protective systems per Cal/OSHA CSO*,* § 1541.1:
   a. Designed in accordance with Appendix A, or C – F:
      i. Appendix A – *Soil Classification*
      ii. Appendix C – *Timber Shoring for Trenches*
      iii. Appendix D – *Aluminum Hydraulic Shoring for Trenches*
      iv. Appendix E – *Alternatives to Timber Shoring*
      v. Appendix F – *Selection of Protective Systems*.
5. Engineered systems:
   a. Designed using manufacturer's data (shields for example):
      i. Data includes specifications, limitations, and/or other tabulated data (tables or charts).
   b. Designed using other tabulated data (tables or charts):
      i. Identified by a California registered professional engineer approving the data. (Approving engineer implies the California professional engineer designing or submitting the excavation plan.)
   c. Designed by a registered professional engineer:
      i. Identified by a California registered professional engineer authorizing the plan. (Authorizing engineer applies to the California professional engineer other than the registered professional engineer designing or submitting the excavation plan.)

Protective system designs (including manufacturer's data) other than those selected directly from tables in Cal/OSHA CSO, § 1541.1, Appendices A - F, will need to be posted at the jobsite during construction of the protective system.

Damaged materials or equipment will need to be reevaluated for use by a competent person or by a registered professional engineer before being put back into use.

Shield systems, including individual members of support systems, must not be subjected to loads exceeding those which they are designed to withstand.

Excavation of material to a level no greater than 2 feet below the bottom of the members of a support system is allowed, but only if the system is designed to resist the forces calculated for the full depth of the excavation so no loss of soil is possible.

2-3.03 Soil Classification

Appendix A, *Soil Classification*, of Cal/OSHA CSO, § 1541.1 contains the soil classification information that may be used for the proper selection of a shoring system. This appendix describes when soil classification information may be used as well as defines soil and soil types (A, B, or C). The section also covers the basics of soil classification, who can classify soil, and how soil classification is to be done by using visual and manual tests.

A competent person or a testing lab must make soil classification determinations by at least one visual and one manual test to classify rock or soil for the proper selection, or for the design, of a shoring system. Classification of the soil is necessary to determine the effective active soil pressures that the shoring system may be subjected to. The tables for the selection of sloping, timber shoring, or aluminum hydraulic shoring, are based on one of three types of soil (A, B, or C).

The three soil types in the Cal/OSHA CSO are described below:

2-3.03A Type A

Cohesive soil with unconfined compressive strength of 1.5 tons per square foot (tsf) or greater.

Examples of this soil type are: clay, silty clay, sandy clay, clay loam, silty clay loam, sandy clay loam, and cemented soils like caliche or hardpan.

No soil is Type A if:

1. The soil is fissured.
2. Vibratory or dynamic loads will be present.
3. The soil has been previously disturbed.
4. Sloped four horizontal to one vertical (4H:1V or greater) where layers dip into the excavation.
5. Other factors preclude Type A classification.

2-3.03B Type B

1. Cohesive soil with unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf, or
2. Granular cohesionless soils including: angular gravel, silt, silty loam, sandy loam, or maybe silty clay loam and sandy clay loam, or
3. Previously disturbed soils not classified as Type C, or
4. Soil that meets the requirements of Type A, but is fissured or subject to vibration, or
5. Dry rock that is not stable, or

6. Type B soil that has sloped (4H:1V or less.) layers that dip towards the excavation.

2-3.03C Type C

1. Cohesive soil with unconfined compressive strength of 0.5 tsf or less, or
2. Granular soil including gravel, sand, and loamy sand, or
3. Submerged soil, or soil from which water is freely seeping, or
4. Submerged rock that is not stable, or
5. Material sloped towards the excavation 4H:1V or steeper in a layered system.

Tables in the Cal/OSHA *CSO* for timber shoring systems consider the effective lateral pressures (**PA**) for a depth (**H**) due to the three different soil types as follows:

Type A: PA = 25H + 72 psf (2 ft. Surcharge)

Type B: PA = 45H + 72 psf (2 ft. Surcharge)

Type C: PA = 80H + 72 psf (2 ft. Surcharge)

Manual testing of soils includes tests for plasticity, dry strength, thumb penetration, and the use of a pocket penetrometer or hand-operated vane shear tester. Samples of soil can be dried to determine relative cohesive content. A few of these tests may be used to determine compressive strength; the other tests may be used to determine relative cohesive properties of the soil. The test procedures are outlined in the complete text of Cal/OSHA CSO, § 1541.1, Appendix A. Note that expansive clays are not mentioned and may need special consideration.

2-3.04 Sloping or Benching Systems

Cal/OSHA CSO, § 1541.1, Appendix B, *Sloping and Benching,* contains specifications for sloping and benching options, including visual diagrams, for excavations 20 feet or less. A registered professional engineer may design alternate configurations. Slopes may be laid back in conformance with the figures in Cal/OSHA CSO, § 1541.1, Appendix B, providing there is no sign of distress, and surcharge loads will not be a factor. Signs of distress include: caving-in of the soil, development of fissures, subsidence, bulging or heaving at the bottom of the excavation, or spalling or raveling at the face of the excavation.

When there is any sign of distress, the slope must be laid back to at least 1/2 horizontal to 1 vertical less than the maximum allowable slope as outlined below.

When surcharge loads from stored material or equipment, operating equipment, or traffic are present, a competent person must determine the degree to which the actual slope must be reduced below the maximum allowable slope and must assure that such reduction is achieved. If site conditions are outside the scope and applications of the figures of Appendix B, an alternative shoring system needs to be designed by a California registered professional engineer.

When surcharge loads from structures are present, underpinning or bracing will be required; otherwise, the structure must be on stable rock or a California registered professional engineer must determine that the excavation work will not pose a hazard to employees.

Cal/OSHA CSO, Appendix B of § 1541.1, Table B-1, *Slope Configurations,* lists the following maximum allowable slopes (H:V) for excavations less than 20 feet deep for the various soil types as shown in Table 2-1.

Table 2-1. Maximum allowable slopes (H:V) for excavations less than 20 feet deep

Soil or Rock Type	Maximum Allowable Slopes (H:V) for Excavations Less Than 20 Feet Deep
Stable Rock	Vertical
Type A	3/4:1
Type B	1:1
Type C	1-1/2:1

Cal/OSHA provides some exceptions and variations to the allowable slopes above. See Appendix B for additional details.

2-3.05 Timber Shoring for Trenches

Cal/OSHA CSO, § 1541.1, Appendix C, *Timber Shoring for Trenches*, contains information and tables that the Contractor may utilize to shore trenches that do not exceed 20 feet in depth with rough or finished timbers in any of the three types of soil. Tables C-1.1 through C-1.3 refer to actual dimensions and not nominal dimensions of the timber, having a minimum $\mathbf{F_b}$ of 850 psi. Tables C-2.1 through C-2.3 list timber members as nominal dimensions, finished (S4S) timbers, having a minimum $\mathbf{F_b}$ of 1500 psi. There is one table for each soil type for each of the timber grading sizes.

CSO, § 1541.1, Appendix C, Item (d)(2), *Limitation of application*, provides a list of conditions when the tables will not be adequate. Thus, either another protective system is to be selected, or an alternative shoring system needs to be designed by a registered professional engineer. Some of the limitations include:

1. Material surcharge loads adjacent to the trench will exceed the load from a 2-foot surcharge. (Adjacent is defined as within the horizontal distance from the edge of the trench equal to the depth of the trench.)
2. Vertical loads on the center of crossbraces exceed 240 pounds.

3. Adjacent surcharge loads from equipment weighing over 20,000 pounds are present.
4. Only the lower portion of a trench is shored, and the remaining portion is sloped or benched unless:
   a. The sloping portion is sloped less than 3H:1V, or
   b. The shoring is selected for full depth excavation.

It is necessary to understand the notes associated with these Cal/OSHA tables which describe their use and additional constraints. The excerpts below are paraphrased from Cal/OSHA CSO, § 1541.1, Appendix C, Item (g), *Notes for all Tables*.

1. When conditions are saturated or submerged, use tight sheeting. Tight sheeting refers to tongue and groove timbers at least 3 inches thick, steel sheet piling, or similar materials able to resist imposed lateral loads including water. Close sheeting refers to placing planks side-by-side as close together as possible.
2. All spacings indicated are center-to-center.
3. Wales are to be installed with greatest dimension horizontal.
4. If the vertical distance from the center of the lowest cross brace to the bottom of the trench is to exceed 2.5 feet, uprights are to be firmly embedded [in the soil], or a mudsill is to be used. A mudsill is a waler placed at the bottom of the trench.

   Maximum distance from lower brace to bottom of trench:

   a. 36 inches for embedded sheeting.
   b. 42 inches when mudsills are used.
5. Trench jacks may be used in place of or in combination with timber struts.
6. Placement of crossbraces: when the vertical spacing of crossbraces is 4 feet, place the top crossbrace no more than 2 feet below the top of the trench. When the vertical spacing of crossbraces is 5 feet, place the top crossbrace no more than 2.5 feet below the top of the trench.

Cal/OSHA CSO, § 1541.1, Appendix C, also contains four example problems demonstrating selection of shoring from the tables.

2-3.06 Aluminum Hydraulic Shoring for Trenches

Cal/OSHA CSO, § 1541.1, Appendix D, *Aluminum Hydraulic Shoring for Trenches*, contains typical installation diagrams, tables, and information for the use of aluminum hydraulic shoring in trenches that do not exceed 20 feet in depth. Tables D-1.1 and D-1.2 are for vertical shores in Type A and B soils. Tables D-1.3 and D-1.4 are for horizontal waler systems in Type B and Type C soils. Type B soils may require sheeting, whereas Type C soils always require sheeting.

For hydraulic cylinder specifications refer to the Cal/OSHA CSO, § 1541.1, Appendix D, Item (d)(2), *Hydraulic cylinders specifications*.

When any of the following conditions exist, the tabular data will not be valid:

1. When vertical loads exceeding 100 pounds will be imposed on the center of hydraulic cylinders.
2. When adjacent surcharge loads are present from equipment weighing in excess of 20,000 pounds.
3. Only the lower portion of a trench is shored, and the remaining portion is sloped or benched unless:
   a. The sloping portion is sloped less than 3H:1V, or
   b. The shoring is selected for full depth excavation.

Footnotes for the aluminum hydraulic shoring will be found in Item (g) of Appendix D to Section 1541.1, immediately preceding the figures. As with the timber tables above, it is necessary to review and understand these notes, with excerpts paraphrased below.

1. Minimum thickness plywood of 1-1/8 inch or 3/4-inch-thick 14-ply arctic white birch (Finland form) may be used in conjunction with aluminum hydraulic shoring to prevent raveling but may not be used as structural members.
2. The tables consider two cylinder sizes with minimum safe working capacities as follows: 2-inch inside diameter with 18,000 pounds axial compressive load at maximum extension, or 3-inch inside diameter with 30,000 pounds axial compressive load at maximum extensions, and as recommended by the product manufacturer.

Cal/OSHA CSO, § 1541.1, Appendix D, also contains four example problems demonstrating selection of shoring from the tables.

2-3.07 Shield Systems

Cal/OSHA CSO, § 1541.1, Appendix E, *Alternatives to Timber Shoring,* Figure 4, *Trench Shields*, contains a few diagrams of manufactured trench shields in various configurations.

The reviewing engineer should be aware that manufacturers will normally furnish engineering data to a supplier who, in turn, will furnish the data to the Contractor. An excavation plan for specific use of the shield must be prepared. The Engineer will determine forces, including surcharges, that are to be resisted, and then make comparisons with manufacturer's data, or with the submitting engineer's computations that define the capacity of the shoring system.

2-4 Manufactured Products

Manufactured trench shoring and worker protection products include screw jacks, hydraulic shores, screw or hydraulic operated frames, work shields, and other devices used to shore a trench and/or protect workers.

The maximum loading which may be applied per Cal/OSHA CSO, § 1541.1, Item (c)(2), *Option (2) – Designs Using Manufacturer's Tabulated Data*, to a manufactured product must not exceed the capacities as given by the manufacturer. These are usually shown in a catalog or brochure published by the manufacturer, or in the form of a letter from the manufacturer pertaining to the use of their product for specific job conditions. This declaration of capacities may be shown on a shop drawing or included in a letter. To be acceptable, it must be signed by the manufacturer, not the Contractor. When professional engineering data accompanies manufactured products, that data may be used with minimum supplemental review.

Be aware that some manufacturer's catalogs do not always present enough engineering data; they may only be sales brochures. Be sure to review the conditions that apply to the data submitted. This is necessary to ascertain that "capacity ratings" and other information were established utilizing the minimum loads (such as surcharges) required by the CCR, Title 8. It may be necessary to request the Contractor to furnish additional engineering data from the manufacturer.

The maximum allowable safe working load, as recommended by the manufacturer, will be based on the use of new or undamaged used material. If the product or its components are not in good condition, it must be determined if the product can function as intended, or if the safe working loads should be reduced. It is the responsibility of the Contractor to furnish proof of load capacity.

In the case of manufactured products which cannot be found in any catalog and the manufacturer is unknown or unable to recommend a safe working load, a load test is required to establish the safe load carrying capacity of that product or device. A load test should be conducted to a predetermined value or to failure. It is recommended to test the device to failure, in which case the safe working load may be taken as 1/2 the ultimate load. This will provide a safety factor of 2 with respect to failure, which is consistent with manufacturer's ratings for concrete form accessories. If it is not possible to test to failure, the working load used for the design should not exceed 1/2 of the maximum load carried during the test.

A non-commercial product generally has less quality control during its fabrication relative to a manufactured product. As such, non-commercial material should have a safety factor of 3.

Load tests witnessed by the Engineer should be documented in the project records and a copy submitted to Structure Construction Headquarters with the authorized excavation plans.

Materials must be properly identified on the excavation plan shop drawing and verified in the field. This is very important when analyzing aluminum members as there are many different alloys.

2-5 Information About Text Formatting in the Construction Safety Orders

In the CCR, Title 8, all subtopics are usually indented the same amount only on the first line of type. The subjects and subheadings format generally conforms to the following example:

Subchapter Title

Article No. Major Heading

Section Number Heading

(a) Lower case letter used for first subtopic.

(1) Number used for subtopic to lower case letter.

(A) Upper case letter used for subtopic to number.

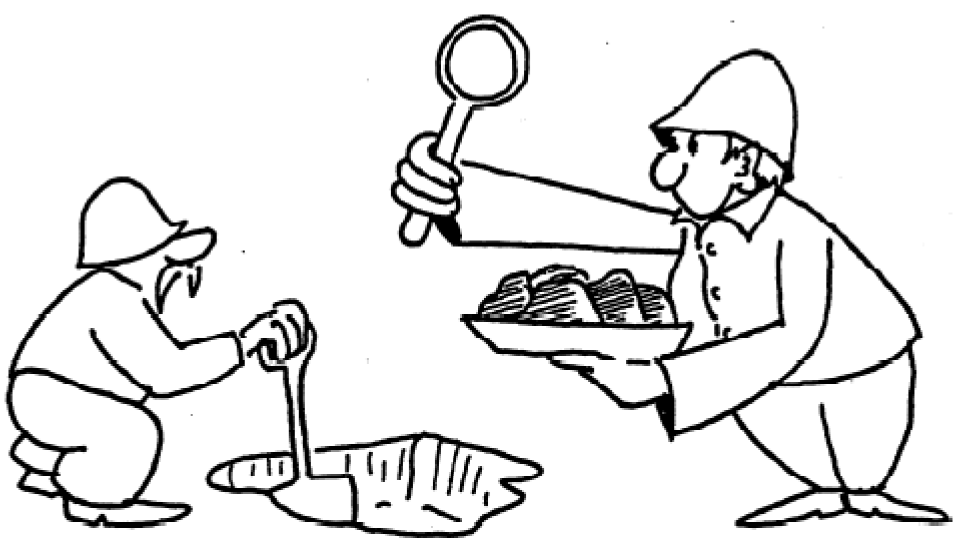
1. Number used for subtopic to uppercase letter.

Another Heading.

JULY 2025

CHAPTER 3

SOILS



George Thompson

Chapter 3: Soils

Table of Contents

Chapter 3: Soils 1
Table of Contents 1
3-1 Introduction 2
3-2 Soil Identification, Classification, Description, and Presentation 2
3-3 Soil Properties and Strength 4
3-4 Standard Penetration Test (SPT) 4
3-5 Cone Penetration Test (CPT) 7
3-6 Field and Laboratory Tests 9
3-7 Shear Strength 10
3-8 Contractor Soil Investigations 11
3-9 Special Ground Conditions 11

3-1 Introduction

To verify the adequacy of a shoring system in soil, it is necessary to be familiar with the properties and expected behaviors of the types of soil in which the excavation is to be made. The lateral earth pressure exerted on a shoring system depends on the soil type, its density or consistency, and other factors such as external loads, the type of retaining system used, and the construction procedure. For most projects, the geotechnical investigation and geotechnical report(s), issued by the Division of Engineering Services - Geotechnical Services, should present sufficient information for the Engineer to perform shoring design and analyses. Contact Geotechnical Services for guidance when additional soil properties are needed for the design review. When the material encountered during the installation of the shoring system differs from the material that was anticipated, contact the shoring system designer. This chapter discusses the Department's resources for soil information and provides guidance on how to use this information to determine parameters necessary for the design or verification of a shoring system.

3-2 Soil Identification, Classification, Description, and Presentation

The Contractor can obtain soil classification characteristics from the information provided in the *Geotechnical Design Report* or *Foundation Report* and corresponding Log of Test Borings, by performing independent sampling and analysis of the soil, or by having a competent person classify the soil per Cal/OSHA Construction Safety Orders, § 1541.1, *Requirements for Protective Systems*, Appendix A, *Soil Classification*. Note that the Construction Safety Orders are found in the California Code of Regulations (CCR) Title 8, Chapter 4, Subchapter 4.

The Cal/OSHA soil classification methods include a series of visual methods as well as a series of manual tests. The Construction Safety Orders, § 1541.1, Appendix A (c), *Requirements*, requires that the classification of soil deposits must be made based on the results of at least one visual and at least one manual analysis as described in paragraph (d), or in other approved methods of soil classification and testing. Some of the acceptable manual tests described in paragraph (d) are similar to those used in the Caltrans *Soil and Rock Logging, Classification, and Presentation Manual*, 2022 Edition *(Soil and Rock Logging Manual)*, including the dry strength and pocket penetrometer tests. The competent person will use the quantitative and qualitative information obtained from the visual and manual tests to classify the soils as either stable rock, Type A, Type B, or Type C soil. Depending on the type of soil classified, an unconfined compressive strength value is assigned. Unconfined compressive strength is defined in the Cal/OSHA standard as "the load per unit area at which a soil will fail in compression."

It is the Engineer's responsibility to verify that the soil properties used by the Contractor's engineer in their shoring design submittal are appropriate. It is recommended that the Engineer contact the author of the Caltrans *Foundation Report* or *Geotechnical Design Report* to discuss and verify. The *Foundation Report* provides geotechnical information for structure items, while the *Geotechnical Design Report* is geared towards roadway items and provides geotechnical information for the entire project footprint.

Caltrans uses geotechnical reports, Log of Test Boring (LOTB) sheets and boring records to present the results of its geotechnical and borehole investigations. LOTB sheets are included in the contract plans for structures. They present the boring logs that both graphically and descriptively convey the soil descriptions and sampling information. The Standard Plans, Sheets A10F and A10G, *Legend – Soil*, provide additional information on soil classification, and Sheet A10H, *Legend – Rock*, provides additional information on rock classification. The *Soil and Rock Logging Manual*, maintained by Geotechnical Services, presents in further detail the Department's practice for identification, classification, description, and presentation of soil and rock for all investigations after December 7, 2009.

Correct interpretation of LOTB sheets, boring records, and related discussions in geotechnical reports requires familiarity with this manual. The following is an overview of the Department's soil presentation practice.

The descriptive sequence for a soil consists of a *group name* and *group symbol*, followed by descriptive components such as density or consistency, color, moisture, etc. The group name and group symbol of a soil, "SANDY lean CLAY (CL)" for example, are determined using one of the following standards:

- ASTM D2488, *Standard Practice for Description and Identification of Soils* (Visual-Manual Procedures), if laboratory testing is not performed.
- ASTM D2487, *Standard Practice for Classification of Soils for Engineering Purposes* (Unified Soil Classification System), if laboratory particle size analysis and plasticity index tests are performed.

The descriptive components following the group name and group symbol are defined in Section 2, *Field Procedures for Soil and Rock Logging, Description, and Identification,* of the *Soil and Rock Logging Manual.* This section provides details of the Department's practice for identifying and describing soil in the field following the group name and group symbol which are defined therein. Section 3, *Procedures for Soil and Rock Description and/or Classification Using Laboratory Test Results,* presents the practice of soil classification and description based on laboratory test results.

Soils are identified or classified as either *coarse-grained* (gravel and sand) or *fine-grained* (silts and clays). Natural soil consists of one or any combination of gravel, sand, silt, or clay, and may also contain boulders, cobbles, and organics.

Coarse-grained soils retain more than 50 percent of material on or above the No. 200 sieve (0.075mm). GRAVEL (G) and SAND (S) are further identified or classified according to their gradation as well-graded (W) or poorly graded (P), SILT content (M), or CLAY content (C). Examples of these are *Well-graded SAND (SW)* or *SILTY SAND (SM)*.

Fine-grained soils pass more than 50 percent of material through the No. 200 sieve. SILT (M), CLAY (C), and ORGANIC SOIL (O) are further identified by visual methods or classified by laboratory plasticity tests as low plasticity (L) or high plasticity (H). Examples of these are *lean CLAY (CL)* or *SANDY SILT (ML).*

3-3 Soil Properties and Strength

Characteristics or properties that help predict the effect of a soil on a shoring system include the particle distribution (% gravel, % sand, % fines [silt & clay]), particle angularity, apparent density or consistency (strength), moisture, and unit weight. The *Soil and Rock Logging Manual* presents the Department's standards of measuring or determining these properties either visually (Section 2) or with laboratory testing (Section 3).

Typically, the Department uses one or more of the following investigative methods to determine a soil's identification, classification, description, and strength:

- Standard Penetration Test (SPT) with visual/manual methods
- Cone Penetration Test (CPT)
- Laboratory testing.

3-4 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) obtains a disturbed sample of soil for visual identification and description, and/or laboratory testing (particle size analysis, plasticity index). The number of hammer blows required to drive the split-spoon sampler a distance of 12 inches into the ground, is referred to as the **N** value. When corrected for the SPT hammer's energy efficiency, it becomes $\mathbf{N_{60}}$. This can be used to determine the apparent density of a granular soil. Empirical relationships to approximate the soil friction angle (ϕ) and density (unit weight), γ, are shown in Table 3-1.

Table 3-1. Properties of Granular Soils

Apparent Density	Relative Density (%)	SPT, N_{60} (blows/ft)	Friction Angle, ϕ (deg)	Unit Weight, γ (pcf)	
				Moist	Submerged
Very Loose	0-15	$N_{60} < 5$	<28	<100	<60
Loose	16-35	$5 \leq N_{60} < 10$	28-30	95-125	55-65
Medium Dense	36-65	$10 \leq N_{60} < 30$	31-36	110-130	60-70
Dense	66-85	$30 \leq N_{60} < 50$	37-41	110-140	65-85
Very Dense	86-100	$N_{60} \geq 50$	>41	>130	>75

Note: Both the LOTB and boring records report the SPT blow count observed in the field as the **N** value, not $\mathbf{N_{60}}$ as used above, to determine the apparent density descriptor. The reader is encouraged to read the *Soil and Rock Logging Manual* on apparent density and Appendix A, *Field Test Procedures*, Section A.8, *Standard Penetration Test,* prior to using Table 3-1. There are a variety of correction factors that can be applied to the **N** value such as for overburden pressure. It is important to know what, if any, correction factors have been applied to the **N** value for the correct interpretation of Table 3-1.

The Division of Engineering Services, Geotechnical Services, has prepared a summary of "simplified typical soil values." For average trench conditions, the Engineer will find the data very useful to establish basic properties or evaluate data submitted by the Contractor; Table 3-2 lists approximate values.

Table 3-2. Simplified Typical Soil Values

Soil Classification	ϕ Friction Angle of the Soil	Density or Consistency	γ Soil Unit Weight (pcf)	K_a Coefficient of Active Earth Pressure	$K_w=K_a\gamma$ Equiv. Fluid Wt. (pcf)
Gravel, Gravel-Sand Mixture, Coarse Sand	41 34 29	Dense Medium Dense Loose	130 120 90	0.21 0.28 0.35	27 34 32
Medium Sand	36 31 27	Dense Medium Dense Loose	117 110 90	0.26 0.32 0.38	30 35 34
Fine Sand	31 27 25	Dense Medium Dense Loose	117 100 85	0.32 0.38 0.41	37 38 34
Fine Silty Sand, Sandy Silt	29 27 25	Dense Medium Dense Loose	117 100 85	0.35 0.38 0.41	41 38 34
Silt	27 25 23	Dense Medium Dense Loose	120 110 85	0.38 0.41 0.44	45 45 37

For active pressure conditions, use a unit weight value of γ = 115 pcf (pounds per cubic feet) minimum when insufficient soils data is known.

It is not the Department's practice to use the SPT test as a means of estimating the shear strength of cohesive soil. Field tests on relatively undisturbed samples, including the pocket penetrometer, Torvane, and laboratory tests such as triaxial, unconfined compression, and direct shear, are considered more accurate and are discussed in the *Soil and Rock Logging Manual*. Field and/or laboratory test results are typically available in the *Foundation Report* and/or *Geotechnical Design Report* issued by Geotechnical Services staff, and it is recommended that the Engineer use those results in their shoring analyses. In the absence of any field or laboratory test results for cohesive soil, the consistency descriptor can be roughly correlated to shear strength and unit weight as shown in Table 3-3.

Table 3-3. Simplified Typical Properties of Cohesive Soils

Consistency	Unconfined Compressive Strength (psf)	Moist Unit Weight (pcf)
Very Soft	0-500	<100-110
Soft	500-1,000	100-120
Medium Stiff	1,000-2,000	110-125
Stiff	2,000-4,000	115-130
Very Stiff	4,000-8,000	120-140
Hard	>8,000	>132

3-5 Cone Penetration Test (CPT)

The Cone Penetration Test (CPT) is used by the Department to determine the in-situ properties of soil. The CPT consists of pushing a conically tipped, cylindrical probe into the ground at a constant rate. The probe is instrumented with strain gauges to measure resisting force against the tip and along the side while the probe is advancing downward. A computer controls the advance of the probe and the acquisition of data, and a continuous record of subsurface information is collected. The results of a CPT are presented on either a LOTB plan sheet or on 8 1/2 x 11 sheets as presented in Figure 3-1 and Figure 3-2.

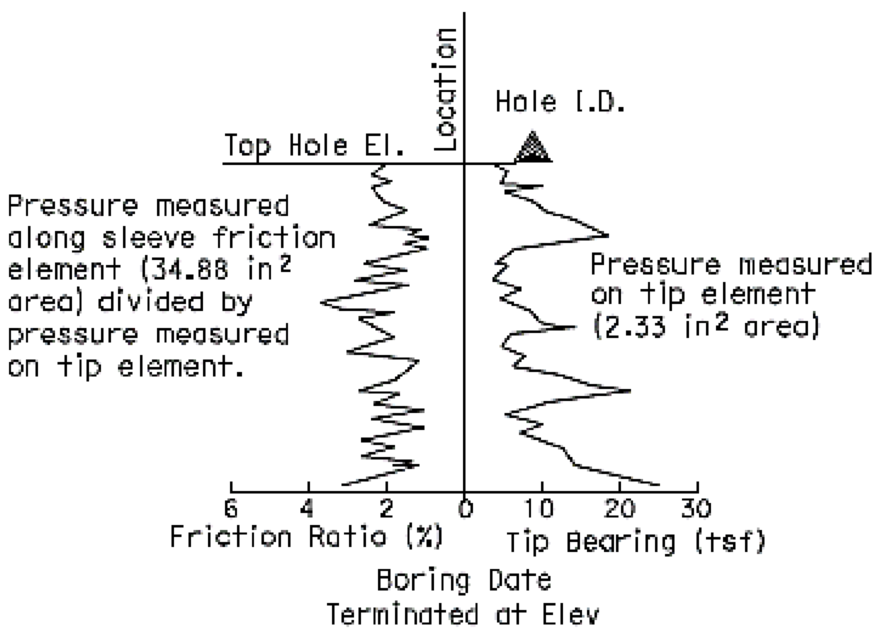


Figure 3-1. Cone Penetration Test (CPT) Boring

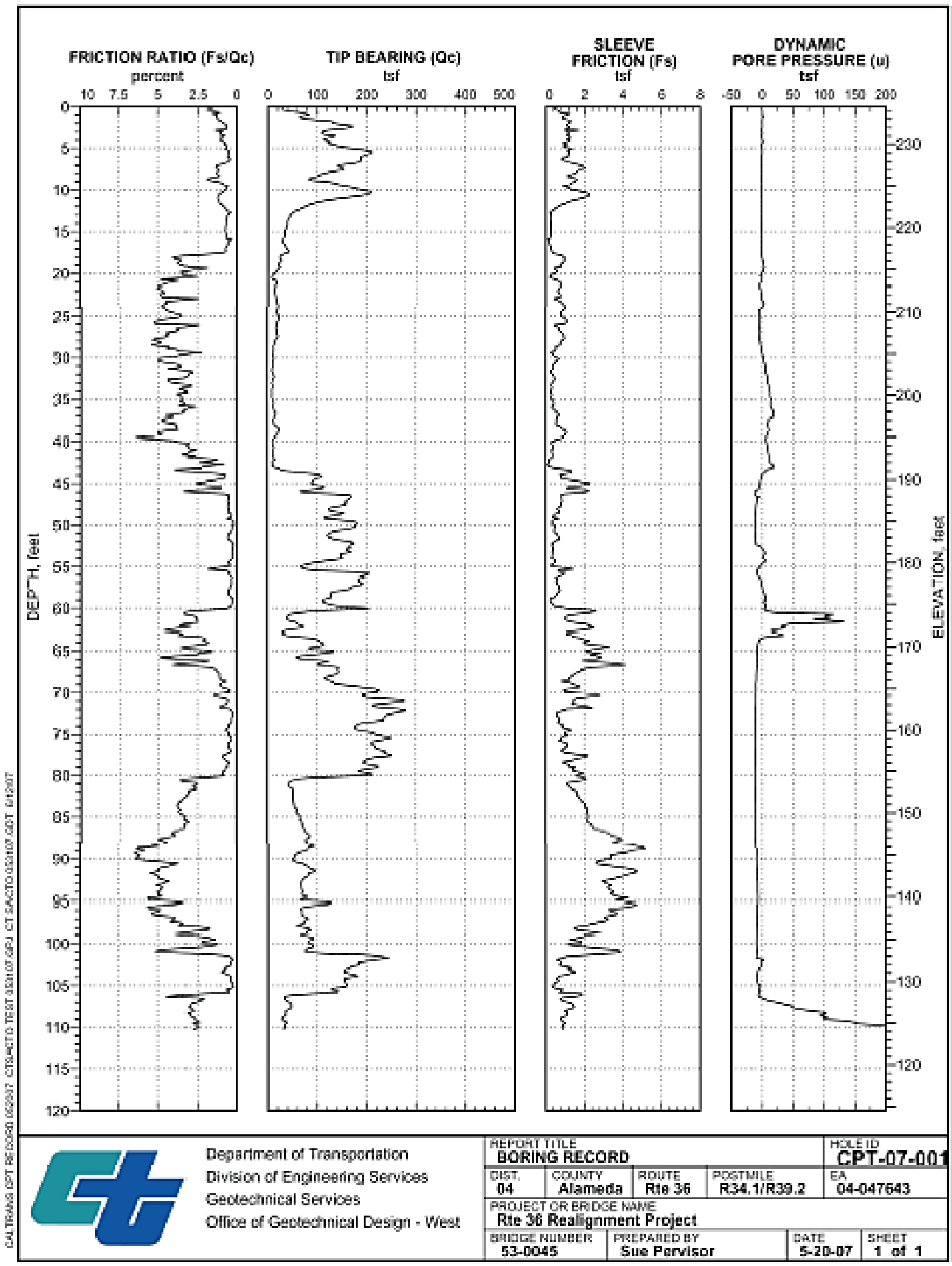


Figure 3-2. Typical CPT Plot

The CPT cannot recover soil samples, so visual/manual soil identification is not possible. However, it is possible to obtain approximate soil identification, relative density for granular soils, and undrained shear strength (**S_u**) for fine-grained soils by using several published relationships. The Engineer should review the appropriate project geotechnical report(s) for discussions relative to soil identification and strength from CPT investigations or contact Geotechnical Services for guidance on the interpretation of CPT data relating to shoring analysis and design.

3-6 Field and Laboratory Tests

Not all methods of evaluating soil shear strength are equally accurate. Therefore, the source of the shear strength data must be considered when evaluating a proposed trenching or shoring system. Table 3-4 presents a list of field and laboratory tests that are used to measure or estimate soil shear strength and an indication of their reliability.

Table 3-4. Field and Laboratory Test Reliability of Soil Shear Strength Measurements

Test Method	Coarse-grained Soil	Fine-grained Soil
Standard Penetration Test (SPT) (ASTM D1586)	Good	Poor
Cone Penetration Test (CPT) (ASTM D3441)	Good	Fair
Pocket Penetrometer	Not applicable	Fair
Torvane (shearvane)	Not applicable	Fair
Vane Shear (ASTM D2573)	Not applicable	Very good
Triaxial Compression (UU,CU) (ASTM D2850)	Very good*	Very good
Unconfined Compression (ASTM D2166)	Not applicable	Very good
Direct Shear (ASTM D3080)	Good*	Fair

*Recovery of undisturbed samples can be difficult

The Torvane and pocket penetrometer tests are field tests you may see noted on the LOTB of the plans. The Department uses these tests to aid in determining the uniformity of the cohesive soil encountered. The Contractor may perform their own tests while excavating. The Contractor may overly emphasize these field tests, but the tests need to be used in conjunction with all the materials information in the logs and geotechnical reports. The pocket penetrometer provides a value for a cohesive soils' unconfined compressive strength, $\mathbf{q_u}$, in tons per square foot. The Torvane (or shearvane) is a soil-testing tool utilized for determination of cohesive soils' undrained shear strength ($\mathbf{S_u}$). The use of a Torvane is an acceptable practice, meeting the requirements of Cal/OSHA *CSO,* § 1541.1, *Requirements for Protective Systems,* Appendix A, *Soil Classification*, subsection (d)(2)(D), *Acceptable Visual and Manual Tests*. In situations where the Contractor is proposing to use a Torvane, consult the SC Falsework Engineer in Structure Construction Headquarters[1] for assistance.

[1] Caltrans internal use only

For projects utilizing a cohesion value, **c**, it is recommended the following conditions be included and addressed with each shoring submittal:

1. Methods used to prevent tension cracks from developing at the top of the excavation.
2. Methods used to prevent water from ponding at the top of and the toe of the slope.
3. Verification that the soil is consistent and homogeneous throughout the excavation.

Additional information on the tests listed in Table 3-4 is in the *Soil and Rock Logging Manual*, Sections 2.5.3, *Consistency of Cohesive Soil*, and Appendix A, *Field Test Procedures*, Section A.2, *Torvane*.

3-7 Shear Strength

The shear strength of soil is measured by two parameters, the angle of internal friction ϕ, which is the resistance to interparticle slip, and the soil cohesion, which is the interparticle attraction of the soil particles. The design of most geotechnical structures requires a quantitative determination of the soil shear strength. One of the fundamental relationships governing soil shear strength is:

$$\tau_f = c + \sigma \tan \phi \qquad (3\text{-}7\text{-}1)$$

Where:

τ_f = Soil Shear Strength at Failure
ϕ = Internal Friction Angle
σ = Normal Stress
c = Soil Cohesion

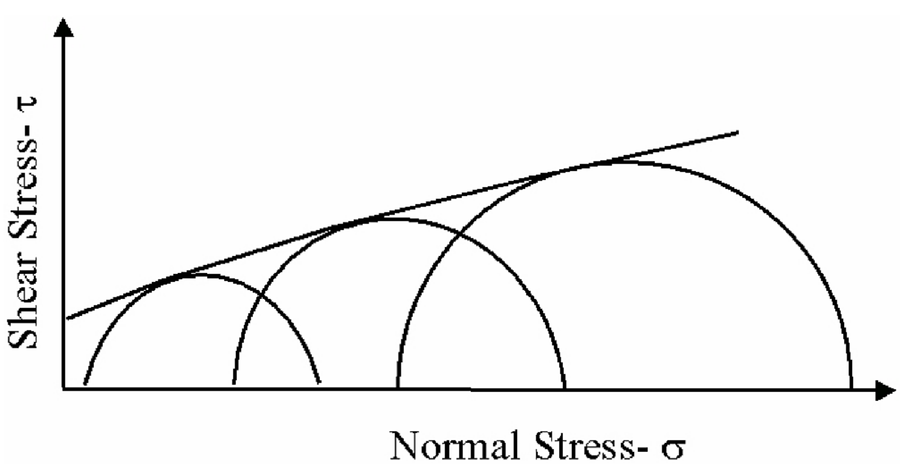


Figure 3-3. Mohr-Coulomb Criteria

In general, the relationship between shear strength and normal stress is not linear for large stress ranges. The strength envelope is a curve that is tangent to the Mohr circles as shown in Figure 3-3. The point of tangency to the Mohr circles represents the stress conditions on the failure plane of the sample.

In fine-grained (cohesive) soils, shear strength is initially insensitive to confining pressure since the strength is derived through cohesion (interparticle attraction). For cohesive soils, the failure criterion simplifies to:

$$\boldsymbol{\tau_f = S_u} \qquad (3\text{-}7\text{-}2)$$

Where $\mathbf{S_u}$ is the undrained shear strength.

Cohesive soils will consolidate or swell over time depending on whether the soil has been loaded or unloaded, respectively. Trenching and shoring work often creates situations where soil loading is reduced, such as in an excavation. A fine-grained soil subjected to unloading will then expand and has the potential to lose shear strength over time.

3-8 Contractor Soil Investigations

The Contractor may elect to have a soils investigation performed to support their shoring design. In this case, the soils information or report will be furnished to the Engineer as part of the supporting data accompanying the shoring plans. Soil test results need to be used with caution. Soil test reports from many soils laboratories or similar sources may or may not include safety factors incorporated in the reported results.

Factors that the Engineer will consider when assigning strength parameters to a soil include:

1. The method with which the soil shear strength was determined (Table 3-4),
2. The variability of the subsurface profile, and
3. The number and distribution of shear strength tests.

3-9 Special Ground Conditions

Occasionally, excavations are made into soil or rock with properties that require special consideration in the design of the shoring system. The special condition must be defined, and the expected behavior of the material during and after excavation and installation of the shoring system, must be understood. Typically, the geotechnical report would identify and discuss the presence of special soil or rock conditions and it is recommended that Geotechnical Services be contacted for assistance with these situations.

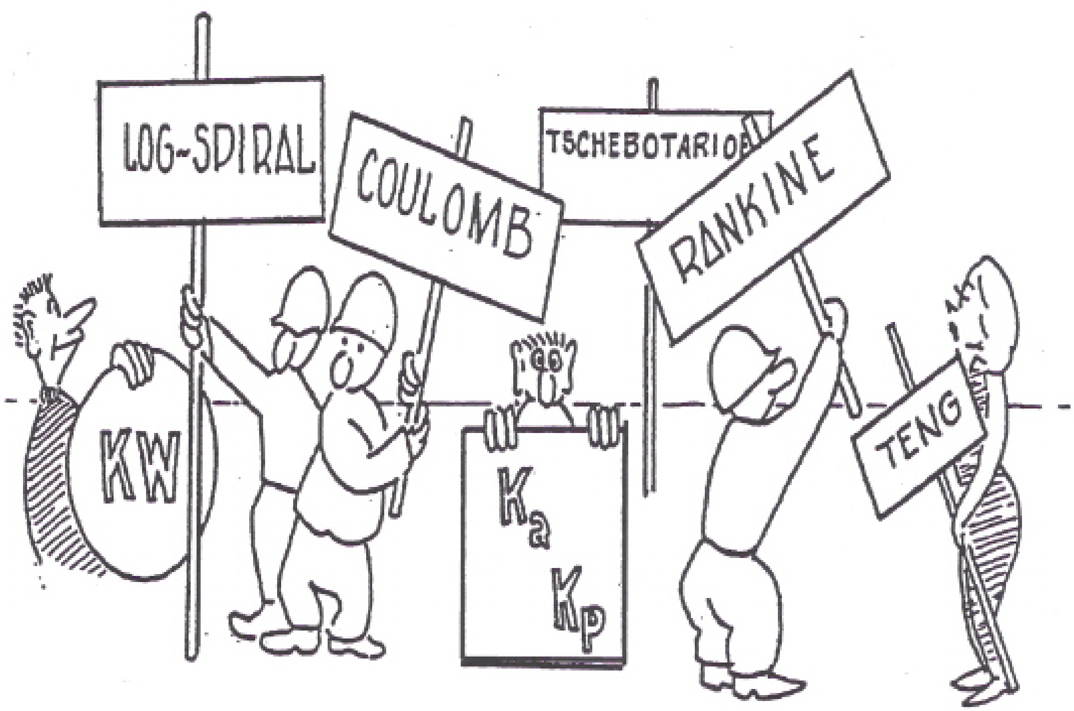
Examples of special ground conditions are:

1. Fractured rock: Adversely oriented bedding or fracturing, which would allow toppling or wedge failure into the excavation, should be identified and accounted for in the design of the shoring system.
2. Organic soil: Organic soils, such as peat, are compressible and subject to decomposition, which can lead to significant volume changes.
3. Clay and shale: Subject to cracking and spalling upon exposure to the atmosphere, swelling and slaking when exposed to water, and weakening when unloaded. Excavating such materials might require protection of the shoring system to help retain natural moisture content to prevent cracking and spalling. Design analyses need to account for the expected disturbed strength of the retained material, which might be weaker than in-situ.
4. Running soil: A soil that cannot stand by itself even for a short term. Running soil will have little shear strength and will flow with virtually no angle of repose in an unsupported condition. The cohesive value (**c**) is equal to zero.
5. Quick condition: Occurs when the upward flow of water through a soil is sufficient to make the soil buoyant and thereby prevent grain interlocking. The soil grains are suspended in the water. A quick condition can be created in an excavation when the water level in the excavation is lower than the level outside, in the surrounding soil. It may best be stabilized by equalizing the water levels. Quick conditions can occur in some silts as well as in sand. For additional information on this topic, refer to Section 10-4, *Piping*, of Chapter 10, *Special Conditions*.

JULY 2025

CHAPTER 4

EARTH PRESSURE THEORY AND APPLICATION



George Thompson

Chapter 4: Earth Pressure Theory and Application

Table of Contents

Chapter 4: Earth Pressure Theory and Application 1
Table of Contents 1
4-1 Introduction 2
4-2 Shoring Types 2
4-2.01 Unrestrained Shoring Systems 2
4-2.02 Restrained Shoring Systems 2
4-2.03 Shoring Movement and Loading 3
4-3 Developing Earth Pressures for Granular Soil 5
4-3.01 At-Rest Lateral Earth Pressure Coefficient (K_0) 8
4-3.02 Active and/or Passive Earth Pressure 9
4-3.03 Summary of Earth Pressure Theories 16
4-4 Developing Earth Pressures for Cohesive Soil 16
4-5 Estimating Maximum Allowable Embankment Slope Angle 22
4-5.01 Active Trial Wedge Method 23
4-5.02 Passive Trial Wedge Method 25
4-5.03 Culmann's Graphical Solution for Active Earth Pressure 26
4-6 Log-Spiral Passive Earth Pressure 28

4-1 Introduction

All shoring systems must be designed to withstand lateral earth pressure, water pressure, and the effect of surcharge loads in accordance with the general principles and guidelines specified in this *California Department of Transportation Trenching & Shoring Manual*. The Simplified Method for cantilevered systems presented is developed around AASHTO and Transportation Research Board National Cooperative Highway Research Program (TRB NCHRP Report 611) equations. The Conventional Method for cantilevered systems presented in the 1984 *USS Steel Sheet Piling Design Manual* is presented here as the Rigorous Method (see Appendix D, *Sheet Piles,* for additional information on this reference). This method is a reintroduction of a past practice used in the pre-2006 *Trenching and Shoring Manual*. The Rigorous Method, as the name implies, produces a more refined or precise analysis.

This chapter will describe the two basic categories of shoring systems, the development of soil pressures behind the shoring, and how to use these pressures to develop the loads the shoring system must resist.

4-2 Shoring Types

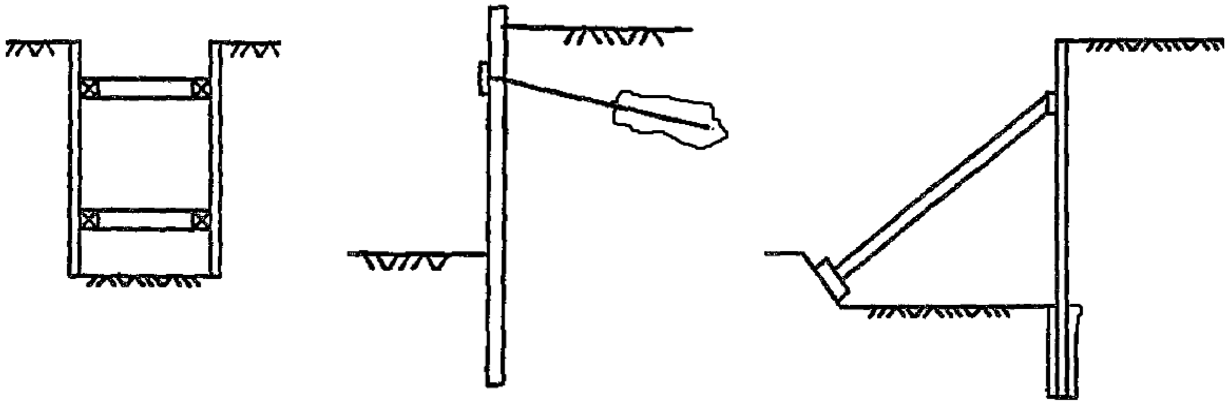
Shoring systems are generally classified as unrestrained (non-gravity cantilevered walls) and restrained (braced or anchored walls). Unrestrained shoring systems rely on structural components of the wall partially embedded in the foundation material to mobilize passive resistance to lateral loads. Restrained shoring systems derive their capacity to resist lateral loads by their structural components being restrained by tension or compression elements connected to the vertical structural members of the shoring system and, additionally, by the partial embedment (if any) of their structural components into the foundation material.

4-2.01 Unrestrained Shoring Systems

Unrestrained shoring systems (non-gravity cantilevered walls) are constructed of vertical structural members consisting of partially embedded soldier piles or continuous sheet piles. This type of system depends on the 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 economical height of this type of wall is generally limited to a maximum of 18 feet.

4-2.02 Restrained Shoring Systems

Restrained shoring systems are either anchored or braced walls, as illustrated in Figure 4-1. They are typically comprised of the same elements as unrestrained (non-gravity cantilevered) walls but derive additional lateral resistance from one or more levels of braces, rakers, or anchors. These systems would also include a trench system, where the sides are braced against each other with two or more levels of struts between the sides.



Braced trench with two rows of struts

Cantilever Wall restrained with a single ground anchor

Cantilever Wall restrained with a single brace (or raker)

Figure 4-1. Examples of Restrained Shoring Systems

These walls are typically constructed in cut situations in which construction proceeds from the top down to the base of the wall. The vertical wall elements should extend below the potential failure plane associated with the retained soil mass. For these types of walls, heights up to 80 feet are economically feasible.

Note - Soil Nail Walls and Mechanically Stabilized Earth (MSE) Walls are not included in this manual. Both of these types of systems are designed by other methods that can be found online with FHWA or AASHTO.

4-2.03 Shoring Movement and Loading

A major issue in providing a safe shoring system design is to determine the appropriate earth-pressure loading diagram. The loads are to be calculated using the appropriate earth pressure theories. The lateral horizontal stresses (σ_h) for both active and passive pressure are to be calculated based on the soil properties and the shoring system. Earth pressure loads on a shoring system are a function of the unit weight of the soil, location of the groundwater table, seepage forces, surcharge loads, and the shoring structure system. Shoring systems that cannot tolerate any movement should be designed for at-rest lateral earth pressure. At-rest lateral earth pressure is when a wall experiences no lateral movement. Shoring systems which can move away from the soil mass should be designed for active earth pressure conditions depending on the magnitude of the tolerable movement. Any movement, which is required to reach the minimum active pressure or the maximum passive pressure, is a function of the wall height and the soil type. A significant amount of movement is necessary to mobilize the full passive pressure. The variation of lateral stress between the active and passive earth pressure values can be brought about only through lateral movement within the soil mass of the backfill as shown in Figure 4-2.

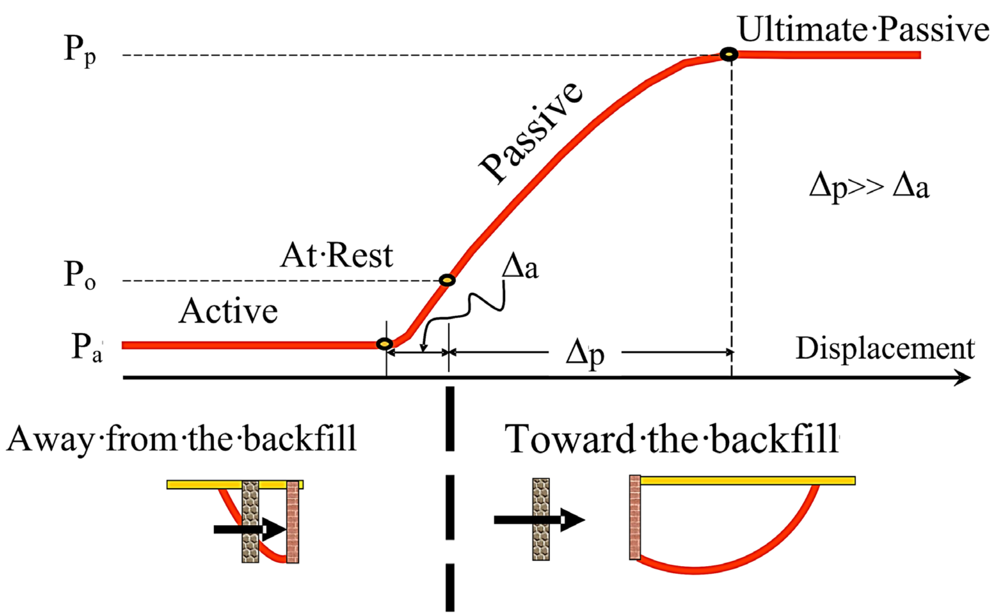


Figure 4-2. Active and Passive Earth Pressure Coefficient as a Function of Wall Displacement

Typical values of these mobilizing movements, relative to wall height, are given in Table 4-1 (Clough 1991).

Table 4-1. Mobilized Wall Movements

Type of Backfill	Value of **Δ/H**	
	Active	Passive
Dense Sand	0.001	0.01
Medium Dense Sand	0.002	0.02
Loose Sand	0.004	0.04
Compacted Silt	0.002	0.02
Compacted Lean Clay	0.01	0.05
Compacted Fat Clay	0.01	0.05

Where:

Δ = the movement of top of wall required to reach minimum active or maximum passive pressure, by tilting or lateral translation.

H = height of wall.

4-3 Developing Earth Pressures for Granular Soil

At present, the methods of analysis in common use for retaining structures are based on Rankine (1857) and Coulomb (1776) theories. Both methods are based on the limit equilibrium approach with an assumed planar failure surface. Developments since 1920, largely due to the influence of Terzaghi (1943), have led to a better understanding of the limitations and appropriate applications of classical earth pressure theories. Terzaghi assumed a logarithmic failure surface. Many experiments have been conducted to validate Coulomb's wedge theory, and it has been found that the sliding surface is not a plane, but a curved surface as shown in Figure 4-3. (Terzaghi 1943).

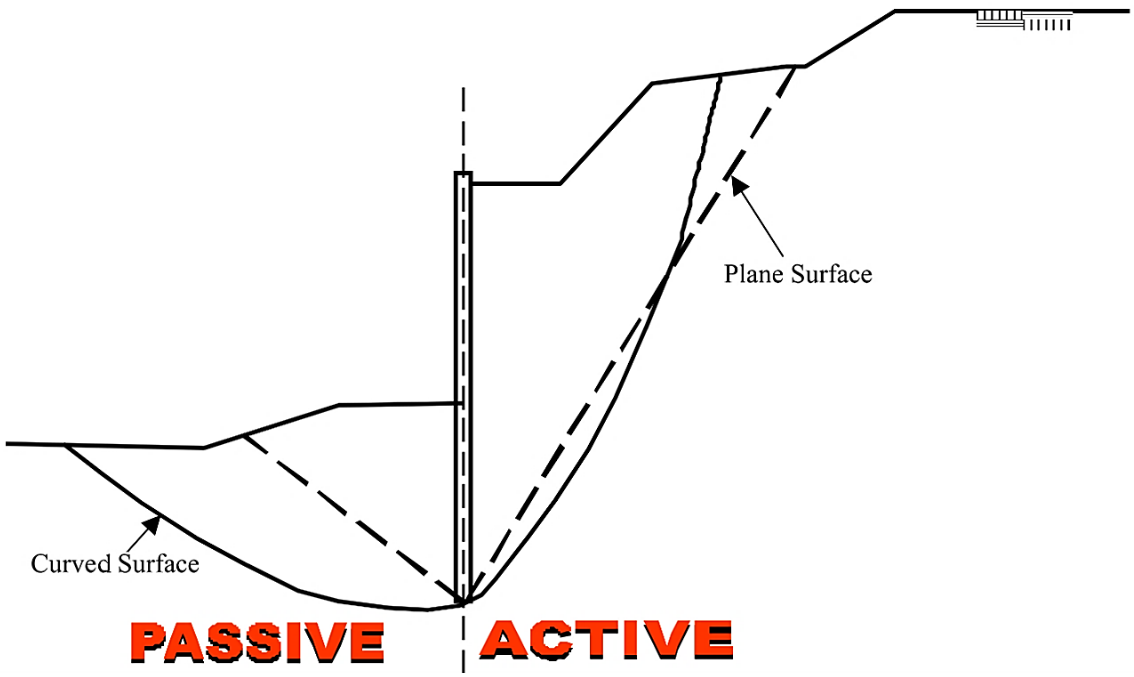


Figure 4-3. Comparison of Plane versus Curved Failure Surfaces

Furthermore, these experiments have shown that the Rankine (1857) and Coulomb (1776) earth pressure theories lead to quite accurate results for the active earth pressure. However, for the passive earth pressure, these theories are accurate only for the backfill of clean, dry sand which would lead to a low wall-interface friction angle between the material and the wall.

For the purpose of the initial discussion, it is assumed that the backfill is level, homogeneous, and the distribution of vertical stress (σ_v) with depth is hydrostatic, as shown in Figure 4-4. The horizontal stress (σ_h) is linearly proportional to depth and is a multiple of vertical stress (σ_v) as shown in Equation 4-3-1.

$$\sigma_h = \sigma_v K = \gamma h K$$ where gamma, γ, is the unit weight of the soil. (4-3-1)

Depending on the wall movement, the coefficient **K** represents the active (**K_a**), passive (**K_p**), or at-rest (**K_o**) earth pressure coefficient in the above equation.

The resultant lateral earth load, **P**, which is equal to the area of the load diagram as calculated in Equation 4-3-2 and illustrated in Figure 4-4, acts at a height of **h/3** above the base of the wall, where **h** is the height of the pressure surface, measured from the surface of the ground to the base of the wall. **P** is the force that causes bending, sliding, and overturning in the wall.

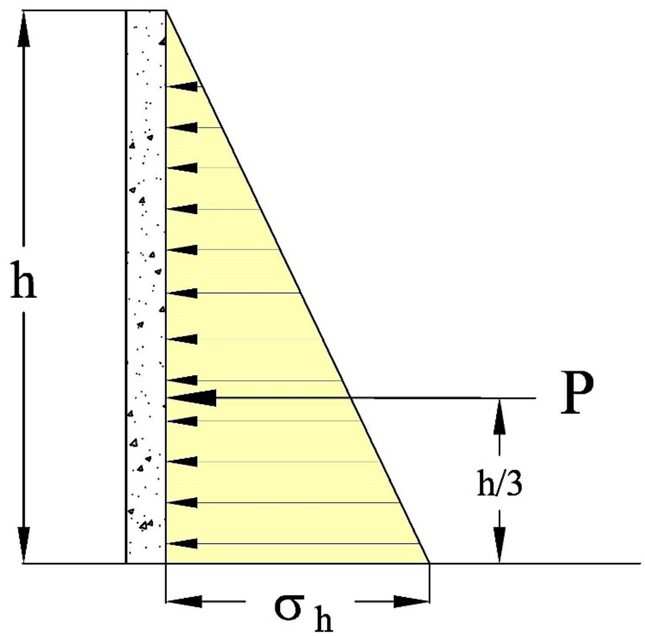


Figure 4-4. Lateral Earth Pressure Variation with Depth

$$\mathbf{P} = \frac{\mathbf{1}}{\mathbf{2}}\boldsymbol{\sigma}_{\mathbf{h}}\mathbf{h} \qquad (4\text{-}3\text{-}2)$$

Depending on the shoring system, the value of the active and/or passive pressure will be determined using either the Rankine, Coulomb, Log-Spiral, or Trial Wedge methods as appropriate.

The state of the active and passive earth pressure depends on the expansion or compression transformation of the backfill from elastic state to state of plastic equilibrium. The concept of the active and passive earth pressure theory can be explained using a continuous anchor block buried near the ground surface for the stability of a sheet pile wall, as shown in Figure 4-5. As a result of wall deflection, Δ, the tie rod is pulled until the active and passive wedges are formed behind and in front of the anchor block. Element P, in front of the anchor block, and element A, at the back of the anchor block, are acted on by two principal stresses: a vertical stress (σ_v) and a horizontal stress (σ_h). In the active case, the horizontal stress (σ_a) is the minor principal stress, and the vertical stress (σ_v) is the major principal stress. In the passive case, the horizontal stress (σ_p) is the major principal stress, and the vertical stress (σ_v) is the minor principal stress. The resulting failure surface within the soil mass corresponding to active and passive earth pressure for the cohesionless soil is shown in Figure 4-6.

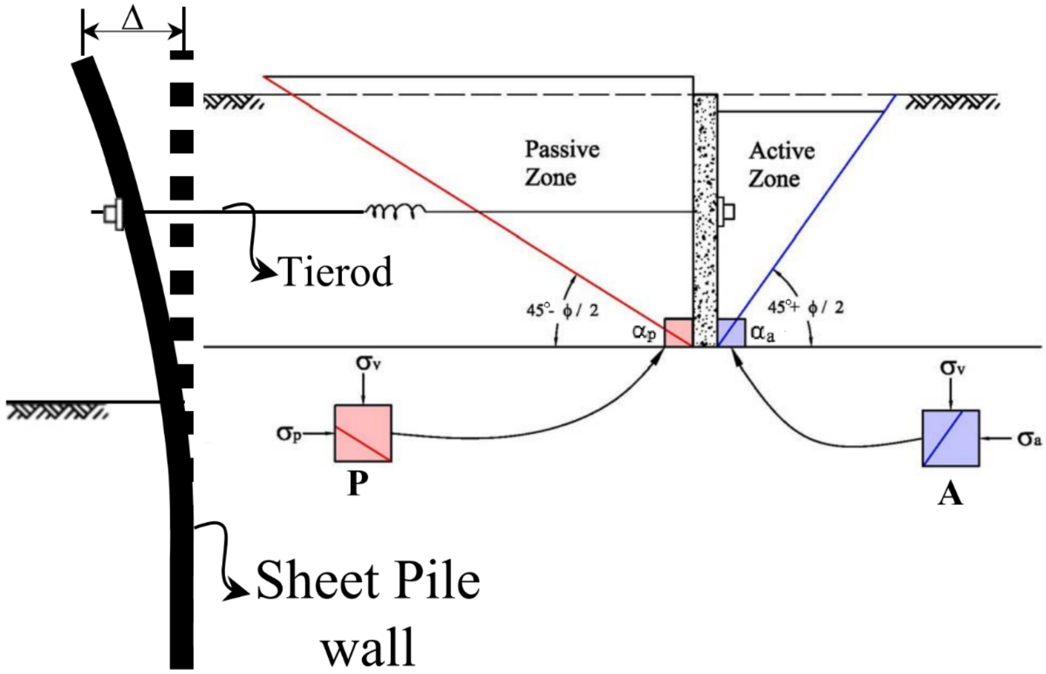


Figure 4-5. Concept of Active and Passive Earth Pressure Theory

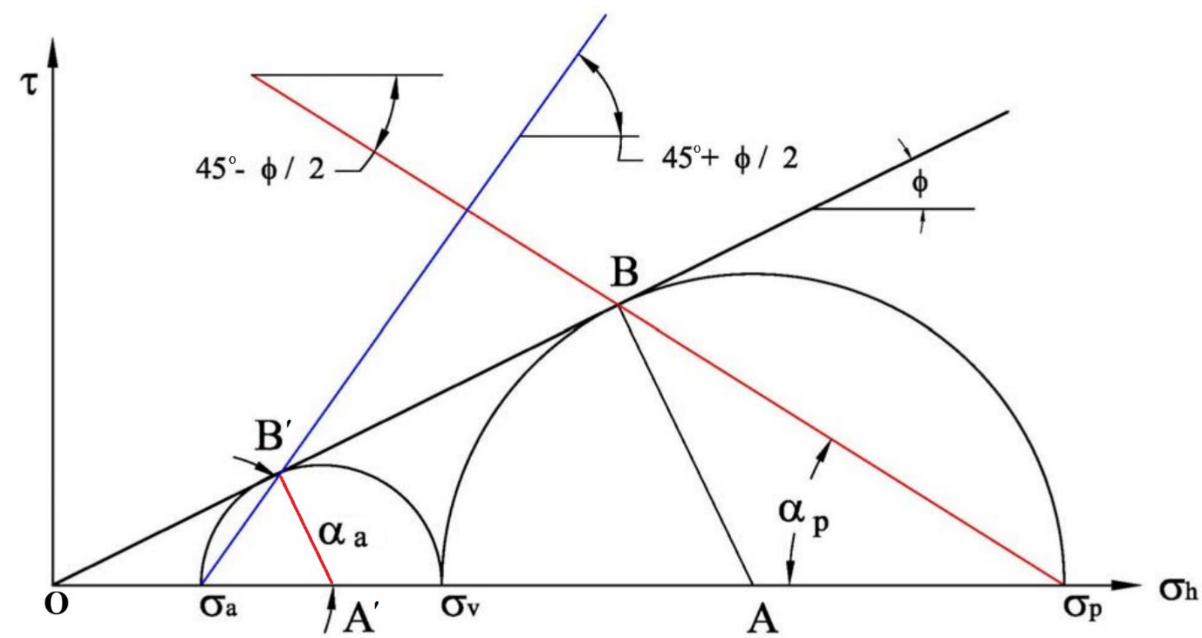


Figure 4-6. Mohr Circle Representation of Earth Pressure for Cohesionless Backfill

From Figure 4-6 above:

$$\sin\emptyset = \frac{A'B'}{OA'} = \frac{\frac{\sigma_v - \sigma_a}{2}}{\frac{\sigma_v + \sigma_a}{2}} \qquad (4\text{-}3\text{-}3)$$

Where **A'B'** is the radius of the small circle:

$$\sin\emptyset = \frac{A'B'}{OA'} = \frac{\sigma_v - \sigma_a}{\sigma_v + \sigma_a} \qquad (4\text{-}3\text{-}4)$$

$$\sigma_v \sin\phi + \sigma_a \sin\phi = \sigma_v - \sigma_a \qquad (4\text{-}3\text{-}5)$$

Collecting Terms:

$$\sigma_a + \sigma_a(\sin\phi) = \sigma_v - \sigma_v(\sin\phi) \qquad (4\text{-}3\text{-}6)$$

$$\sigma_a(1 + \sin\phi) = \sigma_v(1 - \sin\phi) \qquad (4\text{-}3\text{-}7)$$

$$\frac{\sigma_a}{\sigma_v} = \frac{(1 - \sin\phi)}{(1 + \sin\phi)} \qquad (4\text{-}3\text{-}8)$$

From trigonometric identities:

$$\frac{(1 - \sin\phi)}{(1 + \sin\phi)} = \tan^2(45^\circ - \phi/2) \qquad (4\text{-}3\text{-}9)$$

$$\frac{(1 + \sin\phi)}{(1 - \sin\phi)} = \tan^2(45^\circ + \phi/2) \qquad (4\text{-}3\text{-}10)$$

$$K_a = \tan^2(45^\circ - \phi/2),\ \text{where}\ K_a = \frac{\sigma_a}{\sigma_v} \qquad (4\text{-}3\text{-}11)$$

For the passive case:

$$K_p = \frac{\sigma_p}{\sigma_v} = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2(45^\circ + \phi/2) \qquad (4\text{-}3\text{-}12)$$

4-3.01 At-Rest Lateral Earth Pressure Coefficient (K_0)

For a zero lateral strain condition, horizontal and vertical stresses are related by the Poisson's ratio (μ) as follows:

$$K_0 = \frac{\mu}{1 - \mu} \qquad (4\text{-}3\text{-}13)$$

For normally consolidated soils and vertical walls, the coefficient of at-rest lateral earth pressure may be taken as:

$$\mathbf{K_0 = (1 - \sin\ \phi)(1 - \sin\ \beta)} \tag{4-3-14}$$

ϕ = effective friction angle of soil

$\mathbf{K_0}$ = coefficient of at-rest lateral earth pressure

β = slope angle of backfill surface behind retaining wall.

For over-consolidated soils, level backfill, and a vertical wall, the coefficient of at-rest lateral earth pressure may be assumed to vary as a function of the over-consolidation ratio or stress history, and may be taken as:

$$\mathbf{K_0 = (1 - \sin\phi)(OCR)^{\sin\phi}} \tag{4-3-15}$$

Where:

OCR = over consolidation ratio

Note that the equations for the coefficient of at-rest lateral earth pressure are empirical.

4-3.02 Active and/or Passive Earth Pressure

Depending on the shoring system, the value of the active and/or passive pressure can be determined using either the Rankine, Coulomb, or trial wedge methods.

4-3.02A Rankine's Theory

Rankine's theory is the simplest formulation proposed for earth pressure calculations and is based on the following assumptions:

1. The wall is smooth and vertical.
2. There is no friction or adhesion between the wall and the soil.
3. The failure wedge is a plane surface and is a function of soil's friction ϕ and the backfill slope β as shown in Equation 4-3-16 and Equation 4-3-19.
4. Lateral earth pressure varies linearly with depth.
5. The direction of the lateral earth pressure is parallel to the slope of the backfill as shown in Figure 4-7 and Figure 4-8.
6. The resultant earth pressure acts at a distance equal to one-third of the wall height from the base.

Values for the coefficient of active lateral earth pressure using the Rankine's theory may be calculated as shown in Equation 4-3-16.

$$\mathbf{K_a = \cos\ \beta \frac{\cos\ \beta - \sqrt{\cos^2\ \beta - \cos^2\ \phi}}{\cos\ \beta + \sqrt{\cos^2\ \beta - \cos^2\ \phi}}} \quad (4\text{-}3\text{-}16)$$

And the magnitude of active earth pressure can be determined as shown in Figure 4-7 and Equation 4-3-17:

$$\mathbf{P_a = \frac{1}{2}(\gamma)(h^2)(K_a)} \quad (4\text{-}3\text{-}17)$$

The failure plane angle α can be determined as shown in Equation 4-3-18:

$$\boldsymbol{\alpha = \left(45^\circ + \frac{\phi}{2}\right) - \frac{1}{2}\left(\mathrm{Arcsin}\left(\frac{\sin\ \beta}{\sin\ \phi}\right) - \beta\right)} \quad (4\text{-}3\text{-}18)$$

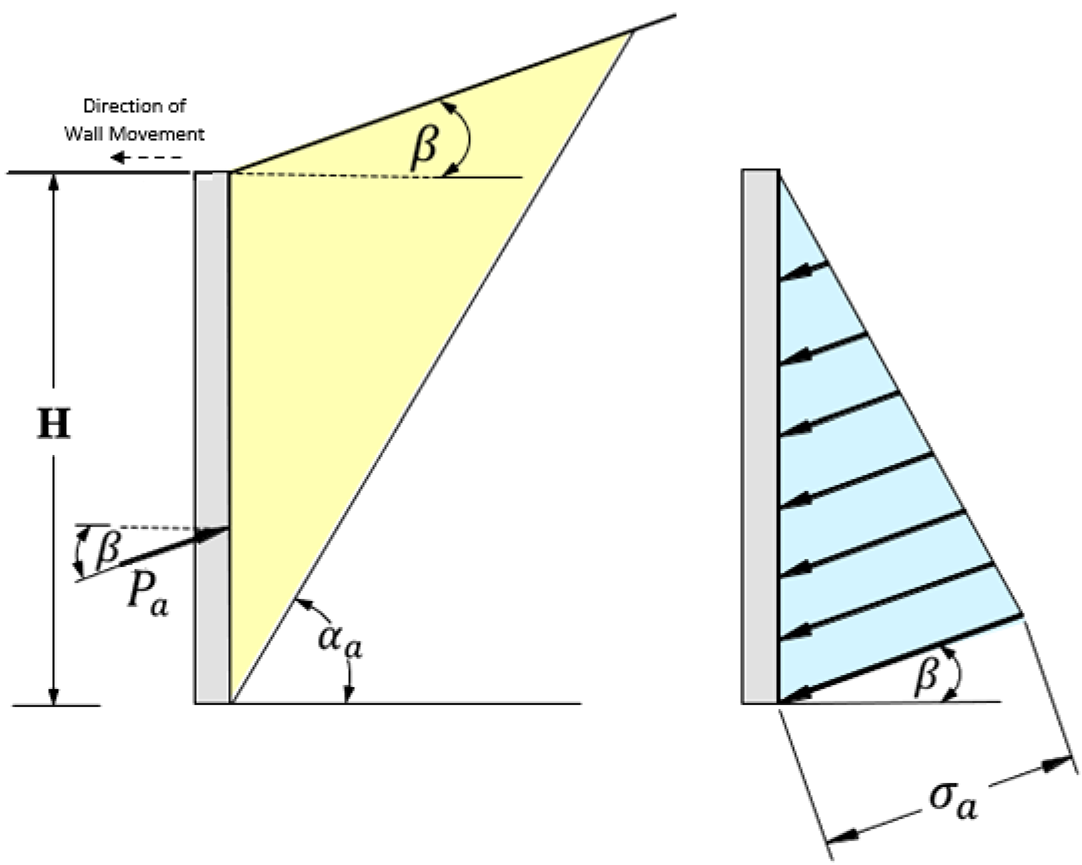


Figure 4-7. Rankine's Active Wedge

Rankine made similar assumptions to his active earth pressure theory to calculate the passive earth pressure. Values for the coefficient of passive lateral earth pressure may be calculated as:

$$\mathbf{K_p = \cos\ \beta \frac{\cos\ \beta + \sqrt{\cos^2\ \beta - \cos^2\ \phi}}{\cos\ \beta - \sqrt{\cos^2\ \beta - \cos^2\ \phi}}} \tag{4-3-19}$$

And the magnitude of passive earth pressure can be determined as shown in Figure 4-8 and Equation 4-3-20:

$$\mathbf{P_P = \frac{1}{2}(\gamma)(h^2)(K_p)} \tag{4-3-20}$$

The failure plane angle α can be determined as shown in Equation 4-3-21:

$$\boldsymbol{\alpha = \left(45^\circ - \frac{\phi}{2}\right) + \frac{1}{2}\left(\mathbf{Arcsin}\ \left(\frac{\sin\ \beta}{\sin\ \phi}\right) + \beta\right)} \tag{4-3-21}$$

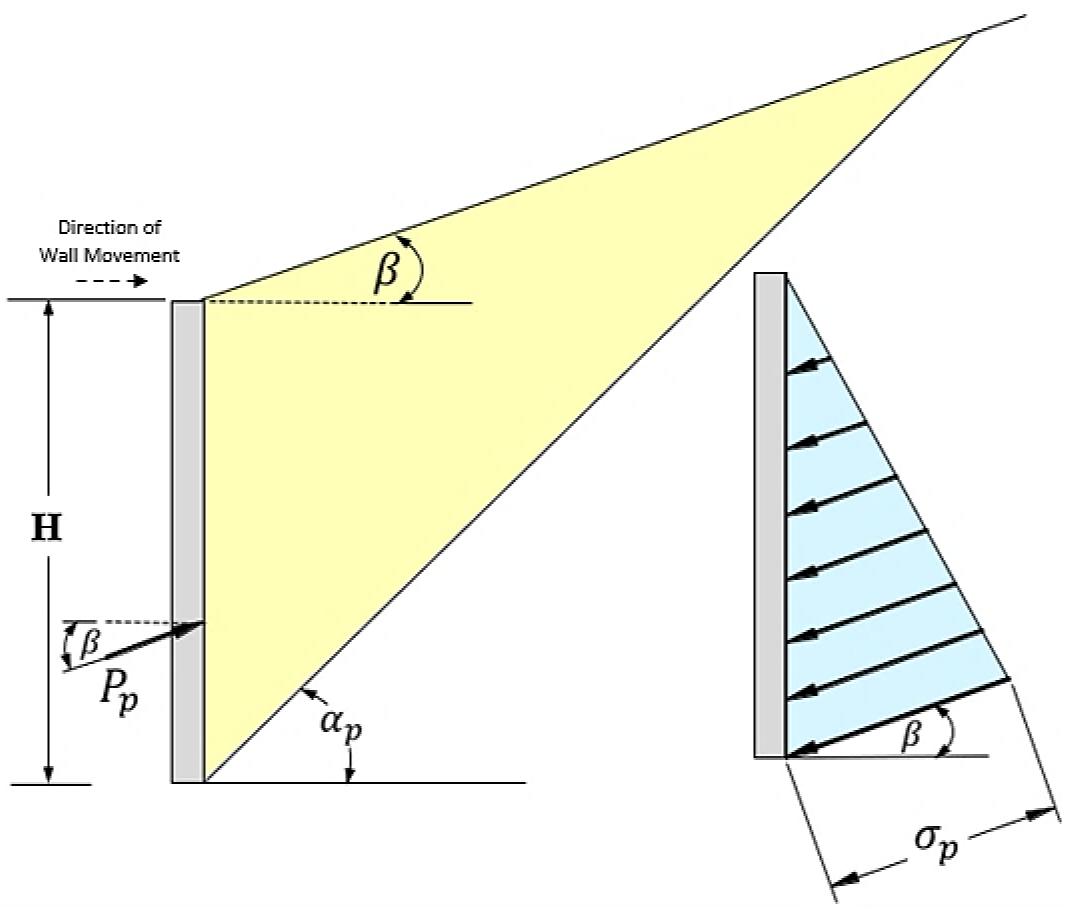


Figure 4-8. Rankine's Passive Wedge

Where:

H	=	Height of pressure surface on the wall
$\mathbf{P_a}$	=	Active lateral earth pressure resultant per unit width of wall
$\mathbf{P_p}$	=	Passive lateral earth pressure resultant per unit width of wall
β	=	Angle from backfill surface to the horizontal
α	=	Failure plane angle with respect to horizontal
ϕ	=	Effective friction angle of soil
$\mathbf{K_a}$	=	Coefficient of active lateral earth pressure
$\mathbf{K_p}$	=	Coefficient of passive lateral earth pressure
γ	=	Unit weight of soil

Although Rankine's equation for the passive earth pressure is provided above, one should not use the Rankine method to calculate the passive earth pressure when the backfill angle is greater than zero ($\beta>0$). As a matter of fact, the $\mathbf{K_p}$ value for both positive ($\beta>0$) and negative ($\beta<0$) backfill slope is identical. This is clearly not correct. Therefore, avoid using the Rankine equation to calculate the passive earth pressure coefficient for sloping ground.

4-3.02B Coulomb's Theory

Coulomb's (1776) earth pressure theory is based on the following assumptions:

1. The wall is rough.
2. There is friction and/or adhesion between the wall and the soil; refer to Table 4-2 for typical values of wall friction.
3. The failure wedge has a plane surface and is a function of the soil friction ϕ, wall friction δ, the backfill slope β, and the slope of the wall ω.
4. Lateral earth pressure varies linearly with depth.
5. The lateral earth pressure acts at an angle δ with a line that is normal to the wall.
6. The resultant earth pressure acts at a distance equal to one-third of the wall height from the base.

Values for the coefficient of active lateral earth pressure may be taken as shown in Equation 4-3-22.

$$\mathrm{K_a} = \frac{\cos^2(\phi - \omega)}{\cos^2\omega\cos(\delta + \omega)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi)\sin(\phi - \beta)}{\cos(\delta + \omega)\cos(\omega - \beta)}}\right]^2} \qquad \text{(4-3-22)}$$

The magnitude of active earth pressure can be determined as shown in Figure 4-9 and Equation 4-3-23.

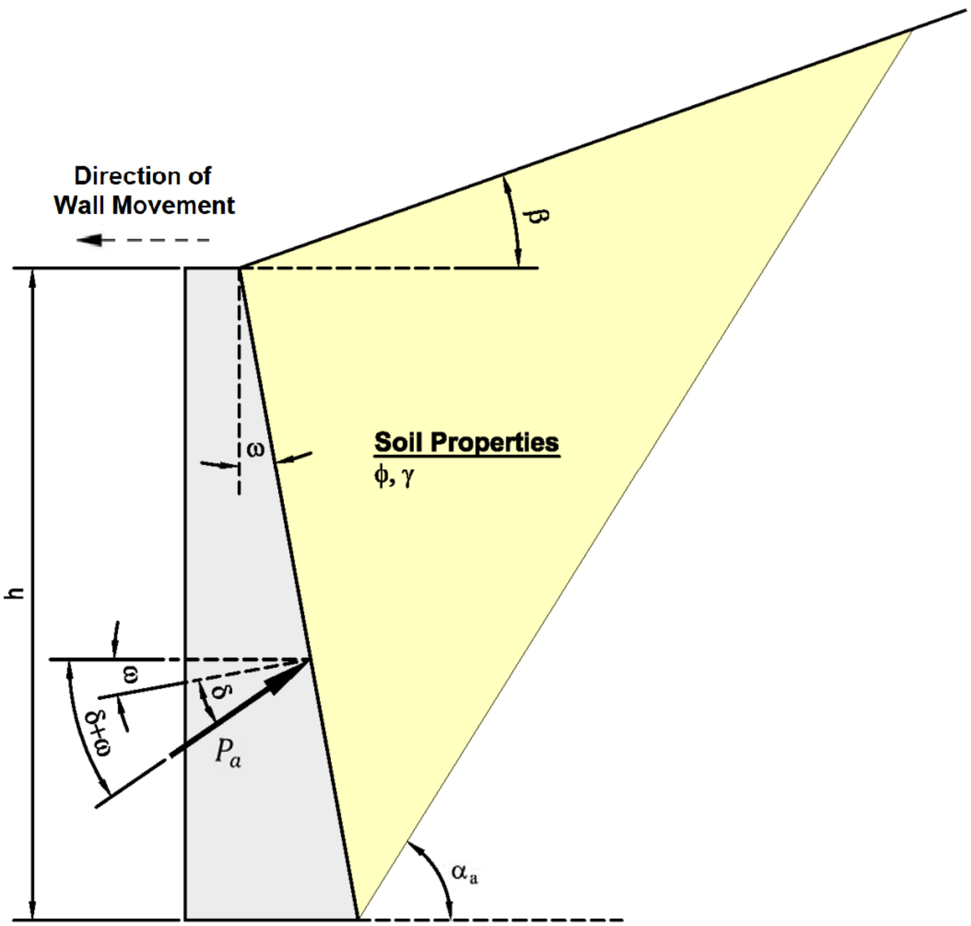


Figure 4-9. Coulomb's Active Wedge

$$\mathbf{P_a = \frac{1}{2}(\gamma)(h^2)(K_a)} \qquad (4\text{-}3\text{-}23)$$

Coulomb's passive earth pressure is derived similarly to his active earth pressure except the inclination of the force is as shown in Figure 4-10. Values for the coefficient of passive lateral earth pressure may be taken as calculated in Equation 4-3-24.

$$\mathbf{K_p = \frac{\cos^2(\phi+\omega)}{\cos^2\omega\cos(\delta-\omega)\left[1-\sqrt{\frac{\sin(\delta+\phi)\sin(\phi+\beta)}{\cos(\delta-\omega)\cos(\beta-\omega)}}\right]^2}} \qquad (4\text{-}3\text{-}24)$$

The magnitude of passive earth pressure can be determined as shown in Figure 4-10 and Equation 4-3-25.

$$\mathbf{P_p = \frac{1}{2}(\gamma)(h^2)(K_p)} \qquad (4\text{-}3\text{-}25)$$

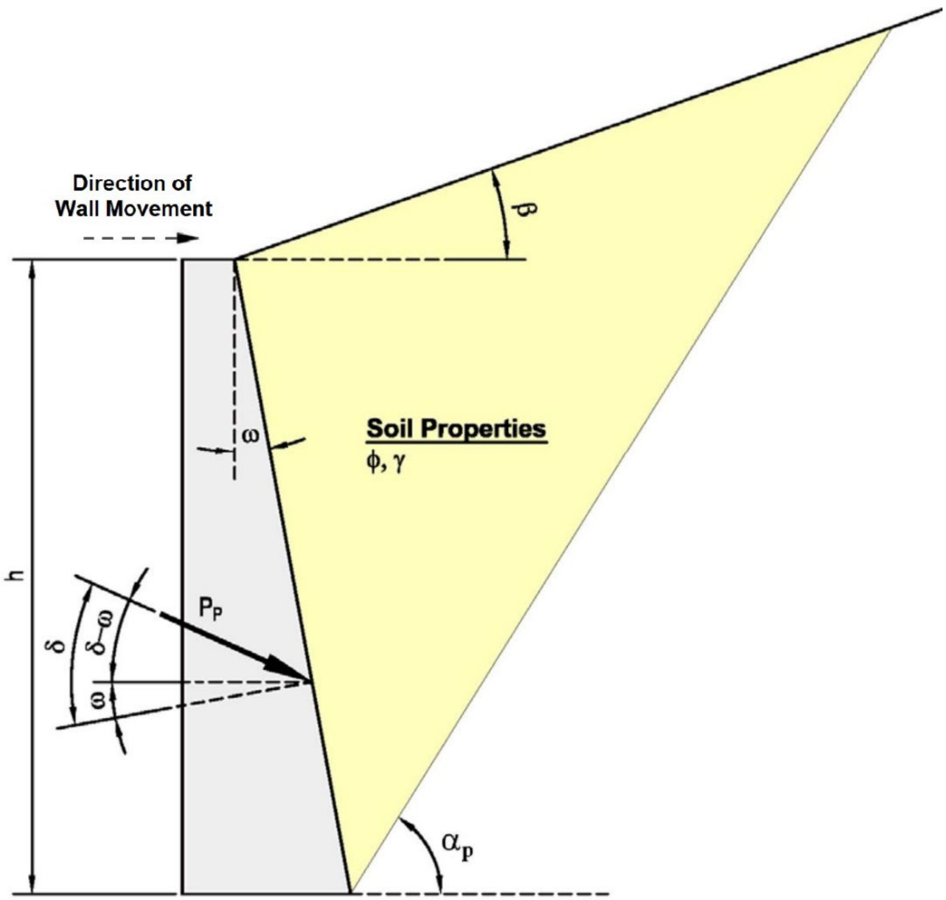


Figure 4-10. Coulomb's Passive Wedge

Where:

h	=	Height of pressure surface on the wall.
$\mathbf{P_a}$	=	Active lateral earth pressure resultant per unit width of wall.
$\mathbf{P_p}$	=	Passive lateral earth pressure resultant per unit width of wall.
δ	=	Friction angle between backfill material and face of wall.
β	=	Angle from backfill surface to the horizontal.
α	=	Failure plane angle with respect to the horizontal.
ω	=	Angle from the face of wall to the vertical.
ϕ	=	Effective friction angle of soil.
$\mathbf{K_a}$	=	Coefficient of active lateral earth pressure.
$\mathbf{K_p}$	=	Coefficient of passive lateral earth pressure.
γ	=	Unit weight of soil.

Table 4-2. Wall Friction

Ultimate Friction Factor for Dissimilar Materials	
INTERFACE MATERIALS	**Friction Angle, δ (°)**
Mass concrete on the following foundation materials:	
• Clean sound rock	35
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29
• Clean fine sand, silty or clayey fine to medium sand	19 to 24
• Fine sandy silt, nonplastic silt	17 to 19
• Very stiff and hard residual or preconsolidated clay................................	22 to 26
• Medium stiff and stiff clay and silty clay	17 to 19
Note: Masonry on foundation materials has similar friction factors.	
Steel sheet piles against the following soils:	
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls........	22
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17
• Silty sand, gravel or sand mixed with silt or clay.....................................	14
• Fine sandy silt, nonplastic silt	11
Formed or precast concrete or concrete sheet piling against the following soils:	
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls..........	22 to 26
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22
• Silty sand, gravel or sand mixed with silt or clay.....................................	17
• Fine sandy silt, nonplastic silt	14
Various structural materials:	
• Masonry on wood in direction of cross grain...	26
• Steel on steel at sheet pile interlocks..	17
• Masonry on masonry, igneous and metamorphic rocks:	
○ Dressed soft rock on dressed soft rock...	35
○ Dressed hard rock on dressed soft rock	33
○ Dressed hard rock on dressed hard rock	29

Note: This table is a reprint of Table C3.11.5.3-1, AASHTO LRFD BDS, 8th ed, 2017

Further discussion of wall friction is included in Section 4-6, *Log-Spiral Passive Earth Pressure.*

4-3.03 Summary of Earth Pressure Theories

There are various pros and cons to the individual earth theories when it comes to practical application of slope angles, friction angles, and failure planes. A summary for each theory is presented below:

- The Rankine formula for passive pressure can only be used correctly when the embankment slope angle, β, equals zero (i.e., level) or is negative. If a large wall friction value can develop, the Rankine theory is not correct and will give less conservative results. Rankine's theory is not intended to be used for determining earth pressures directly against a wall (friction angle does not appear in equations above). The theory is intended to be used for determining earth pressures on a vertical plane within a mass of soil, and therefore its use is to be avoided for passive earth pressure.
- For the Coulomb coefficient of passive earth pressure equation, if the shoring system is vertical and the wall friction angle is equal to zero degrees, the result will be the same as Rankine's for a level ground condition. Since wall friction requires a curved surface of sliding to satisfy equilibrium, the Coulomb formula will give only approximate results since it assumes planar failure surfaces. The accuracy for Coulomb will diminish with increased depth. For passive pressures, the Coulomb formula can also give inaccurate results when there is a large back slope or wall friction angle. Because of these limitations it is also recommended not to use Coulomb for the passive earth pressure.

The Log-Spiral theory was developed because of the unrealistic values of earth pressures that are obtained by theories that assume a straight-line failure plane. The difference between the Log-Spiral curved failure surface and the straight-line failure plane can be large and on the unsafe side for Coulomb passive pressures (especially when wall friction exceeds $\phi/3$). Figure 4-3 illustrates a comparison of the Coulomb and Rankine failure surfaces (planar) versus the Log-Spiral failure surface (curvilinear). More information on Log-Spiral theory can be found in in Section 4-6, *Log-Spiral Passive Earth Pressure,* of this manual.

4-4 Developing Earth Pressures for Cohesive Soil

Neither Coulomb's nor Rankine's theories explicitly incorporated the effect of cohesion in the lateral earth pressure computations. Bell (1952) modified Rankine's solution to include the effect of a backfill with cohesion. The derivation of Bell's equations for the active and passive earth pressure follows the same steps as were used in Section 4-3, *Developing Earth Pressures for Granular Soil*. The derivation is shown below.

For the cohesive soil, Figure 4-11 can be used to derive the relationship for the active and passive earth pressures.

Where: (for Equations 4-4-1 through 4-4-9)

- σ_v = Vertical stress.
- σ_a = Horizontal stress.
- **h** = Height of pressure back of wall.
- P_a = Active lateral earth pressure resultant per unit width of wall.
- P_p = Passive lateral earth pressure resultant per unit width of wall.
- ϕ = Effective friction angle of soil.
- **c** = Effective soil cohesion.
- K_a = Coefficient of active lateral earth pressure.
- K_p = Coefficient of passive lateral earth pressure.
- γ = Unit weight of soil.
- h_{cr} = Height of the tension crack

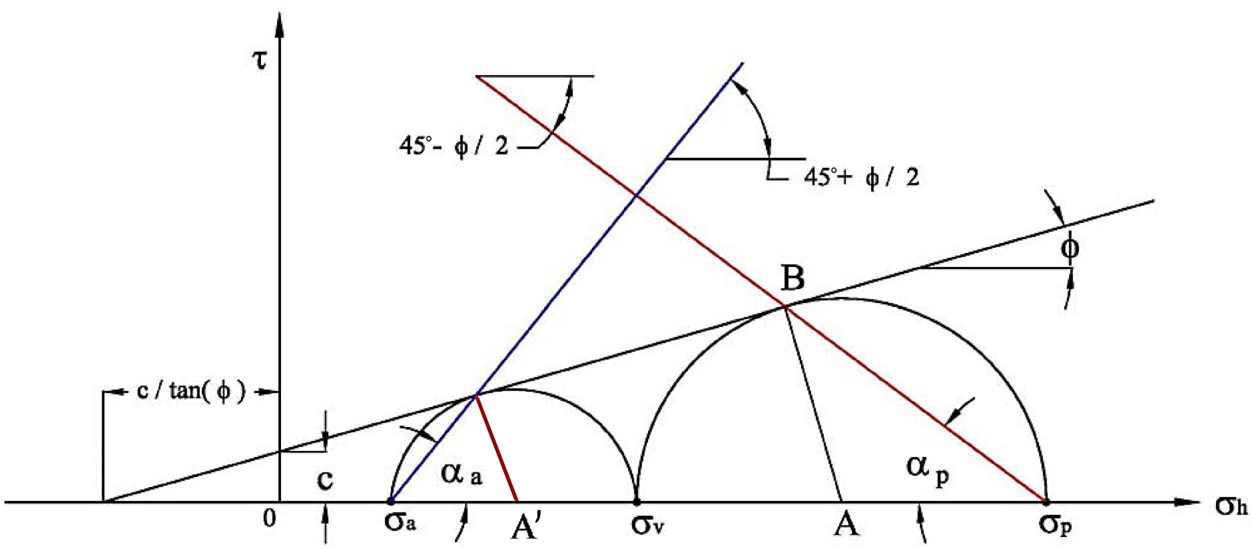


Figure 4-11. Mohr Circle Representation of Earth Pressure for Cohesive Backfill

For the active case:

$$\sin \phi = \frac{\frac{\sigma_v - \sigma_a}{2}}{\frac{\sigma_v + \sigma_a}{2} + \frac{c}{\tan \phi}} \tag{4-4-1}$$

Then,

$$\sigma_v \sin \phi + \sigma_a \sin \phi + 2c(\cos \phi) = \sigma_v - \sigma_a \tag{4-4-2}$$

Collecting terms:

$$\boldsymbol{\sigma_v(1-\sin\phi)=\sigma_a(1+\sin\phi)+2c(\cos\phi)} \qquad (4\text{-}4\text{-}3)$$

Solving for $\boldsymbol{\sigma_a}$:

$$\boldsymbol{\sigma_a=\frac{\sigma_v(1-\sin\phi)}{(1+\sin\phi)}-\frac{2c(\cos\phi)}{(1+\sin\phi)}} \qquad (4\text{-}4\text{-}4)$$

Using the trigonometric identities from above:

$$\boldsymbol{\sigma_a=\sigma_v\tan^2\left(45^\circ-\frac{\phi}{2}\right)-2c\tan(45^\circ-\phi/2)} \qquad (4\text{-}4\text{-}5)$$

$$\boldsymbol{\sigma_a=\sigma_vK_a-2c\sqrt{K_a},\quad \text{where}\quad \sigma_v=\gamma z} \qquad (4\text{-}4\text{-}6)$$

For the passive case:

Solving for $\boldsymbol{\sigma_p}$:

$$\boldsymbol{\sigma_p=\frac{\sigma_v(1+\sin\phi)}{(1-\sin\phi)}+\frac{2c(\cos\phi)}{(1-\sin\phi)}} \qquad (4\text{-}4\text{-}7)$$

$$\boldsymbol{\sigma_p=\sigma_v\tan^2\left(45^\circ+\frac{\phi}{2}\right)+2c\tan(45^\circ+\phi/2)} \qquad (4\text{-}4\text{-}8)$$

$$\boldsymbol{\sigma_p=\sigma_vK_p+2c\sqrt{K_p},\quad \text{where}\quad \sigma_v=\gamma z} \qquad (4\text{-}4\text{-}9)$$

Extreme caution is advised when using the cohesion value (**c**) to evaluate soil stresses. The evaluation of the stress induced by cohesive soils is highly uncertain due to the soil's 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. The development of the tension cracks from the surface to depth, $\mathbf{h_{cr}}$, is shown in Figure 4-12 and the depth of tension crack zone, $\mathbf{h_{cr}}$, can be estimated by Equation 4-4-13.

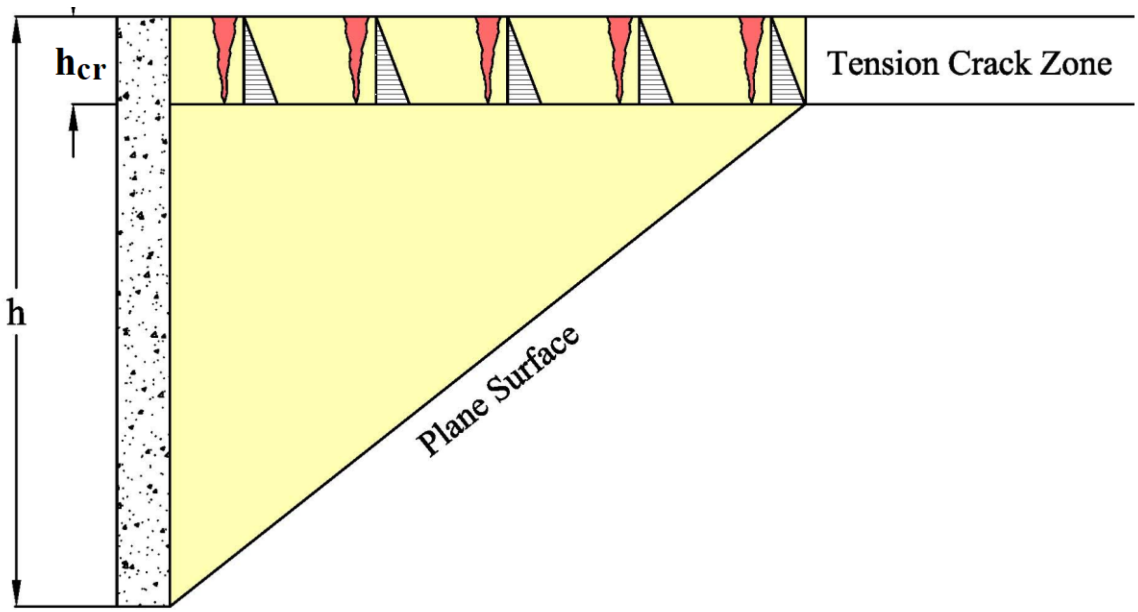


Figure 4-12. Tension Crack with Hydrostatic Water Pressure

As shown in Figure 4-13, the active earth pressure (**σ_a**) normal to the back of the wall at depth, **h**, is equal to:

$$\sigma_a = \gamma h K_a - 2c\sqrt{K_a} \qquad (4\text{-}4\text{-}10)$$

$$P_a = \frac{1}{2}\gamma h^2 K_a - 2c\sqrt{K_a}(h) \qquad (4\text{-}4\text{-}11)$$

According to Equation 4-4-10, the lateral stress (**σ_a**) at some point along the wall is equal to zero, therefore:

$$\gamma h K_a - 2c\sqrt{K_a} = 0 \qquad (4\text{-}4\text{-}12)$$

$$h = h_{cr} = \frac{2c\sqrt{K_a}}{\gamma K_a} \qquad (4\text{-}4\text{-}13)$$

As shown in Figure 4-13, the passive earth pressure (**σ_p**) normal to the back of the wall at depth, **h**, is equal to:

$$\sigma_p = \gamma h K_p + 2c\sqrt{K_p} \qquad (4\text{-}4\text{-}14)$$

$$P_p = \frac{1}{2}\gamma h^2 K_p + 2c\sqrt{K_p}(h) \qquad (4\text{-}4\text{-}15)$$

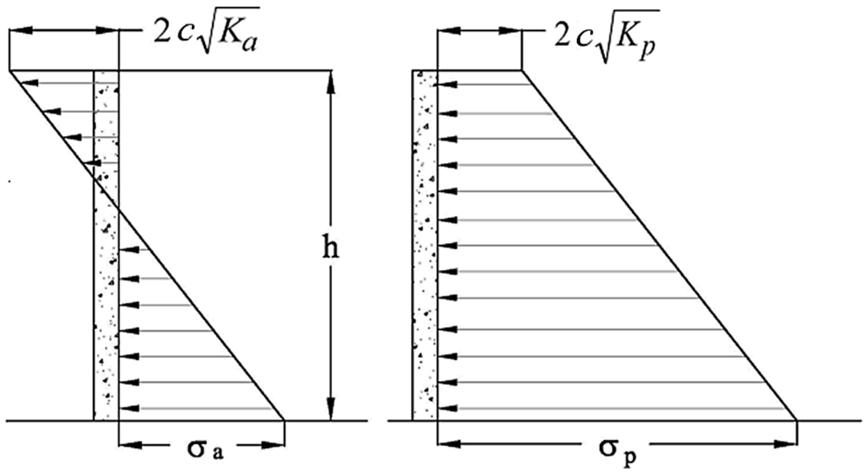


Figure 4-13. Cohesive Soil Active and Passive Earth Pressure Distribution

For shoring systems which support cohesive backfill, the height of the tension zone, $\mathbf{h_{cr}}$, should be ignored, as it is undeterminable when a crack develops, and the cohesive tension in the soil is no longer present. Thus, modified lateral earth pressure distribution acting along the entire wall height should be used (see Figure 4-14 below). The active lateral earth pressure ($\boldsymbol{\sigma_a}$) at the base of the pressure diagram may be used provided its value is not less than the minimum $\mathbf{K_a}$ value of 0.25, times the effective vertical stress ($\sigma_v = \gamma \mathbf{h}$). The vertical particle stress has not changed ($\sigma_v = \gamma \mathbf{h}$), however, because cohesion shifts the pressure diagram over to the left, there is a lower horizontal stress seen by the shoring. Thus, a revised earth pressure coefficient can be calculated and then used for the pressure at any intermediate depth. This new value is referred to as the "apparent active earth pressure coefficient", $\mathbf{K_{apparent}}$. As seen in Equation 4-4-16, $\mathbf{K_{apparent}}$ must be greater than or equal to the established minimum of 0.25.

Any design based on a $\mathbf{K_{apparent}}$ lower than 0.25 must have justification that could include multiple laboratory tests verifying higher values for "**c**", an explanation of the time frame for the excavation, and an explanation of other conditions that may affect the "**c**" value while the shoring is in place.

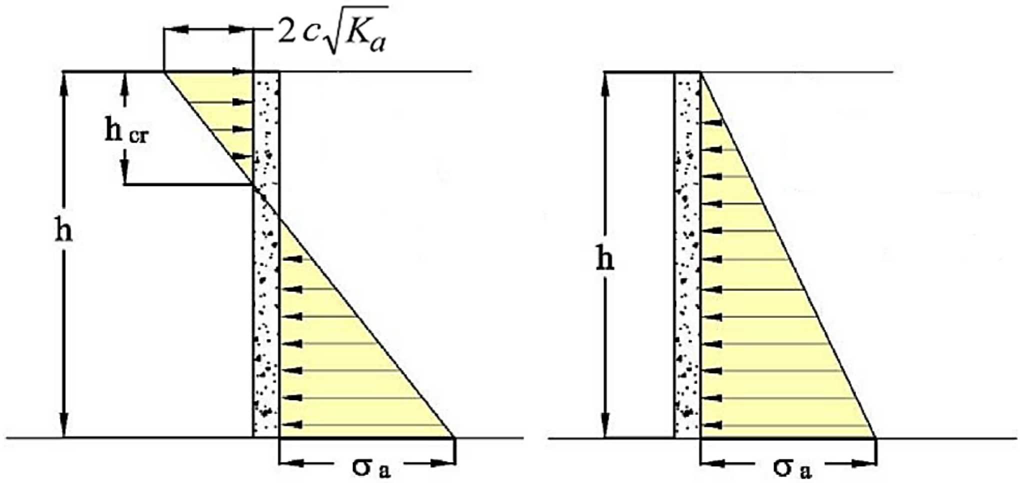


Figure 4-14. Pressure for K apparent

$$\mathbf{K_{apparent}} = \frac{\boldsymbol{\sigma_a}}{\boldsymbol{\gamma}\,\mathbf{h}} \geq \mathbf{0.25} \tag{4-4-16}$$

The effect of surcharges and ground water is not included in the above figure. When water is present, the tension crack will fill with water, and the hydrostatic pressure needs to be considered.

Including the presence of water pressure within the tension zone, the pressure diagram is shown below, in Figure 4-15.

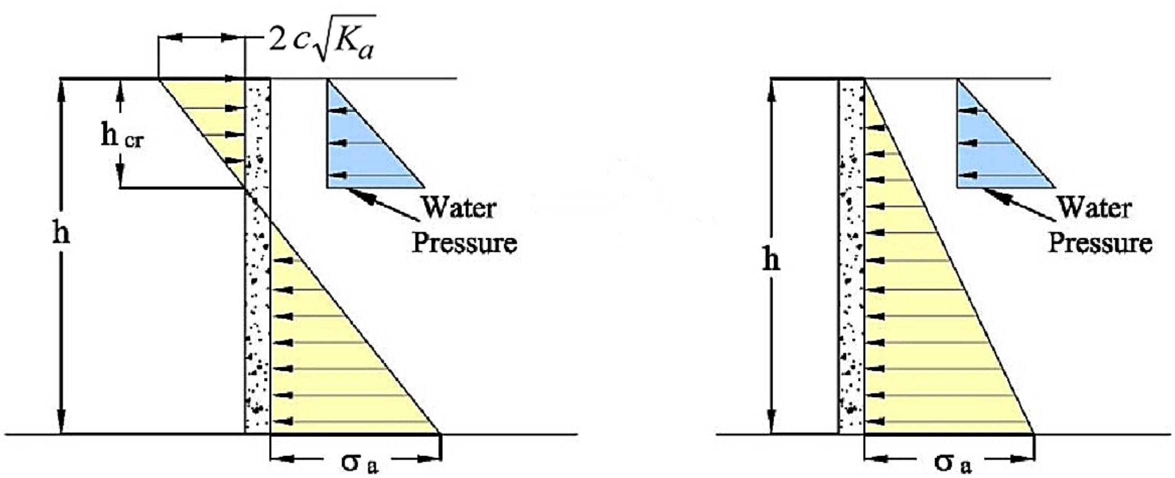


Figure 4-15. Pressure Diagram Depicting Hydrostatic Water Pressure in the Tension Crack Zone

4-5 Estimating Maximum Allowable Embankment Slope Angle

In nature, there are many stable slopes that have slope angles (β) that are larger than the angles of internal friction (ϕ). This is due to the presence of cohesion (**c**). None of the earth pressure theories will work when the slope angle β is larger than friction angle ϕ, even if the shoring system is to be installed in cohesive soil. Mohr Circle representation of the **C-ϕ** soil backfill, where the slope angle β is less than or equal to ϕ, is shown in Figure 4-16.

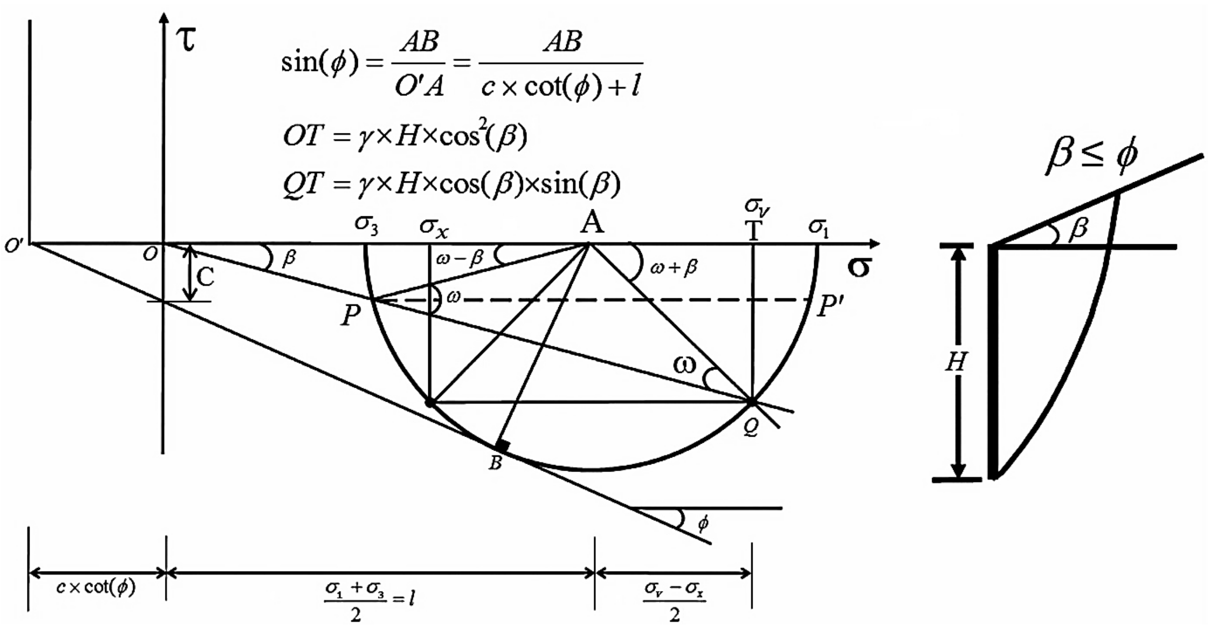


Figure 4-16. Mohr's Circles for Sloping Ground

The following equations are based on the *ASCE Journal of Geotechnical* and *Geoenvironmental Engineering* (February 1997) and are used to estimate the maximum allowable embankment slope angle for **C-ϕ** soil backfill.

$$\sin\ \beta \leq \sin\ \phi + \frac{c}{l}\cos\ \phi \qquad (4\text{-}5\text{-}1)$$

Where:

$$l = \frac{1}{\cos^2\ \phi}\left[\sigma_x + \frac{1}{2}\sin\ (2\phi) - \sqrt{\sigma_v(\cos^2\ \beta - \cos^2\ \phi) + \sigma_x[c\ \sin\ (2\phi)] + c^2\cos^2\ \phi}\right]$$

(4-5-2)

$$\boldsymbol{\sigma_V = \gamma(H\cos\beta)} \tag{4-5-3}$$

$$\boldsymbol{\sigma_x = \gamma(H\cos^2\beta)} \tag{4-5-4}$$

The following sections outline various methods for analyzing shoring systems that have sloping ground conditions.

4-5.01 Active Trial Wedge Method

Figure 4-17 shows the assumptions used to determine the resultant active pressure for sloping ground with an irregular backfill condition applying the wedge theory. This is an iterative process. The failure plane angle (**α_n**) for the wedge varies until the maximum value of the active earth pressure is computed using Equation 4-5-5. The development of Equation 4-5-5 is based on the limiting equilibrium for a general soil wedge. It is assumed that the soil wedge moves downward along the failure surface and along the wall surface to mobilize the active wedge. This wedge is held in equilibrium by the resultant force equal to the resultant active pressure (**P_a**) acting on the face of the wall. Since the wedge moves downward along the face of the wall, this force acts with an assumed wall friction angle (δ) below the normal to the wall to oppose this movement.

For any assumed failure surface defined by angle **α_n** from the horizontal and the length of the failure surface **L_c**, the magnitude of the wedge weight (**W_n**) is the weight of the soil wedge plus the relevant surcharge load. For any failure wedge, the maximum value of **P_a** can be determined using Equation 4-5-5.

$$\mathbf{P_a} = \frac{W_n[\tan(\alpha-\phi)] - C_oL_c[\sin\alpha\tan(\alpha-\phi)+\cos\alpha] - C_aL_a[\tan(\alpha-\phi)\cos(-\omega)+\sin\omega]}{[1+\tan(\delta+\omega)\tan(\alpha-\phi)]\cos(\delta+\omega)}$$

(4-5-5)

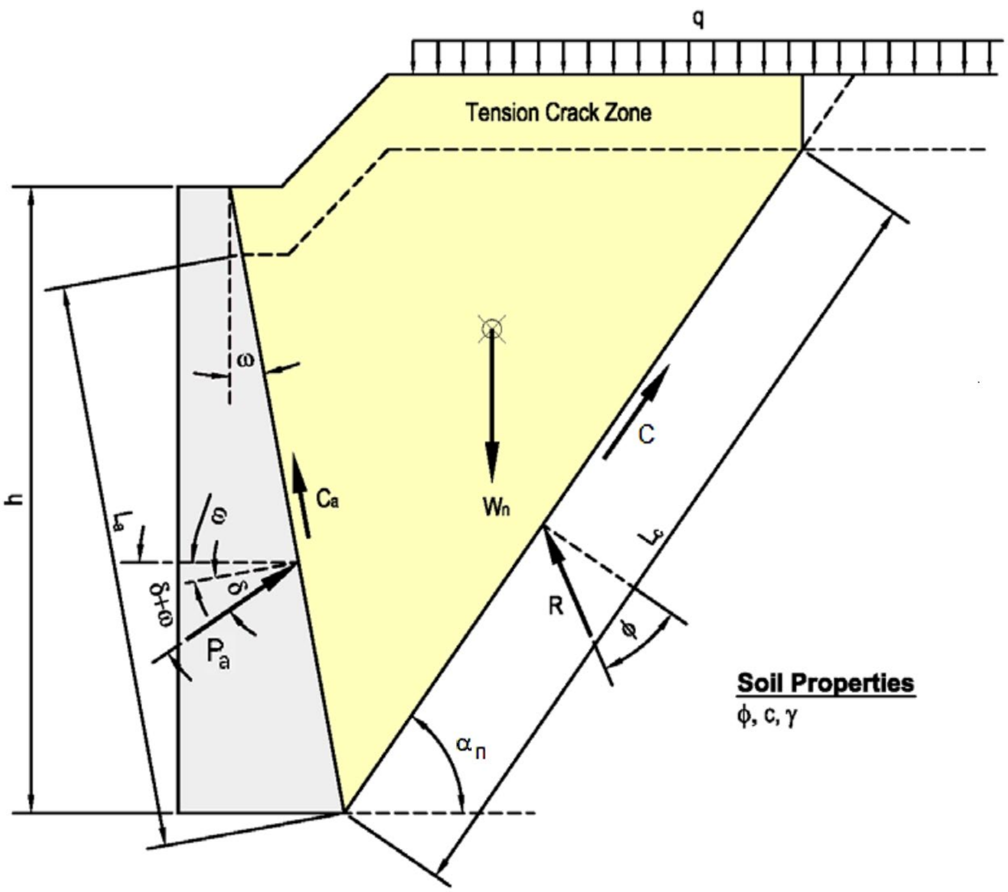


Figure 4-17. Active Trial Wedge

Where:

- **P_a** = Active lateral earth pressure resultant per unit width of wall.
- **W_n** = Weight of soil wedge plus the relevant surcharge loads.
- **δ** = Friction angle between backfill material and back of wall.
- **ϕ** = Effective friction angle of soil.
- **α_n** = Failure plane angle with respect to horizontal.
- **C** = Soil cohesion resultant force.
- **C_a** = Wall-backfill adhesion resultant force.
- **L_c** = Length of the failure plane on which cohesion acts.
- **L_a** = Length of the active wedge along the backwall on which adhesion acts.

Similar to wall friction for granular soils, adhesion is the resistance to slippage along the interface of a wall and a cohesive soil. The textbook, *Foundation Analysis and Design* (4th edition, 1988, by Joseph Bowles) discusses adhesion and places its value as 50 percent to 70 percent of the cohesion.

4-5.02 Passive Trial Wedge Method

Figure 4-18 shows the assumptions used to determine the resultant passive pressure for a broken back slope condition applying the trial wedge theory. Using the limiting equilibrium for a given wedge, Equation 4-5-6 calculates the passive earth pressure on a wall. The same iterative procedure is used as in the active case. However, the failure surface angle (α_n) is varied until the minimum value of passive pressure $\mathbf{P_p}$ is attained.

$$\mathbf{P_p} = \frac{W_n[\tan(\alpha+\phi)] + C_o L_c[\sin\alpha\tan(\alpha+\phi) + \cos\ \alpha] + C_a L_a[\tan(\alpha+\phi)\cos(-\omega) + \sin\ \omega]}{[1-\tan(\delta+\omega)\tan(\alpha+\phi)]\cos(\delta+\omega)}$$

(4-5-6)

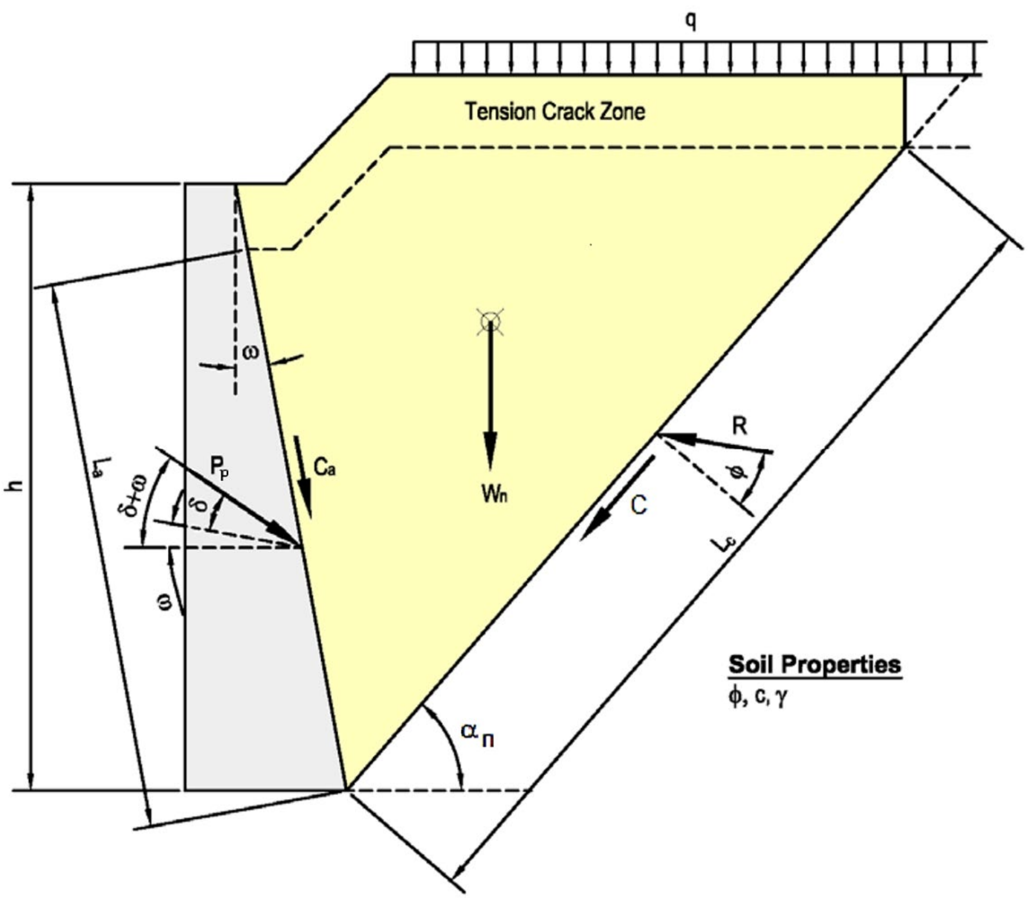


Figure 4-18. Passive Trial Wedge

Where:

- P_p = Passive lateral earth pressure resultant per unit width of wall.
- W_n = Weight of soil wedge plus the relevant surcharge loads.
- δ = Friction angle between backfill material and back of wall.
- ϕ = Effective friction angle of soil.
- α_n = Failure plane angle with respect to horizontal.
- C = Soil cohesion resultant force.
- C_a = Wall-backfill adhesion resultant force.
- L_c = Length of the failure plane on which cohesion acts.
- L_a = Length of the active wedge along the backwall on which adhesion acts.

4-5.03 Culmann's Graphical Solution for Active Earth Pressure

Culmann (1866) developed a convenient graphical solution procedure to calculate the active earth pressure for retaining walls for irregular backfill and surcharges. Figure 4-19 shows a failure wedge and a force polygon acting on the wedge. The forces per unit width of the wall to be considered for equilibrium of the wedge are as follows:

Where:

- c = Soil cohesion value
- K_a = Rankine active earth pressure coefficient
- ϕ = Soil friction angle (displayed as φ in some images)
- γ = Unit weight of soil

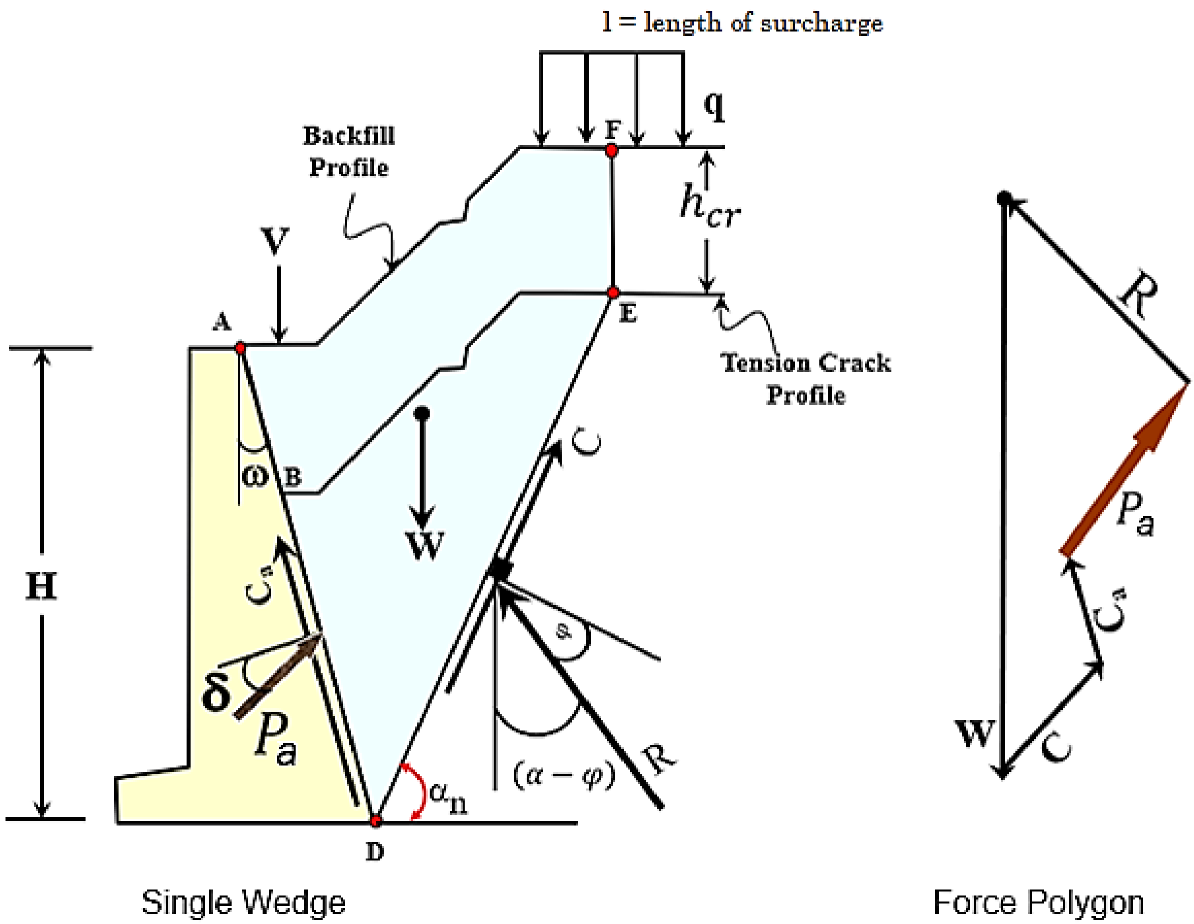


Figure 4-19. Single Wedge and Force Polygon

1. **W** = Weight of the wedge including weight of the tension crack zone and the surcharges with a known direction and magnitude.

$$\mathbf{W} = \mathbf{ABDEFA}_{\mathbf{area}}\ (\boldsymbol{\gamma}) + \mathbf{q}(\mathbf{1}) + \mathbf{V} \tag{4-5-7}$$

2. $\mathbf{C_a}$ = Adhesive force along the backfill of the wall with a known direction and magnitude.

$$\mathbf{C_a} = \mathbf{c}(\mathbf{BD}) \tag{4-5-8}$$

3. **C** = Cohesive force along the failure surface with a known direction and magnitude.

$$\mathbf{C} = \mathbf{c}(\mathbf{DE}) \tag{4-5-9}$$

4. **h_{cr}** = Height of the tension crack from Equation 4-4-13.

$$\mathbf{h_{cr}} = \frac{\mathbf{2c\sqrt{K_a}}}{\boldsymbol{\gamma}\mathbf{K_a}} \qquad (4\text{-}5\text{-}10)$$

5. **R** = Resultant of the shear and normal forces acting on the failure surface DE with the direction known only.

6. **P_a** = Active force of wedge with only the direction known.

To determine the maximum active force against a retaining wall, several trial wedges must be considered and the force polygons for all the wedges must be drawn to scale; this is illustrated in Appendix A, *Additional Theory For Inquiring Minds*, where further explanation of the Culmann's Graphical Solution for Active Earth Pressure is included.

4-6 Log-Spiral Passive Earth Pressure

Figure 4-3 from earlier in this chapter shows a simplified shoring system that has been sufficiently extended below the dredge line. The shoring system is stable when the active earth pressure developed on the high side of the wall is opposed by much higher passive earth pressure on the low side. It can be seen that the sliding surface for active earth pressure is practically a straight line, whereas a straight line cannot approximate the sliding surface for passive earth pressure. A detailed discussion on the nonlinear failure surfaces for passive pressures is included in Appendix A, *Additional Theory for Inquiring Minds*. A comparison between the "composite spiral-straight line" and the full Log-Spiral analysis is presented.

As mentioned in previous sections, Rankine's theory should not be used to calculate the passive earth pressure forces for a shoring system because it does not account for wall friction. While Coulomb's theory to determine the passive earth pressure force accounts for the angle of wall friction (δ), the theory assumes a linear failure surface. The result is an error in Coulomb's calculated force since the actual sliding surface is curved rather than planar. Coulomb's theory gives increasingly erroneous values of passive earth pressure as the wall friction (δ) increases. Therefore, Coulomb's theory could lead to unsafe shoring system designs because the calculated value of passive earth pressure would become higher than the soil could generate.

Terzaghi (1943) suggested that combining a logarithmic spiral and a straight line could represent the failure surface. Morrison and Ebeling (1995) suggested a single arc of the logarithmic spiral could realistically represent the failure surface.

Estimating the value of **K_p** for the shoring system in granular soils can be accomplished using the chart in Figure 4-20. This procedure requires that the values of δ, β, and ϕ are known. The failure surface is represented by a logarithmic spiral and a straight line. The procedure is shown below. For conditions that deviate from those described in Figure 4-20, the passive pressure may be calculated by using a trial procedure based on the trial wedge theory or a logarithmic spiral method.

1. Given δ, β, and ϕ.
2. Calculate ratios $^{\delta}/_{\phi}$ and $^{\beta}/_{\phi}$.
3. Determine initial $\mathbf{K_p}$ for $^{\beta}/_{\phi}$ from Figure 4-20.
4. Determine reduction factor **R** using the ratio of $^{\delta}/_{\phi}$.
5. Calculate final $\mathbf{K_p}' = \mathbf{R} \times \mathbf{K_p}$.

Example:

Wall Friction δ = 14° Slope above β = 27° Soil Friction ϕ = 32°

$^{\beta}/_{\phi}$ = 27/32 = 0.84

Enter the chart with the values of ϕ = 32° and $^{\beta}/_{\phi}$ = 0.84 to determine the initial $\mathbf{K_p}$.

Interpolating between the +0.8 and +1 lines, the initial value for $\mathbf{K_p}$ is approximately 19.

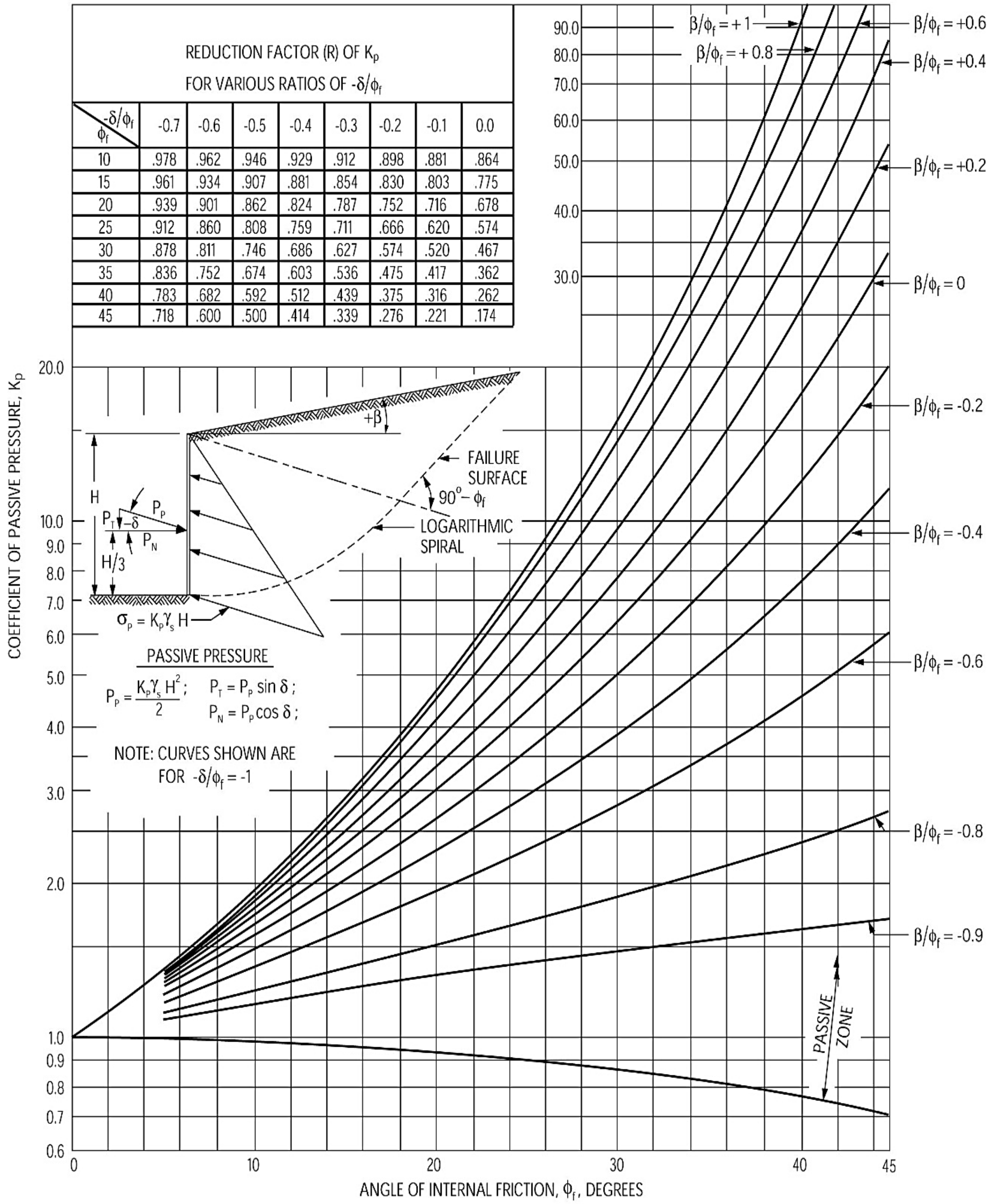
Use the table in the upper left portion of the chart to determine the reduction value, **R**, using $^{\delta}/_{\phi}$ = 0.44 and ϕ = 32°. Using interpolation, the value for **R** is 0.679.

	0.5	0.44	0.4
30	0.746	0.710	0.686
32	0.717	0.679	0.653
35	0.674	0.631	0.603

Matrix 4-1. Matrix for Interpolation of R

Multiply the initial value of $\mathbf{K_p}$ by the **R** value.

The $\mathbf{K_p}$ value used for your shoring check is $\mathbf{K_p}' = \mathbf{RK_p}$ = 0.679(19) = 13



REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

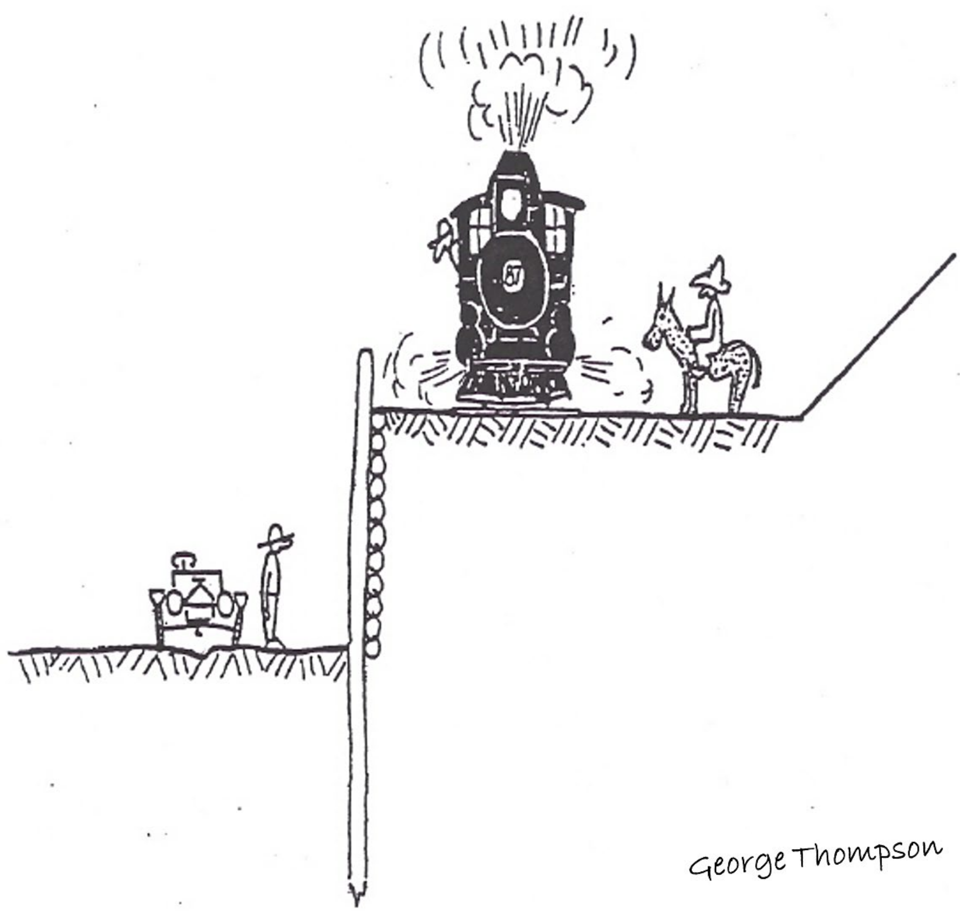
ϕ_f \ $-\delta/\phi_f$	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536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

Figure 4-20. Passive Earth Pressure Coefficient (Caquot and Kerisel, 1948)

JULY 2025

CHAPTER 5

SURCHARGES



Chapter 5: Surcharges

Table of Contents

Chapter 5: Surcharges 1
Table of Contents 1
5-1 Surcharge Loads 2
5-1.01 Minimum Construction Surcharge Load 2
5-1.02 Uniform Surcharge Loads 3
5-1.03 Boussinesq Loads 4
5-1.04 Traffic Loads 7
5-1.05 Alternate Surcharge Loading (Traffic) 12

5-1 Surcharge Loads

This chapter will present how to account for loads on the retained soil. There could be buildings, roadways, railroads, construction equipment, and/or materials adjacent to the excavation. There is a minimum surcharge that must always be applied. It is necessary for the Engineer to always consider a shoring system in relation to its surroundings as well as to the construction methods and equipment that will be implemented. Assumptions made with regards to these additional loads are best recorded on the shop drawings for the protective system.

A surcharge load is any load which is imposed upon the surface of the soil close enough to the excavation to cause a lateral pressure to act on the system in addition to the basic earth pressure. Groundwater will also cause an additional pressure, but it is not a surcharge load. Water is not classified as a surcharge load as it is a force acting against the sheets directly and not as a load acting on the soil retained by the shoring system.

5-1.01 Minimum Construction Surcharge Load

A minimum lateral construction surcharge of 72 psf ($\boldsymbol{\sigma_h}$) must be applied to the shoring system. The Cal/OSHA tables account for a similar surcharge. This load must be applied to a minimum depth of 10 feet (**Hs**) below the uppermost level of the soil retained by the shoring system, as illustrated in Figure 5-1. This is the minimum surcharge loading that must be applied to any shoring system regardless of whether or not the system is actually subjected to a surcharge load. Surcharge loads which produce lateral pressures greater than 72 psf would be used in lieu of this prescribed minimum.

This surcharge is intended to provide for the normal construction loads imposed by small vehicles, equipment, materials, and workers in the area adjacent to the trench or excavation and should be added to all basic earth pressure diagrams. This minimum surcharge can be compared to a soil having parameters of γ = 109 pcf and **Ka** = 0.33 for a depth of 2 feet [(0.33)(109)(2) = 72 psf].

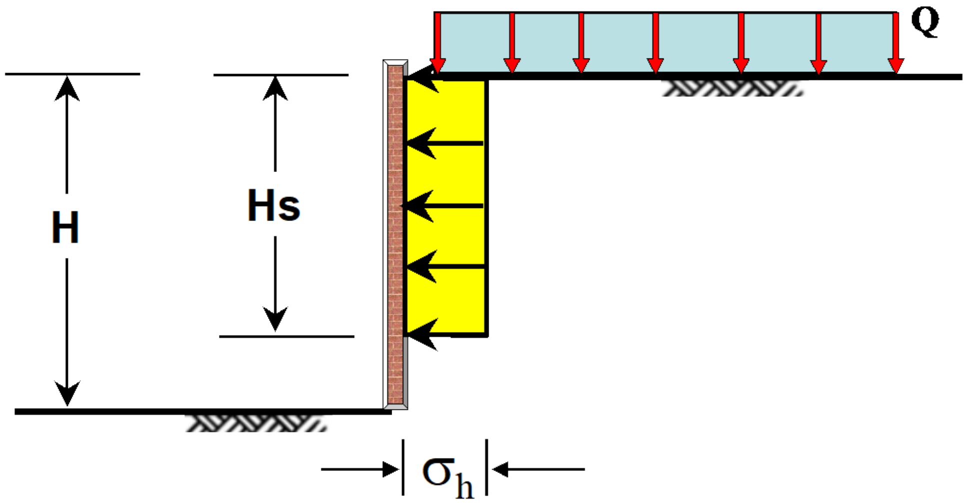


Figure 5-1. Minimum Lateral Surcharge Load

5-1.02 Uniform Surcharge Loads

Where a uniform surcharge is present, a constant horizontal earth pressure must be added to the basic lateral earth pressure. This constant earth pressure may be taken as:

$$\sigma_h = (K)(Q) \tag{5-1-1}$$

Where:

σ_h = Constant horizontal earth pressure due to uniform surcharge.

K = Coefficient of lateral earth pressure due to surcharge for the following conditions:

- Use **K_a** for active earth pressure.
- Use **K_o** for at-rest earth pressure.

Q = Uniform surcharge applied to the wall backfill surface within the limits of the active failure wedge.

5-1.03 Boussinesq Loads

Typically, there are three (3) types of Boussinesq Loads. They are as follows:

5-1.03A Strip Load

Strip loads are loads such as highways and railroads that are generally parallel to the wall.

The general equation for determining the horizontal pressure at distance, **h**, below the ground line may be referred to as the Wayne C. Teng equation (see Figure 5-2):

$$\boldsymbol{\sigma_h = \frac{2Q}{\pi}[\beta_R - \sin\beta\cos(2\alpha)]} \tag{5-1-2}$$

Where $\boldsymbol{\beta_R}$ is in radians.

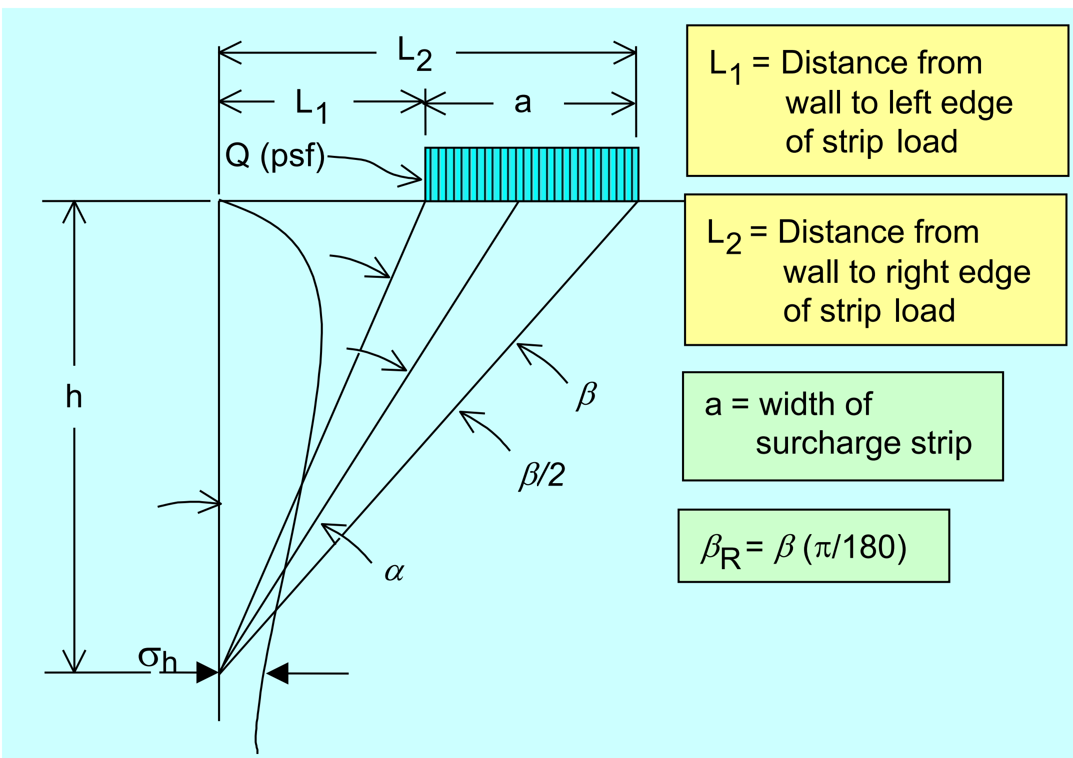


Figure 5-2. Boussinesq Type Strip Load

5-1.03B Line Load

A line load is a load such as a continuous wall footing of narrow width, or a similar load, generally parallel to the wall. K-railing could be considered to be a line load.

The general equation for determining the pressure at distance, **h** = **n·H**, below the ground line is (see Figure 5-3):

For **m** ≤ 0.4:

$$\boldsymbol{\sigma_h = \frac{Q_l}{H}\frac{0.2n}{(0.16+n^2)^2}} \quad (5\text{-}1\text{-}3)$$

For **m** > 0.4:

$$\boldsymbol{\sigma_h = 1.28\frac{Q_l}{H}\frac{m^2 n}{(m^2+n^2)^2}} \quad (5\text{-}1\text{-}4)$$

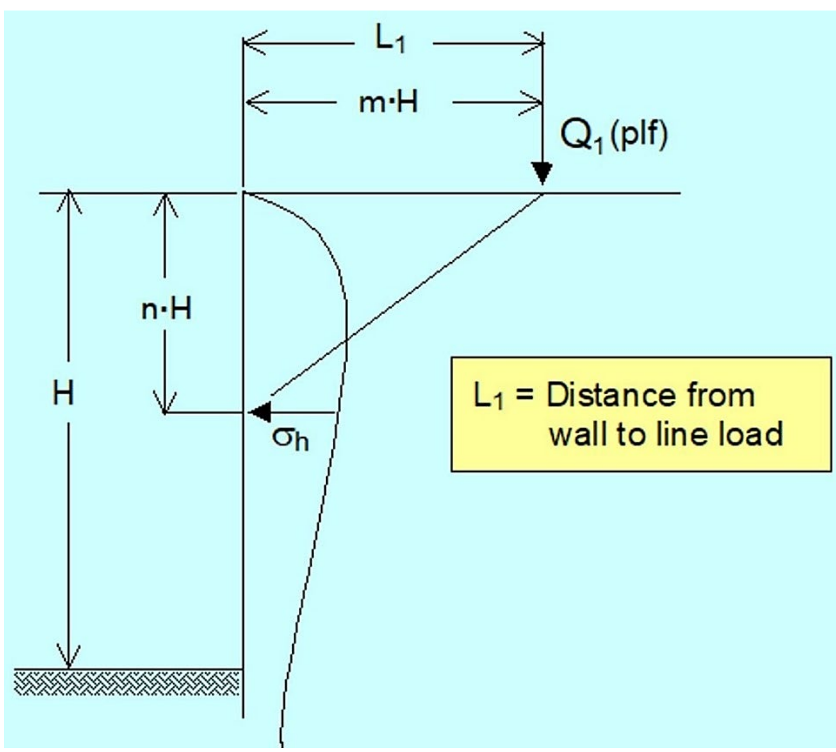


Figure 5-3. Boussinesq Type Line Load

5-1.03C Point Load

Point loads are loads such as outrigger loads from a concrete pump or crane. A wheel load from a concrete truck may also be considered a point load when the concrete truck is adjacent to an excavation and in the process of unloading. The truck could be positioned either parallel or perpendicular to the excavation.

The general equation for determining the horizontal pressure at distance, **h** = **n·H**, below the ground line is (see Figure 5-4):

For **m** ≤ **0.4**:

$$\boldsymbol{\sigma_h = 0.28\frac{Q_p}{H^2}\frac{n^2}{(0.16 + n^2)^3}} \qquad (5\text{-}1\text{-}5)$$

For **m > 0.4**:

$$\boldsymbol{\sigma_h = 1.77\frac{Q_p}{H^2}\frac{m^2n^2}{(m^2 + n^2)^3}} \qquad (5\text{-}1\text{-}6)$$

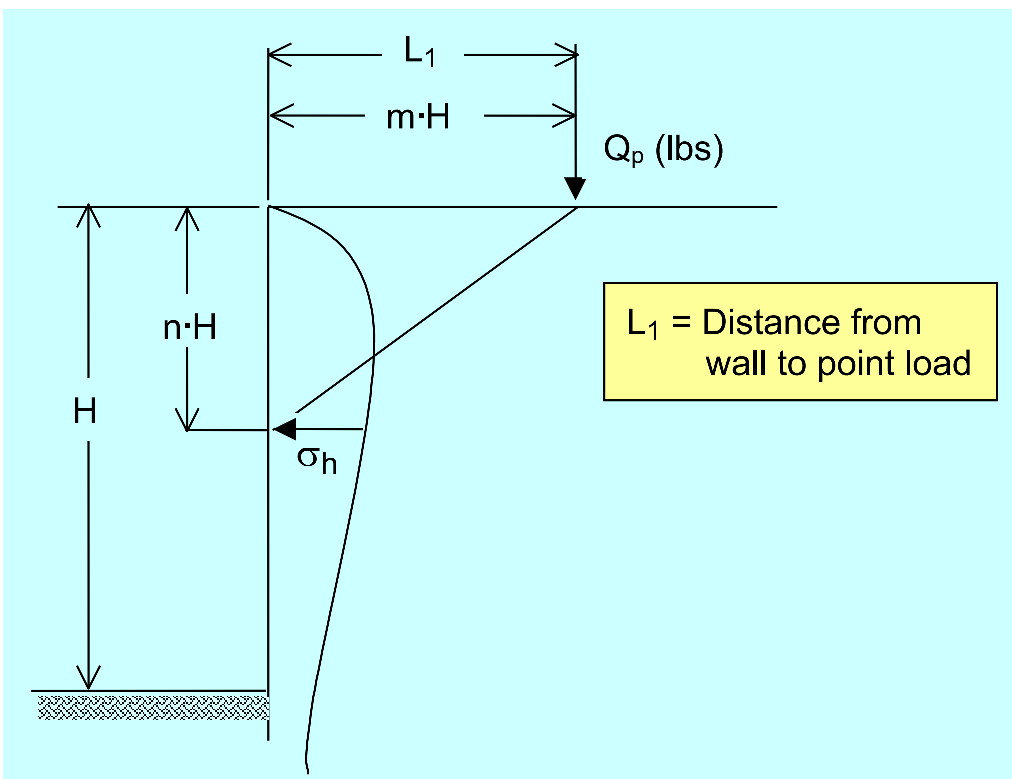


Figure 5-4. Boussinesq Type Point Load

In addition, σ_h is further adjusted by the following when the point is further away from the line closest to the point load (see Figure 5-5):

$$\sigma_h' = \sigma_h \cos^2[(1.1)\theta] \tag{5-1-7}$$

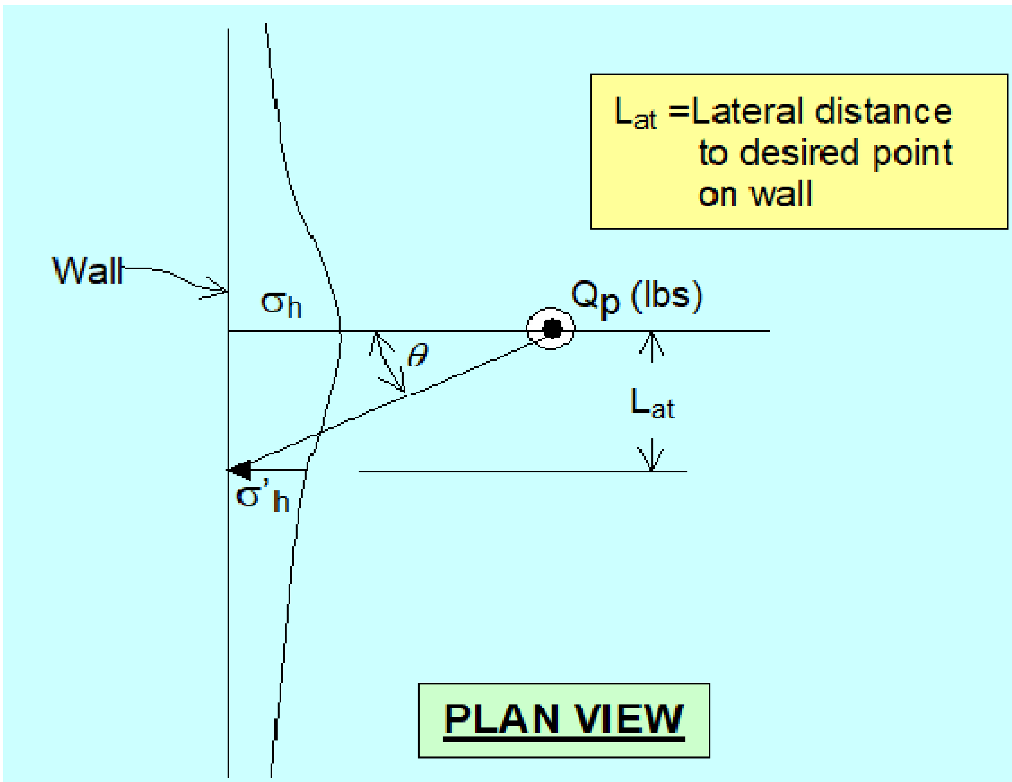


Figure 5-5. Boussinesq Type Point Load with Lateral Offset

5-1.04 Traffic Loads

Traffic near an excavation is one of the more commonly occurring surcharge loads. Trying to analyze every possible scenario would be time consuming and not very practical. For normal situations, a vertical surcharge load of 300 psf spread over the width of the traveled way should be sufficient.

The following example compares the pressure diagrams for a **Q** = 300 psf load (using the Boussinesq Strip method) and that of an HS-20 truck, using individual point loads from the tires centered in a 12-foot lane adjacent to the shoring, as illustrated in Figure 5-6. The total depth of excavation is 10 feet.

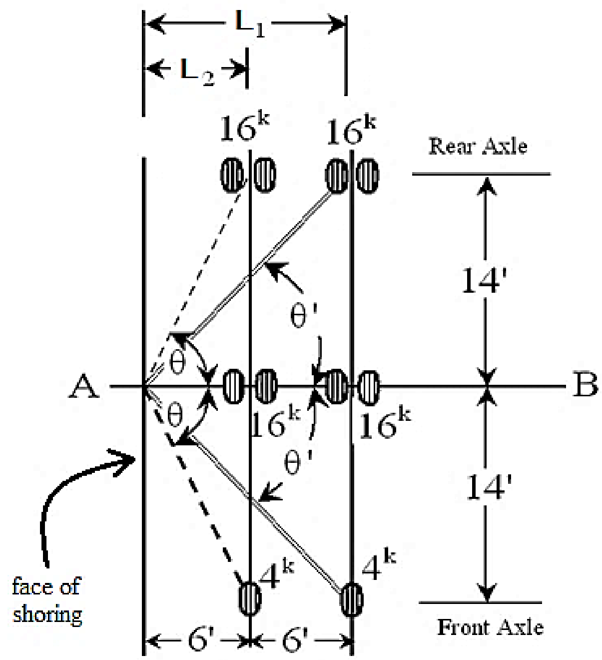


Figure 5-6. Example problem of HS-20 traffic loads adjacent to excavation

Solving to determine **n** for various depths (Table 5-1):

n = depth/H

Table 5-1. Values of 'n' for various depths

Depth	n
2'	0.2
4'	0.4
6'	0.6
8'	0.8
10'	1.0

$$\mathbf{L_1 = m_1 H \therefore m_1 = \frac{L_1}{H} = \frac{12}{10} = 1.2} \qquad (5\text{-}1\text{-}8)$$

$$\mathbf{L_2 = m_2 H \therefore m_2 = \frac{L_2}{H} = \frac{6}{10} = 0.6} \qquad (5\text{-}1\text{-}9)$$

Since **m** is greater than 0.4 in both cases, use Equation 5-1-6.

Since we are checking for the added load at the AB line, the adjustment for the horizontal angle must be calculated. For loads at an angle to AB, see Equation 5-1-7.

Horizontal adjustment:

Front and rear right wheels:

$$\theta = 66.8°, \therefore \cos^2[(1.1)(66.8°)] = 0.08 \qquad (5\text{-}1\text{-}10)$$

Front and rear left wheels:

$$\theta' = 49.4°, \therefore \cos^2[(1.1)(49.4°)] = 0.34 \qquad (5\text{-}1\text{-}11)$$

Now create the equations for each of the six-point loads:

Right rear wheels:

$$\sigma_h = \frac{(0.08)(1.77)(16{,}000)(0.6^2)(n^2)}{10^2(0.6^2 + n^2)^3} = 8.16n^2/(0.36 + n^2)^3 \qquad (5\text{-}1\text{-}12)$$

Left rear wheels:

$$\sigma_h = \frac{(0.34)(1.77)(16{,}000)(1.2^2)(n^2)}{10^2(1.2^2 + n^2)^3} = 138.7n^2/(1.44 + n^2)^3 \qquad (5\text{-}1\text{-}13)$$

Right center wheels:

$$\sigma_h = \frac{(1.77)(16{,}000)(0.6^2)(n^2)}{10^2(1.2^2 + n^2)^3} = 102.0n^2/(0.36 + n^2)^3 \qquad (5\text{-}1\text{-}14)$$

Left center wheels:

$$\sigma_h = \frac{(1.77)(16{,}000)(1.2^2)(n^2)}{10^2(1.2^2 + n^2)^3} = 407.8n^2/(1.44 + n^2)^3 \qquad (5\text{-}1\text{-}15)$$

Right front wheels:

$$\sigma_h = \frac{(0.08)(1.77)(4{,}000)(0.6^2)(n^2)}{10^2(0.6^2 + n^2)^3} = 2.04n^2/(0.36 + n^2)^3 \qquad (5\text{-}1\text{-}16)$$

Left front wheels:

$$\sigma_h = \frac{(0.34)(1.77)(4{,}000)(1.2^2)(n^2)}{10^2(1.2^2 + n^2)^3} = 34.7n^2/(1.44 + n^2)^3 \qquad (5\text{-}1\text{-}17)$$

Combine and simplify similar equations:

a) $$\sigma_H = \frac{8.16n^2}{(0.36 + n^2)^3} + \frac{102.0n^2}{(0.36 + n^2)^3} + \frac{2.04n^2}{(0.36 + n^2)^3} = \frac{(112.2)n^2}{(0.36 + n^2)^3}$$

$$\sigma_H = \frac{(112.2)(n^2)}{(0.36 + n^2)^3} \qquad (5\text{-}1\text{-}18)$$

$$\sigma_H = \frac{138.7n^2}{(1.44 + n^2)^3} + \frac{407.8n^2}{(1.44 + n^2)^3} + \frac{34.7n^2}{(1.44 + n^2)^3} = \frac{581.2n^2}{(1.44 + n^2)^3}$$

$$\sigma_H = \frac{(581.1)(n^2)}{(1.44 + n^2)^3} \qquad (5\text{-}1\text{-}19)$$

The following Table 5-2 will sum up the point load values at the 2-foot increments. The last column in the table uses the 300 psf surcharge tables in Appendix C, *Surcharges – Tabular Values*. The HS-20 truck was centered on the lane, thus the near edge of the lane is 3 feet away from the shoring. Hence the beginning of the 300 psf load is 3 feet away, and the far edge is 15 feet away. The resultant pressures at 2-foot increments are also illustrated in Figure 5-7.

Table 5-2. Summary of wheel point loads versus 300 psf traffic load

Depth (ft)	**n** from above	a) σ_H	b) σ_H	$\sum \sigma_H$	300 psf (from App. C)
0	0.0	0.0	0.0	0.0	0.0
2	0.2	70.1	7.2	77.3	150.1
4	0.4	127.7	22.7	150.4	171.5
6	0.6	108.2	35.9	144.1	149.3
8	0.8	71.8	41.3	113.1	121.4
10	1.0	44.6	40.0	84.6	96.4

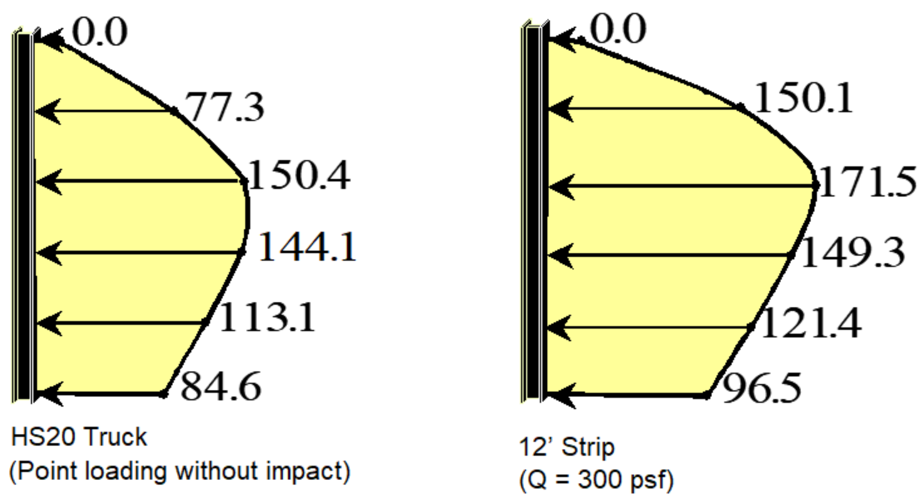


Figure 5-7. Plotted comparision of point loads versus 300 psf for traffic

Conclusion: Strip load of **Q** = 300 psf compares favorably to a point load evaluation for HS-20 truck loadings.

5-1.04A Example 5-1 Sample Problem – Surcharge Loads

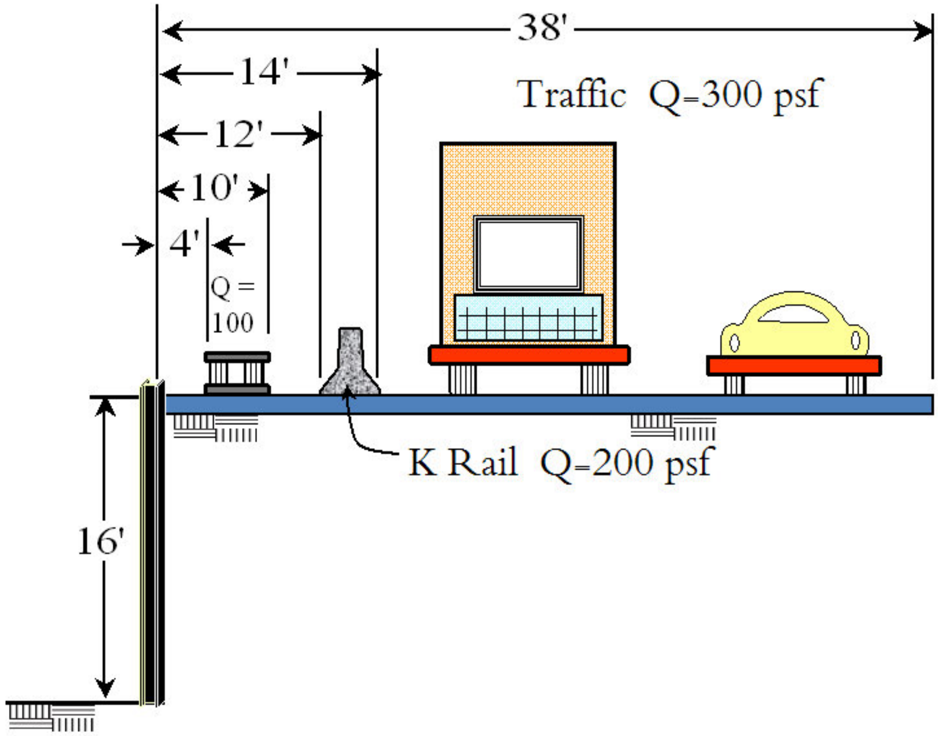


Figure 5-8. Surcharge Loads

Use the surcharge tables in Appendix C to determine the loading on the shoring at 2-foot increments for the three loadings illustrated above in Figure 5-8. The tables are set for a 300 psf surcharge; thus for the K-rail load of 200 psf, use 2/3 of the value in the table, and for the 100 psf load, use 1/3 of the tabulated value, as illustrated in Table 5-3.

Table 5-3. Surcharge Lateral Pressures (psf)

Depth (ft)	**Q** = 100	**Q** = 200	**Q** = 300	Sum
0.1	1.9	0.3	1.7	72*
2	30.2	5.8	33.8	72*
4	35.7	10.1	63.7	109.5
6	29.5	12.3	87.1	128.9
8	21.9	12.7	103.3	137.9
10	15.9	11.9	112.6	140.4
12	11.5	10.5	116.4	138.4
14	8.5	9.0	116.1	133.6
16	6.3	7.6	112.9	126.8

* Minimum construction surcharge load.

5-1.05 Alternate Surcharge Loading (Traffic)

An acceptable alternative to the Boussinesq analysis described above consists of imposing estimated surcharges behind the shoring system, such that the resulting pressure diagram is a rectangle extending to the computed depth of the shoring system and of a uniform width of 100 psf, as illustrated in Figure 5-9. Generally, alternative surcharge loadings are limited to traffic and light equipment surcharge loads. Other loadings due to structures, stockpiles of soil, materials, or heavy equipment will need to be considered separately.

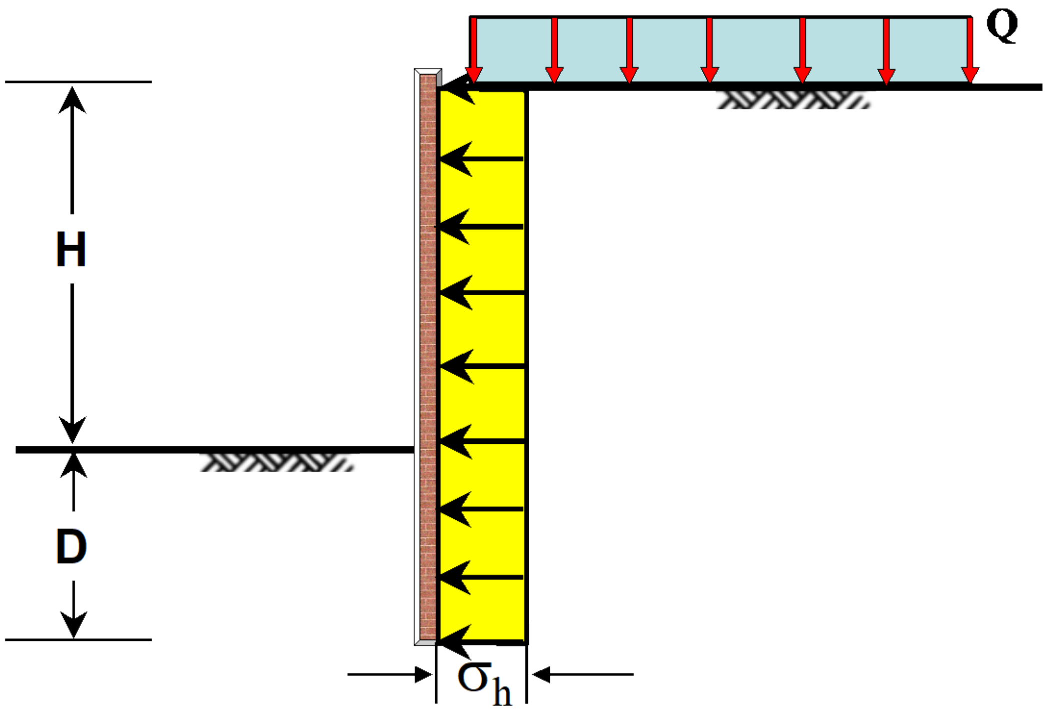
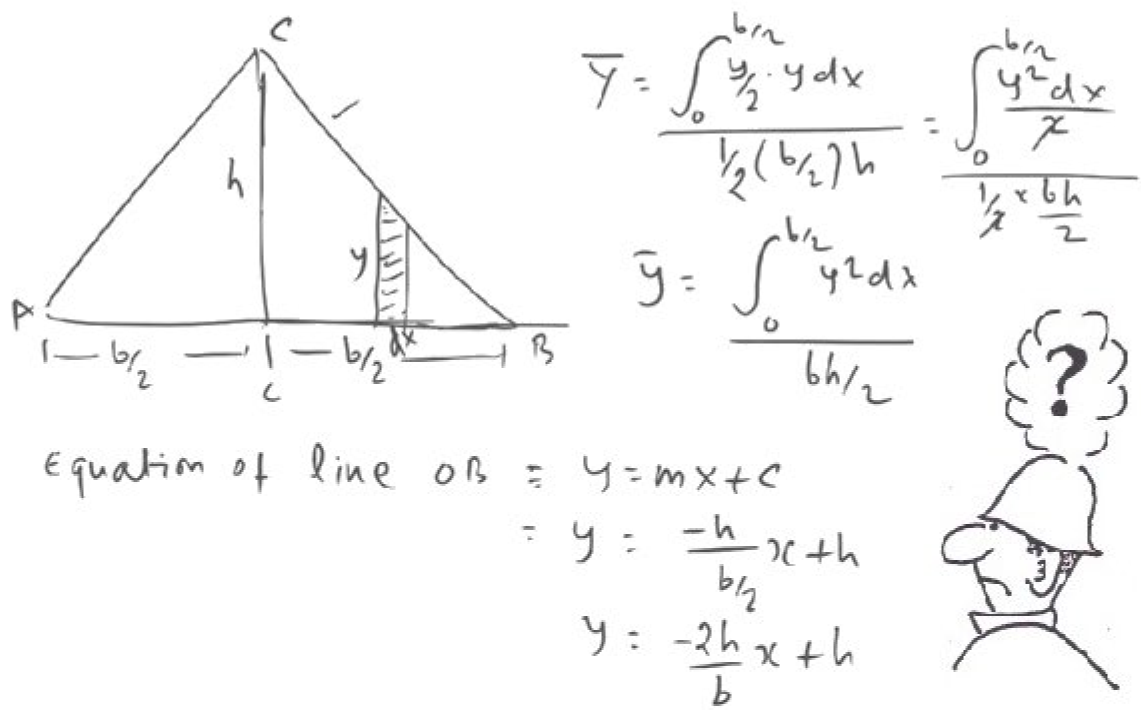


Figure 5-9. Alternate Traffic Surcharge Loading

JULY 2025

CHAPTER 6

STRUCTURAL DESIGN OF SHORING SYSTEMS



Chapter 6: Structural Design of Shoring Systems

Table of Contents

Chapter 6: Structural Design of Shoring Systems 1
Table of Contents 1
6-1 Introduction 2
6-2 Allowable Working Stresses 2
6-2.01 Timber 2
6-2.02 Steel 3
6-3 Mechanics of Stress Analysis 4
6-4 Overstress 4
6-5 Lagging 5
6-5.01 Example Lagging Calculations 8

6-1 Introduction

It is Structure Construction (SC) practice to review the trenching and shoring problems using Allowable Stress Design (ASD), as specified in the *Standard Specifications* for falsework design and the American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual for Railway Engineering.* The *Guidelines for Temporary Shoring*[1] published by Burlington Northern & Santa Fe Railway (BNSF) and Union Pacific Railroad (UPRR) is more readily available than the AREMA manual (note that Bridge Design has links to various Railroad guidelines in the *Bridge Design Processes and Procedures Manual* (BDPPM[1]) Section 5.1, *Railroad Overview).* This chapter summarizes the allowable values that the reviewer should use for timber and structural steel. For aluminum and concrete members, use the latest acceptable national standard. For timber connections, use the current *National Design Specification for Wood Construction (NDS)* printed by American Wood Council. Historically, these allowable values have provided shoring systems that are rigid and capable to support the earth pressures due to dry and/or saturated soils.

6-2 Allowable Working Stresses

6-2.01 Timber

For timber shoring analysis, reference *Falsework Manual,* Chapter 5*, Analysis*, for guidance on working with allowable working stress values.

Shoring adjacent to railroads is to be designed and reviewed in accordance with railroad requirements. Specific railroad requirements can be found in the railroad's *Guidelines for Temporary Shoring,* which is based primarily on the AREMA manual, Chapter 7, *Timber Structures*, Appendix 4, *Temporary Structures*. Chapter 9, *Railroads*, of this manual presents additional details on shoring systems involving railroads.

Appendix C, *Timber Shoring for Trenches*, of the Cal/OSHA Construction Safety Orders (CSO), § 1541.1, *Requirements for Protective Systems,* defines minimum timber member sizes to use in a shoring system consisting of uprights, walers, and cross bracing members for excavations not exceeding 20 feet in depth and only with the minimum surcharge. Member substitutions for shoring systems to be used in conjunction with the timber tables in Appendix C require they be manufactured members of equivalent strength. Some alternatives to timber shoring are also shown in Appendix E, *Alternatives to Timber Shoring*, of § 1541.1 of the Cal/OSHA CSO*.* Note that the Construction Safety Orders are found in the California Code of Regulations (CCR) Title 8, Division 1, Chapter 4, Subchapter 4.

[1] Caltrans internal use only

6-2.02 Steel

The maximum allowable stresses are based on the assumed use of structural steel conforming to the American Society for Testing and Materials (ASTM) Grade. Unless otherwise identified, assume the material to be ASTM Grade A36. Since in general, the load carrying capacity of steel beams will be limited by stress, not deflection, the use of higher strength steels may be beneficial. The maximum allowable stress for an identified steel grade must not exceed the limit specified in the current American Institute of Steel Construction (AISC) *Steel Construction Manual*.

When lagging is placed against the front flange of vertical shoring elements, such as wide flange (W) or HP sections, it may be assumed that the entire length of the beam is laterally supported for compression flange buckling (bending) due to the support provided by the lagging. This is the best practice for placement of lagging. If the lagging is placed on the back side of the soldier pile, the soldier pile will need to be checked for compression flange buckling. The allowable bending stress, $\mathbf{F_b}$, could be reduced significantly depending on if the structural section is determined as compact, non-compact, or partially compact as defined in the *AISC Steel Construction Manual*.

For determining allowable stresses for steel members (excluding sheet piles), use the ASD requirements of the AISC *Steel Construction Manual* specifications. Keep the following considerations in mind when analyzing steel components of shoring:

1. Since it is the Contractor’s responsibility to design the shoring, there are several standards for the grade of steel the Contractor can choose from to design the sheet piling for temporary shoring. Ensure the check of the Contractor’s temporary system adheres to the standards the Contractor is using.
2. For shoring members strictly in compression, determine the allowable axial compressive stress using the AISC *Steel Construction Manual* and assume a steel member with pinned ends.
3. Shoring systems with elements carrying large point loads should be reviewed for web yielding and web crippling. For additional details see Section 5-4.07, *Web Yielding,* and Section 5-4.08, *Web Crippling*, in the SC *Falsework Manual,* and Chapter J, *Design of Connections*, Section J10, *Flanges and Webs with Concentrated Forces,* of the AISC *Steel Construction Manual.*
4. For bolted connections use the most current version of the AISC *Steel Construction Manual*.

For additional details see Section 5-4.14, *Welding Steel Members*, in the SC Falsework Manual for a review of basic information of welded connections.

Railroads have different allowable stress requirements. See Chapter 9, *Railroad,* for information related to the AREMA *Manual for Railway Engineering* requirements and those specific to individual railroad companies.

6-3 Mechanics of Stress Analysis

Use the accepted structural mechanics formulas and theories to perform stress analysis. Review members of the shoring system for flexure, shear, compression, and bearing. Review the shoring system for overall stability; see Chapter 10, *Special Conditions*, of this manual. The *Falsework Manual* provides detailed information on stress analysis for temporary members. This information is applicable for the review of the timber and steel elements of the shoring system. The most relevant and useful sections include the following:

- Chapter 4, *Design Considerations*, Section 4-3, *Beam Continuity*,
- Chapter 5, *Analysis*, Section 5-2, *Timber Members*,
- Chapter 5, *Analysis*, Section 5-3, *Timber Fasteners*, and
- Chapter 5, *Analysis*, Section 5-4, *Steel Members*.

The material to follow is a brief overview, followed by an example checking the lagging for a soldier pile shoring system. See Table 6-1 for some basic structural mechanics formulas related to flexural stress and axial compression.

Table 6-1. Common Structural Mechanics Formulas

Topic	Formula	Variables
Flexural stress (bending)	$\mathbf{f_b = \frac{M}{S} \text{ or } \frac{Mc}{I}}$ (6-3-1)	**M** = Bending Moment **S** = Section Modulus **c** = distance from the neutral axis to extreme fiber **I** = moment of inertia of section about the neutral axis
Axial Compression	$\mathbf{f_c = \frac{P}{A}}$ (6-3-2)	**P** = Applied Load **A** = Area of Member

6-4 Overstress

Short term increases to allowable stresses are allowed (to a maximum of 133 percent) except in the following situations when:

1. Excavations are in place more than 90 days.
2. Dynamic loadings are present (pile driving, traffic, etc.).
3. Excavations are adjacent to railroads.
4. Analyzing horizontal struts.

6-5 Lagging

Lagging is placed between the flanges of either wide flange (W) or HP piles. The practice of installing lagging behind the back flange of the soldier piling is not recommended because the potential arching action of the supported soil will be destroyed. Also, the unsupported length of the compression flange of the beam will be affected. Lagging placed behind the front flange may be wedged back to provide tight soil to lagging contact. Voids behind lagging should be filled with compacted material. Lagging may be installed with a maximum spacing up to 1-1/2 inches between lagging members to permit seepage through the wall system. Movement of soil through the lagging spaces can be prevented by placing or packing straw, hay, or similar material into the spaces. Filter fabric behind the lagging members is usually used for permanent structures.

Construction grade lumber is the most common material used for lagging. Treated lumber is beneficial to use when it is expected that the lagging will remain in place for a long period of time, or permanently. If the use of treated lumber is proposed, check to see that it complies with your contract and permit requirements, especially in and near water sources. Treated lumber is typically Hem-Fir, not Douglas fir, and Hem-Fir has lower reference stress values.

The lagging bridges and retains soil between piles and transfers the lateral soil load to the soldier pile system. Due to the flexibility of the lagging and the soil arching capability, as shown in Figure 6-1, the soil pressure against the lagging may be reduced. For the arching effect to occur, the backside of the soldier pile must bear against the soil.

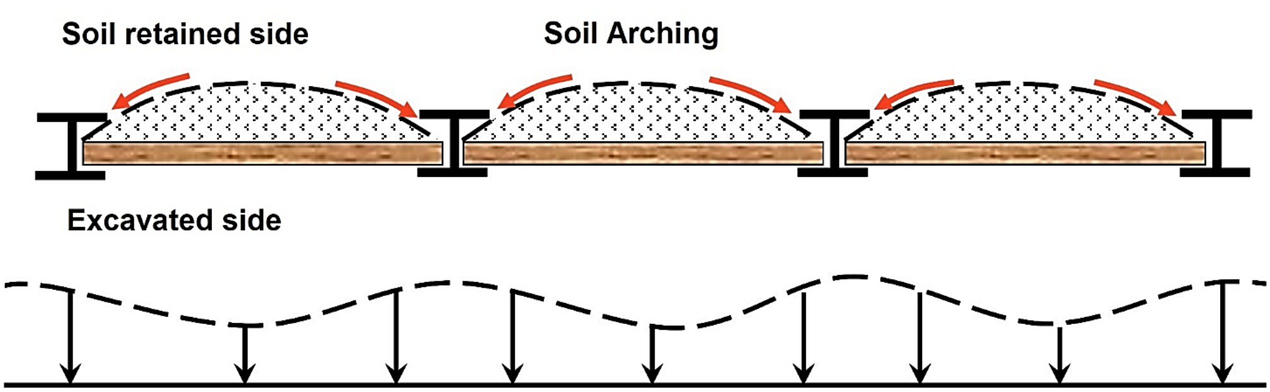


Figure 6-1. Soil Arching

Lateral soil movement within the failure wedge induces soil arching behind the lagging. This soil movement causes the lagging to flex outward. The arching process induces a redistribution of soil pressure away from the center of the lagging toward the much stiffer soldier pile support. Because of this, the design load on the lagging may be taken as 0.6 times the theoretical or calculated earth pressure. Studies have shown that a maximum lagging pressure of 400 psf should be expected when surcharges are not affecting the system. Without soil arching, the pressure redistribution would not occur, and reduced lagging loads should not be considered.

- Lagging design load = 0.6 (shoring design load).
- Maximum lagging load may be 400 psf when no surcharges are present.

The stress analysis of lagging will follow the procedures given in Section 5-2, *Timber Members,* of the *Falsework Manual*.

The Federal Highway Administration (FHWA) published recommended minimum timber thickness for construction grade rough cut Douglas fir. Table 6-2, below, lists lagging recommendations for the following soil classification groupings:

1. Competent Soils: These soils include high internal friction angle sand or granular material or stiff to very stiff clays.
2. Difficult Soils: These soils consist of loose to low internal friction angle cohesionless material, silty sands, and over consolidated clays which may expand laterally, especially in deep excavations.
3. Potentially Dangerous Soils: These soils consist of soft clays, silts below the water table, and clayey sands below the water table. For these soils, the appropriateness of lagging is questionable.

The tabular values may be used for lagging where soil arching behind the lagging can develop. Tabular values should not be used for excavations adjacent to existing facilities, including railroads. Analyze the lagging used in conjunction with surcharges separately.

Table 6-2. FHWA Recommended Minimum Timber Thickness

RECOMMENDED THICKNESS OF ROUGH CUT WOOD LAGGING WHEN SOIL ARCHING WILL BE DEVELOPED* (FOR LOCATIONS WITHOUT SURCHARGE LOADINGS)								
Soil Description	Unified Soil Classification System Group Symbol	Depth	for clear spans of:					
			5′	6′	7′	8′	9′	10′
COMPETENT SOILS								
Silts or fine sand and silt above 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 $\gamma H/C < 5$.	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 over-consolidated 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 (appropriateness of lagging is questionable)								
Soft clays $\gamma H/C > 5$.	CL, CH	0′ to 15′	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″			

*Adapted and revised from the April 1976 Federal Highway Administration Report No. FHWA-RD-130.

6-5.01 Example Lagging Calculations

Check the proposed lagging for adequacy.

Given:

- Soldier Pile spacing is 8.5 feet on-center, with a clear-span of 7.5 feet.
- The lagging proposed is 4 x 12 DF #2, (8 feet long members), spaced at 12 inches on center.
- A surcharge is present from an adjacent haul road. The maximum combined soil and surcharge load on the lagging is 580 psf.

Solution:

Step 1: First determine the allowable stresses for shear, bending, and compression perpendicular to the grain per NDS (2018).

Calculate allowable horizontal shear stress:

Reference design value in shear $\mathbf{F_v}$ = 180 psi (NDS Supplement Table 4A)

Applicable adjustment factors from NDS Table 4.3.1:

$\mathbf{C_D}$	= 1.15	Duration Factor (less than two months) NDS Table 2.3.2
$\mathbf{C_M}$	= 0.97	Wet Service Factor NDS Supplement Table 4A (Assume > 19% moisture content)
$\mathbf{C_t}$	= 1.0	Temperature Factor NDS Table 2.3.3 (Temp up to 100ºF)
$\mathbf{C_i}$	= 1.0	Incising Factor NDS Table 4.3.8

Adjusted design value:

$$\mathbf{F_v'} = \mathbf{F_v}(\mathbf{C_D})(\mathbf{C_M})(\mathbf{C_t})(\mathbf{C_i}) = \mathbf{201psi} \tag{6-5-1}$$

Calculate allowable compression stress perpendicular to grain:

Reference design value in compression perpendicular to grain, $\mathbf{F_{c\perp}}$ = 625 psi (NDS Supplement Table 4A)

Adjustment factors from NDS Table 4.3.1:

$\mathbf{C_M}$	= 0.67	Wet Service Factor NDS Supplement Table 4A (Assume > 19% moisture content)
$\mathbf{C_t}$	= 1.0	Temperature Factor NDS Table 2.3.3 (Temp up to 100ºF)
$\mathbf{C_i}$	= 1.0	Incising Factor NDS Table 4.3.8
$\mathbf{C_b}$	= 1.0	Bearing Area Factor NDS 3.10.4

Adjusted design value:

$$\mathbf{F_{c_\perp}}' = \mathbf{F_{c_\perp}}(\mathbf{C_M})(\mathbf{C_t})(\mathbf{C_i})(\boldsymbol{C_b}) = \mathbf{419\ psi} \tag{6-5-2}$$

Calculate allowable bending stress:

Reference design value in bending $\mathbf{F_b}$ = 900 psi for DF #2 (NDS Supplement Table 4A)

Applicable adjustment factors from NDS Table 4.3.1:

$\mathbf{C_D}$	= 1.15	Duration Factor (less than two months; do *not* use Construction load duration for lagging)
$\mathbf{C_M}$	= 1.0	Wet Service Factor NDS Supplement Table 4A (Assume > 19**%** moisture content $(F_b)(C_F) \le 1150$ psi)
$\mathbf{C_t}$	= 1.0	Temperature Factor NDS Table 2.3.3 (Temp up to 100ºF)
$\mathbf{C_L}$	= 1.0	Beam Stability Factor NDS 4.4.1
$\mathbf{C_F}$	= 1.1	Size Factor NDS Supplement Table 4A
$\mathbf{C_{fu}}$	= 1.1	Flat Use Factor NDS Supplement Table 4A (note orientation of lagging)
$\mathbf{C_i}$	= 1.0	Incising Factor NDS Table 4.3.8
$\mathbf{C_r}$	= 1.0	Repetitive Member Factor NDS 4.3.9 (no load distributing element)

Adjusted design value:

$$\mathbf{F_b}' = \mathbf{F_b}\,(\mathbf{C_D})(\mathbf{C_M})(\mathbf{C_L})(\mathbf{C_t})(\mathbf{C_F})(\mathbf{C_i})(\mathbf{C_{fu}})(\mathbf{C_r}) = \mathbf{1252\ psi} \qquad (6\text{-}5\text{-}3)$$

<u>Step 2: Next perform the lagging stress calculations based on the variables below:</u>

- $\mathbf{A_b}$ = l_b x w_b is the Bearing Area at the lagging ends for compression perpendicular to the grain
- $\mathbf{w_b}$ = 11.25 inches is the Bearing Width
- $\mathbf{l_b}$ = 3 inches is the Bearing Length (lagging centered over clear span)
- $\mathbf{L_{clr}}$ = is the clear span for wood lagging elements
- **w** = is the soil pressure on the lagging
- **V** = is the shear load

Calculate the uniform load, *w*, on the lagging, including soil and surcharge loads:

$$\mathbf{w} = \mathbf{f} \times \mathbf{W_L} = \mathbf{0.6(580)} \approx \mathbf{348\ psf} \qquad (6\text{-}5\text{-}4)$$

Where 0.6 is the soil lagging arching factor discussed above.

Determine the lagging span length $\mathbf{L_{clr}}$:

Pile spacing is 8.5 feet and the pile width is 12 inches. $\mathbf{L_{Clear}}$ = 7.5 feet

Calculate the shear value, V in the lagging:

$$V = \frac{w \times L_{clear}}{2} = \frac{348 \text{ psf}(1 \text{ ft})\ 7.5 \text{ ft}}{2} = 1{,}305 \text{ lbs} \quad (6\text{-}5\text{-}5)$$

Calculate horizontal shear stress:

V = 1,305 lbs

$$f_v = \frac{3V}{2A} = \frac{3(1305)}{2(3.5)(11.25)} \approx 50 \text{ psi} \quad (6\text{-}5\text{-}6)$$

50 psi ≤ 201 psi, which is the **F_v′** Allowable calculated above (6-5-1). **OK**

Calculate the compression stress perpendicular to grain:

Use equation to check the bearing (**$f_{c\perp}$**).

$$f_{c\perp} = \frac{V}{l_b \times w_b} \quad (6\text{-}5\text{-}7)$$

$$f_{c\perp} = \frac{1305\text{lb}}{3\text{in} \times 11.25\text{in}} \approx 39 \text{ psi} \quad (6\text{-}5\text{-}8)$$

39 psi ≤ 419psi, which is the **$F_{c\perp}$′** Allowable calculated above (6-5-2).

Calculate bending stress:

First determine the actual bearing length, **L**, required by adding the required bearing length to **L_{clr}.**

Rearrange Equation 6-5-7 above and modify to include the allowable compression stress perpendicular to grain, to solve for **l_b**:

$$l_b = \frac{V}{(F_{c_\perp}{}')(w_b)} \quad (6\text{-}5\text{-}9)$$

$$l_b = \frac{1305}{(419)(11.25)} \approx 0.28 \text{ in} \quad (6\text{-}5\text{-}10)$$

This is less than the 3 inches assumed, so our assumption was OK.

Thus, the span length is:

$$L = 7.5' + \frac{0.28}{12} = 7.52 \text{ ft} \tag{6-5-11}$$

Calculate the moment in the lagging (recall it is 1 foot on center), the lagging section modulus, and finally the bending stress within the lagging (note the lagging orientation when calculating the section modulus):

$$M = \frac{wL^2}{8} = \frac{(348)(1 \text{ ft})(7.52)^2}{8} = 2460 \text{ ft-lb} \tag{6-5-12}$$

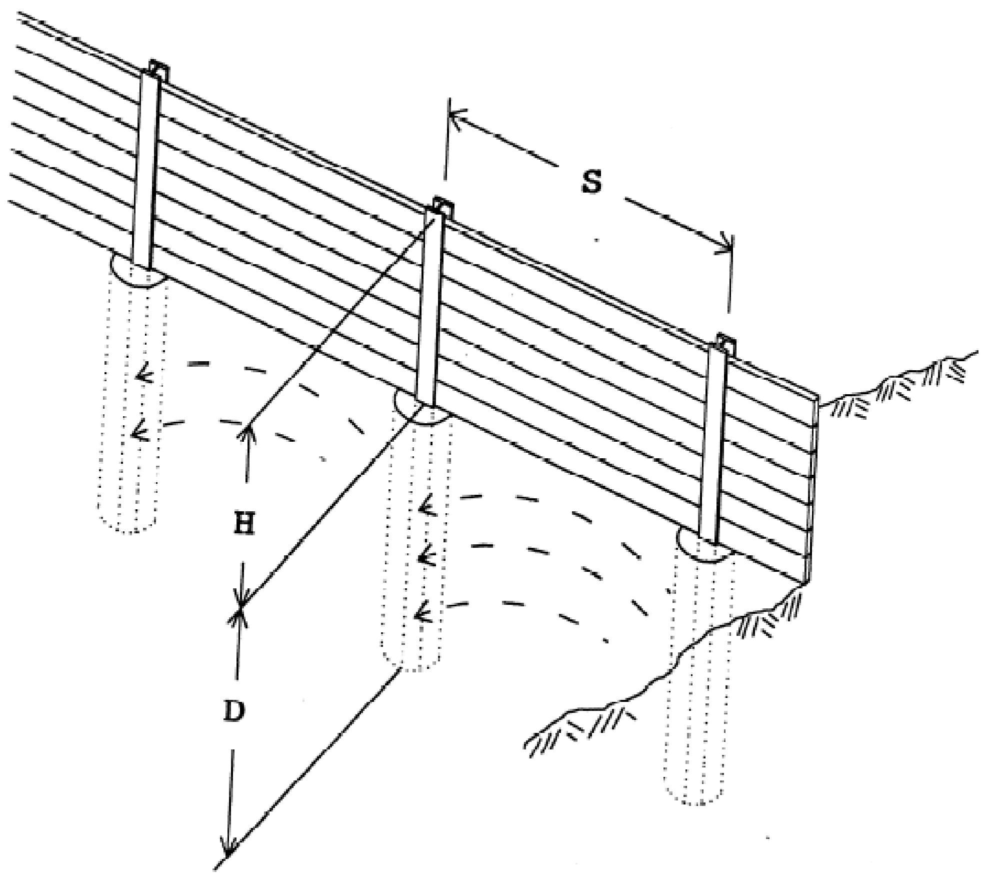
$$S = \frac{bh^2}{6} = \frac{(11.25)(3.5)^2}{6} = 23.0 \text{ in}^3 \tag{6-5-13}$$

$$f_b = \frac{M}{S} = \frac{2460(12)}{23.0} \approx 1283 \text{ psi} \tag{6-5-14}$$

1283 psi > 1252 psi, which is the $\mathbf{F_b'}$ allowable calculated above (Eq. 6-5-3). The lagging fails to meet the bending stress criteria; therefore the Contractor needs to resubmit a new lagging proposal.

CHAPTER 7

UNRESTRAINED SHORING SYSTEMS



Chapter 7: Unrestrained Shoring Systems

Table of Contents

Chapter 7: Unrestrained Shoring Systems 1

Table of Contents 1

7-1 Types of Unrestrained Shoring Systems 2

7-2 Effective Pile Width 3

7-3 System Deflection 5

7-4 Soil Pressure Distribution for Layered Soil 7

7-5 Lateral Earth Pressures for Unrestrained Shoring Systems 8

7-5.01 The Rigorous Method 9

7-5.02 The Simplified Method 11

7-5.03 Variations of Pressure Diagrams for Other Soil Configurations 12

7-5.04 Example 7-1A: Cantilevered Soldier Pile Wall by Rigorous Method ... 17

7-5.05 Example 7-1B: Cantilevered Soldier Pile Wall by Simplified Method .. 26

7-5.06 Example 7-2: Cantilevered Soldier Pile Wall, Simplified Method with Two Soil Layers above the Excavation 32

7-5.07 Example 7-3: Cantilevered Soldier Pile Wall, Simplified Method 35

7-1 Types of Unrestrained Shoring Systems

Unrestrained shoring systems or non-gravity cantilevered walls (hereafter simply cantilevered) are constructed of vertical structural members consisting of partially embedded continuous sheet piles or soldier piles. Continuous sheet pile retaining walls may be constructed with steel sheet piles with interlocking edges, driven precast pre-stressed concrete sheet piles, or other materials. The sheet piles are driven side by side into the ground and form a continuous vertical wall. Because of the large deflections that may develop, cantilever sheet pile retaining walls are mainly used for temporary excavations not greater than about 18 feet. However, the use of struts and/or walers can increase the wall height, making it a restrained system; see Chapter 8, *Restrained Shoring Systems,* of this manual. Figure 7-1 shows a typical cantilever sheet pile retaining wall for permanent work with a concrete cap.

Figure 7-1. Sheet Pile Wall with Cap Beam

Soldier pile retaining walls may be constructed with driven piles (steel, treated timber, or precast concrete), or the piles may be placed in drilled holes and backfilled with concrete, slurry, sand, pea-gravel, or similar approved material. A soldier pile may also be a cast-in-place reinforced concrete pile. Lagging is placed between soldier pile vertical elements and could be treated timber, reinforced shotcrete, reinforced cast-in-place concrete, precast concrete panels, or steel plates. This type of wall depends on passive resistance of the foundation material and the moment resisting capacity of the vertical structural members for stability. The maximum height is limited to competence of the foundation material and the moment resisting capacity of the vertical structural members. Figure 7-2 shows a typical soldier pile retaining wall with timber lagging.

Figure 7-2. Soldier Pile Wall with Cap Beam

The general design procedure for soldier pile walls is to assume one half the pile spacing on either side of the pile acts as a panel loaded with active soil pressures and surcharge loading above the depth of the excavation. The portion of the soldier pile below the depth of the excavation is likewise loaded with both active and passive soil pressures and surcharge loading. Resistance to the lateral movement or overturning (about any point) of the soldier pile is furnished by the passive resistance of the soil below the excavation on both sides of the soldier pile.

7-2 Effective Pile Width

The effective width (**d**) of a soldier pile is generally considered to be the dimension of the soldier pile taken parallel to the line of the wall for driven piles or drilled piles backfilled with material other than concrete such as gravel. The effective width of the soldier piles may be taken as the diameter of the drilled hole when 4-sack or better concrete is used below the excavation line. Soil arching, however, can greatly increase the effective width described above as shown in Figure 7-3 below. Arching of the soil between soldier piles can increase the effective width of a soldier pile up to 3 times for granular soil and 2 times for cohesive soils.

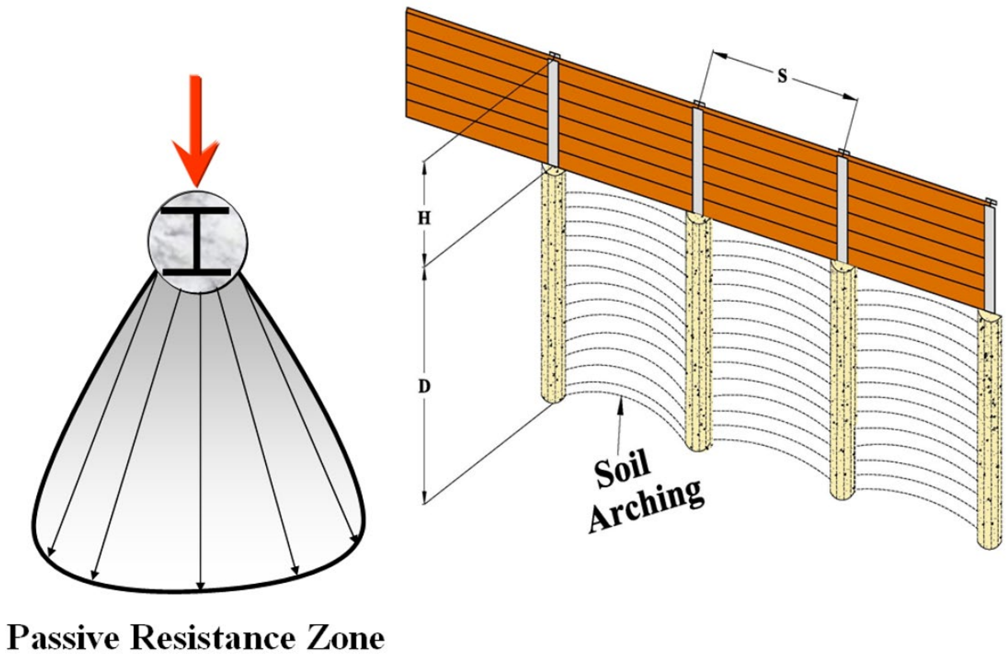


Figure 7-3. Soldier Pile with Soil Arching Below Excavation

Numerous full-scale pile experiments have shown the passive resistance in front of an isolated pile is a three-dimensional model for the sake of analysis. Therefore, the passive resistance in front of a pile must be adjusted due to the effect of soil arching beyond the effective pile width.

The soil arching factor for granular soils is a function of the soil friction angle (ϕ) as shown below.

Arching Capability Factor, $\mathbf{f} = 0.08 \times \emptyset \ \leq 3$ (7-2-1)

The Adjusted Pile Width is limited to actual pile spacing.

Adjusted Pile Width $=$ (Effective Width $\times\, \mathbf{f}$) $\leq$ Pile Spacing (7-2-2)

The arching capability for cohesive soil ranges between 1 and 2 as shown in Figure 7-4.

CONSISTENCY	VERY SOFT	SOFT	MEDIUM	STIFF	VERY STIFF	HARD
q_u = unconfined comp. strength (PSF)	500	1000	2000	4000	8000	
Unit Weight (PCF) Saturated	100-120	110-130		120-140		130+
Arching Capability	1 to 2	1 to 2	2	2	2	

VERY SOFT: Exudes from fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.

Figure 7-4. Arching Capability for Cohesive Soil

Below the excavation depth, the adjusted pile width is used for the passive resistance in front of the pile. Any active loadings (including surcharge loadings) on the back of the pile below the excavation depth use only the effective width of the pile, effectively fixing the arching factor at 1. Again, the adjusted pile width cannot exceed the pile spacing.

7-3 System Deflection

The point of fixity, or the point of zero (0) deflection, of the “shoring support” of a cantilevered system is a significant assumption made for the analysis of a shoring system. The point of fixity, below the excavation line as shown in Figure 7-5 below, will affect the pile embedment, shears, moments, and deflection. The point of fixity is defined as a percentage of the embedment depth, **D,** which varies from 0 to 0.75D. For unrestrained shoring systems in most stiff to medium dense soils, a value of 0.25D may be assumed and this value is used as the default value in the Structure Construction (SC) Trenching & Shoring Check Program. A greater value may be used for loose sand or soft clay.

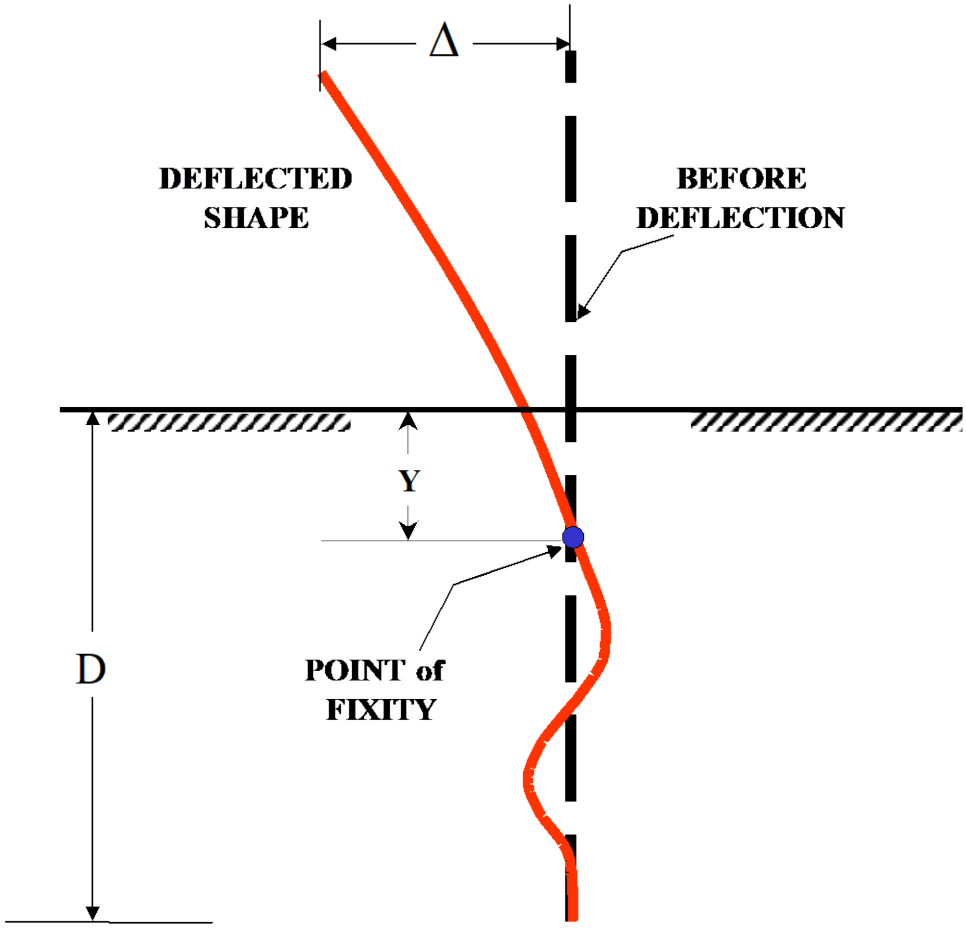


Figure 7-5. Deflected Shape for Unrestrained System

Calculating deflections of temporary shoring systems can be complicated. The total deflection of a shoring system is a combination of the deflection of the structural supporting member due to bending, and the movement of the entire system (usually thought of as rotation) within the embedded portion of the system. Deflection calculations are required for any shoring system adjacent to the railroad or high-risk structures. Generally, the taller a shoring system becomes, the more likely it is to be subject to large lateral deflections. The amount of allowable deflection or movement is inversely proportional to the sensitivity to movement of what is being shored. Thus, it will be up to the Engineer's judgment as to what degree of analysis will be performed. Bear in mind that except for the railroad as discussed in Chapter 9, *Railroads*, of this manual, there are no guidelines on the maximum allowable lateral deflection of the shoring system. For other high-risk structures, allowable deflections are on a case-by-case basis.

Calculations to approximate shoring deflection are normally performed per standard beam analysis methods. This approximation does not account for other factors that may be contributory, such as rotation of a cantilevered shoring or long-term movement of the ground anchors for a restrained system. The deflection can either be determined from double integration of the moment diagram or by multiplying the area under the moment diagram times its moment arm beginning from the top of the pile to a depth '**D**' below the dredge line. Although these methods described above are for standard beam analysis, it should be pointed out that shoring systems do not necessarily act as standard beams with point supports. Instead, for calculating a realistic deflection for a shoring system a Soil-Structure Interaction (SSI) analysis using a **p-y** approach or a finite element method, must be performed. The SSI method of analysis is beyond the scope of this manual and the Engineer is encouraged to contact the SC Falsework Engineer in SC HQ in Sacramento. An example problem showing deflection calculations can be found in Appendix B, *Example Problems*.

7-4 Soil Pressure Distribution for Layered Soil

As discussed in Chapter 4, *Earth Pressure Theory and Application*, the horizontal pressure exhibited is a function of the soil unit weight, the depth, and the earth pressure coefficient. Thus, in a uniform soil the pressure generally grows with depth. When there are layered soils in the system, the pressure diagram can develop discontinuities at the divisions between soil layers due to the property changes. As depicted in Figure 7-6 below, these interface pressure changes are represented by using sigma, **σ**, with a "**+**" or a "**-**" to represent the pressure based on the upper soil properties and lower soil properties, respectively. Thus, for a shoring system in layered soils, it is very important to be mindful of these nuances while developing the soil pressure distribution for each individual soil layer as shown in Figure 7-6.

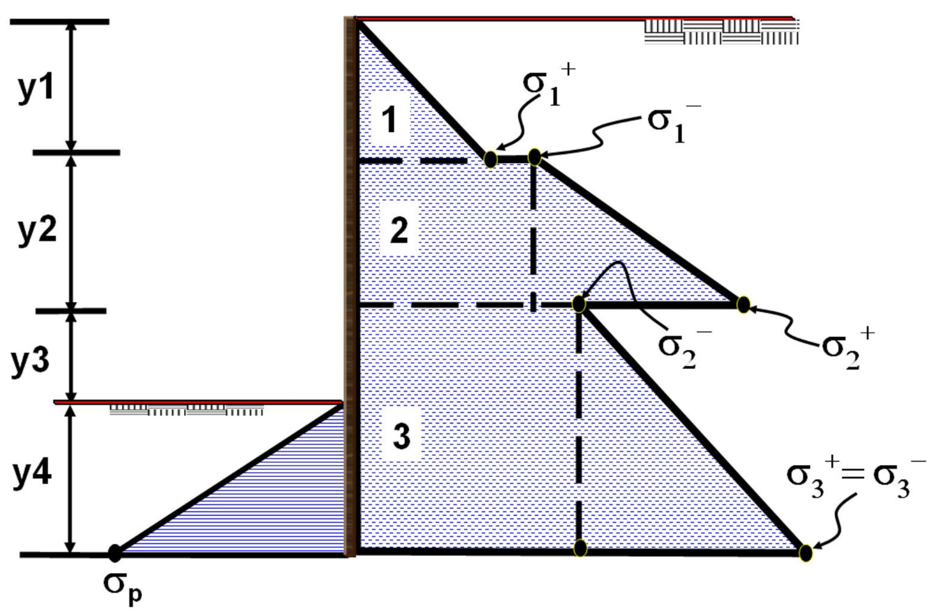


Figure 7-6. Pressure Diagram of Multilayer Soil Pressure

The vertical pressure of the soil in layer 1 is the unit weight of the soil multiplied by the depth. The horizontal pressure against the shoring at this depth is dependent on the $\mathbf{K_a}$

of the soil. Thus, sigma, σ, will have different values at the depth of **y1** due to the unique soil properties of layer 1 and layer 2. Figure 7-6 above, depicts layer 1 as having a smaller value of $\mathbf{K_a}$ and layer 2 having a larger $\mathbf{K_a}$. This manual uses the nomenclature of σ^+ to represent the pressure on the shoring "just above" the layer change and σ^- to represent the pressure on the shoring "just below" the layer change.

7-5 Lateral Earth Pressures for Unrestrained Shoring Systems

Cantilever retaining walls are analyzed by assuming that the vertical structural member rotates at point O, at the distance $\mathbf{D_O}$ below the excavation line, as illustrated in Figure 7-7. As a result, the mobilized active pressure develops above point O in the back of the wall and below point O in the front of the wall. The mobilized passive pressure develops in front of the wall above point O and at the back of the wall below point O. The realistic load distribution is shown in Figure 7-8b, and the idealized pressure distribution is shown in Figure 7-8c.

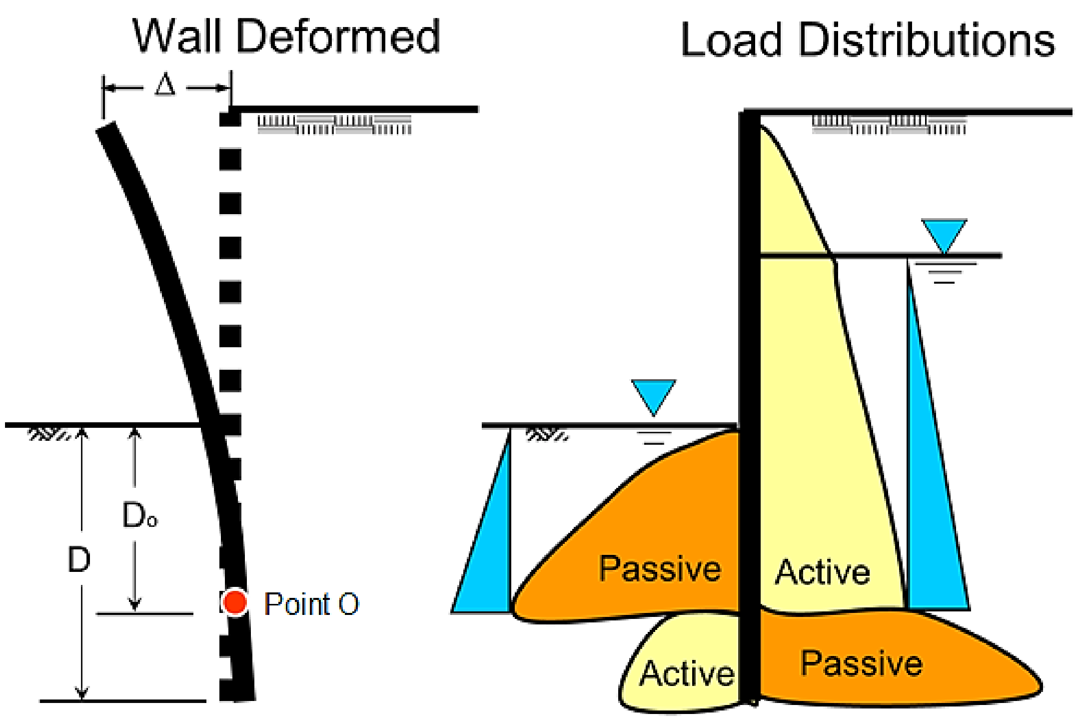


Figure 7-7. Non-Gravity Cantilever Retaining Walls Loading Diagram

There are at least two acceptable methods for analyzing the soil pressures for the unrestrained systems; the Rigorous Method and the Simplified Method. Examples of these methods are detailed below. Both of these methods solve for the embedment depth of the system to be stable by summarizing the moments these soil pressures exert to “Drive” the system over into the excavation, and offsetting the soil pressures “Resisting” these moments from the opposite direction. In the examples below, these will be referred to as “Driving Moments” and “Resisting Moments” within the shoring system.

7-5.01 The Rigorous Method

For the Rigorous Method, the idealized load distribution is shown in Figure 7-8. The load distribution is a combination of active and passive pressure, and it extends below point O down to the tip of pile.

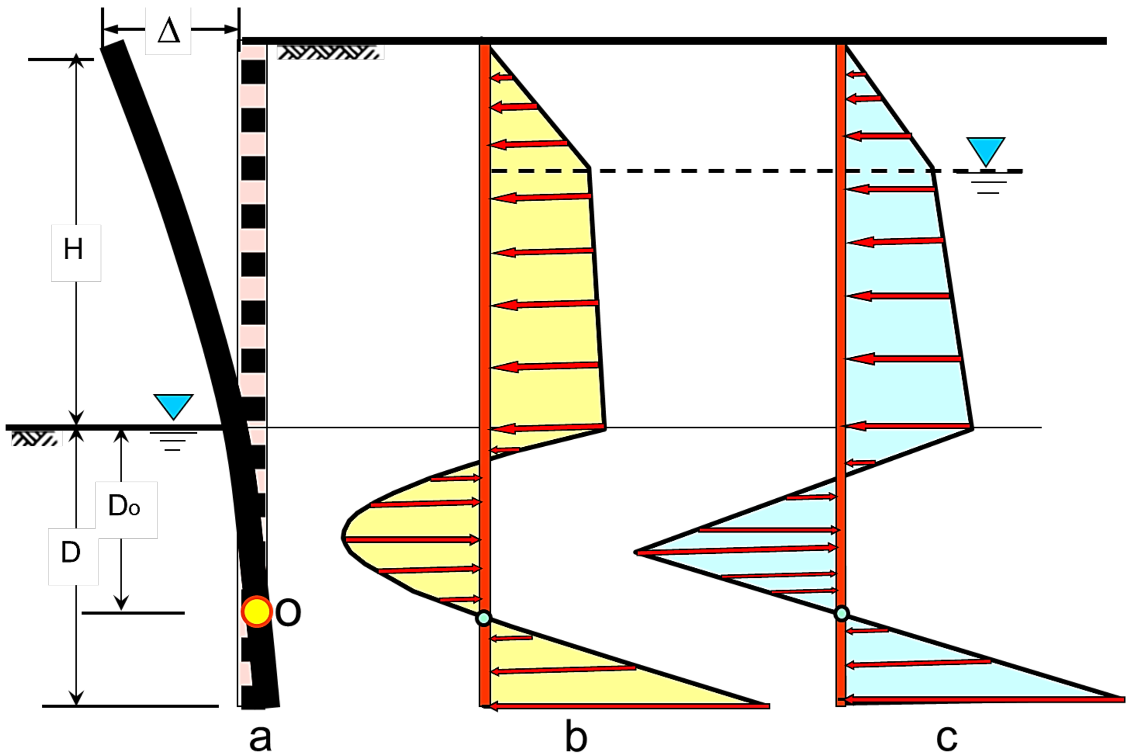


Figure 7-8. Cantilever Sheet Pile Walls Idealized Loading Diagram

Figure 7-9 shows the shear and moment diagrams for a continuous sheet pile wall with an idealized pressure distribution for the Rigorous Method. Shoring utilizing sheet piles will have a different pressure distribution below the excavation (see Figure 7-9) than a soldier pile (see Figure 7-10) due to pile spacing and effective pile width. Use this method to solve for the embedment depth (**D**), maximum shear, maximum moment, and deflection.

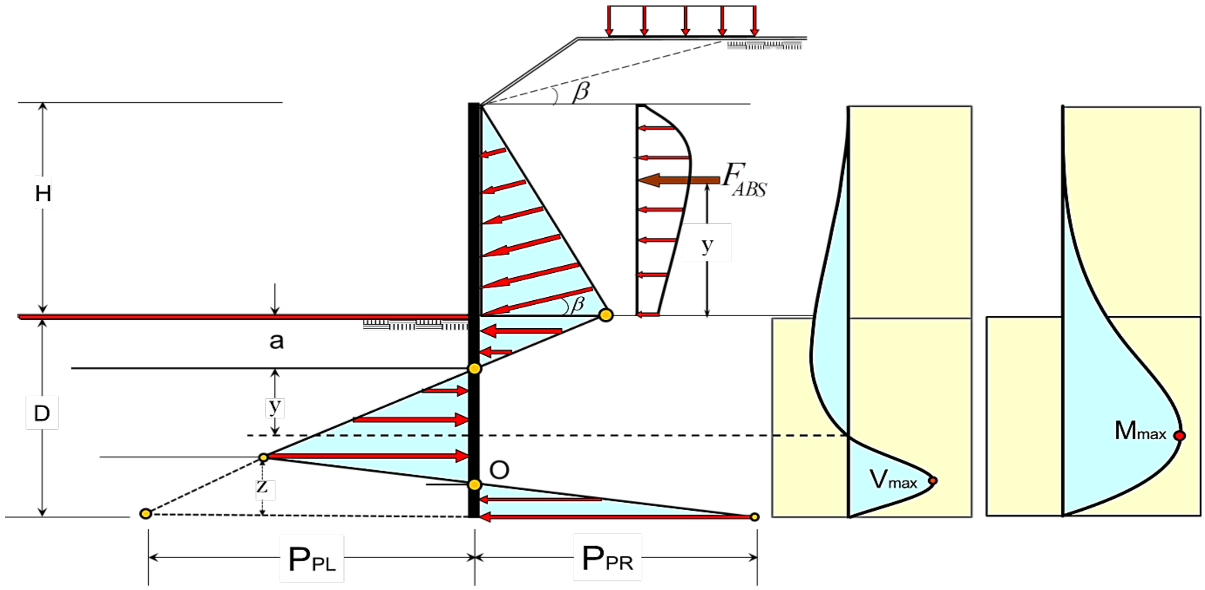


Figure 7-9. Sheet Pile Walls Idealized Pressure Distribution and Shear and Moment Diagram

For the sheet pile system shown above, the loading analysis is simpler as the wall loadings can be calculated on a per-foot basis. The figure below for a soldier pile system must account for the pile spacing as a tributary area loaded on the retained side, and the pile spacing plus soil arching, on the resistive side. This is what produces the discontinuity (step down) in the pressure diagram visible in the figure. This difference also applies to using the Simplified Method, which is discussed next.

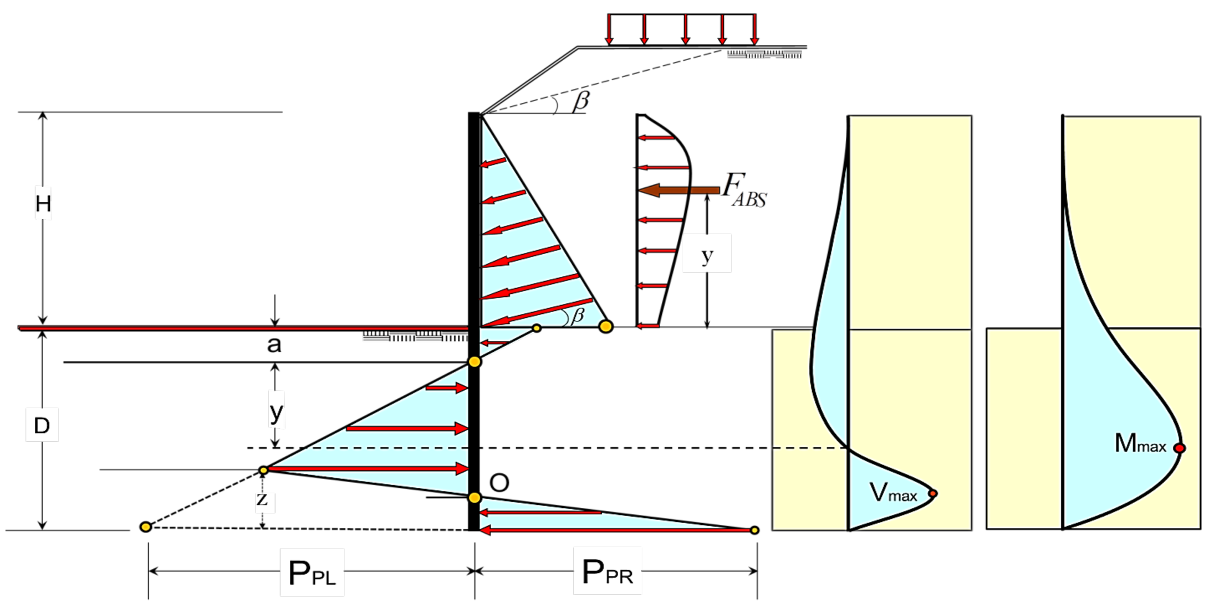
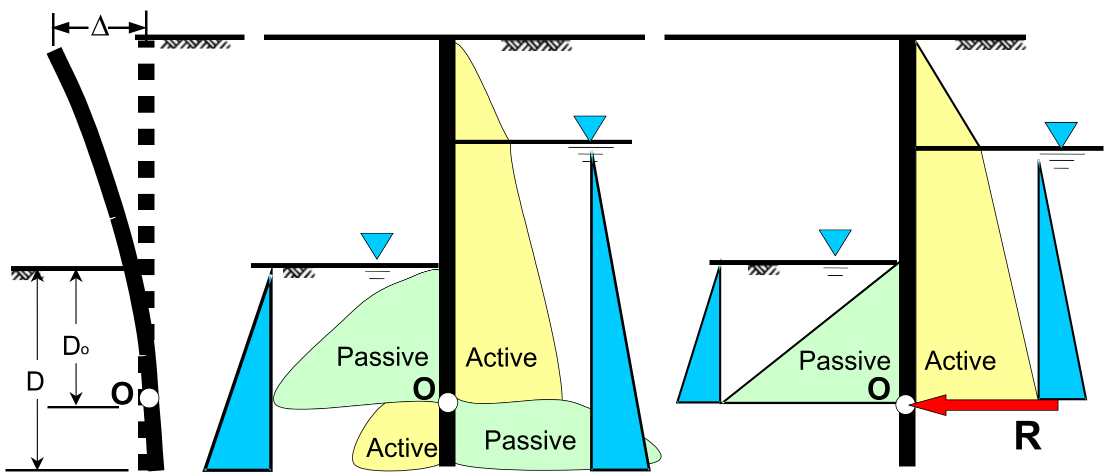


Figure 7-10. Soldier Pile Idealized Pressure Distribution and Shear and Moment Diagram

7-5.02 The Simplified Method

Load distributions for typical cantilever earth retaining systems using the simplified procedure are shown in Figure 7-11. Cantilever retaining walls are analyzed by assuming that the vertical structural member rotates at point O, at the distance **D_O**, below the excavation line as shown in Figure 7-11(a). The realistic load distribution is shown in (b). As a result, the mobilized active pressure develops above point O in the back of the wall and below point O in the front of the wall. The mobilized passive pressure develops in front of the wall above point O, and at the back of the wall below point O.

The simplified load distribution is shown in Figure 7-11(c). Force **R** is assumed at point O to compensate the resultant net active and passive pressure below point of rotation at point O. The calculated depth, **D**, is determined by increasing **D_O** by 20 percent to approximate the total embedment depth of the vertical wall element. The 20 percent increase is not a factor of safety. It accounts for the rotation of the length of vertical wall element below point O as shown in Figure 7-11(a). (**D** = **1.2D_O**, AASHTO 3.11.5.6).



(a)- Wall Deformed (b) - Load (c) - Simplified Load Simplification

Figure 7-11. Cantilever Sheet Pile Walls Idealized Loading Diagram

The following procedure is used for the check of a cantilever wall using the Simplified Method, as illustrated in (c) of Figure 7-11:

1. Calculate active/passive earth pressure to an arbitrary point, O, at the depth of **D_o** below the excavation line.
2. Take moments about point O to eliminate force **R** and determine embedment depth **D_O**. There will be driving moments tipping the system forward, and resisting moments working to keep the system upright. These become balanced at the depth **D_O** .

3. Increase $\mathbf{D_O}$ by 20 percent ($\mathbf{D}$ = 1.2 D_O).
4. Calculate **R** by summation of forces in horizontal direction (**R** ≤ 0; if **R** is larger than zero, increase **D**).
5. Calculate maximum bending moment ($\mathbf{M_{max}}$) and maximum shear force ($\mathbf{V_{max}}$) to check the vertical structural member.
6. Then check the lagging and any other members of the system (see Chapter 6, *Structural Design of Shoring Systems*).

7-5.03 Variations of Pressure Diagrams for Other Soil Configurations

Below are examples of various pressure distributions for analysis of unrestrained shoring systems with the Simplified Method, based on differing soil profiles.

For a shoring system with a single layer of granular soil, Figure 7-12 may be used to determine the lateral earth pressure distribution.

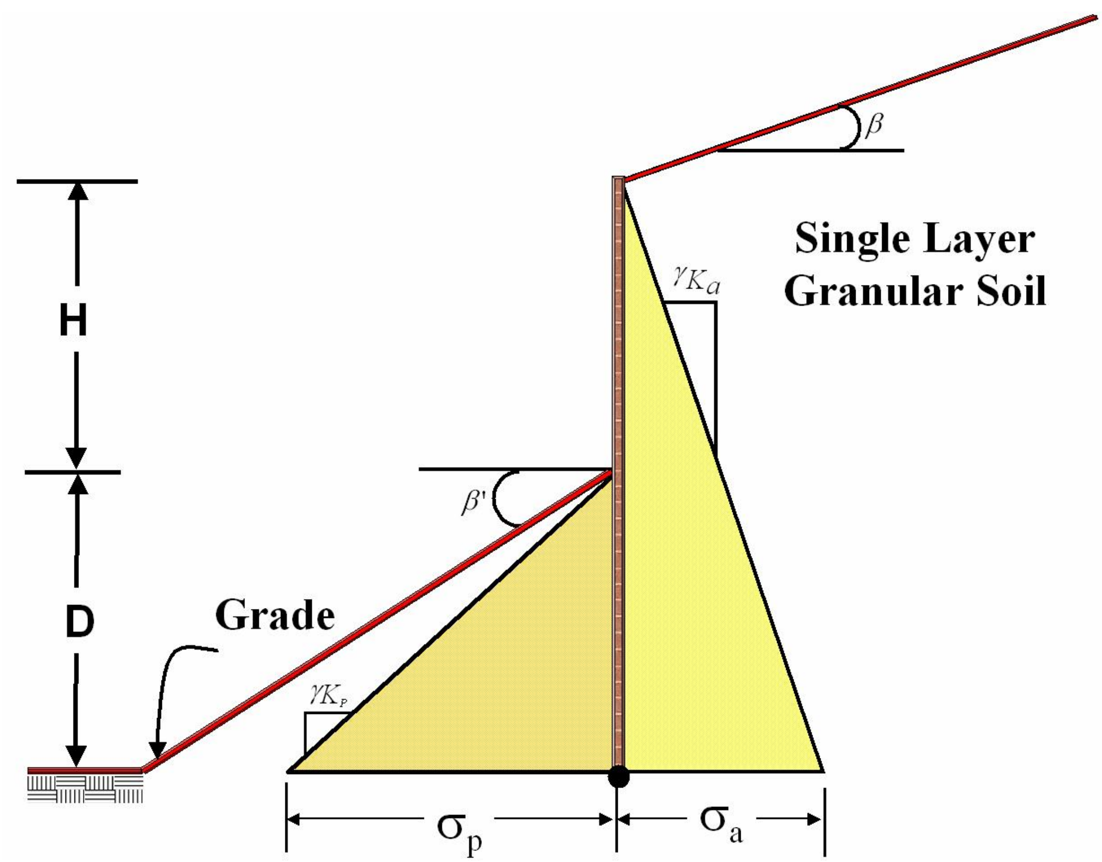


Figure 7-12. Loading Diagram for Single Layer

For a shoring system with a multi-layer granular soil, Figure 7-13 may be used to determine the lateral earth pressure distribution.

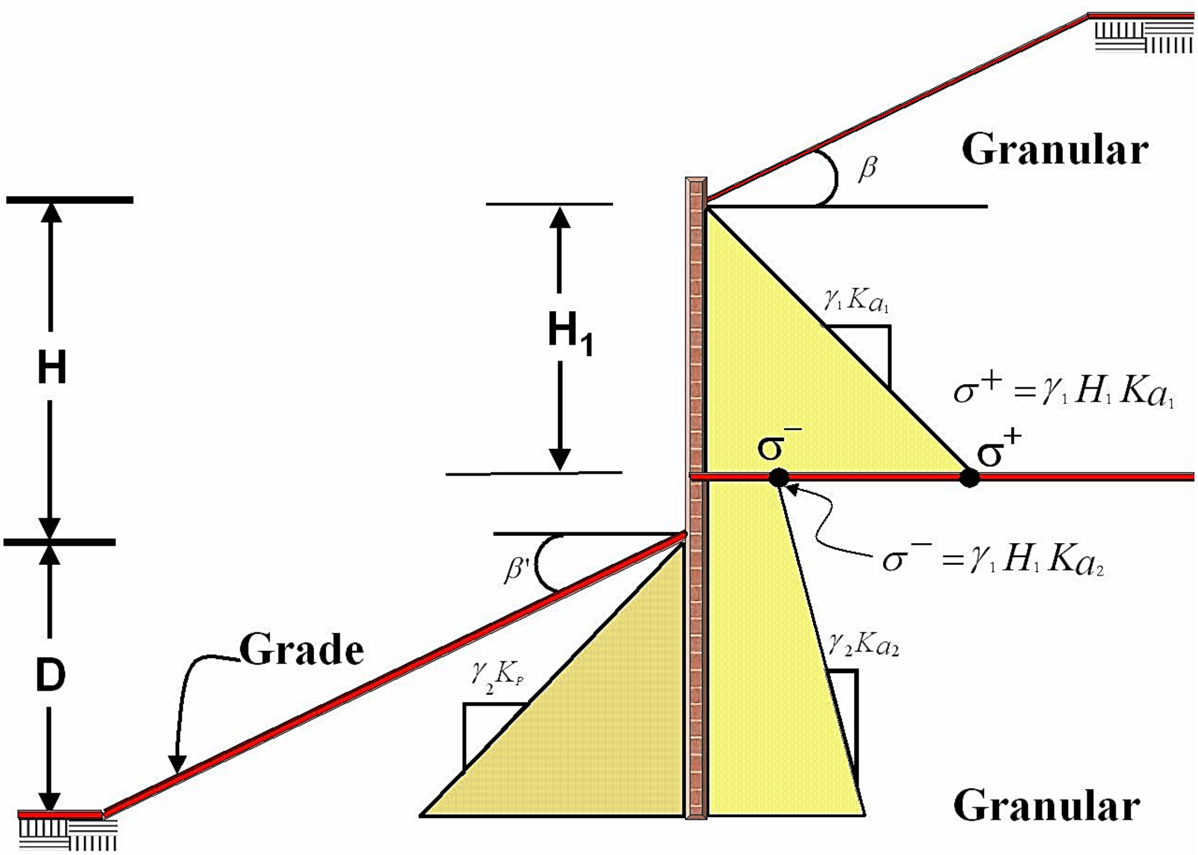


Figure 7-13. Loading Diagram for Multi-Layer Granular Soil

For a shoring system that is embedded in granular soils and retaining cohesive soils, Figure 7-14 may be used to determine the lateral earth pressure distribution.

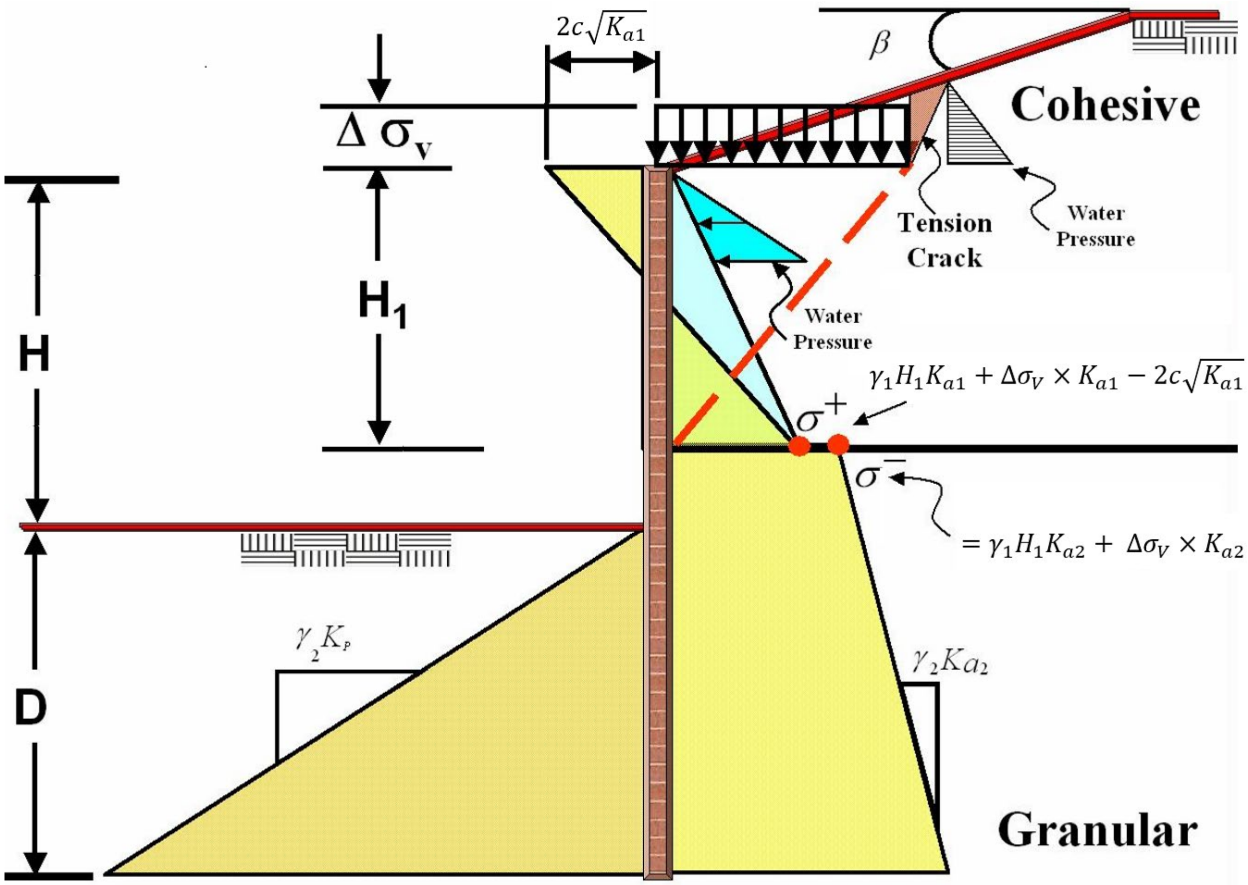


Figure 7-14. Loading Diagram for Multi-Layer – Cohesive over Granular Soil

For a shoring system that is embedded in (supported by) cohesive soils and retaining granular soil, Figure 7-15 may be used to determine the lateral earth pressure distribution.

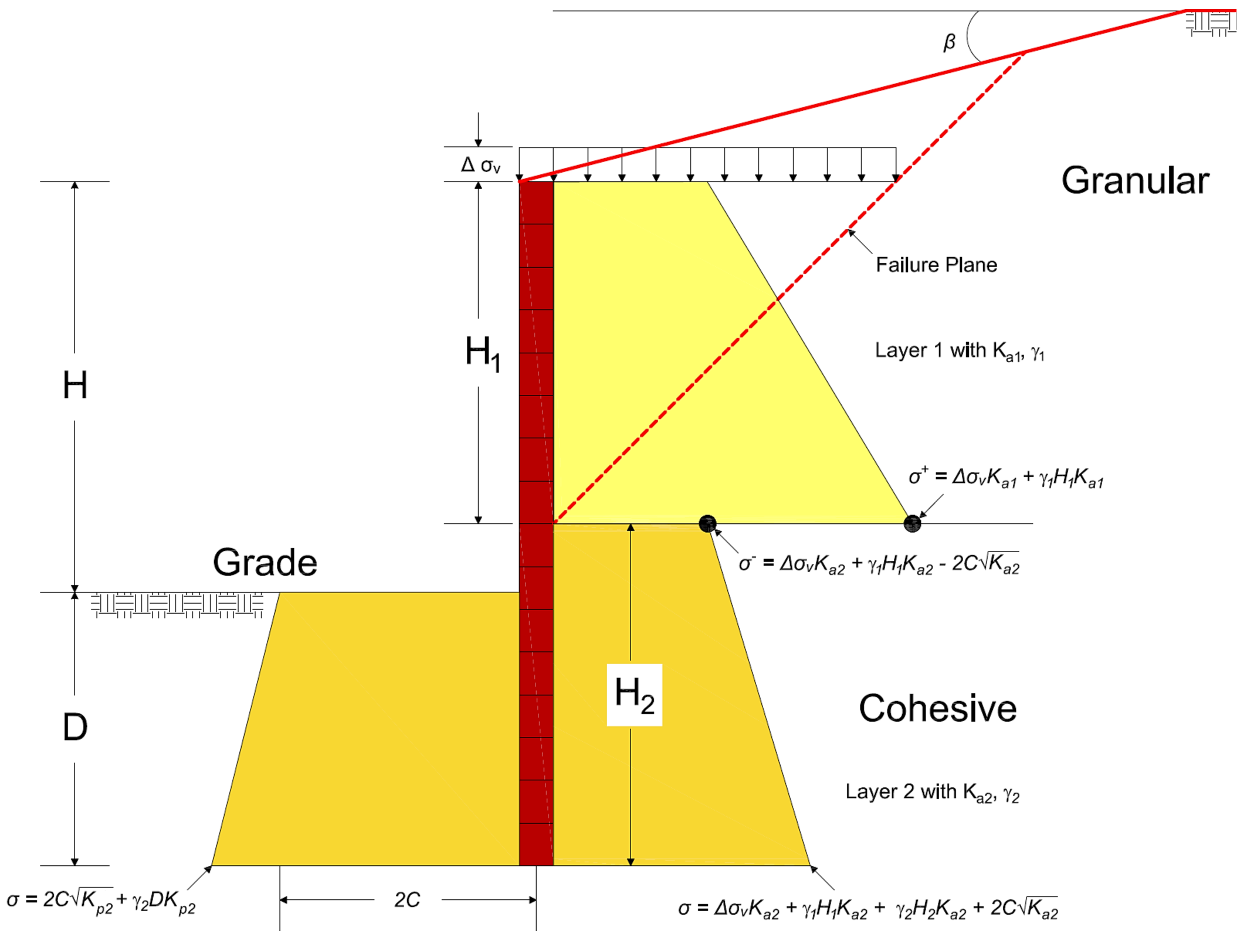


Figure 7-15. Loading Diagram for Granular over Cohesive Soil

For a shoring system with a multi-layer cohesive soil, Figure 7-16 may be used to determine the lateral earth pressure distribution.

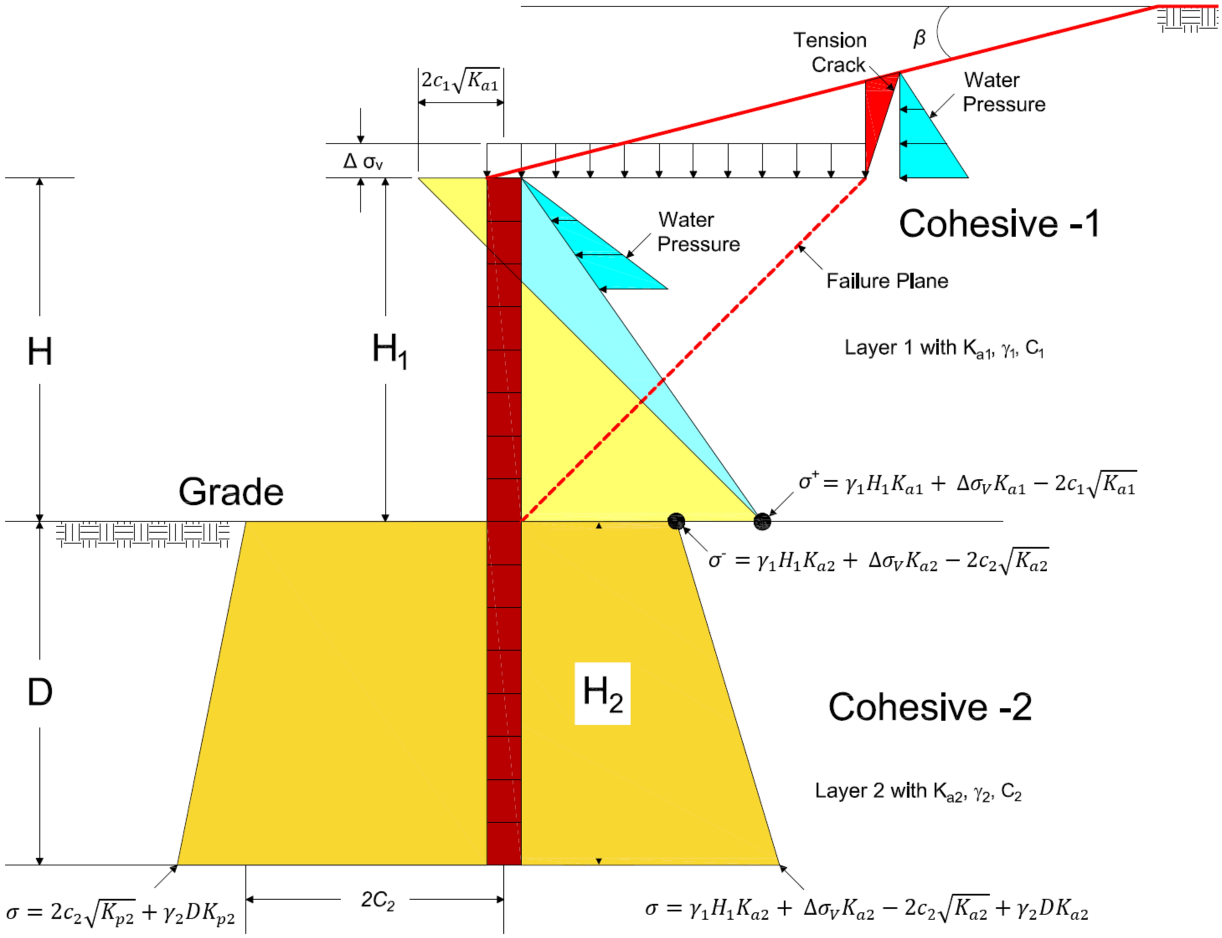


Figure 7-16. Loading Diagram for Multi-Layer Cohesive Soil

The active lateral earth pressure on the embedded wall element can be determined for Figure 7-12 through Figure 7-16 with these general steps:

1. Treat the sloping backfill above the top of the wall within the active failure wedge as an additional surcharge (**$\Delta\sigma_v$**).
2. The portion of the negative loading at the top of the wall due to cohesion is ignored.
3. Any hydrostatic pressure in the tension crack needs to be considered.

7-5.04 Example 7-1A: Cantilevered Soldier Pile Wall by Rigorous Method

Using the Rigorous Method perform a shoring check for a W14 x 120 cantilevered soldier-pile-lagging wall with piles placed at 8 feet on center, encased in 2-foot diameter holes, filled with 4-sack concrete. The pressure of 72 psf is the minimum lateral construction surcharge acting on the timber lagging that is caused by typical construction loading. The soil properties are shown below, in Figure 7-17.

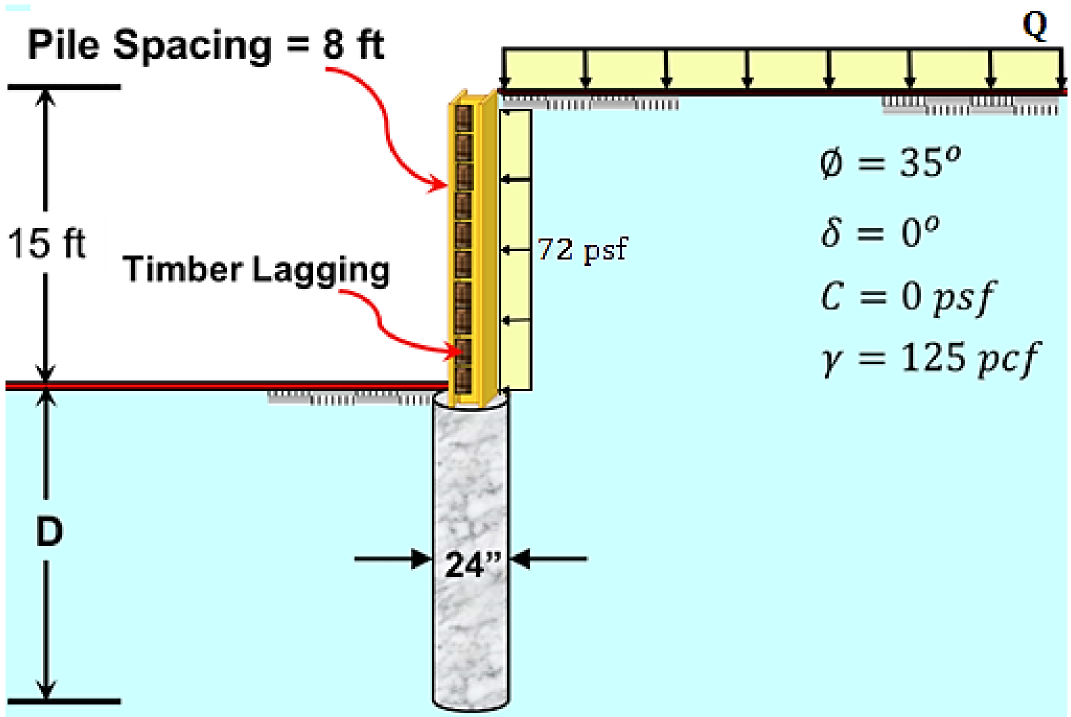


Figure 7-17. Cantilevered Soldier-Pile-Lagging Wall Example Properties

Solve for the following using the Rigorous Method:

1. Calculate active & passive earth pressures
2. Determine pile embedment, **D**
3. Calculate maximum shear & moment
4. Calculate service deformation
5. Calculate timber lagging deflection.

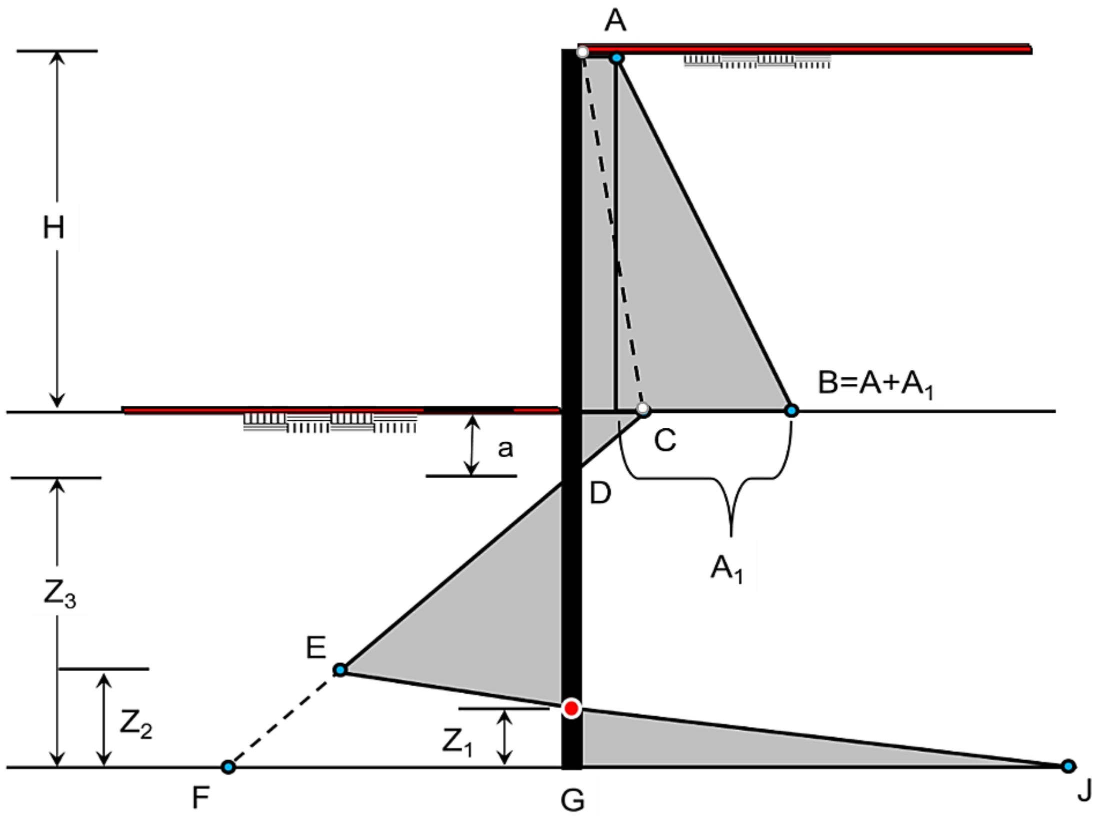


Figure 7-18. Rigorous Assumed Loading Diagram

Determine Active and Passive Earth Pressures:

Calculate active and passive earth pressure coefficients: since the wall friction (***δ***) is zero, use Rankine earth pressure theory to calculate the active and passive earth pressure coefficients, as outlined in Chapter 4.

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \tag{7-5-1}$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \tag{7-5-2}$$

Note: Rankine Theory tends to underestimate the passive earth pressure. It is recommended to use the Log-Spiral-Rankine Model to calculate the passive earth force.

Since this is a soldier pile system, soil arching for the passive resistance needs to be included. Calculate the arching factor, **f** :

$$f = 0.08\,\emptyset = (0.08 \times 35) = 2.8 \tag{7-5-3}$$

From the given information, lowercase "**a**" can be easily calculated and will be needed to find the pressures at each point. The calculations below use the slope of the pressure change with depth (slope of the line). The slope is based on the combination of the active and passive earth pressure coefficients and begins with the pressure at point C. The depth of "**a**" is at the point the earth pressure is equal to zero.

$$0 = \sigma(\text{at C}) - \text{a}[\text{slope of the line}] \tag{7-5-4}$$

$$(\text{slope of the line}) = \left[\gamma\left(\text{K}_\text{p}\text{f} - \text{K}_\text{a}\right)\right] \tag{7-5-5}$$

$$0 = \sigma(\text{at C}) - \text{a}\left[\gamma\left(\text{K}_\text{p}\text{f} - \text{K}_\text{a}\right)\right] \tag{7-5-6}$$

$$\text{a} = \frac{\sigma(\text{at C})}{\gamma(\text{pcf}) \times \left(\text{K}_\text{p}\text{f} - \text{K}_\text{a}\right)} \tag{7-5-7}$$

$$\text{a} = \frac{125\ \text{pcf} \times 15\ \text{ft} \times 0.271}{(125\ \text{pcf})((3.69 \times 2.8) - 0.271)} = \frac{508\ \text{psf}}{1258\ \text{pcf}} = 0.404\ \text{ft} \tag{7-5-8}$$

Note: In the above equation, **f** is the arching capability factor. This factor is applied to passive pressures below the excavation line for soldier pile systems. Calculate the earth pressure distribution in kips/ft at each node of the diagram. This implies multiplying each pressure to account for the soldier pile spacing at the various points in Figure 7-18.

- Point A - Lateral load due to minimum construction surcharge above the excavation line only (Note in this example the minimum construction surcharge is taken farther than 10 feet below the top of the excavation):

$$\text{A} = 0.072 \times 8 = 0.576\ \text{kips/ft} \tag{7-5-9}$$

- Point A_1 - Active lateral load at excavation level on the wall:

$$\text{A}_1 = 0.125 \times 15 \times 0.271 \times 8 = 4.065\ \text{kips/ft} \tag{7-5-10}$$

- Point C - Active lateral load at excavation level from top on the soldier pile:

$$\text{C} = 0.125 \times 15 \times 0.271 \times 2 = 1.01625\ \text{kips/ft} \tag{7-5-11}$$

- Point F - Passive lateral load in front of the dredge line at embedment depth:

$$\text{F} = (0.125 \times \text{Z}_3 \times ((3.69 \times 2.8) - 0.271) \times 2) = 2.51525\text{Z}_3\ \text{kips/ft} \tag{7-5-12}$$

- Point J - Active lateral load distribution at embedment depth:

$$\mathrm{J} = \big(0.125 \times (\mathrm{Z}_3 + 0.404) \times \big((3.69 \times 2.8) - 0.271\big) \times 2\big) + (0.125 \times 15 \times 3.69 \times 2.8 \times 2) = 2.51525\ \mathrm{Z}_3 + 39.7612\ \text{ kips/ft} \qquad (7\text{-}5\text{-}13)$$

Calculate resultant earth forces (**P**) and apply $\sum \mathbf{F} = \mathbf{0}$. The applied forces on the wall are the areas of the distributed loads, as illustrated in Figure 7-19.

1. Calculate active earth force due to surcharge full height of the wall **H**, **P_{sur}**:

$$\mathrm{P_{sur}} = 0.576 \frac{\text{kips}}{\text{ft}} \times 15\ \text{ft} = 8.64\ \text{kips} \qquad (7\text{-}5\text{-}14)$$

2. Calculate active earth force above dredge line, **P_1**:

$$\mathrm{P}_1 = \frac{1}{2} \times 4.065 \frac{\text{kips}}{\text{ft}} \times 15\ \text{ft} = 30.4875\ \text{kips} \qquad (7\text{-}5\text{-}15)$$

3. Calculate active earth forces below dredge line, **P_2**:

$$\mathrm{P}_2 = \frac{1}{2} \times 1.01625 \frac{\text{kips}}{\text{ft}} \times 0.404 = 0.2053\ \text{kips} \qquad (7\text{-}5\text{-}16)$$

4. Calculate passive earth forces below dredge line. For simplification, take (Area FEJ) – (Area FDG):

$$\text{Area FEJ} = \mathrm{P}_3 = \frac{1}{2} \times \left(2.51525\ \mathrm{Z}_3 \frac{\text{kips}}{\text{ft}} + \left(2.51525\ \mathrm{Z}_3 + 39.7612\ \frac{\text{kips}}{\text{ft}}\right)\right) \times \mathrm{Z}_2 = 2.51525\ \mathrm{Z}_3\mathrm{Z}_2 + 19.881\ \mathrm{Z}_2\ \text{kips} \qquad (7\text{-}5\text{-}17)$$

$$\text{Area FDG} = \mathrm{P}_4 = \frac{1}{2} \times 2.51525\ \mathrm{Z}_3 \frac{\text{kips}}{\text{ft}} \times \mathrm{Z}_3\ \text{ft} = 1.257625\ \mathrm{Z}_3^2\ \text{ kips} \qquad (7\text{-}5\text{-}18)$$

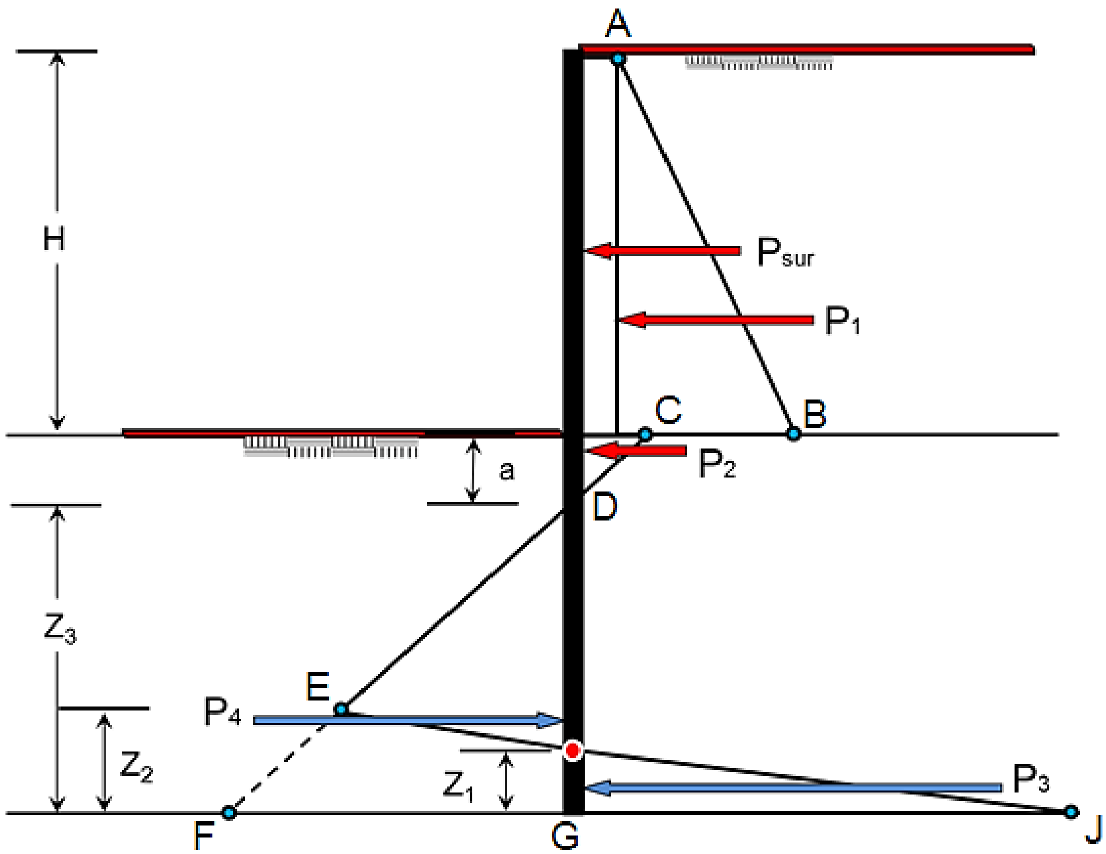


Figure 7-19. Force Diagram for Rigorous Method

Set up equations sum of forces and sum of moments to solve for variables **Z_2** and **Z_3**:

$$\Sigma F = 0$$

$$8.64 + 30.4875 + 0.2053 + (2.51525\ Z_3 Z_2\ +\ 19.881\ Z_2) - 1.257625\ Z_3^2 = 0 \tag{7-5-19}$$

Simplify and solve for **Z_2**:

$$Z_2 = \frac{1.257625 Z_3^2 - 39.3328}{2.51525 Z_3\ +\ 19.881} \tag{7-5-20}$$

$$\Sigma M = 0$$

$$
\begin{aligned}
&(8.64 \times (Z_3 + 0.404 + 7.5)) + \left(30.4875 \times (Z_3 + 0.404 + 5)\right) \\
&\quad + \left(0.2053 \times \left(Z_3 + \frac{2(0.404)}{3}\right)\right) \\
&\quad + \left((2.51525 Z_3 Z_2 \ + \ 19.881 Z_2) \times \frac{Z_2}{3}\right) \\
&\quad - \left(1.257625\, Z_3^2 \times \left(\frac{Z_3}{3}\right)\right) = 0
\end{aligned}
\tag{7-5-21}
$$

Simplify and collect like terms:

$$39.3328\, Z_3 + 233.1003 + 0.83842\, Z_3 Z_2^2 + 6.627\, Z_2^2 - 0.41921\, Z_3^3 = 0 \tag{7-5-22}$$

Solve for $\mathbf{Z_2}$ and $\mathbf{Z_3}$ by using iteration to achieve both simplified equations to equal 0:

$$Z_2 = 3.351 \text{ ft} \quad \& \quad Z_3 = 13.122 \text{ ft} \tag{7-5-23}$$

Determine Embedment Depth (without a Safety Factor):

$$\text{Total Embedment Depth} = Z_3 + \text{a} = 13.122 + 0.404 = 13.526 \text{ ft} \tag{7-5-24}$$

Calculate Maximum Shear:

Maximum shear occurs when the load diagram crosses zero (see Figure 7-20). In this case, the loading crosses zero at two locations so the area of the load diagram has to be calculated before the first zero point and after the second zero point. The largest value of the two areas will be $\mathbf{V_{max}}$. Usually, it will be the area of loading below the pivot point (second zero load location) because this is where the largest passive pressure is acting at the base of the wall.

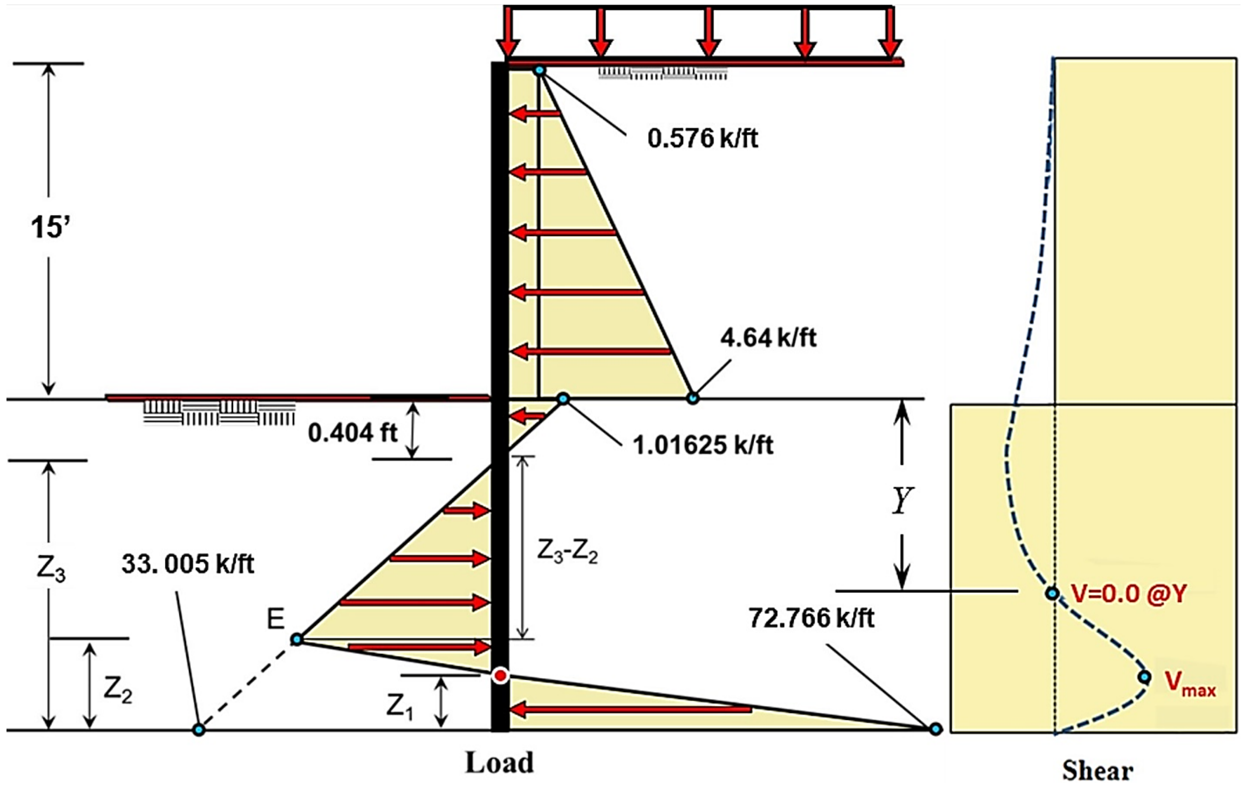


Figure 7-20. Net Load and Shear Diagram

Find pressure (kips/ft) at point E, using similar triangles:

$$\frac{33.005\ \text{kips/ft}}{13.122\ \text{ft}} = \frac{\text{E}}{(13.122 - 3.351)\text{ft}} \quad => \quad \text{E} = 24.576\ \text{Kips/ft} \tag{7-5-25}$$

Use similar triangles again to calculate $\mathbf{Z_1}$:

$$\frac{3.351\ \text{ft}}{(24.576 + 72.766)\ \text{kips/ft}} = \frac{\text{Z}_1}{72.766\ \text{kips/ft}} \quad => \quad \text{Z}_1 = 2.505\ \text{ft} \tag{7-5-26}$$

Calculate Shear, $\mathbf{V_{max}}$:

$$\text{V}_{\text{max}} = \frac{1}{2} \times \left(72.766 \frac{\text{kips}}{\text{ft}}\right) \times (2.505\ \text{ft}) = 91.14\ \text{kips} \tag{7-5-27}$$

Calculate Maximum Moment:

The maximum moment is located at distance **Y** below the excavation line where the shear is equal to zero, as illustrated in Figure 7-21. Therefore, the summation of horizontal forces at the distance **Y** must be set to equal zero.

Passive earth pressure at **Y** below the dredge line (Y = y + 0.404 ft):

$$P_P = \frac{1}{2}[0.125 \times 1 \times y((3.69 \times 2.8) - 0.271) \times 2]\, y$$
$$= 1.257625y^2 \ \text{kips/ft} \quad (7\text{-}5\text{-}28)$$

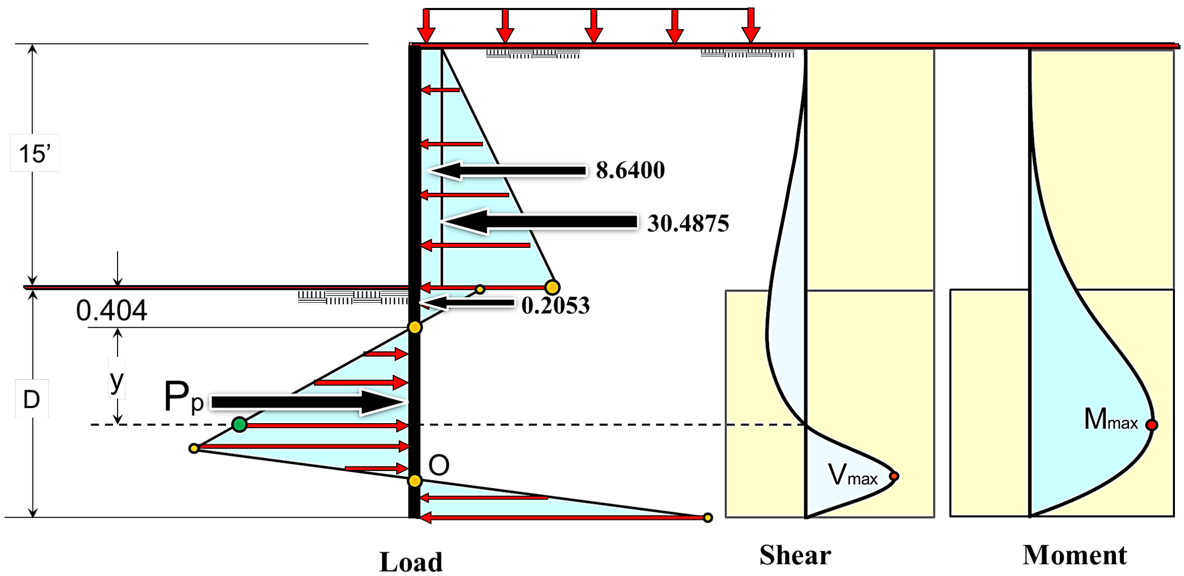


Figure 7-21. Location of Zero Shear and Maximum Moment for Soldier Pile

Set up equation, sum of forces: $\sum F_x = 0$ (7-5-29)

$$1.257625y^2 = 8.64 + 30.4875 + 0.2053 \quad (7\text{-}5\text{-}30)$$

$$1.257625y^2 = 39.3328 => \quad y = 5.592 \text{ ft} \quad (7\text{-}5\text{-}31)$$

$$Y = 5.592 + 0.404 = 6.00 \text{ ft (below the dredge line)} \quad (7\text{-}5\text{-}32)$$

$$M_{max} = \begin{cases} 8.64\times(7.5+6.00)+30.4875\times(5+6.00)+0.2053\times\left(6.00-\frac{0.404}{3}\right) \\ -\ 1.257625\times5.592^2\times\left(\frac{5.592}{3}\right) \end{cases} \quad (7\text{-}5\text{-}33)$$

$$M_{max} = 379.90 \text{ kip-ft} \quad (7\text{-}5\text{-}34)$$

The Shear and Moment Diagram along the pile length is shown in the Figure 7-22 and the Deflection Diagram is shown in Figure 7-23. These are from the SC Trenching and Shoring (T&S) Program.

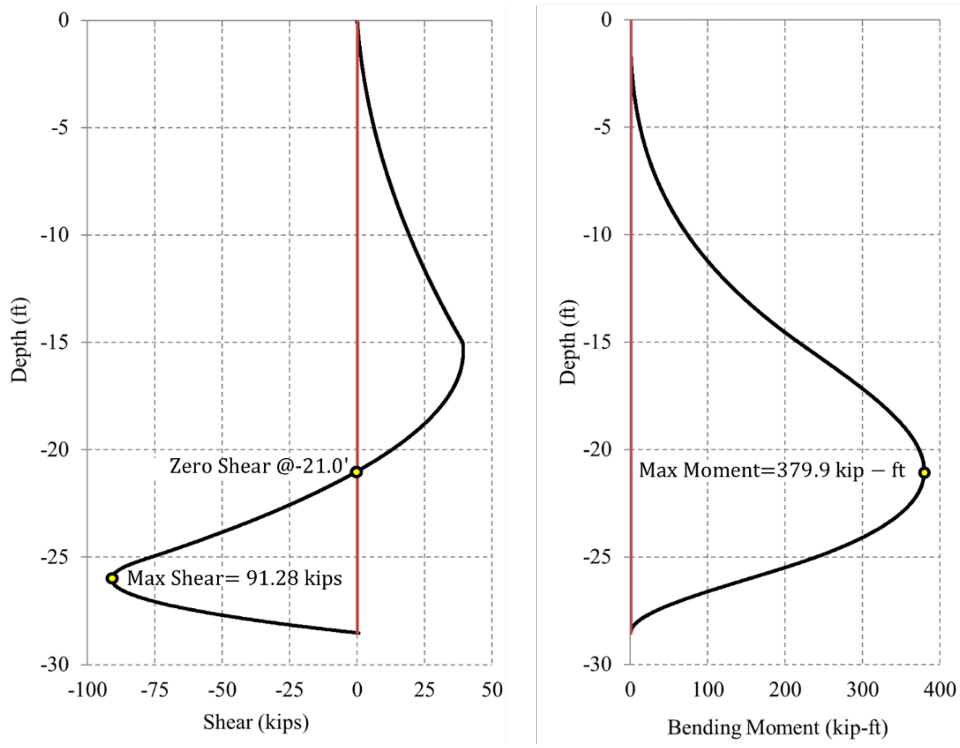


Figure 7-22. Shear and Moment Diagram (CT_T&S Program)

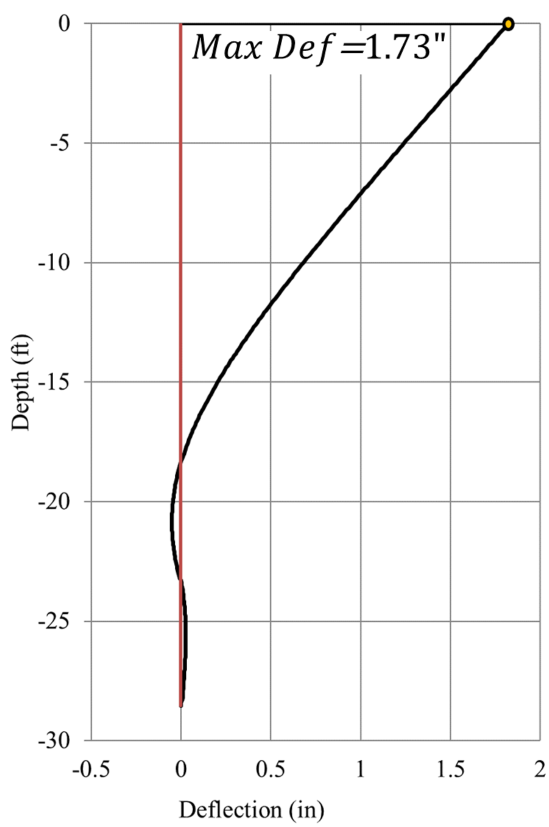


Figure 7-23. Deflection (CT_T&S Program)

Example lagging calculations are based on Chapter 6, *Structural Design of Shoring Systems*, Section 6-5, *Lagging*. A reduction factor of 0.6 will be used to reduce the soil distribution behind the lagging. See Section 6-5.01, *Example Lagging Calculations,* of this manual.

7-5.05 Example 7-1B: Cantilevered Soldier Pile Wall by Simplified Method

Perform a shoring check for a W14x120 cantilevered soldier-pile-lagging wall with piles at 8 feet on center encased in 2-foot diameter holes filled with 4-sack concrete, using the AASHTO simplified procedure. The pressure of 72 psf is the minimum lateral construction surcharge acting on the timber lagging that is caused by typical construction loading. The soil properties are shown below.

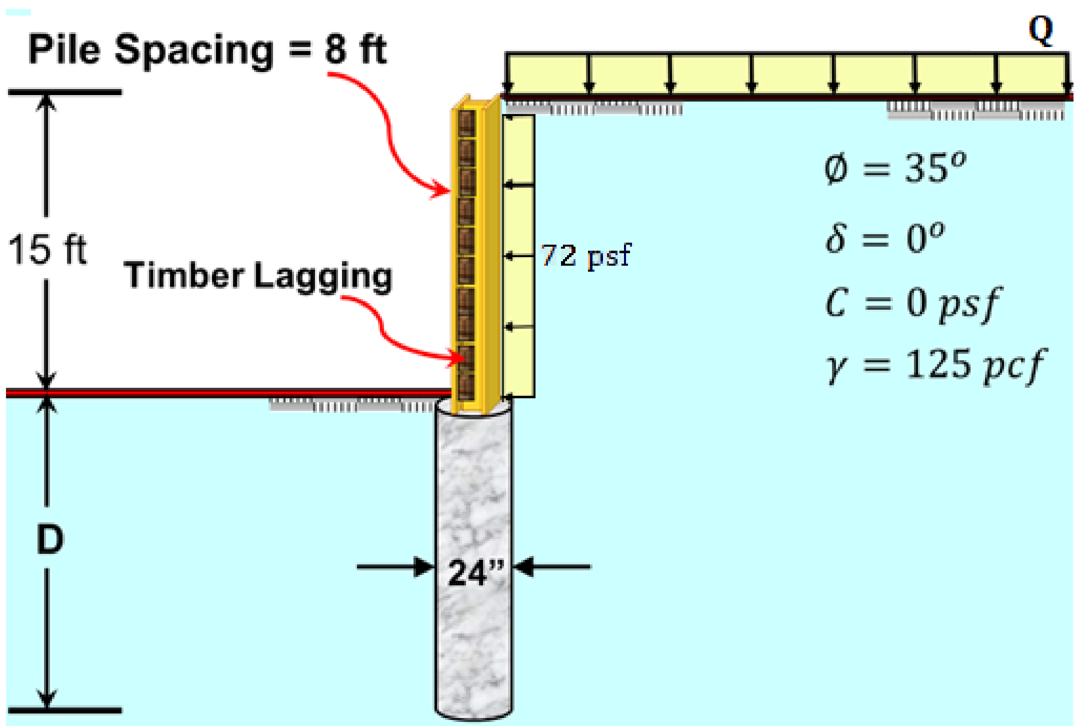


Figure 7-24. Cantilevered Soldier-Pile-Lagging Wall Cross-Section

Use these steps:

1. Calculate active & passive earth pressures
2. Determine pile embedment, **D**
3. Calculate maximum shear & moment
4. Calculate service deformation
5. Calculate timber lagging deflection.

Determine active and passive earth pressures:

- Calculate active and passive earth pressure coefficients. Since the wall friction (***δ***) is zero, use Rankine earth pressure theory to calculate the active and passive earth pressure coefficients:

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \tag{7-5-35}$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \tag{7-5-36}$$

Note: Rankine theory tends to underestimate the passive earth pressure. It is recommended to use the Log-Spiral model to compute the passive earth force.

- Calculate earth pressure distribution:

Lateral load due to minimum construction surcharge above the excavation line only:

$$\sigma_{sur} = 0.072 \text{ ksf} \tag{7-5-37}$$

Lateral load distribution at excavation level:

$$\sigma = 0.125 \text{ x } 15 \text{ x } 0.271 = 0.508 \text{ ksf} \tag{7-5-38}$$

Active lateral load distribution of the soil below the dredge line at depth $\mathbf{D_o}$:

$$\sigma_{AD_O} = 0.508 + (0.271 \text{ x } 0.125 \text{ x } D_O) = (0.508 + 0.0339\ D_O)\text{ksf} \tag{7-5-39}$$

Passive Lateral load distribution in front of the wall, at depth $\mathbf{D_o}$:

$$\sigma_{pD_O} = 0.125 \text{ x } 3.69 \text{ x } D_O = 0.461\ D_O \text{ ksf} \tag{7-5-40}$$

Calculate resultant earth forces:

1. Calculate active earth force due to surcharge $\mathbf{P_{AS}}$:

$$P_{As} = 15 \text{ ft} \times 8 \text{ ft} \times 0.072 \text{ ksf} = 8.64 \text{ kips} \tag{7-5-41}$$

2. Calculate active earth force above dredge line using 8-foot spacing, $\mathbf{P_{A1}}$:

$$P_{A1} = \frac{15}{2}\ \text{ft} \times 8\ \text{ft} \times 0.508\ \text{ksf} = 30.48\ \text{kips} \quad (7\text{-}5\text{-}42)$$

3. Calculate active earth force below dredge line using 2-foot hole diameter, $\mathbf{P_{A2}}$:

$$P_{A21} = D_0 \times 2 \times 0.508 = 1.016\ D_0\ \text{kips} \quad (7\text{-}5\text{-}43)$$

$$P_{A22} = 0.0339 D_0 \times \left(\frac{D_0}{2}\right) \times 2 = 0.0339\ D_0^2\ \text{kips} \quad (7\text{-}5\text{-}44)$$

- Calculate passive earth force below dredge line using 2-foot hole diameter and soil arching capability factor, $\mathbf{P_p}$:

$$f = 0.08 \times \emptyset = (0.08 \times 35) = 2.8 \quad (7\text{-}5\text{-}45)$$

$$P_p = 0.461 D_0 \times \left(\frac{D_0}{2}\right) \times 2 \times f = 0.461 D_0 \times \left(\frac{D_0}{2}\right) \times 5.60 = 1.291 D_0^2\ \ \text{kips} \quad (7\text{-}5\text{-}46)$$

These calculated values are summarized in Figure 7-25.

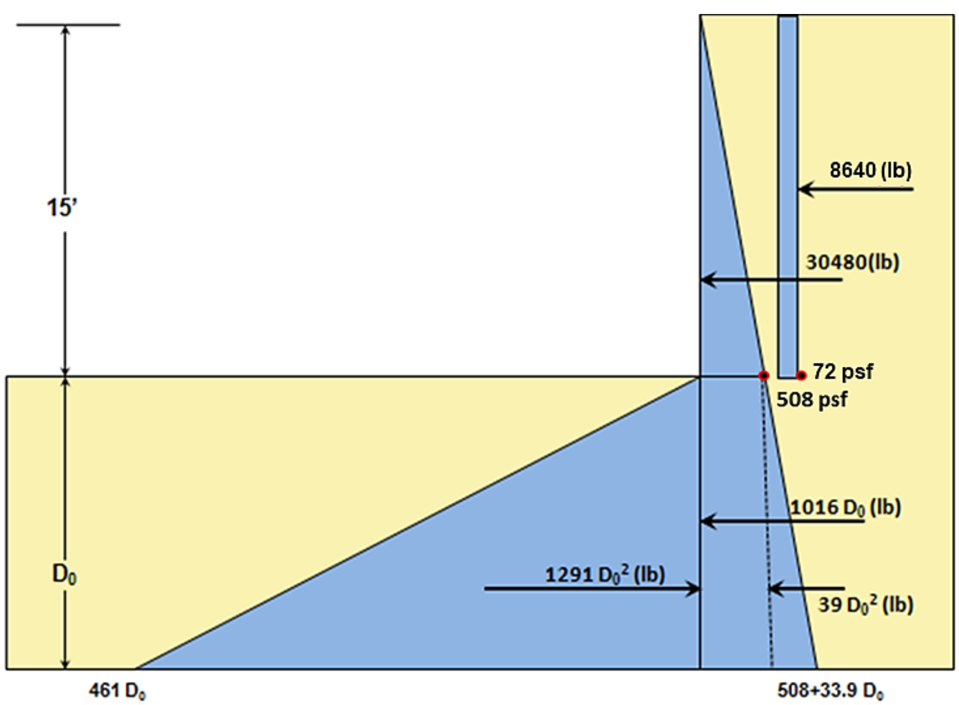


Figure 7-25. Force Diagram

Calculate driving and resisting moments as shown in Table 7-1 and Table 7-2:

Table 7-1. Driving Moments

Driving Force (kips)	Arm (ft)	Driving Moment **M_{DR}**
8.64	$7.5 + D_0$	$64.8 + 8.64D_0$
30.48	$5 + D_0$	$152.4 + 30.48D_0$
$1.016D_0$	$\frac{D_0}{2}$	$0.508D_0^2$
$0.0339D_0^2$	$\frac{D_0}{3}$	$0.0113D_0^3$

Table 7-2. Resisting Moments

Resisting Force (kips)	Arm (ft)	Resisting Moment **M_{RS}**
$1.291D_0^2$	$\frac{D_0}{3}$	$0.430D_0^3$

$$M_{DR} = 0.0113\ D_0^3 + 0.508\ D_0^2 + 30.48\ D_0 + 152.4 + 8.64\ D_0 + 64.8 \tag{7-5-47}$$

$$M_{RS} = 0.43\ D_0^3 \tag{7-5-48}$$

Calculate embedment depth:

$$M_{DR} = 0.0113\ D_0^3 + 0.508\ D_0^2 + 30.48\ D_0 + 152.4 + 8.64\ D_0 + 64.8 \tag{7-5-49}$$

$$D_0^3 - 1.2133\ D_0^2 - 93.432\ D_0 - 518.75 = 0 \tag{7-5-50}$$

Solving for **D_o**: $D_0 = 12.272$ ft (7-5-51)

Determine **D** to account for pile embedment required below point O.

$$D = 1.2D_0 = 14.73\ \text{ft} \tag{7-5-52}$$

Note that this embedment depth, **D**, does not have a safety factor applied.

Calculate maximum moment:

The maximum moment is located at distance **Y** below the excavation line where the shear is equal to zero. Therefore, the summation of horizontal forces at the distance **Y** must be set to equal zero.

$$\sum F_x = 0 \tag{7-5-53}$$

$$1.2571Y^2 - 1.016Y - 39.12 = 0 \tag{7-5-54}$$

$$Y^2 - 0.808Y - 31.119 = 0 \quad => \quad Y = 5.997 \text{ ft (below the dredge line)} \tag{7-5-55}$$

$$V_{max} = 1.291 \times 12.272^2 - 8.64 - 30.48 - 1.016 \text{ x } 12.272 - 5.11 = 137.729 \text{ kips} \tag{7-5-56}$$

$$M_{max} = \left\{ \begin{array}{c} 8.64\times(7.5+5.997)+30.48\times(5+5.997)+1.016\times5.997\times\left(\frac{5.997}{2}\right) \\ +0.0339\times5.997^2\times\left(\frac{5.997}{3}\right) \\ -1.291\times5.997^2\times\left(\frac{5.997}{3}\right) \end{array} \right\} \tag{7-5-57}$$

$$M_{max} = 379.697 \text{ kip-ft} \tag{7-5-58}$$

Figure 7-26 displays the shear and moment diagram using the Simplified Method, and Figure 7-27 displays the deflection diagram.

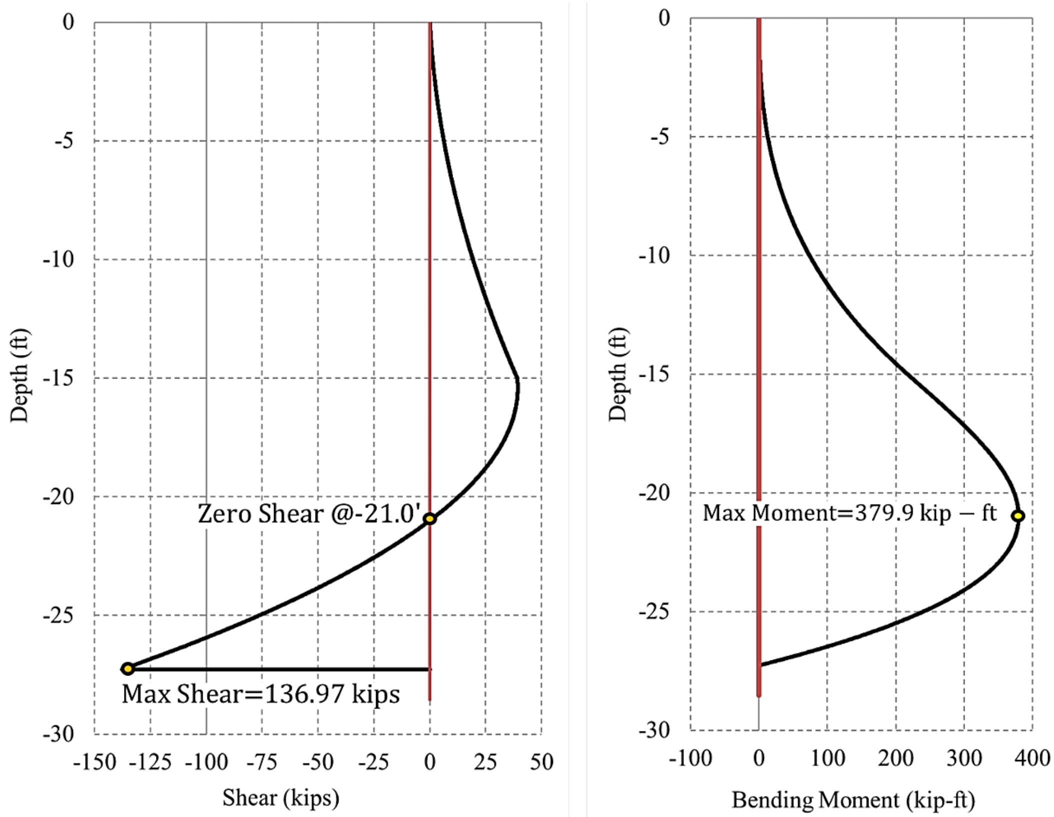


Figure 7-26. Shear and Moment Diagram (CT_T&S Program)

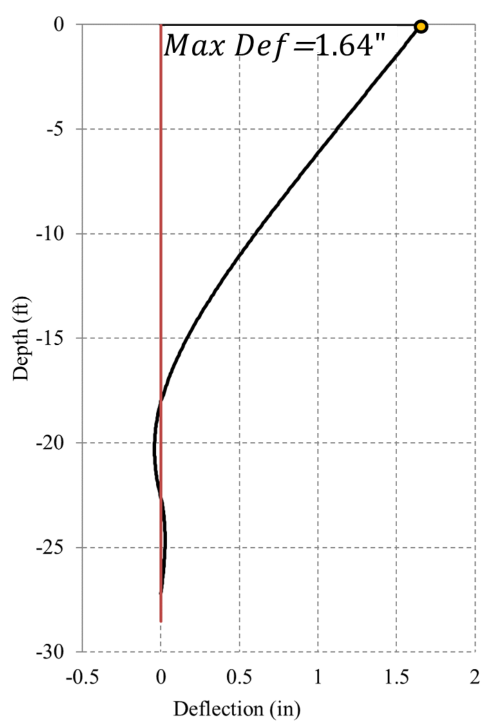


Figure 7-27. Deflection by Simplified Method (CT_T&S Program)

Table 7-3 below is a summary of these two methods of shoring system analysis. The Simplified Method is inherently slightly more conservative for embedment depth and shear. As a reminder to the reader, both the Rigorous and the Simplified methods only apply to unrestrained (cantilevered) shoring systems.

Table 7-3. Comparison of Results Between Simplified Method and Conventional Method

Characteristic	Conventional	Simplified
Depth, **D** (ft)	13.53	*14.73
Shear, **V** (kips)	91.28	136.97
Moment, **M** (kip-ft)	379.90	379.70
Deflection Δ (in)	1.73	1.64

Note: Simplified depth **D**, is increased by 20% (**D** = 1.2D_o, where D_o = 12.3). The depth, **D**, shown above only used a safety factor of 1.0.

7-5.06 Example 7-2: Cantilevered Soldier Pile Wall, Simplified Method with Two Soil Layers above the Excavation

For a shoring system subjected to the lateral load of two different soil layers above the excavation, calculate the total required horizontal force using Rankine earth pressure theory. See Example 5 from Appendix B, *Example Problems*, for a related example.

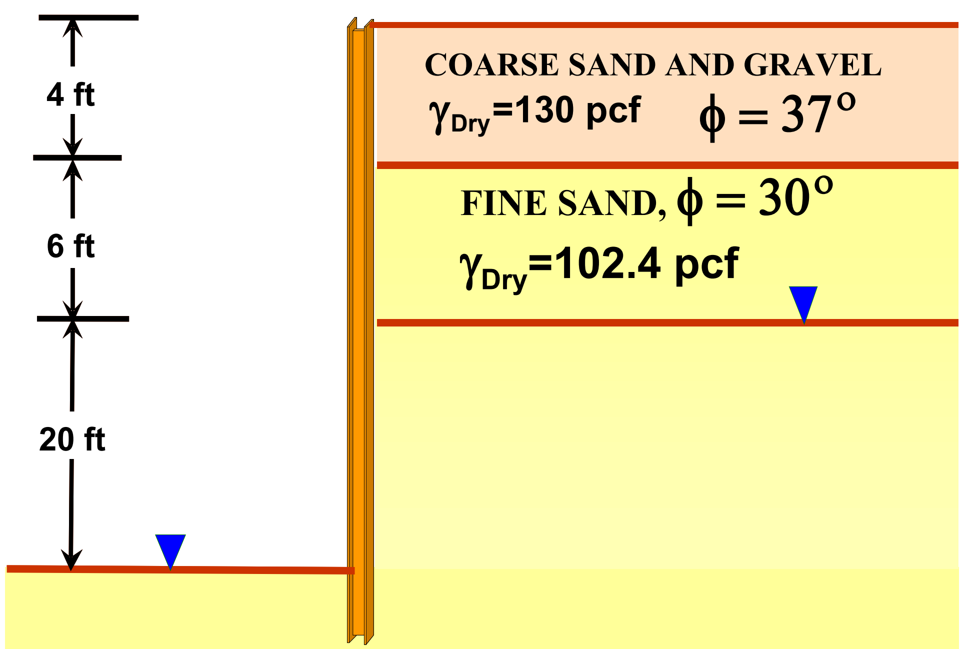


Figure 7-28. Diagram of Shoring Cross Section and Soil Properties

Determine:

- Calculate and plot earth pressure distribution.
- Calculate the total force on the shoring system.

Solution:

$$K_{a_1} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{37}{2}\right) = 0.249 \quad (7\text{-}5\text{-}59)$$

$$K_{a_2} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{30}{2}\right) = 0.333 \quad (7\text{-}5\text{-}60)$$

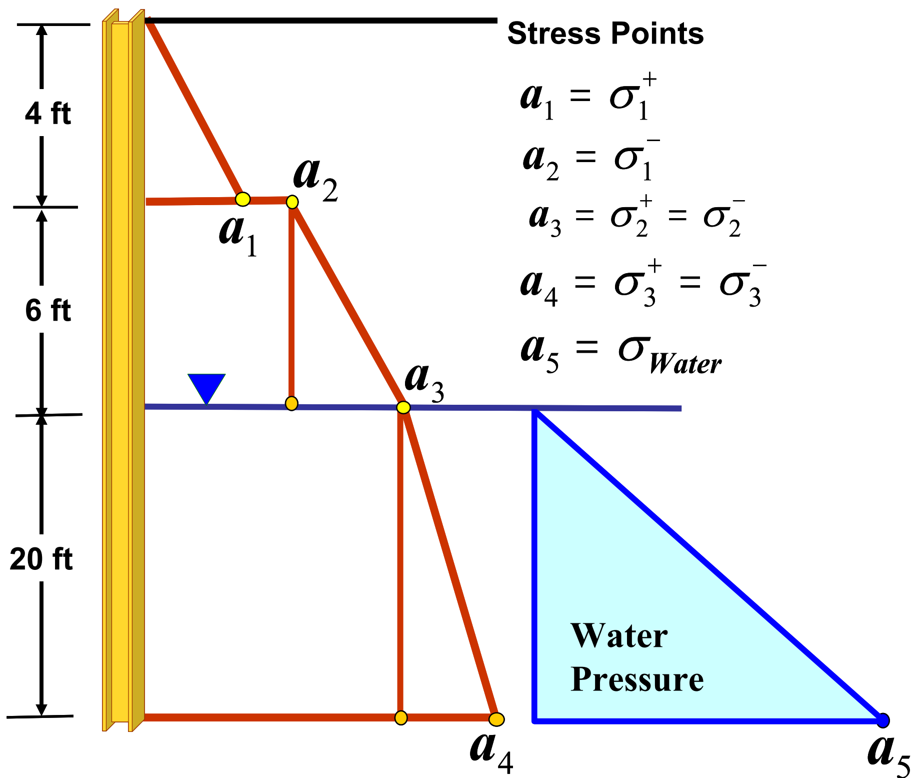


Figure 7-29. Pressure Loading Diagram

In Figure 7-29 above and the analysis below, the subscripted numbers refer to the soil layer. The superscripted **+** refers to the stress at the indicated soil layer due the material above the layer line based on $\mathbf{K_a}$ of that soil. The superscripted **–** refers to the stress at the indicated soil layer for the material above the layer line based on the $\mathbf{K_a}$ of the soil below the layer line.

$$\sigma_1^+ = (130\text{pcf})(4\text{ft})(0.249) = 129.48\text{ psf} \quad (7\text{-}5\text{-}61)$$

$$\sigma_1^- = (130\text{pcf})(4\text{ft})(0.333) = 173.16\text{ psf} \quad (7\text{-}5\text{-}62)$$

$$\sigma_2^+ = 173.16 + (102.40\text{pcf})(6\text{ft})(0.333) = 377.76\text{ psf} \quad (7\text{-}5\text{-}63)$$

$$\sigma_2^+ = \sigma_2^- = 377.76\text{ psf} \quad (7\text{-}5\text{-}64)$$

$$\sigma_3^+ = \sigma_3^- = 377.76 + (102.40 - 62.40)(20)(0.333) = 644.16\text{ psf} \quad (7\text{-}5\text{-}65)$$

Water Pressure (at excavation line):

$$\sigma_{a5} = 20(62.4\text{pcf}) = 1{,}248.0\text{ psf} \quad (7\text{-}5\text{-}66)$$

Driving Forces:

$$F_1 = \frac{1}{2}(4\text{ft})(129.48\text{ psf}) = 258.96\text{ lb/ft} \quad (7\text{-}5\text{-}67)$$

$$F_2 = (6\text{ft})(173.16\text{ psf}) = 1{,}038.96\text{ lb/ft} \quad (7\text{-}5\text{-}68)$$

$$F_3 = \frac{1}{2}(6\text{ft})(377.76 - 173.16\text{ psf}) = 613.80\text{ lb/ft} \quad (7\text{-}5\text{-}69)$$

$$F_4 = (20\text{ft})(377.76\text{ psf}) = 7{,}555.20\text{ lb/ft} \quad (7\text{-}5\text{-}70)$$

$$F_5 = \frac{1}{2}(20\text{ft})(644.16 - 377.76\text{ psf}) = 2{,}664.00\text{ lb/ft} \quad (7\text{-}5\text{-}71)$$

$$F_6 = \frac{1}{2}(20\text{ft})(1248\text{ psf}) = 12{,}480\text{ lb/ft} \quad (7\text{-}5\text{-}72)$$

Net Forces:

$$F_{TOTAL} = 24{,}610.92\text{ lb/ft} \quad (7\text{-}5\text{-}73)$$

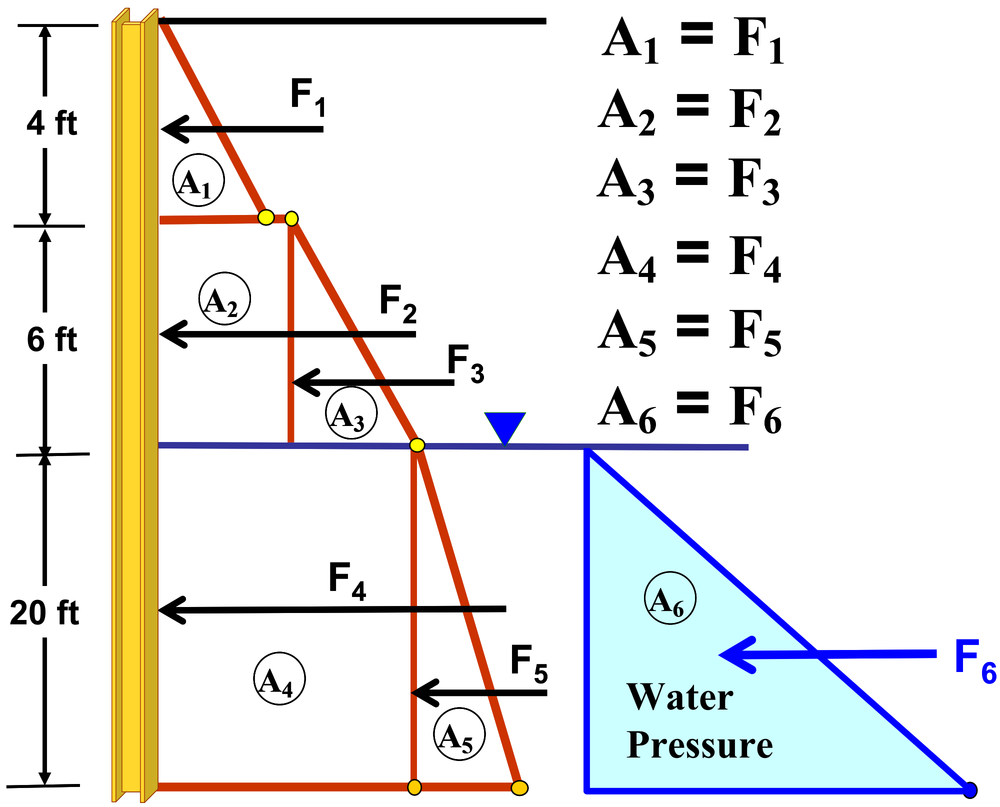


Figure 7-30. Force Loading Diagram

7-5.07 Example 7-3: Cantilevered Soldier Pile Wall, Simplified Method

Using the Simplified Method, check the adequacy of the cantilevered soldier pile wall in granular 2-layered soil with a negative slope in the front of the wall. The soldier pile is an HP12X84, 50 ksi steel beam placed in a 2-foot diameter hole filled with 4-sack concrete. Refer to Figure 7-31.

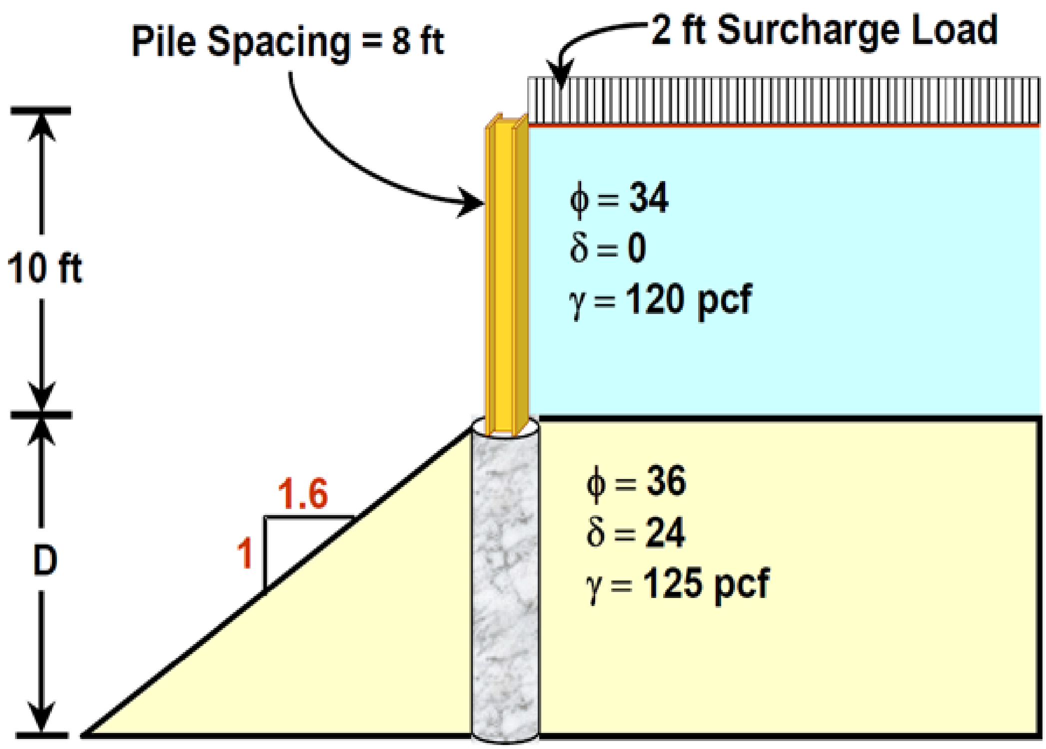


Figure 7-31. Soldier Pile Wall with Sloping Ground, Example 7-3

Determine:

1. Active & passive earth pressures
2. Pile embedment, **D**
3. Maximum moment.

Solution:

Calculate the active & passive earth pressures for layer 1:

$$K_{a_1} = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{34}{2}\right) = 0.283 \qquad (7\text{-}5\text{-}74)$$

Use Coulomb theory to calculate active earth pressure in layer 2. Note that due to layer 2 having a friction angle, **δ**, between the soil and the shoring, the active and passive pressures act at an angle (to the horizontal), and thus will need to be converted to the horizontal value.

$$K_{a2} = \frac{\cos^2\phi}{\cos\delta\left[1+\sqrt{\frac{\sin(\delta+\phi)\sin\phi}{\cos\delta}}\right]^2} \tag{7-5-75}$$

$$K_{a2} = \frac{\cos^2(36)}{\cos(24)\left[1+\sqrt{\frac{\sin(24+36)\sin(36)}{\cos(24)}}\right]^2} = 0.235 \tag{7-5-76}$$

The passive horizontal earth pressure coefficient $\mathbf{K_{ph}}$ is calculated using the method outlined in Section 4-6, *Log-Spiral Passive Earth Pressure,* and Figure 4-20, as shown below:

Calculate **δ/ϕ**:

$$\frac{\boldsymbol{\delta}}{\boldsymbol{\phi}} = \frac{24}{36} = 0.67 \tag{7-5-77}$$

Calculate **β/ϕ**:

$$\frac{\boldsymbol{\beta}}{\boldsymbol{\phi}} = -\frac{32}{36} = -0.89 \tag{7-5-78}$$

Where beta (**β)** is the slope of the backfill.

Use Log-Spiral to determine the passive soil coefficient, $\mathbf{K_p}$**,** using Figure 4-20. Determine $\mathbf{K_p}$ from Figure 4-20: $\mathbf{K_p = 1.65}$

Determine the reduction factor **R**, using the ratio of **δ/ϕ** (thru interpolation of Figure 4-20): **R = 0.8**

$$\mathbf{K'_p} = \mathbf{K_p} \times \ \mathbf{R} = 1.65 \times 0.8 = \ 1.32 \tag{7-5-79}$$

$\mathbf{K_p}$ is acting at an angle due to the wall friction angle, δ, of 24 degrees. Thus, calculate $\mathbf{K_p}$ acting in the horizontal, $\mathbf{K_{ph}}$:

$$\mathbf{K_{ph}} = \mathbf{K'_p} \times \ \mathbf{cos(\boldsymbol{\delta})} = 1.32 \times \ \cos(24) = 1.20 \tag{7-5-80}$$

Where:

δ = Friction angle between soil and shoring members (in this case, it is with the embedded piles).

Φ = Effective friction angle of soil.

K_a = Coefficient of active lateral earth pressure.

K_p = Coefficient of passive lateral earth pressure.

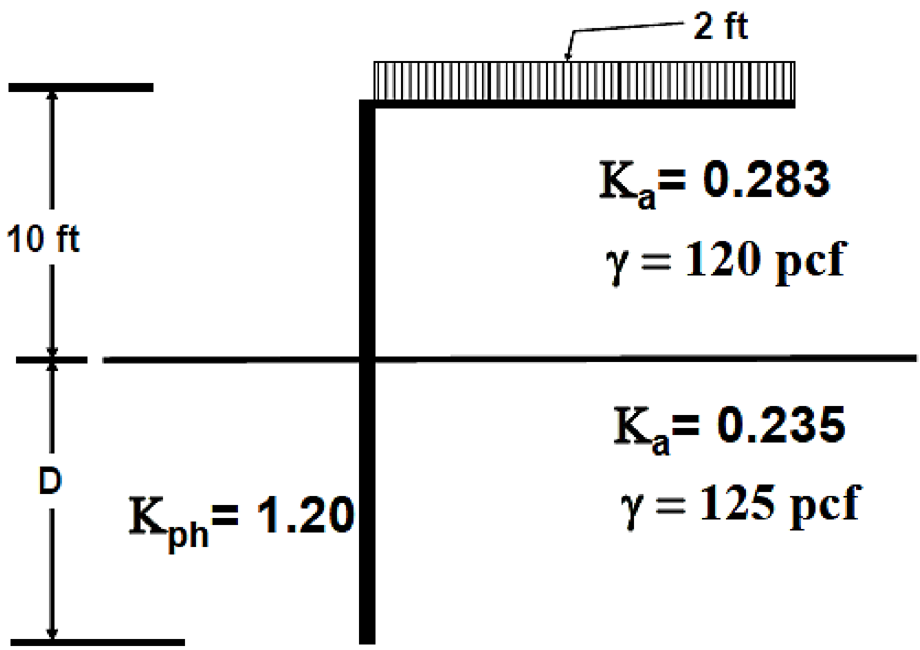


Figure 7-32. Active and Passive Earth Pressure Coefficients

Calculate earth pressure distribution:

Lateral load due to surcharge above the excavation line only:

$$\sigma_{Sur} = (120) \times (2) \times (0.283) = 68 \text{ psf; use } 72 \text{ psf minimum} \qquad (7\text{-}5\text{-}81)$$

Note: Surcharge is assumed to act uniformly for the top 10 feet only.

Lateral load distribution for the first layer:

$$\sigma^{+} = 72 + [(120) \times (10) \times (0.283)] = 411.6 \text{ psf; use } 412 \text{ psf} \qquad (7\text{-}5\text{-}82)$$

Lateral load distribution for the second layer at the soil boundary:

$$K_{ah} = K_a \times \cos(\delta) = 0.235 \times \cos(24) = 0.215 \quad (7\text{-}5\text{-}83)$$

$$\sigma^{-} = (120) \times (10) \times (0.215) = 258 \text{ psf} \quad (7\text{-}5\text{-}84)$$

Lateral load distribution for the second layer at depth **D**:

$$\sigma_D = 258 + (125)(0.215)D = (258 + 26.88D) \text{ psf} \quad (7\text{-}5\text{-}85)$$

Passive lateral load distribution for the second layer in the front of depth **D**:

$$\sigma_{PD} = (125)(1.2)D = 150D \text{ psf} \quad (7\text{-}5\text{-}86)$$

Calculate active earth pressure due to surcharge $\mathbf{P_{AS}}$:

$$P_{AS} = 72 \times 10 = 720 \text{ plf} \quad (7\text{-}5\text{-}87)$$

Calculate active earth pressure for the first soil layer $\mathbf{P_{A1}}$:

$$P_{A1} = \left[(412 - 72)\left(\frac{10}{2}\right)\right] = 1700 \text{ plf} \quad (7\text{-}5\text{-}88)$$

Calculate active earth pressure for the second soil layer $\mathbf{P_{A2}}$:

$$P_{A21} = 258D \text{ plf} \quad (7\text{-}5\text{-}89)$$

$$P_{A22} = \left[26.88 \times D \times \left(\frac{D}{2}\right)\right] = 13.44D^2 \text{ plf} \quad (7\text{-}5\text{-}90)$$

Calculate passive earth pressure for the second soil layer $\mathbf{P_p}$:

$$P_P = \left[150 \times D \times \left(\frac{D}{2}\right)\right] = 75D^2 \text{ plf} \quad (7\text{-}5\text{-}91)$$

Because the pile spacing is equal to 4 times the effective width of the pile and the arching factor is limited to a maximum of 3, the arching factor will be used to increase the effective pile width for the passive forces below the dredge line. Only the effective width of the pile **should be used** for the active forces below the dredge line because the arching factor is not applicable there. Figure 7-33 provides the total pressure for each area per foot of wall. Use these in reference to Table 7-4 and Table 7-5 for calculating the Driving and Resisting moments.

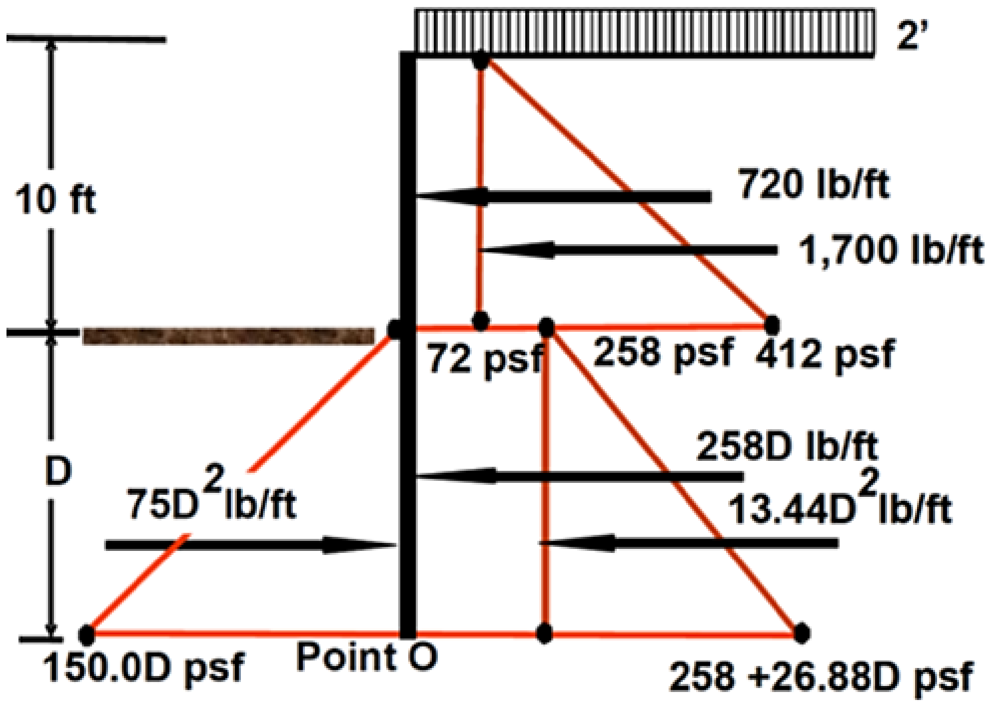


Figure 7-33. Pressure Diagram

Table 7-4. Calculate Driving Moment (M_{DR}) about Point "O" using Figure 7-33

Driving Force = **P_a** × Spacing	Moment Arm(ft.)	Driving Moment **M_{DR}**
720 × 8 = 5760	5+D	5760D+28800
1700 × 8 = 13600	10/3+D	13600D+45333
258D × 2 = 516D	D/2	258 D^2
13.44D^2 × 2 = 26.88D^2	D/3	8.96 D^3

Table 7-5. Calculate the Resisting Moment (M_{RS}) about Point "O" using Figure 7-33

Resisting Force = **P_p** × Spacing	Moment Arm(ft.)	Resisting Moment **M_{RS}**
75D^2 × (5.76) = 432D^2	D/3	144 D^3

$$M_{DR} = 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}92)$$

$$M_{RS} = 144D^3 \quad (7\text{-}5\text{-}93)$$

For the external stability use the safety factor of 1.3 to calculate the embedment depth and then increase it by 20 percent to account for the Simplified Method.

$$\frac{M_{RS}}{M_{DR}} = 1.3 \quad (7\text{-}5\text{-}94)$$

$$\left(\frac{144}{1.3}\right)D^3 - 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}95)$$

$$D^3 - 2.53D^2 - 190.2D - 728.2 = 0 \ \longrightarrow D = 16.6\text{ ft} \quad (7\text{-}5\text{-}96)$$

Increase **D** by 20 percent:

$$16.6 \times 1.20 \longrightarrow D = 19.9\text{ ft} \quad (7\text{-}5\text{-}97)$$

Calculate embedment depth using a factor of safety (FS) equal to 1.0 for the purposes of calculating the shear and moments.

$$\frac{M_{RS}}{M_{DR}} = 1.0 \quad (7\text{-}5\text{-}98)$$

$$144D^3 - 8.96D^3 - 258D^2 - 19360D - 74134 = 0 \quad (7\text{-}5\text{-}99)$$

$$D^3 - 1.91D^2 - 143.3D - 549 = 0 \ \longrightarrow D = 14.4\text{ ft} \quad (7\text{-}5\text{-}100)$$

Increase **D** by 20 percent:

$$14.4 \times 1.20 \rightarrow D = 17.3\text{ ft} \quad (7\text{-}5\text{-}101)$$

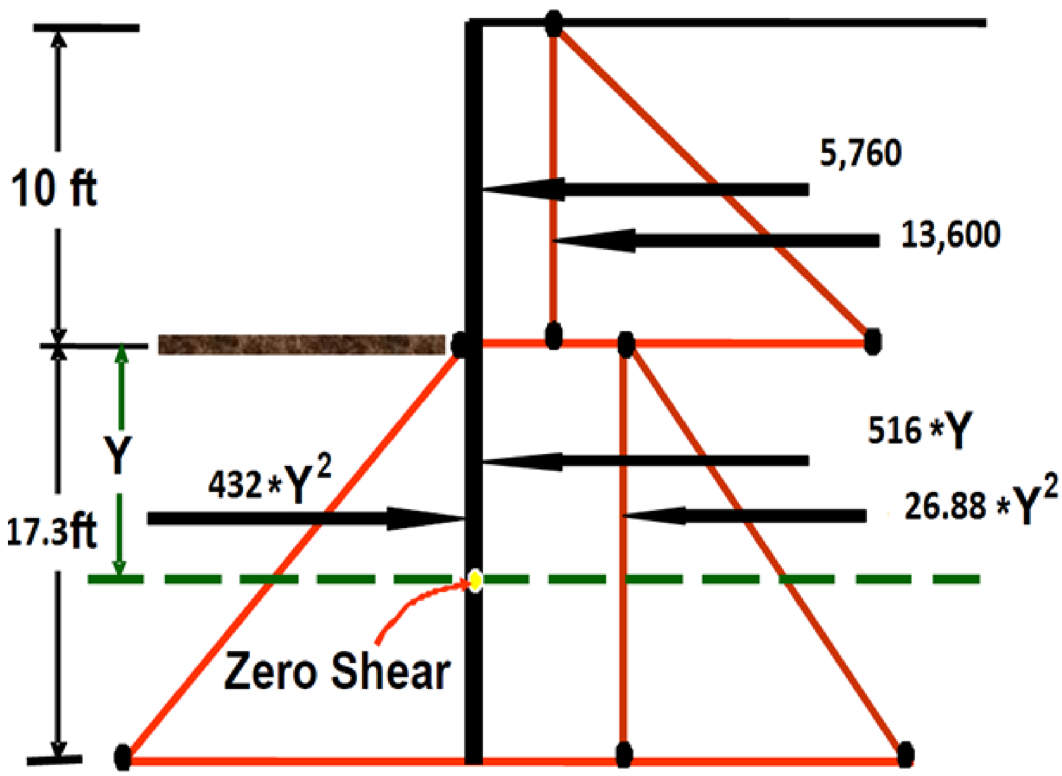


Figure 7-34. Location of Zero Shear and Maximum Moment

Calculate zero shear location using Figure 7-34:

$$432\mathrm{Y}^2 - 26.88\mathrm{Y}^2 - 516\mathrm{Y} - 19360 = 0 \quad (7\text{-}5\text{-}102)$$

$$\mathrm{Y}^2 - 1.3\mathrm{Y} - 47.8 = 0 \quad (7\text{-}5\text{-}103)$$

$$\mathrm{Y} = 7.59 \text{ ft Below the dredge line} \quad (7\text{-}5\text{-}104)$$

Based on zero shear location, maximum moment can be calculated as below:

$$\mathrm{M_{max}} = 5760(5 + 7.59) + 13600\left(\frac{10}{3} + 7.59\right) + (516 \times 7.59)\left(\frac{7.59}{2}\right) + (26.88 \times 7.59^2)\left(\frac{7.59}{3}\right) - (432 \times 7.59^2)\left(\frac{7.59}{3}\right) \quad (7\text{-}5\text{-}105)$$

$$\mathrm{M_{max}} = 176893 \text{ lb-ft} \quad (7\text{-}5\text{-}106)$$

$$F_b = 0.60F_y = 0.60(50\ \text{ksi}) = 30\ \text{ksi} \tag{7-5-107}$$

Recall: S_x = 106 in^3 for an HP 12x84

$$S_{Required} = \frac{M_{max}}{F_b} = \frac{176.893\ \times\ 12}{30} = 70.8\ \text{in}^3 < 106\ \text{in}^3\ :\ \text{O.K} \tag{7-5-108}$$

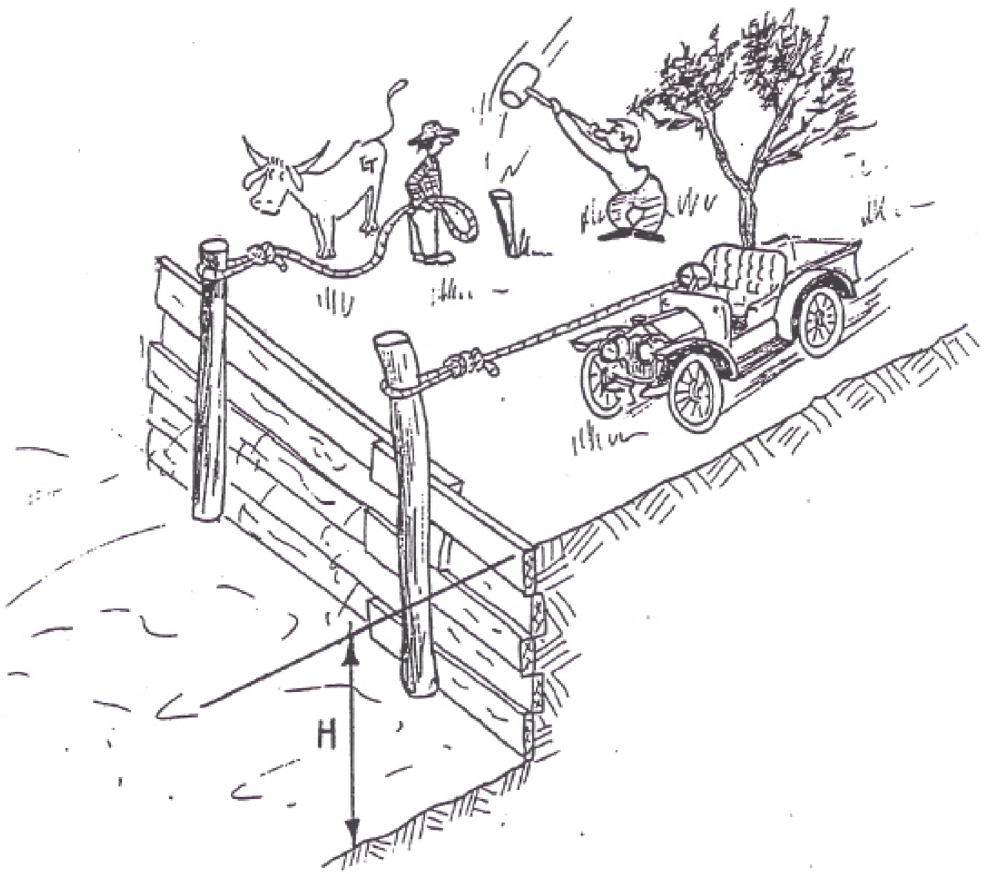
In summary, using the Simplified Method, the adequacy of the 10-foot cantilevered soldier pile wall for a granular 2-layered soil was completed to calculate the active and passive earth pressures, the pile embedment (**D**), and the maximum moment. Deflection and structural integrity of the lagging material were not included in this example.

The pile embedment required was determined to be 19.9 feet with a safety factor of 1.3. Use this value to compare with the Contractor's value in the shop drawings. The calculated maximum moment was used to verify the adequacy of the steel beam provided and was acceptable. Keep in mind that the calculations would have to be done if any of the variables change, such as excavation height, pile spacing, beam type, or soil conditions. See Example 4 from Appendix B, *Example Problems*, for a related example.

JULY 2025

CHAPTER 8

RESTRAINED SHORING SYSTEMS



Chapter 8: Restrained Shoring Systems

Table of Contents

Chapter 8: Restrained Shoring Systems 1
Table of Contents 1
8-1 Lateral Earth Pressures for Restrained Shoring Systems 2
8-2 Cohesionless Soils 2
8-3 Cohesive Soils 5
8-3.01 Stiff to Hard 5
8-3.02 Soft to Medium Stiff 5
8-4 Calculation Procedures 7
8-4.01 Single Ground Anchor/Brace System 7
8-4.02 Multiple Ground Anchor/Brace System 8
8-4.03 Deflection 10
8-4.04 Example 8-1, Single Ground Anchor Sheet Pile Wall 11
8-4.05 Example 8-2 Multiple Ground Anchor Soldier Pile Wall 21

8-1 Lateral Earth Pressures for Restrained Shoring Systems

An anchored wall of exposed height (**H**) over which soil is retained with an embedded depth (**D**) will include either structural supports (braced) or ground anchors, as illustrated in Figure 8-1. An anchored wall type is usually used when the restrained height is so large that it is impractical to have a cantilevered system, or when the estimated lateral displacement of a cantilevered system is too large, considering existing structures or utilities behind the wall.

The lateral earth pressure acting on the wall increases with depth and a triangular pressure is used for cantilevered systems. For braced or ground anchor walls this is not the case. A trapezoidal-shaped apparent earth pressure distribution needs to be developed for this type of wall system.

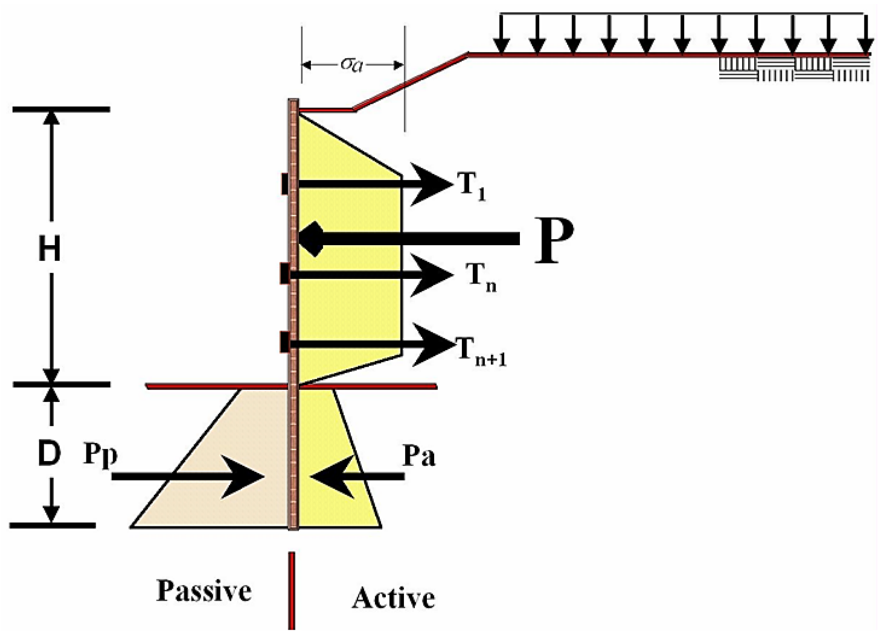


Figure 8-1. Lateral Earth Pressure for Anchored/Braced Wall

8-2 Cohesionless Soils

The lateral earth pressure distribution for braced or anchored walls constructed in cohesionless soils may be determined using Figure 8-2 for single braced/ground anchor walls and Figure 8-3 for multiple braced/ground anchor walls.

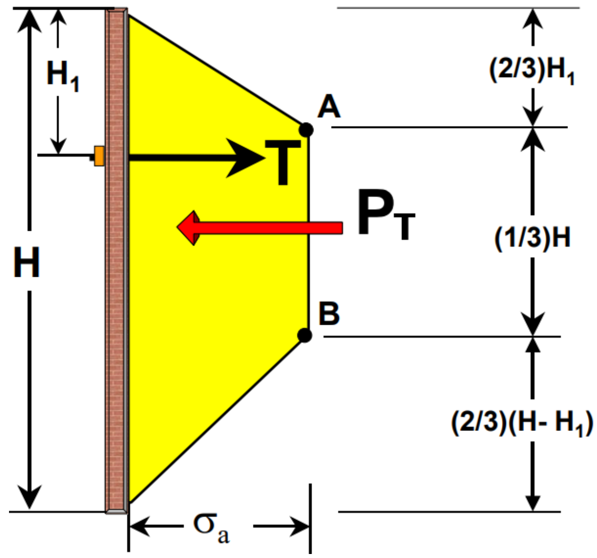


Figure 8-2. Pressure Diagram for Single Anchored/Braced Wall in Cohesionless Soil

The maximum ordinate (σ_a) of the pressure diagram is determined as follows:

For walls with a single level of anchors or braces:

$$\sigma_a = \frac{f \times P}{\left(\frac{2}{3}\right)H} = \frac{1.3 \times P}{\left(\frac{2}{3}\right)H} \quad \text{(8-2-1)}$$

The factor **f** is a constant used to convert a triangular pressure distribution to a trapezoidal pressure distribution. Note that although the triangular pressure distributions are not shown for Figures 8-2 and 8-3, they are important in calculating the resultant forces for the trapezoidal pressure distributions, as illustrated.

The resultant force for the triangular pressure distribution is:

$$P = \left(\frac{1}{2}\gamma\right)(H^2)(K_a) \quad \text{(8-2-2)}$$

The resultant force for the trapezoidal pressure distribution is:

$$P_T = 0.65[\gamma(H^2)(K_a)] \quad \text{(8-2-3)}$$

Setting the triangular pressure distribution equal to trapezoidal pressure distribution:

$$P_T = \left[\frac{1}{2}\gamma(H^2)(K_a)\right]f = 0.65[\gamma(H^2)(K_a)] \Rightarrow f = \frac{0.65\gamma[(H^2)(K_a)]}{\left(\frac{1}{2}\gamma\right)[(H^2)(K_a)]} = 1.3$$

(8-2-4)

Therefore:

$$\mathbf{P_T = 1.3P} \tag{8-2-5}$$

The lateral active horizontal earth pressure for the multilevel anchored wall is shown in Figure 8-3:

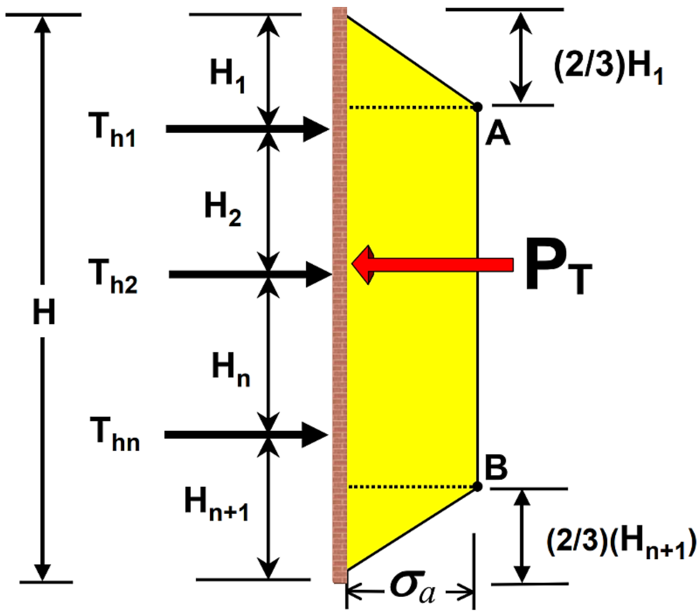


Figure 8-3. Pressure Diagram for Multi-Anchored/Braced Wall in Cohesionless Soil

Equation 8-2-6 may be used to calculate the active earth pressure for the multiple ground anchor wall.

$$\boldsymbol{\sigma_a = \frac{1.3P}{[H - 1/3(H_1 + H_{n+1})]}} \tag{8-2-6}$$

Where:

- $\boldsymbol{\sigma_a}$ = Maximum ordinate of pressure diagram
- γ = Unit weight of the soil
- $\mathbf{K_a}$ = Coefficient of active earth pressure
- **P** = Resultant lateral load from the triangular earth pressure distribution
- $\mathbf{P_T}$ = Resultant lateral load from the trapezoidal earth pressure distribution
- **H** = Wall height
- $\mathbf{H_1}$ = Distance from ground surface at top of wall to uppermost level of anchors
- $\mathbf{H_{n+1}}$ = Distance from the grade at bottom of a wall to lowermost level of anchors
- **n** = Number of anchors
- $\mathbf{T_{hn}}$ = Horizontal component of the anchor force at level **n**

8-3 Cohesive Soils

The lateral earth pressure distribution for cohesive soils is related to the stability number ($\mathbf{N_S}$), which is defined as:

$$\mathbf{N_s} = {}^{\boldsymbol{\gamma}\mathbf{H}}/_{\mathbf{c_u}} \tag{8-3-1}$$

Where:

γ = Total unit weight of soil

H = Wall height

$\mathbf{c_u}$ = Average undrained shear strength of soil = average undrained cohesion of soil

Note: The undrained cohesion is equal to the unconfined compressive strength divided by 2. This is true when the undrained angle of internal friction phi (ϕ) is equal to zero degrees.

8-3.01 Stiff to Hard

For braced or anchored walls in stiff to hard cohesive soils with the stability number ($\mathbf{N_s}$) less than or equal to 4, the lateral earth pressure may be determined using Figure 8-4, with the maximum ordinate ($\boldsymbol{\sigma_a}$) of the pressure diagram determined as:

$$\boldsymbol{\sigma_a} = \mathbf{0.2}(\boldsymbol{\gamma})(\mathbf{H}) \text{ to } \mathbf{0.4}(\boldsymbol{\gamma})(\mathbf{H}) \tag{8-3-2}$$

Where:

$\boldsymbol{\sigma_a}$ = Maximum ordinate of trapezoidal pressure diagram

γ = Total unit weight of soil

H = Wall height

Note that the stability number is a dimensionless variable. A higher stability number ($\mathbf{N_s}$) means a slope requires a higher shear strength to maintain stability. As such, in situ actual field condition needs to be determined and best engineering judgment is needed to use the applicable factor between 0.2 and 0.4.

8-3.02 Soft to Medium Stiff

The lateral earth pressure on a restrained shoring system in soft to medium stiff cohesive soils with the stability number ($\mathbf{N_s}$) equal to or larger than 6 may be determined using Figure 8-4, for which the maximum ordinate ($\boldsymbol{\sigma_a}$) of the pressure diagram is determined as:

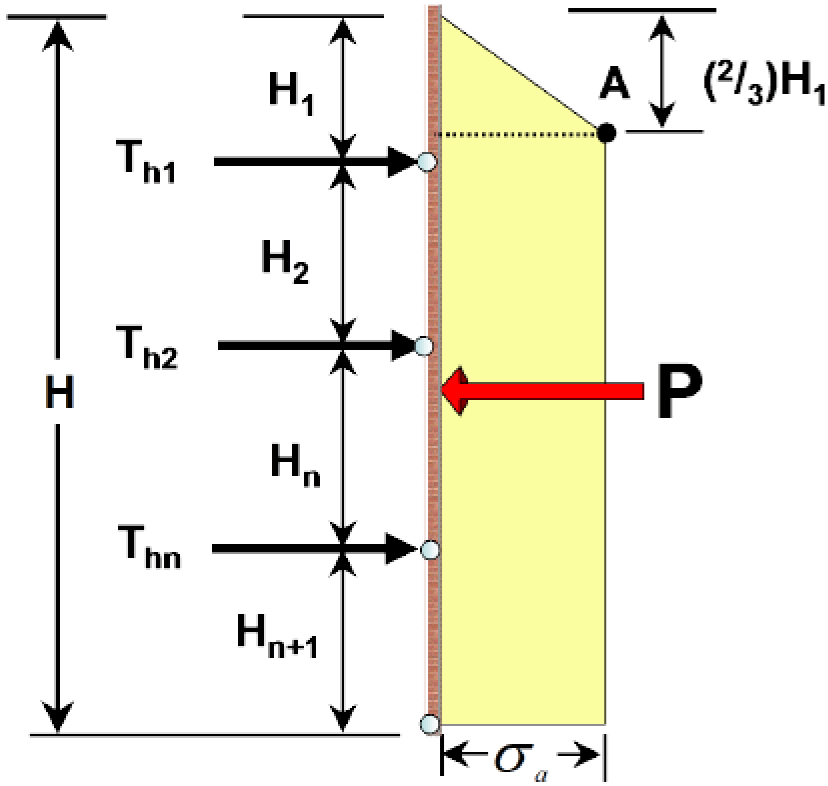


Figure 8-4. Pressure Diagram for Multi Anchored/Braced Wall for Cohesive Backfill

$$\boldsymbol{\sigma_a = K_a \gamma H} \tag{8-3-3}$$

Where:

σ_a = Maximum ordinate of pressure diagram

K_a = Coefficient of active lateral earth pressure

γ = Total unit weight of soil

H = Wall height

For soils with **4 < N_S< 6**, use the larger σ_a from Equations 8-3-2 and 8-3-3

The coefficient of active lateral earth pressure (**K_a**) may be determined using Equation (8-3-4) per FHWA and not to be less than 0.25.

$$\boldsymbol{K_a = 1 - \frac{4(c_{u1})}{(\gamma)(H)} + 2\sqrt{2}\frac{D}{H}\left[1 - \frac{5.14(c_{u2})}{(\gamma)(H)}\right] \geq 0.25} \tag{8-3-4}$$

Where:

c_{u1} = Undrained shear strength of retained soil above the excavation line (the upper cohesive soil layer)

c_{u2} = Undrained shear strength of soil below the excavation line (the lower cohesive soil layer)

γ = Total unit weight of retained soil

H = Wall height

D = Depth from the grade in front of the wall to the potential failure surface below.

Be aware of the following, when using Equation 8-3-4. This equation addresses the condition where there are two layers of cohesive soil. The excavation is done in the upper layer of cohesive soil, and the lower layer of cohesive soil is directly under the excavation line. The lower level of cohesive soil has an undrained shear strength that is significantly less than the undrained shear strength for the upper layer of cohesive soil. For this equation, the results may be unexpected when the shear strength of the lower soil is approximately 80 percent or more of the shear strength of the upper soil. Although there are conditions where the $\mathbf{K_a}$ value from Equation 8-3-4 is negative, the equation gives reasonable results when the shear strength of the lower soil is significantly less than the shear strength of the upper soil.

8-4 Calculation Procedures

8-4.01 Single Ground Anchor/Brace System

The following procedure is used for the analysis of a single ground anchor/brace system wall including any surcharge as shown in Figure 8-5:

1. Determine the earth pressure coefficients using the classical earth pressure theories described in the previous section.
2. Convert the active earth pressure above the excavation line to a trapezoidal earth pressure.
3. Take moments about the ground anchor to calculate the embedment depth **D**, using a factor of safety of 1.3.
4. Take moments about the ground anchor to calculate embedment depth **D**, using a factor of safety of 1.0 to calculate ground anchor load **T** in the following step.
5. Set summation of forces equal to zero in horizontal direction to calculate ground anchor/brace force **T**.
6. Calculate maximum bending moment ($\mathbf{M_{MAX}}$) and maximum shear force ($\mathbf{V_{MAX}}$) to analyze the vertical structural member and lagging.

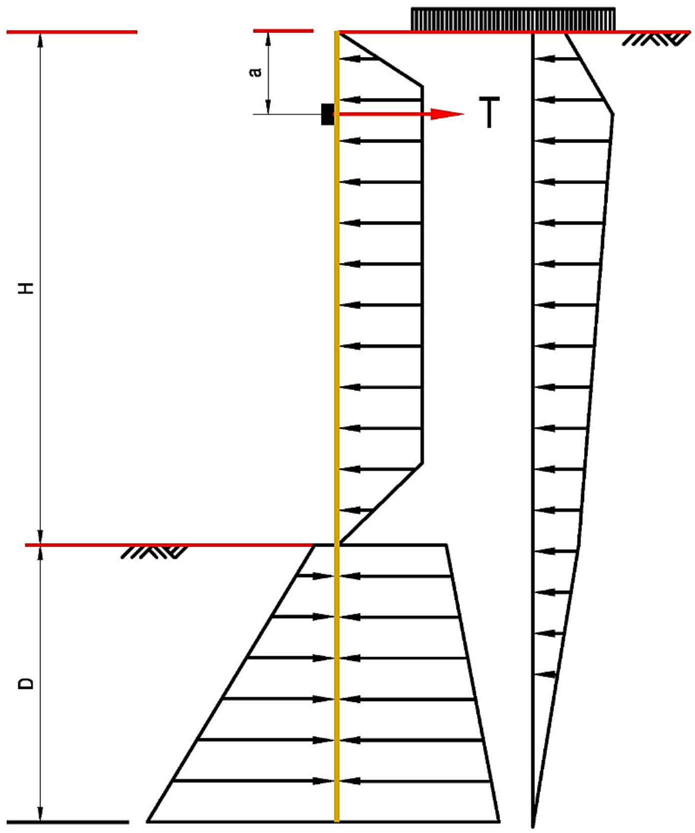


Figure 8-5. Single Ground anchor System

8-4.02 Multiple Ground Anchor/Brace System

Depending on the soil properties, use one of the trapezoidal soil's pressure diagrams shown in Figure 8-3 and Figure 8-4 for the analysis of multiple ground anchor systems. Figure 8-6 shows a simple trapezoidal pressure diagram for a multiple ground anchor system. The beam is divided into three types of spans.

- Starting Cantilever Span $\mathbf{S_1}$
- Interior Spans $\mathbf{S_n}$
- Embedment Span $\mathbf{S_D}$

Per FHWA, the two methods used to calculate the embedment depth, **D**, and ground anchor load, **T**, are the Hinge Method and the Tributary Area Method. The Tributary Area Method balances only summation of forces, which results in a large moment at the tip of the pile. The Hinge Method satisfies the force and moment equilibrium in that the shear and moment equal zero at the tip of the pile. Both finite element and beam spring models show the same trend.

The detailed procedure is shown and described below.

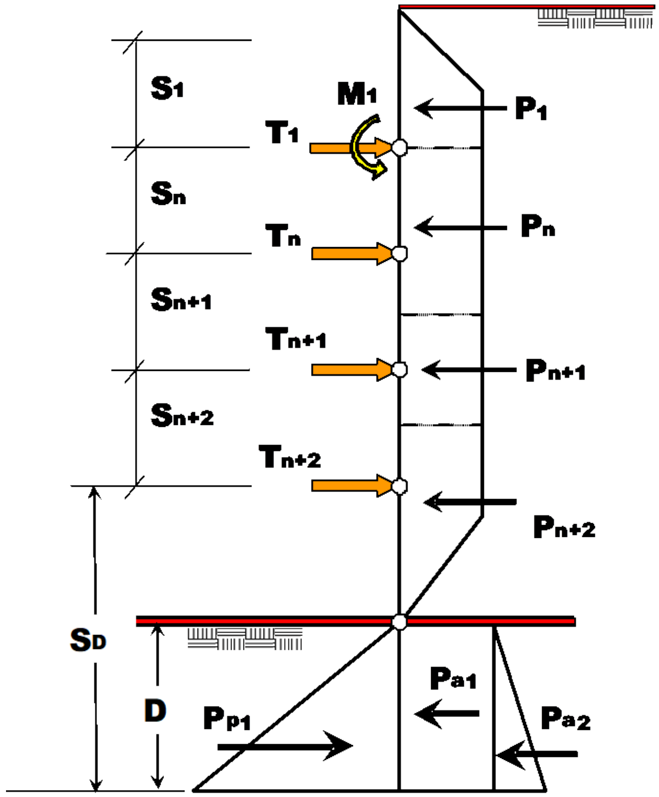


Figure 8-6. Multiple Ground Anchor System

P_a = Active lateral earth pressure below dredge line

P_p = Passive lateral earth pressure below dredge line

The Hinge Method as shown in Figure 8-6 and Figure 8-7 is used to solve multiple ground anchor/brace systems and includes the following steps:

1. Take moments **M_1** about the upper-level ground anchor due to cantilever action of the soil pressure above the upper ground anchor. The moments at the remaining ground anchors are assumed to be zero (0).
2. Use a combination of the moment **M_1** and tributary areas to calculate the remaining ground anchor loads, except the last ground anchor load.
3. Calculate last ground anchor load **T_{n+2}.**
   a. Calculate embedment depth **D** by taking moments about the last ground anchor. (Set Driving Moment = Resisting Moment.)
   b. Set summation of forces equal to zero in horizontal direction to calculate the last ground anchor load **T_{n+2}.**
4. Take moments about the last ground anchor to calculate embedment depth **D** using a factor of safety (**FS = 1.3**) for external stability.

$$\mathbf{M_1} = \text{Moment Due to load } \mathbf{P_1} \,\&\, \mathbf{P_2}$$

$$\mathbf{T_{1U}} = (\mathbf{P_1} + \mathbf{P_2})$$

$$\mathbf{T_{1L}} = \left(\frac{\mathbf{P_3}}{2} + \frac{\mathbf{M_1}}{\mathbf{S_1}}\right)$$

$$\mathbf{T_{2U}} = \left(\frac{\mathbf{P_3}}{2} - \frac{\mathbf{M_1}}{\mathbf{S_1}}\right)$$

$$\mathbf{T_{2L}} = \left(\frac{\mathbf{P_4}}{2}\right)$$

$$\mathbf{T_{3U}} = \left(\frac{\mathbf{P_4}}{2}\right)$$

$$\mathbf{T_{3L}} = \left(\frac{\mathbf{P_5}}{2}\right)$$

$$\mathbf{T_{4U}} = \left(\frac{\mathbf{P_5}}{2}\right)$$

$$\mathbf{T_{4L}} = (\mathbf{P_6} + \mathbf{P_7} + \mathbf{P_{a1}} + \mathbf{P_{a2}} - \mathbf{P_{p1}})$$

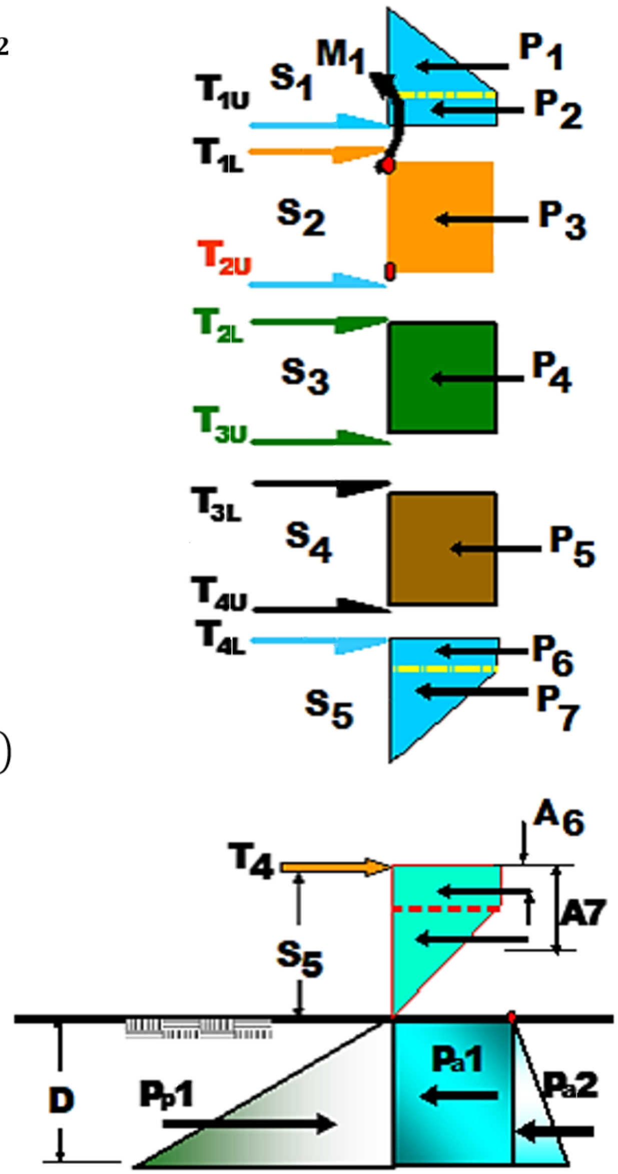


Figure 8-7. Detail Hinge Method for Ground Anchor Analysis

8-4.03 Deflection

A general discussion of deflection for unrestrained temporary shoring systems was described in the previous chapter. The same approach applies when calculating the deflection of restrained shoring systems. Similar to a simple beam analysis, the deflection at the supports along the vertical element of the shoring system is assumed to be zero (0), as shown in Figure 8-8. The point of fixity varies from 0.25**D** to 0.8**D** below the excavation level, and is a function of the effective pile diameter and soil type.

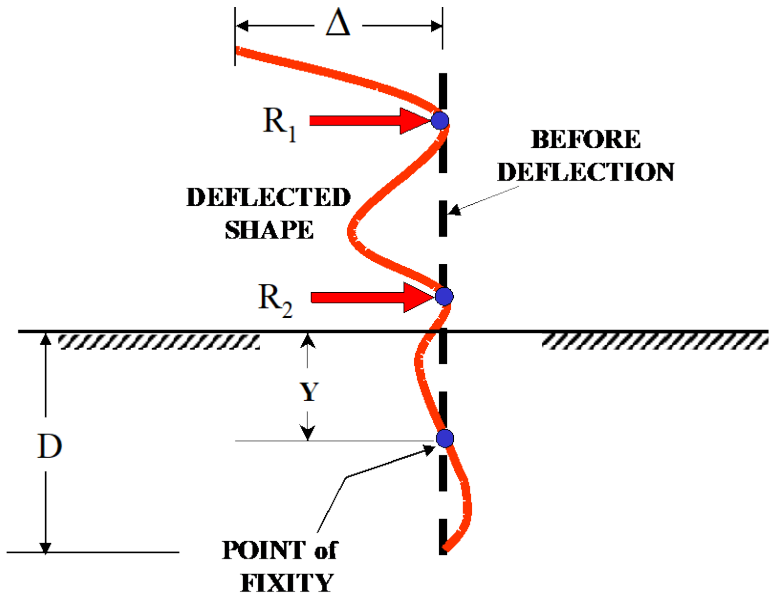


Figure 8-8 Deflected Shape for Restrained System

8-4.04 Example 8-1, Single Ground Anchor Sheet Pile Wall

Check the adequacy of a single ground anchor sheet pile wall with a single soil layer as illustrated below in Figure 8-9, with the ground anchors spaced at every 10 feet. Assume that the sheet pile section is a PZ22, with a steel grade of 42 ksi. Note that pressure and stress calculations will be performed on a typical one-foot strip of the sheet pile wall.

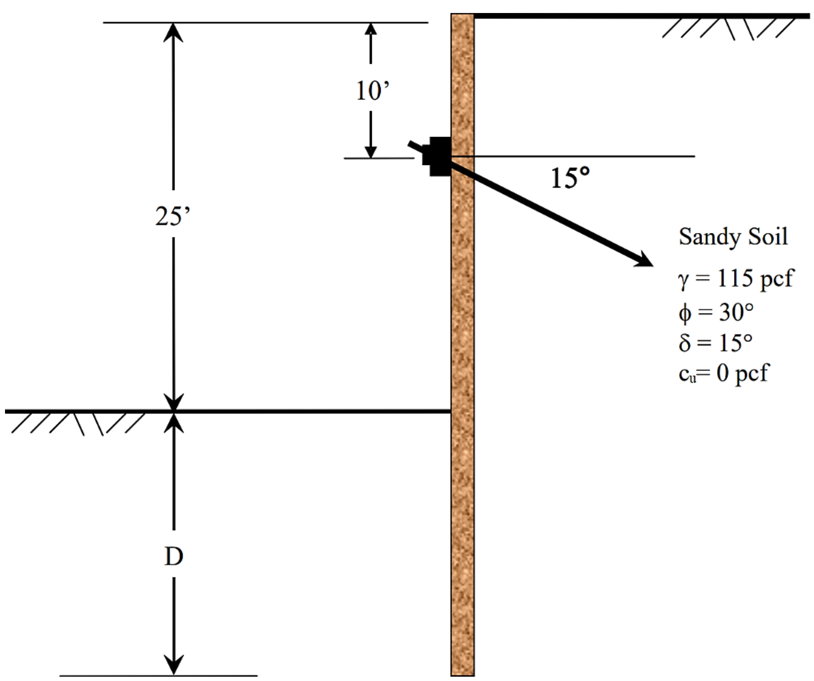


Figure 8-9. Example 8-1 Given Information

Determine:

1. Active & passive earth pressures
2. Pile embedment **D** with **FS = 1.3**
3. Ground anchor load with **FS = 1.0**
4. Maximum shear, maximum moment.

Structural properties of sheet pile section PZ22 are:

a. Section modulus per foot of wall width: **S** = 18.10 in^3
b. Moment of inertia per foot of wall width: **I** = 84.70 in^4
c. Radius of gyration per foot of wall width: **r** = 3.62 in
d. Area per foot of wall width:

$$\mathbf{A} = \frac{\mathbf{I}}{\mathbf{r^2}} = \frac{84.7 \text{ in}^4}{(3.62 \text{ in})^2} = 6.46 \text{ in}^2 \qquad (8\text{-}4\text{-}1)$$

Develop the pressure diagram:

From Rankine's Theory: $\mathbf{K_a}$ = 0.33. Using the method outlined in Section 4-6, *Log-Spiral Passive Earth Pressure*, and Figure 4-20, *Passive Earth Pressure Coefficient (Caquot and Kerisel, 1948)*, and considering the wall friction angle (δ) is equal to 15 degrees, obtain the following:

- $\boldsymbol{\phi} = 30°$, $\boldsymbol{\beta} = 0°$, yields $\beta/\phi = 0$
- initial $\mathbf{K_p} \approx 6.3$
- $-(\delta/\phi) = (-15/30) = -0.5$
- $\mathbf{R} = 0.746$

$$\mathbf{K_{ph}} = \mathbf{K_p} = 0.746 \text{ x } 6.3 = 4.7 \qquad (8\text{-}4\text{-}2)$$

The lateral earth pressure distribution for the analysis of anchored walls constructed in cohesionless soils may be determined using Figure 8-10.

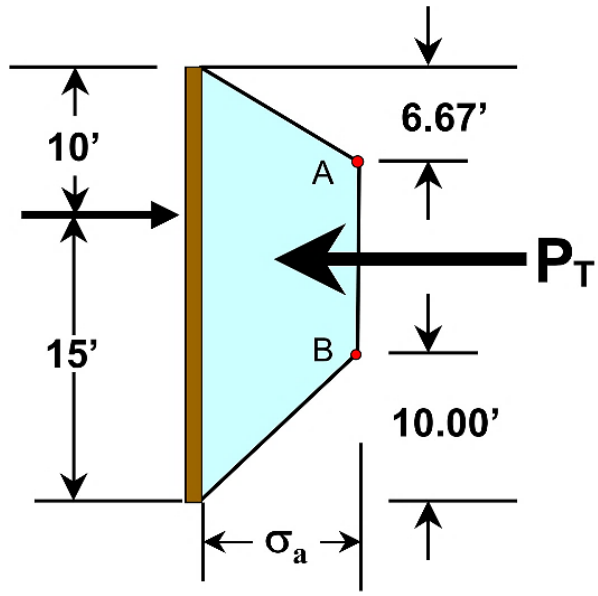


Figure 8-10. Pressure Diagram for Single Ground Anchor Wall

The maximum ordinate (σ_a) of the pressure diagram is determined as follows:

$$\boldsymbol{\sigma_a = \frac{1.3P}{(2/3)H} = \frac{P_T}{(2/3)H}} \qquad (8\text{-}4\text{-}3)$$

Where the total active earth pressure for a triangular pressure distribution is calculated as follows:

$$\mathbf{P = \frac{1}{2}\gamma H^2 K_a} \qquad (8\text{-}4\text{-}4)$$

Using Equation 8-2-5: $\mathbf{P_T = 1.3 \times P}$

$$P = \left(\frac{1}{2}\right)(115)(25^2)\left(\frac{1}{3}\right) = 11{,}980\ \text{lb/ft} \qquad (8\text{-}4\text{-}5)$$

$$P_T = 1.3P = 1.3 \times 11{,}980 = 15{,}574\ \text{lb/ft} \qquad (8\text{-}4\text{-}6)$$

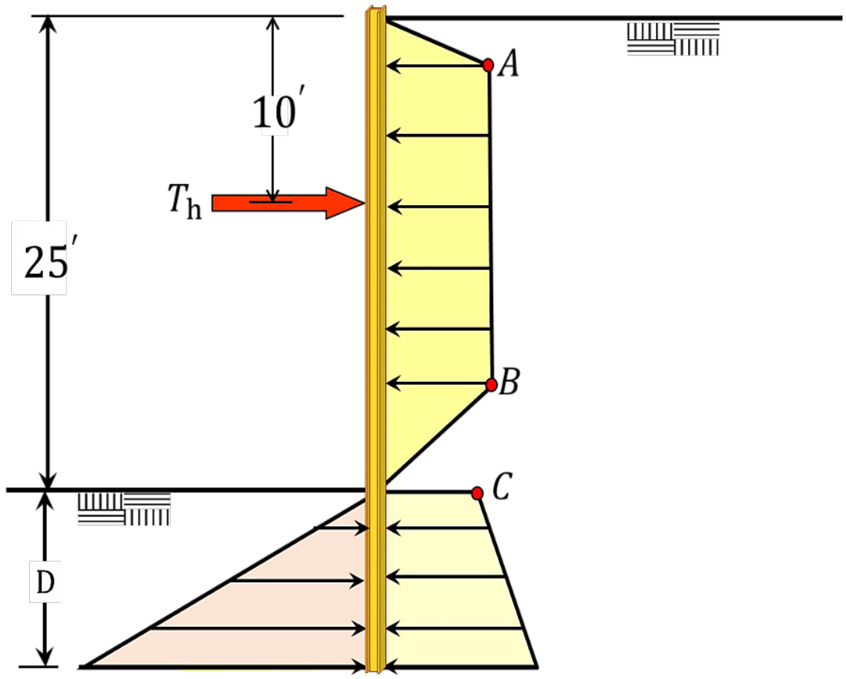


Figure 8-11. Pressure Diagram.

Active stress at the points A and B as shown in Figure 8-11:

$$\sigma_a = \frac{15{,}574}{(^2/_3)25'} = 934.4 \text{ psf} \tag{8-4-7}$$

Active stress at the dredge line point C:

$$\sigma_c = (115)(25')\left(\frac{1}{3}\right) = 958.3 \text{ psf} \tag{8-4-8}$$

$$\mathbf{FS} = \frac{\mathbf{M_R}}{\mathbf{M_D}} \tag{8-4-9}$$

Let **FS = 1.3**

$$M_R = 1.3M_D \tag{8-4-10}$$

Take moments about the ground anchor:

$$M_D = \begin{bmatrix} (934.4)\left(\frac{6.67}{2}\right)\left(3.33+\frac{6.67}{3}\right)-(934.4)(8.33)(0.835) \\ -(934.4)\left(\frac{10}{2}\right)(8.33)-(958.3)\left(15+\frac{D}{2}\right)D \\ -(0.5)(115)\left(\frac{1}{3}\right)\left(15+\frac{2}{3}D\right)D^2 \end{bmatrix}$$

$$= -\,12.778D^3 \;-\; 766.65D^2 \;-\; 14374.5D \;-\; 28111.6 \qquad (8\text{-}4\text{-}11)$$

As determined above: **K_p** is 4.7.

$$M_R = \frac{1}{2}(115)(4.7)\left(15' + \frac{2}{3}D\right)D^2 \qquad (8\text{-}4\text{-}12)$$

$$M_R \;=\; 180.167D^3 \;+\; 4053.75D^2 \qquad (8\text{-}4\text{-}13)$$

$$M_R \;+ 1.3\,M_D = 0 \qquad (8\text{-}4\text{-}14)$$

Solve for **D**.

$$D^3 + 18.7D^2 - 114.53D - 223.98 = 0 \qquad (8\text{-}4\text{-}15)$$

$$D \approx 6.09 \text{ ft} \qquad (8\text{-}4\text{-}16)$$

Solve for ground anchor force **T** by setting the resisting moment equal to driving moment as shown below:

$$\mathbf{M_R} = \mathbf{M_D} \qquad (8\text{-}4\text{-}17)$$

Find **D′:**

$$D'^3 + 19.64D'^2 - 85.88D' - 167.95 = 0 \qquad (8\text{-}4\text{-}18)$$

$$D' \approx 4.89 \text{ ft} \qquad (8\text{-}4\text{-}19)$$

$$\sum F_X = 0 \qquad (8\text{-}4\text{-}20)$$

$$\begin{bmatrix} -\left(\frac{25+8.33}{2}\right)(934.4) - \frac{1}{2}(115)(4.89^2)\frac{1}{3} - (958.3)(4.89) + \\ \frac{1}{2}(115)(4.89^2)(4.7) \end{bmatrix}(10) + T_H = 0$$

(8-4-21)

$$T_H = 143.87 \text{ kips} \quad \text{and} \quad T = \frac{143.87}{\cos(15°)} = 148.95 \text{ kips} \tag{8-4-22}$$

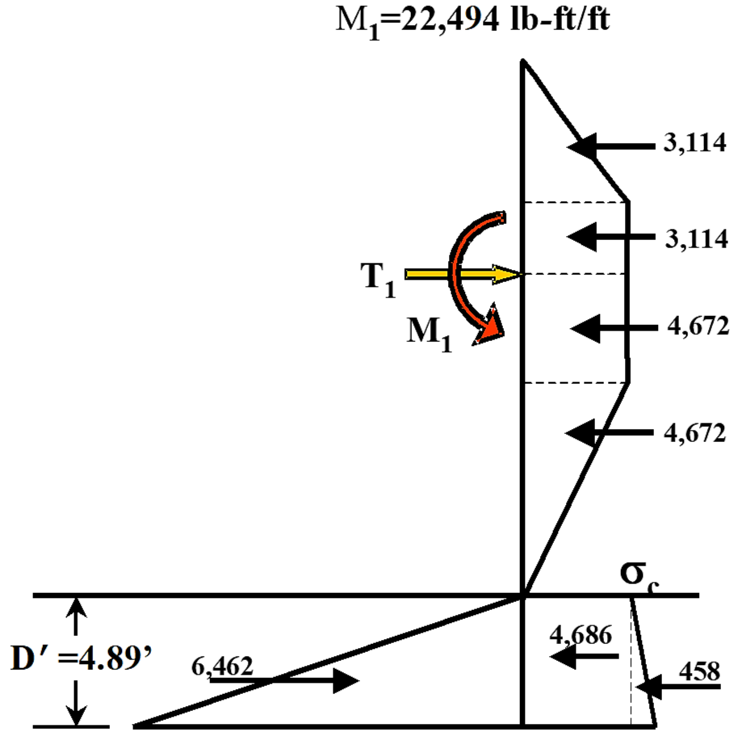


Figure 8-12. Pressure Diagram for Single-Ground Anchor Above Dredge Line Based on $M_R = M_D$

The maximum shear in the sheet pile is located at the ground anchor.

$$T_{1U} = \frac{1}{2}(934.4)(6.667') + (934.4)(3.333') = 3{,}114 + 3{,}114 = 6{,}228 \text{ lb/ft} \tag{8-4-23}$$

$$T_{1L} = \frac{1}{2}(934.4)(10') + (934.4)(5') + (958.3)(4.89') + \frac{1}{2}(115)\left(\frac{1}{3} - 4.7\right)(4.89')^2 =$$
$$4{,}672 + 4{,}672 + 4{,}686 - 6{,}004 = 8{,}026 \text{ lb/ft}$$

(8-4-24)

Maximum shear is **T_{1L}** = 8,026 lb/ft. Recall the area of the sheet pile is 6.46 in^2/ ft, thus:

$$f_v = \frac{8{,}026 \text{ lbs/ft}}{6.46 \text{ in}^2\text{/ft}} = 1{,}242 \text{ psi/ft} \; < \; 16{,}800 \text{ psi} \tag{8-4-25}$$

$$[F_v = 42{,}000 \text{ psi} \times 0.4 = 17{,}000 \text{ psi}] \therefore \tag{8-4-26}$$

PZ22 is <u>satisfactory in shear</u>.

Determine moment $\mathbf{M_1}$ at top of ground anchor due to cantilever loads:

M_1 = F_1 x Moment arm of triangular load + F_2 x Moment arm of rectangular load

$$\mathrm{M_1} = \frac{1}{2}(934.4)(6.667)\left(3.333 + \frac{6.667}{3}\right) + (934.4)(3.333)\left(\frac{3.333}{2}\right)$$
$$= 17303.877 + 5190.063 = 22{,}494 \text{ lb-ft/ft} \qquad (8\text{-}4\text{-}27)$$

Determine moment at zero shear below the ground anchor. Please refer to Figure 8-13 for the shear diagram of single ground anchor. The point of zero shear is either located below the bottom of excavation or it is located between the ground anchor and the bottom of excavation. For this particular example problem, when the summation of forces in the horizontal direction includes the area below the bottom of excavation, a quadratic equation results with two possible roots. As shown below, one root lies at depth **D** but is not the root we are looking for. The other root is negative and therefore, cannot be used:

$$\sum \mathrm{F_H} = 8026 - 4672 - 4672 - 958.3\mathrm{y}' - \frac{1}{2}(115)\left(\frac{1}{3} - 4.7\right)\left(\mathrm{y}'\right)^2$$
$$= -1318 - 958.3\mathrm{y}' + 251.09\mathrm{y}'^2 = 0 \qquad (8\text{-}4\text{-}28)$$

Solving:

$$\mathrm{y}'^2 - 3.816\mathrm{y}' - 5.25 = 0 \qquad (8\text{-}4\text{-}29)$$

yields:

$$\mathrm{y}' = 4.89 \text{ ft and } \mathrm{y}' = -1.07 \text{ ft} \qquad (8\text{-}4\text{-}30)$$

Since the second root is invalid, the point of zero shear must be located above the bottom of the excavation. Further, it can be surmised that the point of zero shear is located within the sloping portion of the load diagram below the ground anchor since:

$$\mathrm{T_{1L}} - (934.4)(5') = 8{,}026 \text{ lbs} - 4{,}672 \text{ lbs} = 3354 \text{ lbs/ft} > 0 \qquad (8\text{-}4\text{-}31)$$

The slope of the load line just above the dredge line is:

$$^{934.4\text{ psf}}\!/_{10'} = 93.44 \text{ psf/ft} \qquad (8\text{-}4\text{-}32)$$

Solving for **y'**:

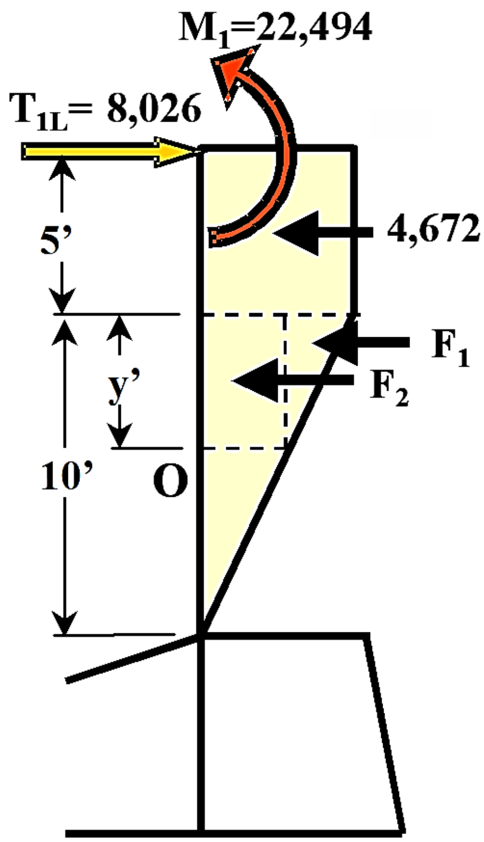


Figure 8-13. Zero Shear

$$(8{,}026 - 4{,}672) = \frac{1}{2}(934.4 + 934.4 - 93.44\text{y}')\text{y}'$$

$$(2)(3{,}354) = 1{,}868.8\text{y}' - 93.44\text{y}'^2$$

$$\therefore 93.44\text{y}'^2 - 1{,}868.8\text{y}' + 6{,}708 = 0$$

$$\text{y}' = 4.69' \tag{8-4-33}$$

The point of zero shear is located $5'+4.69' = 9.69'$ below **T1**. Taking moments about the point of zero shear (**O**) in Figure 8-13:

$$\text{F}_1 = \frac{1}{2}(93.44)(4.69')(4.69') = 1{,}027.6\text{ lbs/ft} \tag{8-4-34}$$

$$\text{F}_2 = (93.44)(10' - 4.69')(4.69') = 2{,}326.7\text{ lbs/ft} \tag{8-4-35}$$

$$\mathrm{M}_{1-\mathrm{tip}} = \left\{ \begin{matrix} (8{,}026)(9.69') - (4{,}672)\left(\frac{5'}{2} + 4.69'\right) - \frac{2}{3}(1{,}027)(4.69') \\ -\frac{1}{2}(2{,}326)(4.69') - 22{,}494 \end{matrix} \right\} \quad (8\text{-}4\text{-}36)$$

$$\mathrm{M}_{1-\mathrm{tip}} = 77{,}772 - 33{,}592 - 3{,}211 - 5{,}455 - 22{,}494 = 13{,}020 \text{ ft-lbs/ft} \quad (8\text{-}4\text{-}37)$$

Therefore, the maximum moment is at **T_1**:

$$\mathrm{M}_1 = 22{,}494 \text{ ft-lbs/ft} \quad (8\text{-}4\text{-}38)$$

Check the bending stress in the sheet pile section:

$$\mathrm{f_b} = \frac{22{,}494 \text{ ft-lb/ft } \times 12 \text{ in/ft}}{18.10 \text{ in}^3/\text{ft}} = 14{,}913 \text{ psi} \quad (8\text{-}4\text{-}39)$$

$$\mathrm{F_b} = 42{,}000 \text{ psi} \times 0.6 \approx 25{,}000 \text{ psi} \therefore \quad (8\text{-}4\text{-}40)$$

Therefore, PZ22 is satisfactory in bending.

The process to check deflection is discussed in Section 7-3, *System Deflection*, and illustrated in Example 9-1 from Chapter 9, *Railroad*, as well as in Appendix B, *Example Problems*; it will not be calculated for this example. The deflection diagram in Figure 8-14 represents the deflected shape of the PZ22 sheet pile based on the Moment Area method; therefore, use these values with caution. The deflection due to the cantilever is 0.20 inches. The maximum deflection is 0.23 inches and is located about 9.6 feet below **T_1**. The respective diagrams for pressure, shear, moment, and deflection for the example with single ground anchors, are shown below in Figure 8-14, and are for information only. These results were obtained using the Caltrans Trenching and Shoring Check Program.

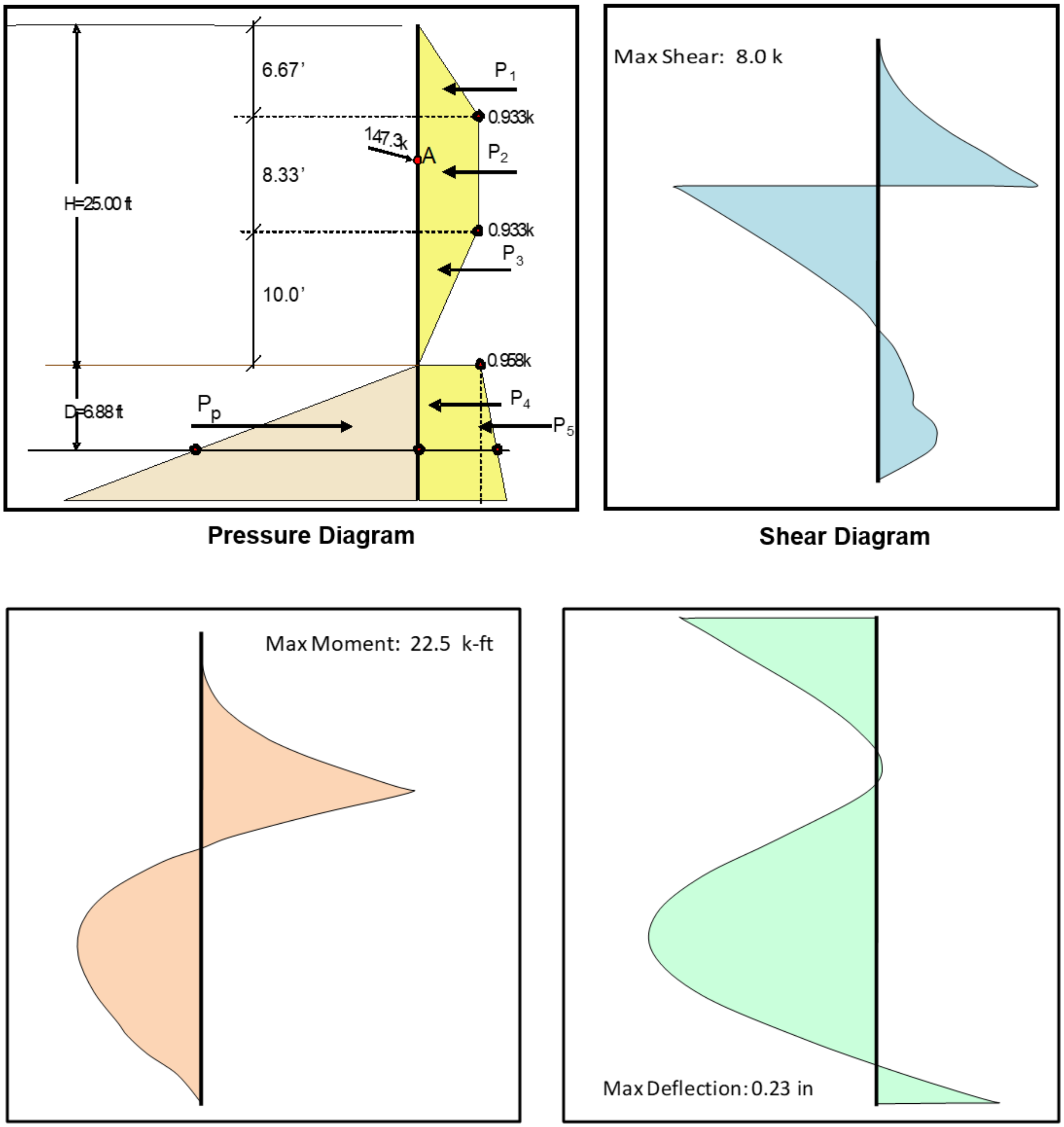


Figure 8-14. Pressure, Shear, Moment, and Deflection Diagrams (CT-T&S Program)

8-4.05 Example 8-2 Multiple Ground Anchor Soldier Pile Wall

Check the adequacy of the multi-ground anchor cantilevered soldier pile wall in one-layered soil. The soldier piles are HP8 x 36 steel beams, at 8 feet spacing, placed in 2-foot-diameter holes filled with 4-sack concrete.

Given:

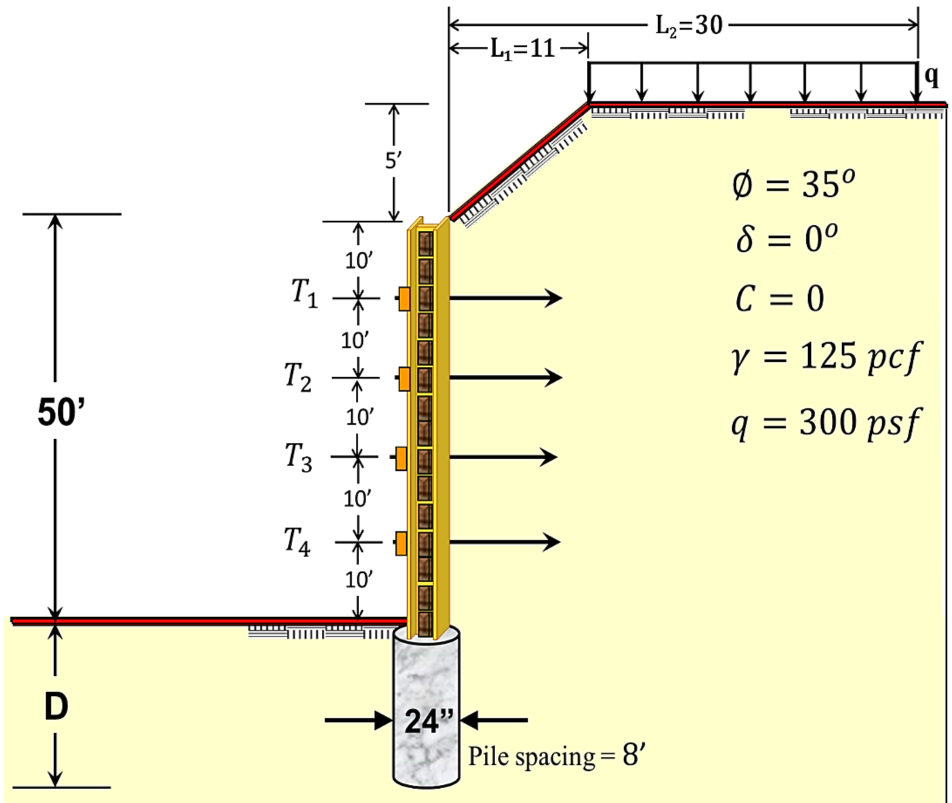


Figure 8-15. Multiple Ground Anchor Soldier Pile Wall, Example 8-2

Determine:

1. Active & passive earth pressures
2. Pile embedment **D**
3. Ground anchor loads
4. Maximum shear, maximum moment

Solution:

In the case of multiple ground anchor walls, the lateral active horizontal earth pressure for the typical multilevel anchors wall is shown in Figure 8-16.

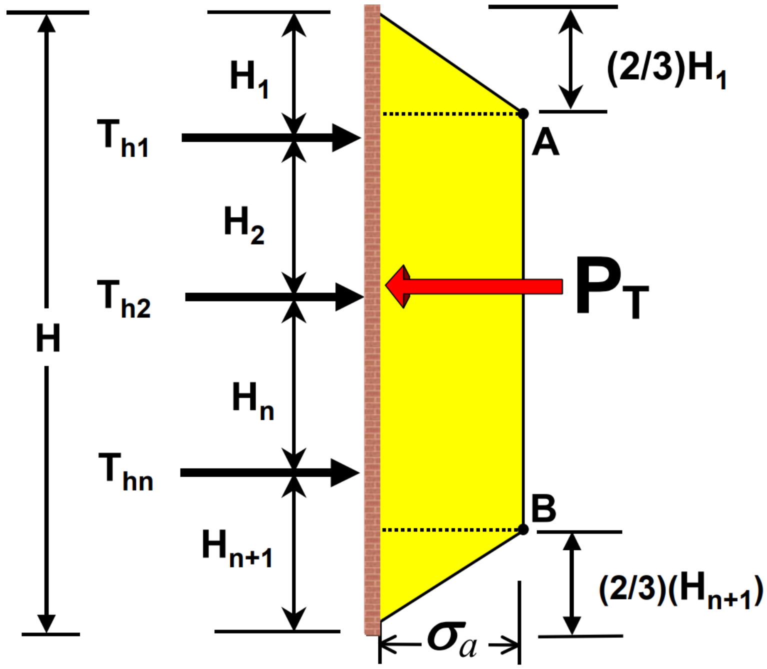


Figure 8-16. Typical Multi Ground Anchor Wall System

$$\boldsymbol{\sigma_a} = \frac{\mathbf{P_T}}{[\mathbf{H} - \frac{\mathbf{1}}{\mathbf{3}}(\mathbf{H_1} + \mathbf{H_5})]} \qquad (8\text{-}4\text{-}41)$$

Where:

- $\boldsymbol{\sigma_a}$ = Maximum ordinate of pressure diagram
- **P** = Total lateral load required to be applied to the wall (per foot of wall)
- **H** = Wall height
- $\mathbf{H_1}$ = Distance from ground surface at top of wall to uppermost level of anchors
- $\mathbf{H_{n+1}}$ = Distance from the grade at bottom of a wall to lowermost level of anchors
- $\mathbf{T_{hn}}$ = Horizontal component of the anchor force at level **n**
- **n** = Number of anchors

The coefficients of active and passive earth pressures are calculated using Trial Wedge method (see Chapter 4, Section 4-5.01, *Active Trial Wedge Method*, and 4-5.02, *Passive Trial Wedge Method*). When using the Trial Wedge method for sloping ground, remember that the earth pressure coefficient will vary with depth because of the variations of the wedge weight. For this example, a wall height of 50 feet is assumed so the results presented herein are valid only for this height. Other heights will vary somewhat, and further trial and error may be necessary to obtain the desired accuracy. For instance, at the depth of 55 feet, $\mathbf{K_a}$ = 0.313 vs. 0.310 at depth of 60 feet resulting in slight increase in embedment depth and ground anchor loads.

$\mathbf{K_a} = 0.310;\ (K_a = \text{Active earth pressure coefficient})$

$\mathbf{K_P} = 3.690;\ (K_P = \text{Passive earth pressure coefficient})$

The lateral earth pressure distribution for the analysis of braced or anchored walls constructed in cohesionless soils may be determined using Figure 8-16. The maximum ordinate (**σ_a**) of the pressure diagram is determined as follows:

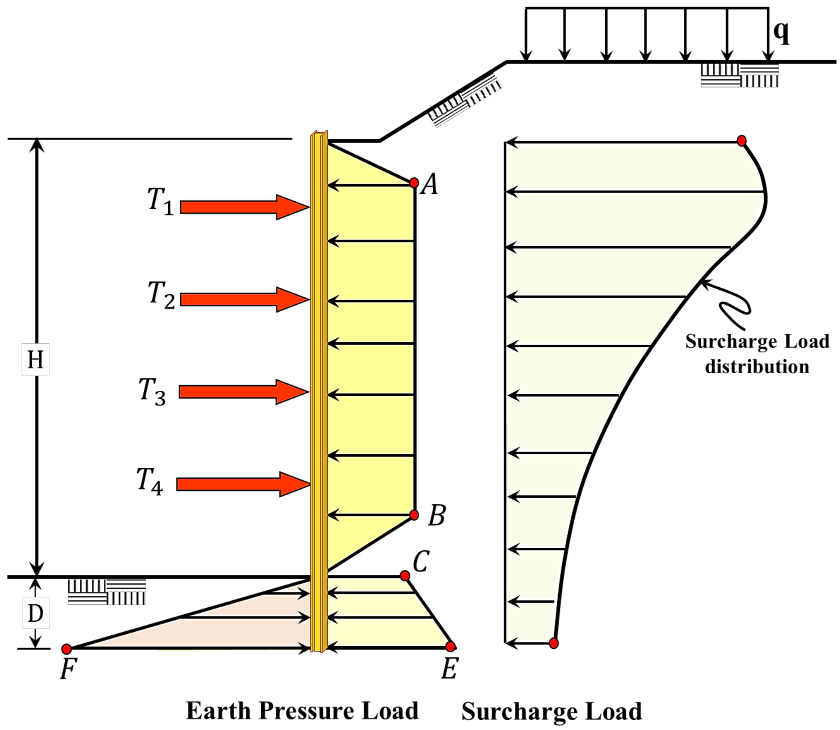


Figure 8-17. Pressure Diagram for Multi Ground Anchor Wall System with Boussinesq Load

Lateral load distribution at excavation line: (point C) as shown in Figure 8-17:

$$\sigma_{a1} = \Upsilon \times (H = 50\ \text{ft}) \times K_a = (125)(50)(0.310) = 1937.5\ \text{psf} \qquad (8\text{-}4\text{-}42)$$

Lateral load distribution at **D** ft below excavation line: (point E):

$$\sigma_{a2} = \sigma_{a1} + \Upsilon \times (D) \times K_a = 1937.5\ \text{psf} + (125)(D)(0.310) \qquad (8\text{-}4\text{-}43)$$
$$= (1937.5 + 38.75D)\ \text{psf}$$

Calculate passive earth pressure at depth **D** below excavation line (point F):

$$\sigma_P = \Upsilon \times (H = D) \times K_P = (125)(D)(3.690) = (461.25D)\ \text{psf} \qquad (8\text{-}4\text{-}44)$$

$$P = \frac{1}{2} \times \Upsilon \times H^2 \times K_{a1} = \frac{1}{2}(125)(50^2)(0.310) = 48{,}438 \text{ lb/ft} \tag{8-4-45}$$

$$P_T = 1.3 \times P = 1.3(48438) = 62{,}969 \text{ lb/ft} \tag{8-4-46}$$

Active stress at the points A and B as shown in Figure 8-16 and Figure 8-17:

$$\sigma = \frac{62969}{\left[50 - \frac{1}{3}(10 + 10)\right]} = 1453 \text{ psf} \tag{8-4-47}$$

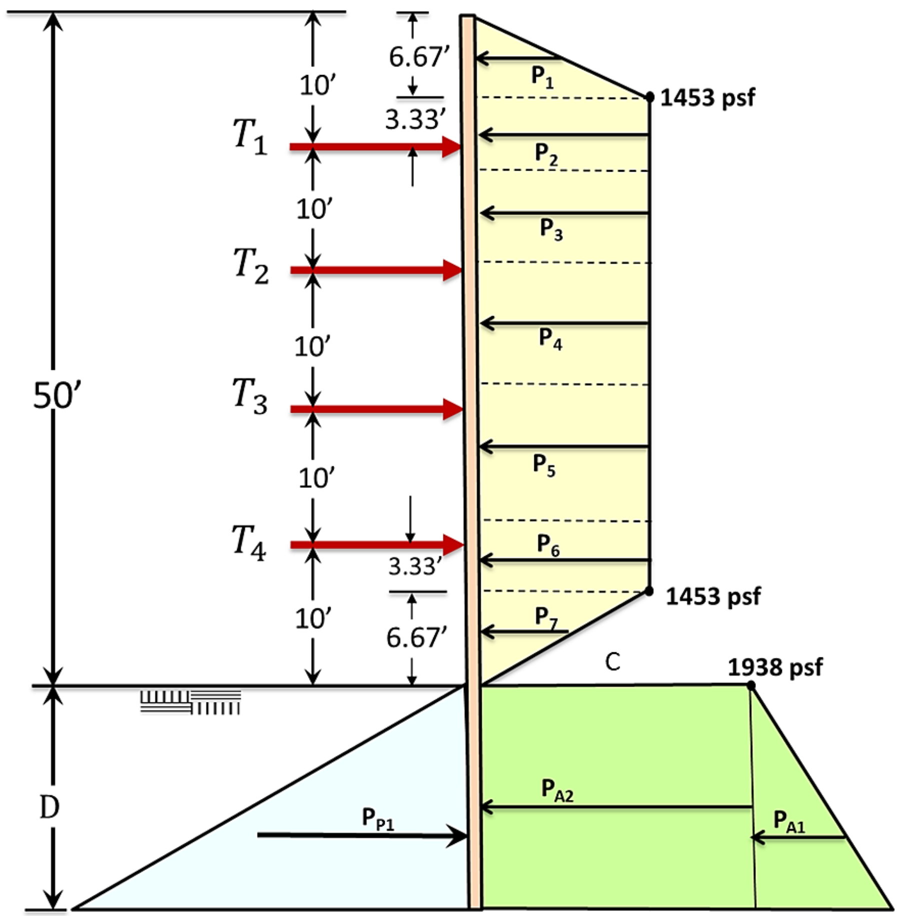


Figure 8-18. Trapezoidal Pressure Diagram Distribution

Surcharge Load:

In addition to the loads due to earth pressure, the surcharge load needs to be considered. To follow are the steps required to calculate the additional loads due to surcharge load. See Chapter 9, *Railroad*, for additional information on the Boussinesq-Type strip load.

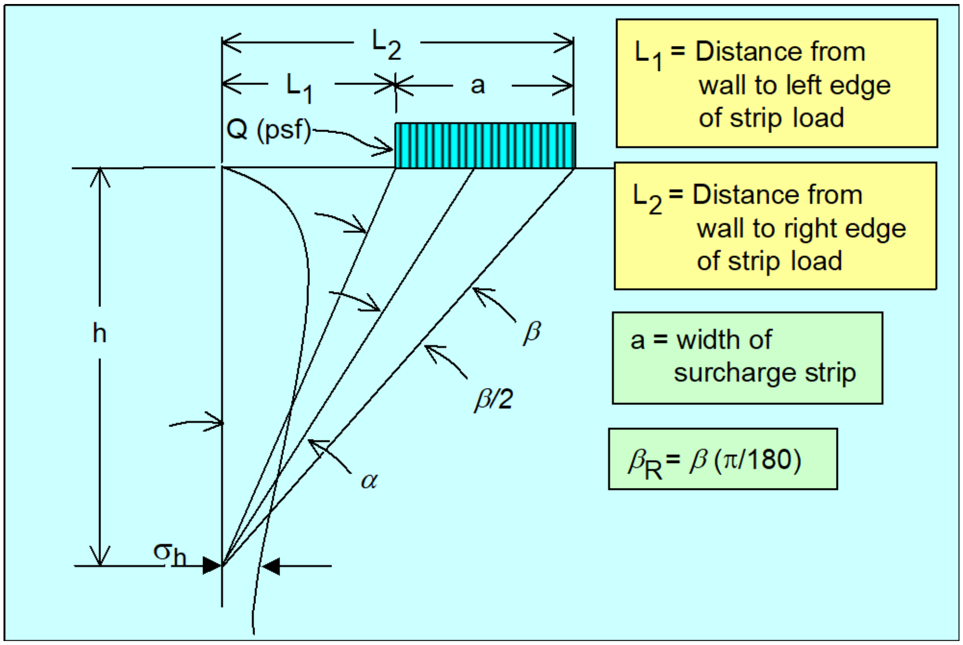


Figure 8-19. Boussinesq-Type Strip Load with the Wayne C. Teng Equation

Load at top of the shoring:

$$\beta = \left[\tan^{-1}\left(\frac{L_2}{h}\right) - \tan^{-1}\left(\frac{L_1}{h}\right)\right] = \tan^{-1}\left(\frac{30}{5}\right) - \tan^{-1}\left(\frac{11}{5}\right) = 14.98° \quad (8\text{-}4\text{-}48)$$

$$\beta_R = (14.98)\frac{\pi}{180} = 0.261 \text{ Rad} \quad (8\text{-}4\text{-}49)$$

$$\alpha_R = \tan^{-1}\left(\frac{L_1}{h}\right) + \frac{\beta}{2} = \tan^{-1}\left(\frac{11}{5}\right) + \frac{14.98}{2} = 73.04° \quad (8\text{-}4\text{-}50)$$

$$\alpha_R = 73.04 \times \frac{\pi}{180} = 1.275 \text{ Rad} \quad (8\text{-}4\text{-}51)$$

$$q = 300 \text{ psf} \quad (8\text{-}4\text{-}52)$$

$$\sigma_h = 2q \times \frac{\beta_R - \sin\beta\cos 2\alpha_R}{\pi} \quad (8\text{-}4\text{-}53)$$

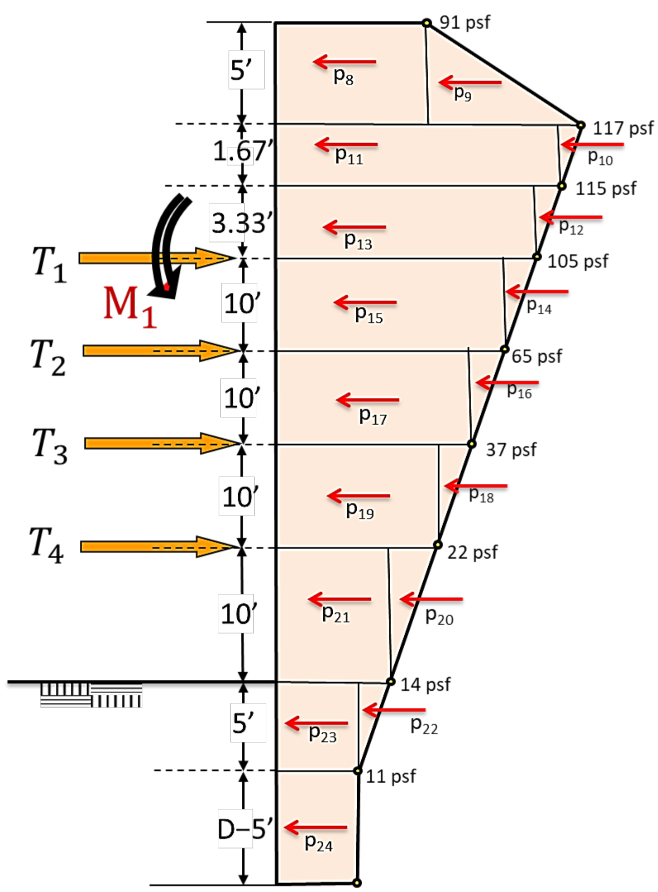
$$\sigma_h = 2 \times 300 \times \left[\frac{0.261 - \sin(0.261)\cos(2 \times 1.275)}{\pi}\right] = 91 \text{ psf} \quad (8\text{-}4\text{-}54)$$

This equation for the increase in horizontal pressure due to the vertical surcharge load is the Wayne C. Teng Equation, which is shown in Section 5, Appendix of the *Guidelines for Temporary Shoring*, published by BNSF and UPRR (12/07/21). Note that the line for the alpha (α) angle is the line that is in the center of the beta (β) angle; it is not the line that goes through the center of the vertical surcharge load.

Table 8-1. Boussinesq Values at Various Depths

Depth (ft.)	Horizontal Load (psf)	Location
0	91	Top of shoring
5	117	5 ft down
10	105	10 ft down
50	14	Dredge line
55	11	5 ft below dredge line

Load diagram due to surcharge is shown in Figure 8-20 below (not to scale):



Note: $\mathrm{M}_1 = 42{,}187$ lb-ft

Figure 8-20. Load Diagram Due to Boussinesq Load

Load Calculations due to Boussinesq Load (P_8 through P_{24}):

$$P_8 = (91)(5')(8') = 3640 \text{ lb} \quad (8\text{-}4\text{-}55)$$

$$P_9 = (\tfrac{1}{2})(5')(117-91)(8') = 520 \text{ lb} \quad (8\text{-}4\text{-}56)$$

$$P_{10} = (\tfrac{1}{2})(1.67')(117-115)(8') = 14 \text{ lb} \quad (8\text{-}4\text{-}57)$$

$$P_{11} = (115)(1.67')(8') = 1537 \text{ lb} \quad (8\text{-}4\text{-}58)$$

$$P_{12} = (\tfrac{1}{2})(3.33')(115-105)(8') = 134 \text{ lb} \quad (8\text{-}4\text{-}59)$$

$$P_{13} = (105)(3.33')(8') = 2798 \text{ lb} \quad (8\text{-}4\text{-}60)$$

$$P_{14} = (\tfrac{1}{2})(10')(105-65)(8') = 1600 \text{ lb} \quad (8\text{-}4\text{-}61)$$

$$P_{15} = (65)(10')(8') = 5200 \text{ lb} \quad (8\text{-}4\text{-}62)$$

$$P_{16} = (\tfrac{1}{2})(10')(65-37)(8') = 1120 \text{ lb} \quad (8\text{-}4\text{-}63)$$

$$P_{17} = (37)(10')(8') = 2960 \text{ lb} \quad (8\text{-}4\text{-}64)$$

$$P_{18} = \left(\frac{1}{2}\right)(10')(37-22)(8') = 600 \text{ lb} \quad (8\text{-}4\text{-}65)$$

$$P_{19} = (22)(10')(8') = 1760 \text{ lb} \quad (8\text{-}4\text{-}66)$$

$$P_{20} = \left(\frac{1}{2}\right)(10')(22-14)(8') = 320 \text{ lb} \quad (8\text{-}4\text{-}67)$$

$$P_{21} = (14)(10')(8') = 1120 \text{ lb} \quad (8\text{-}4\text{-}68)$$

$$P_{22} = \left(\frac{1}{2}\right)(5')(14-11)(2') = 15 \text{ lb} \quad (8\text{-}4\text{-}69)$$

$$P_{23} = (11)(5')(2') = 110 \text{ lb} \quad (8\text{-}4\text{-}70)$$

$$P_{24} = (11)(D-5')(2') = (22D-110) \text{ lb} \quad (8\text{-}4\text{-}71)$$

$$\mathbf{M_1 = \text{Moment Due to load } P_1 \,\&\, P_2}$$

$$\mathbf{T_{1U} = (P_1 + P_2)}$$

$$\mathbf{T_{1L} = \left(\frac{P_3}{2} + \frac{M_1}{S_1}\right)}$$

$$\mathbf{T_{2U} = \left(\frac{P_3}{2} - \frac{M_1}{S_1}\right)}$$

$$\mathbf{T_{2L} = \left(\frac{P_4}{2}\right)}$$

$$\mathbf{T_{3U} = \left(\frac{P_4}{2}\right)}$$

$$\mathbf{T_{3L} = \left(\frac{P_5}{2}\right)}$$

$$\mathbf{T_{4U} = \left(\frac{P_5}{2}\right)}$$

$$\mathbf{T_{4L} = \left(P_6 + P_7 + P_{a1} + P_{a2} - P_{p1}\right)}$$

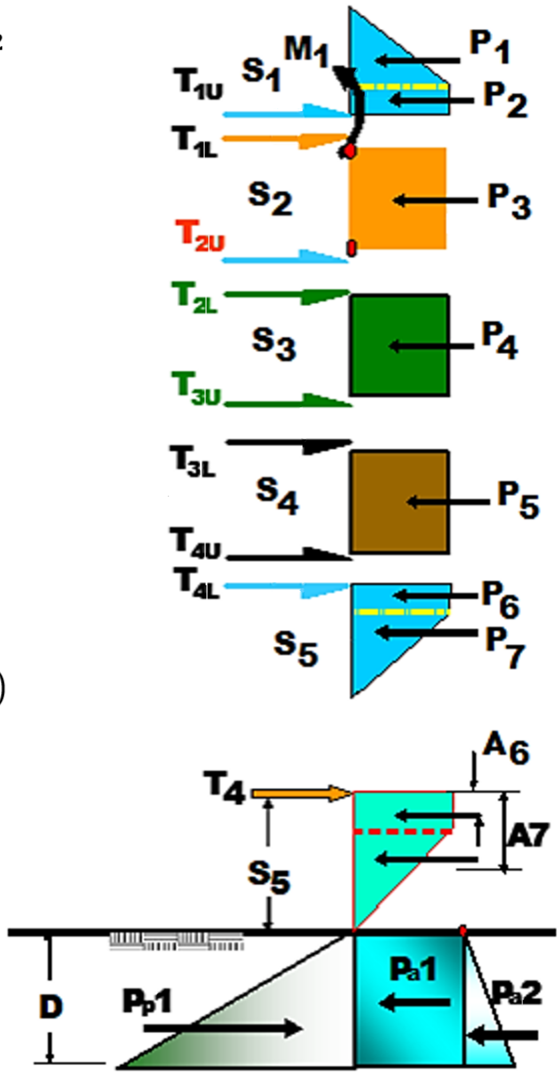


Figure 8-21. Detailed Analysis of Multi Ground Anchor Wall

For soldier piles, an arching factor needs to be calculated and applied to passive forces below the dredge line. The arching factor is not applied to active forces below the dredge line. Assume that the effective width of the piles is 2 feet.

$$\text{Arching Factor} = 0.08\phi = 0.08(35) = 2.80 \quad (8\text{-}4\text{-}72)$$

1. Determine the forces due to trapezoidal pressure distribution using the dimensions illustrated in Figure 8-18:

$$P_1 = \frac{1}{2}(6.67)(1453)(8') = 38{,}766 \text{ lb} \quad (8\text{-}4\text{-}73)$$

$$P_2 = (3.33)(1453)(8') = 38{,}708 \text{ lb} \quad (8\text{-}4\text{-}74)$$

$$P_3 = (10)(1453)(8') = 116{,}240 \text{ lb} \quad (8\text{-}4\text{-}75)$$

$$P_4 = (10)(1453)(8') = 116{,}240 \text{ lb} \quad (8\text{-}4\text{-}76)$$

$$P_5 = (10)(1453)(8') = 116{,}240 \text{ lb} \quad (8\text{-}4\text{-}77)$$

$$P_6 = (3.33)(1453)(8') = 38{,}708 \text{ lb} \quad (8\text{-}4\text{-}78)$$

$$P_7 = \frac{1}{2}(6.67)(1453)(8') = 38{,}766 \text{ lb} \quad (8\text{-}4\text{-}79)$$

2. Determine the moment **M₁** at top ground anchor due to cantilever loads:

$$M_1 = 38{,}766\left[3.33 + \frac{1}{3}(6.67)\right] + 38{,}708\left(\frac{3.33}{2}\right) \quad (8\text{-}4\text{-}80)$$

$$M_1 = 279{,}729 \text{ lb-ft} \quad (8\text{-}4\text{-}81)$$

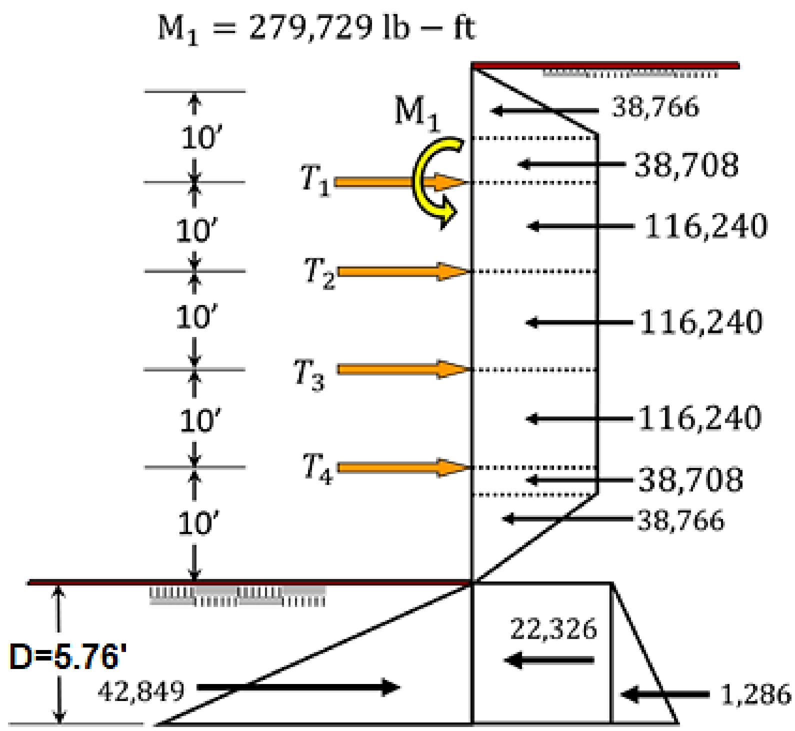


Figure 8-22. Pressure Diagram for Multi-Ground Anchor Above Dredge Line

* Note: **D** is calculated below for **FS = 1.0**:

3. Determine ground anchor loads $\mathbf{T_1}$ through $\mathbf{T_3}$ and component $\mathbf{T_{4U}}$:

Component $\mathbf{T_{4L}}$ will be determined after **D** is calculated. Note, the subscript letters “U” refers to Upper and “L” refers to Lower components of each ground anchor.

$$\mathrm{T_{1U}} = \mathrm{P_1} + \mathrm{P_2} = 38766 + 38708 = 77{,}474 \text{ lb} \tag{8-4-82}$$

$$\mathrm{T_{1L}} = \left(\frac{116240 \text{ lb}}{2}\right) + \left(\frac{279729\text{lb}}{10}\right) = 86{,}093 \text{ lb} \tag{8-4-83}$$

$$\mathrm{T_1} = (77474 \text{ lb} + 86093 \text{ lb}) = 163{,}567 \text{ lb} \tag{8-4-84}$$

$$\mathrm{T_{2U}} = \left(\frac{116240 \text{ lb}}{2}\right) - \left(\frac{279729 \text{ lb}}{10}\right) = 30{,}147 \text{ lb} \tag{8-4-85}$$

$$\mathrm{T_{2L}} = \left(\frac{116240 \text{ lb}}{2}\right) = 58{,}120 \text{ lb} \tag{8-4-86}$$

$$\mathrm{T_2} = 58120 \text{ lb} + 30147 \text{ lb} = 88{,}267 \text{ lb} \tag{8-4-87}$$

$$\mathrm{T_{3U}} = \left(\frac{116240 \text{ lb}}{2}\right) = 58{,}120 \text{ lb} \tag{8-4-88}$$

$$\mathrm{T_{3L}} = \left(\frac{116240 \text{ lb}}{2}\right) = 58{,}120 \text{ lb} \tag{8-4-89}$$

$$\mathrm{T_3} = 58120 \text{ lb} + 58120 \text{ lb} = 116{,}240 \text{ lb} \tag{8-4-90}$$

Total tieback loads including Boussinesq Loads in parenthesis:

$$\mathrm{T_1} = 163{,}567 + (8{,}642 + 7{,}885) = 180{,}094 \tag{8-4-91}$$

$$\mathrm{T_2} = 88{,}267 + (-1{,}085 + 2{,}227) = 89{,}409 \text{ lb} \tag{8-4-92}$$

$$\mathrm{T_3} = 116{,}240 + (1{,}853 + 1{,}280) = 119{,}373 \text{ lb} \tag{8-4-93}$$

For T_2 and T_3, the 1st numerical value is tieback load due to earth pressure (Figure 8-22). The subsequent additive numerical value in parentheses are tieback load due to surcharge (Figure 8-20).The first number is load above the tieback, second number is load below the tieback.

4. Loads due to Boussinesq on the bottom cantilever:

$$P_{20} = \left(\frac{1}{2}\right)(10')(22-14)(8') = 320\ \text{lb} \tag{8-4-94}$$

$$P_{21} = (14)(10')(8') = 1120\ \text{lb} \tag{8-4-95}$$

$$P_{22} = \left(\frac{1}{2}\right)(5')(14-11)(2') = 15\ \text{lb} \tag{8-4-96}$$

$$P_{23} = (11)(5')(2') = 110\ \text{lb} \tag{8-4-97}$$

$$P_{24} = (11)(D-5')(2') = (22D-110)\ \text{lb} \tag{8-4-98}$$

5. Determine **D′** to calculate **T₄** by taking moments about **T₄**:

$$M_D = \left\{\begin{array}{l} \left[38708\left(\frac{3.33}{2}\right)\right] + \left[38766\left(\frac{1}{3}\times 6.67 + 3.33\right)\right] + \left[1938D\left(\frac{D}{2}+10\right)(2')\right] \\ \quad + \left[\left(\frac{38.75D^2}{2}\right)\left(\frac{2D}{3}+10\right)(2')\right] + \left[1120\left(\frac{10}{2}\right)\right] + \left[320\left(\frac{10}{3}\right)\right] \\ \quad + \left[110\left(\frac{5}{2}+10\right)\right] + \left[15\left(\frac{5}{3}+10\right)\right] + \left[(22D-110)\left(\frac{D}{2}-\frac{5}{2}+15\right)\right] \end{array}\right\} \tag{8-4-99}$$

$$M_D = 25.83D^3 + 2336.5D^2 + 38980D + 286570 \tag{8-4-100}$$

$$M_R = \left(\frac{461.25D^2}{2}\right)\left(\frac{2}{3}D + 10\right)(2.80)(2') \tag{8-4-101}$$

$$M_R = 861D^3 + 12915D^2 \tag{8-4-102}$$

Set $\mathbf{M_D}$ = $\mathbf{M_R}$ and solve for embedment depth “**D**”:

$$\mathbf{M_D = M_R} \tag{8-4-103}$$

$$D^3 + 12.67D^2 - 46.67D - 343.07 = 0 \tag{8-4-104}$$

$$\mathbf{D = 5.76\ ft} \tag{8-4-105}$$

For external stability, use the safety factor of 1.5:

$$\frac{M_R}{M_D} = 1.5 \tag{8-4-106}$$

$$\left(\frac{861D^3}{1.5} + \frac{12915D^2}{1.5}\right) - 25.83D^3 - 2336.5D^2 - 38980D - 286570 \tag{8-4-107}$$

$$\mathbf{D = 7.46\ ft} \tag{8-4-108}$$

6. Determine the lower component T_{4L} of ground anchor T_4, **which is due to earth pressure,** and calculate its load:

$$T_{4U} = \left(\frac{116240\ \text{lb}}{2}\right) = 58{,}120\ \text{lb} \quad (8\text{-}4\text{-}109)$$

$$P_{A1} = \frac{1}{2}(38.75)(5.76^2)(2') = 1285.6\ \text{lb} \quad (8\text{-}4\text{-}110)$$

$$P_{A2} = (1938)(5.76)(2') = 22{,}325.8\ \text{lb} \quad (8\text{-}4\text{-}111)$$

$$P_{P1} = \frac{1}{2}(461.25)(5.76^2)(2') \times 2.80 = 42{,}848.9\ \text{lb} \quad (8\text{-}4\text{-}112)$$

$$T_{4L} = 38708 + 38766 + 1285.6 + 22325.8 - 42848.9 \rightarrow\ T_{4L} = 58{,}236.5\ \text{lb} \quad (8\text{-}4\text{-}113)$$

$$T_4 = 58120\ \text{lb} + 58236.5\ \text{lb} = 116{,}356.5\ \text{lb} \quad (8\text{-}4\text{-}114)$$

Include Boussinesq load (**T_{4U} +T_{4L}**) in parenthesis for final **T_4**:

$$T_{4L}\ \text{due to surcharge} = 22D + 1455 = 22 \times 5.76 + 1455 = 1582\ \text{lb} \quad (8\text{-}4\text{-}115)$$

$$T_4 = 116{,}356.5\ \text{lb} + (1{,}080 + 1{,}582)\ \text{lb} \approx 119{,}019\ \text{lb} \quad (8\text{-}4\text{-}116)$$

Shear Diagram:

@10′:

$$V_1 = 38766 + 38708 + 3640 + 520 + 14 + 1537 + 134 + 2798 = 86{,}117\text{lb} \quad (8\text{-}4\text{-}117)$$

$V_2 = T_1 - 86117 = 180094 - 86117 = 93{,}977\ \text{lb}$ = **V_{MAX}** (<u>Max Shear</u>) (8-4-118)

@20′:

$$V_1 = \left(\frac{116{,}240}{2} - 27{,}973\right)\ \text{lb} - \left(\frac{5{,}200}{2} + \frac{1{,}600}{3} - 4{,}218\right) = 31{,}232\ \text{lb} \quad (8\text{-}4\text{-}119)$$

$$V_2 = T_2 - 31232\ \text{lb} = 89409 - 31232 = 58{,}177\ \text{lb} \quad (8\text{-}4\text{-}120)$$

@30′:

$$V_1 = \frac{116{,}240}{2}\ \text{lb} + \left(\frac{1{,}120}{3} + \frac{2{,}960}{2}\right)\text{lb} = 59{,}973\ \text{lb} \tag{8-4-121}$$

$$V_2 = T_3 - 59973\ \text{lb} = 119373 - 59973 = 59{,}400\ \text{lb} \tag{8-4-122}$$

@40′:

$$V_1 = \frac{116{,}240}{2}\text{lb} + \left(\frac{600}{3} + \frac{1{,}760}{2}\right)\text{lb} = 59{,}200\ \text{lb} \tag{8-4-123}$$

$$V_2 = T_4 - 59200 = 119019 - 59200 = 59{,}819\ \text{lb} \tag{8-4-124}$$

Note that the values for T_2, T_3, and T_4 are the total loads, including Boussinesq loads.
@55.76′:

$$V = 0 \tag{8-4-125}$$

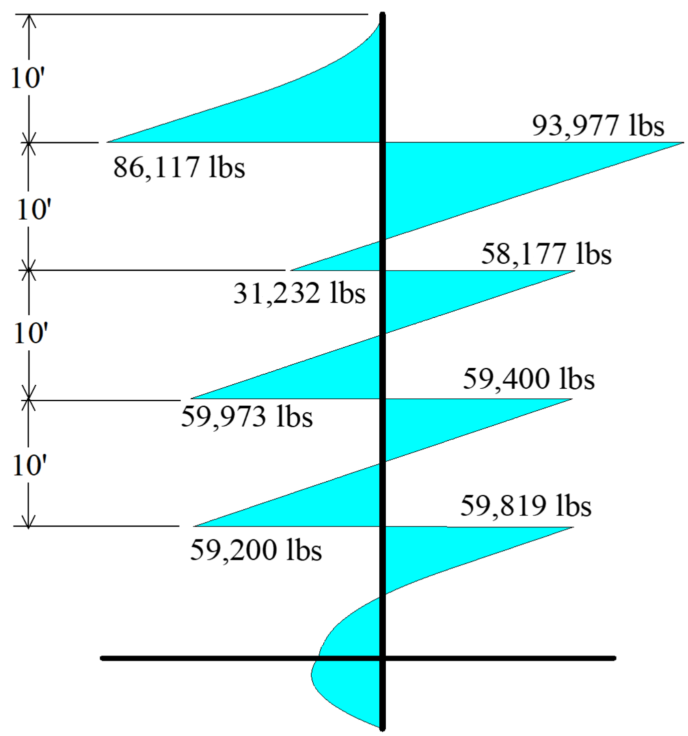


Figure 8-23. Shear Diagram

Moment Diagram:

By inspection, the maximum moment is at the location of the first tie, 10 feet below the top of the wall. Here is how the maximum moment is calculated:

@ First 10':

$$M_{max} = \left\{ \begin{array}{l} 38{,}766\left[\left(\frac{6.67}{3}\right) + 3.33\right] + 38{,}708\left(\frac{3.33}{2}\right) + 3{,}640\left(\frac{5}{2} + 1.67 + 3.33\right) \\ \qquad +520\left(\frac{5}{3} + 1.67 + 3.33\right) + 14\left[\left(\frac{2}{3} \times 1.67\right) + 3.33\right] \\ \qquad +1{,}537\left[\left(\frac{1.67}{2}\right) + 3.33\right] + 134\left(\frac{2}{3} \times 3.33\right) + 2{,}798\left(\frac{3.33}{2}\right) \end{array} \right\}$$

(8-4-126)

$$M_{max} = 321{,}916 \text{ lb-ft}$$ (8-4-127)

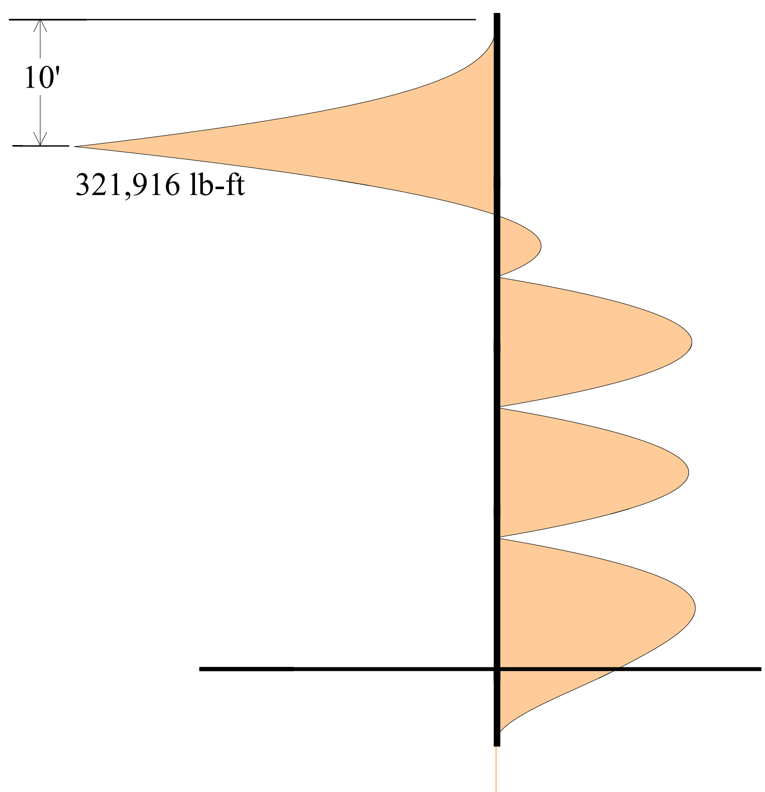


Figure 8-24. Moment Diagram

For the sake of comparison, the Pressure, Shear, Moment, and Deflection Diagrams generated by the Caltrans Trenching & Shoring Program are illustrated below, in Figure 8-25. Compared to the hand calculations, the maximum shear force is essentially identical, and the maximum moment is very similar.

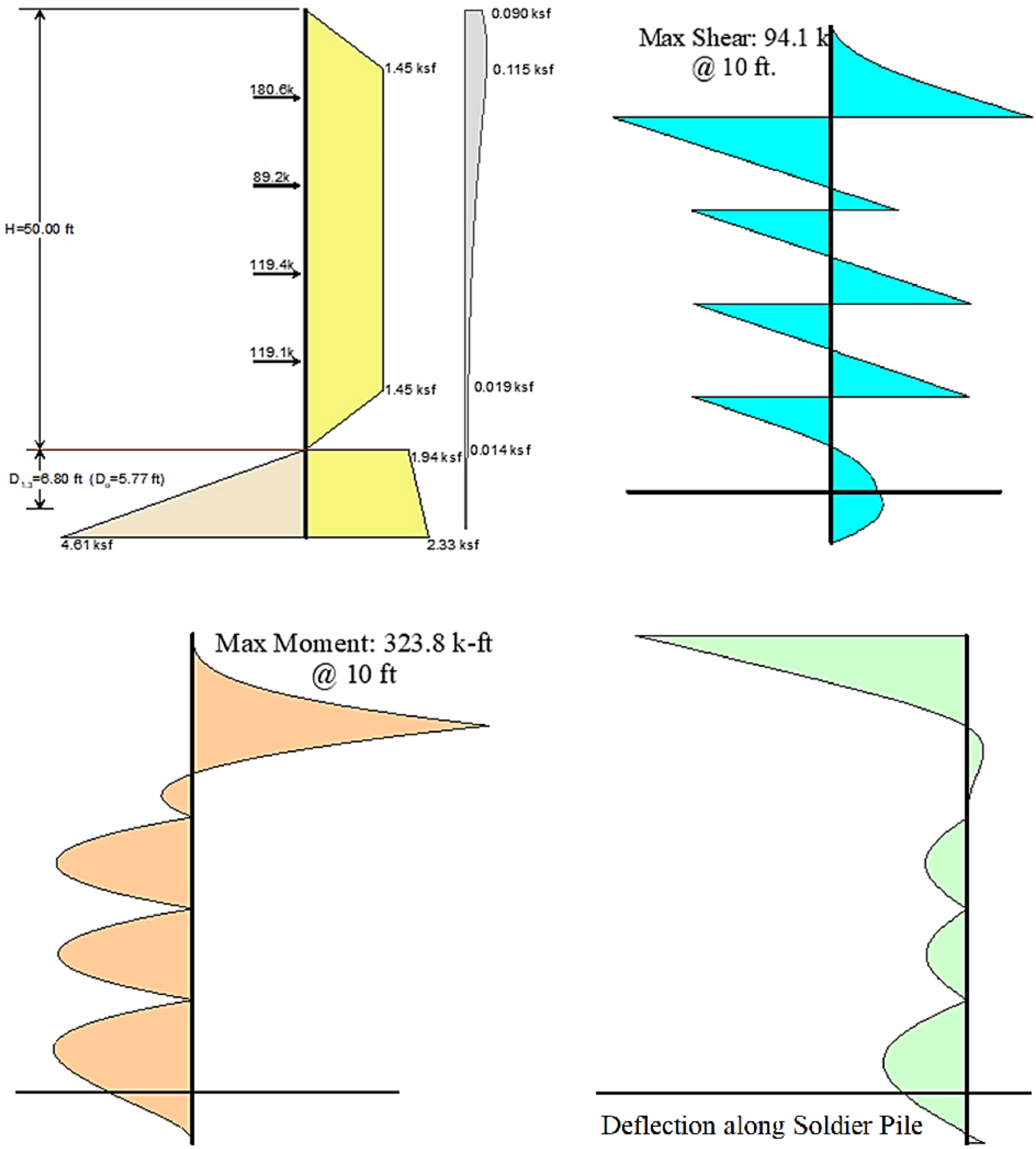
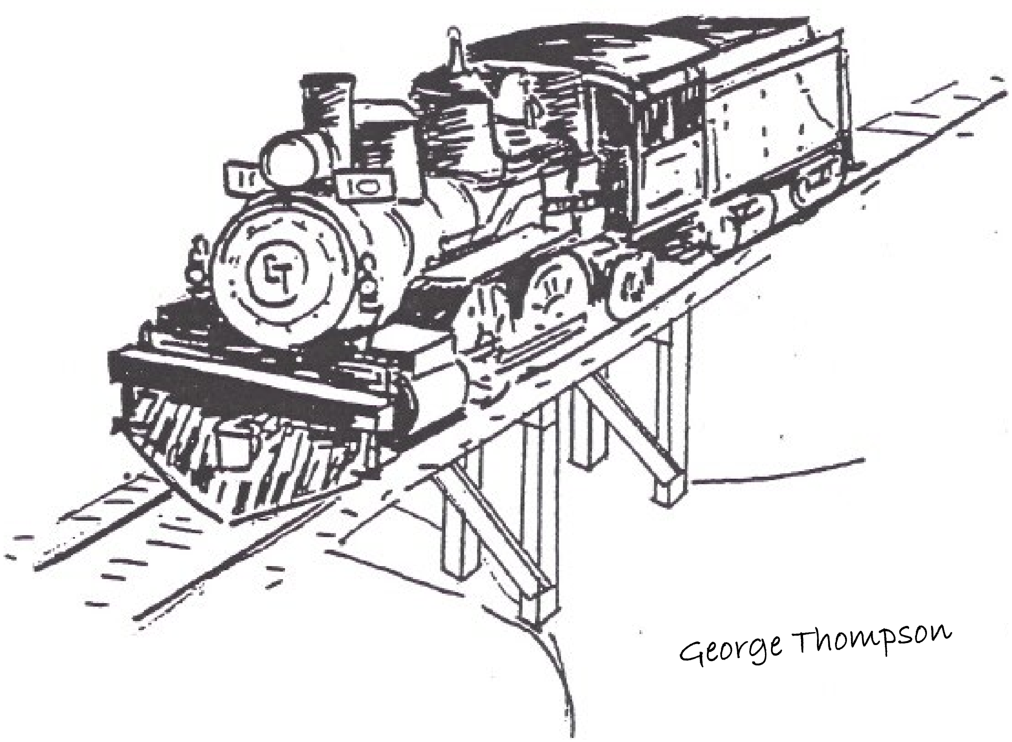


Figure 8-25. Pressure, Shear, Moment, and Deflection Diagrams (CT-T&S Program)

CHAPTER 9

RAILROADS



Chapter 9: Railroads

Table of Contents

Chapter 9: Railroads 1
Table of Contents 1
9-1 Introduction 2
9-2 Summary of Typical Railroad Requirements from the Guidelines 4
9-3 Track Monitoring Requirements for Railroads 6
9-3.01 Monitoring 6
9-3.02 Contingency Plan 7
9-4 Example 9-1 (Railroad Example) 8

9-1 Introduction

Shoring adjacent to railroads presents additional challenges in both the review and construction phases. For the purposes of this manual, the term “Railroad” will refer to the Burlington Northern and Santa Fe Railway (BNSF) and the Union Pacific Railroad (UPRR). In the course of the work, Structure Construction (SC) engineers may encounter other railways such as light rail and commuter trains like Bay Area Rapid Transit (BART) and Southern California Regional Rail Authority (SCRRA). For these other railways, it is acceptable to use the same guidelines presented here unless there are specific instructions from the concerned railway.

Review the UPRR general shoring requirements and the *Guidelines for Temporary Shoring* published by BNSF and UPRR, hereafter simply referred to as GUIDELINES (note that Bridge Design has various Railroad references in Appendix 5.1, *Railroad Overview*, of the *Bridge Design Processes and Procedures Manual*[1], including this resource- and that the version referenced is from December 2021). The GUIDELINES were designed as a supplement to the American Railway Engineering and Maintenance-of-Way Association (AREMA) *Manual of Recommended Practice for Railway Engineering and Maintenance of Way*. When reviewing shoring that encroaches on railroad right-of-way, always verify that the most current editions of both documents are being used. When the railroad requirements conflict with the Department’s or Cal/OSHA specifications, always use the more conservative guidance.

The *Contract Specifications*, Section, 7-1.02K(6)(b), *Legal Relations and Responsibility to the Public – Laws – Labor Code – Occupation Safety and Health Standards – Excavation Safety*, sets the allowable review time for shop drawings for protective system involving the Railroad to 65 days. This includes the railroad’s review time, which is typically 5-6 weeks. Thus, it is important for the Structure Representative (SR) to perform the review and forward to the SC Falsework Engineer promptly, who then sends the submittal to the Railroad. Contracts with Railroad involvement will include an additional section in the Special Provisions directing the Contractor to documents that will include general requirements for the design and construction of temporary shoring and provide reference to additional information and requirements. The *Information Handout* in the bid package contains requirements of the railroad company involved. Below are other sections of the *Contract Specifications,* related to Railroads:

1. Section 2-1.06B, *Bidding – Bid Documents* – Supplemental Project Information
2. Section 5-1.20C, *Control of Work – Coordination with Other Entities – Railroad Relations*
3. Section 5-1.36, *Control of Work – Property and Facility Preservation*
4. Section 5-1.*36B, Control of Work – Property and Facility Preservation – Railroad Property.*

[1] Caltrans internal use only

The field Engineer will be responsible for reviewing the submittal package for compliance and accuracy in the same manner as any other shoring system. Special attention should be paid to the plan and calculation requirements in the GUIDELINES. Submissions of the plans and calculations to the Railroad are to be routed through the Structure Construction Headquarters in Sacramento (SC HQ) in accordance with the procedure set forth in Section 1-6, *Railroad Relations and Requirements,* of this manual and BCM C-11, *Shop Drawing Review of Temporary Structures.* The SC Falsework Engineer[1] will be the field Engineer's single point of contact with the Railroad through the submittal phase. The Contractor may not begin work on any part of the shoring system until Caltrans receives written approval from the Railroad.

Live loads for Railroads are based on the Cooper E80 loading. Cooper E80 is designed to approximate two locomotives with 80 kips per axle pulling an infinite train of 8 kips per foot as shown in Figure 9-1.

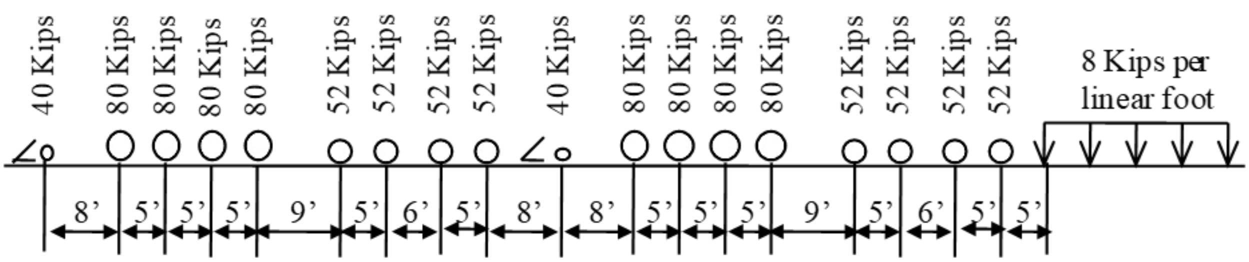


Figure 9-1. Cooper E80 Loading from the Appendix of the GUIDELINES

The lateral pressure from this loading will be determined using the Boussinesq Strip Loading procedure; see Chapter 5, Section 5-1.03, *Boussinesq Loads*. Since the live loading is considered to be dynamic and produce vibrations, the use of wall friction in the earth pressure calculations should not be considered above the bottom of excavation. When using the railroad (RR) live load (LL) curves, the plot of the curve always starts at the elevation of the topsoil level being retained by the shoring system as shown in Figure 9-2.

[1] Caltrans internal use only

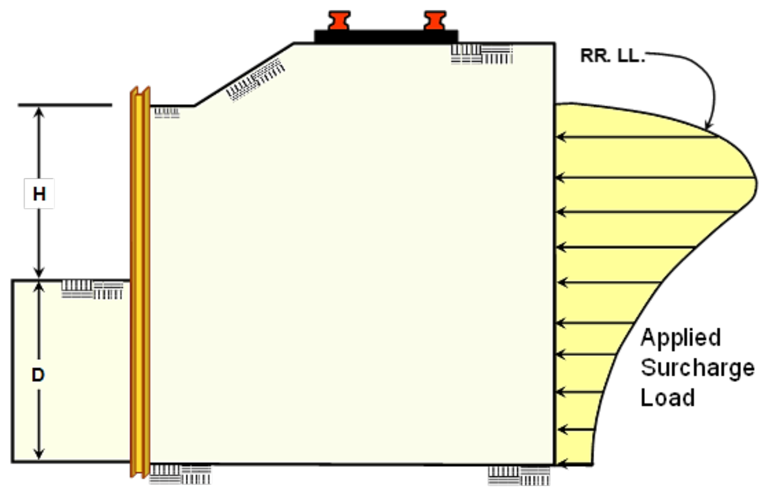


Figure 9-2. Pressure diagram for Boussinesq Strip Load

9-2 Summary of Typical Railroad Requirements from the Guidelines

To follow are some of the typical Railroad requirements from the GUIDELINES that SC Staff must review and verify compliance with for work on or adjacent to Railroad property:

1. No excavations to be closer than 15 feet from centerline of track.
2. Use compacted pea gravel to backfill soldier piles provided:
    a. Groundwater is below bottom of hole.
    b. The difference between the hole diameter and the diagonal pile dimension is 12 inches or more.
    c. The pea gravel is placed in 8-inch lifts and vibrated.
3. Concrete for encasing soldier piles must be 3000 psi and approved for use by the Railroad.
4. Shoring within Zone A shall be placed prior to excavation.
5. Shoring in Zone A is designed for railroad live load (Cooper E80).
6. Shoring in Zones A & B must be stamped by a licensed professional engineer.

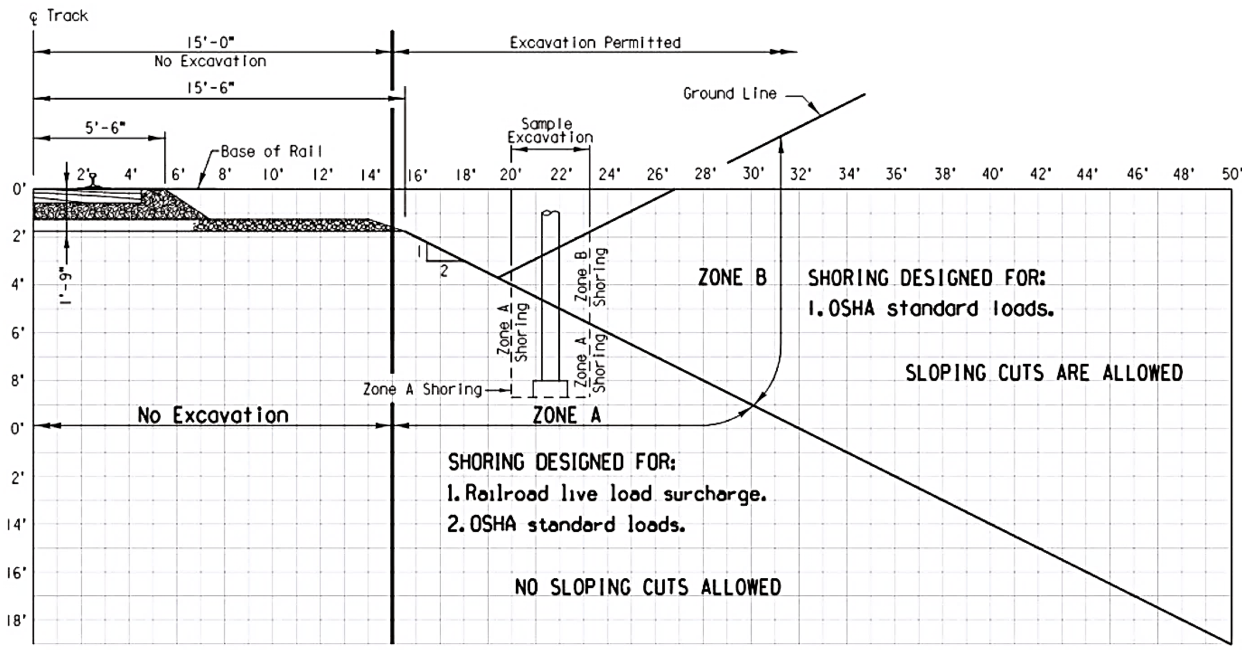


Figure 9-3. Shoring Zones from the GUIDELINES, Section 3, Figure 1

7. Borings for soil identification should be within 50 feet of proposed shoring and additional borings are required when shoring exceeds 250 feet in length.
8. Cantilevered sheet pile walls maximum height is 10 feet in Zone A and 12 feet in Zone B.
9. Cantilevered soldier pile walls maximum height is 8 feet in Zone A and 12 feet in Zone B.
10. Shoring in place for longer than 18 months shall be designed using AREMA requirements for permanent structures.
11. The minimum factor of safety is 1.5 by reducing the passive earth pressure resistance by 0.67.
12. The additional embedment for fixing the tip of the soldier pile is determined by using the Simplified Method and the appropriate factor of safety.
13. Only Rankine and Log Spiral Methods may be used for passive pressures.
14. Assume cohesion value (**c**) = 0 unless local experience by a licensed Geotechnical Engineer determines a higher value.
15. Factor of safety for anchor blocks = 2. See Chapter 10, *Special Conditions*, for determining anchor block capacities.

Remember that other rail entities may have different requirements, thus the Engineer must review the agreements for the individual project and location.

9-3 Track Monitoring Requirements for Railroads

When working in and around BNSF/UPRR railroads, track monitoring and contingency plans are required to perform any construction work. The information below is a summary of the UPRR Guidelines for Track & Ground Monitoring; note that the version referenced is dated April 2021. Other entities may have different requirements.

9-3.01 Monitoring

1. Deflection Limits:
   a. The total deflection of the shoring system is to include accumulated elastic deflections of individual member and passive deflection of resisting soil mass. The figure below is from the GUIDELINES, Table 2 – *Deflection Criteria*.

Horizontal distance from shoring to track C/L measured at a right angle from track	Maximum horizontal movement of shoring system	Maximum acceptable horizontal or vertical movement of rail
15' < S < 18'	3/8"	1/4"
18' < S < 25'	1/2"	1/4"
S > 25'	1% of shoring height above excavation line	-

Figure 9-4. Deflection Criteria from GUIDELINES, Section 3.8, Table 2

   b. Monitoring Targets:
      i. Track monitoring shall not require track access other than to place the track monitoring targets.
      ii. Monitoring targets should be placed such that monitoring is possible when a train is present. However, monitoring during the passing of a train is not required.
      iii. Adhesive backed reflective targets may be attached to the side of the rail temporarily. Targets should be removed once monitoring phase is complete.
   c. Monitoring Plan:
      i. If the top of rail does deflect more than 1/4 inch, all operations shall stop until the matter is resolved.
      ii. Provide established contingency plan (see below) in the event of ground loss and/or the rail deviates 1/4 inch vertical or horizontal.
      iii. Establish a benchmark in the vicinity of the construction. Establish locations for shooting elevations on the top of rail at each area of construction.

Example locations for shooting rail elevations would be at:

1. The centerline of an under-track crossing (required)
2. Both outside edges of the crossing (i.e., for a box culvert)
3. Multiple locations from the crossing centerline or excavation edge (i.e., 10, 20, 30, 40 and 50 feet, as required by the Railroad on a case-by-case basis).

iv. Monitoring shall be continuous and recorded in a field logbook dedicated for this purpose. Copies of these field log entries shall be made available to all concerned parties upon request at any time during construction.

v. Monitoring shall commence once any construction activity is within Zone A. See Figure 9-3 (above).

vi. Monitoring shall continue after installation is complete, for 7 days, or as required by the Railroad.

9-3.02 Contingency Plan

1. The Contractor shall supply contingency plan(s), which anticipate reaching the Threshold and Shutdown values, for all construction activities which may result in horizontal and/or vertical track deflection.
   a. Track monitoring values:
      i. Threshold value = 1/8 inch permanent vertical or horizontal deflection.
      ii. Shutdown value = 1/4 inch permanent vertical or horizontal deflection.
2. The contingency plans shall provide means and methods, with options if necessary.
3. The Contractor should anticipate the need to implement each contingency plan with required materials, equipment, and personnel.
   a. Once the Threshold value is met, the Contractor shall determine the appropriate contingency plan(s) and immediately discuss this plan with, and receive approval confirmation from, the Railroad.
   b. Once the Shutdown value is exceeded, all project work shall stop, and the chosen contingency plan must commence.
4. The Railroad may choose to allow and/or require the immediate implementation of specific approved contingency plans, submitted by the Contractor, if the deflection criteria in Figure 9-4 are met.

9-4 Example 9-1 (Railroad Example)

Cantilevered Soldier Pile Wall by Rigorous Method

Use the rigorous method to perform a shoring check for a W12 x 336 cantilevered soldier-pile-lagging wall with piles at 8 feet on center encased in 2-foot diameter holes, which are filled with 4-sack concrete; the centerline of the train track is 14 feet from the shoring. The soil properties are shown in Figure 9-5.

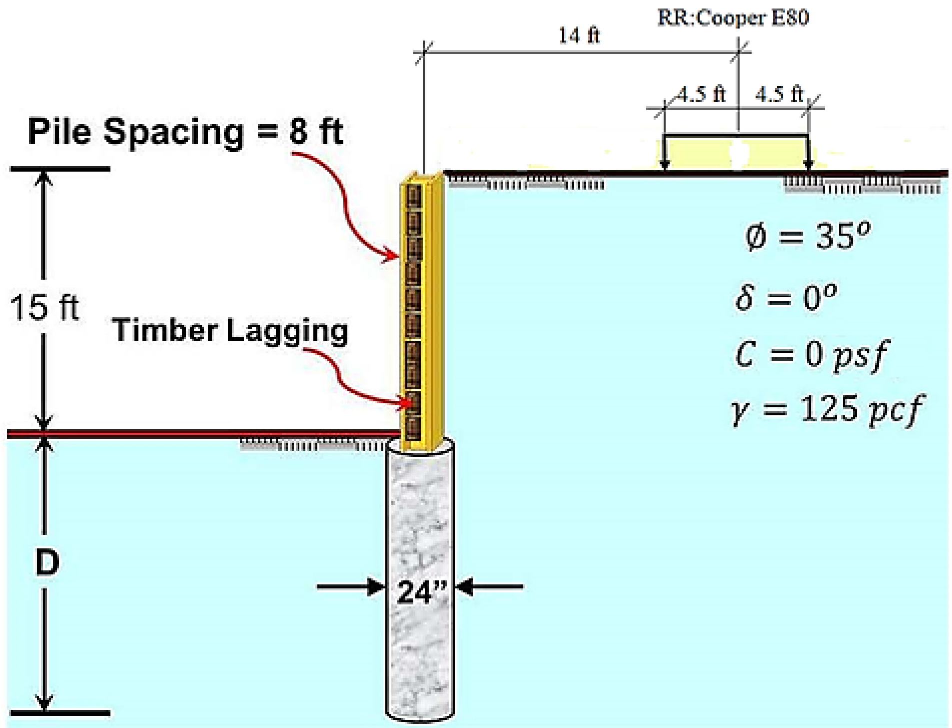


Figure 9-5. Cantilevered Soldier-Pile-Lagging Wall Example Properties

1. Calculate Train Surcharge
2. Calculate Active & Passive Earth Pressures
3. Determine Pile Embedment, **D**
4. Calculate Maximum Shear & Moment
5. Calculate Service Deformation
6. Calculate Timber Lagging Deflection

Determine Train Surcharge:

Surcharge based on E80 Cooper Load:

$$\mathbf{q_s} = \frac{\textbf{Axle Load}}{(\textbf{Axle Spacing})(\textbf{Track} + \mathbf{H_1})};\ \mathrm{H_1} = 0\ \therefore\ \mathrm{q_s} = \frac{80{,}000}{(5)(9+0)} = 1{,}778\ \mathrm{psf}$$

(9-4-1)

Axle Load: Maximum load per Railroad Axle in lbs.

Axle Spacing: Minimum distance of spacing between Railroad Axles in feet.

Track: Length of Railroad Tie in feet.

H1: Height of backfill slope between bottom of tie and top of retaining system in feet. Per Union Pacific RR manual, the height of the backfill slope should be added to the track length when calculating the appropriate surcharge for the Boussinesq Load.

This surcharge is then transformed into a Boussinesq Load (note that the Boussinesq Strip Load equation referenced in the GUIDELINES is commonly referred to as the Wayne C. Teng Equation). To follow is a sample calculation to determine the Boussinesq Load at a depth of 5 feet:

$$\boldsymbol{\sigma_h} = \mathbf{2q_s}\frac{\boldsymbol{\beta_R} - \mathbf{sin\beta cos2\alpha}}{\boldsymbol{\pi}} \quad (9\text{-}4\text{-}2)$$

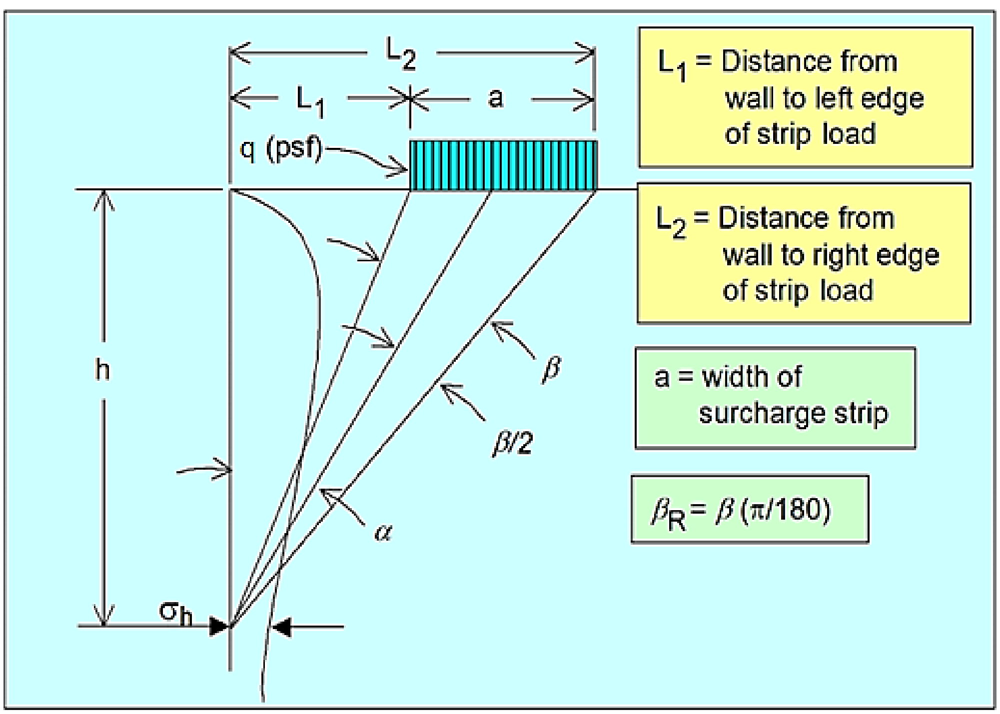


Figure 9-6. Boussinesq Type Strip Load

$$q_S = 1{,}778 \text{ psf}; \ L_1 = 9.5 \text{ ft}; \ L_2 = 18.5 \text{ ft}; \ H = 5\text{ft} \tag{9-4-3}$$

$$\beta = \sin^{-1}\left(\frac{L_2}{\sqrt{L_2^2 + h^2}}\right) - \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) = \sin^{-1}\left(\frac{18.5}{19.164}\right) - \sin^{-1}\left(\frac{9.5}{10.735}\right) = 12.627° \tag{9-4-4}$$

$$\alpha = \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) + \frac{1}{2}\beta = \sin^{-1}\left(\frac{9.5}{10.735}\right) + \frac{1}{2}(12.627°) = 68.560° \tag{9-4-5}$$

$$\beta_R = \beta\left(\frac{\pi}{180}\right) = 12.627°\left(\frac{\pi}{180}\right) = 0.2204 \text{ Rad.} \tag{9-4-6}$$

$$\sigma_h = 2(1{,}778)\frac{0.2204 - \sin(12.627°)\cos(2 \times 68.560°)}{\pi} = 430.79 \approx \mathbf{431\ psf}$$

(horizontal pressure)

(9-4-7)

The above procedure is used to determine the horizontal loads at specific intervals. Alternatively, Section 5.2, *Chart – Live Load Pressure Due to E80 Loading* (found in the Appendix of the GUIDELINES), can be used to obtain the horizontal loads due to the railroad surcharge at various depths. See Figure 9-7 and Table 9-1 below, and note that the GUIDELINES refers to this as the Boussinesq surcharge pressure.

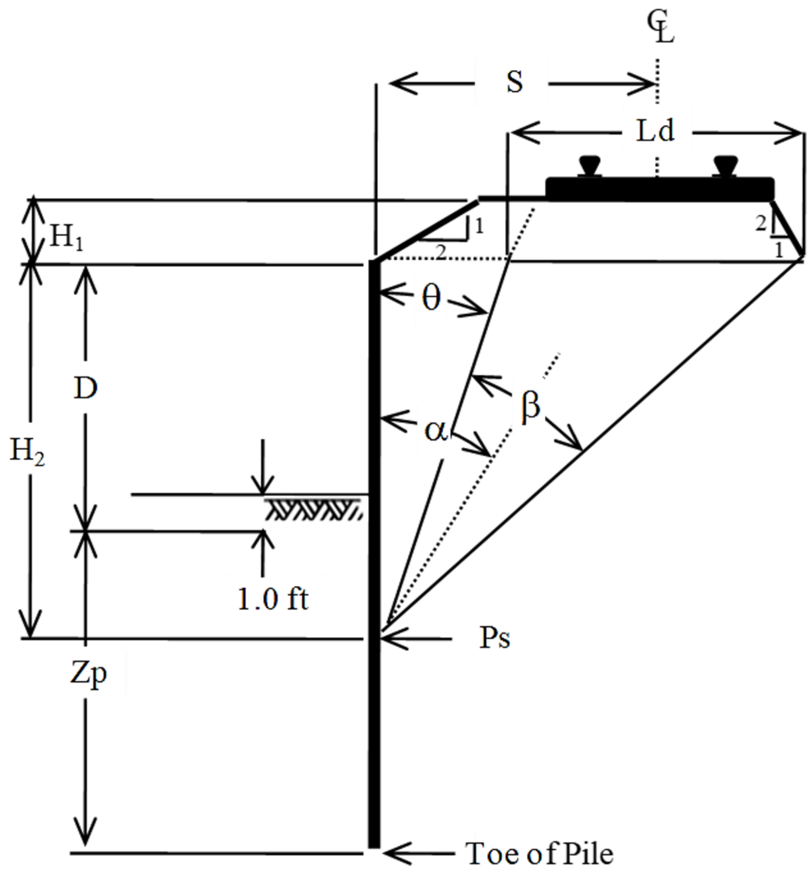


Figure 9-7, Railroad Adjacent to Soldier Pile Wall

Where:

- **P_s** = Lateral pressure due to live load.
- **ϕ** = Angle of internal friction, degrees.
- **L_d** = Length of tie (9 feet) plus **H**.
- **H_1** = Height from the bottom of tie to the top of shoring.
- **H_2** = Depth of point being evaluated with Boussinesq equation.
- **S** = Distance perpendicular from centerline of track to the face of shoring.
- **D** = Top of shoring to one foot below dredge line.
- **Z_p** = The minimum embedment depth.

Table 9-1. Horizontal Loads (Boussinesq Surcharge Pressures) at Various Depths

Depth (ft)	Load (psf)	Location
0	0	Top of shoring
5	431	
10	449	
15	326	Bottom of Excavation
30	98	Bottom of shoring
32	85	Last iteration

The general pressure diagram is shown below in Figure 9-8:

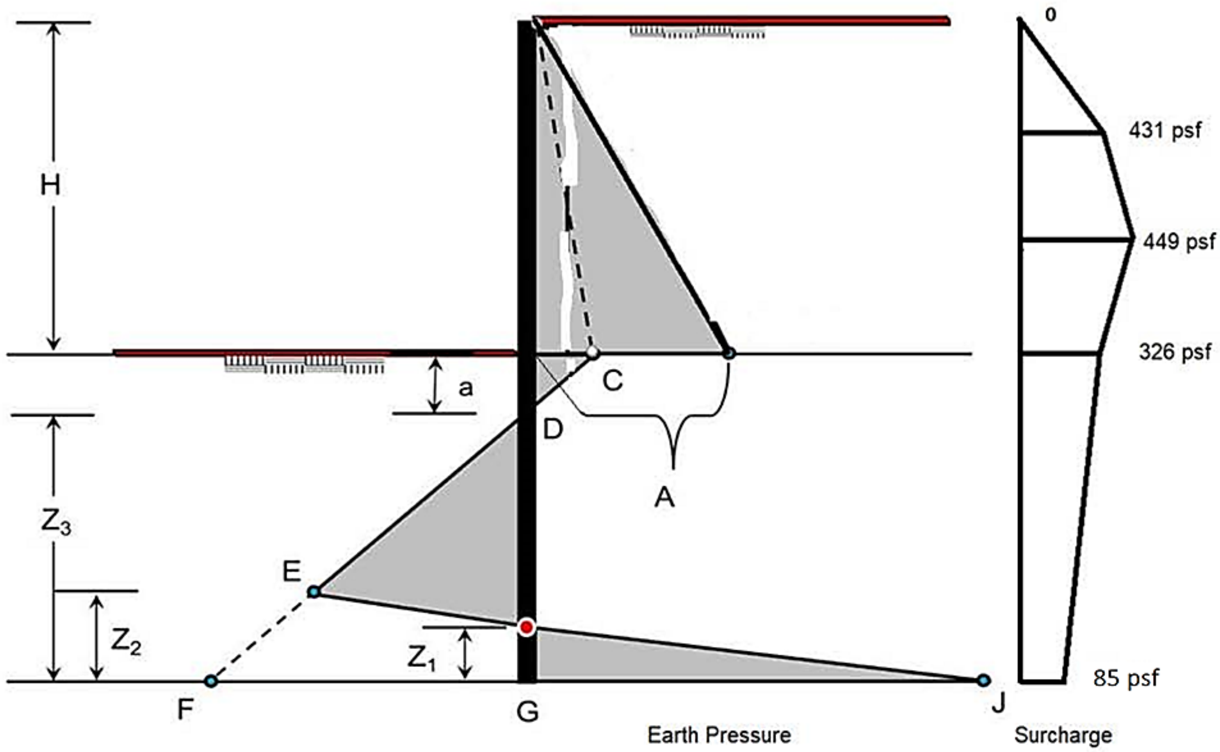


Figure 9-8. Rigorous Pressure Diagram and Horizontal Load from Surcharge

Determine Active and Passive Earth Pressures

Calculate active and passive earth pressure coefficients: Since the wall friction (**δ**) is zero, use Rankine's earth pressure theory to calculate the active and passive earth pressure coefficients (see Chapter 4, *Earth Pressure Theory and Application)*.

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \quad (9\text{-}4\text{-}8)$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \quad (9\text{-}4\text{-}9)$$

Note: Rankine's theory tends to underestimate the passive earth pressure. It is recommended to use the Log-Spiral-Rankine Model to compute the passive earth force.

From the given information, lowercase "**a**" can easily be calculated and will be needed to find the pressures at each point. The calculations below use the slope of the pressure with depth, based on the combination of the active and passive earth pressure coefficients and beginning with the pressure at point **C**. The depth of "**a**" is at the point where the earth pressure is equal to zero.

$$0 = \sigma(\text{at C})(\text{in ksf}) - \text{a}\left[\gamma(\text{kcf}) \times \left(K_p f - K_a\right)\right] \quad (9\text{-}4\text{-}10)$$

$$\text{a} = \frac{\sigma(\text{at excavation line}, \gamma H K_a)(\text{ksf})}{\gamma(\text{kcf}) \times \left(K_p f - K_a\right)}$$

$$= \frac{0.508\ \text{ksf}}{(0.125\ \text{kcf})(3.69 \times 2.8 - 0.271)} = 0.404\ \text{ft} \quad (9\text{-}4\text{-}11)$$

Note: In the above equation, "$\boldsymbol{f}$" is the arching capability factor. This factor is applied to **passive pressures below the excavation** for soldier pile systems.

$$\boldsymbol{f} = 0.08 \times \emptyset = (0.08 \times 35) = 2.8 \quad (9\text{-}4\text{-}12)$$

Calculate the earth pressure distribution in kip/ft at each node of the diagram. This implies multiplying each pressure to account for the soldier pile spacing at the various points in Figure 9-8.

- Point A - Active lateral load at excavation level on the wall:

$$A = 0.125 \times 15 \times 0.271 \times 8 = 4.065\,\frac{\text{kip}}{\text{ft}} \quad (9\text{-}4\text{-}13)$$

- Point C - Active lateral load at excavation level on the soldier pile:

$$\mathrm{C} = 0.125 \times 15 \times 0.271 \times 2 = 1.01625 \frac{\mathrm{kip}}{\mathrm{ft}} \tag{9-4-14}$$

- Point F - Passive lateral load in front of the dredge line at embedment depth:

$$\mathrm{F} = \big(0.125 \times \mathrm{Z}_3 \times \big((3.69 \times 2.8) - 0.271\big) \times 2\big) = 2.51525\mathrm{Z}_3 \ \frac{\mathrm{kip}}{\mathrm{ft}} \tag{9-4-15}$$

- Point J - Active lateral load distribution at embedment depth:

$$\begin{aligned}\mathrm{J} = &\big(0.125 \times (\mathrm{Z}_3 + 0.404) \times \big((3.69 \times 2.8) - 0.271\big) \times 2\big) \\ &+ (0.125 \times 15 \times 3.69 \times 2.8 \times 2) = 2.51525\ \mathrm{Z}_3 + 39.7612\ \frac{\mathrm{kip}}{\mathrm{ft}}\end{aligned} \tag{9-4-16}$$

Calculate resultant earth forces (**P**) and apply $\sum \mathrm{F} = 0$. The applied forces on the wall are the areas of the distributed loads.

1. Calculate active earth force due to RR surcharge:

$$\mathrm{P}_{s1} = \frac{1}{2}(431\mathrm{psf} \times 5\ \mathrm{ft}) \times 8 = 8.62\ \mathrm{kips, at\ 3.33\ ft\ from\ top\ of\ wall.} \tag{9-4-17}$$

$$\mathrm{P}_{s2} = (431\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 17.24\ \mathrm{kips, at\ 7.5\ ft\ from\ top\ of\ wall.} \tag{9-4-18}$$

$$\mathrm{P}_{s3} = \frac{1}{2}(18\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 0.36\ \mathrm{kips, at\ 8.33\ ft\ from\ top\ of\ wall.} \tag{9-4-19}$$

$$\mathrm{P}_{s4} = \frac{1}{2}(123\ \mathrm{psf}\times 5\ \mathrm{ft}) \times 8 = 2.46\ \mathrm{kips, at\ 11.67ft\ from\ top\ of\ wall.} \tag{9-4-20}$$

$$\mathrm{P}_{s5} = (326\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 13.04\ \mathrm{kips, at\ 12.5\ ft\ from\ top\ of\ wall.} \tag{9-4-21}$$

$$\mathrm{P}_{s6} = \frac{1}{2}(241\ \mathrm{psf}\times 17\mathrm{ft}) \times 2.0 = 4.10\mathrm{kips, at\ 20.67ft\ from\ top\ of\ wall.} \qquad 9\text{-}4\text{-}22)$$

$$\mathrm{P}_{s7} = (85\ \mathrm{psf} \times 17\ \mathrm{ft}) \times 2.0 = 2.89\ \mathrm{kips, at\ 23.5\ ft\ from\ top\ of\ wall.} \tag{9-4-23}$$

2. Calculate active earth force above dredge line, **P₁**:

$$\mathrm{P}_1 = \frac{1}{2} \times 4.065 \frac{\mathrm{kip}}{\mathrm{ft}} \ \times 15\ \mathrm{ft} = 30.4875\ \mathrm{kips} \tag{9-4-24}$$

3. Calculate active earth forces below dredge line, **P₂**:

$$P_2 = \frac{1}{2} \times 1.01625 \frac{\text{kip}}{\text{ft}} \times 0.404 = 0.2053 \text{ kips} \tag{9-4-25}$$

4. Calculate passive earth forces below dredge line. For simplification, take (Area FEJ) and (Area FDG):

$$\text{Area FEJ} = P_3 = \frac{1}{2} \times \left(2.51525\, Z_3 \frac{\text{kip}}{\text{ft}} + \left(2.51525\, Z_3 + 39.7612 \frac{\text{kip}}{\text{ft}}\right)\right) \times Z_2$$

$$= 2.51525\, Z_3 Z_2 + 19.881\, Z_2 \text{ kips} \tag{9-4-26}$$

$$\text{Area FDG} = P_4 = \frac{1}{2} \times 2.51525\, Z_3 \frac{\text{kip}}{\text{ft}} \times Z_3 \text{ ft} = 1.257625\, Z_3^2 \text{ kips} \tag{9-4-27}$$

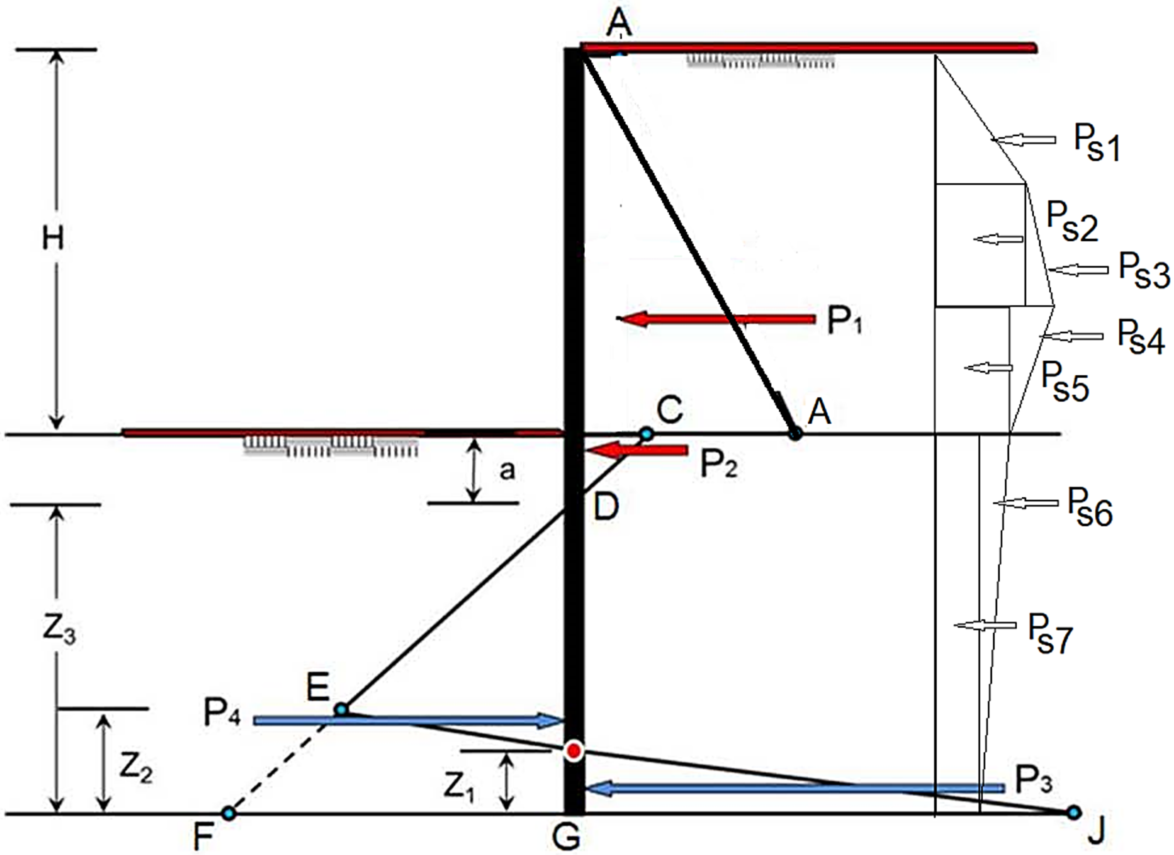


Figure 9-9. Force Diagram for Rigorous Method

Assemble a force diagram as illustrated in Figure 9-9, to display the forces and their points of application. Set up equations sum of forces and sum of moments to solve for variables **Z2** and **Z3**:

$$\Sigma F = 0 \tag{9-4-28}$$

$$P_1 + P_2 + P_3 + P_{S1} + P_{S2} + P_{S3} + P_{S4} + P_{S5} + P_{S6} + P_{S7} - P_4 = 0$$
$$30.4875 + 0.2053 + (2.51525\ Z_3Z_2 + 19.881\ Z_2) + 8.62 + 17.24 + 0.36 + 2.46 + 13.04 + 4.10 + 2.89 - 1.257625\ Z_3^2 = 0 \tag{9-4-29}$$

Simplify and solve for **Z2**:

$$Z_2 = \frac{1.257625 Z_3^2 - 79.3998}{2.51525 Z_3 + 19.881} \tag{9-4-30}$$

$$\Sigma M_G = 0 \tag{9-4-31}$$

$$\begin{aligned}
&\left(30.4875 \times (Z_3 + 0.404 + 5)\right) + \left(0.2053 \times \left(Z_3 + \frac{2(0.404)}{3}\right)\right) \\
&+ \left((2.51525 Z_3 Z_2 + 19.881 Z_2) \times \frac{Z_2}{3}\right) \\
&+ \left(8.62 \times (Z_3 + 0.404 + 11.67)\right) + \left(17.24 \times (Z_3 + 0.404 + 7.5)\right) \\
&+ \left(0.36 \times (Z_3 + 0.404 + 6.67)\right) + \left(2.46 \times (Z_3 + 0.404 + 3.33)\right) \\
&+ \left(13.04 \times (Z_3 + 0.404 + 2.5)\right) + \left(4.097 \times (Z_3 + 0.404 - 5.67)\right) \\
&+ \left(2.89 \times (Z_3 + 0.404 - 8.5)\right) - \left(1.257625\ Z_3^2 \times \left(\frac{Z_3}{3}\right)\right) = 0
\end{aligned} \tag{9-4-32}$$

Simplify and collect like terms:

$$79.3998\ Z_3 + 409.7808 + 0.83842\ Z_3 Z_2^2 + 6.627\ Z_2^2 - 0.41921\ Z_3^3 = 0 \tag{9-4-33}$$

Solve for **Z2** and **Z3** by using iteration to achieve both simplified equations to equal 0:

$$Z_2 = 4.8925\ \text{ft} \quad \& \quad Z_3 = 17.7148\ \text{ft} \tag{9-4-34}$$

Determine Embedment Depth (without a Safety Factor):

$$\text{Total Embedment Depth} = Z_3 + a = 17.7148 + 0.404 = 18.1188\ \text{ft}$$

(9-4-35)

Calculate Maximum Shear:

Maximum shear occurs when the load diagram crosses zero. In this case, the loading crosses zero at two locations, so the area of the load diagram has to be calculated before the first zero point and after the second zero point. The largest value of the two areas will be $\mathbf{V_{max}}$. Usually, it will be the area of loading below the pivot point (second zero load location) because this is where the largest passive pressure is acting at the base of the wall.

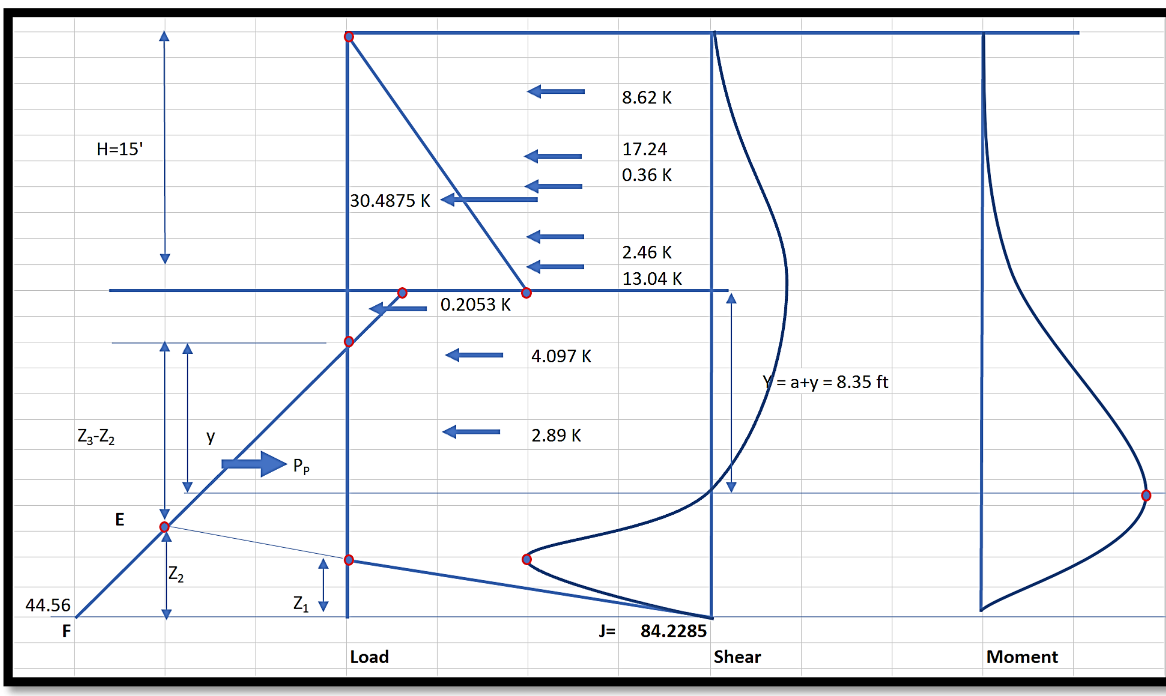


Figure 9-10. Pressure, Shear, and Moment Diagram

As illustrated in Figure 9-10, find pressure (kip/ft) at point **E** using similar triangles:

$$\frac{F}{Z_3} = \frac{E}{(Z_3 - Z_2)} \quad => \quad E = \frac{(Z_3 - Z_2)F}{Z_3} \qquad (9\text{-}4\text{-}36)$$

$$E = \frac{(17.7148\ \text{ft} - 4.8925\ \text{ft})44.56\frac{\text{kip}}{\text{ft}}}{17.7148\ \text{ft}} = 32.2534\frac{\text{kip}}{\text{ft}} \qquad (9\text{-}4\text{-}37)$$

Use similar triangles again to calculate $\mathbf{Z_1}$:

$$\frac{4.8925\ \text{ft}}{(32.2534 + 84.2285)\frac{\text{kip}}{\text{ft}}} = \frac{Z_1}{84.2285\frac{\text{kip}}{\text{ft}}} \quad => \quad Z_1 = 3.5378\ \text{ft} \qquad (9\text{-}4\text{-}38)$$

Calculate shear, $\mathbf{V_{max}}$:

$$V_{max} = \frac{1}{2} \times \left(84.2285\frac{\text{k}}{\text{ft}}\right) \times (3.5378\ \text{ft}) = 148.992\ \text{kips} \qquad (9\text{-}4\text{-}39)$$

Calculate Maximum Moment:

The maximum moment is located at distance **Y** below the excavation line where the shear is equal to zero. Therefore, the summation of horizontal forces at the distance **Y** must be set to equal zero.

Passive earth pressure at **Y** below the dredge line (Y = y+0.404):

$$P_p = \frac{1}{2}\left[0.125 \times y \times \left((3.69 \times 2.8) - 0.271\right) \times 2\right] \times y = 1.257625y^2\ \frac{\text{k}}{\text{ft}} \qquad (9\text{-}4\text{-}40)$$

Set up equation for sum of forces:

$$\Sigma F_x = 0 \qquad (9\text{-}4\text{-}41)$$

$$1.257625y^2 = 30.4875 + 0.2053 + 8.62 + 17.24 + 0.36 + 2.46 + 13.04 + 4.097 + 2.89$$
$$1.257625y^2 = 79.3998 => \quad y = 7.946\ \text{ft}$$

(9-4-42)

$$Y = 7.946 + 0.404 = 8.35\ \text{ft (below the dredge line)} \qquad (9\text{-}4\text{-}43)$$

$$M_{max} = \begin{cases} M_+ = 30.4875(5+8.35) + 0.2053\left(8.35 - \frac{0.404}{3}\right) + 8.62(11.67+8.35) + 17.24(7.5+8.35) \\ +0.36(6.67+8.35) + 2.46(3.33+8.35) + 13.04(2.5+8.35) + 5.74(15+8.35-20.67) \\ +4.05(15+8.35-23.5) = 1044.921\ \text{kips} - \text{ft} \\ M_- = 1.257625 \times 8.086^2 \times \left(\frac{8.086}{3}\right) = 221.631\ \text{kips} - \text{ft} \end{cases}$$

(9-4-44)

$$M_{max} = 1044.921\ \text{kips} - \text{ft} \ \text{(as illustrated in Figure 9-11)} \qquad (9\text{-}4\text{-}45)$$

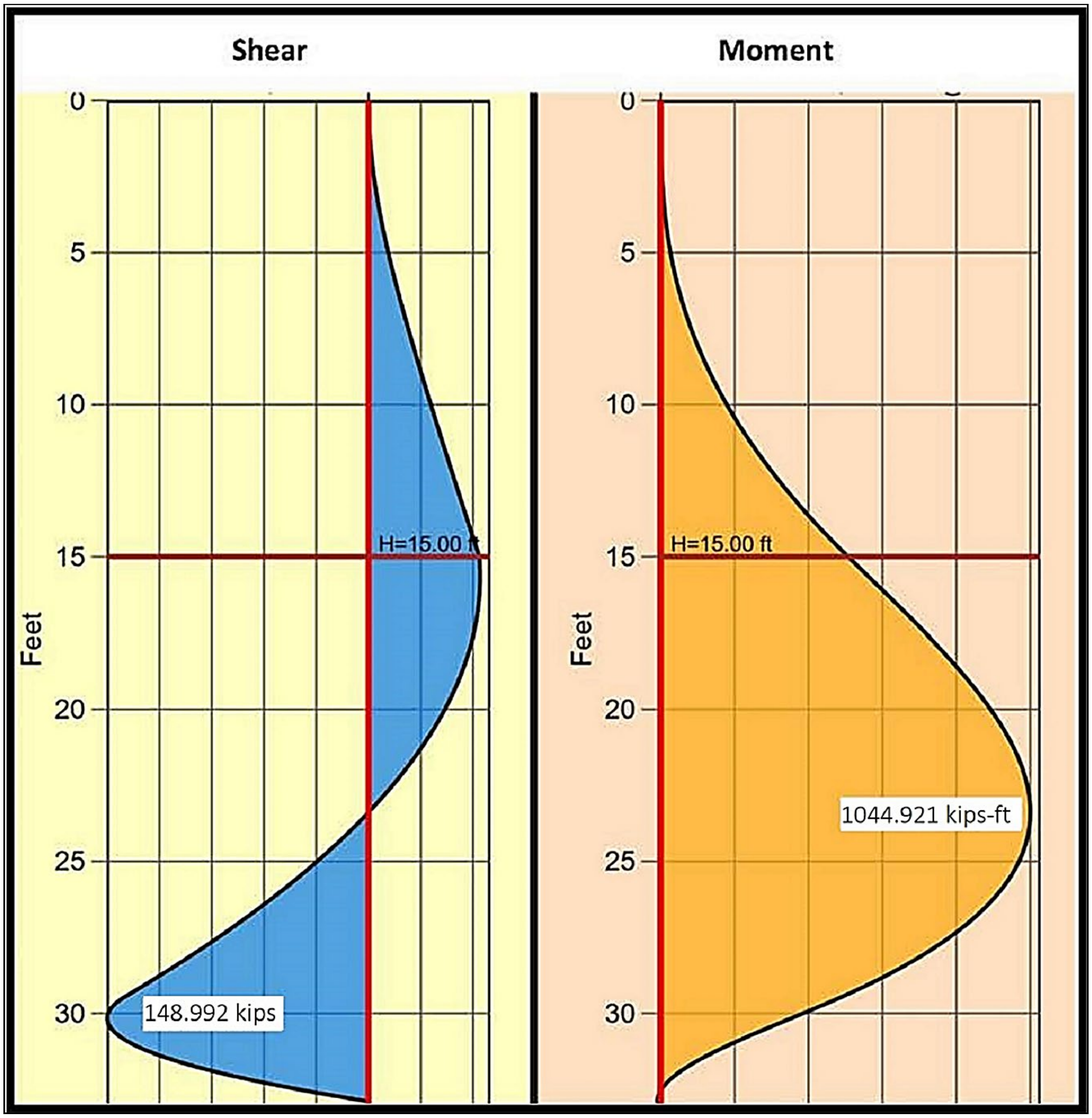


Figure 9-11. Shear and Moment Diagram

W12x336 Beam Shear and Moment:

Shear, **V** = 148.992 Kips.

$$(A = d \times t_w = 16.8 \times 1.78 = 29.904 \text{ in}^2, \text{AISC Table } 1-1) \qquad (9\text{-}4\text{-}46)$$

Compare actual versus allowable shear stress:

$$f_V = \frac{V}{A} = \frac{148{,}992 \text{ lb}}{29.904 \text{ in}^2} = 4{,}982 \text{ psi} < 14{,}400 \text{ psi } (0.4\ F_Y) \ \textbf{OK.} \qquad (9\text{-}4\text{-}47)$$

Max Moment, 1044.921 kips-ft.

$$(S_X = 483\ \text{in}^3, \text{AISC Table } 1-1) \tag{9-4-48}$$

Compare actual versus allowable bending stress:

$$f_b = \frac{M}{S_X} = \frac{1044921 \times 12\ \text{lb} - \text{in}}{483\ \text{in}^3} = 25{,}960\ \text{psi} > 23{,}760\ \text{psi},\ (0.66F_Y)\ \textbf{Not Good}. \tag{9-4-49}$$

The soldier pile W12x336 does **not** meet the requirements for bending stress. Consider increasing the soldier pile member size or decreasing the soldier pile spacing.

Calculate Maximum Deflection:

Horizontal movement or deflection of shoring systems, as described in Chapter 7, *Unrestrained Shoring Systems,* Section 7-3, *System Deflection,* can only be roughly approximated because soils do not apply pressures as true equivalent fluids, even in the totally active state. An initial deflection calculation can be made by structural mechanics procedures (moment area – M/EI); sound engineering judgment should be used to analyze the results. Various factors can affect the movement of the shoring system, including soil type, stage construction, and the duration of time that the shoring is in service. Monitoring or performance testing is also important. Illustrated below in Figure 9-12, is the deflection obtained from CT_T&S Program; note that the values shown are a close approximation.

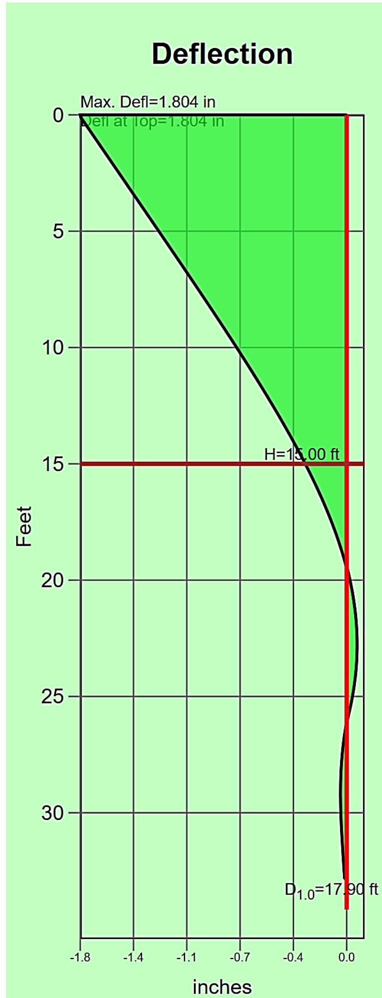


Figure 9-12. Deflection (CT_T&S Program)

To comply with the deflection limit in Table 2, *Deflection Criteria*, of the GUIDELINES, consider increasing the soldier pile member size or reducing the soldier pile spacing.

For lagging calculations, see Chapter 6, *Structural Design of Shoring Systems*, Section 6-5.01, *Example Lagging Calculations*.

CHAPTER 10

SPECIAL CONDITIONS

Chapter 10: Special Conditions

Table of Contents

Chapter 10: Special Conditions 1
10-1 Special Conditions 2
10-2 Use of Anchor Blocks 3
10-2.01 Anchor Block in Cohesionless Soil 5
10-2.02 Anchor Block in Cohesionless Soil Where 1.5 ≤ D/H ≤ 5.5 – Alternative Method 10
10-2.03 Anchor Block in Cohesive Soil Near the Ground Surface d ≤ H/2 11
10-2.04 Anchor Blocks in Cohesive Soil Where d ≥ H/2 12
10-2.05 Example 10-1 Problem – Anchor Blocks 14
10-3 Heave 16
10-3.01 Factor of Safety against Heave 19
10-3.02 Example 10-2 Problem – Heave Factor of Safety 20
10-4 Piping 21
10-4.01 Hydraulic Forces on Cofferdams and Other Structures 22
10-5 Slope Stability 25
10-5.01 Rotational Slides 26
10-5.02 Fellenius Method 29
10-5.03 Bishop Method 33
10-5.04 Translational Slide 35
10-5.05 Stability Analysis of Shoring Systems 39

10-1 Special Conditions

The best shoring system design in the world is of little value if the soil being supported does not act as contemplated by the designer. Potential adverse soil properties and changing conditions need to be considered and monitored. This chapter reviews several of the more common challenges.

Proper placement of anchor blocks (similar to ground anchors) is one challenge, and easily overlooked. Anchors placed within a soil failure wedge will not provide the resistance value needed when soil movement in the active zone occurs. Additional information regarding anchors may be found in the *USS Steel Sheet Piling Design Manual* (note that additional information on this resource can be found in Appendix D, *Sheet Piles*).

Another challenge is soil movement within the shoring system. Cohesive soils tend to expand and may push upward into an excavation. Expanding soils may also produce additional forces on the shoring system, and may induce lateral movement of the shoring system. Soil rising in an excavation indicates that soil is settling somewhere else. Water rising in an excavation can lead to quick conditions, while water moving horizontally can transport soil particles, possibly leaving unwanted voids at critical locations. Soil heave (movement of the soil in the bottom of the excavation due the soil pressures outside the system) is another condition to be aware of.

Excavating in an area with a high water table or within a waterway is another challenging condition. A cofferdam shoring system is generally employed for these conditions. This chapter reviews the topic of ”piping” and stream flow pressures against the exposed sided of a cofferdam in which an unbalanced hydrostatic head occurs. The sizing of seal-course concrete often used at the bottom of a cofferdam is discussed in the Structure Construction (SC) *Foundation Manual.*

The last challenge presented is stability of the soils around the shoring system. A consideration for the stability of exposed slopes, and that of global stability of the system, is the potential for a failure due to slippage of the soil around the shoring system along a surface offering the least amount of resistance. This potential is present for most types of shoring systems, with the exception being two-sided systems in level ground (i.e., a traditional trench). Although global failures usually happen suddenly, occasionally there are indications of small slope movements hours and sometimes days before a global failure.

Sample situations of the above are included on the following pages.

10-2 Use of Anchor Blocks

Lateral support for sheet pile and/or soldier pile walls can be provided by tie rods that extend to an anchor block (also called deadman anchor) sufficiently behind the face of the excavation. Each rod can be connected to a single anchor block or multiple blocks spaced along the rod. Occasionally, a shoring design may utilize not an individual block, but a continuous block (wall) running parallel to the excavated face. See Figure 10-1**.** Tie rod spacing is generally determined by the need to limit the deflection of the shoring face, and the maximum moment of the soldier pile.

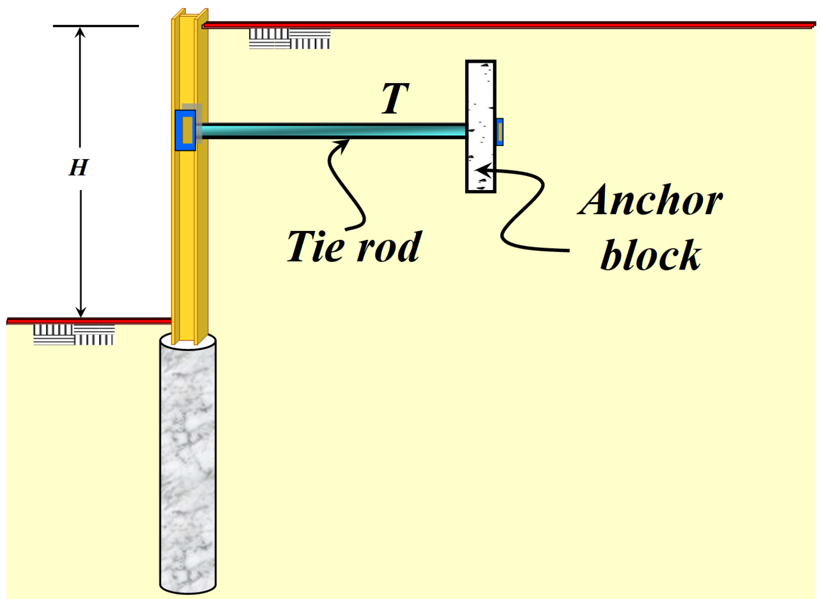


Figure 10-1. Anchor Block and Tie Rod

Using anchor blocks is simply another means for designing a restrained system, rather than using drilled ground anchors or struts. Use of an anchor block differs from ground anchors by how they develop resistance. An anchor block system's resistance is through the passive pressure developed in front of the anchor block, rather than soil-ground anchor bond strength along the bonded section of a drilled hole.

The size, shape, depth, and location of an anchor block affect the resistance capacity developed by that anchor. Figure 10-2 illustrates how the distance of the anchor block from the wall affects capacity.

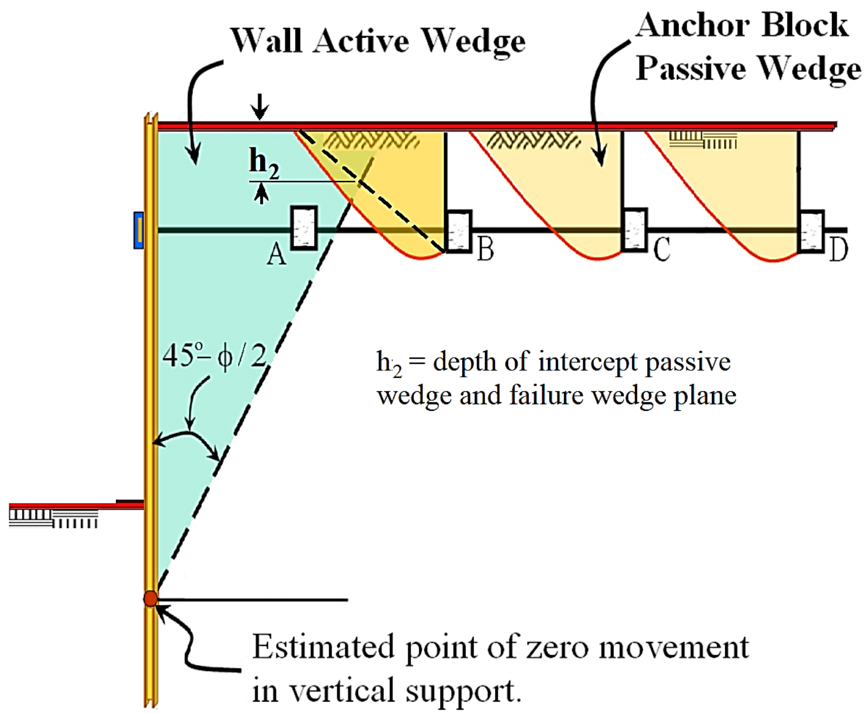


Figure 10-2. Anchor Block Position Relative to Wall Face and Failure Plain

1. Anchor block A is located inside active wedge and offers no resistance.
2. Anchor block B resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).
   - Anchor reduction: (Granular soils)

$$\boldsymbol{\Delta P_p = \frac{\gamma h_2^2 (K_p - K_a)}{2}} \tag{10-2-1}$$

   - This reduced portion becomes a load transferred to the wall as well.
3. Anchor block C develops full capacity but increases pressure on the wall.
4. Anchor block D is similar to Anchor block C. Anchor block D develops full capacity but increases pressure on the wall.

To follow are some considerations when dealing with anchor blocks. Anchor blocks should be placed against firm, undisturbed, or recompacted soil, and a safety factor of 2 is recommended for all anchor blocks. The capacities of anchor blocks are, of course, a function of the soil parameters. Other factors which affect the resistance of an anchor block include the depth of the anchor relative to the ground surface, and the proportions of the block and its spacing; i.e., whether it behaves as a continuous or a singular element. In the following section, we will examine properly located anchors at position **D** in Figure 10-2 (assuming cohesionless soils), beginning with a continuous block at or near the ground surface.

10-2.01 Anchor Block in Cohesionless Soil

10-2.01A Case A - Anchor blocks at or near the ground surface; d ≤ H/2

The forces acting on an anchor block at the ground surface are shown in Figure 10-3.

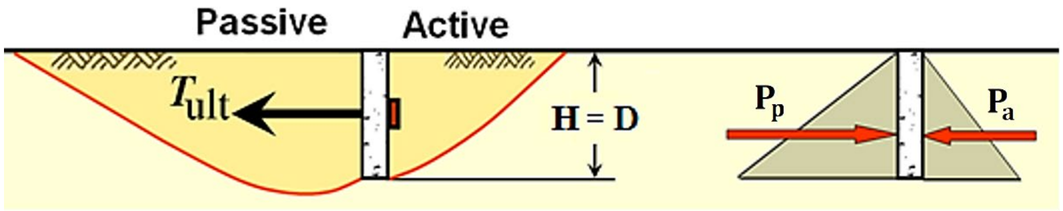


Figure 10-3. Anchor Block in Cohesionless Soil at the Ground Surface

When the block is not at the surface but the depth, **d**, is within **H/2,** it is assumed the influence of the block does extend to the ground surface as shown in Figure 10-4.

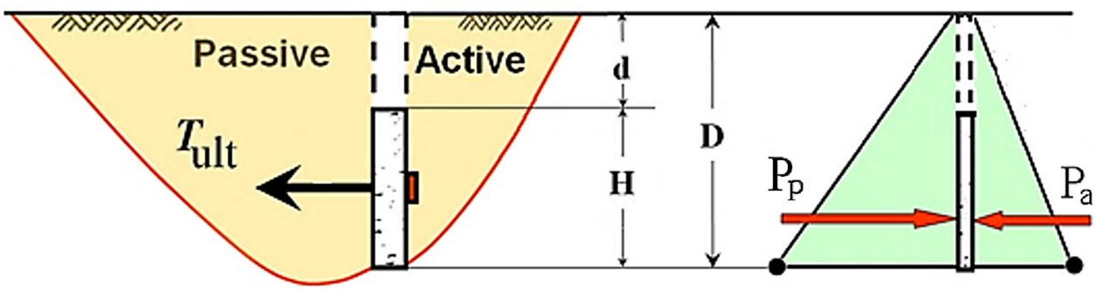


Figure 10-4. Continuous Anchor Block in Cohesionless Soil within H/2 of Ground Surface

Note: For Figures 10-3, 10-4, 10-8, and 10-11, the right side of each figure does not show the force pulling on the anchor block. That force is equal to **T**, and the maximum value for **T** is equal to $\mathbf{T_{ult}}$.

The basic equation for a continuous anchor block to calculate the ultimate capacity is shown in Equation 10-2-1. If **L** is assumed to be 1 foot, the result is the capacity in pounds per linear foot of the block.

$$\mathbf{T_{ult}} = \mathbf{L}(\mathbf{P_p} - \mathbf{P_a}) \quad (10\text{-}2\text{-}2)$$

Where:

$$\mathbf{P_a} = \mathbf{K_a}\boldsymbol{\gamma}\frac{\mathbf{D^2}}{\mathbf{2}} \quad (10\text{-}2\text{-}3)$$

$$\mathbf{P_p} = \mathbf{K_p}\boldsymbol{\gamma}\frac{\mathbf{D^2}}{\mathbf{2}} \quad (10\text{-}2\text{-}4)$$

Substituting Equation 10-2-3 and Equation 10-2-4 into Equation 10-2-2 then:

$$\mathbf{T_{ult}} = \boldsymbol{\gamma} \frac{\mathbf{D^2}}{\mathbf{2}} \left(\mathbf{K_p} - \mathbf{K_a}\right)\mathbf{L} \tag{10-2-5}$$

L = Length of the anchor block (depicted as "**w**" in Figure 10-5).

The conventional earth pressure theories using two-dimensional conditions corresponding to long (continuous) walls can be used to calculate the resistance force against the anchor block movement. An anchor block is considered continuous when its length exceeds its height by three or more times. The anchor block is otherwise considered to be isolated and has the advantage of an increased capacity by considering a three-dimensional analysis as described below.

In the case of isolated or short anchor blocks (see Figure 10-5 and 10-6 below), a larger passive pressure may develop because of three-dimensional effects due to a curved failure surface at both ends of the block, resulting in a wider passive zone in front of the anchor block, as shown below.

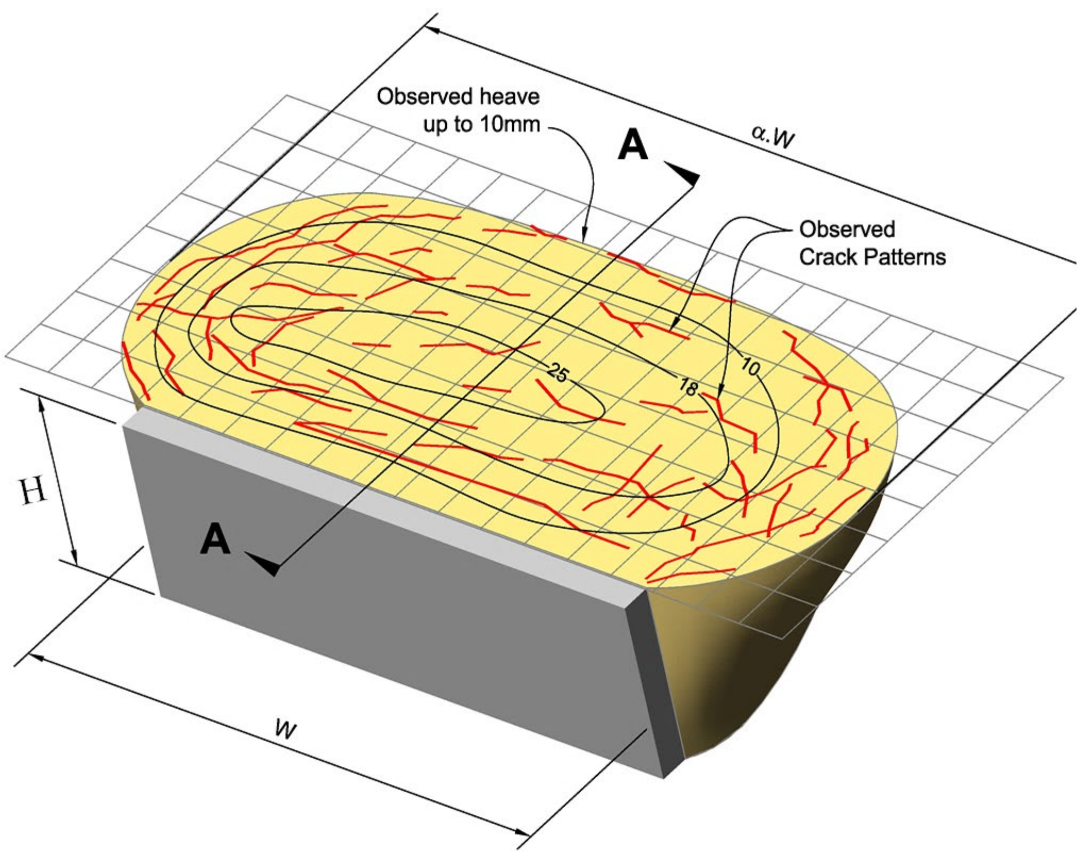


Figure 10-5. Anchor Block in 3D (Shamsabadi, A., Nordal, S., 2006)

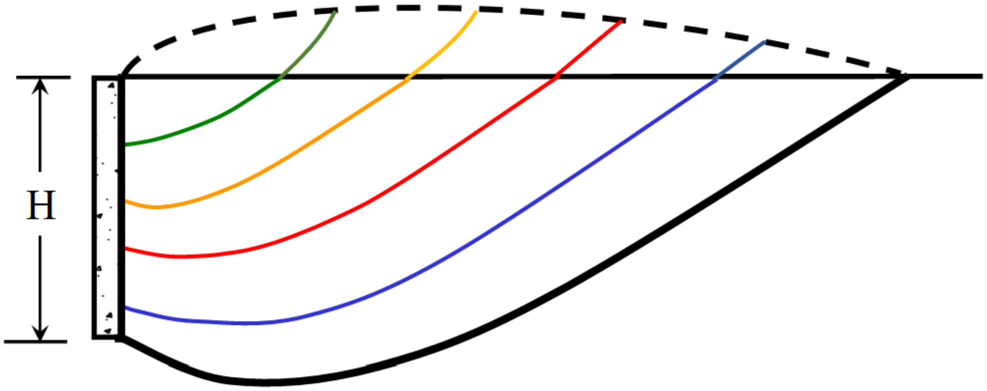


Figure 10-6. Section A-A (Shamsabadi, A., et al., 2007)

The ratio between three-dimensional and two-dimensional soil resistance varies with the soil friction angle and the depth below the ground surface. N. K. Ovesen studied and performed 32 different model tests for fully mobilized anchor blocks in granular soil. The resulting figures can be used to estimate the magnitude of the three-dimensional effects. Ovesen's method utilizes a three-dimensional factor, **R**, based on his test results, which differentiate the results of isolated versus continuous blocks. Figure 10-7 illustrates isolated anchor blocks near the ground surface.

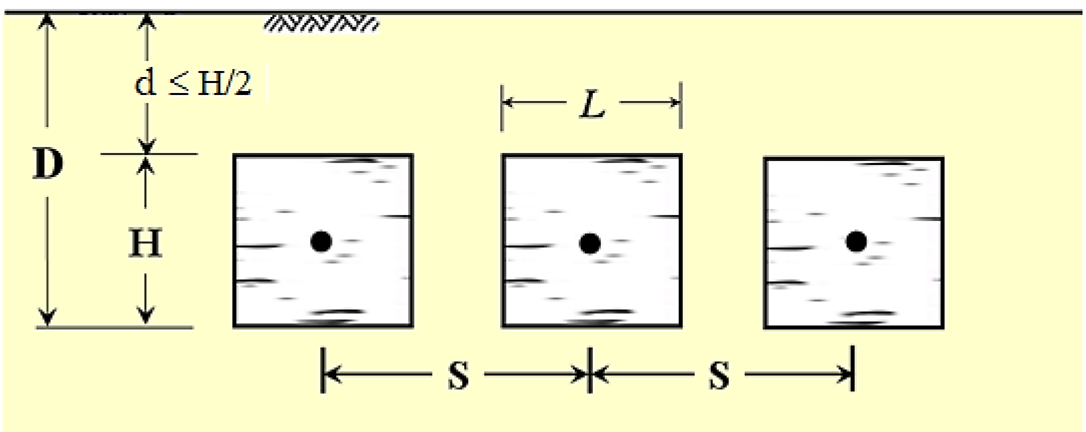


Figure 10-7. Isolated Anchor Blocks in Cohesionless Soil

$$\mathbf{T_{ult}} = \mathbf{R}\left[\boldsymbol{\gamma}\frac{\mathbf{D^2}}{\mathbf{2}}\boldsymbol{\Delta}\mathbf{KL}\right] \quad (10\text{-}2\text{-}6)$$

$$\boldsymbol{\Delta}\mathbf{K} = \left(\mathbf{K_p} - \mathbf{K_a}\right) \quad (10\text{-}2\text{-}7)$$

L = Length of the anchor block.

$$\mathbf{R}/_{\Delta\mathbf{K}} = \mathbf{1} + \Delta\mathbf{K}^{2/3}\left[\mathbf{1.1E^4} + \frac{\mathbf{1.6B}}{\mathbf{1}+\mathbf{5}\frac{\mathbf{L}}{\mathbf{H}}} + \frac{\mathbf{0.4\Delta KE^3B^2}}{\mathbf{1}+\mathbf{0.05}\frac{\mathbf{L}}{\mathbf{H}}}\right] \qquad (10\text{-}2\text{-}8)$$

Where:

$$\mathbf{B} = \mathbf{1} - \left(\frac{\mathbf{L}}{\mathbf{S}}\right)^{2} \qquad (10\text{-}2\text{-}9)$$

and,

$$\mathbf{E} = \mathbf{1} - \frac{\mathbf{H}}{\mathbf{d}+\mathbf{H}} \qquad (10\text{-}2\text{-}10)$$

10-2.01B Case B - Anchor block failure surfaces do not extend to the ground surface 1.5 ≤ D/H≤ 5.5

The forces acting on an anchor, which is not near the ground surface, are shown in Figure 10-8.

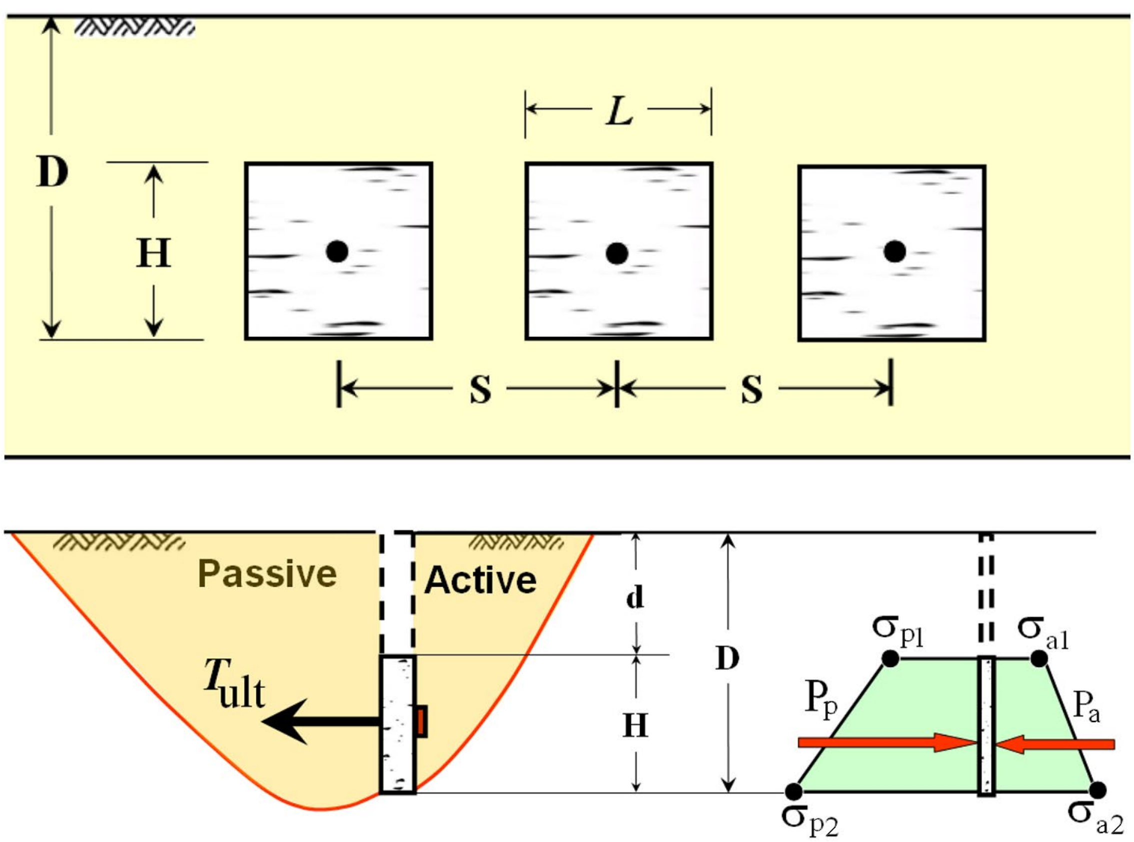


Figure 10-8. Anchor Block not near the Ground Surface: 1.5 ≤ D/H≤ 5.5

The basic equation to calculate the capacity of a continuous anchor block with length **L**, not extended near the ground surface, is shown in Equation 10-2-2.

$$\mathbf{T_{ult} = L\left(P_p - P_a\right)}$$

Where the $\mathbf{P_a}$ and $\mathbf{P_p}$ are the resultant forces from the areas of active and passive earth pressure developed in the front and back of the anchor block, as shown in Figure 10-8 and Equations 10-2-11 and 10-2-14.

$$\mathbf{P_a = \left[\frac{\sigma_{a1} + \sigma_{a2}}{2}\right] H} \tag{10-2-11}$$

Where:

$$\boldsymbol{\sigma_{a1} = \gamma d K_a} \tag{10-2-12}$$

and,

$$\boldsymbol{\sigma_{a2} = \gamma D K_a} \tag{10-2-13}$$

$$\mathbf{P_p = \left[\frac{\sigma_{p1} + \sigma_{p2}}{2}\right] H} \tag{10-2-14}$$

Where:

$$\boldsymbol{\sigma_{p1} = \gamma d K_p} \tag{10-2-15}$$

and,

$$\boldsymbol{\sigma_{p2} = \gamma D K_p} \tag{10-2-16}$$

Substituting Equation 10-2-11 and Equation 10-2-14 into Equation 10-2-2 then:

$$\mathbf{T_{ult} = L\left[\frac{1}{2}\left(\left(\boldsymbol{\gamma} dK_p + \boldsymbol{\gamma} DK_p\right) - \left(\boldsymbol{\gamma} dK_a + \boldsymbol{\gamma} DK_a\right)\right)\right] H} \tag{10-2-17}$$

In case of isolated and short anchor blocks, the Ovesen's three-dimensional factor (**R**) must be estimated using Equation 10-2-8. Then use the following equation for $\mathbf{T_{ult}}$.

$$\mathbf{T_{ult} = R\,(L)\left[\frac{1}{2}\left(\left(\boldsymbol{\gamma} dK_p + \boldsymbol{\gamma} DK_p\right) - \left(\boldsymbol{\gamma} dK_a + \boldsymbol{\gamma} DK_a\right)\right)\right] H} \tag{10-2-18}$$

10-2.02 Anchor Block in Cohesionless Soil Where 1.5 ≤ D/H ≤ 5.5 – Alternative Method

The chart shown in Figure 10-9 is based on sand of medium density (ϕ = 32.5 degrees). For other values of ϕ, a linear correlation may be made from (ϕ / 32.5 degrees). The chart is valid for ratios of depth to height of anchor (**D/H**), between 1.5 and 5.5.

For square anchor blocks, the value from the chart (**K_P'**) is larger than the value for continuous anchor blocks (**K_p**). This is because the failure surface is larger than the actual dimensions of the anchor block. In testing it is determined to be approximately twice the width.

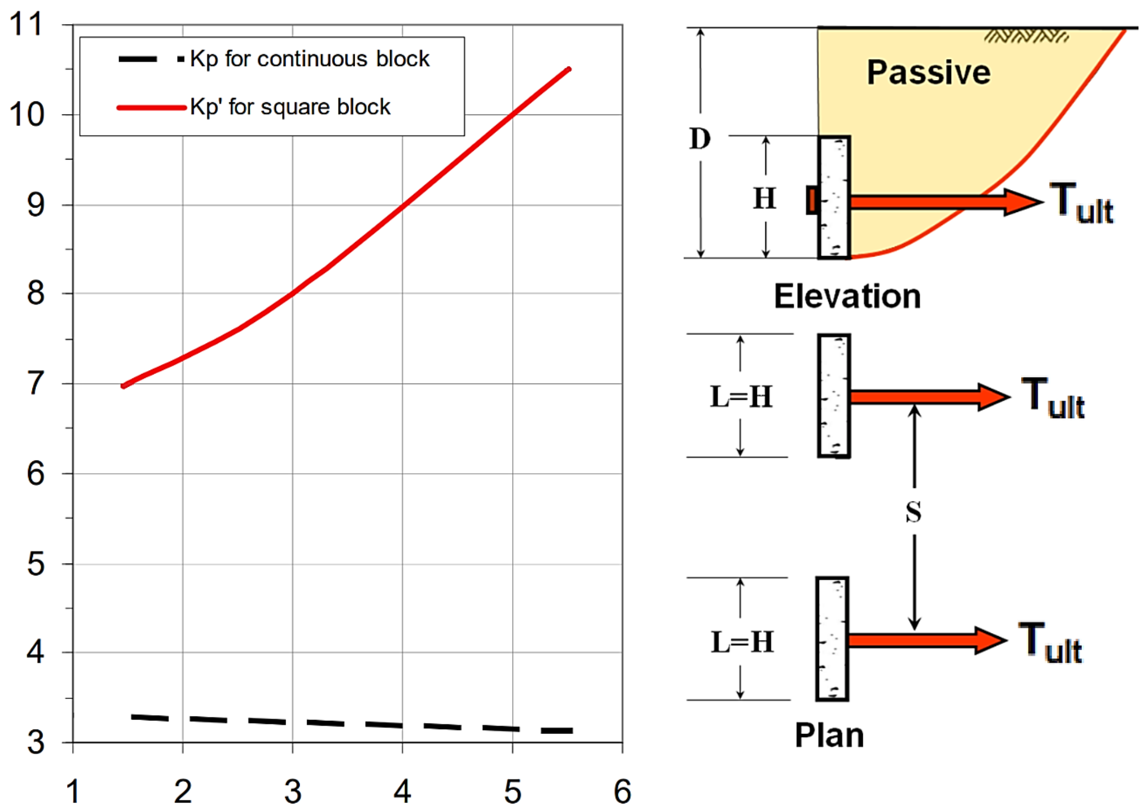


Figure 10-9. Anchor Block in Cohesionless Soil 1.5 ≤ D/H ≤ 5.5 – Alternate Method

$$\mathbf{T_{ult}} = [\boldsymbol{\gamma}\mathbf{D^2K_P}'\,(\mathbf{L})]/\mathbf{2} \qquad (10\text{-}2\text{-}19)$$

10-2.03 Anchor Block in Cohesive Soil Near the Ground Surface d ≤ H/2

Recall from Chapter 4 that for cohesive soil, the pressure diagrams for the active and passive forces look as shown in Figure 10-10. Thus the forces acting on an anchor are shown in Figure 10-11. For the case of **d ≤ H/2**, where **H** is the height of the block, it is assumed that the anchor essentially extends to the ground surface. The capacity of the anchor depends upon whether it is considered continuous or short.

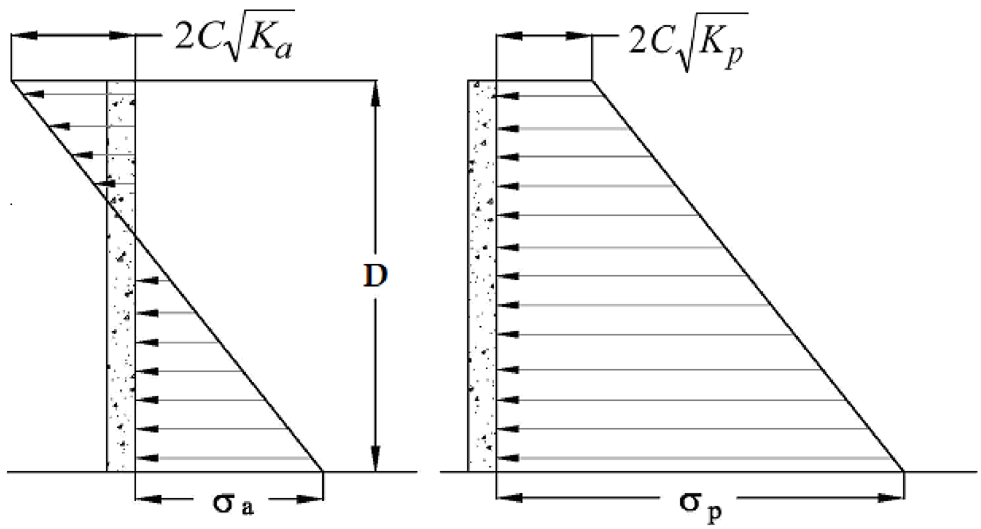


Figure 10-10. Cohesive Soil Pressure Diagrams

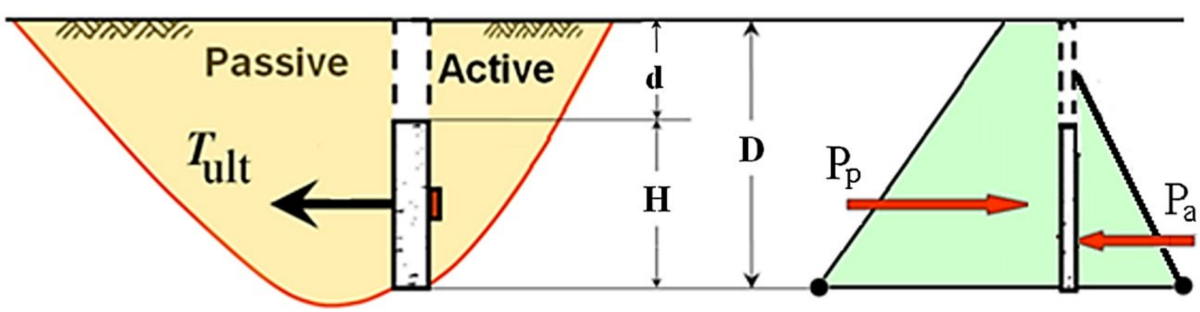


Figure 10-11. Anchor Block in cohesive soil near the ground surface d ≤ H/2

Where:

$$\sigma_p = \gamma D K_p + 2C\sqrt{K_p} \quad (10\text{-}2\text{-}20)$$

$$\sigma_a = \gamma D K_a - 2C\sqrt{K_a} \quad (10\text{-}2\text{-}21)$$

The pressure diagram shown in Figure 10-10 for cohesive soils assumes short load duration. Over a period of years, creep is likely to alter the pressure diagram. Therefore, conservative assumptions should be used in the analysis, such as **c** = 0 and ϕ = 27°.

The basic equation is:

$$\mathbf{T_{ult} = L(P_p - P_a)} \tag{10-2-22}$$

Where: **L** = Length of anchor block.

For continuous anchor blocks:

$$\mathbf{P_p = \frac{\gamma D^2 K_p}{2} + 2CD\sqrt{K_p}} \tag{10-2-23}$$

$$\mathbf{P_a = \frac{\left(\gamma D K_a - 2C\sqrt{K_a}\right)\left(D - \frac{2C}{\gamma}\right)}{2}} \tag{10-2-24}$$

It is recommended that the tension zone be neglected.

For short anchor blocks where $\mathbf{H \leq L}$:

$$\mathbf{T_{ult} = L(P_p - P_a) + 2CD^2} \tag{10-2-25}$$

10-2.04 Anchor Blocks in Cohesive Soil Where d ≥ H/2

The chart shown in Figure 10-12 was developed through testing for anchor blocks other than near the surface. The chart relates a dimensionless coefficient (**R**) to the ratios of depth to height of an anchor (**D/H**) to determine the capacity of the anchor block. The chart applies to continuous anchors only. Figure 10-12 is from *Strength of Deadmen Anchors in Clay*, Thomas R. Mackenzie, Master's Thesis Princeton University, Princeton, New Jersey, 1955.

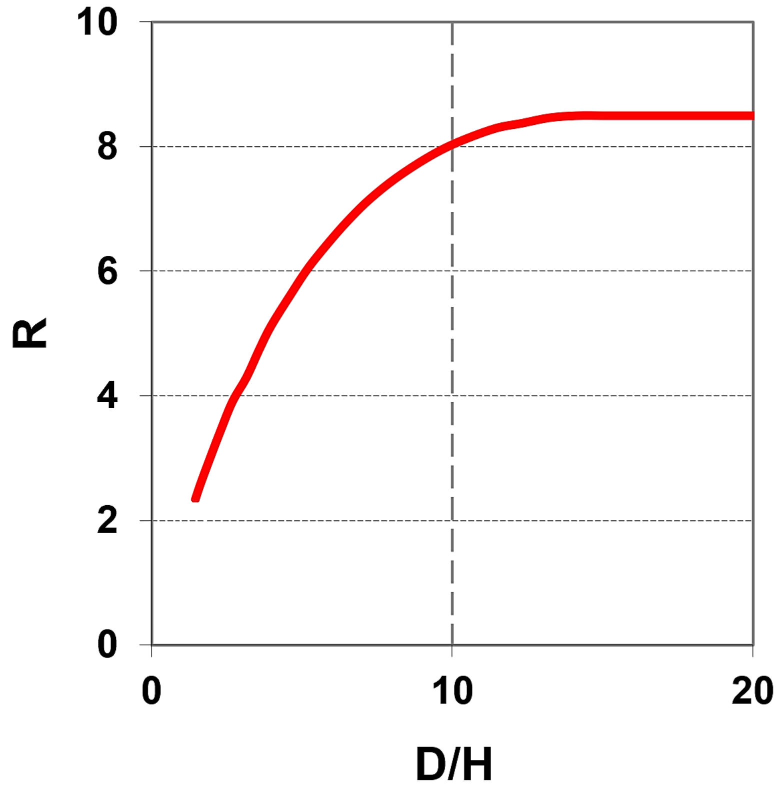


Figure 10-12. Anchor Block in Cohesive Soil d ≥ H/2

$\mathbf{P_{ult}} = \mathbf{RCHL}$ with a maximum value of $\mathrm{R} = 8.5.$

When using the graph in Figure 10-12, the reader needs to check that they are using a value of **D/H** that is greater than or equal to 1.5.

10-2.05 Example 10-1 Problem – Anchor Blocks

Given:

Check the adequacy of the Contractor's anchor blocks in the proposed shoring system shown in Figure 10-13. The 2 foot wide by 2 foot long anchor blocks are to be buried 3 feet below the ground surface. The required tie load on the wall is 11,000 lbs.

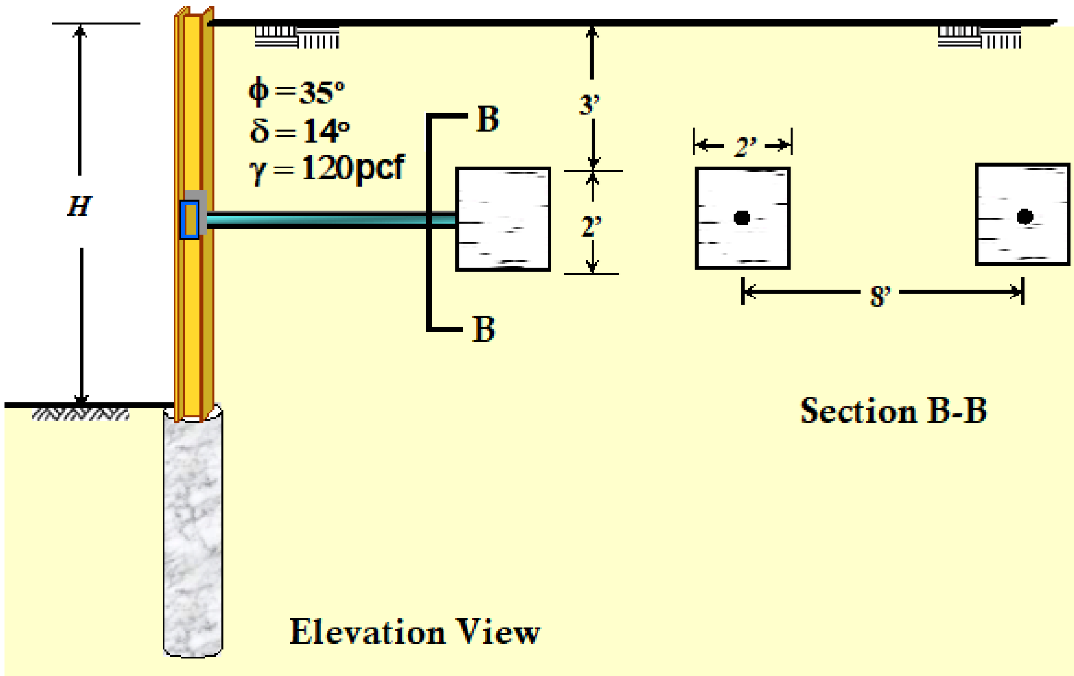


Figure 10-13. Anchor Block Example 10-1

Solution:

Step 1: Calculate active and passive earth pressure in the front and back of the anchor block. Begin by calculating the active and passive coefficients. Using the Coulomb equations, $\mathbf{K_a}$ = 0.27 and $\mathbf{K_p}$ = 6.27. The soil is cohesionless, and the cohesion value (**c** value) is equal to zero.

Since wall friction, δ, is included in the given information, these calculations take into account this friction (see Chapter 4, Section 4-3, *Developing Earth Pressures for Granular Soil,* for additional information on these topics).

$$\sigma_{a1} = \gamma d k_a \times \cos(\delta) = 120 \times 3 \times 0.27 \times \cos(14°) = 94.31 \qquad (10\text{-}2\text{-}26)$$

$$\sigma_{a2} = \gamma H k_a \times \cos(\delta) = 120 \times 5 \times 0.27 \times \cos(14°) = 157.19 \qquad (10\text{-}2\text{-}27)$$

$$P_a = \left[\frac{94.31 + 157.19}{2}\right] 2 = 251.50 \qquad (10\text{-}2\text{-}28)$$

$$\sigma_{p1} = \gamma d K_p \times \cos(\delta) = 120 \times 3 \times 6.27 \times \cos(14°) = 2{,}190.15 \quad (10\text{-}2\text{-}29)$$

$$\sigma_{p2} = \gamma H K_p \times \cos(\delta) = 120 \times 5 \times 6.27 \times \cos(14°) = 3{,}650.25 \quad (10\text{-}2\text{-}30)$$

$$P_p = \left[\frac{2{,}190.15 + 3{,}650.25}{2}\right]2 = 5{,}840.40 \quad (10\text{-}2\text{-}31)$$

Step 2: Use Ovesen's theory to estimate the magnitude of the three-dimensional effects **R**, using Equation 10-2-8.

$$R = 1 + \Delta K^{2/3}\left[1.1E^4 + \frac{1.6B}{1 + 5\frac{L}{H}} + \frac{0.4\Delta K E^3 B^2}{1 + 0.05\frac{L}{H}}\right]$$

$$\Delta K_{horz} = (K_p - K_a)\cos(\delta) = (6.27 - 0.27)\cos(14°) = 5.82 \quad (10\text{-}2\text{-}32)$$

$$B = 1 - \left(\frac{L}{S}\right)^2 = 1 - \left(\frac{2}{8}\right)^2 = 0.94 \quad (10\text{-}2\text{-}33)$$

$$E = 1 - \frac{H}{d + H} = 1 - \frac{2}{3 + 2} = 0.60 \quad (10\text{-}2\text{-}34)$$

$$R = 1 + 5.82^{2/3}\left[1.1 \times 0.6^4 + \frac{1.6 \times 0.94}{1 + 5\frac{2}{2}} + \frac{0.4 \times 5.82 \times 0.60^3 \times 0.94^2}{1 + 0.05\frac{2}{2}}\right] = 3.64$$

(10-2-35)

R = 3.64 > 2.0

Use: R=2 (see below)

The **R** value is the Ovesen factor that is equal to the 3-dimensional ultimate load divided by the 2-dimensional ultimate load. If the calculated **R** value is more than 2, then the **R** value that is used should be equal to 2.

Step 3: Calculate ultimate anchor block capacity, $\mathbf{T_{ult}}$.

$$T_{ult} = R \times (P_p - P_a) \times L = 2 \times (5{,}840.40 - 251.50) \times 2 = 22{,}355.6 \text{ lb/ft}$$

(10-2-36)

Where **L** is the length of the anchor block.

$$FS = \frac{T_{ult}}{T} = \frac{22{,}355.6}{11{,}000.0} = 2.03 \quad (10\text{-}2\text{-}37)$$

10-3 Heave

The condition of heave can occur in soft plastic clays when the depth of the excavation is sufficient to cause the surrounding clay soil to displace vertically with a corresponding upward movement of the material in the bottom of the excavation.

The possibility of heave and slip circle failure in soft clays, and in the underlying clay layers, should be checked when the Stability Number ($\mathbf{N_O}$) exceeds 6.

Stability Number, $\mathbf{N_O = \gamma\, H/c}$ (10-3-1)

Where:

γ = Unit weight of the soil in pcf

$\mathbf{H}$ = Height of the excavation in ft

$\mathbf{c}$ = Cohesion of soil in psf

Braced cuts in clay may become unstable as a result of heaving of the bottom of the excavation. Terzaghi (1943) analyzed the factor of safety of long braced excavations against bottom heave. The failure surface for such a case is shown in Figure 10-14. The vertical load per unit length of the trench length at the bottom of the supports along line ***dc*** is the driving force to create heave in pounds per length of trench (plf). It can be calculated:

$$\mathbf{Q = W + (0.7B)q - S} \qquad (10\text{-}3\text{-}2)$$

Where:

$\mathbf{Q}$ = Vertical load per unit length of trench.

$\mathbf{W}$ = Weight of soil column per unit length of trench $\mathbf{W = \gamma\, H\,(0.7B)}$.

$\mathbf{B}$ = Width of open excavation in feet.

$\mathbf{q}$ = Surcharge loading in psf.

$\mathbf{S}$ = Resistance of soil due to cohesion over depth of excavation,

S = (cH) in pound per unit length of the trench.

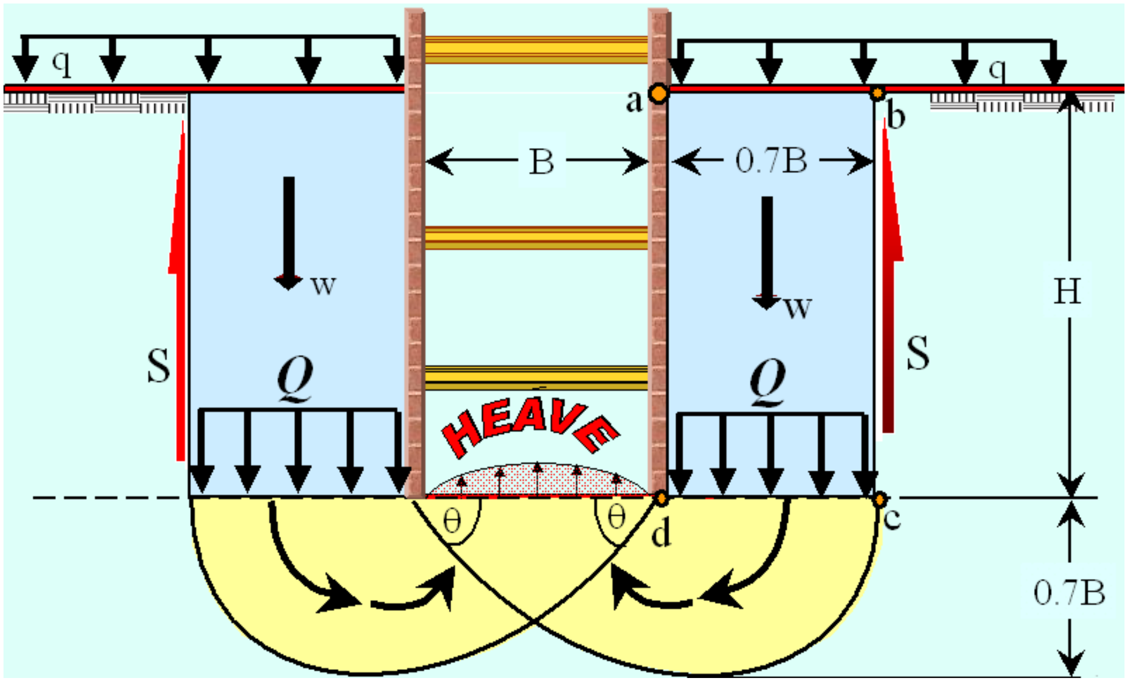


Figure 10-14. Bottom Heave

Through considering the mechanics of heave, the driving force may be treated as a load per unit length on a continuous foundation at the level of ***dc***, with the width of **0.7B**, and thus compare it to the bearing capacity analysis of a footing.

The resisting force is based on Terzaghi's bearing capacity theory by considering the driving force, **Q**, as a unit load from a foundation. The equation for the net ultimate load-carrying capacity per unit length per Terzaghi is:

$$\mathbf{Q_U = cN_C(0.7B)} \qquad (10\text{-}3\text{-}3)$$

$\mathbf{Q_U}$ = Ultimate load carry capacity per unit length = Ultimate bearing capacity

$\mathbf{c}$ = Cohesion of soil in psf

$\mathbf{N_c}$ = Bearing capacity factor from Figure 10-15 and Equation 10-3-4

$\mathbf{B}$ = Width of open excavation in feet

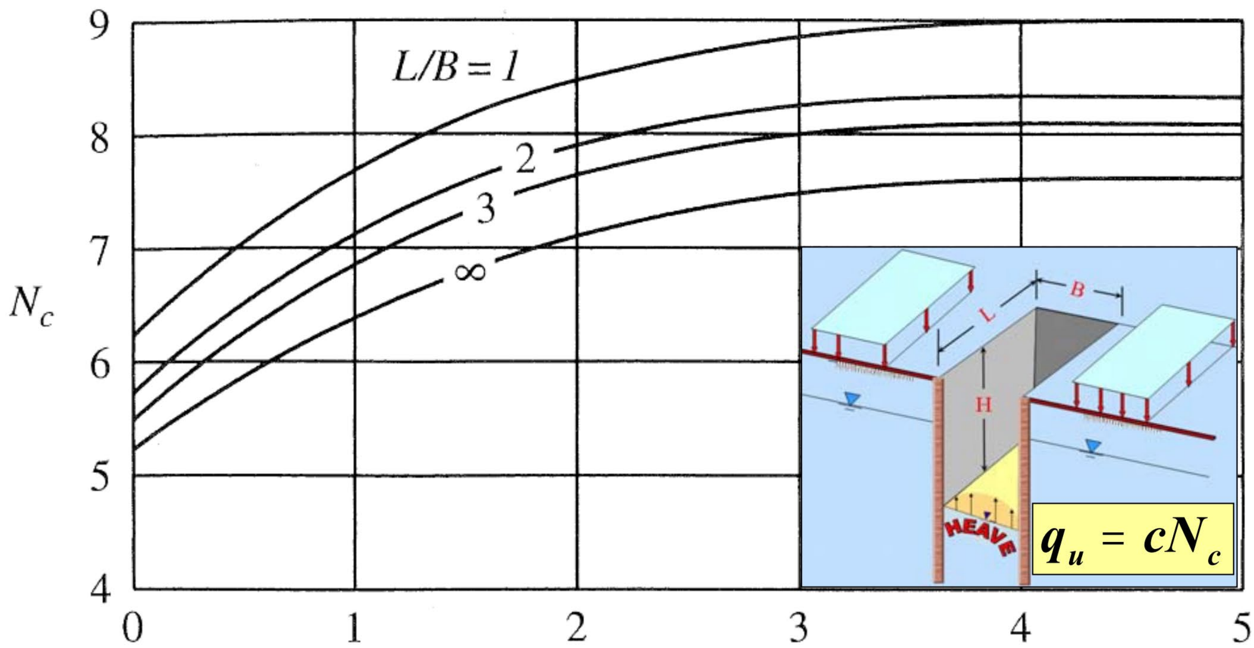


Figure 10-15. Bearing Capacity Factor from Bjerrum and Eide (1956)

The bearing capacity factor, **Nc**, shown in Figure 10-15, varies with the ratios of **H/B** and **L/B**. In general, for **H/B**:

$$\mathbf{N_{c(rec\,tan\,gle)}} = \mathbf{N_{c(square)}}\left(\mathbf{0.84 + 0.16\frac{B}{L}}\right) \text{ [from Bjerrum and Eide (1956)]} \tag{10-3-4}$$

Where:

$\mathbf{N_{c(square)}}$ = Bearing capacity factor based on L/B=1

$\mathbf{B}$ = Width of excavation in feet

$\mathbf{L}$ = Length of excavation in feet

If the analysis indicates that heave is probable, modifications to the shoring system may be needed. The sheeting may be extended below the bottom of the excavation into a more stable layer, or for a distance of one-half the width of the excavation (typically valid only for excavations where **H>B**). When submerged or when installed in clay, another possible solution could be to over-excavate and construct a counterweight to the heaving force. Be aware that strutting a wall near its bottom will not prevent heave. Strutting only resists the lower shoring sides from rotating into the excavation.

10-3.01 Factor of Safety against Heave

The factor of safety (FS) against bottom heave as shown in Figure 10-16 is:

$$\mathbf{FS} = \frac{\mathbf{F_{RS}}}{\mathbf{F_{DR}}} = \frac{\mathbf{Q_U}}{\mathbf{Q}} \geq \mathbf{1.5} \qquad (10\text{-}3\text{-}5)$$

Where:

$\mathbf{F_{RS}}$ = Resisting Force = $\mathbf{Q_U}$ from Equation 10-3-3.

$\mathbf{F_{DR}}$ = Driving Force = **Q** from Equation 10-3-2.

Note that Equation 10-3-2 has a term of negative **S.**
S (resistance of soil due to cohesion over depth of excavation) is equal to **cH**, and is modelled as a negative driving force.

To reduce the risk of heave, it is recommended that a minimum safety factor of 1.5 should be used.

This factor of safety is based on the assumption that the clay layer is homogeneous, and at least to a depth of **0.7B** below the bottom of the excavation. However, if a hard layer of rock or rocklike material is within **0.7B** of the bottom of the excavation, then the failure surface will need to be modified to some extent. The depth to the hard layer of rock or rocklike material is **D**. **D** is measured from the excavation line to the top of the rock or rocklike material, and that depth is less than or equal to **0.7B**. The following steps would be made for this condition:

1. In Figure 10-14, the vertical distance of **0.7B** would be changed to **D**.
2. In Figure 10-14, the horizontal distance of **0.7B** would be changed to **D**.
3. In Equation 10-3-2, W would be changed from γ**H** (**0.7B**) to γ**HD**.
4. In Equation 10-3-2, the width for the surcharge would be changed from **0.7B** to **D**. The term of (**0.7B**)**q** would be changed to (**D**)**q**.
5. In Equation 10-3-2, the value of **S** (resistance of soil due to cohesion) would not change.
6. In Equation 10-3-3, the width for the bearing capacity would be changed from **0.7B** to **D**. The value of $\mathbf{Q_u}$ would be changed from $\mathbf{cN_c}$(**0.7B**) to $\mathbf{cN_c(D)}$.

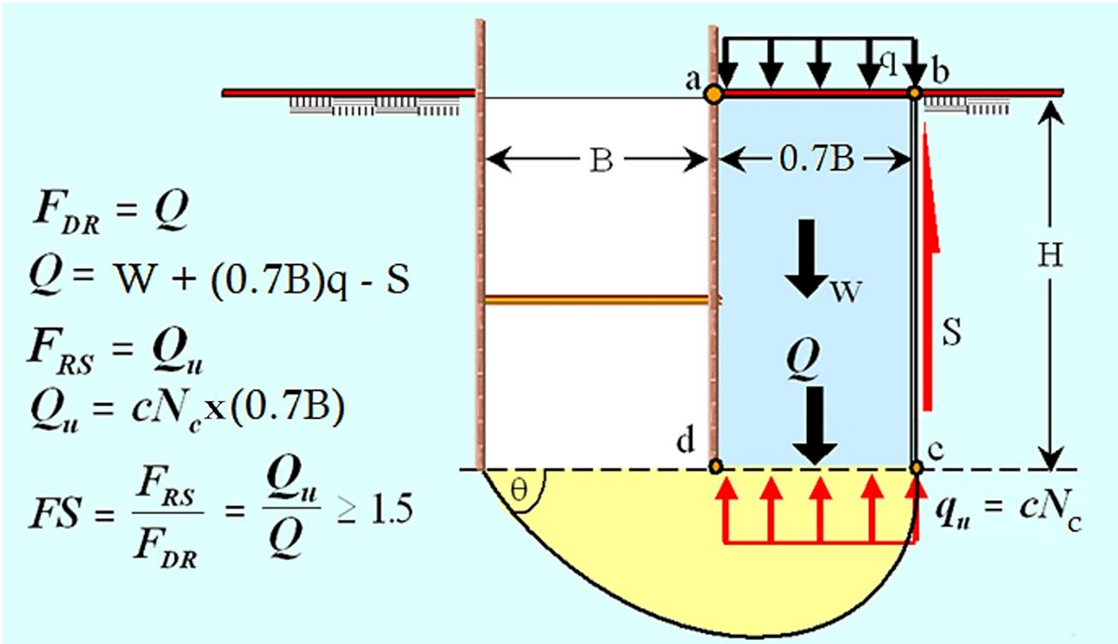


Figure 10-16. Driving and Resisting Forces and the Factor of Safety

10-3.02 Example 10-2 Problem – Heave Factor of Safety

Given: **H** = 30', **B** = 15', **L** = 45'

q = 300 psf, **c** = 500 psf, γ = 120 pcf

Solution:

$\mathbf{N_{c(square)}}$ as determined from Figure 10-15 for H/B= 2 is 8.5. This is the value for L/B=1. However, the L/B value for this example is = 3.

$\mathbf{N_c}$ from Figure 10-15 (for H/B = 2 and L/B=3) is = 7.6.

Bearing capacity = **c x $\mathbf{N_c}$ x (0.7B)** = (500 psf) x (7.6) x (0.7 x 15 feet) ≈ 40 kip /ft

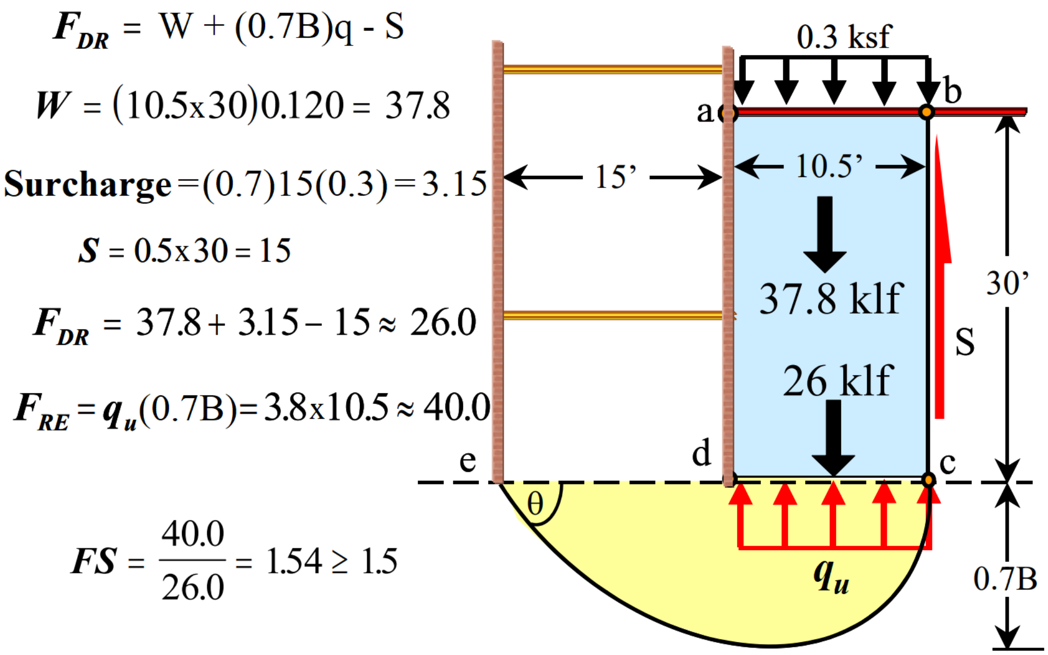


Figure 10-17. Heave Example Problem Calculations

(Note: figure only shows one side for simplicity)

10-4 Piping

For excavations in pervious materials (sands), the condition of piping can occur when an unbalanced hydrostatic head exists. This pressure difference causes an upward flow of water through the soil and into the bottom of the excavation. This is also known as quick conditions, and may be visible as a sand boil. If the piping is allowed to continue, this movement of water into the excavation will transport material and will cause settlement of the soil adjacent to the excavation. The passive resistance of embedded members will be reduced as a result.

One solution to this problem is to equalize the unbalanced hydraulic head by either allowing the excavation to fill with water or lower the water table outside the excavation by dewatering. If dewatering is used, the flow rate into the excavation will decrease, the shear strength of the soil will increase, and the soil will stop acting as a liquid. On Caltrans projects, one of the common methods used to protect or mitigate against piping is the use of a seal course. Refer to the *Foundation Manual*, Chapter 12, *Cofferdams and Seal Courses*, for additional information regarding seal course construction.

If the embedded length of the shoring system member is long enough, the condition of piping should not develop. The *USS Steel Sheet Piling Design Manual* contains charts on page 65 giving lengths of sheet pile embedment, which will result in an adequate factor of safety against piping. These charts are of particular interest and a good resource for cofferdams.

10-4.01 Hydraulic Forces on Cofferdams and Other Structures

Moving water imposes not only normal forces acting on the normal projection of the cofferdam, but also substantial forces in the form of eddies that can act along the sides of sheet piles as shown in Figure 10-18. The drag force, **D** (in pounds), is calculated with Equation 10-4-1 [from Ratay (1984)]:

$$\mathbf{D} = (\mathbf{A})(\mathbf{C_d})(\boldsymbol{\gamma}_\mathbf{w})\frac{\mathbf{V}^2}{\mathbf{2g}} \qquad (10\text{-}4\text{-}1)$$

Where:

- **A** = Projected area of the obstruction normal to the current in ft^2
- $\mathbf{C_d}$ = Coefficient of drag
- $\boldsymbol{\gamma}_\mathbf{w}$ = Water unit weight in lbs/ft^3
- **V** = Velocity of the current in ft/sec
- **g** = Acceleration due to gravity in ft/sec^2

In English units, the numerical value of γ_ω is approximately equal to **2g**, without regard to units. (Recall that the weight of water is 62.4 lbs/ft^3, and the acceleration of gravity is 32.2 ft/s^2). Thus, Equation 10-4-1 can be simplified as follows:

$$\mathbf{D} = (\mathbf{A})(\mathbf{C_d})(\mathbf{V}^2) \qquad (10\text{-}4\text{-}2)$$

Where:

- **A** = as defined above
- **V** = as defined above
- $\mathbf{C_d}$ = Coefficient of drag, lbs sec^2/ft^4 (Note: $\mathbf{C_d}$ is not dimensionless in the above equation for **D** to be in lbs.)
- **D** = Drag force in lbs

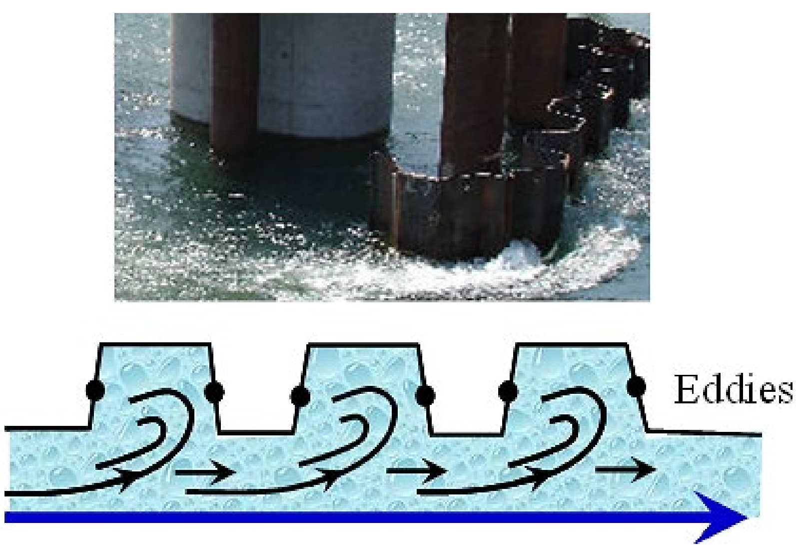


Figure 10-18. Hydraulic Forces on Cofferdams

Considering the roughness along the sides of the obstructions (as for a sheet pile cofferdam) the practical value for C_d = 2.0.

$$\mathbf{D} = \mathbf{2AV^2} \qquad (10\text{-}4\text{-}3)$$

The drag load, **D**, is applied in the same manner as a wind rectangular load on the loaded height of the obstruction (falsework or guyed elements).

Example: Determine the drag force on a 6-foot wide sheet pile cofferdam placed vertically in water with average depth of 6 feet flowing at 4 feet per second. For this example: C_d = 2.0.

Projected Area $= 6(6) = 36\ \text{ft}^2$.

$\mathbf{D} = 2(36)(4)^2 = 1{,}152\ \text{lbs.}$

The drag load, **D,** may then be added as an additional live load force distributed over the projected area of the cofferdam. If applied as a point load it would be placed at the centroid of the projected area, i.e., at the center of the 6-ft by 6-ft area for the example above. It is more appropriate to apply as a per square foot-loading, as illustrated in the calculation below. The applied load per square-foot generally would not govern for the stress in the sheet piles.

Drag Load = 1152 lbs/36 ft^2 = 32 psf

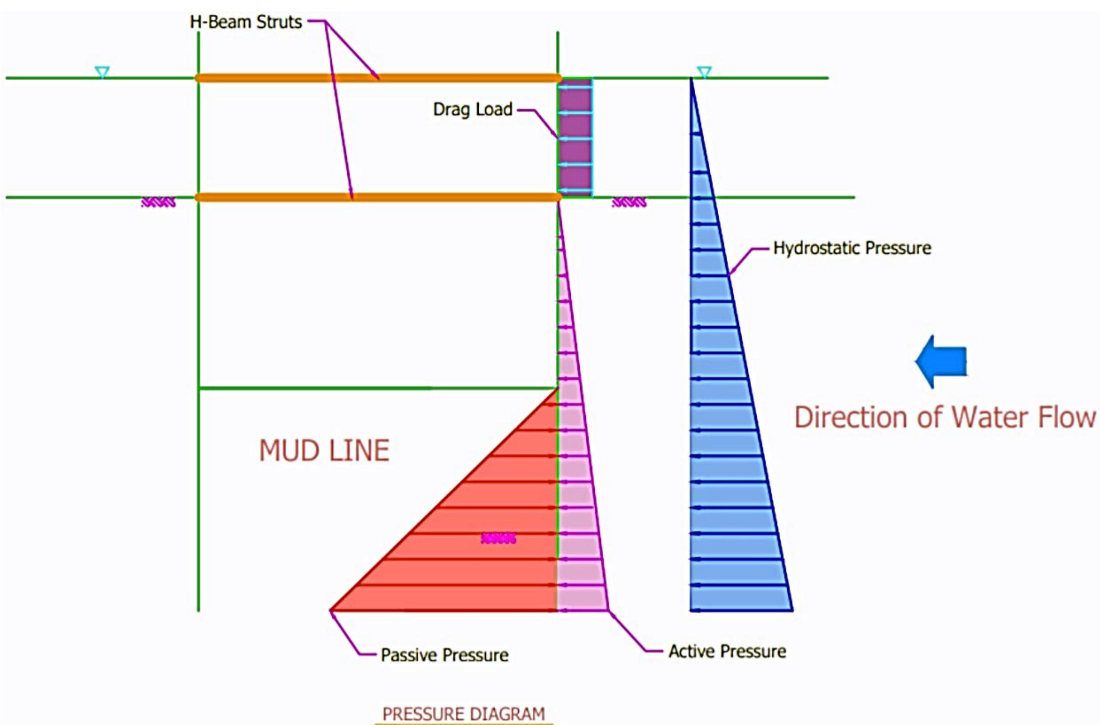


Figure 10-19. Drag and Hydraulic Forces on a Cofferdam

For the example above, illustrated in Figure 10-19, the Engineer may determine that the additional live load is negligible compared to the overall loads to the cofferdam system. It will likely be the primary contributor to the loading for the upper strut both in the final condition and in the sequence of construction. However, for a system nearing allowable capacities, the additional drag force produced by the water flow could require consideration.

The characteristic of a cofferdam differs from a typical sheet pile system only in that it is intended to address both hydrostatic and flowing water. This added variable requires additional attention to the installation sequence proposed by the Contractor, as each step will require analysis of the system. Always obtain and review the Contractor's installation sequence. While the procedures vary by contractor, a typical installation may include the following:

1. Construct and install outside waler-ring template and anchor per the Contractor's submittal.
2. Install sheet piling to specified elevation.
3. Dewater cofferdam to elevation described in the Contractor's submittal to facilitate the installation of the first level of interior bracing. Verify calculations for cofferdam system in this configuration.
4. Install interior waler and struts and any required diagonal cross bracing struts. Verify cofferdam load capacities with bracing system installed.
5. Place concrete filler required at diagonal cross bracing struts.
6. Confirm Contractor's seal course thickness is adequate to resist hydrostatic pressure and verify use of proper cofferdam vent during seal course placement as required by the *Contract Specifications*, Section 19-3.03 B(4), *Earthwork – Structure Excavation and Backfill – Construction – Cofferdams.*
7. Excavate to specified elevation (usually bottom of seal course).
8. Place seal course concrete underwater and cure.
9. Dewater cofferdam. Verify cofferdam calculations at this configuration, fully dewatered, and braced with seal course placed.
10. Clean and prepare top of seal course for footing subgrade.

It should be noted that the Contractor's cofferdam system may require multiple levels of interior waler and struts, and the procedure of excavation and dewatering noted in the submittal should be followed closely.

10-5 Slope Stability

When the ground surface is not horizontal at the construction site, a component of gravity may cause the soil to move in the direction of the slope. Slopes fail in different ways. Figure 10-20 shows some of the most common patterns of slope failure in soil. The slope failure of rocks is beyond the scope of this manual.

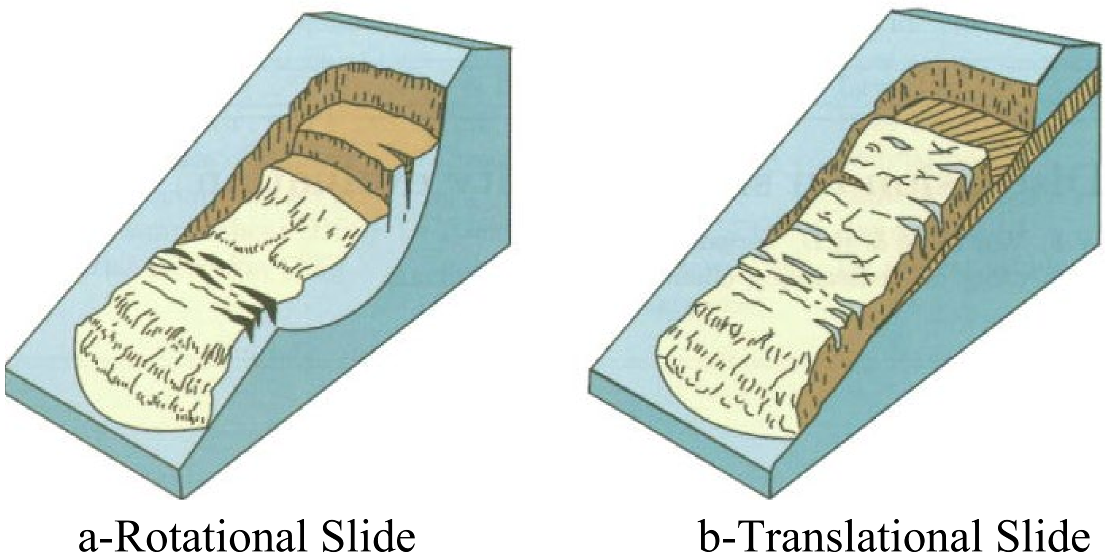


Figure 10-20. Common Pattern of Soil Slope Failure (USGS)

A stability analysis prepared by a Geotechnical engineer or Geologist should be requested from the Contractor when it appears that shoring or a cut slope presents a possibility of some form of slip failure. The discussion and examples that follow are intended to give the Engineer the ability to anticipate conditions needing further analysis. Geotechnical Services in Sacramento has the capability of performing computer-aided stability analysis to verify a contractor's submitted analysis, and it is recommended the Engineer utilize this resource when they are tasked with performing a slope stability review.

The discussion and examples that follow for Fellenius and Bishop methods are provided to simply show the concept of the failure mechanism and analysis thereof. It is very important to be in agreement on the soil parameters with the Contractor, as these will have a significant effect on the factor of safety calculated, especially cohesion.

The fundamental assumption of the limit-equilibrium method is that failure occurs when a mass of a soil slides along a slip surface as shown in Figure 10-20. The popularity of limit-equilibrium methods is primarily due to their relative simplicity, and the many years of experience analyzing slope failures.

Construction surcharges such as equipment and stockpiled materials, may cause excavation instabilities and should be considered when performing a slope stability analysis. The slope stability analysis involves the following:

1. Obtain surface geometry, stratigraphy, and subsurface information.
2. Determine soil properties.
3. Determine soil-structure interaction, such as the presence of sheet piles, soldier piles, ground anchors, soil nails, and so forth.
4. Determine surcharge loads.
5. Perform slope stability analysis to calculate the minimum factor of safety against failure for various stage constructions.

The stability of an excavated slope is expressed in terms of the lowest factor of safety found*,* utilizing multiple potential failure surfaces. Circular solutions to slope stability have been developed primarily due to the ease of this geometry during the computational procedure. The most critical failure surface will be dependent on site geology and other factors mentioned above. However, the most critical failure surface is not necessarily circular, as shown in Figure 10-20 (Rotational Slide) and Figure 10-21. Non-circular failure surfaces can be caused by adversely dipping bedding planes, zones of weak soil, or unfavorable ground water conditions.

10-5.01 Rotational Slides

Stability analysis of slopes with circular failure surfaces can be explained using a method of slices. Figure 10-21a shows an arc or a circle, AB, representing a trial failure surface. The soil above the trial surface is divided into a number of slices and given an incremental designation. The forces acting on a typical slice “**i**” are shown as b, c and d of Figure 10-21. The ordinary method of slices (Figure 10-21b), which is the simplest method, does not consider interslice forces acting on the side of the slices. The Simplified Bishop’s Method of Slices (Figure 10-21c) accounts only for the horizontal interslice forces while more refined methods, such as Spencer’s solution (Figure 10-21d), account for both vertical and horizontal interslice forces acting on each side of the slice.

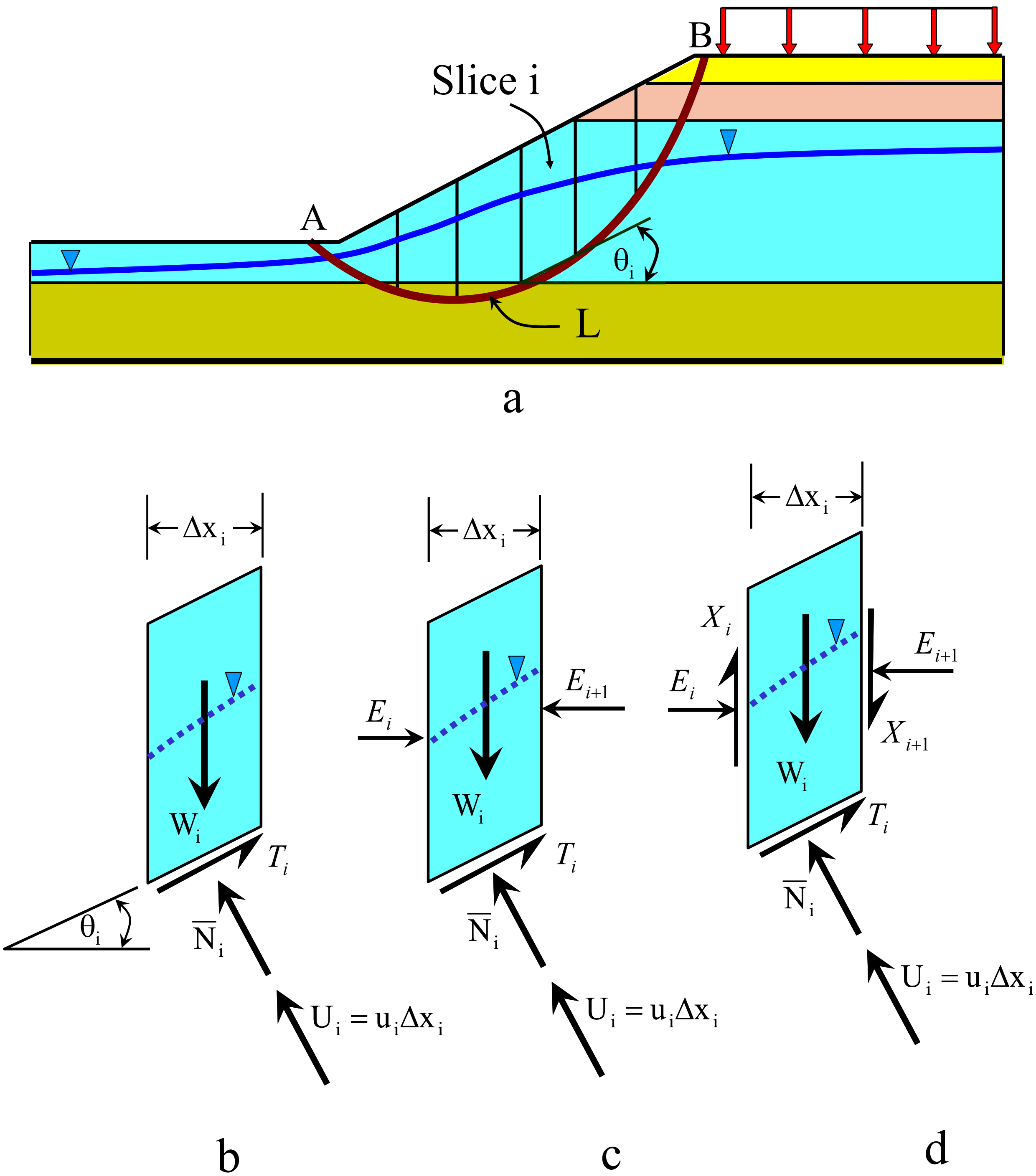


Figure 10-21. Method of Slices and Forces Acting on a Slice

Variations of this method used for investigating the factor of safety for potential stability failure include:

- Fellenius Method of Slices (Figure 10-21b).
- Simplified Bishop Method of Slices (Figure 10-21c).
- Spencer and Janbu Method of Slices (Figure 10-21d).

Also known as Ordinary Method of Slices or Swedish Circle, the Fellenius Method was published in 1936. The Simplified Bishop Method (1955) also uses the method of slices to find the factor of safety for the soil mass. The failure is assumed to occur by rotation of a mass of soil on a circular slip surface centered on a common point as shown in Figure 10-22.

The basic equation for each of these methods is:

$$\mathbf{FS} = \frac{\overline{\mathbf{C}}\mathbf{L} + \tan\overline{\boldsymbol{\phi}}\sum_{i=1}^{i=n}\overline{\mathbf{N}}_i}{\sum_{i=1}^{i=n}\mathbf{W}_i\sin\boldsymbol{\theta}_i} \tag{10-5-1}$$

Nomenclature

FS = Factor of safety

$\mathbf{FS_a}$ = Assumed factor of safety

i = Represents the current slice

$\bar{\phi}$ = Friction angle based on effective stresses

$\bar{C}$ = Cohesion intercept based on effective stresses

$\mathbf{W_i}$ = Weight of the slice

$\overline{\mathbf{N}}_i$ = Effective normal force

$\boldsymbol{\theta}_i$ = Angle from the horizontal of a tangent at the center of the slice along the slip surface.

$\mathbf{T_i}$ = Shear force at base of slice.

$\mathbf{u_i}$ = Pore-water pressure force on a slice

$\mathbf{U_i}$ = Resultant neutral (pore-water pressure) force

$\Delta \mathbf{l_i}$ = Length of the failure arc cut by the slice. Note that as the slices get smaller, the values of $\Delta \mathbf{x_i}$ and $\Delta \mathbf{l_i}$ converge.

$\Delta \mathbf{x_i}$ = the width of a slice

L = Length of the entire failure arc

Note: The angle, **θ**, (measured from the horizontal) is shown in Figure 10-21 and is equal to the angle that is measured from the vertical in Figure 10-22. In Figure 10-21, the shear force at the base of the slice (**T**i) acts at this angle, and this angle is measured from the horizontal.

For major excavations in side slopes, slope stability failure for the entire system should be investigated.

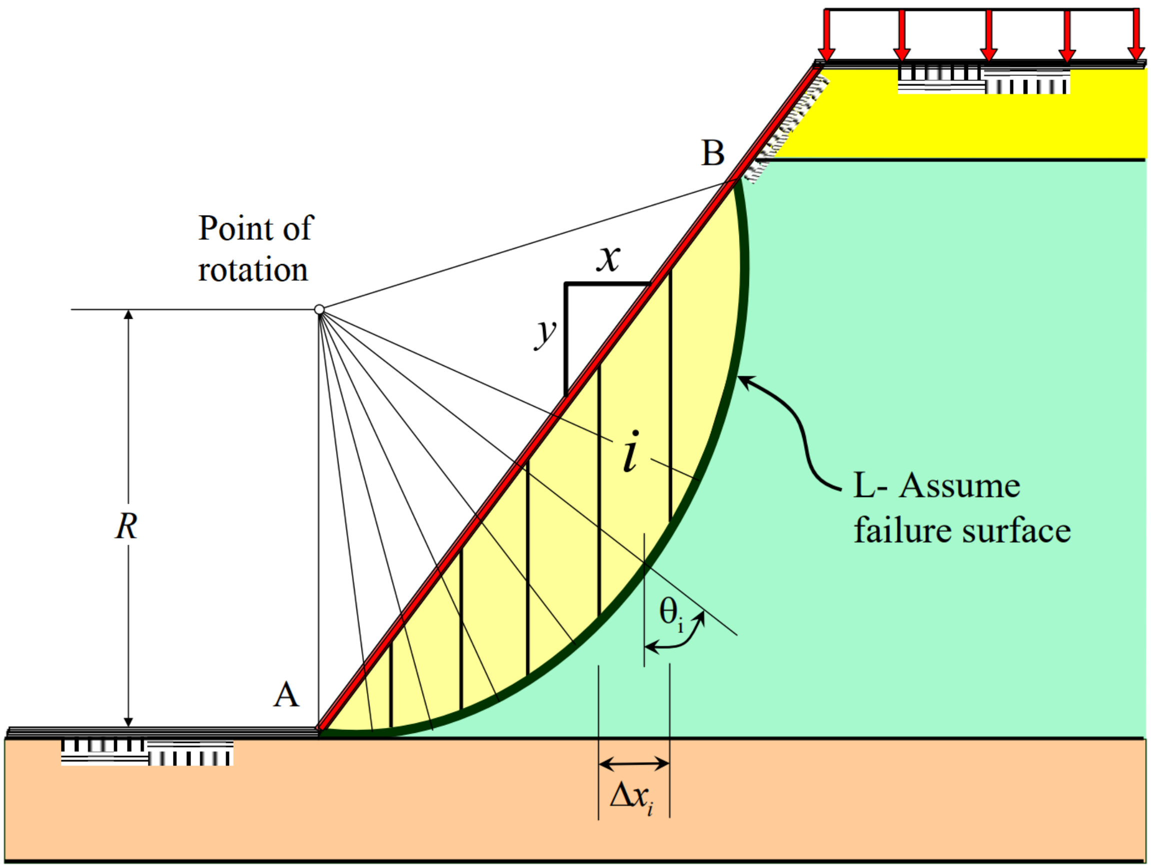


Figure 10-22. A Trial Surface and Potential Slices for Fellenius, Simplified Bishop, and Spencer and Janbu, Methods of Slices

10-5.02 Fellenius Method

This method assumes that for any slice, the forces acting upon its sides have a resultant of zero in the direction normal to the failure arc. This method is conservative but is widely used in practice because of its early origins and simplicity.

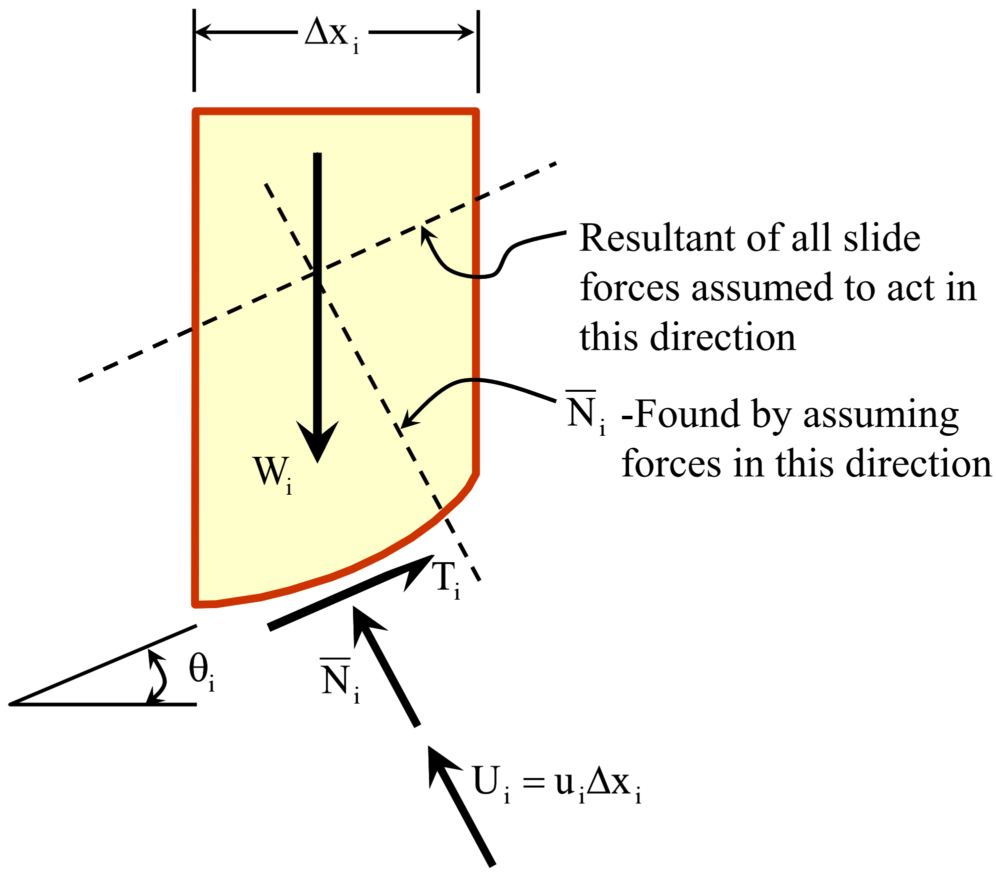


Figure 10-23. Slice i, Fellenius Method

$$\bar{\mathbf{N}}_{\mathbf{i}} = \mathbf{W_i} \cos \theta_{\mathbf{i}} - \mathbf{u_i}\, \Delta \mathbf{l_i} \tag{10-5-2}$$

Recall that for small slices,

$$\Delta \mathbf{x_i} \approx \Delta \mathbf{l_i} \tag{10-5-3}$$

Thus, the basic equation becomes:

$$\mathbf{FS} = \frac{\bar{\mathbf{C}}\mathbf{L} + \tan \bar{\boldsymbol{\phi}} \sum_{i=1}^{i=n} (\mathbf{W_i} \cos \boldsymbol{\theta}_{\mathbf{i}} - \mathbf{u_i} \Delta \mathbf{x_i})}{\sum_{i=1}^{i=n} \mathbf{W_i} \sin \boldsymbol{\theta}_{\mathbf{i}}} \tag{10-5-4}$$

The procedure is to investigate many possible failure planes, with different centers and radii, to identify the possible failure arc with the lowest factor of safety. Since this may take hundreds of iterations and is thus ideally solved with specialized software, Structure Construction staff should contact the DES Geotechnical unit for assistance.

10-5.02A Example 10-3 Problem – Fellenius Method

Given: γ = 115 pcf $\bar{\phi}$ = 30° $\bar{C}$ = 200 psf No Groundwater

Solution:

The trial failure mass is divided into 6 slices with equal width as shown in Figure 10-24. Each slice makes an angle θ with respect to horizontal as shown; note relationship between angles from vertical and horizonal.

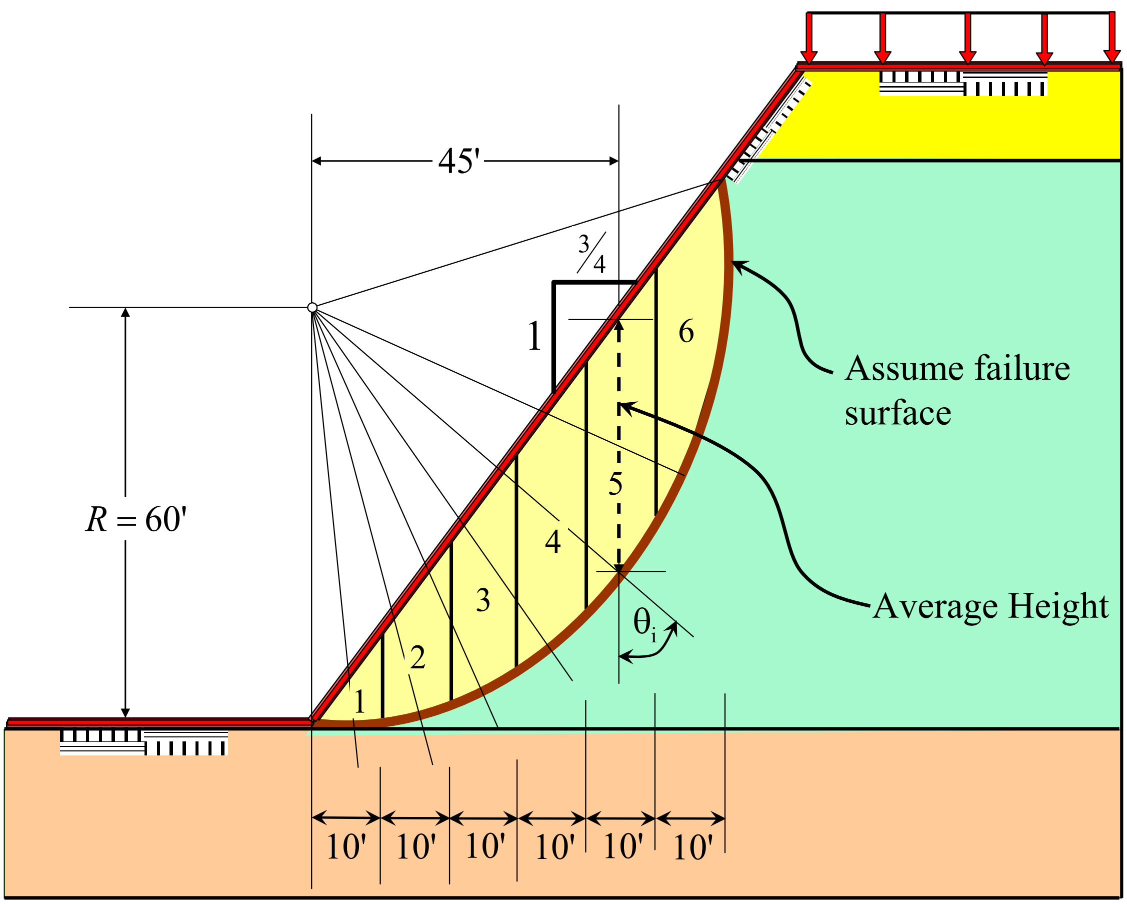


Figure 10-24. Example of Fellenius and Bishop Method of Slices

Table 10-1. Fellenius Table of Slices, Part 1

Angles θ_i (°)	Average Height (ft)	Slice Weights (kips/ft)
$\theta_1 = \sin^{-1}(5/60) = 4.78°$	6.46	$W_1 = (6.46)(10)(0.115) = 7.43$
$\theta_2 = \sin^{-1}(15/60) = 14.48°$	18.09	$W_2 = (18.09)(10)(0.115) = 20.81$
$\theta_3 = \sin^{-1}(25/60) = 24.62°$	27.88	$W_3 = (27.88)(10)(0.115) = 32.06$
$\theta_4 = \sin^{-1}(35/60) = 35.69°$	35.40	$W_4 = (35.40)(10)(0.115) = 40.71$
$\theta_5 = \sin^{-1}(45/60) = 48.59°$	39.69	$W_5 = (39.69)(10)(0.115) = 45.64$
$\theta_6 = \sin^{-1}(55/60) = 66.44°$	37.31	$W_6 = (37.31)(10)(0.115) = 42.91$

Table 10-2. Fellenius Table of Slices, Part 2

Slice	θ_i (°)	W_i (kips/ft)	$W_i \sin\theta_i$ (kips/ft)	$W_i \cos\theta_i$ (kips/ft)	$\bar{N}_i$ (kips/ft)
1	4.78	7.43	0.62	7.40	7.40
2	14.48	20.81	5.20	20.15	20.15
3	24.62	32.06	13.36	29.14	29.14
4	35.69	40.71	23.75	33.07	33.07
5	48.59	45.64	34.23	30.19	30.19
6	66.44	42.91	39.33	17.15	17.15
			Σ = 116.49		Σ = 137.09

L = 113.55 ft (by geometry)

$$\text{FS} = \frac{(0.2)\,(113.55) + (0.577)\,(137.09)}{116.49} = 0.87 < 1 \qquad (10\text{-}5\text{-}5)$$

This is the value for one trial failure plane. Additional trials are necessary to determine the critical one that gives the minimum factor of safety. The slope for this sample problem is deemed to be unstable since the computed safety factor determined by this single calculation is less than one.

10-5.03 Bishop Method

This method assumes that the forces acting on the sides of any slice have a zero resultant in the vertical direction.

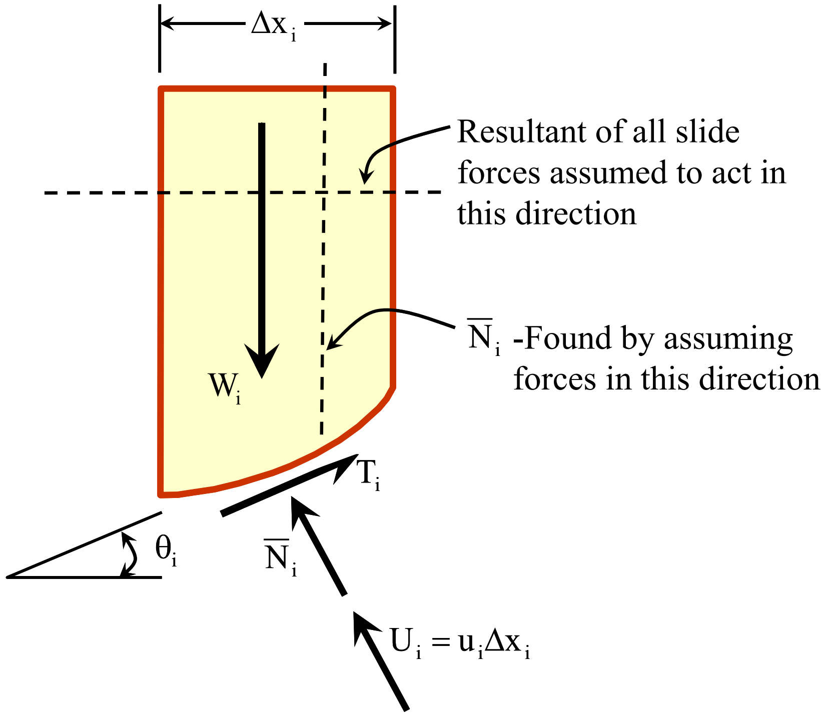


Figure 10-25. Slice i, Bishop Method

$$\overline{\mathbf{N}}_i = \frac{W_i - u_i \Delta x_i - \dfrac{\bar{C}\Delta x_i \tan\theta_i}{FS_a}}{\cos\theta_i \left\{1 + \dfrac{\tan\theta_i \tan\bar{\phi}}{FS_a}\right\}} \qquad (10\text{-}5\text{-}6)$$

The basic equation becomes:

$$FS = \frac{\sum_{i=n}^{i=n}\left(\dfrac{\bar{C}\Delta x_i + (W_i - u_i \Delta x_i)\tan\bar{\phi}}{M_i}\right)}{\sum_{i=n}^{i=n} W_i \sin\theta_i} \qquad (10\text{-}5\text{-}7)$$

Where:

$$M_i = \cos\theta_i \left\{1 + \frac{\tan\theta_i \tan\bar{\phi}}{FS_a}\right\} \qquad (10\text{-}5\text{-}8)$$

For the Bishop Method, the factor of safety must be assumed (FS_a) and trial-and-error iterations are required to determine the solution. The assumed FS_a converge on the factor of safety for that trial failure plane. Close agreement between the assumed FS_a and the calculated FS indicate that the selection of the center and radius is near the target value.

10-5.03A Example 10-4 Problem – Bishop Method

Given: γ = 115 pcf ϕ = 30° θ = 200 psf No Groundwater

Solution:

Table 10-3. Bishop Table of Slices, Part 1

Column	A	B	C	D	E	F	G
Slice	θ_I (deg)	W_i (k/ft)	$\bar{C}\,\Delta x_i$ (k/ft)	$W_i \tan\bar{\phi}$ (k/ft)	$\cos\theta_i$	$\tan\theta_i \tan\bar{\phi}$	C + D (kips/ft)
1	4.78	7.43	2	4.29	1.00	0.05	6.29
2	14.48	20.81	2	12.01	0.97	0.15	14.01
3	24.62	32.06	2	18.51	0.91	0.26	20.51
4	35.69	40.71	2	23.50	0.81	0.41	25.50
5	48.59	45.64	2	26.35	0.66	0.65	28.35
6	66.44	42.91	2	24.77	0.40	1.32	26.77

Table 10-4. Bishop Table of Slices, Part 2

Column	Ha	Hb	Ia	Ib	J
Slice	M_i FS_a = 1.5	M_i FS_a = 0.8	G/Ha (kips/ft) FS_a = 1.5	G/Hb (kips/ft) FS_a = 0.8	$W_i \sin\theta_I$ (kips/ft)
1	1.03	1.06	6.11	5.93	0.62
2	1.06	1.15	13.21	12.18	5.20
3	1.07	1.21	19.17	16.95	13.36
4	1.04	1.23	24.52	20.72	23.75
5	0.95	1.20	29.84	23.63	34.23
6	0.75	1.06	35.69	25.25	39.33
			Σ = 128.54	Σ = 104.66	Σ = 116.49

For FS_a = 1.5:

$$FS = \frac{128.54}{116.49} = 1.10 \qquad (10\text{-}5\text{-}9)$$

For FS_a = 0.8:

$$FS = \frac{104.66}{116.49} = 0.90 \qquad (10\text{-}5\text{-}10)$$

The factor of safety for this trial converges to ≈ 0.9. Again, this is the value for one trial failure plane. Additional trials are necessary to determine the critical one that gives the minimum factor of safety.

If ground water were present, pore pressure would need to be considered. The values are most typically calculated based on field measured water levels.

10-5.04 Translational Slide

For excavations with soil layers dipping toward the excavation, or when there is a definite plane of weakness near the base of the slope, the slope may fail along a plane parallel to the weak strata as shown in Figure 10-26. This surface would be assumed along the interface of the upper sliding soil and the weaker soil below it.

The movement of the soil mass within the failure surface is translational rather than rotational. Methods of analysis that consider blocks or wedges sliding along plane surfaces must be used to analyze slopes with a specific plane of weakness. Note that for Figure 10-26, the soil layers depicted are at an incline that would also need to be accounted for. A down sloping angle in the direction of the larger active mass would contribute to the forces that the passive block would need to resist.

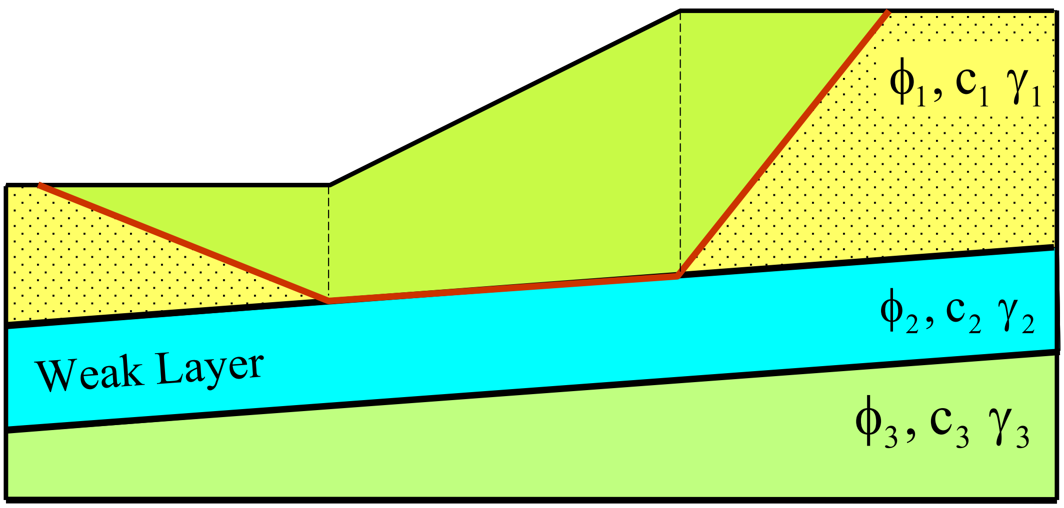


Figure 10-26. Mechanism of Translational Slide

Figure 10-27 also depicts a stratified soil consisting of three layers and a potential sliding mass. The force equilibrium of the blocks or wedges is more sensitive to shear forces than moment equilibrium as shown in Figure 10-27. The potential failure mass consists of an upper or active Block A, a central or neutral Block B, and a lower or passive Block P. The active earth pressure from Block A tends to initiate translational movement. This movement is opposed by the passive resistance to sliding of Block P and by shearing resistance along the base of central Block B. The critical failure surface can be located using an iterative process as explained previously.

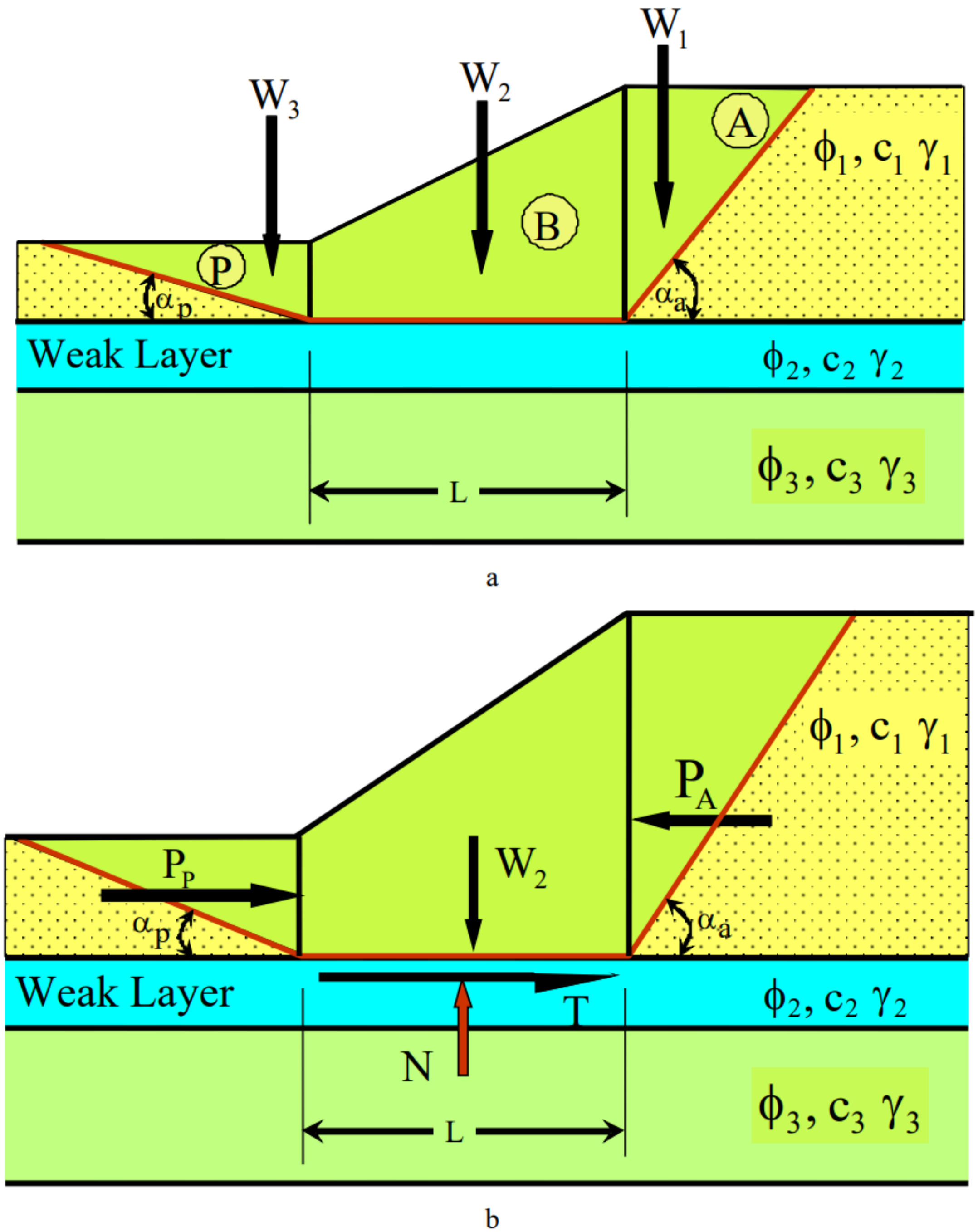


Figure 10-27. Mechanism of Translational Slide

The factor of safety of the slope against translational sliding is established by the ratio of resisting to driving forces. The resisting force is a function of passive pressure at the toe of the slope and the shearing resistance along the base of Block B. The driving force is the active earth pressure due to thrust of Block A. Thus the factor of safety can be expressed as follows:

$$\mathbf{FS} = \frac{\mathbf{T} + \mathbf{P_p}}{\mathbf{P_a}} \tag{10-5-11}$$

In which:

$$\mathbf{T} = \mathbf{c_2} \times \mathbf{L} + \mathbf{W_2} \times \tan \boldsymbol{\phi}_2 \tag{10-5-12}$$

Where:

T	=	tangential resistance force at the base of Block B
$\mathbf{c_2}$	=	unit cohesion along base of the Block B
L	=	length of base of Block B
$\mathbf{P_a}$	=	resultant active pressure on Block B $= W_1 \tan(\alpha_a - \phi_1)$
$\mathbf{P_p}$	=	passive pressure on Block B $= W_3 \tan(\alpha_P + \phi_1)$
$\mathbf{W_1}$	=	weight of section of Block A
$\mathbf{W_2}$	=	weight of section of Block B
$\mathbf{W_3}$	=	weight of section of Block P
α_a	=	failure plane angle with horizontal for active pressure
α_p	=	failure plane angle with horizontal for passive pressure
ϕ_1	=	internal friction angle of soil for Block A
ϕ_2	=	internal friction angle of weaker underlying soil
FS	=	factor of safety

Additional notes and observations for the translational slide:

1. Block B is the middle block, and it is the key element. A free body diagram can be drawn for Block B.
2. The bottom of Block B is horizontal in Figure 10-27. If the bottom of block B has a slope downward to the left, then the factor of safety would be reduced.
3. There is only one driving force that is acting on Block B, and that driving force is the resultant Rankine active earth pressure force on the right side of Block B.
4. At the bottom of Block B, there is a resisting force from the cohesion of the weak layer (due to the cohesion value for the weak layer), and there is a resisting force from the friction of the weak layer (due to the phi (ϕ) angle for the weak layer).
5. On the left side of Block B, there is a resisting force from the resultant Rankine passive earth pressure.
6. The factor of safety is equal to the sum of the resisting forces divided by the sum of the driving forces.

10-5.04A Example 10-5 Problem – Translational Slide

Calculate the factor of safety for a translational slide for a given failure surface, as shown below in Figure 10-28.

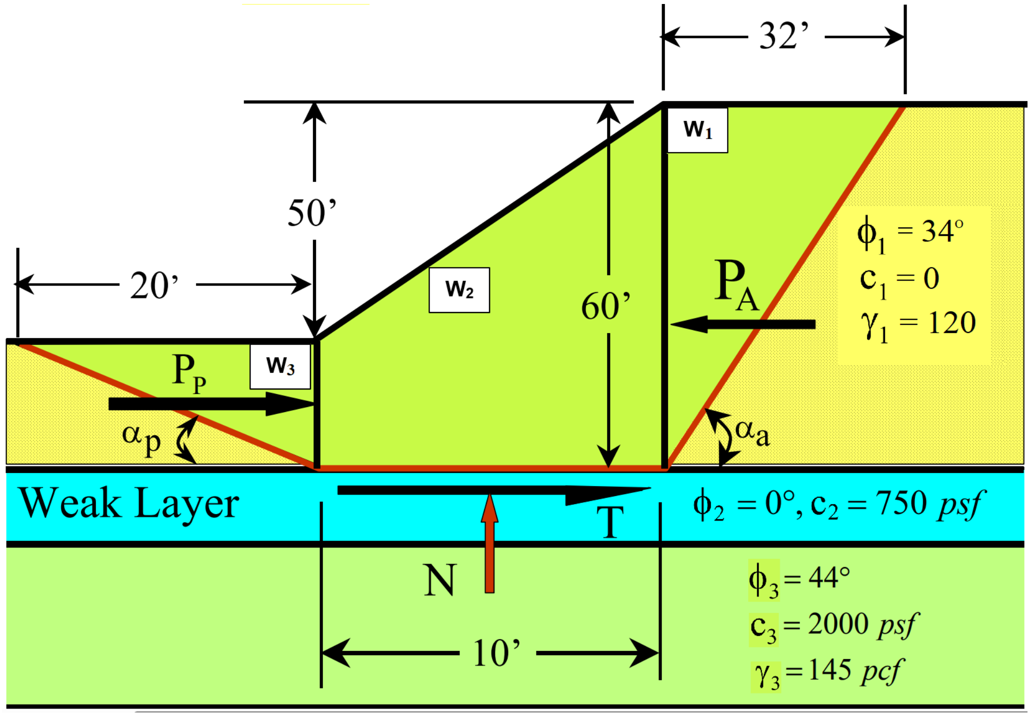


Figure 10-28. Example of a Translational Slide

Solution:

By geometry: $\alpha_a = 62°$ $\alpha_p = 26.6°$

Note that these values for α_a and α_p are close to the values one would obtain when following the more rigorous method outlined in Section 4-3.02, *Active and/or Passive Earth Pressure*, of this manual.

Calculating the weight of the soil blocks:

$$W_1 = \frac{(32')(60')}{2}\left(\frac{120}{1000}\right) = 115.2\ \text{kip/ft} \tag{10-5-13}$$

$$W_2 = \frac{(10' + 60')(10')}{2}\left(\frac{120}{1000}\right) = 42.0\ \text{kip/ft} \tag{10-5-14}$$

$$W_3 = \frac{(20')(10')}{2}\left(\frac{120}{1000}\right) = 12.0\ \text{kip/ft} \tag{10-5-15}$$

The tangential resistance force at the bottom of the block with weight **W2**, due to the cohesion of the weak layer is calculated as:

$$T = W_2 \mathrm{Tan}(\phi_2) + c_2 L = (42)\big(\mathrm{Tan}(0°)\big) + \frac{(750\ \mathrm{psf})\big(10'\big)}{1000\ \mathrm{lb/k}} = 7.5\ \mathrm{kip/ft} \tag{10-5-16}$$

Calculate the resultant active and passive forces acting on the soil block with weight **W2**:

$$P_a = (115.2)(\mathrm{Tan}(62° - 34°)) \approx 61.3\ \mathrm{kip/ft} \tag{10-5-17}$$

$$P_p = (12.0)(\mathrm{Tan}(26.6° + 34°)) \approx 21.3\ \mathrm{kip/ft} \tag{10-5-18}$$

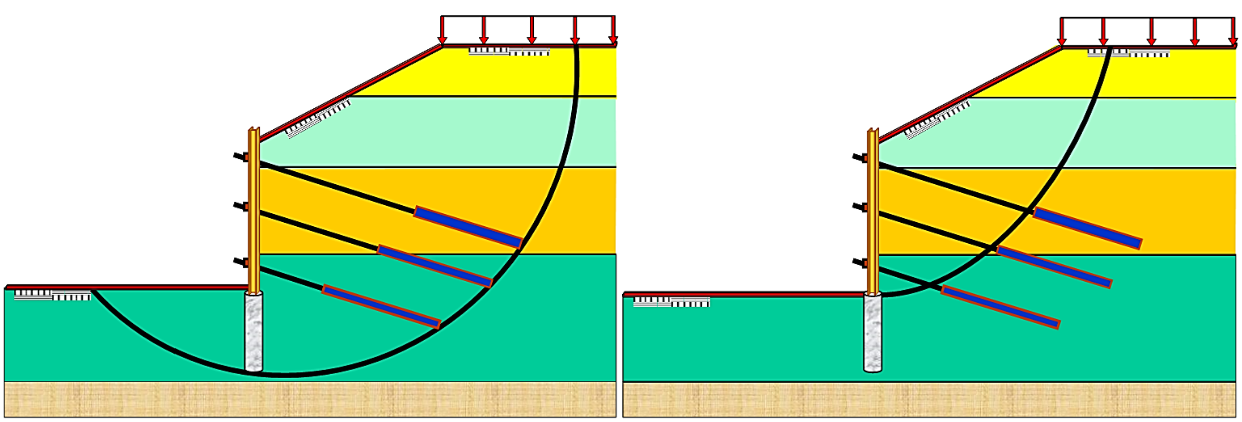
$$FS = \frac{7.5 + 21.3}{61.3} = 0.47 \tag{10-5-19}$$

Note that if the length of **W2** was increased to 100 feet, the FS ≈ 1.57.

Depending on the reliability of the soil properties, the duration, the level of risk desired, as well as other considerations, a factor of safety of 1.5 may be appropriate. A discussion with DES Geotechnical is always appropriate.

10-5.05 Stability Analysis of Shoring Systems

Deep-seated stability failure should be investigated for major shoring systems such as ground anchor walls. The slip surface passes behind the anchors and underneath the base tip of the vertical structural members as shown in Figure 10-29. A minimum factor of safety of 1.25 is required for the deep-seated stability failure. Local system failure should also be investigated for major ground anchor systems as shown in Figure 10-29**.** The trial surface must extend to the depth of the excavation to calculate the minimum factor of safety of 1.25. The un-bonded length must extend beyond the failure surface.



a. Deep Seated b. Local System

Figure 10-29. Stability Failure Modes

JULY 2025

CHAPTER 11

CONSTRUCTION CONSIDERATIONS AND FINAL SUMMARY

Chapter 11: Construction Considerations and Final Summary

Table of Contents

Chapter 11: Construction Considerations and Final Summary 1
11-1 Construction 2
11-2 Encroachment Permit Projects 4
11-3 Ground Anchor Restrained Shoring Systems 7
11-3.01 Engineering Analysis and Construction Sequence 7
11-3.02 Components of Ground Anchor Systems 7
11-3.03 Ground Anchor 9
11-3.04 Forces on the Vertical Members 12
11-3.05 Testing Temporary Ground Anchors 12
11-3.06 Wall Movement and Settlement 17
11-3.07 Lock-off Force 17
11-3.08 Corrosion Protection 18
11-3.09 General Steps for Checking Ground Anchor Shoring Submittal 19
11-3.10 Estimating Long Term Creep 19
11-4 Summary 21

11-1 Construction

The integrity of a shoring system, like any other structure, is dependent upon the adequacy of the design, the quality of the materials used, and the quality of the workmanship. Frequent and thorough inspection of the excavation and the shoring system during all stages of construction must be performed by qualified personnel. An awareness of the changing conditions is essential. The following is a list of common considerations:

1. Check to ensure the Contractor has a current excavation permit from Cal/OSHA. The annual permits are valid from January 1 to December 31 and must be renewed each year. A project permit will be project specific and will be valid during the life of the project.
2. Prior to the beginning of excavation work, become familiar with all aspects of the authorized plans, the location of the work, assumptions made, available soils data, ground water conditions, surcharge loads expected, sequence of operations, location of utilities and underground obstructions, and any other factors that may impact the work at the site. In addition, know who the Contractor has designated as the competent person.
3. Since the primary function of the shoring is the protection of the workers, adjacent property, and the public, it is essential that the inspector be knowledgeable with the minimum safety requirements.
4. Assess all soil being excavated to confirm that it is consistent with the Log of Test Borings and/or with what is contemplated in the excavation plan. Note any differences between the soil anticipated, and what is actually encountered. Discuss the differences with the Contractor.
5. Monitor for changes in the groundwater conditions.
6. As the excavation progresses, be alert for indicators of distress such as the development of tension cracks, or subsidence of soil near the excavation.
7. If the excavation is sloped back without shoring, the need for inspection remains. Sloughing and cave-ins can occur, especially after significant rain events. Confirm that the slope configurations are per the authorized excavation plan.
8. For shored excavations, check that the shoring members size and spacing conform with the authorized excavation plans. The sequence of operations shown on the plans must be followed. Check for full bearing at the ends of jacks and struts and make sure they are secure and will not fall out under impact loads.
9. Review all the materials for quality, integrity, and the strength-grade specified to avoid potential failure of structural elements. Also, check members for visible signs of bending, buckling, and crushing.

10. Manufactured products, such as hydraulic jacks, screw jacks, and trench shields, should be installed and used according to the manufacturer's recommendations.
11. If a ground anchor system is used, the ground anchors must be installed per the authorized plan.
12. When cables are used in conjunction with anchors, they must not be wrapped around sharp corners. Thimbles should be used, and cable clamps installed properly.
13. Surcharge loads need to be monitored so they do not exceed the design loads anticipated for the system.
14. Weather conditions may have an adverse effect on excavations, and some materials, especially clays, may fail due to change in moisture content. Some situations may benefit by protecting the slopes with sheeting or other stabilizing material.
15. Good workmanship makes an excavation safer and easier to inspect. Trouble spots are easier to detect when the excavation is uniform and straight.
16. Vibrations from dynamic loadings, such as vibratory compaction equipment, pile driving, or blasting operations, require additional monitoring of the system.
17. Verify that the Contractor has notified utility owners prior to commencing work if their facilities are within a horizontal distance that is equal to 5 times the excavation depth.

    Underground Service Alert:

    811 or 1-800-227-2600

Northern California (USA)	www.usanorth811.org
Southern California (USA)	www.digalert.org
Statewide	www.call811.com

18. Encourage the use of benchmarks to monitor ground movement in the vicinity of the shoring system before, during, and after excavation. The benchmarks should be monitored for horizontal and vertical displacement. In general, ground settlement accompanies shoring deflection.
19. Egress provisions such as ladders, ramps, stairways, or other means must be provided in excavations 4 feet or over in depth, so that no more than 25 feet of lateral travel is required to exit trench excavations.
20. Adequate protection from hazardous atmospheres must be provided. Air monitoring and other confined space regulations must be followed and documented.
21. Employees must be protected from the hazards of accumulating water, loose or falling debris, or potentially unstable structures.

22. Daily inspections, inspections after storms, and those otherwise required for hazardous conditions are to be made by the Contractor's competent person. Inspections are to be conducted before the start of the work and as needed throughout the shift. The competent person will need to check for potential cave-ins, indications of system failure, and hazardous atmospheres. When the competent person finds a hazardous situation, that person must have the authority to remove the endangered employees from the area until the necessary corrective action has been taken to ensure their safety.
23. Adequate physical barrier protection is to be provided at all excavations. All wells, pits, shafts, etc. must be barricaded or covered. Upon completion of exploration and similar operations, temporary shafts, etc. must be backfilled.

Contractors sometimes propose alternative shoring methods such as wire mesh MSE style walls, "burrito" style retention systems, and other proprietary systems. These will need to be reviewed against the published manufacturer's design criteria and limitations.

11-2 Encroachment Permit Projects

An encroachment permit is required for projects performed by others within State highway right-of-way or adjacent to State highways, including those done under a cooperative agreement, such as a Capital Improvement Project. The Contractor, builder, or owner must apply for and be issued an encroachment permit by the District Permit Engineer.

If the scope of work requires excavation and shoring, plans for this work must accompany the permit application. The plan must be reviewed and authorized by the District Permit Engineer prior to a permit being issued. The Department has an obligation with respect to trenching and shoring work. Be informed of legal responsibilities and requirements; see Chapter 1, *Legal Requirements.*

Although many of the encroachment permit projects are quite simple, some may require complex shoring systems. The District Permit Engineer, on receipt of an application for an encroachment permit, will decide if technical assistance is necessary to review the plan. The plan may be routed to Structure Maintenance, Bridge Design, or Structure Construction for review and commenting.

The plan must conform to all applicable requirements as outlined in Chapter 2, *Cal/OSHA Overview.* It must also conform to the requirements set forth in the permit application. The review process is similar to the process for a typical State contract, except that all correspondence regarding authorization or rejection of the plan must be routed through the District Encroachment Permit Office.

Note that an engineer who prepares shoring plans for an encroachment permit project may not necessarily use the recommended allowable stresses given in this manual. Keep this in mind, when reviewing these types of shoring plans. Acceptance should be based on what is required for a State project (i.e., within the recommended procedures that govern our own improvements), with consideration being given to the background of the Contractor, the work to be done, and the degree of risk involved. Remember, geotechnical engineering is not an exact or precise science.

In order for the State to review and authorize an encroachment permittee's excavation plan or proposed shoring system, a detailed plan of the work to be done must be submitted. At a minimum, the shoring plan must contain the following information:

Encroachment Permit No. (Contractor):

Contractor:	Name, address, phone
Owner:	For whom the work is being done. Include contract number or designation.

Owner Encroachment Permit No.:

Location:	Road, street, highway stationing, etc. indicating the scope or extent of the project.
Purpose:	A description of what the trench or excavation is for (sewer line, retaining wall, etc.).
Soil Profile:	A description of the soil, including the basis of identification, such as surface observation, test borings, observation of adjacent work in the same type of material, reference to a soils investigation report, etc.
Surcharge Loadings:	Any loads, including normal construction loads that are adjacent to the excavation or trench, should be identified and shown on the plans with all pertinent dimensions; examples are highways, railroads, existing structures, etc. The lateral pressures due to these loads will then be added to the basic soil pressures. The minimum surcharge is to be used where not exceeded by above loading considerations.
Excavation/Trenching & Shoring Plan:	The plan for simple excavation work can be in the form of a letter covering the items required. For more complex systems, a complete description of the shoring system, including all members, materials, spacing, etc., is required. The plan may be in the form of a drawing or referenced to the applicable portions of the Construction Safety Orders. In accordance with California Labor Code (CA law), if a shoring system varies from Title 8 of the safety orders, then the shoring plans must be prepared

	and signed by an engineer who is registered as a civil engineer in the State of California.
Manufactured Data:	Catalogs or engineering data for a product should be identified in the plan as supporting data. All specific items and applicable conditions must be outlined on the submittal.
Construction Permit:	Any plan or information submitted should confirm a permit has been secured from Cal/OSHA to perform the excavation work. This is not an authorization of the shoring system by Cal/OSHA.
Inspection:	The Contractor's plan must designate who the competent person on site will be.

The Engineer will review a Contractor's shoring plan in accordance with applicable specifications and the Construction Safety Orders. Deviations from Cal/OSHA or different approaches will be considered, providing adequate supporting data such as calculations, soils investigations, manufacturer's engineering data and references, are submitted. This Caltrans *Trenching & Shoring Manual* is the primary resource available to assist the Engineer during the shoring plan review process.

The District Encroachment Permit Engineer and their staff are responsible for fieldwork inspection. However, there will be occasions where the complexity of the excavation and/or shoring requires assistance from Structure Construction (SC). For major encroachment permit projects, the District may request that SC assigns an Engineer as a representative of the District Permit Engineer. Remember that the administrative or control procedure is different from typical State construction contracts because SC is assisting the District Permit Engineer as a representative of the District Permit Engineer, not the Resident Engineer. Major corrections must be routed through the District Permit Engineer. If there are difficulties with compliance, the District Permit Engineer has the authority to withdraw the encroachment permit, which would have the effect of stopping the work. Close communication between SC and the District Permit Engineer is very important during all phases of the encroachment permit project.

Prior to the start of excavation work, verify that the Contractor and/or owner have all of the proper permits to do the work and have properly notified Underground Service Alert. The work is then monitored to verify that the excavation and/or shoring work is in conformance with the authorized plan.

For more information regarding the encroachment permit process, contact your local District Permit Engineer. Additional resources are found within the Caltrans Traffic Operations Internet page, under Encroachment Permits.

11-3 Ground Anchor Restrained Shoring Systems

Restrained shoring systems are generally considered higher risk due to several factors. The first of these is that these systems are often retaining a greater height of soil. Another risk is the additional elements of the system that must perform as intended and that the sequence of installation and removal affects the loading and performance of the shoring system. There are also risks involved with additional drilling or disturbance below or behind the shoring, such as damaging utilities. This section will focus specifically on ground anchors used as a restraining element.

11-3.01 Engineering Analysis and Construction Sequence

The construction sequence for a restrained shoring system, sheet pile or soldier pile, must be considered when making an engineering analysis. This is true whether the restraint is strutting, a buried anchor block, or a drilled ground anchor. Different loads are imposed on the system before and after the completion of a level of intermediate supports. An analysis for each stage of the system's installation should be performed and an analysis for each stage of support removal during backfilling operations may also be needed.

11-3.02 Components of Ground Anchor Systems

There are many variations or configurations of ground anchor systems. The tension element of a ground anchor may be either prestressing strands or bars using either single or multiple elements. Ground anchors may be alternatively secured against walers or piles.

Figure 11-1 illustrates a typical temporary ground anchor. In this diagram, a bar tendon system is shown; strand systems are similar.

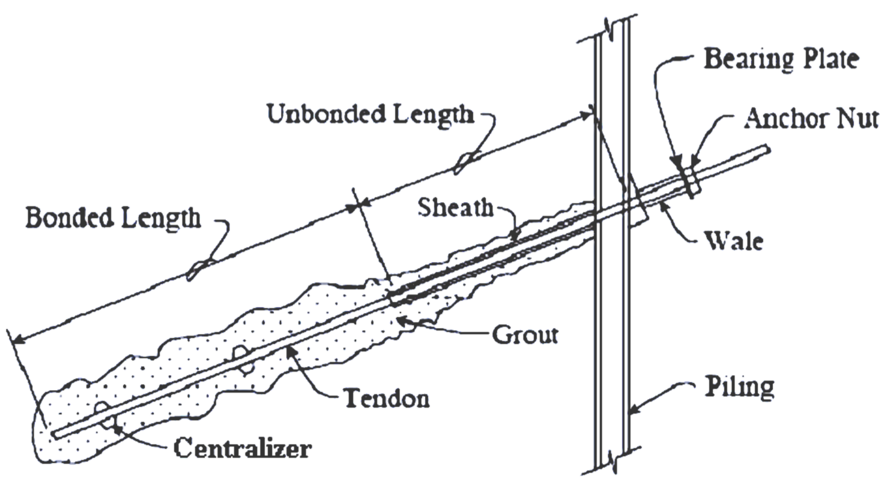


Figure 11-1. Typical Temporary Ground Anchor

The more common components, criteria, and materials used in conjunction with ground anchor shoring systems are listed below:

Piling — Sheet piling and soldier piles. See Chapter 6, *Structural Design of Shoring Systems*, for common materials and allowable stresses.

Waler — These components transfer the resultant of the earth pressure from the piling to the ground anchor. An overstress of 33 percent to the allowable design value is permitted for the walers when proof testing the ground anchor. Anchors for temporary work are often anchored directly against the soldier piling through holes or slots made in the flanges, eliminating the need for walers. Bearing stiffeners and flange cover plates are generally added to the pile section to compensate for the loss of section. A structural analysis of this cut section should always be required.

Tendon — Ground anchor tendons are generally the same high strength bars or strands used in prestressing structural concrete.
The anchorage of the ground anchor tendons at the shoring members consists of (1) bearing plates and anchor nuts for bar tendons, and (2) bearing plates, anchor head, and strand wedges for strand tendons.
The details of the anchorage must accommodate the inclination of the ground anchor relative to the face of the shoring members. Items that may be used to accomplish this are shims or wedge plates placed between the bearing plate and soldier pile or between the wale and sheet piling or soldier piles. For bar tendons, spherical anchor nuts with special bearing washers plus wedge washers, if needed, or specially machined anchor plates may be used.

The tendon should be centered within the drilled hole throughout the entire length. This is accomplished by the use of centralizers (spacers) adequately spaced to prevent the tendon from contacting the sides of the drilled hole or by installation with the use of a hollow stem auger.

Stress — Allowable tensile stress values are based on a percentage of the ultimate tensile strength ($\mathbf{F_{pu}}$) of the anchor. The common value for these are indicated below:

Bars: $\mathbf{F_{pu}} = 150$
Strand: $\mathbf{F_{pu}} = 270$ ksi
(Check manufacturers data for actual ultimate strength.)

Allowable tensile stresses:

At design load: $\mathbf{F_t} \leq 0.6\ \mathbf{F_{pu}}$
At proof load: $\mathbf{F_t} \leq 0.8\ \mathbf{F_{pu}}$
(Both conditions must be checked.)

The relationship between the proof load and the design load can be thought of in this manner: the proof load provides a value that could be considered the maximum value to use for the soil, and the design load applies a safety factor of 1.3 to the proof load.

Grout A flowable Portland cement mixture of grout or concrete encapsulates the tendon and fills the drilled hole within the bonded length. Generally, a neat cement grout is used in drilled holes of diameters up to 8 inches. A sand-cement mixture is used for hole diameters greater than or equal to 8 inches. An aggregate concrete mix is commonly used in very large holes. Type I or II cement is commonly recommended for ground anchors. Type III cement may be used when high early strength is desired. Grout, with very few exceptions, should always be injected at the bottom of the drilled hole. This method ensures complete grouting and will displace any water that has accumulated in the hole.

11-3.03 Ground Anchor

There are several different types of ground anchors. The capacity of a traditional ground anchor depends on a number of interrelated factors discussed later. Alternative anchor products have been developed over the last several years and their analysis would follow the individual published data from the manufacturers. Examples of these include screw anchors, under the product name CHANCE Helical Anchors, and deadman-style anchors, under the product name of Manta Ray Earth Anchors.

The most typical shape of drilled holes for ground anchors is a “straight” shafted drilled hole. Only incidental variations in the sides of the drilled hole are found based on the drilling method and material encountered. Occasionally, a drilled hole will be mechanically enlarged at the end or at multiple points to enhance the ground anchor’s capacity by utilizing a combination of perimeter bond and bearing against the soil, creating a belled hole. A hole with this mechanical widening is referred to as “under-reamed.” This can only be done in soils with sufficient cohesion to prevent collapsing. Figure 11-2 depicts what these drilled holes might look like.

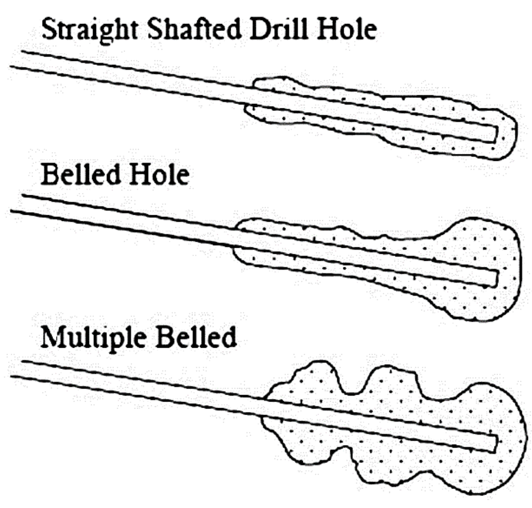


Figure 11-2. Types of drilled hole shapes

The presence of water, either introduced during drilling or existing as ground water, can cause significant reduction in anchor capacity when using a rotary drilling method in some cohesive soils (generally the softer clays).

The grout for a ground anchor is generally placed by tremie or low-pressure grouting methods. High-pressure grouting is seldom used for temporary ground anchor systems, as it is a secondary step, known as post-grouting.

Post-grouting of ground anchors has been used successfully to increase the capacity of an anchor. This method involves the placing of high-pressure grout, 150 psi or higher, in a previously cast anchor. Post-grouting fractures the previously placed anchor grout, disperses new grout into the anchor zone, and expands it. This process compresses the soil and forms an enlarged bulb of grout, thereby increasing the anchor capacity. Post-grouting is done through a separate grout tube installed with the anchor tendon. The separate grout tube will generally have sealed ports uniformly spaced along its length which open under pressure, allowing the grout to exit into the previously formed anchor.

Due to the many factors involved, the determination of anchor capacity can vary quite widely. Proof tests or performance tests of the ground anchors are needed to confirm the anchor capacity. A Federal publication titled *Tiebacks*, the FHWA/RD-82/047 report on ground anchors, provides considerable information for estimating ground anchor capacities for the various types of ground anchors; note that this report may be requested through the Caltrans Transportation Library.

Bond capacity is the ground anchor's resistance to pull out, which is developed by the interaction of the anchor grout (or concrete) surface with the soil along the bonded length.

Determining or estimating the bond (resisting) capacity is a prime element in the design of a ground anchor.

Some shoring designs may include a soils laboratory report, which will contain a recommended value for the bond capacity to be used for ground anchor design. The appropriateness of the value of the bond capacity will only be proven during ground anchor testing.

For most of the temporary shoring work normally encountered, the ground anchors will be straight shafted with low-pressure grout placement. Placement of grout is done with a tube that starts at the bottom of the hole. As grout placement progresses, the tube is extracted, similar to the placement of concrete using a tremie, which utilizes gravity pressure. For these conditions the following criteria can generally be used for estimating the ground anchor capacity.

The determination of the bonded length, $\mathbf{L_b}$, and capacity of the ground anchor is solely the responsibility of the Contractor and is subsequently verified by testing. The Engineer's review of the Contractor's plan must include a check of the unbonded length of the ground anchor. The minimum distance between the front of the bonded zone and the active failure surface behind the wall must not be less than **H/5**. In no case will the minimum distance be less than 5 feet. The unbonded length must not be less than 15 feet. See Figure 11-3 for an illustration of the unbonded length.

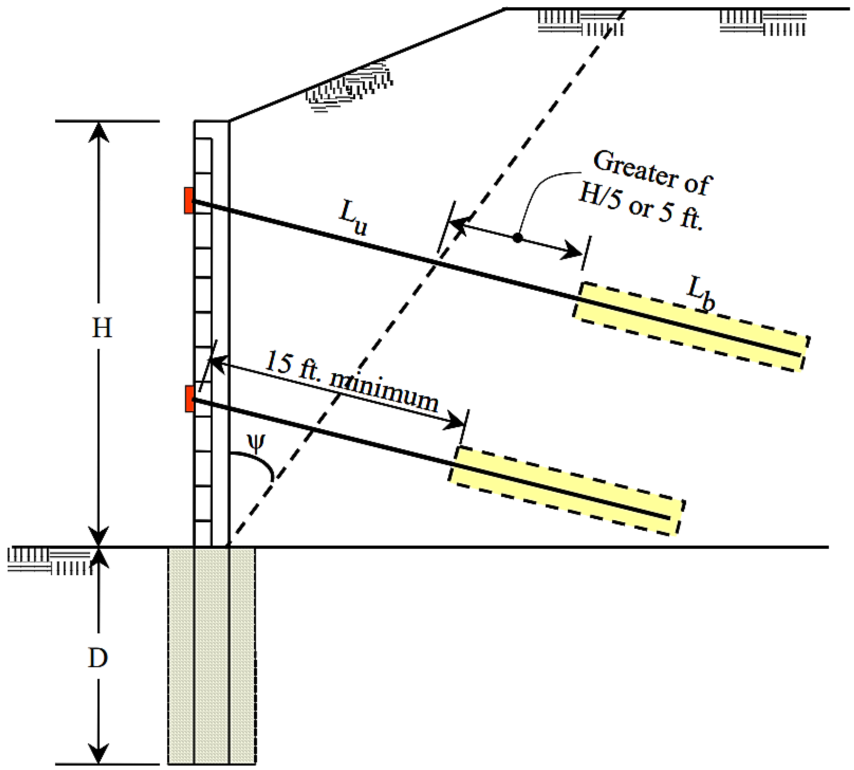


Figure 11-3. Unbonded Length Criteria

The ultimate capacity of the ground anchor is defined as follows:

$$P_{ult} = \pi d L_b S_b$$

Where:

- **d** = Diameter of drilled hole
- **L_b** = Bonded length of the ground anchor
- **L_u** = Unbonded length of the ground anchor
- **S_b** = Bond strength (lbs/ ft^2)
- **Ψ** = Angle between assumed failure plane and vertical. For the Rankine active condition, the angle is **$45° - \phi / 2$**.

The bond strength for ground anchors depends on the number of interrelated factors listed below.

1. Location – amount of effective overburden pressure above the ground anchor.
2. Drilling method and drilled hole configuration.
3. Strength properties, type, and relative density of the soil.
4. Grouting method and pressure.
5. Tendon type, size, and shape.

Since the bond strength is affected by the Contractor’s means and methods, the shoring plans should list the methods they intend to use. The assumed bond strength must be included in the data that is submitted by the Contractor, just as any other soil property, and is preferably included within a geotechnical report. Geotechnical Services staff of the Division of Engineering Services (DES) are available for consultation for concerns or other information regarding bond strength.

11-3.04 Forces on the Vertical Members

Ground anchors are generally inclined; therefore, the vertical component of the ground anchor force must be resisted by the vertical member through skin friction on the embedded length of the piling in contact with the soil and by end bearing. Problems with ground anchor walls have occurred because of excessive downward wall movement when this downward movement was not properly accounted for.

The vertical capacity of the shoring system should be checked, particularly for (1) shoring embedded in loose granular material or soft clays, (2) ground anchors with angles steeper than the standard 15 degrees, and (3) when there are multiple rows of ground anchors. The Engineer is reminded to contact Caltrans Geotechnical Services for assistance when performing a check of the vertical capacity of the shoring elements.

11-3.05 Testing Temporary Ground Anchors

The Contractor is responsible for providing a reasonable test method for verifying the capacity of the ground anchors after installation. Anchors are tested to verify they can sustain the design load over time without excessive movement. The need to test anchors is more important when the system will support, or is adjacent to, existing structures, and when the system will be in place for an extended period of time.

In the Contractor’s test methods, the Contractor should consider the degree of risk to the adjacent surroundings and structures when determining the number of ground anchors tested, the duration of the tests, the allowable movement, and the allowable load loss. High-risk situations would include cases where settlement or other damage would be experienced by adjacent facilities. Table 11-1 is a recommended list of minimum criteria for testing temporary ground anchors.

Generally, the shoring plans with ground anchors should include:

1. Ground anchor load testing criteria, which should minimally consist of proof load test values.
2. Frequency of testing (number of anchors to be tested).
3. Test load duration (to capture anchor creep).
4. Allowable movement or loss of load permissible during the testing time frame.
5. The anticipated life of the shoring system.
6. The remedial measures that are to be taken when, or if, test anchors fail to meet the specified criteria.

Pressure gauges or load cells used for determining test loads should have been recently calibrated by a certified lab, they should be clean and not abused, and they should be in good working order. The calibration dates should be determined and recorded. Calibration dates within one year are generally acceptable.

A ground anchor that does not satisfy the testing criteria may still have some value. The Contractor will need to revise the shoring plan to address this. Often the revision will be able to utilize extra capacity of adjacent ground anchors to make up for the reduced value, or an additional ground anchor will be placed to supplement the low value ground anchor.

11-3.05A Proof Testing

Applying a sustained proof load to a ground anchor and measuring anchor movement over a specified period of time is the typical method of proof testing ground anchors. Proof testing may begin after the grout has achieved the desired strength. A specified number of the ground anchors will be proof tested by the method specified on the Contractor's authorized plans (see Table 11-1).

Table 11-1. Recommended Ground Anchor Proof Test Criteria

Test Load	Load Hold Duration	% of Ground Anchors to be Load Tested
Cohesionless Soils Normal Risk 1.2 to 1.3 of Design Load	10 minutes	10% for each soil type encountered
High Risk ≥1.3 Design Load	10 minutes	20% to 100%
Cohesive Soils Normal Risk 1.2 to 1.3 Design Load	30 minutes	10%
High Risk ≥1.3 Design Load	60 minutes	30% to 100%
Adverse conditions When ground water is present or in soft clays	60 minutes for 10 minutes for	10%, and 90% (remaining)

The unbonded length, L_u, of a ground anchor is left ungrouted prior to and during testing (see Figure 11-3) as standard practice. This ensures that only the bonded length, L_b, is carrying the proof load during testing. It is not desirable to have loads transferred to the soil through grout (or concrete) in the unbonded region since this length is considered to be within the zone of the failure wedge and would not contribute to the resisting portion of the system. This also prevents the grout column in the hole from bearing against the wall facing, which would produce erroneous test results.

Forensic investigations on Caltrans specific ground anchors have shown that ground anchors in small diameter holes (6 inches or less) develop most of their capacity in the bonded length despite any additional grout which may be in the unbonded length zone. Regardless, grout in the unbonded zone is to be avoided so there is a clear separation between the wall and the soil restrained, and that of the anchors' bonded zone. This phenomenon is not true for larger diameter ground anchors.

Generally, the Contractor will specify an alignment load of 5 percent to 10 percent of the proof load, which is initially applied to the tendon to secure the jack against the anchor head and stabilize the setup. The load is then increased until the proof load is achieved. Generally, a maximum amount of time is specified to reach proof load. Once the proof load is attained, the load hold period begins. Movement of the ground anchor is normally measured by using a dial indicator gauge that is positioned as shown in Figure 11-4. The dial indicator gauge is mounted on a tripod that is independent of the ground anchor and shoring.

The tip of the dial indicator gauge is positioned against a flat surface perpendicular to the centerline of the tendon; this can be a plate secured to the tendon. The piston of the jack may be used in lieu of a plate if the jack is not going to have to be cycled during the test. As long as the dial indicator gauge is mounted independently of the shoring system, only movement of the anchor, due to the proof load, will be measured. Continuous jacking to maintain the specified proof load during the load hold period is essential to offset losses resulting from anchor creep or movement of the shoring into the supporting soil.

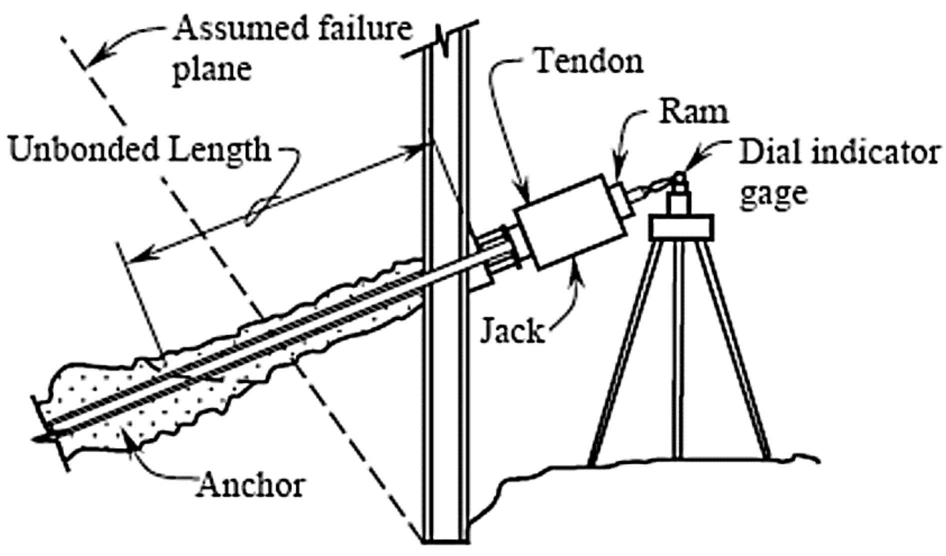


Figure 11-4. Proof Testing Equipment Setup Example

Measurements from the dial indicator gauge are taken periodically during the load hold period in accordance with the Contractor's authorized shoring plan. The total movement measured during the load hold period of time is compared to the allowable value indicated on the Contractor's authorized shoring plans to determine the acceptability of the anchor.

It is important that the proof load be reached quickly. When excessive time is taken to reach the proof load or the proof load is held for an excessive amount of time before beginning the measurement of creep movement, the creep rate indicated will not be representative. For the creep measurement to be accurate, the starting time must begin when the proof load is first reached.

As an alternative to measuring creep movement with a dial indicator gauge, the Contractor may propose a "lift-off test". A "lift-off test" compares the force on the ground anchor at seating to the force required to lift the anchor head off of the bearing plate. The comparison should be made over a specified period of time. The lost force can be converted into creep movement to provide an estimate of the amount of creep over the life of the shoring system.

Use of the "lift-off test" may not accurately predict overall anchor movement. During the time period between lock-off and lift-off, the ground anchor may creep, or the wall may move into the soil, or both. These two components cannot be differentiated. If the test is done accurately, results are likely to be a conservative measure of anchor movement. Structure Construction recommends the use of a dial indicator gauge to monitor creep rather than lift-off tests.

11-3.05B Performance Testing

Performance testing is similar to, but more extensive, than proof testing. Performance testing is used to establish the movement behavior for a ground anchor at a particular site. Performance testing is not normally specified for temporary shoring, but it can be utilized to identify the causes of anchor movement. Performance testing consists of incremental loading and unloading of a ground anchor in conjunction with measuring movement.

11-3.05C Evaluation of Creep Movement

Long-term ground anchor creep can be estimated from measurements taken during initial short term proof testing. This process involves extrapolating measurements taken during proof testing to determine the anticipated total creep over the period the shoring system is in use; the anchor creep is roughly modeled by a curve which is a logarithmic function of time.

The general formula listed below for the determination of the anticipated long-term creep is only an estimate of the potential anchor creep and should be used in conjunction with periodic monitoring of the wall movement. This formula does not accurately predict anchor creep for soft cohesive soils.

Based on the assumed creep behavior, the following formula can be utilized to evaluate the long-term effects of creep:

General formula for long term creep:

$$\boldsymbol{\Delta_{2-3} = C\left[\log_{10}\left(\frac{T_3}{T_2}\right)\right]} \qquad (11\text{-}3\text{-}1)$$

Where:

$$\mathbf{C} = \Delta_{1-2} \Big/ \left[\boldsymbol{\log_{10}\left(\frac{T_2}{T_1}\right)}\right] \qquad (11\text{-}3\text{-}2)$$

C = Constant determined using the movement from $\mathbf{T_1}$ to $\mathbf{T_2}$. This constant is typically measured in inches. For the period from $\mathbf{T_2}$ to $\mathbf{T_3}$, the value of **C** is already known, and the value of Δ can be calculated with that value of **C**.

$\boldsymbol{\Delta}$ = Creep movement (inches) specified on the plans for times $\mathbf{T_1}$, $\mathbf{T_2}$, or $\mathbf{T_3}$. (or measured in the field)

$\mathbf{T_1}$ = Time of first movement measurement during load hold period. (usually within 1 minute after proof load is applied)

$\mathbf{T_2}$ = Time of last movement measurement during load hold period.

$\mathbf{T_3}$ = Time the shoring system will be in use.

If using a “lift-off test” to estimate the creep movement, the following approximation needs to be made for substitution into the above equation (11-3-2):

$$\boldsymbol{\Delta_{1-2} = (P_1 - P_2)\frac{L_U{}'}{AE}} \qquad (11\text{-}3\text{-}3)$$

Where:

$\mathbf{P_1}$ = Force at seating

$\mathbf{P_2}$ = Force at lift-off

$\mathbf{L_u'}$ = $L_u + \Delta L$

$\boldsymbol{\Delta}\mathbf{L}$ = 3 to 5 feet of anchor length necessary for the jack and anchor wedges

$\boldsymbol{\Delta_{1\text{-}2}}$ = Amount of movement during the load hold period.

A = Area of strand or bar in anchor

E = Modulus of elasticity of the strand or bar in anchor

Example 11-1A demonstrates the calculation of long-term creep by Equation 11-3-1 and by the lift-off method in Equation 11-3-3.

11-3.06 Wall Movement and Settlement

As a rule of thumb, the settlement of the soil behind a ground anchor wall, where the ground anchors are locked-off at a high percentage of the design force, can be approximated as equal to the movement at the top of the wall caused by anchor creep and deflection of the piling. Reference is made to Section 7-3 titled *System Deflection* of Chapter 7, *Unrestrained Shoring Systems*.

If a shoring system is to be in close proximity to an existing structure where settlement might be detrimental, significant deflection and creep of the shoring system would not be acceptable. If a shoring system will not affect permanent structures or when the shoring does not support something like a haul road, reasonable lateral movement and settlement may be tolerated.

11-3.07 Lock-off Force

The lock-off force is the percentage of the required design force that the anchor wedges or anchor nut are seated at after seating losses. A value of $\mathbf{0.8T_{DESIGN}}$ is typically recommended as the lock-off force but lower or higher values may be used to achieve specific design needs.

One method for obtaining the proper lock-off force for strand systems is to insert a shim plate under the anchor head equal to the elastic elongation of the tendon produced by a force equal to the proof load minus the lock-off load. A correction for seating of the wedges in the anchor head is often subtracted from the shim plate thickness. To determine the thickness of the shim plate you may use the following equation:

$$\mathbf{t_{shim}} = \frac{\left(\mathbf{P_{proof}} - \mathbf{P_{lockoff}}\right)\mathbf{L_u}'}{\mathbf{AE}} - \boldsymbol{\Delta}\mathbf{L} \qquad (11\text{-}3\text{-}4)$$

Where:

- $\mathbf{t_{shim}}$ = thickness of shim
- $\mathbf{P_{proof}}$ = Proof load (kips)
- $\mathbf{P_{lockoff}}$ = Lock-off load (kips)
- **A** = Area of tendon steel (bar or strands) (in^2)
- **E** = Modulus of Elasticity of anchor (strand or bar) (psi)
- $\mathbf{L_u'}$ = The unbonded length of the anchor (ft)
- **ΔL** = 3 to 5 feet of anchor length necessary for the jack and anchor wedges

The seating loss for threaded bar systems is much less than that for strand systems and can vary between 0 and 1/16 of an inch.

For strand systems, seating loss can vary between 3/8 and 5/8 of an inch. The seating loss should be determined by the designer of the system and verified during installation. Oftentimes, wedges are mechanically seated, minimizing seating loss and resulting in the use of a lesser value for the seating loss.

After seating the wedges in the anchor head at the proof load, the tendon is loaded, the shim is removed, and the whole anchor head assembly is seated against the bearing plate.

11-3.08 Corrosion Protection

The Contractor's submittal must address potential corrosion of the tendon after it has been stressed.

For very short-term installations in non-corrosive sites, corrosion protection may not be necessary. The exposed steel may not be affected by a small amount of corrosion that occurs during its life.

For longer term installations, grouting of the bonded and unbonded length is generally adequate to minimize corrosion in most non-corrosive sites. Encapsulating or coating any un-grouted portions (anchor head, bearing plate, wedges, strand, etc.) of the ground anchor system may be necessary to guard against corrosion.

For long-term installations or installations in corrosive sites, more elaborate corrosion protection schemes may be necessary. (Grease is often used as a corrosion inhibitor). Figure 11-5 depicts tendons encapsulated in pre-greased and pre-grouted plastic sheaths generally used for permanent installations.

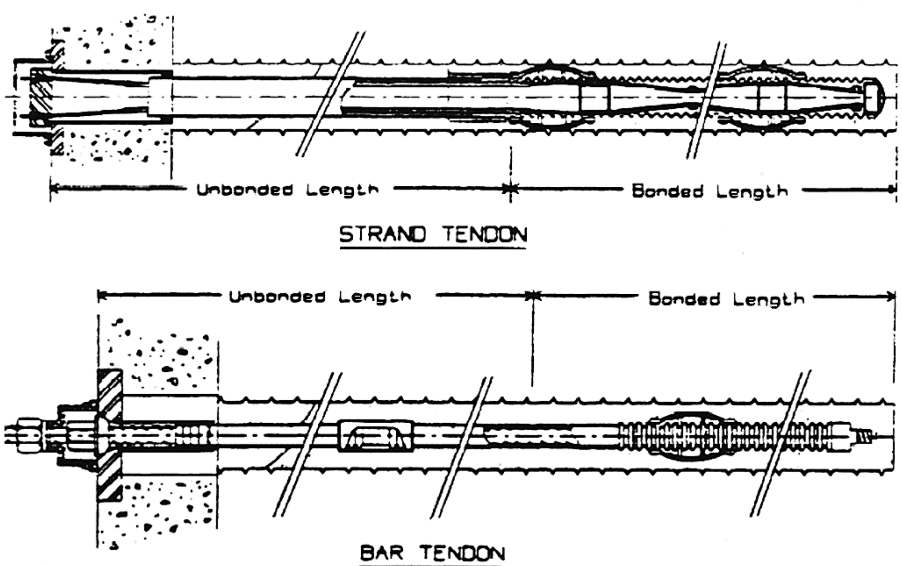


Figure 11-5. Bar or Strand Tendons

11-3.09 General Steps for Checking Ground Anchor Shoring Submittal

1. Review submittal for completeness.
2. Determine $\mathbf{K_a}$ and $\mathbf{K_p}$. In the rare cases where the shoring is not allowed to move sufficiently, determine $\mathbf{K_o}$ in lieu of $\mathbf{K_a}$.
3. Develop pressure diagrams.
4. Determine forces.
5. Determine the moments at the top of the pile above the highest ground anchor.
6. Solve for depth (**D**), for both lateral, and ground anchor force ($\mathbf{T_H}$). Confirm "**D**" is sufficient to resist the vertical loads.
7. Check pile section.
8. Check anchor capacity.
9. Check miscellaneous details.
10. Check adequacy of ground anchor test procedure.
11. Review corrosion proposal.
12. General: Consider effects of wall deflection and subsequent soil settlement on any surface feature behind the shoring wall.

11-3.10 Estimating Long Term Creep

Determine the long-term effects of creep.

11-3.10A Example 11-1A Measurement and Time Method

Given:

The shoring plans indicate that a proof load must be applied in 2 minutes or less, and then the load must be held for ten minutes. The test begins immediately upon reaching the proof load value. Measurements of movement are to be taken at 1, 4, 6, 8 and 10 minutes. The proof load is to be 133 percent of the design load. The maximum permissible movement between 1 and 10 minutes of time is 0.1 inch. All ground anchors are to be tested. The system is anticipated to be in place for 1 year.

Given:

$\Delta_{\mathbf{Allow}}$ = 0.1 inches

$\mathbf{T_1}$ = 1 minute

$\mathbf{T_2}$ = 10 minutes

Solution:

$$T_3 = (1Y)\left(365\,\frac{D}{Y}\right)\left(24\,\frac{H}{D}\right)\left(60\,\frac{M}{H}\right) = 525{,}600 \text{ minutes} \qquad (11\text{-}3\text{-}5)$$

Utilizing Equation 11-3-2 above:

$$C = \frac{\Delta_{1-2}}{\left[\log_{10}\left(\frac{T_2}{T_1}\right)\right]} = \frac{0.1 \text{ inch}}{\left[\log_{10}\left(\frac{10}{1}\right)\right]} = 0.1 \text{ inch} \qquad (11\text{-}3\text{-}6)$$

Estimated long-term creep movement from Equation 11-3-1 above:

$$\Delta_{2-3} = (C)\log_{10}\left(\frac{T_3}{T_2}\right) = (0.1\,)\log_{10}\left(\frac{525{,}600}{10}\right) = 0.47 \text{ inches} \qquad (11\text{-}3\text{-}7)$$

The proof load and duration of test are reasonable and exceed the recommended values shown in Table 11-1. Applying the proof load in a short period of time and beginning measurements immediately upon reaching that load produce meaningful test results which can be compared to the calculated long-term creep movement for the anchor.

If the shoring system were in close proximity to an existing structure that could not tolerate 1/2 inch of wall deflection, the design may not be acceptable. If the shoring will not affect permanent structures or when the shoring does not support something like a haul road, the anticipated movement may be acceptable.

11-3.10B Example 10-1B Lift-Off Load Method

Given:

"Lift-off test" will be performed 24 hours after wedges are seated. The force at seating the wedges will be 83,000 pounds and the lift-off force will be no less than 67,900 pounds.

L_u' = **L_u + ΔL** ≈ 20 ft (which is the unbonded length of 15′ + 5′ for the strand length in the jack)

A = 0.647 in²

E = 28x10^6 psi

T_1 = 1 minute, this is the time until the wedges are seated (at the start of the 24 hours)

Solution:

$$\Delta_{1-2} = (P_1 - P_2)\frac{L_U{}'}{AE} \quad (11\text{-}3\text{-}8)$$

$$\approx \frac{(83{,}000 - 67{,}900)(20)(12)}{(0.647)(28\text{x}10^6)} \approx 0.2 \text{ inch} \quad (11\text{-}3\text{-}9)$$

$$C \approx \frac{0.2}{\left[\log_{10}\left(\frac{(24)(60)}{1}\right)\right]} = 0.063 \text{ inch} \quad (11\text{-}3\text{-}10)$$

Estimated long term creep at one year:

T_2 = 24 hours = 1 day (end of the “lift-off test”)

T_3 = 1 year = 365 days

$$\Delta_{2-3} \approx (C)\log_{10}\left(\frac{T_3}{T_2}\right) = (0.063 \text{ inch})\log_{10}\left(\frac{365 \text{ days}}{1 \text{ day}}\right) \quad (11\text{-}3\text{-}11)$$

$\boldsymbol{\Delta_{2-3}}$= **0.16 inch** [from the end of the “lift-off test” (T_2) to the end of the 1 year (T_3)]

11-4 Summary

The Department has an obligation with respect to trenching and shoring work. Be informed of legal responsibilities and requirements (Refer to Chapter 1, *Legal Requirements,* and Chapter 2*, Cal/OSHA Overview*).

Soil Mechanics (Geotechnical Engineering) is not a precise science. Be aware of the effects that assumptions can make. Simplified engineering analysis procedures can be used for much of the trenching and shoring work that will be encountered.

The actual construction work is of equal importance compared to the engineering design or planning. The Contractor and the Engineer must always be alert to changing conditions and must take appropriate action. Technical assistance is available. The Engineer at the jobsite must be able to recognize when assistance is required. The need for good engineering judgment is essential.

Work involving railroads requires additional controls and specific administrative procedures.

The following is a summary of Caltrans requirements and procedures in regard to trenching, excavation, and shoring work:

1. The law (State Statute, §137.6, as discussed in Section 1-4, *State Statutes* of Chapter 1) requires that a California registered engineer review the Contractor's plans for temporary structures in connection with State highway work. Shoring plans are included in this category.
2. The Engineer will ascertain that the Contractor has obtained a proper excavation or trenching permit from Cal/OSHA before any work starts, and that the permit (or copy) is properly posted at the work site.
3. If the trench is less than or equal to 20 feet deep and the Contractor submits a plan in accordance with the standard details found within the Cal/OSHA Construction Safety Orders, it is not necessary to have the plans prepared by a professional engineer. The Engineer will confirm that the Contractor's plan does indeed conform to the applicable standard details found in Cal/OSHA, and need not make an independent engineering analysis.
4. If a trench is over 20 feet in depth, or if the Cal/OSHA standard details are not applicable, the plans must be prepared by a professional engineer.
5. If the Contractor's shoring plan deviates from the Cal/OSHA Construction Safety Orders, the plan must be prepared by a California registered professional engineer and the reviewing engineer will perform an independent engineering analysis.
6. When shoring plans are designed by firms specializing in temporary support systems, soil restraint systems or sloping systems, good engineering judgment is to be used for the review. Shoring designs by such firms may appear less conservative when analyzed using the methods proposed in this manual. Consequently, the shoring plan may need to be reviewed in the manner in which it was designed. Obtain assistance from the SC Falsework Engineer as needed.
7. For any shoring work that requires review and authorization by a Railroad, the SC Falsework Engineer[1] will be the liaison between the project and the Railroad. The Structure Representative will submit the Contractor's shoring plans to SC Sacramento after review. The review should be complete, so that the plans are ready for authorization. The Structure Representative should inform the Contractor of the proper procedure, and the time required for Railroad review and authorization.
8. Any revisions to the shoring plans should be done by the plan originator or by his/her authorized representative.

[1] Caltrans internal use only

APPENDIX A

ADDITIONAL THEORY FOR INQUIRING MINDS

Appendix A: Additional Theory for Inquiring Minds

Table of Contents

Appendix A: Additional Theory for Inquiring Minds 1
Table of Contents 1
A-1 Introduction 2
A-2 Log Spiral Passive Earth Pressure 2
A-2.01 Composite Failure Surface 2
A-2.02 Noncomposite Log Spiral Failure Surface 8
A-3 Culmann's Graphical Solution for Active Earth Pressure 18
A-3.01 Example A-1: Culmann Graphical Method 23
A-3.02 Example A-2: Critical Active Wedge Method 26
A-3.03 Example A-3 (2023 AREMA Manual Page 8-5-20) 28
A-3.04 Example A-4 (2023 AREMA Manual Page 8-5-20) 31
A-3.05 Example A-5 (2023 AREMA Manual Page 8-5-21) 33
A-3.06 Example A-6 (2023 AREMA Manual Page 8-5-21) 37

A-1 Introduction

The information within this appendix is provided for additional reference, and includes some additional details and some advanced topics. These sections are excerpted out of the 2011 *Trenching and Shoring Manual*. These sections were removed to streamline the manual contents to the most relevant information that is commonly used in the review of excavation plans. These sections provide mainly alternative approaches for analysis comparison of various approaches. All these are good for edification but not essential for the review task at hand. As such, no updates to the latest referenced guidance are provided. This is purely archival information. If the reader is to utilize any of these approaches, then it is incumbent upon the reader to find and utilize the latest references noted within this appendix.

A-2 Log-Spiral Passive Earth Pressure

As mentioned in previous sections, Rankine's theory should not be used to calculate the passive earth pressure forces for a shoring system because it does not account for wall friction, and it assumes a linear failure plane. While Coulomb's theory to determine the passive earth pressure force accounts for the angle of wall friction (δ), the theory assumes a linear failure surface. The result is an error in Coulomb's calculated force since the actual sliding surface is curved rather than planar. Coulomb's theory gives increasingly erroneous values of passive earth pressure as the wall friction (δ) increases. Therefore, Coulomb's theory could lead to unsafe shoring system designs because the calculated value of passive earth pressure would become higher than the soil could generate. Terzaghi (1943) suggested that combining a logarithmic spiral and a straight line could represent the failure surface. Morrison and Ebeling (1995) suggested a single arc of the logarithmic spiral could realistically represent the failure surface. Both methods (Terzaghi 1943 composite failure surface and Morison and Ebeling 1995) are implemented in this *Trenching and Shoring Manual.*

A-2.01 Composite Failure Surface

The composite failure surface will be examined first. As seen in Figure A-1,the logarithmic spiral portion of the failure surface (BD) is governed by the height of the wall (AB), the location of the center of the logarithmic spiral arc (O), and the soil's internal friction angle (ϕ). However, the curved failure surface will be circular (**R = R_o**) in cohesive soil (for total stress analysis, $\phi = 0$). The spiral surface is given as:

$$\mathbf{R = R_o e^{\theta \tan \phi}} \qquad \text{(A-2-1)}$$

R_o is obtained from triangle OAB. The upper portion of DE is a straight line, which is tangent to the curve BD at point D. DE makes an angle α_1 with the horizontal as given in Equation 4-3-21 of Chapter 4, *Earth Pressure Theory and Application*.

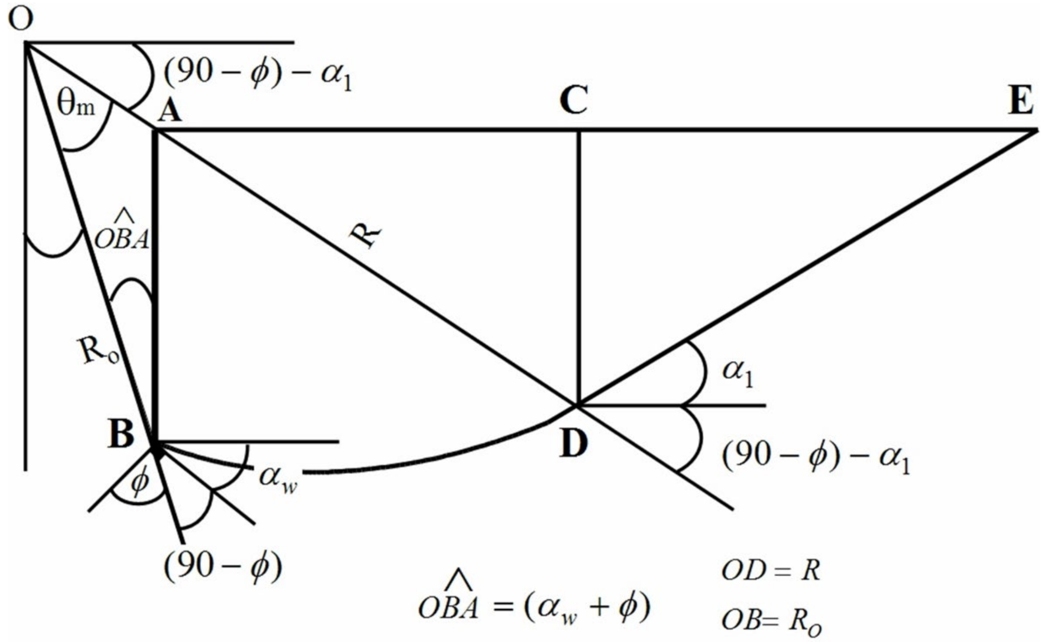


Figure A-1. Geometry of the developing mobilized failure plane (Shamsabadi, et al,. 2005)

The logarithmic spiral leaves the wall at the takeoff angle (α_w) at radius OB and intersects the conjugate failure surface wedge CDE. AD lies on a ray of the logarithmic spiral zone that must pass through the center of the logarithmic spiral arc. As a result, the location of the center of the Log-Spiral curve (O) can be accurately defined based on the subtended angle θ_m. Either moment equilibrium or force equilibrium can be used to calculate the passive earth pressure force per unit length of the wall. Several authors have calculated passive earth pressure coefficients using log spiral failure surfaces (moment method and method of slices), circular failure surfaces and elliptical failure surfaces. The shoring engineer has the option to use any of these methods.

A-2.01A Force Equilibrium Procedures

The log spiral surface at the bottom of the wall, illustrated in Figure A-1, starts with the takeoff angle (α_w), which is calculated as follows:

$$\boldsymbol{\alpha_w = \left(45 - \frac{\phi}{2}\right) - \alpha_p} \quad \text{(A-2-2)}$$

The angle α_w has a positive value when it is above the horizontal and a negative when it is below the horizontal.

$$\boldsymbol{\alpha_p = \frac{1}{2}\tan^{-1}\left[\frac{2\ K(\tan\ \delta)}{K-1}\right]} \quad \text{(A-2-3)}$$

Where δ is the wall interface friction angle that varies from zero to its full value δ (where $\delta = \delta_{ult}$) as a function of ϕ. The coefficient **K** is the horizontal to vertical stress ratio given in Equation A-2-4.

$$\mathbf{K} = (\mathbf{A1} + \mathbf{A2})/\mathbf{A3} \qquad \text{(A-2-4)}$$

Where:

$$\mathbf{A1} = \mathbf{1} + \sin^2 \phi + \frac{\mathbf{C}}{\sigma_z} \sin(2\phi) \qquad \text{(A-2-5)}$$

$$\mathbf{A2} = 2\cos\phi\left(\sqrt{\left(\tan\phi + \frac{\mathbf{C}}{\sigma_z}\right)^2 + \tan^2\delta\left[4\left(\left(\frac{\mathbf{C}}{\sigma_z}\right)^2 + \frac{\mathbf{C}}{\sigma_z}\tan\phi\right) - 1\right]}\right)$$

(A-2-6)

$$\mathbf{A3} = \cos^2\phi + 4\tan^2\delta \qquad \text{(A-2-7)}$$

$$\sigma_z = \gamma\mathbf{H} \qquad \text{(A-2-8)}$$

The value of θ_m can be obtained from the following relationships:

$$\theta_m = \alpha_1 - \alpha_w \qquad \text{(A-2-9)}$$

Where α_1 is the failure angle of slice 1 in Figure A-2.

Therefore, the value of θ_m, can be obtained both from the geometry of the composite failure surface and/or from the state of the stresses of a soil element at the bottom of the wall. The geometry of the failure surface presented in Figure A-1 can be established using Equations A-2-2, A-2-3, and A-2-9. It should be noted that the direction of the takeoff angle (α_w) is a function of the wall-soil interface friction angle (δ), the angle of internal friction (ϕ), the cohesion (**c**) of the soil, and the wall height (**H**). Once the geometry of the failure plane is established then the failure mass can be divided into slices as shown in Figure A-2. Earth pressure $\mathbf{P_{ph}}$ is then calculated by summation of forces in the vertical and horizontal direction for all slices using Equation A-2-10.

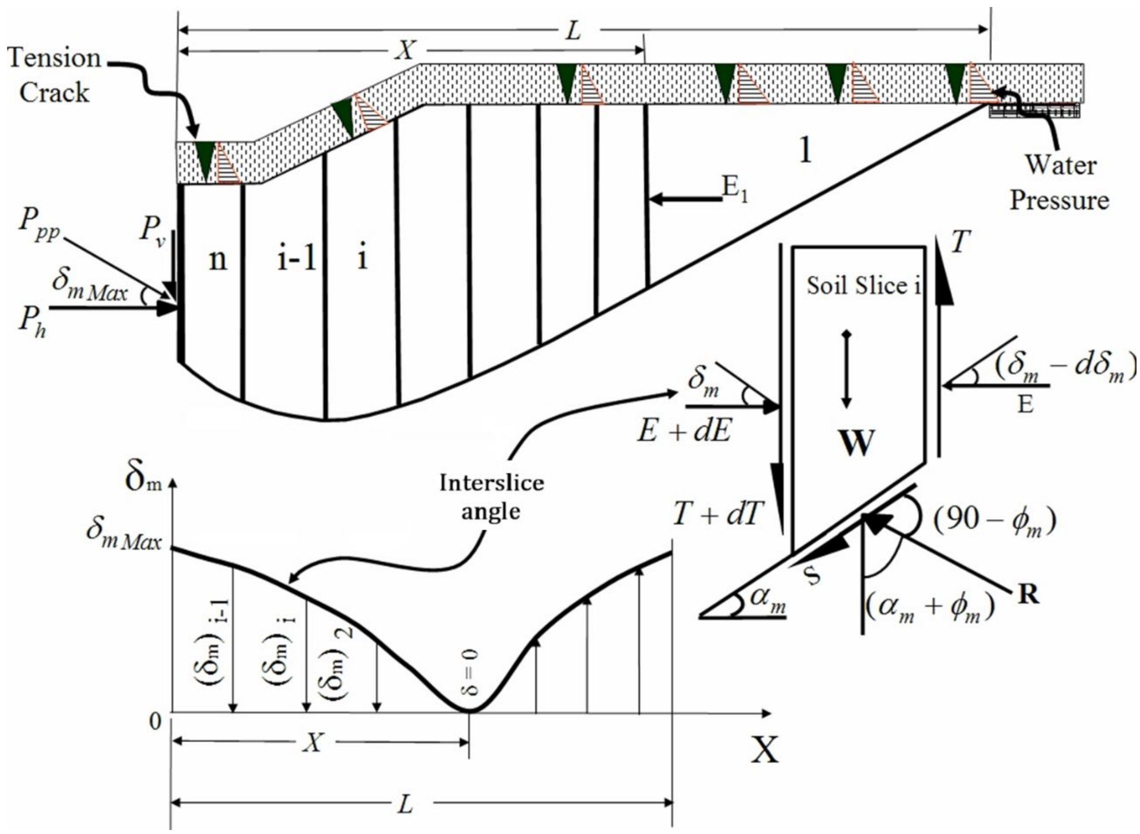


Figure A-2. Geometry of the Failure Surface and Associated Interslice Forces

$$\mathbf{P_h = \frac{\sum_{i=1}^{n} \; dE}{[1 - \tan\delta \tan(\alpha_w + \phi)]}} \qquad \text{(A-2-10)}$$

Where:

$$\mathbf{dE = \frac{W\tan(\alpha + \phi) + (C)(L)[\sin\;\alpha \tan(\alpha + \phi) + \cos\;\alpha]}{1 - \tan\delta \tan(\alpha + \phi)}} \qquad \text{(A-2-11)}$$

By dividing the resisting wall force $\mathbf{P_h}$ by $\mathbf{0.5\gamma H^2}$, one obtains the horizontal passive pressure coefficient ($\mathbf{K_{ph}}$) which is expressed as:

$$\mathbf{K_{ph} = \frac{2P_{ph}}{\gamma H^2}} \qquad \text{(A-2-12)}$$

A-2.01B Moment Equilibrium Procedures

The passive earth pressure $\mathbf{P_p}$ can be determined by summing moments (rather than forces as described above) about the center of the log spiral point O considering those forces acting on the free body associated with the weight and cohesion respectively. This is a two-step process, which is solved by method of superposition. Considering the weight of the free body diagram shown in Figure A-3, $\mathbf{P_p}$ can be determined as follows:

$$\mathbf{E_w} = \frac{(\mathbf{w_{ABDF}})(\mathbf{L_2}) + (\mathbf{P_R})(\mathbf{L_3})}{\mathbf{L_1}} \qquad \text{(A-2-13)}$$

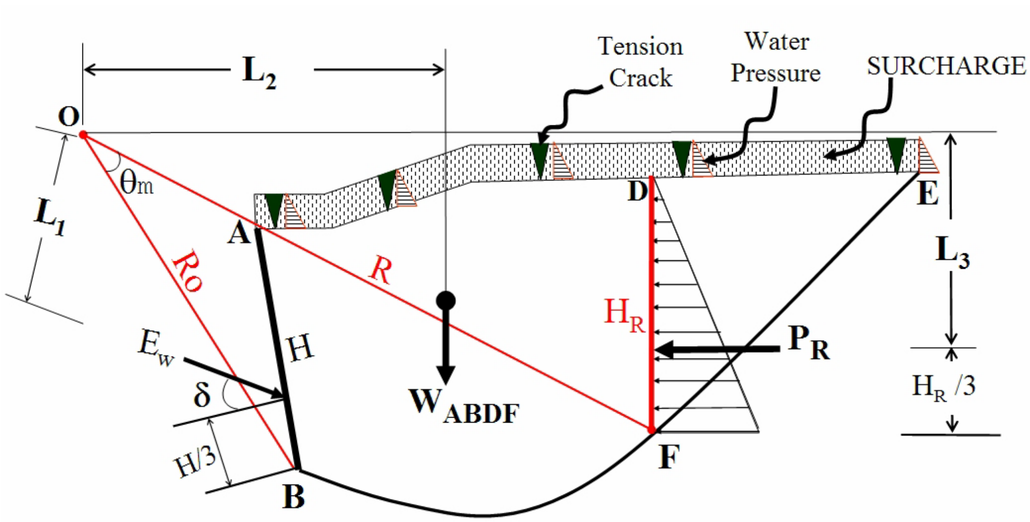


Figure A-3. Geometry of the Failure Surface due to Weight

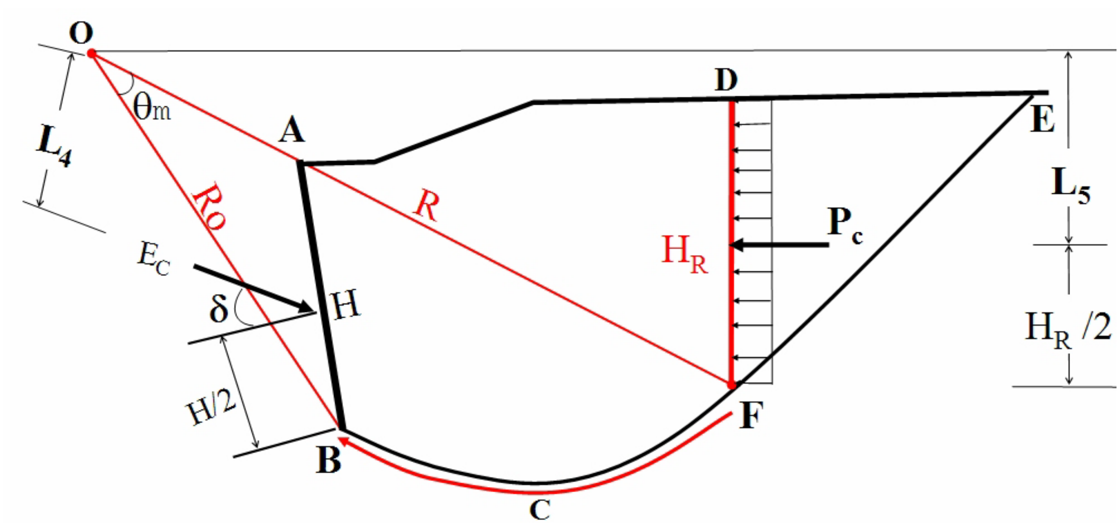


Figure A-4. Geometry of the Failure Surface Due to Cohesion

Considering the cohesion part of the backfill only as shown in Figure A-3 the passive earth pressure due to cohesion (**E_C**) can be determined by the summation of moments about the center of log spiral point O as follows:

$$\mathbf{E_C = \frac{M_C + (P_C)(L_5)}{L_4}} \quad \text{(A-2-14)}$$

Where:

$$\mathbf{M_C = \frac{C + P_C}{\tan\ \varphi}\left(R^2 - R_0^2\right)} \quad \text{(A-2-15)}$$

For the cohesive soil where the soil friction is equal to zero:

$$\mathbf{M_C = (C)(\theta)(R^2) M_C = (C)(\theta)(R^2)} \quad \text{(A-2-16)}$$

W_{ABDF} = Weight of log spiral section and the surcharge weight.
H_R = Height of left side of Rankine section.
P_R = Horizontal force component of Rankine section DFE.
P_C = Horizontal force component of Rankine section DFE due to cohesion.
E_W = Total lateral earth pressure due to weight.
E_C = Total lateral earth pressure due to cohesion.
M_C = Moment due to cohesion due to log spiral section.

Equations A-2-14 and A-2-15 are obtained using the following procedures:

1. Calculate earth pressure on vertical face of DEF using Rankine's equation.
2. Calculate weight of the zone ABDF including the surcharge.
3. Take the moment about point O.

The total lateral earth pressure due to weight and cohesion is the summation of A-2-13 and A-2-14.

$$\mathbf{P_p = E_w + E_c} \quad \text{(A-2-17)}$$

The passive earth pressure force **Pp** is obtained by summing **E_W** and **E_C**. However, this may not be the unique solution to the problem as only one trial surface is examined. The value of passive pressure **Pp** must be determined for several trial surfaces as shown in Figure A-5 until the minimum value of **Pp** is attained. Note that failure surface 2 is the critical failure surface.

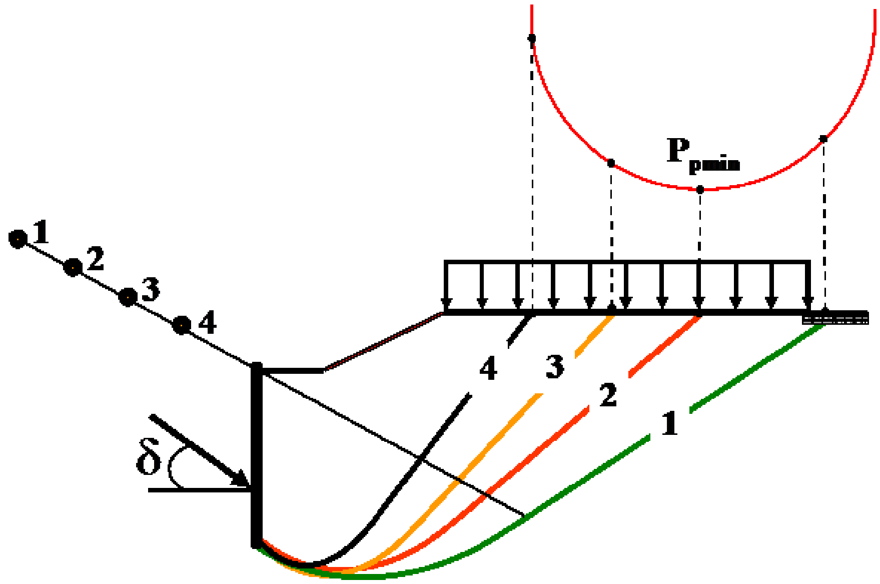


Figure A-5. Moment Method

A-2.02 Noncomposite Log Spiral Failure Surface

As shown in Figure A-6, it is assumed that a single arc of the log spiral curve can represent the entire failure surface. Note: do not confuse this discussion with the topic of global stability, which is alluded to in Chapter 10, *Special Conditions,* Section 10-5, *Slope Stability*. As described previously, the equations of the force equilibrium and or moment equilibrium methods are applied to calculate the passive or active force directly without breaking the failure surface into an arc of logarithmic spiral zone and a Rankine zone.

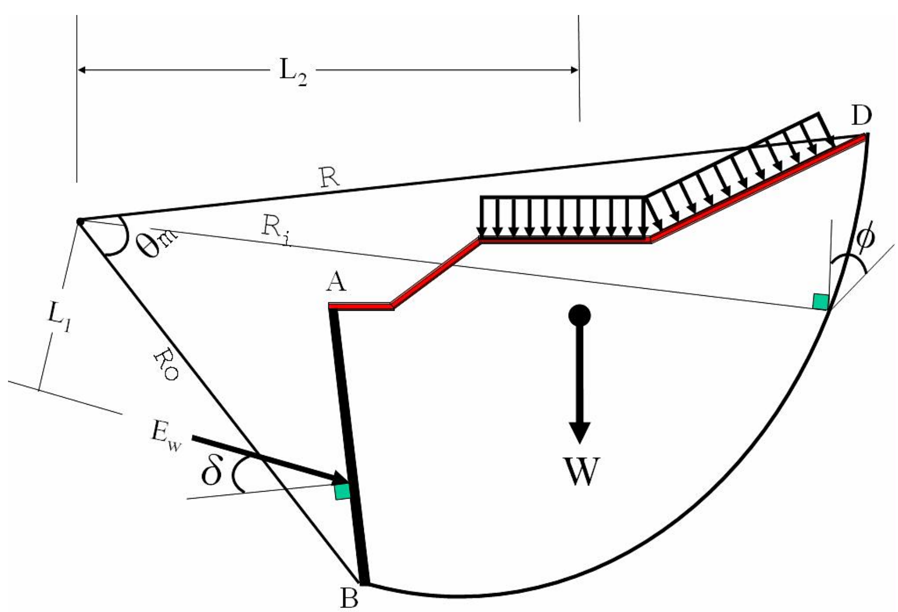


Figure A-6. Mobilized Full Log Spiral Failure Surface

A-2.02A Force Equilibrium Method

The formulation regarding the limit equilibrium method of slices (Shamsabadi, *et al*., 2007, 2005) does not change since Rankine's zone was treated as a single slice. The entire mass above the failure surface BD is divided into vertical slices and Equations A-2-10 through A-2-12 are used to calculate the magnitude of the earth pressure.

A-2.02B Moment Equilibrium Method

Only slight modifications are done to the moment limit equilibrium equations by removing the Rankine components of Equations A-2-13 and A-2-14. From Figure A-6:

$$\mathbf{E_w} = \frac{(\mathbf{W_{ABD}})(\mathbf{L_2})}{\mathbf{L_1}} \tag{A-2-18}$$

The cohesion component from Figure A-7:

$$\mathbf{E_c} = \frac{\mathbf{M_c}}{\mathbf{L_4}} \tag{A-2-19}$$

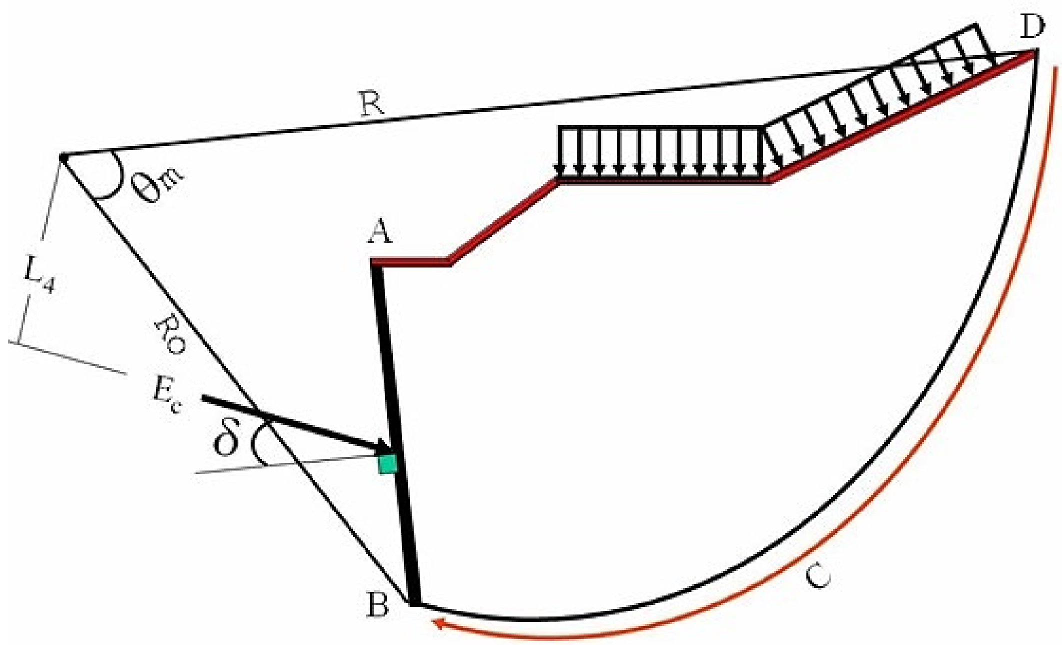


Figure A-7. Mobilized Full Log Spiral Failure Surface for Cohesion Component

The passive earth pressure coefficients for various methods are listed in the following tables for zero slope backfill (**β = 0º**). For sloping backfill, the value of $\mathbf{K_{ph}}$ should be determined by the using Figure 4-20, *Passive Earth Pressure Coefficient (Caquot and Kerisel, 1948*) from Chapter 4, *Earth Pressure Theory and Application*, and the appropriate corresponding Figure A-8 through A-14.

δ	φ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.46	3.00	3.69	4.60
5°	1.91	2.33	2.86	3.52	4.44	5.66
10°	2.07	2.59	3.26	4.13	5.31	6.96
15°	2.17	2.77	3.55	4.60	6.05	8.13
20°		2.91	3.79	5.01	6.72	9.22
25°			4.00	5.36	7.33	10.26
30°				5.68	7.88	11.27
35°					8.39	12.23
40°						13.09

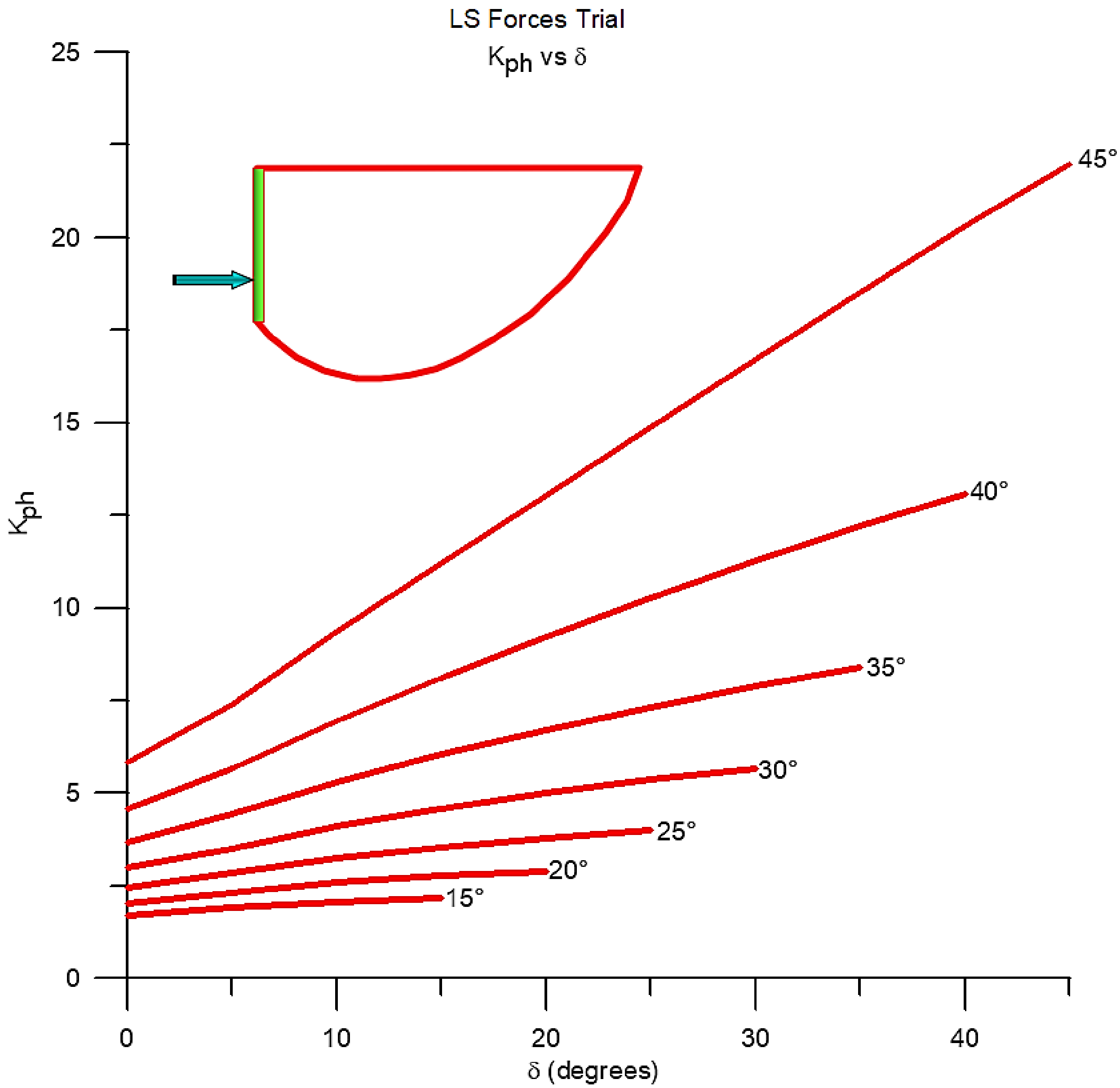


Figure A-8. Log Spiral – Forces Method – Full Log Spiral – Trial

δ	φ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.46	3.00	3.69	4.60
5°	1.92	2.36	2.90	3.61	4.54	5.81
10°	2.07	2.59	3.26	4.14	5.33	6.99
15°	2.39	2.78	3.56	4.61	6.06	8.13
20°		3.27	3.82	5.01	6.72	9.22
25°			4.56	5.42	7.35	10.27
30°				6.56	8.00	11.32
35°					9.79	12.46
40°						15.40

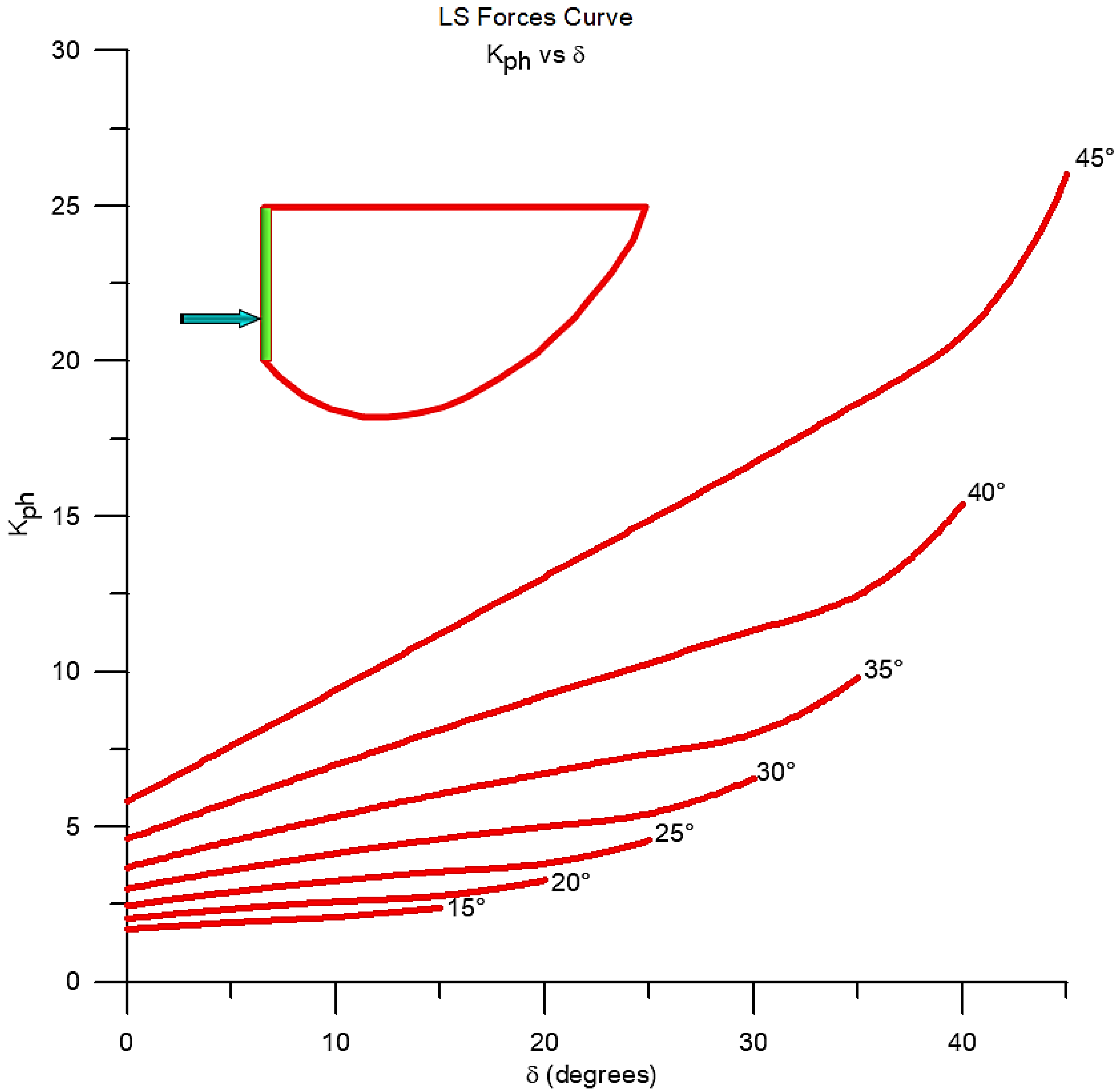


Figure A-9. Log Spiral – Forces Method – Full Log Spiral – No Trial

δ	φ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.47	3.00	3.69	4.60
5°	1.92	2.35	2.90	3.60	4.53	5.80
10°	2.05	2.57	3.24	4.11	5.29	6.94
15°	2.13	2.71	3.49	4.53	5.96	8.00
20°		2.83	3.66	4.85	6.52	8.96
25°			3.87	5.09	6.99	9.81
30°				5.45	7.37	10.56
35°					8.02	11.24
40°						12.47

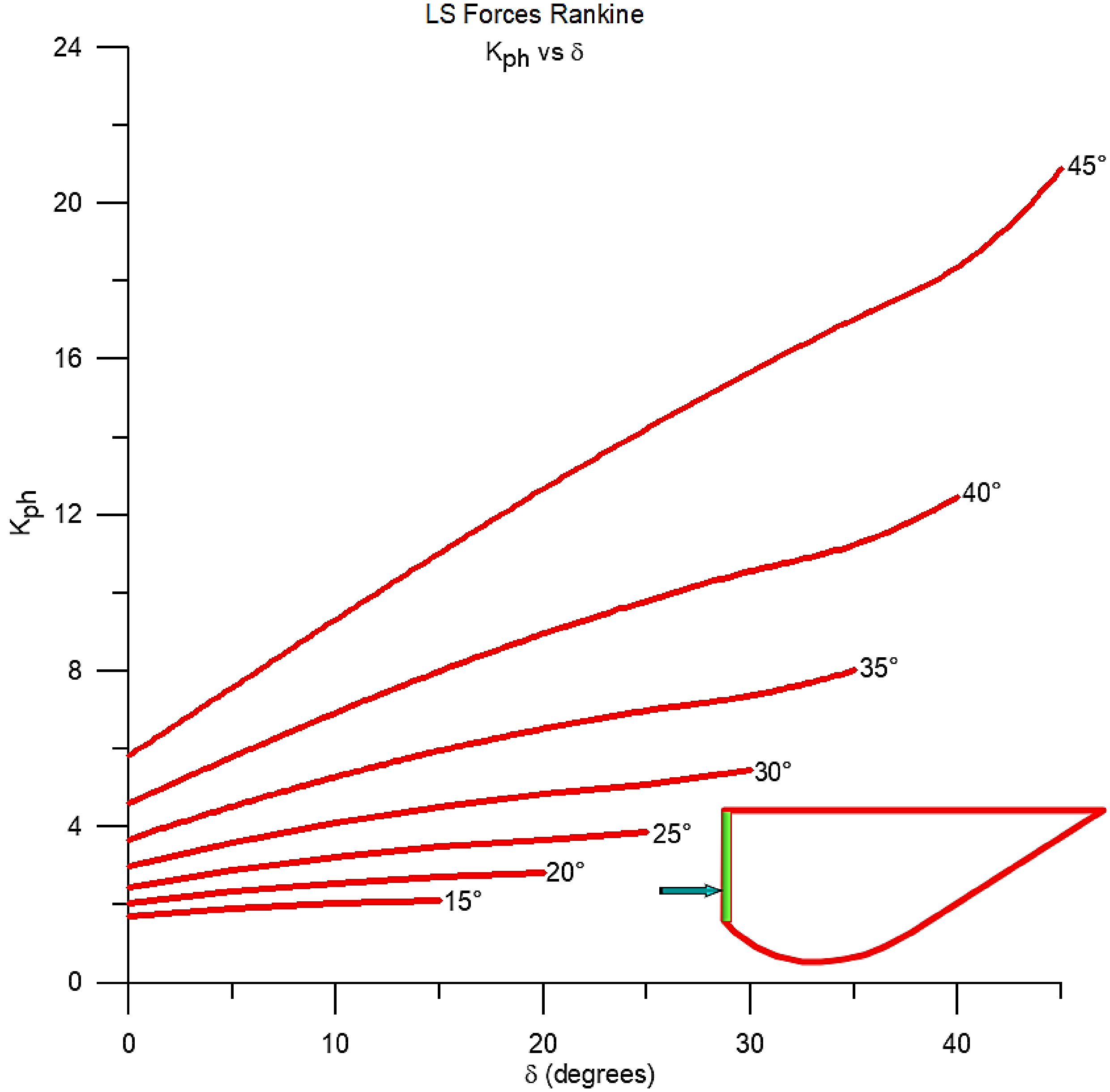


Figure A-10. Log Spiral – Forces Method – Composite Failure Surface

δ	φ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.47	3.00	3.69	4.60
5°	1.88	2.29	2.81	3.47	4.34	5.53
10°	2.03	2.52	3.14	3.96	5.06	6.60
15°	2.25	2.73	3.47	4.46	5.83	7.80
20°		3.07	3.77	4.94	6.62	9.10
25°			4.34	5.42	7.41	10.47
30°				6.38	8.21	11.87
35°					9.93	13.35
40°						16.61

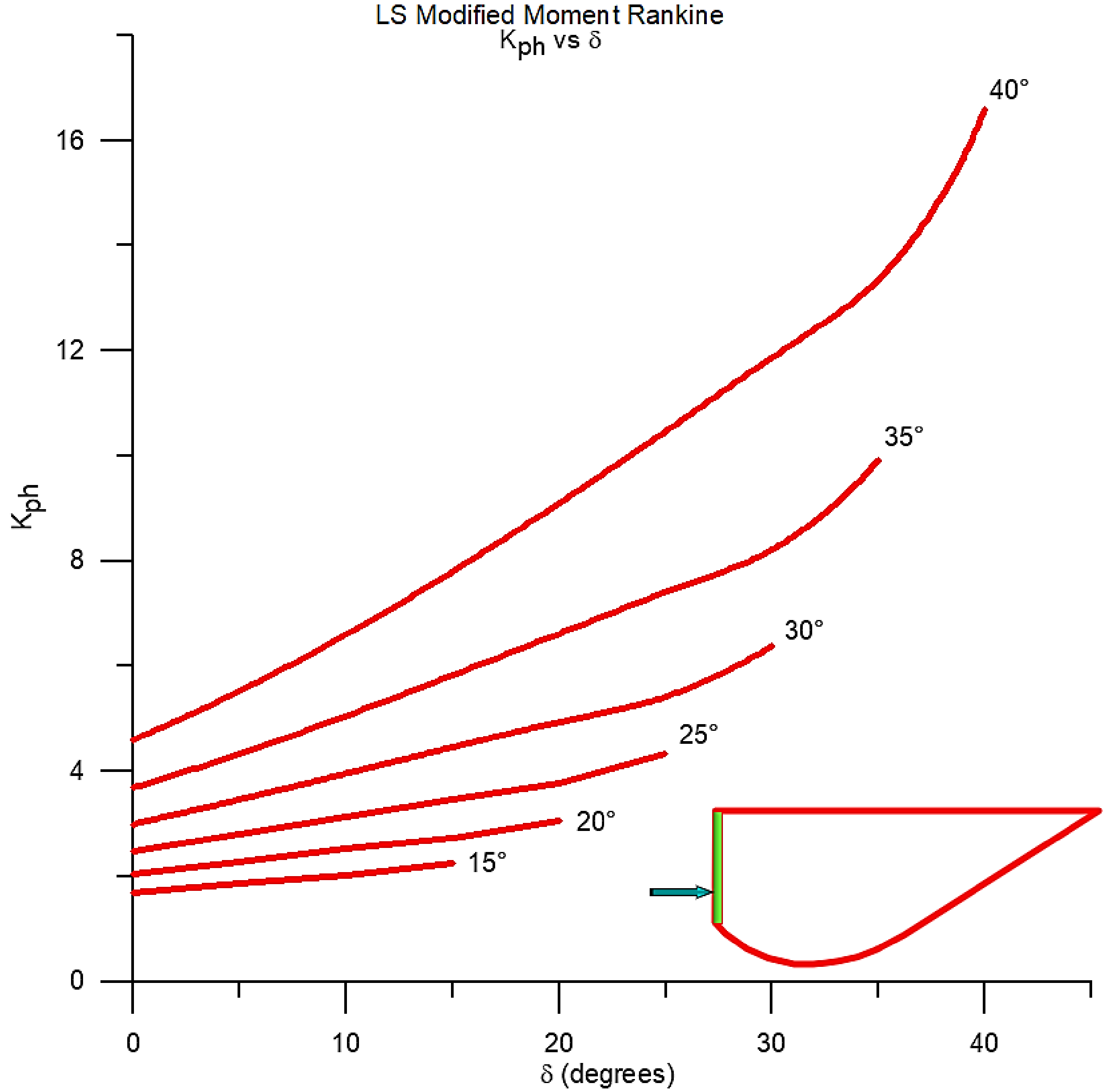
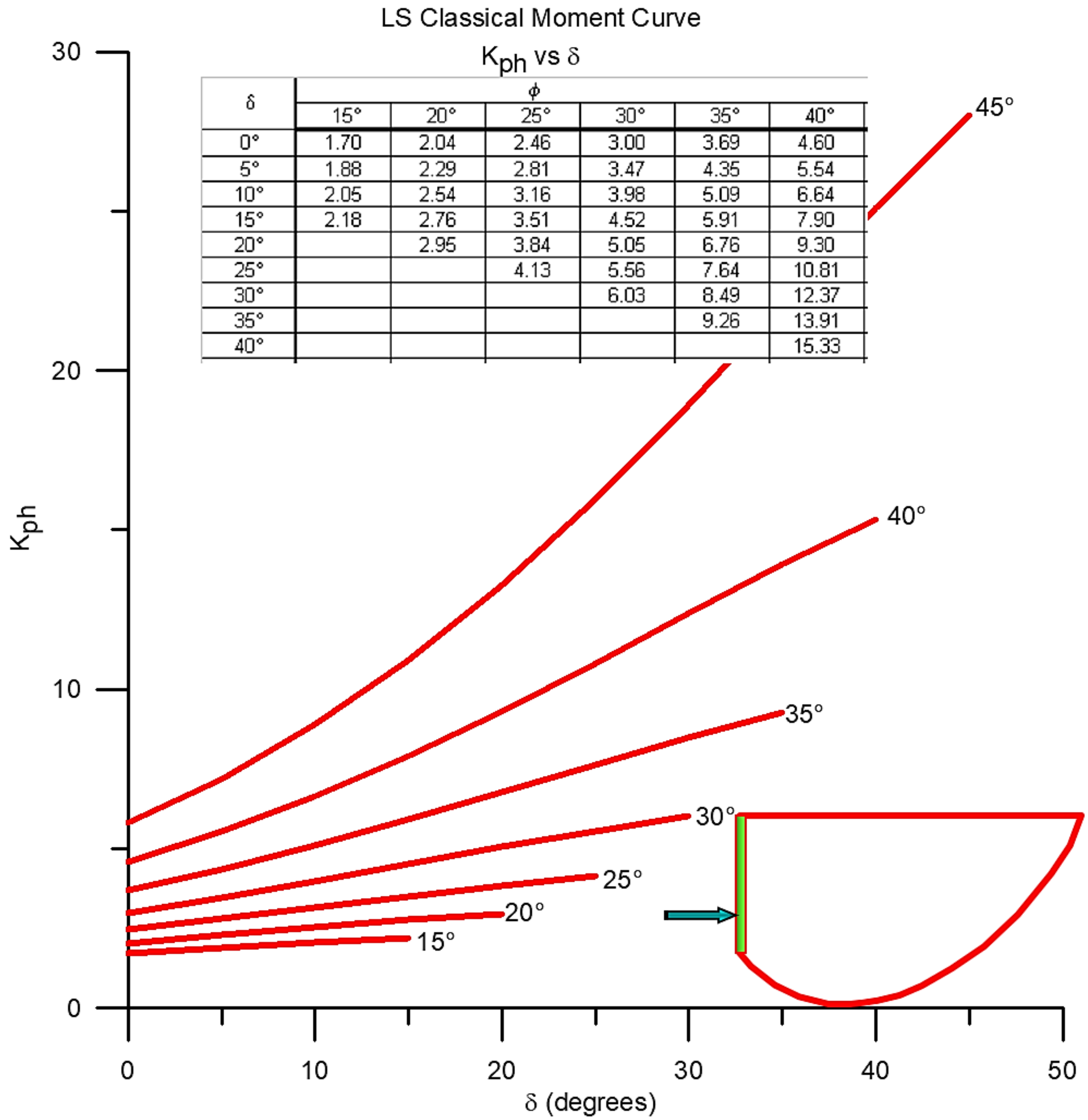


Figure A-11. Log Spiral – Modified Moment Method – Composite Failure Surface



δ	ϕ 15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.46	3.00	3.69	4.60
5°	1.88	2.29	2.81	3.47	4.35	5.54
10°	2.05	2.54	3.16	3.98	5.09	6.64
15°	2.18	2.76	3.51	4.52	5.91	7.90
20°		2.95	3.84	5.05	6.76	9.30
25°			4.13	5.56	7.64	10.81
30°				6.03	8.49	12.37
35°					9.26	13.91
40°						15.33

Figure A-12. Log Spiral – Moment Method – Full Log Spiral Failure Surface

δ	ϕ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.04	2.47	3.00	3.69	4.60
5°	1.88	2.29	2.80	3.47	4.34	5.53
10°	2.03	2.52	3.14	3.96	5.06	6.59
15°	2.15	2.72	3.47	4.46	5.83	7.78
20°		2.89	3.76	4.94	6.62	9.09
25°			4.01	5.39	7.41	10.47
30°				5.78	8.15	11.87
35°					8.82	13.24
40°						14.45

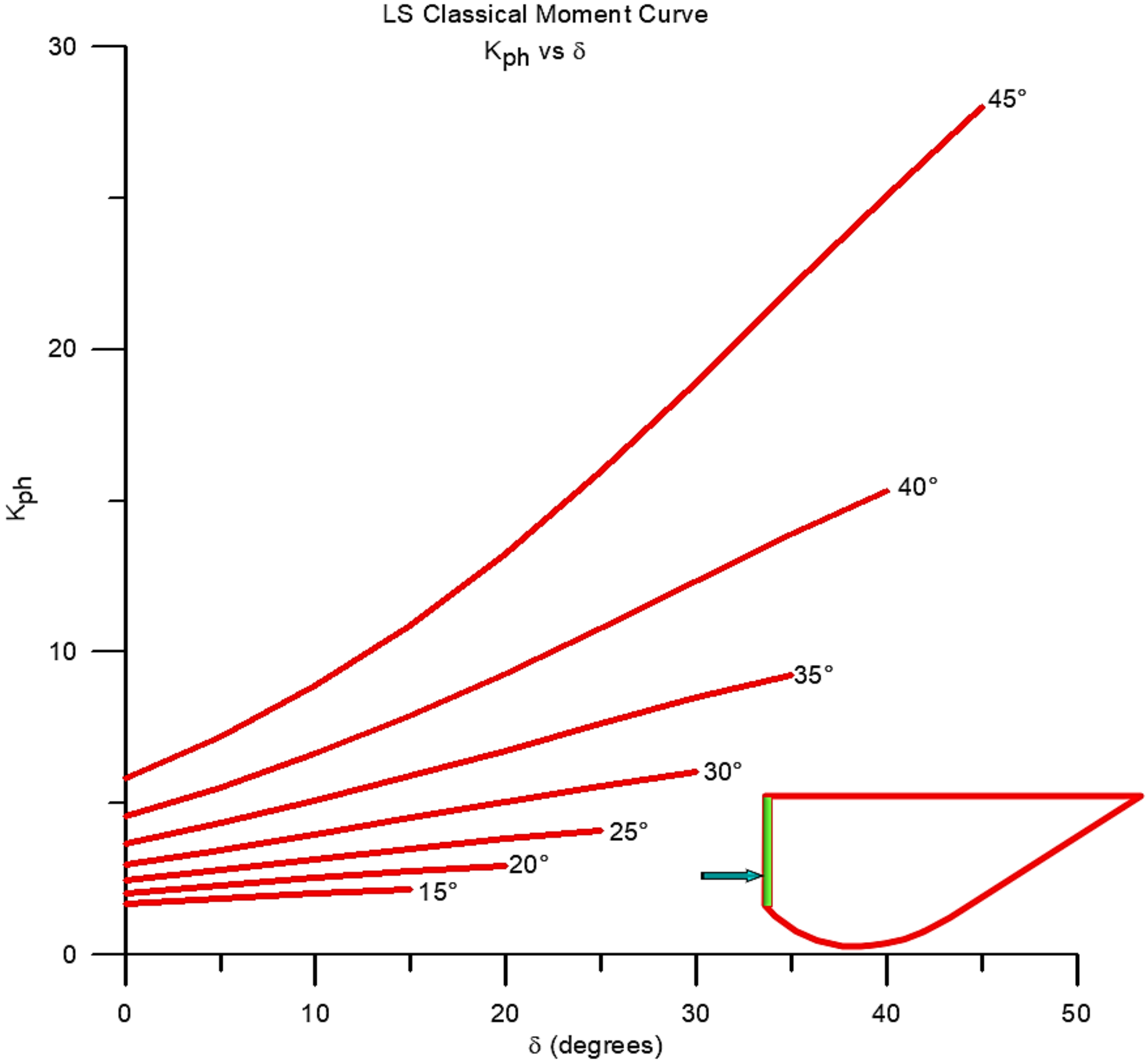


Figure A-13. Log Spiral – Moment Method – Composite Failure Surface

δ	φ					
	15°	20°	25°	30°	35°	40°
0°	1.70	2.05	2.47	3.05	3.76	4.77
5°	1.89	2.32	2.87	3.63	4.59	6.02
10°	2.09	2.60	3.27	4.22	5.48	7.41
15°	2.20	2.87	3.71	4.87	6.48	8.99
20°		3.02	4.06	5.59	7.59	10.77
25°			4.31	6.09	8.77	12.87
30°				6.53	9.58	14.91
35°					10.40	16.55
40°						18.20

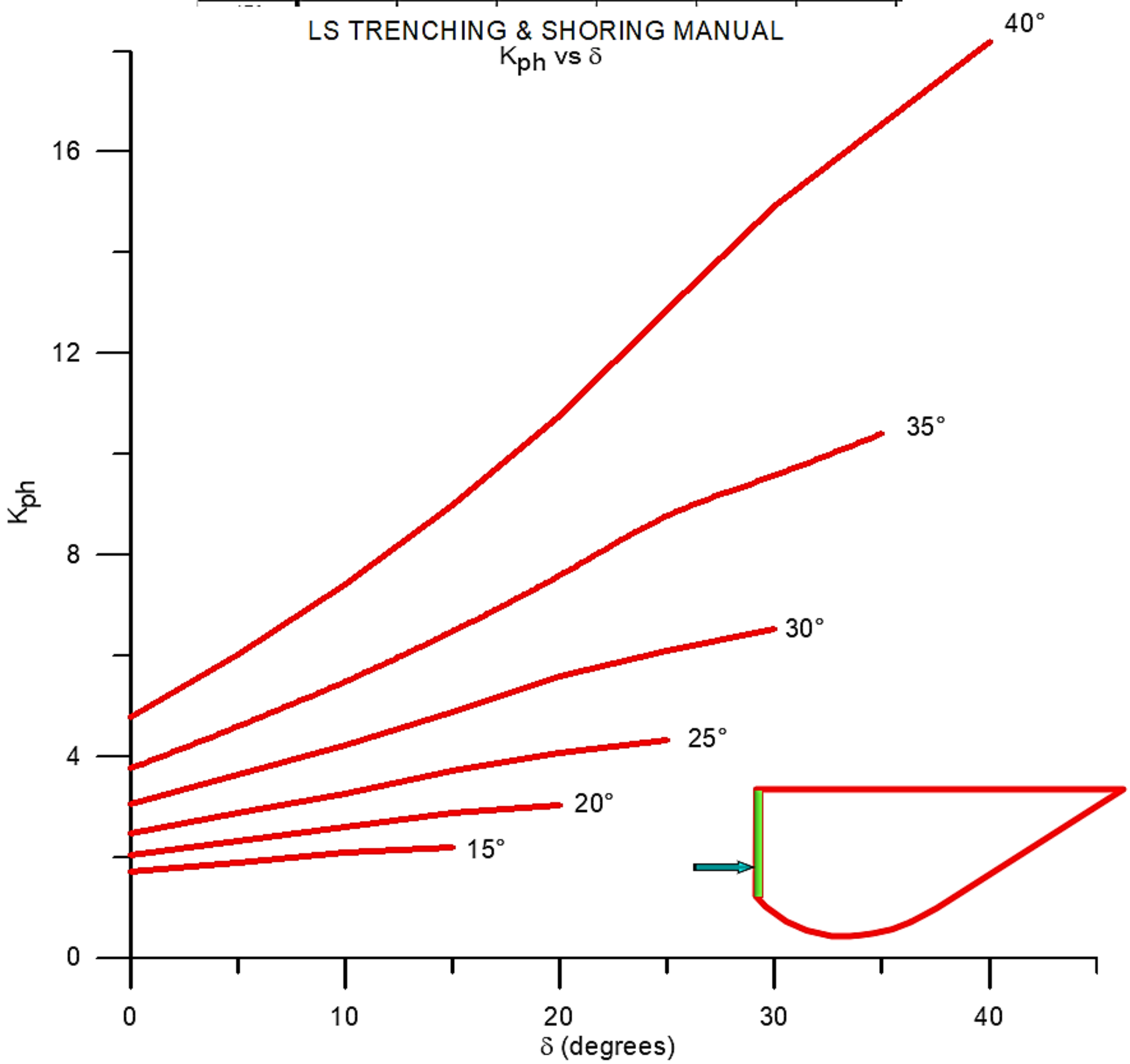


Figure A-14. Log Spiral (see Figure A-7)

Figure A-15 shows the values of $\mathbf{K_{ph}}$ computed by the various log spiral and straight-line methods described above and shown in Figure 4-3, *Comparison of Plane versus Curved Failure Surfaces* from Chapter 4*, Earth Pressure Theory and Application*. When the wall interface friction (δ) is less than about 1/3 of the backfill soil friction angle (ϕ), the value of $\mathbf{K_{ph}}$ does not differ significantly. However, for large values of wall interface friction angle (δ), the values of $\mathbf{K_{ph}}$ should be determined by the using the log spiral methods. Note that the values listed in the following table are for the purposes of comparison of the various methods with zero slope backfill (**β = 0º**).

ϕ	δ/ϕ	K_{ph}						
		Straight Line		Log Spiral				
		Rankine	Coulomb/ Trial Wedge	Forces			Moment	
				Trial	Curve	Composite	Curve	Composite
30°	0	3.00	3.00	3.00	3.00	3.00	3.00	3.00
	1/3	3.00	4.14	4.13	4.14	4.11	3.98	3.96
	2/3	3.00	6.11	5.01	5.01	4.85	5.05	4.94
	1	3.00	10.10	5.68	6.56	5.45	6.03	5.78
35°	0	3.69	3.69	3.69	3.69	3.69	3.69	3.69
	1/3	3.69	5.68	5.57	5.58	5.53	5.36	5.31
	2/3	3.69	9.96	7.13	7.14	6.84	7.35	7.15
	1	3.69	22.97	8.39	9.79	8.02	9.26	8.82
40°	0	4.60	4.60	4.60	4.60	4.60	4.60	4.60
	1/3	4.60	8.15	7.75	7.76	7.66	7.46	7.37
	2/3	4.60	18.72	10.60	10.62	10.07	11.33	10.94
	1	4.60	92.59	13.09	15.40	12.47	15.33	14.45

Figure A-15. K_{ph} Based on Straight or Curved Rupture Lines with Zero Slope Backfill ($\beta = 0º$)

A-3 Culmann's Graphical Solution for Active Earth Pressure

As presented in Section 4-5.03, of Chapter 4, *Earth Pressure Theory and Application,* Culmann (1866) developed a convenient graphical solution procedure to calculate the active earth pressure for retaining walls for irregular backfill and surcharges. Figure A-16 shows a failure wedge and a force polygon acting on a single wedge. The forces per unit width of the wall to be considered for equilibrium of the wedge are as follows:

Where:

c = Soil cohesion value

K_a = Rankine active earth pressure coefficient

ϕ = Soil friction angle

γ = Unit weight of soil

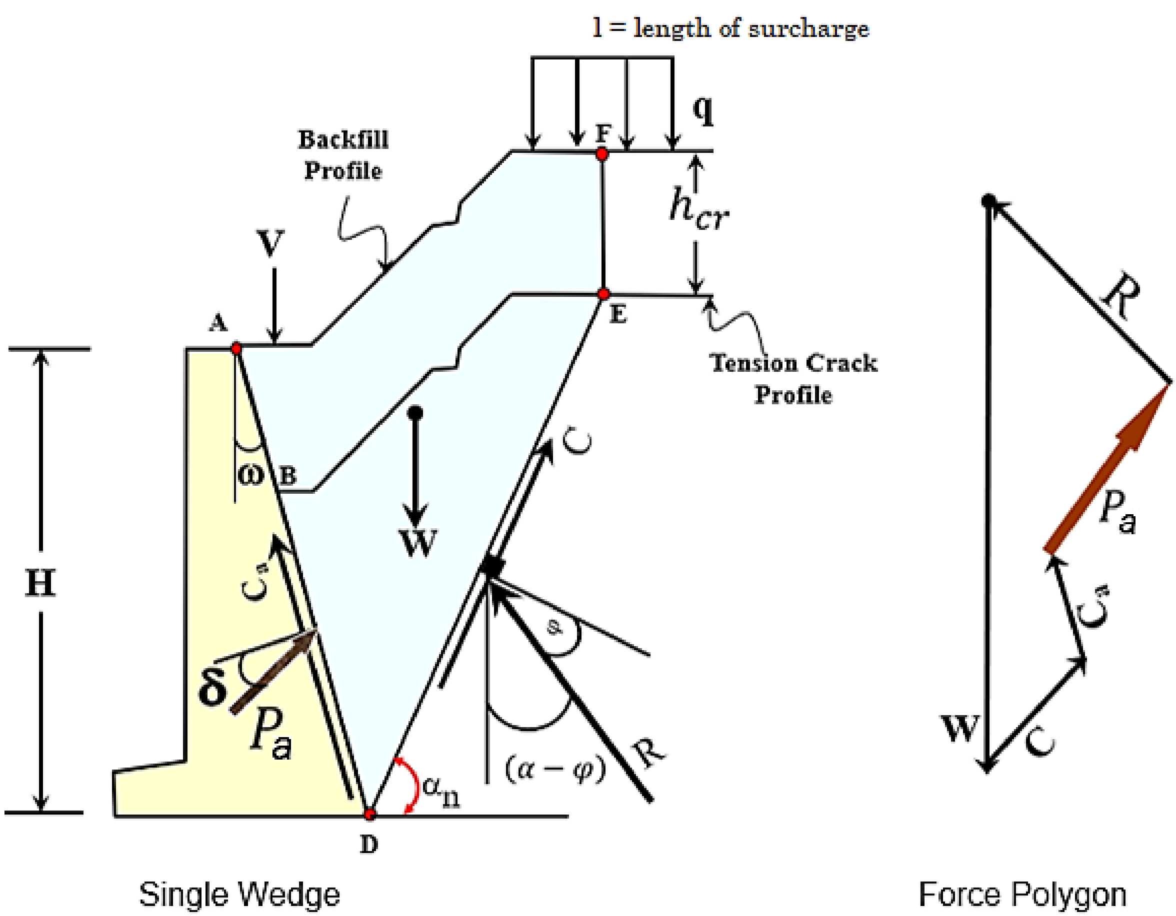


Figure A-16. Single Wedge and Force Polygon

1. **W** = Weight of the wedge including weight of the tension crack zone and the surcharges with a known direction and magnitude.

$$\mathbf{W} = \mathbf{ABDEFA_{area}}\ (\boldsymbol{\gamma}) + \mathbf{q}(\mathbf{1}) + \mathbf{V} \qquad \text{(A-3-1)}$$

2. **C_a** = Adhesive force along the backfill of the wall with a known direction and magnitude.

$$\mathbf{C_a} = \mathbf{c}(\mathbf{BD}) \qquad \text{(A-3-2)}$$

3. **C** = Cohesive force along the failure surface with a known direction and magnitude.

$$\mathbf{C} = \mathbf{c}(\mathbf{DE}) \qquad \text{(A-3-3)}$$

4. **h_{cr}** = Height of the tension crack from Equation 4-4.13.

$$\mathbf{h_{cr}} = \frac{\mathbf{2c\sqrt{K_a}}}{\boldsymbol{\gamma}\mathbf{K_a}} \qquad \text{(A-3-4)}$$

5. **R** = Resultant of the shear and normal forces acting on the failure surface DE with the direction known only.

6. **P_a** = Active force of wedge with only the direction known.

To determine the maximum active force against a retaining wall, several trial wedges must be considered and the force polygons for all the wedges must be drawn to scale.

For greater precision the analysis would include multiple wedges. At a minimum a wedge should address each discontinuity in the retained embankment. It can be broken down further still, for example at the edges or changes of surcharges. To determine the maximum active force against a retaining wall, several trial wedges must be considered and the force polygons for all the wedges must be drawn to scale as shown in Figure A-17.

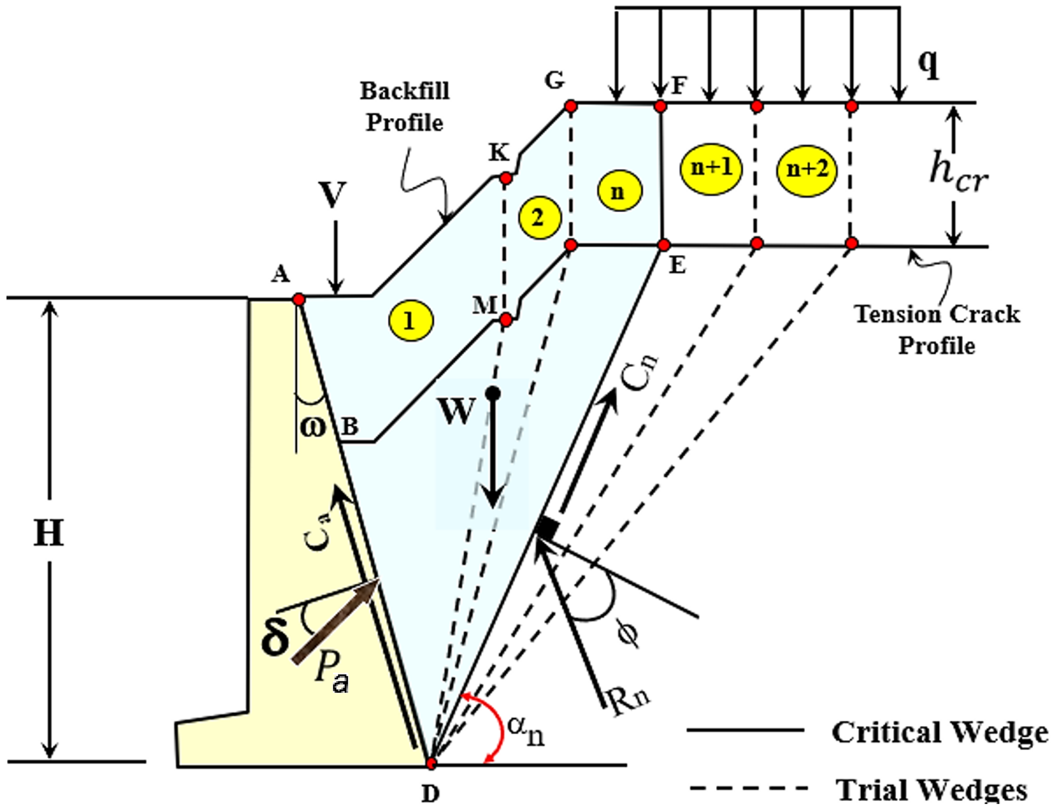


Figure A-17. Culmann Trial Wedges

The procedure for estimating the maximum active force, $\mathbf{P_a}$, as shown in Figure A-17 and Figure A-18, is described as follows:

1. Draw the lines for the tensile crack profile parallel to the backfill profile with height equal to $\mathbf{h_{cr}}$.
2. Draw several trial wedges to intersect the tension crack profile line.
3. Draw the vectors to represent the weight of wedges per unit width of the wall including the surcharges.
4. Draw adhesion force vector $\mathbf{C_a}$ acting along the face of the wall.
5. Draw cohesion force vector $\mathbf{C_n}$ acting along the failure surfaces.
6. Draw the active force vector $\mathbf{P_a}$.
7. Draw the resultant force vector **R** acting on the failure place.
8. Repeat steps 2 through 7 until all trial wedges are complete.
9. Draw a smooth curve through these points as shown in Figure A-18. A cubic spline function is used in CT-Flex computer program to draw the smooth line between point $\mathbf{P_1}$ through point $\mathbf{P_{n+2}}$.
10. Draw dashed line TT' through the left end of force vectors $\mathbf{P_a}$, as shown in Figure A-18.

11. Draw a parallel line to line TT' that is tangent to the above curve to measure maximum active earth pressure length as shown in Figure A-18.
12. Draw a line parallel to the force vectors $\mathbf{P_a}$ that begins at TT' and ends at the intersection point of the tangent line to the curved line above. This is the maximum active pressure force vector $\mathbf{P_{a,max}}$.

The maximum active pressure shown in Figure A-17 and Figure A-18 is obtained as:

$$\mathbf{P}_{\boldsymbol{a}} = (\text{length of } \mathbf{L}) \ \mathbf{x} \ (\text{load scale } \boldsymbol{\lambda}) \qquad \text{(A-3-5)}$$

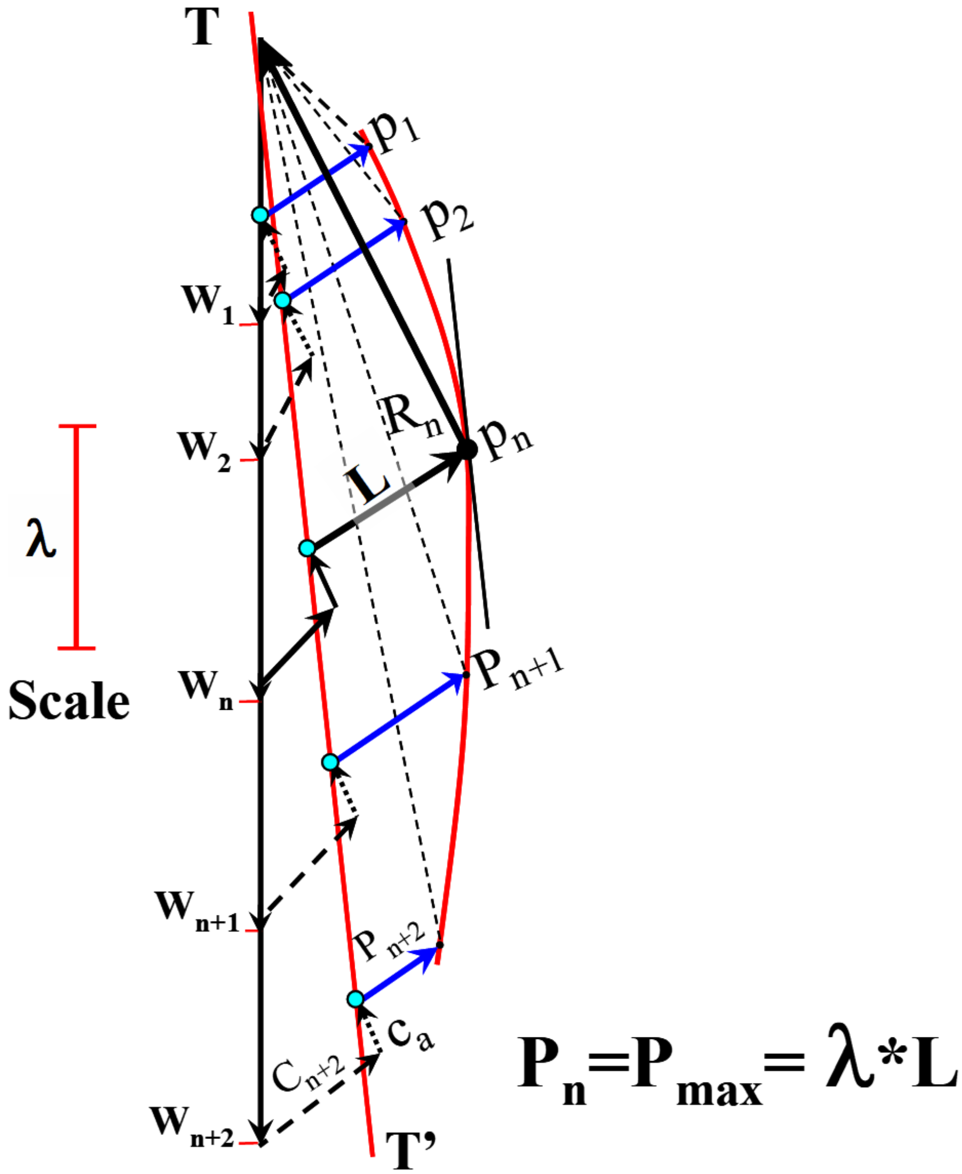


Figure A-18. Culmann Graphical Solution to Scale

The following sections of this appendix contain six example problems. These example problems can be described as follows:

1. Example A-1 has a gravity retaining wall, with both adhesion and cohesion forces. The solution is found using the Culmann Graphical Solution (Trial Wedge Method).
2. Example A-2 has the same gravity retaining wall that is shown in Example A-1. The solution is found using the Caltrans Trenching and Shoring Check Program to obtain the Critical Active Wedge.
3. Example A-3 has a cantilever retaining wall that is the cantilever retaining wall in the 2023 American Railway Engineering and Maintenance-of-Way Association (*AREMA) Manual for Railway Engineering* (Page 8-5-20, Figure C-8-5-4). This example has no adhesion component. The solution is found using the Culmann Graphical Solution (Trial Wedge Method).
4. Example A-4 has the same cantilever retaining wall that is shown in Example A-3. The solution is found using the Caltrans Trenching and Shoring Check Program to obtain the Critical Active Wedge.
5. Example A-5 has a gravity retaining wall that is shown in the 2023 *AREMA Manual for Railway Engineering* (Page 8-5-21, Figure C-8-5-5). This example has no adhesion component. The solution is found using the Culmann Graphical Solution (Trial Wedge Method).
6. Example A-6 has the same gravity retaining wall that is shown in Example A-5. The solution is found using the Caltrans Trenching and Shoring Check Program to obtain the Critical Active Wedge.

A-3.01 Example A-1: Culmann Graphical Method

Calculate the maximum active earth pressure using the Culmann graphical method for a gravity retaining wall given in Figure A-19 using the following backfill properties. (Note that this is not a cantilever retaining wall). Also, please note that in these examples, the units of kips are sometimes displayed as **k**, as in Figure A-20 and Equation A-3-8.

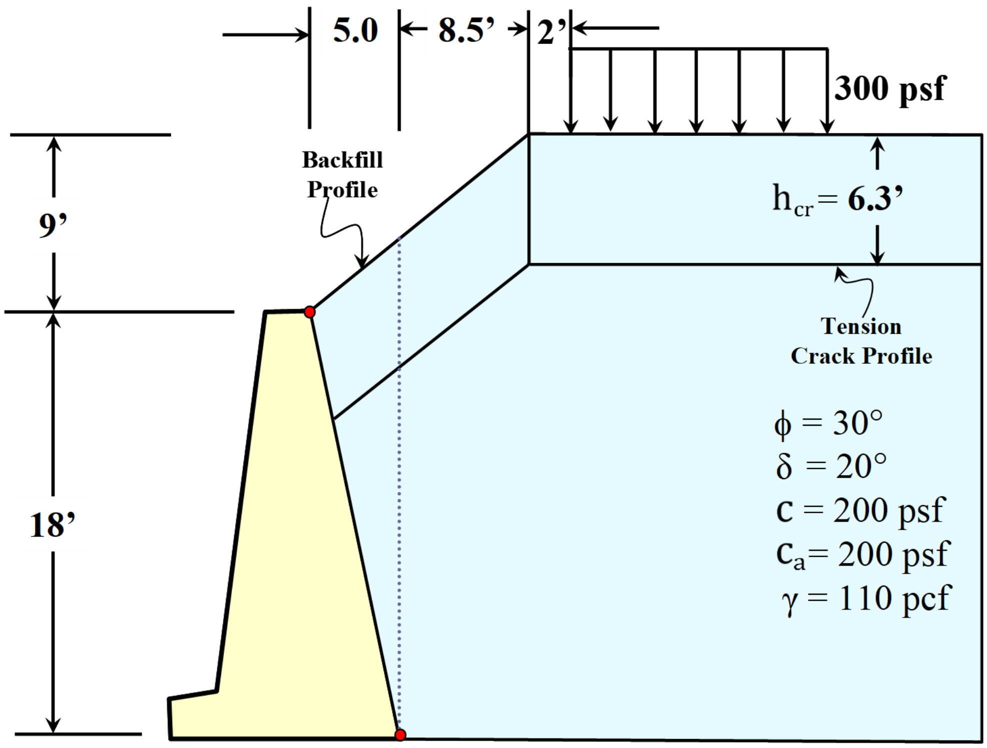
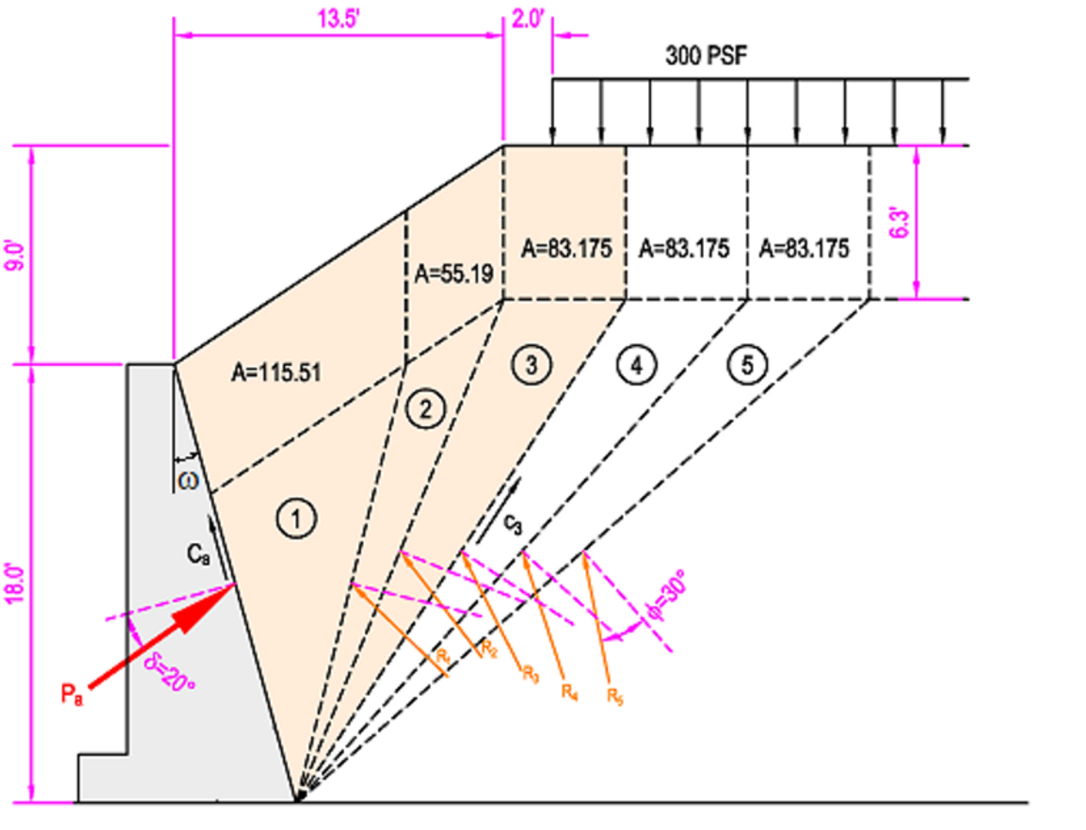


Figure A-19. Retaining Wall with Irregular Backfill by Culmann Method



Geometry			Weight Components					Cohesion Comp.	
Wedge	Backfill Profile Coordinates		Wt (k)	Σ Wt (k)	Sur. Wt (k)	Σ Sur. (k)	Total Wt (k)	Length (ft)	Coh (k)
	X	Y							
1	4.51	18.04	12.72	12.72	0	0	12.72	18.6	3.72
2	8.51	20.70	6.09	18.81	0	0	18.81	22.38	4.48
3	13.52	20.70	9.17	27.98	0.91	0.91	28.89	24.72	4.94
4	18.53	20.70	9.17	37.15	1.50	2.41	39.56	27.78	5.56
5	23.54	20.70	9.18	46.33	1.50	3.91	50.24	31.34	6.27

Figure A-20. Culmann Trial Wedge Method to Scale and Graphical Method Results

Solution:

As shown in Figure A-20, several trial wedges are drawn. The weight of each of these wedges, the adhesive force at the wall interface and cohesive force along the failure surface are computed as is shown below.

The active earth pressure due to soil-wall interaction is constant for all wedges and is calculated as shown below.

Determine wall angle:

$$\omega = \tan^{-1}\left(\frac{5'}{18'}\right) = 15.52° \quad \text{(A-3-6)}$$

$$L_a = \frac{(18 - 6.3)}{\cos(15.52)} = 12.14 \text{ ft} \quad \text{(A-3-7)}$$

$$C_a = c_a L_a = (0.2)(12.14) = 2.43 \text{ k/ft} \quad \text{(A-3-8)}$$

Where **L_a** is the length of the active wedge along the backwall and **C_a** is the active earth pressure due to wall-backfill adhesion properties.

The force polygon for all the wedges and maximum active force using scaling factors are shown in Figure A-21. The maximum earth pressure is about 8.5 kips/ft.

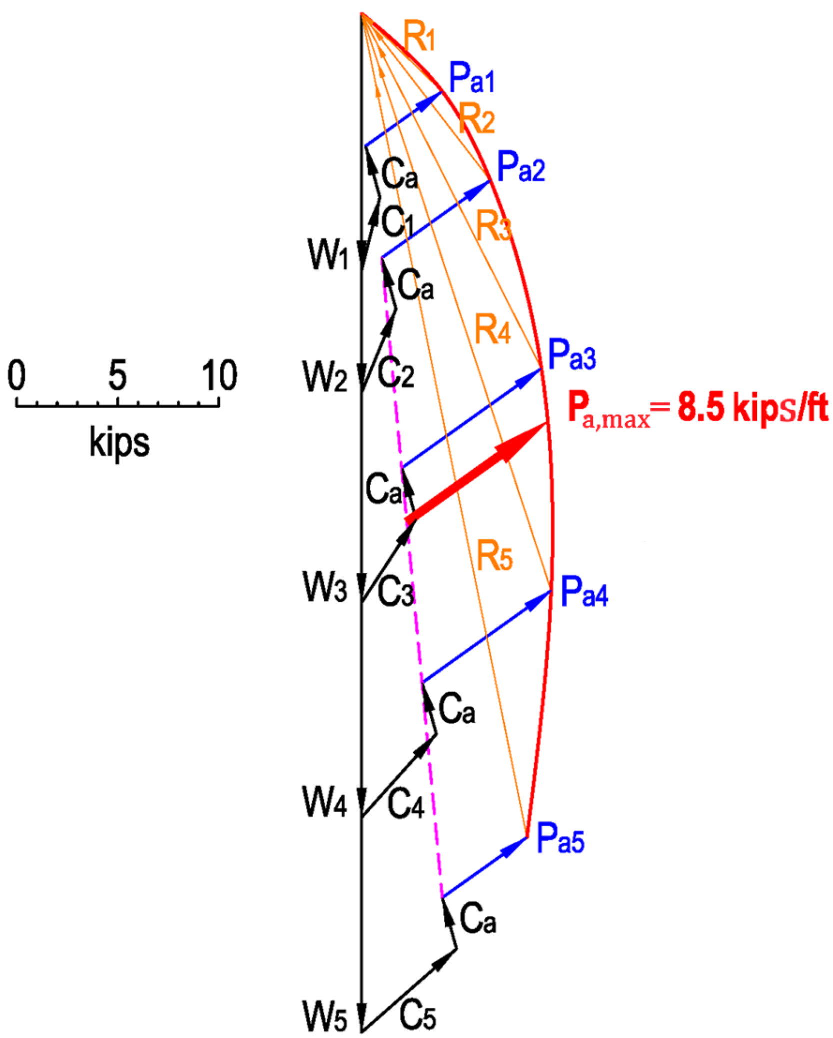


Figure A-21. Culmann Graphical Solution Using Force Polygon

A-3.02 Example A-2: Critical Active Wedge Method

Repeat Example A-1 Culmann Graphical Method, (gravity retaining wall) using ~~trial~~ critical active wedge method. Figure A-22 illustrates the most critical failure surface developed using the *Caltrans Trenching and Shoring Check Program.*

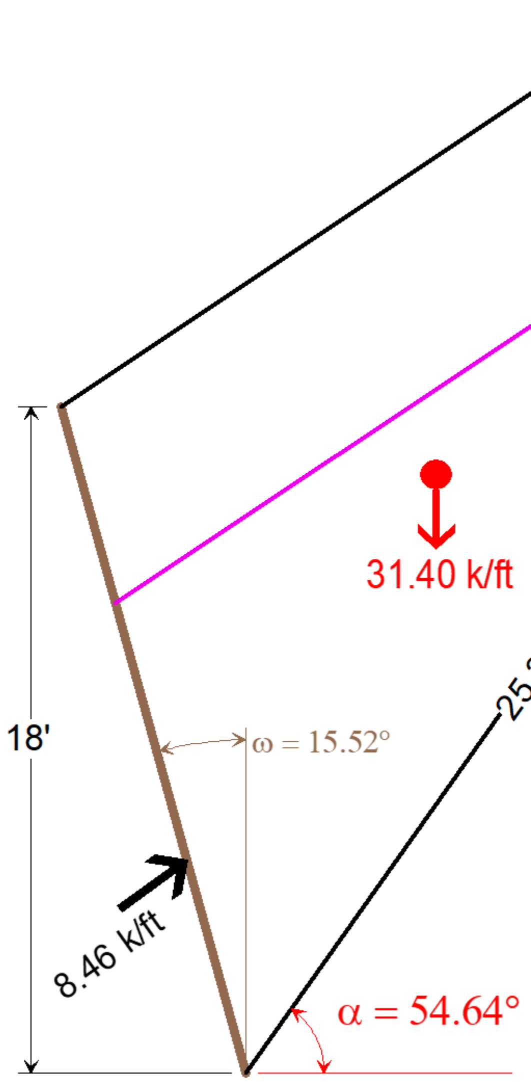


Figure A-22. Critical Active Wedge

Use Equation A-3-9 (from Section 4-5.01, *Active Trial Wedge Method*) to calculate the active earth pressure.

$$P_a = \frac{W[\tan(\alpha - \phi)] - C_0 L_c[\sin\alpha \tan(\alpha - \phi) + \cos\ \alpha] - C_a L_a[\tan(\alpha - \phi)\cos(-\omega) + \sin\ \omega]}{[1 + \tan(\delta + \omega)\tan(\alpha - \phi)]\cos(\delta + \omega)}$$

(A-3-9)

Caltrans Trenching and Shoring Check Program is used to calculate the failure plane angle (α) and the length of the critical failure surface ($\mathbf{L_c}$).

$$\mathbf{P_a = \frac{WT - COH - ADH}{[1 + \tan(\delta + \omega)\tan(\alpha - \phi)]\cos(\delta + \omega)}} \quad \text{(A-3-10)}$$

Calculate the weight contribution from the weight of the wedge and the weight of the surcharge (WT):

$$\mathrm{WT} = \mathrm{W}[\tan(\alpha - \phi)] = (31.40)[\tan(54.64 - 30)] = 14.40\ \mathrm{k/ft} \quad \text{(A-3-11)}$$

Calculate adhesion (ADH) component.

$$\mathrm{L_a} = \frac{(18 - 6.3)}{\cos(15.52)} = 12.14\ \mathrm{ft.} \quad \text{(A-3-12)}$$

$$\begin{aligned}\mathrm{ADH} &= \mathrm{C_a L_a}[\tan(\alpha - \phi)\cos(-\omega) + \sin(-\omega)] \\ &= (0.2)(12.14)[\tan(54.64 - 30)\cos(-15.52) + \sin(-15.52)] = 0.422\ \mathrm{k/ft}\end{aligned} \quad \text{(A-3-13)}$$

Calculate cohesion (COH) component.

$$\begin{aligned}\mathrm{COH} &= \mathrm{C_O L_c}[\sin\alpha\tan(\alpha - \phi) + \cos\ \alpha] \\ &= (0.2)(25.39)[\sin(54.64)\tan(54.64 - 30) + \cos(54.64)] = 4.837\ \mathrm{k/ft}\end{aligned} \quad \text{(A-3-14)}$$

Substitute WT, COH, and ADH into Equation A-3-10.

$$\mathrm{P_a} = \frac{14.40 - 4.84 - .422}{[1 + \tan(20 + 15.52)\tan(54.64 - 30)]\cos(20 + 15.52)} = 8.46\ \mathrm{k/ft} \quad \text{(A-3-15)}$$

In Example A-1, the calculated value was 8.5 kips/ft. This value of 8.46 kips/ft is slightly different than the value of 8.5 kips/ft due to the graphical nature of the calculation.

A-3.03 Example A-3 (2023 AREMA Manual Page 8-5-20)

The following example is taken from the 2023 *AREMA Manual for Railway Engineering* (Page 8-5-20; note that the 2023 manual references 2002 throughout). This is a cantilever retaining wall. In the AREMA Manual, this retaining wall is called a retaining wall with a heel. The vertical surcharge pressure is 500 psf and acts over a width of 14 feet; there is no adhesion component in this example.

Calculate the maximum active earth pressure using the **Culmann graphical method** for a retaining wall with a heel (earth pressure at line AB) given in Figure A-23 (AREMA page 8-5-12).

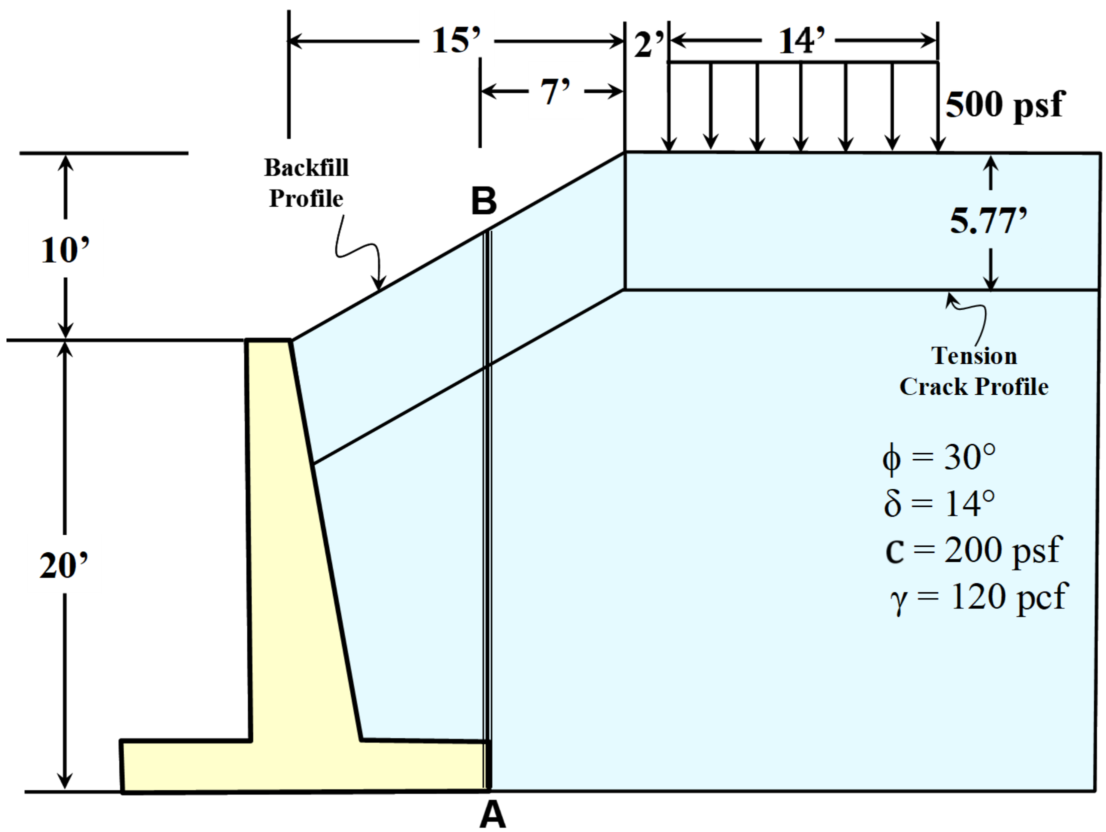
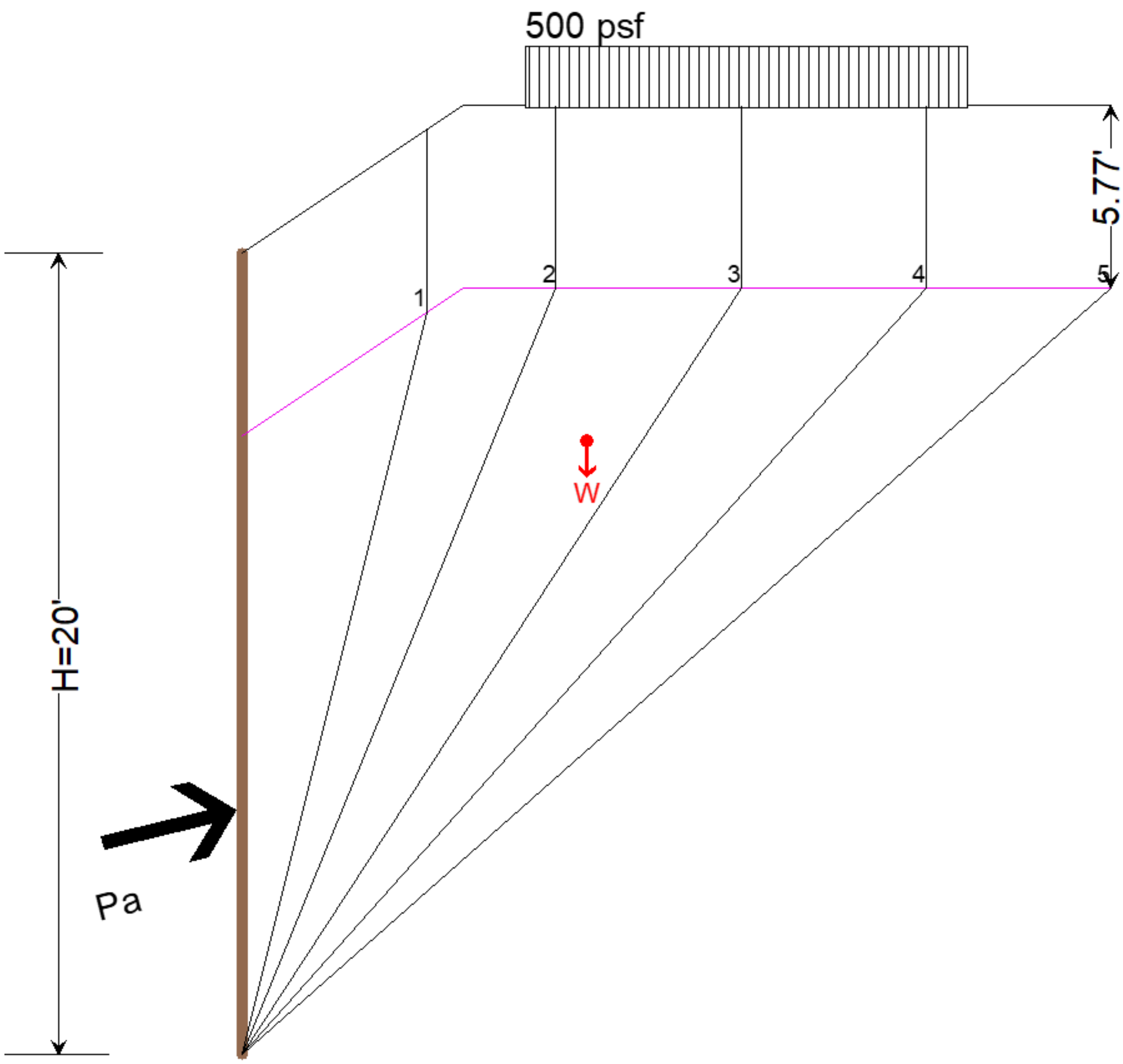


Figure A-23. Retaining Wall by Culmann Method (AREMA Manual page 8-5-20)

As shown in Figure A-24 several trial wedges are drawn. The weight of each of these wedges, the adhesive force at the wall interface and cohesive force along the failure surface are computed as is shown below.



Geometry			Weight Components					Cohesion Comp.	
Wedge	Backfill Profile Coordinates		Wt (k)	Σ Wt (k)	Sur. Wt (k)	Σ Sur. (k)	Total Wt (k)	Length (ft)	Coh (k)
	X	Y							
1	5.87	23.48	10.95	10.95	0	0	10.95	24.20	4.84
2	9.96	24.23	8.47	19.42	0.48	0.48	19.90	26.20	5.24
3	15.83	24.23	12.58	32.00	2.93	3.41	35.41	28.94	5.79
4	21.69	24.23	12.58	44.58	2.93	6.34	50.92	32.52	6.50
5	27.55	24.23	12.58	57.16	0.66	7.00	64.16	36.69	7.34

Figure A-24. Culmann Trial Wedge and Trial Wedge Method Results

The force polygon for all the wedges and maximum active force using scaling factors are shown in Figure A-25. The maximum earth pressure is about 11.35 kips/ft. In the 2023 *AREMA Manual for Railway Engineering*, the value was calculated as 11.1 kips/ft. This value of 11.35 kips/ft is slightly different than the value of 11.1 kips/ft due to the graphical nature of the calculation.

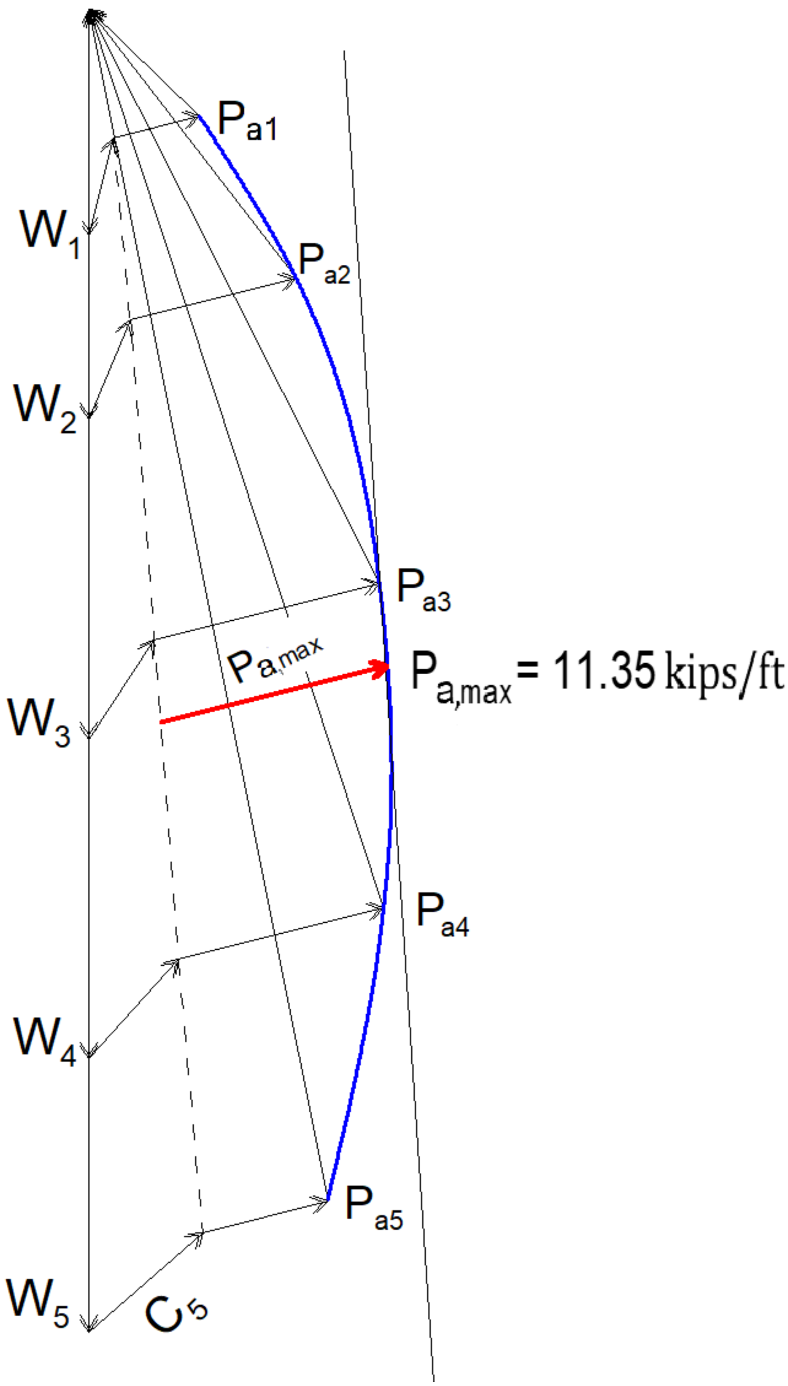


Figure A-25. Culmann Graphical Solution Using Force Polygon

A-3.04 Example A-4 (2023 AREMA Manual Page 8-5-20)

The following example is taken from the 2023 *AREMA Manual for Railway Engineering*

Repeat Example A-3, a cantilever retaining wall (AREMA Manual, page 8-5-20), using the **critical active wedge method**. Figure A-26 illustrates the most critical failure surface developed using the *Caltrans Trenching & Shoring Check Program*.

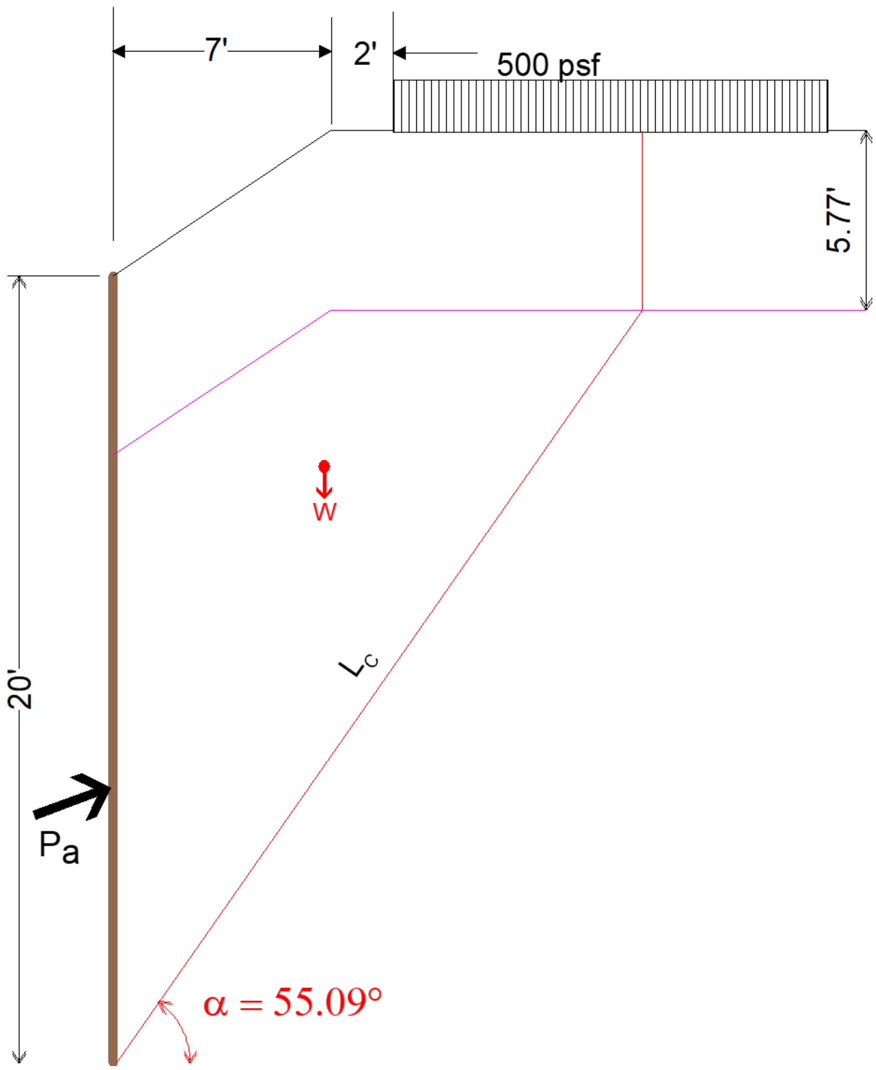


Figure A-26. Critical Active Wedge Method

Equation A-3-16, without the adhesion component, is used to calculate the active earth pressure (**P_a**). *Caltrans Trenching & Shoring Check Program* is used to calculate the failure plane angle (α) and the length of the critical failure surface (**L_c**).

$$\mathbf{P_a} = \frac{\mathbf{W[tan\,(\alpha - \phi)] - C_O L_a[sin\,\alpha\;\;tan\,(\alpha - \phi) + cos\;\;\alpha]}}{\mathbf{[1 + tan\,\delta\,tan\,(\alpha - \phi)]cos\;\;\delta}} \quad \text{(A-3-16)}$$

or

$$\mathbf{P_a} = \frac{\mathbf{WT - COH}}{\mathbf{[1 + tan\,\delta\,tan\,(\alpha - \phi)]cos\;\;\delta}} \quad \text{(A-3-17)}$$

Calculate the weight contribution from the weight of the wedge and the weight of the surcharge (WT):

$$\mathrm{WT} = \mathrm{W}[\tan(\alpha - \phi)] = (38.31)[\tan(55.09 - 30)] = 17.92\ \mathrm{k/ft} \quad \text{(A-3-18)}$$

Calculate cohesion (COH) component.

$$\begin{aligned}\mathrm{COH} &= \mathrm{C_O L_c}[\sin\alpha\tan\,(\alpha - \phi) + \cos\;\;\alpha] \\ &= (0.2)(29.55)[\tan(55.09 - 30)\sin(55.09) + \cos(55.09)] = 5.65\ \mathrm{k/ft}\end{aligned}$$

(A-3-19)

Substitute WT and COH into Equation A-3-17.

$$\mathrm{P_a} = \frac{(17.92 - 5.65)}{[1 + \tan\,(14)\tan\,(55.09 - 30)]\cos\,(14)} = 11.32\ \mathrm{k/ft} \quad \text{(A-3-20)}$$

In Example A-3 (Culmann Graphical Method), the calculated value was 11.35 kips/ft, which is very close to this value of 11.32 kips/ft (Trial Wedge Method).

A-3.05 Example A-5 (2023 AREMA Manual Page 8-5-21)

Calculate the maximum active earth pressure using the Culmann graphical method for a gravity retaining wall given in Figure A-27 using the backfill properties as shown; this example has no adhesion component. Note that this is not a cantilever retaining wall; the AREMA Manual refers to it as a retaining wall without a heel. A snapshot of page 8-5-21 is provided in Figure A-29.

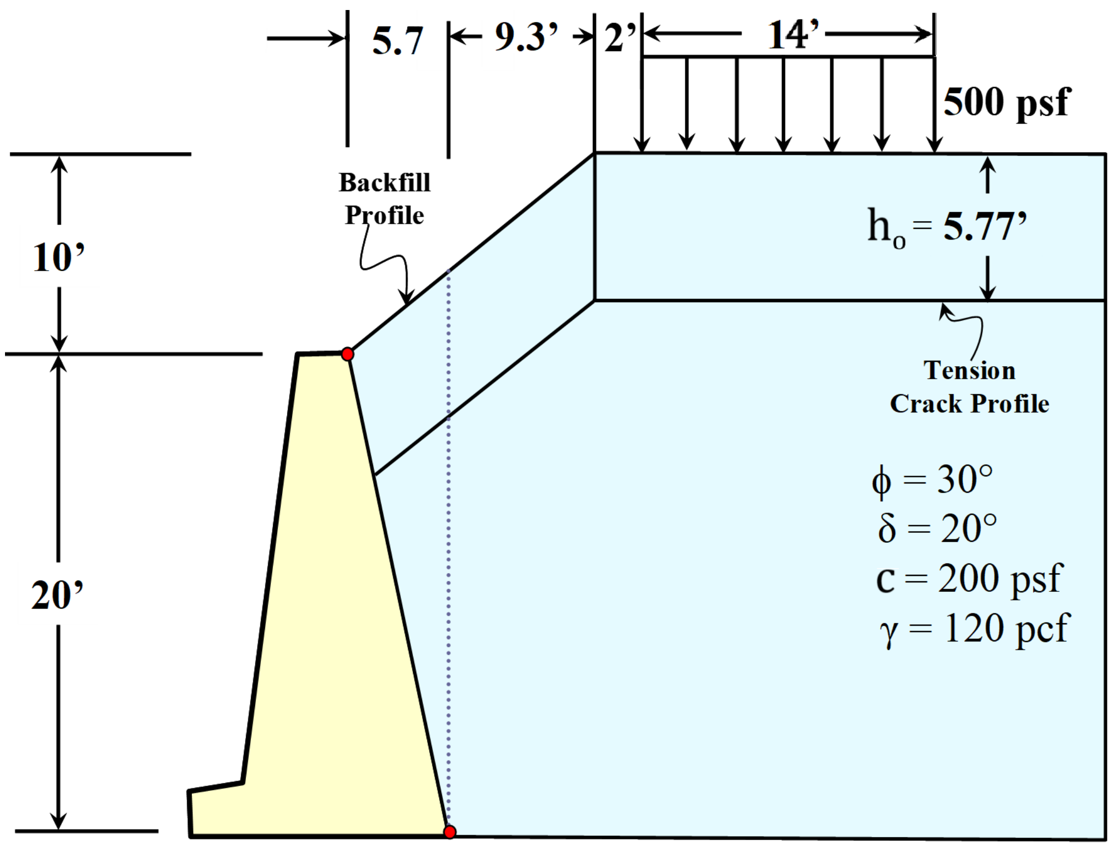
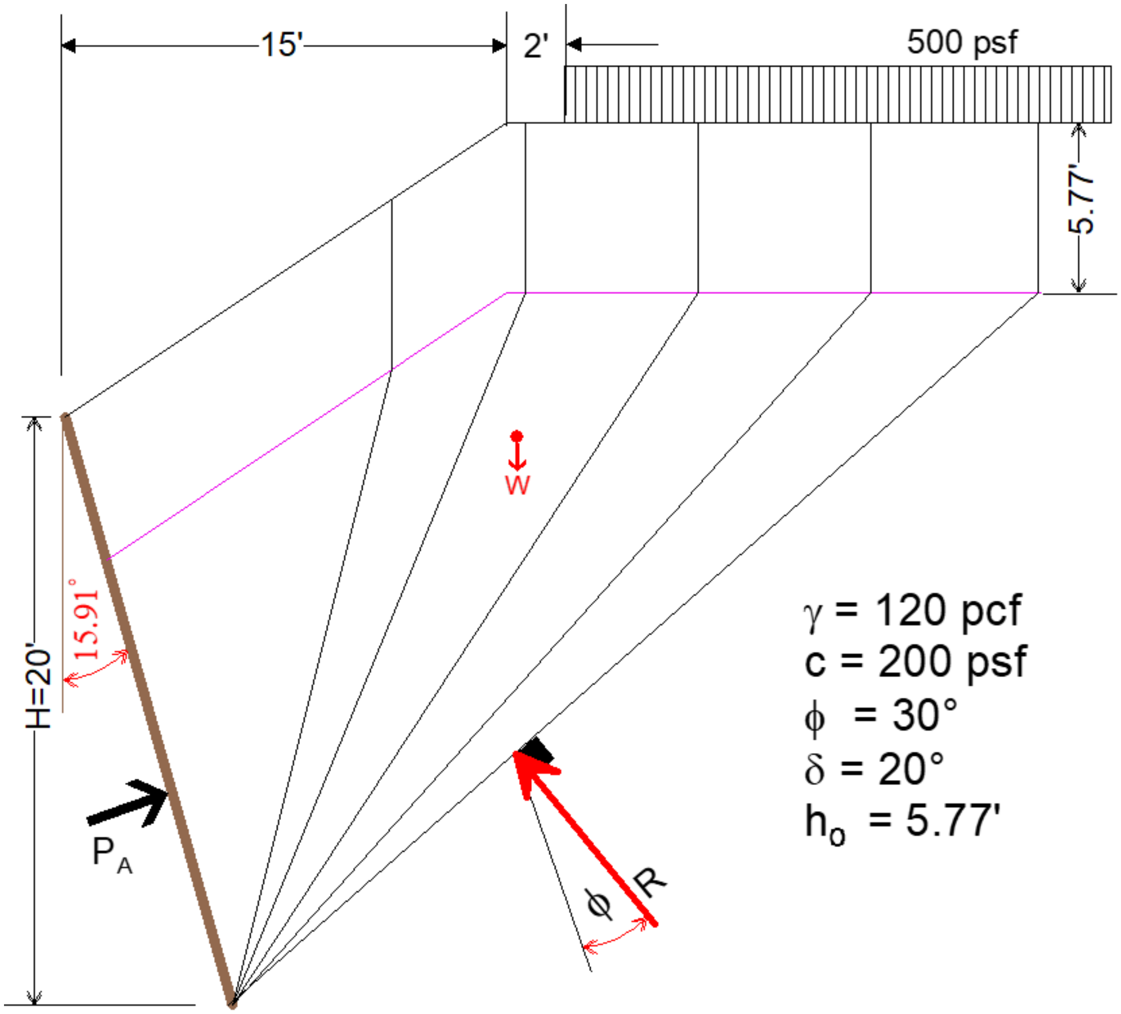
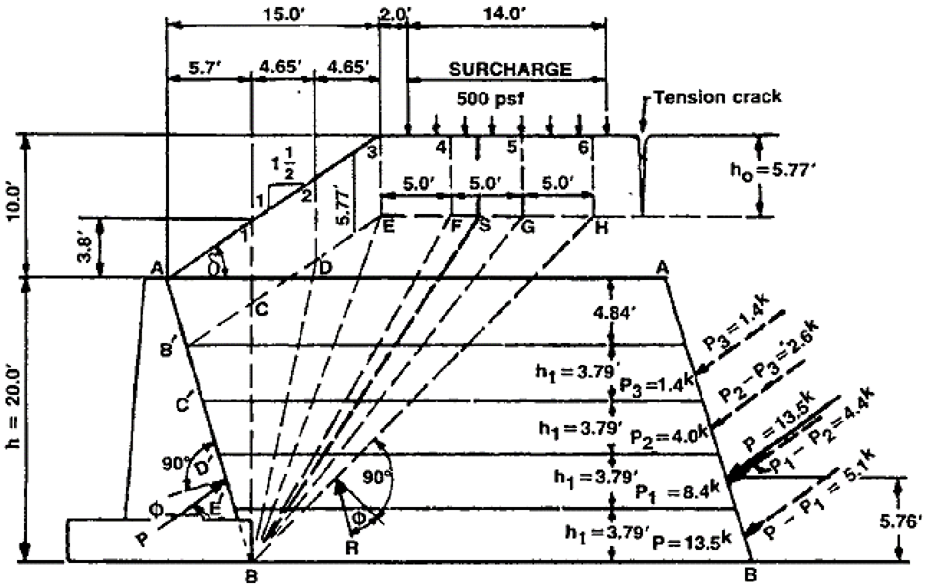


Figure A-27. Gravity Retaining Wall with Irregular Backfill by Culmann Method



Geometry			Weight Components					Cohesion Comp.	
Wedge	Backfill Profile Coordinates X	Backfill Profile Coordinates Y	Wt *(k)*	Σ Wt *(k)*	Sur. Wt *(k)*	Σ Sur. *(k)*	Total Wt *(k)*	Length *(ft)*	Coh *(k)*
1	5.41	21.64	17.74	17.74	0	0	17.74	22.30	4.46
2	9.96	24.23	8.33	26.07	0	0	26.07	26.20	5.24
3	15.83	24.23	12.58	38.65	2.26	2.26	40.91	28.94	5.79
4	21.69	24.23	12.58	51.23	2.93	5.19	56.42	32.52	6.50
5	27.55	24.23	12.58	63.81	7.00	7.00	70.81	36.69	7.34

Figure A-28. Culmann Trial Wedge Method and Graphical Method Results



w	=	Unit weight of soil = 120 pcf
c	=	Cohesion per unit of area = 200 psf (Should usually be neglected)
ϕ	=	Angle of internal friction of soil = 30°
δ	=	Slope of backfill
ϕ'	=	Direction of P = $\frac{2}{3}\phi$ = 20°
h_o	=	$\frac{2c}{w}\tan(45° + \frac{\phi}{2}) = 5.77'$

WEDGE VECTORS

Wedge	Area		Σ Area	Σ Wt.	Σ Surchg.	Total Wt. (Kips)
1	$\frac{(20 + 3.8) \times 5.7}{2}$	= 67.8	67.8	8.1	0	8.1
2	$\frac{18.03 \times 4.65}{2} + 5.77 \times 4.65$	= 68.7	136.5	16.4	0	16.4
3	Same as 2	= 68.7	205.2	24.6	0	24.6
4	$\frac{24.23 \times 5.0}{2} + 5.77 \times 5.0$	= 89.4	294.6	35.4	1.5	36.9
5	Same as 4	= 89.4	384.0	46.1	4.0	50.1
6	Same as 4	= 89.4	473.4	56.8	6.5	63.3

COHESION VECTORS

Wedge	Length	Cohesion (Kips)
1	18.0	3.6
2	21.6	4.3
3'	26.0	5.2
4	28.1	5.6
5	31.0	6.2
6	34.3	6.9

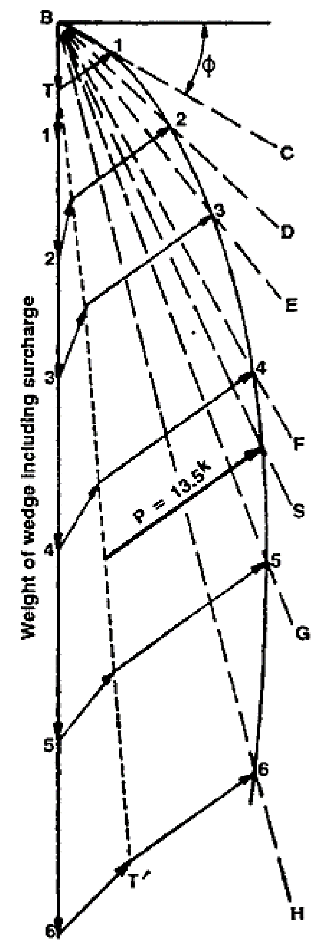


Figure C-8-5-5. Earth Pressure Computation – Walls without Heels

Figure A-29. Snapshot of AREMA Manual, Page 8-5-21

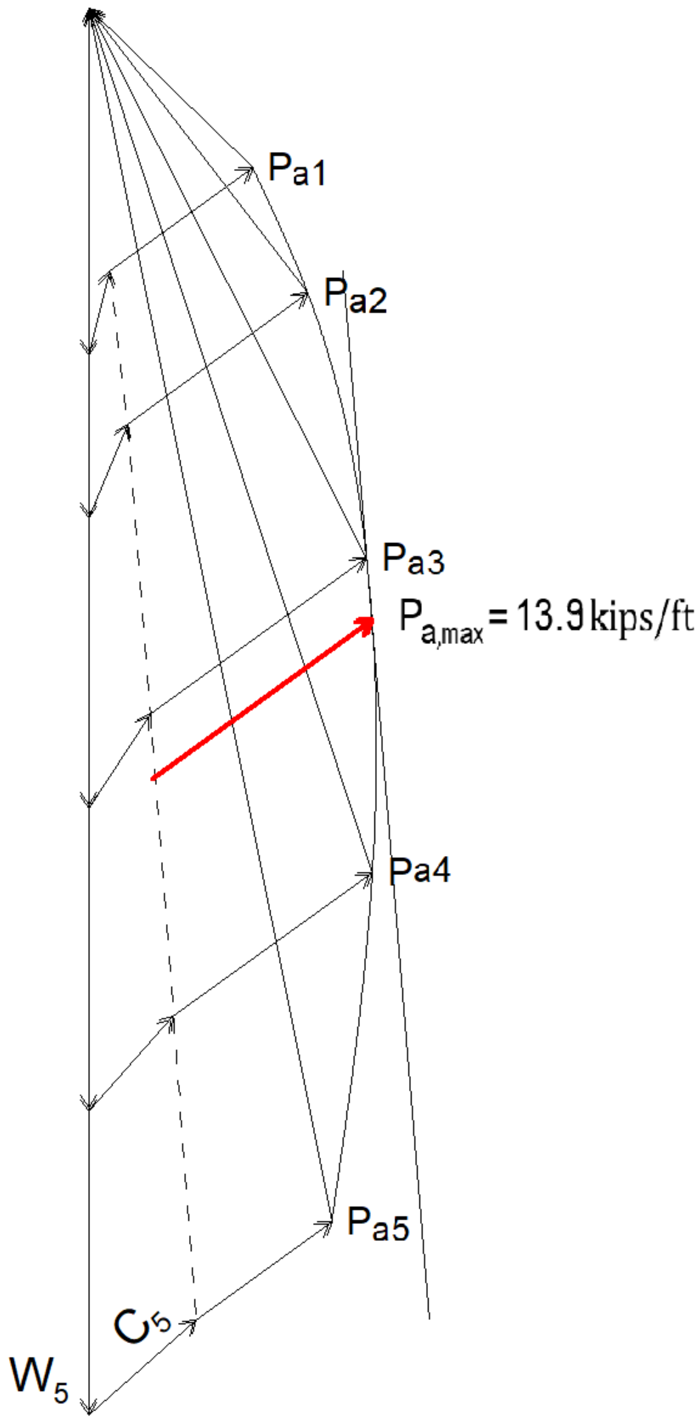


Figure A-30. Culmann Force Polygon

In the 2023 *AREMA Manual for Railway Engineering* (Page 8-5-21), the value was 13.5 kips/ft. This value of 13.9 kips/ft is slightly different due to the graphical nature of the calculation.

A-3.06 Example A-6 (2023 AREMA Manual Page 8-5-21)

Repeat Example A-5, using the critical active wedge method. This is the gravity retaining wall that is shown on Page 8-5-21 in the 2023 AREMA Manual, and is referred to as a retaining wall without a heel. Figure A-31 illustrates the most critical failure surface developed using the *Caltrans Trenching & Shoring Check Program.*

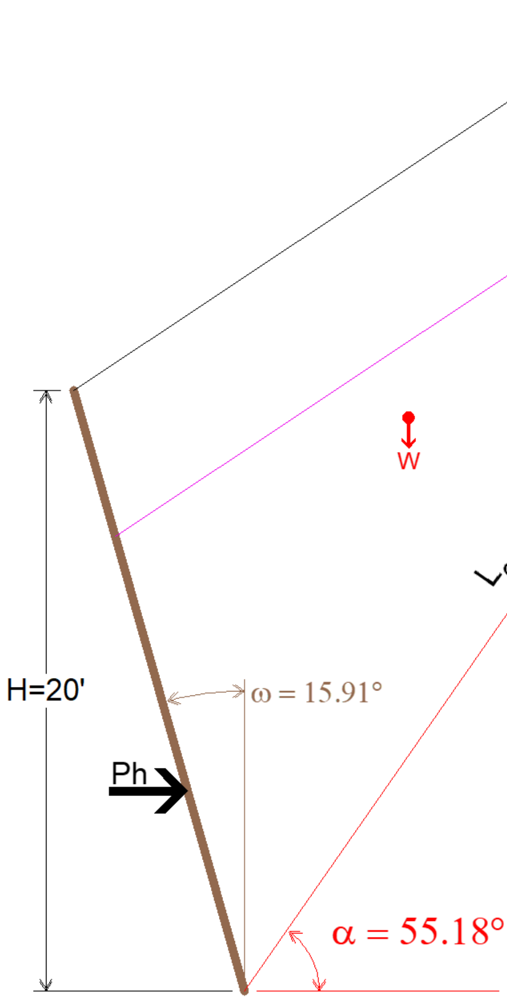


Figure A-31. Critical Active Wedge

In general, Equation A-3-21 is used to calculate the active earth pressure.

$$\mathbf{P_a} = \frac{\mathbf{W[\tan(\alpha - \phi)] - C_0\ L_c[\sin\alpha \tan(\alpha - \phi) + \cos\ \alpha] - C_a L_a[\tan(\alpha - \phi)\cos(-\omega) + \sin\ \omega]}}{\mathbf{[1 + \tan(\delta + \omega)\tan(\alpha - \phi)]\cos(\delta + \omega)}}$$

(A-3-21)

Caltrans Trenching and Shoring Check Program is used to calculate the failure plane angle (α) and the length of the critical failure surface (**L_c**). Without the adhesion component, the calculation is simplified as illustrated below.

$$\mathbf{P_a = \frac{W[\tan(\alpha - \phi)] - C_O L_c[\sin\alpha \tan(\alpha - \phi) + \cos\ \alpha]}{[1 + \tan(\delta + \omega)\tan(\alpha - \phi)]\cos(\delta + \omega)}} \quad \text{(A-3-22)}$$

or

$$\mathbf{P_a = \frac{WT - COH}{[1 + \tan(\delta + \omega)\tan(\alpha - \phi)]\cos(\delta + \omega)}} \quad \text{(A-3-23)}$$

Calculate the weight contribution from the weight of the wedge and the weight of the surcharge (WT):

$$\mathrm{WT} = \mathrm{W}[\tan(\alpha - \phi)] = (43.65)[\tan(55.18 - 30)] = 20.51\ \mathrm{k/ft} \quad \text{(A-3-24)}$$

Calculate cohesion (COH) component.

$$\begin{aligned}\mathrm{COH} &= \mathrm{C_O L_c}[\sin\alpha\tan(\alpha - \phi) + \cos\ \alpha] \\ &= 0.2(29.52)[\tan(55.18 - 30)\sin(55.18) + \cos(55.18)] = 5.65\ \mathrm{k/ft}\end{aligned} \quad \text{(A-3-25)}$$

Substitute WT and COH into Equation A-3-23.

$$\mathrm{P} = \frac{(20.51 - 5.65)}{[1 + \tan(20 + 15.91)\tan(55.18 - 30)]\cos(20 + 15.91)} = 13.70\ \mathrm{k/ft} \quad \text{(A-3-26)}$$

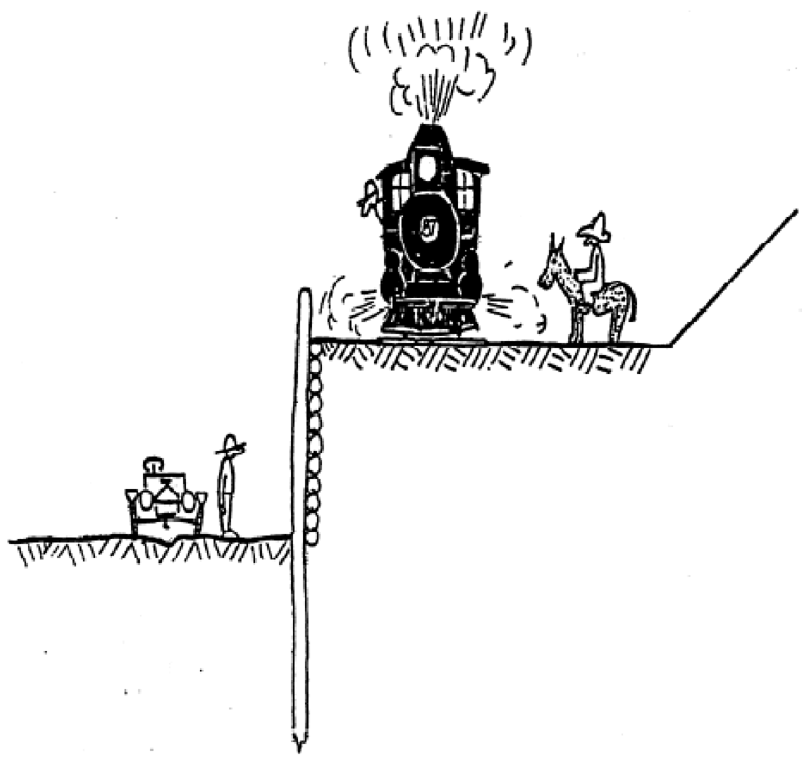
In the 2023 *AREMA Manual for Railway Engineering* (Page 8-5-21), the value was 13.5 kips/ft. This value of 13.70 kips/ft is slightly different due to the graphical nature of the calculation.

Appendix B is not available at this time. Pending revisions are expected to be finalized in August 2025.

JULY 2025

APPENDIX C

SURCHARGES – TABULAR VALUES



George Thompson

Appendix C: Surcharges - Tabular Values

Table of Contents

Appendix C: Surcharges - Tabular Values 1

Table of Contents 1

C-1 Tabular Values for Strip Loads 2

C-2 Additional Notes: Boussinesq versus Wayne C. Teng Equations 16

C-1 Tabular Values for Strip Loads

The tabular values listed beginning on page C-6 may be used to obtain horizontal pressures due to vertical surcharge loadings.

The values in the tables, beginning with Table C-2a, are for a Boussinesq strip surcharge of **Q** = 300 psf using the Wayne C. Teng Equation from 1962. They are calculated for a length of surcharge beginning at the face of the excavation (**L_0**) to the end of the strip load **(L_2)** as illustrated in Figure C-1. Surcharge pressures are listed for one-foot increments of excavation to a depth of 20 feet.

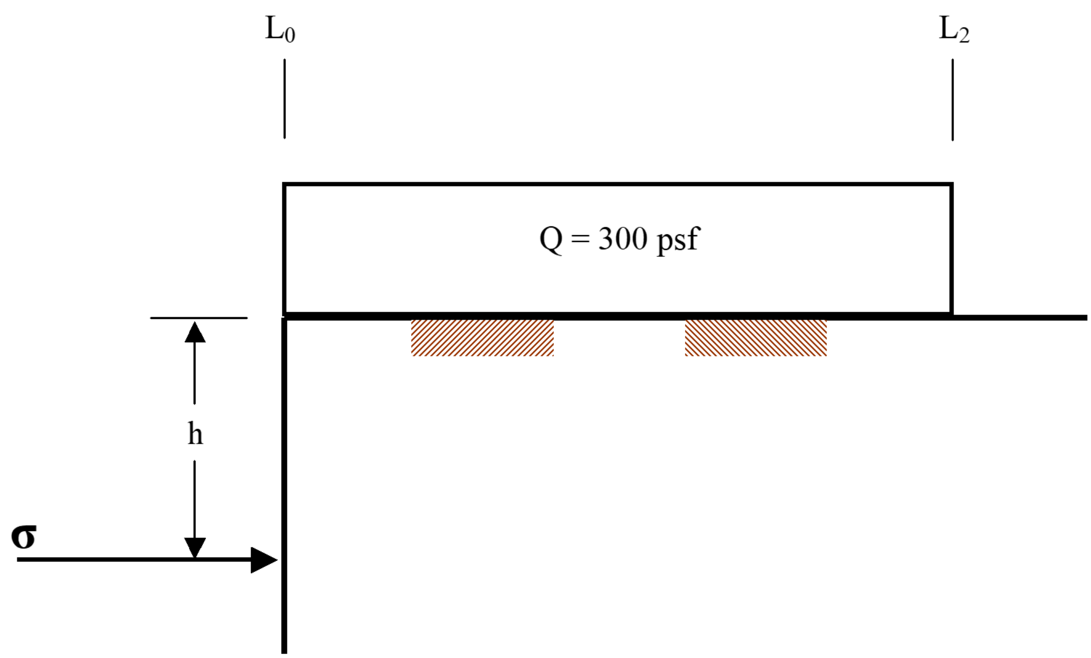


Figure C-1. Typical 300 psf Vertical Surcharge Starting at Face of Excavation

For surcharges not beginning at the face of the excavation (**L_0**), subtract tabular values for distance **L_1** from the tabular values for **L_2** to determine the surcharge load for 300 psf, as illustrated in Figure C-2. Then if needed, prorate the **Q** value by using the ratio **Q/300**. Below is a simple example on how to use the surcharge table. Note: When **L_0** = 0, the horizontal pressure at **h** = 0 is equal to **Q** (300 psf).

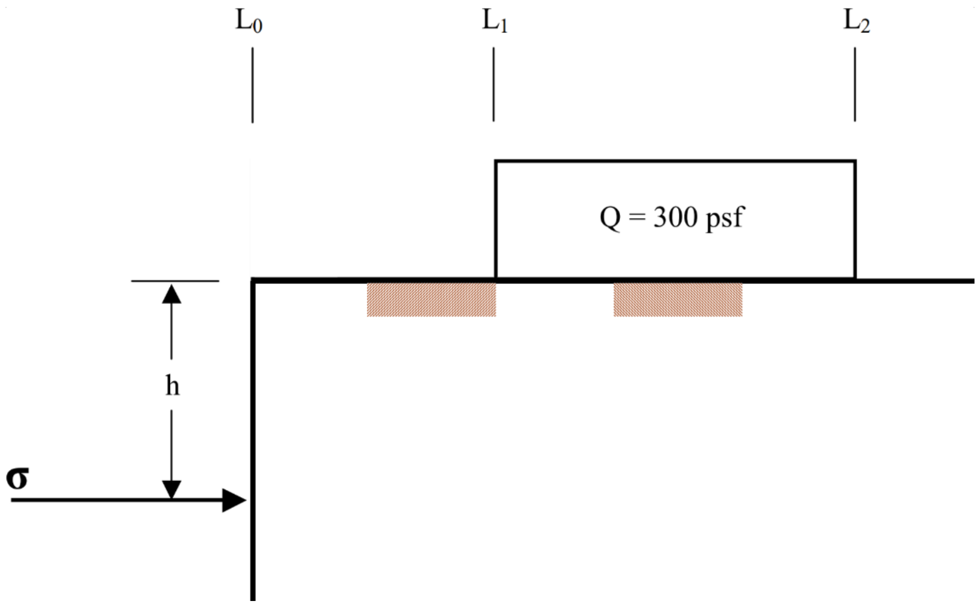


Figure C-2. Typical 300 psf Vertical Surcharge Starting Away from Face of Excavation

<u>Example 1:</u>

Given:

Begin Boussinesq strip load 6 feet from excavation, $\mathbf{L_1}$ = 6′

End Boussinesq strip load 20 feet from excavation, $\mathbf{L_2}$ = 20′

Surcharge load **Q** = 250 psf vertical.

Determine from the tables the given surcharge at a depth of 12 feet:

The surcharge load is **Q** = 250 psf vertical.

Look up the tabular value for the Boussinesq strip load 6 feet from excavation, $\mathbf{L_1}$ = 6′:

$\boldsymbol{\sigma}$ = 12.16 psf horizontal

Look up the tabular value for the Boussinesq strip load 20 feet from excavation, $\mathbf{L_2}$ = 20′:

$\boldsymbol{\sigma}$ = 112.53 psf horizontal

Determine surcharge pressure at **h** = 12′:

$$\sigma_{12} = \frac{250}{300}\,(112.53 - 12.16) \ = 83.6 \text{ horizontal} \qquad \text{(C-1-1)}$$

Example 2:

Given: Surcharge loadings as shown in Figure C-3 (note - figure not to scale).

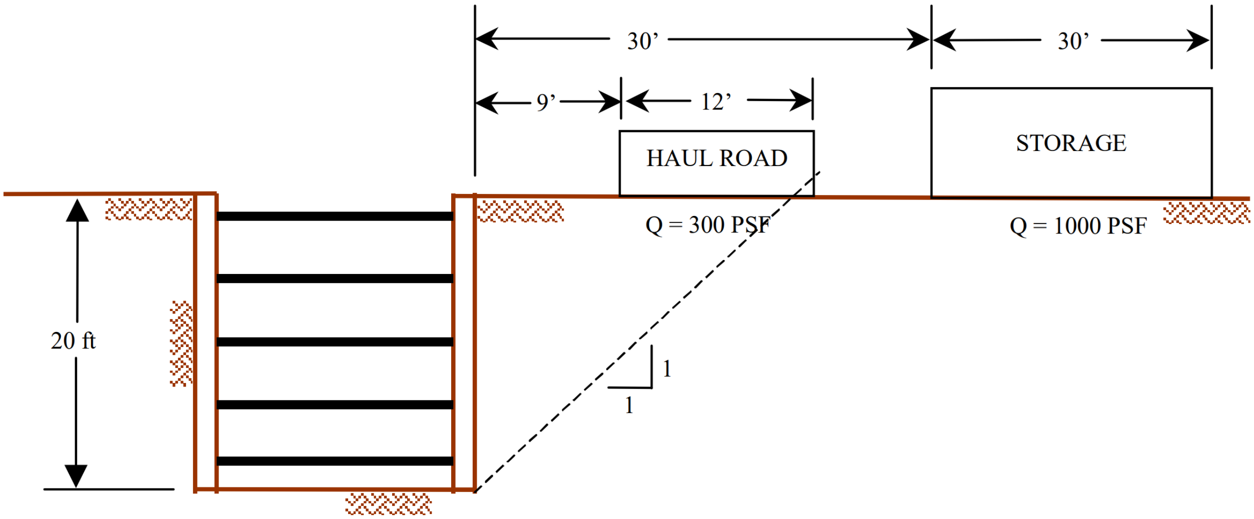


Figure C-3. Excavation Near a Haul Road and Storage Building with Various Surcharge Loads

1. Determine horizontal surcharge pressures at 5-foot increments of depth starting at the ground surface.

2. Compare tabular strip load values to the alternative loading (using 100 psf horizontal, in place of the Haul Road) as illustrated in Table C-1.

   Sample calculations for depth = 10 feet:

   For the haul road:

$$\sigma_{10} = 140.99 - 44.99 = 96.00 \text{ psf} \qquad \text{(C-1-2)}$$

   For the building:

$$\sigma_{10} = \frac{1000}{300}(237.49 - 181.25) = 187.47 \text{ psf} \qquad \text{(C-1-3)}$$

   Building $\boldsymbol{\sigma_{10}}$ + Road $\boldsymbol{\sigma_{10}}$ = 187.47 + 96.0 = 283.47 psf

   Building $\boldsymbol{\sigma_{10}}$ + Road @ 100 psf = 187.47 + 100 = 287.47 psf (alternative)

Table C-1. Tabulated Values of Combined Horizontal Surcharges for Example 2

Depth	Building **σ**	Road **σ**	Sum of **σ**'s	Building **σ** + 100 psf
0	0.00	0.00	72.00 min	100.00
5	102.77	90.26	193.03	202.77
10	187.47	96.00	283.47	287.47
15	244.03	72.26	316.29	344.03
20	272.33	49.99	322.32	372.33

Table C-2a. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	1	2	3	4	5	6	7	8
1	54.51	135.06	181.25	208.27	225.57	237.49	246.16	252.74
2	12.16	54.51	99.55	135.06	161.47	181.25	196.40	208.27
3	4.15	24.15	54.51	85.43	112.53	135.06	153.52	168.69
4	1.85	12.16	31.23	54.51	77.97	99.55	118.58	135.06
5	0.97	6.81	18.95	35.70	54.51	73.39	91.21	107.48
6	0.57	4.15	12.16	24.15	38.76	54.51	70.29	85.43
7	0.36	2.70	8.18	16.88	28.13	40.97	54.51	68.07
8	0.24	1.85	5.73	12.16	20.85	31.23	42.64	54.51
9	0.17	1.32	4.15	8.99	15.77	24.15	33.70	43.94
10	0.13	0.97	3.10	6.81	12.16	18.95	26.92	35.70
11	0.09	0.74	2.37	5.27	9.53	15.08	21.73	29.24
12	0.07	0.57	1.85	4.15	7.59	12.16	17.73	24.15
13	0.06	0.45	1.47	3.33	6.13	9.92	14.61	20.11
14	0.05	0.36	1.19	2.70	5.02	8.18	12.16	16.88
15	0.04	0.30	0.97	2.22	4.15	6.81	10.20	14.27
16	0.03	0.24	0.81	1.85	3.47	5.73	8.63	12.16
17	0.03	0.20	0.67	1.55	2.93	4.86	7.36	10.42
18	0.02	0.17	0.57	1.32	2.50	4.15	6.32	8.99
19	0.02	0.15	0.49	1.13	2.14	3.58	5.46	7.81
20	0.02	0.13	0.42	0.97	1.85	3.10	4.75	6.81

Table C-2b. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	9	10	11	12	13	14	15	16
1	257.90	262.06	265.47	268.32	270.73	272.81	274.61	276.19
2	217.79	225.57	232.04	237.49	242.14	246.16	249.66	252.74
3	181.25	191.77	200.67	208.27	214.84	220.56	225.57	230.01
4	149.24	161.47	172.05	181.25	189.31	196.40	202.68	208.27
5	122.07	135.06	146.57	156.79	165.88	173.99	181.25	187.78
6	99.55	112.53	124.34	135.06	144.75	153.52	161.47	168.69
7	81.20	93.64	105.26	116.02	125.94	135.06	143.42	151.10
8	66.39	77.97	89.06	99.55	109.39	118.58	127.12	135.06
9	54.51	65.08	75.43	85.43	94.97	104.01	112.53	120.53
10	44.99	54.51	64.03	73.39	82.47	91.21	99.55	107.48
11	37.36	45.85	54.51	63.17	71.70	80.03	88.08	95.81
12	31.23	38.76	46.57	54.51	62.45	70.29	77.97	85.43
13	26.27	32.93	39.95	47.18	54.51	61.84	69.10	76.22
14	22.24	28.13	34.41	40.97	47.70	54.51	61.32	68.07
15	18.95	24.15	29.77	35.70	41.86	48.15	54.51	60.86
16	16.25	20.85	25.87	31.23	36.84	42.64	48.55	54.51
17	14.02	18.09	22.58	27.41	32.53	37.85	43.33	48.90
18	12.16	15.77	19.79	24.15	28.81	33.70	38.76	43.94
19	10.60	13.81	17.42	21.36	25.59	30.07	34.75	39.57
20	9.29	12.16	15.39	18.95	22.81	26.92	31.23	35.70

Table C-2c. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	17	18	19	20	21	22	23	24
1	277.58	278.82	279.93	280.93	281.84	282.66	283.41	284.10
2	255.47	257.90	260.09	262.06	263.84	265.47	266.95	268.32
3	233.95	237.49	240.67	243.55	246.16	248.55	250.73	252.74
4	213.28	217.79	221.87	225.57	228.95	232.04	234.87	237.49
5	193.67	199.00	203.85	208.27	212.33	216.05	219.47	222.64
6	175.26	181.25	186.74	191.77	196.40	200.67	204.62	208.27
7	158.16	164.65	170.63	176.15	181.25	185.98	190.38	194.46
8	142.42	149.24	155.58	161.47	166.95	172.05	176.81	181.25
9	128.03	135.06	141.62	147.77	153.52	158.91	163.95	168.69
10	114.98	122.07	128.76	135.06	140.99	146.57	151.84	156.79
11	103.21	110.26	116.96	123.32	129.35	135.06	140.46	145.58
12	92.63	99.55	106.19	112.53	118.58	124.34	129.83	135.06
13	83.16	89.89	96.39	102.65	108.66	114.42	119.94	125.21
14	74.70	81.20	87.51	93.64	99.55	105.26	110.75	116.02
15	67.17	73.39	79.48	85.43	91.21	96.82	102.24	107.48
16	60.47	66.39	72.23	77.97	83.59	89.06	94.39	99.55
17	54.51	60.12	65.69	71.21	76.63	81.95	87.15	92.21
18	49.21	54.51	59.81	65.08	70.29	75.43	80.48	85.43
19	44.50	49.49	54.51	59.53	64.52	69.47	74.36	79.16
20	40.30	44.99	49.74	54.51	59.28	64.03	68.74	73.39

Table C-2d. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	25	26	27	28	29	30	31	32
1	284.74	285.32	285.87	286.37	286.84	287.28	287.69	288.07
2	269.57	270.73	271.81	272.81	273.74	274.61	275.43	276.19
3	254.60	256.31	257.90	259.38	260.77	262.06	263.26	264.40
4	239.90	242.14	244.22	246.16	247.97	249.66	251.25	252.74
5	225.57	228.30	230.83	233.20	235.41	237.49	239.44	241.27
6	211.67	214.84	217.79	220.56	223.14	225.57	227.86	230.01
7	198.27	201.83	205.16	208.27	211.20	213.96	216.55	219.00
8	185.41	189.31	192.96	196.40	199.63	202.68	205.56	208.27
9	173.14	177.32	181.25	184.96	188.46	191.77	194.90	197.86
10	161.47	165.88	170.05	173.99	177.72	181.25	184.60	187.78
11	150.43	155.03	159.38	163.51	167.43	171.15	174.69	178.05
12	140.02	144.75	149.24	153.52	157.59	161.47	165.17	168.69
13	130.24	135.06	139.65	144.04	148.23	152.23	156.05	159.71
14	121.08	125.94	130.60	135.06	139.33	143.42	147.34	151.10
15	112.53	117.39	122.07	126.57	130.90	135.06	139.05	142.89
16	104.56	109.39	114.07	118.58	122.93	127.12	131.16	135.06
17	97.14	101.93	106.57	111.06	115.41	119.62	123.68	127.61
18	90.26	94.97	99.55	104.01	108.33	112.53	116.59	120.53
19	83.88	88.49	93.00	97.40	101.68	105.85	109.89	113.83
20	77.97	82.47	86.89	91.21	95.43	99.55	103.57	107.48

Table C-2e. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	33	34	35	36	37	38	39	40
1	288.43	288.77	289.09	289.40	289.68	289.95	290.21	290.45
2	276.91	277.58	278.22	278.82	279.39	279.93	280.45	280.93
3	265.47	266.47	267.42	268.32	269.16	269.97	270.73	271.46
4	254.15	255.47	256.72	257.90	259.02	260.09	261.10	262.06
5	242.99	244.62	246.16	247.62	249.00	250.31	251.56	252.74
6	232.04	233.95	235.77	237.49	239.12	240.67	242.14	243.55
7	221.31	223.50	225.57	227.54	229.41	231.18	232.87	234.48
8	210.85	213.28	215.60	217.79	219.88	221.87	223.76	225.57
9	200.67	203.33	205.87	208.27	210.57	212.75	214.84	216.83
10	190.80	193.67	196.40	199.00	201.48	203.85	206.11	208.27
11	181.25	184.31	187.21	189.99	192.64	195.17	197.60	199.92
12	172.05	175.26	178.32	181.25	184.06	186.74	189.31	191.77
13	163.20	166.54	169.74	172.80	175.74	178.55	181.25	183.85
14	154.71	158.16	161.47	164.65	167.70	170.63	173.44	176.15
15	146.57	150.12	153.52	156.79	159.94	162.97	165.88	168.69
16	138.80	142.42	145.89	149.24	152.47	155.58	158.58	161.47
17	131.40	135.06	138.59	142.00	145.29	148.47	151.53	154.50
18	124.34	128.03	131.60	135.06	138.39	141.62	144.75	147.77
19	117.64	121.35	124.94	128.42	131.79	135.06	138.22	141.29
20	111.28	114.98	118.58	122.07	125.46	128.76	131.95	135.06

Table C-2f. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	41	42	43	44	45	46	47	48
1	290.69	290.91	291.12	291.32	291.51	291.70	291.88	292.04
2	281.40	281.84	282.26	282.66	283.05	283.41	283.77	284.10
3	272.15	272.81	273.44	274.04	274.61	275.16	275.69	276.19
4	262.97	263.84	264.67	265.47	266.22	266.95	267.65	268.32
5	253.87	254.95	255.98	256.96	257.90	258.81	259.67	260.50
6	244.88	246.16	247.38	248.55	249.66	250.73	251.76	252.74
7	236.02	237.49	238.89	240.23	241.52	242.75	243.94	245.07
8	227.30	228.95	230.53	232.04	233.49	234.87	236.21	237.49
9	218.73	220.56	222.30	223.97	225.57	227.11	228.59	230.01
10	210.34	212.33	214.22	216.05	217.79	219.47	221.09	222.64
11	202.14	204.27	206.31	208.27	210.16	211.97	213.71	215.39
12	194.13	196.40	198.58	200.67	202.68	204.62	206.48	208.27
13	186.33	188.73	191.02	193.24	195.36	197.42	199.39	201.30
14	178.75	181.25	183.66	185.98	188.22	190.38	192.46	194.46
15	171.39	173.99	176.50	178.92	181.25	183.51	185.68	187.78
16	164.26	166.95	169.55	172.05	174.47	176.81	179.07	181.25
17	157.36	160.12	162.80	165.38	167.88	170.29	172.63	174.89
18	150.69	153.52	156.26	158.91	161.47	163.95	166.36	168.69
19	144.26	147.14	149.93	152.64	155.26	157.80	160.27	162.66
20	138.07	140.99	143.82	146.57	149.24	151.84	154.35	156.79

Table C-2g. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	49	50	51	52	53	54	55	56
1	292.21	292.36	292.51	292.66	292.79	292.93	293.06	293.18
2	284.43	284.74	285.04	285.32	285.60	285.87	286.12	286.37
3	276.67	277.14	277.58	278.01	278.43	278.82	279.21	279.58
4	268.96	269.57	270.16	270.73	271.28	271.81	272.32	272.81
5	261.29	262.06	262.79	263.50	264.18	264.83	265.47	266.08
6	253.69	254.60	255.47	256.31	257.12	257.90	258.66	259.38
7	246.16	247.21	248.22	249.19	250.13	251.03	251.90	252.74
8	238.72	239.90	241.04	242.14	243.20	244.22	245.21	246.16
9	231.37	232.69	233.95	235.18	236.35	237.49	238.59	239.64
10	224.13	225.57	226.96	228.30	229.59	230.83	232.04	233.20
11	217.01	218.56	220.07	221.52	222.91	224.27	225.57	226.84
12	210.01	211.67	213.28	214.84	216.34	217.79	219.20	220.56
13	203.13	204.91	206.62	208.27	209.87	211.42	212.92	214.37
14	196.40	198.27	200.08	201.83	203.52	205.16	206.74	208.27
15	189.81	191.77	193.67	195.50	197.28	199.00	200.67	202.28
16	183.37	185.41	187.39	189.31	191.17	192.96	194.71	196.40
17	177.08	179.20	181.25	183.25	185.18	187.05	188.86	190.63
18	170.95	173.14	175.26	177.32	179.32	181.25	183.14	184.96
19	164.98	167.23	169.41	171.53	173.59	175.59	177.53	179.42
20	159.17	161.47	163.71	165.88	168.00	170.05	172.05	173.99

Table C-2h. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	57	58	59	60	61	62	63	64
1	293.30	293.42	293.53	293.64	293.74	293.84	293.94	294.03
2	286.61	286.84	287.06	287.28	287.49	287.69	287.88	288.07
3	279.93	280.28	280.61	280.93	281.24	281.55	281.84	282.12
4	273.28	273.74	274.18	274.61	275.02	275.43	275.81	276.19
5	266.66	267.23	267.78	268.32	268.83	269.33	269.81	270.28
6	260.09	260.77	261.42	262.06	262.67	263.26	263.84	264.40
7	253.56	254.34	255.10	255.84	256.55	257.24	257.90	258.55
8	247.08	247.97	248.83	249.66	250.47	251.25	252.01	252.74
9	240.67	241.66	242.62	243.55	244.45	245.32	246.16	246.98
10	234.33	235.41	236.47	237.49	238.48	239.44	240.37	241.27
11	228.06	229.24	230.39	231.50	232.57	233.61	234.63	235.61
12	221.87	223.14	224.38	225.57	226.73	227.86	228.95	230.01
13	215.77	217.13	218.45	219.72	220.96	222.17	223.34	224.47
14	209.76	211.20	212.60	213.96	215.27	216.55	217.79	219.00
15	203.85	205.37	206.84	208.27	209.66	211.01	212.33	213.60
16	198.04	199.63	201.18	202.68	204.14	205.56	206.93	208.27
17	192.34	194.00	195.61	197.18	198.70	200.18	201.63	203.03
18	186.74	188.46	190.14	191.77	193.36	194.90	196.40	197.86
19	181.25	183.04	184.77	186.46	188.11	189.70	191.26	192.78
20	175.88	177.72	179.51	181.25	182.95	184.60	186.21	187.78

Table C-2i. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	65	66	67	68	69	70	71	72
1	294.12	294.21	294.30	294.38	294.47	294.54	294.62	294.70
2	288.25	288.43	288.60	288.77	288.93	289.09	289.25	289.40
3	282.40	282.66	282.92	283.17	283.41	283.65	283.88	284.10
4	276.55	276.91	277.25	277.58	277.91	278.22	278.53	278.82
5	270.73	271.17	271.60	272.01	272.42	272.81	273.19	273.56
6	264.94	265.47	265.98	266.47	266.95	267.42	267.87	268.32
7	259.18	259.79	260.38	260.95	261.51	262.06	262.58	263.10
8	253.46	254.15	254.82	255.47	256.11	256.72	257.32	257.90
9	247.78	248.55	249.30	250.03	250.73	251.42	252.09	252.74
10	242.14	242.99	243.82	244.62	245.40	246.16	246.90	247.62
11	236.56	237.49	238.39	239.26	240.11	240.94	241.75	242.53
12	231.04	232.04	233.01	233.95	234.87	235.77	236.64	237.49
13	225.57	226.64	227.69	228.70	229.68	230.64	231.58	232.49
14	220.17	221.31	222.42	223.50	224.55	225.57	226.57	227.54
15	214.84	216.05	217.22	218.36	219.47	220.56	221.61	222.64
16	209.58	210.85	212.08	213.28	214.46	215.60	216.71	217.79
17	204.39	205.72	207.01	208.27	209.50	210.70	211.87	213.00
18	199.28	200.67	202.02	203.33	204.62	205.87	207.09	208.27
19	194.25	195.69	197.10	198.46	199.80	201.10	202.37	203.61
20	189.31	190.80	192.25	193.67	195.05	196.40	197.72	199.00

Table C-2j. Horizontal Pressure at Depth h, for Uniform Loadings from L_0 To L_2 or L_1 to L_2

h	73	74	75	76	77	78	79	80
1	294.77	294.84	294.91	294.97	295.04	295.10	295.17	295.23
2	289.54	289.68	289.82	289.95	290.08	290.21	290.33	290.45
3	284.32	284.53	284.74	284.94	285.13	285.32	285.51	285.69
4	279.11	279.39	279.67	279.93	280.19	280.45	280.69	280.93
5	273.92	274.27	274.61	274.94	275.27	275.58	275.89	276.19
6	268.75	269.16	269.57	269.97	270.36	270.73	271.10	271.46
7	263.60	264.08	264.55	265.02	265.47	265.90	266.33	266.75
8	258.47	259.02	259.56	260.09	260.60	261.10	261.58	262.06
9	253.38	254.00	254.60	255.18	255.76	256.31	256.86	257.39
10	248.32	249.00	249.66	250.31	250.94	251.56	252.16	252.74
11	243.30	244.04	244.77	245.47	246.16	246.83	247.49	248.13
12	238.31	239.12	239.90	240.67	241.42	242.14	242.85	243.55
13	233.38	234.24	235.08	235.90	236.71	237.49	238.25	239.00
14	228.48	229.41	230.30	231.18	232.04	232.87	233.69	234.48
15	223.64	224.62	225.57	226.50	227.41	228.30	229.16	230.01
16	218.85	219.88	220.89	221.87	222.83	223.76	224.68	225.57
17	214.11	215.20	216.25	217.29	218.29	219.28	220.24	221.18
18	209.44	210.57	211.67	212.75	213.81	214.84	215.85	216.83
19	204.82	206.00	207.15	208.27	209.38	210.45	211.50	212.53
20	200.26	201.48	202.68	203.85	204.99	206.11	207.21	208.27

C-2 Additional Notes: Boussinesq versus Wayne C. Teng Equations

To follow are some notes and observations to assist the reader with differentiating between Boussinesq and the more recent Wayne C. Teng equations for calculating the effects of surcharge loads.

Section 5-1.03, *Boussinesq Loads*, in this manual introduces the topic of Boussinesq surcharge loads. Boussinesq equations were developed in 1885 using the Theory of Elasticity. The equations were developed for homogeneous and isotropic materials. Although soils are not isotropic, the Boussinesq equations can be used to determine increases in vertical stress from a surcharge load.

Equation 5-1-2 in this manual is not a Boussinesq equation from 1885; it is the Wayne C. Teng Equation for non-yielding walls (rigid walls) from 1962. While it was based on the Boussinesq equations from 1885, it has been modified by experiments and is an empirical equation. The Wayne C. Teng Equation has the following conditions:

1. The wall does not yield.
2. The Wayne C. Teng Equation is a plastic solution. In contrast, the Boussinesq equation is an elastic solution.
3. The alpha (α) angle is different in different sources. Most sources state that the alpha angle is the angle for the line that goes through the center of the strip load. Those sources include the U.S. Army Corps of Engineers and the textbook by Braja Das. The Union Pacific Railroad does not use that alpha angle. The Union Pacific Railroad has a figure that shows that the alpha angle is centered inside of the Beta (β) angle. For the Union Pacific, the line for the alpha angle does not go to the center of the strip load. The Caltrans *Trenching and Shoring Manual* also shows that the alpha angle is centered inside of the Beta angle.
4. For the Wayne C. Teng Equation, the horizontal stress is not a function of the phi (ϕ) angle.
5. For the Wayne C. Teng Equation, the coefficient of earth pressure for the soil is equal to one (1.0). For the Boussinesq Equation, the coefficient of earth pressure is equal to the coefficient for the soil condition that is estimated. Typically, the coefficient is the active earth pressure coefficient, the at-rest earth pressure coefficient, or the passive earth pressure coefficient. If the soil is in the active condition, then the coefficient of earth pressure is the active earth pressure coefficient.
6. Wayne C. Teng Equation has a factor of two (2.0) and is the equation for a rigid wall (non-yielding wall). The U.S. Army Corps of Engineers states that the Wayne C. Teng Equation is correct for a rigid wall, but a factor of 1 (not a factor of 2)

should be used for a flexible wall [U.S. Army Corps of Engineers EM 1110-2-2502 "Engineering and Design for Retaining and Flood Walls" September 29, 1989, Page 3-50].

7. The Union Pacific Railroad uses the Wayne C. Teng Equation for Cooper E80 loads and estimates that all walls are rigid. The Union Pacific Railroad states that it's equation for Cooper E80 loads is the "Boussinesq Strip Load Equation," and the equation they use is the Wayne C. Teng Equation from 1962.

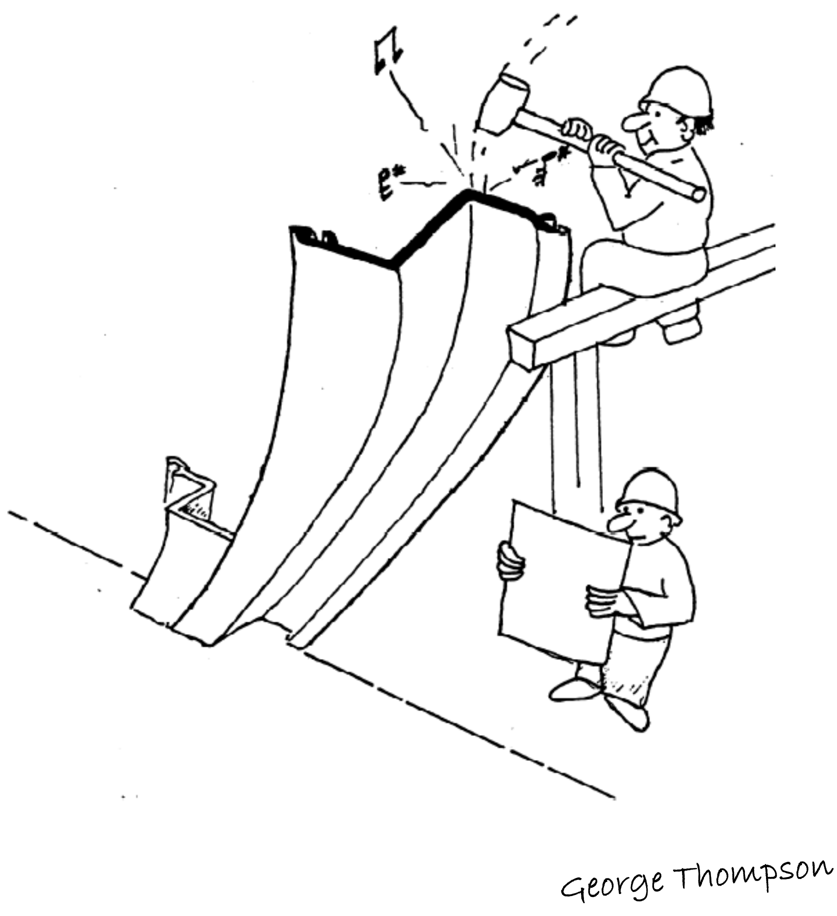
For further information on this topic, refer to these resources:

- The U.S. Army Corps of Engineers, *Engineer Manual*, *Retaining and Flood Walls*
- The Union Pacific Railroad, *UPRR & BNSF Guidelines for Temporary Shoring*
- Boussinesq Equations from the University of Massachusetts at Lowell.

JULY 2025

APPENDIX D

SHEET PILES



Appendix D: Sheet Piles

Table of Contents

Appendix D: Sheet Piles 1
Table of Contents 1
D-1 Steel Sheet Piling Products 2
D-2 Z Sheet Pile 2
D-3 L.B. Foster Piling Products 9
D-3.01 Domestically produced by Gerdau Ameristeel and delivered by L.B. Foster 9
D-3.02 High Efficiency "Z" Section Steel Sheet Pile 11
D-3.03 Heavy Duty "Z" Section Sheet Pile 12

D-1 Steel Sheet Piling Products

Steel sheet piling is a manufactured construction product with a mechanical connection "interlock" at both ends of the section. These mechanical connections interlock with one another to form a continuous wall of sheet piling. Steel sheet pile applications are typically designed to create a rigid barrier for earth and water, while resisting the lateral pressures of those bending forces. The shape or geometry of a section lends to the structural strength. In addition, the soil in which the section is driven has numerous mechanical properties that can affect the performance.

Steel sheet piling is classified in two construction applications, permanent and temporary. A permanent application is "stay-in-place" where the sheet piling wall is driven and remains in the ground. A temporary application provides access and safety for construction in a confined area. Once the work is completed, the sheet piling is removed. A valuable resource for this topic is the 1984 *USS Steel Sheet Piling Design Manual*. Alternatively, Structure Construction Staff may access it at this link[1].

D-2 Z Sheet Pile

Z sections are considered one of the most efficient piles available. Having the interlocks located at the outer fibers of the wall, assures the designer of their published section modulus.

Z-Piles are commonly used for Cantilevered, Tied-Back, King Pile and Combi-Wall retaining systems. Additional applications also include load bearing bridge abutments.

The following figures (D-1 through D-4) and tables (D-1 and D-2) furnish selected properties for various steel sheet piles, and were generally taken from the 1996 *Trenching and Shoring Manual,* with some updates.

Minimum grade of steel is A328 for which **Fb** = 25 ksi.
For sheet piles manufactured prior to 1940 or those with no identified grade of steel, use **Fb** = 22 ksi (A36 equivalent)

Most suppliers also furnish high strength steel, such as A572 (grade 50); in this case, use **Fb** = 30 ksi.

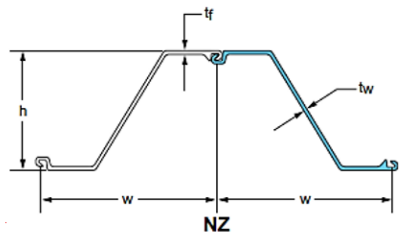
[1] Caltrans internal use only

Section	Width (w)	Height (h)	Thickness		Weight		Section Modulus		Moment of Inertia	Cross Sectional Area	Coating Area	
			Flange	Web	Pile	Wall	Elastic	Plastic			Both Sides of Single[1]	Wall Surface
	in (mm)	in (mm)	in (mm)	in (mm)	lb/ft (kg/m)	lb/ft^2 (kg/m)	in^3/ft (cm^3/m)	in^3/ft (cm^3/m)	in^4/ft (cm^4/m)	in^2/ft (cm^2/m)	ft^2/ft (m^2/m)	ft^2/ft^2 (m^2/m^2)
Hoesch 1207	27.56 700	12.32 313	0.339 8.6	0.335 8.5	45.42 67.6	19.78 96.6	22.4 1205	26.3 1415	137.9 18,833	5.81 123.1	5.60 1.71	1.22 1.22
Hoesch 1307	27.56 700	12.36 314	0.378 9.6	0.374 9.5	49.66 73.9	21.63 105.6	24.3 1307	28.7 1541	150.1 20,494	6.36 134.6	5.60 1.71	1.22 1.22
Hoesch 1407	27.56 700	12.40 315	0.417 10.6	0.413 10.5	53.96 80.3	23.49 114.7	26.2 1409	31.0 1667	162.3 22,156	6.90 146.1	5.60 1.71	1.22 1.22
Hoesch 1208	30.31 770	13.84 354	0.335 8.5	0.335 8.5	49.19 73.2	19.45 95.0	24.2 1301	27.0 1540	137.9 18,833	5.71 121.0	6.13 1.87	1.21 1.21
Hoesch 1308	30.31 770	13.84 354	0.354 9.0	0.354 9.0	51.47 76.6	20.37 99.5	25.3 1357	30.3 1609	150.1 20,494	5.98 126.8	6.13 1.87	1.21 1.21
Hoesch 1408	30.31 770	13.88 355	0.375 9.5	0.375 9.5	53.82 80.1	21.29 104.0	26.3 1412	31.3 1678	162.3 22,156	6.25 132.5	6.13 1.87	1.21 1.21
Hoesch 1707	27.56 700	16.54 420	0.335 8.5	0.335 8.5	49.19 73.2	21.42 104.6	32.2 1731	37.8 2031	265.9 36,304	6.29 133.2	6.10 1.86	1.33 1.33
Hoesch 1807	27.56 700	16.54 420	0.354 9.0	0.354 9.0	51.54 76.7	22.42 109.5	33.6 1804	39.5 2121	277.5 37,894	6.59 139.5	6.10 1.86	1.33 1.33
Hoesch 1907	27.56 700	16.57 421	0.375 9.5	0.375 9.5	53.82 80.1	23.45 114.5	34.9 1878	41.2 2212	289.2 39,485	6.89 145.9	6.10 1.86	1.33 1.33
Hoesch 2007	27.56 700	16.57 421	0.394 10.0	0.394 10.0	56.18 83.6	24.45 119.4	36.3 1951	42.9 2303	300.8 41,076	7.19 152.2	6.10 1.86	1.33 1.33
Hoesch 2407	27.56 700	17.28 439	0.433 11.0	0.443 11.0	63.90 95.1	27.83 135.9	45.2 2432	52.5 2819	390.9 53,379	8.18 173.1	6.34 1.93	1.39 1.39
Hoesch 2607	27.56 700	17.32 440	0.480 12.2	0.480 12.2	68.88 102.5	30.00 146.5	48.5 2606	56.4 3030	420.0 57,329	8.82 186.6	6.34 1.93	1.39 1.39
Hoesch 2807	27.56 700	17.36 441	0.512 13.0	0.512 13.0	73.85 109.9	32.15 157.0	51.7 2779	60.3 3241	448.8 61,279	9.45 200.0	6.34 1.93	1.39 1.39
Hoesch 3607	27.56 700	19.65 499	0.669 17.0	0.433 11.0	79.36 118.1	34.57 168.8	67.0 3600	76.5 4111	657.8 89,826	10.16 215.0	6.72 2.05	1.45 1.45
Hoesch 3807	27.56 700	19.69 500	0.709 18.0	0.472 12.0	84.60 125.9	36.84 179.9	70.7 3800	81.1 4357	695.8 95,004	10.83 229.2	6.72 2.05	1.45 1.45
Hoesch 4007	27.56 700	19.72 501	0.748 19.0	0.512 13.0	89.84 133.7	39.12 191.0	74.4 4000	85.7 4604	733.7 100,184	11.49 243.3	6.72 2.05	1.45 1.45
Hoesch 4807	27.56 700	19.96 507	0.925 23.5	0.571 14.5	104.89 156.1	45.69 223.1	88.6 4760	102.0 5478	883.7 120,667	13.42 284.1	6.72 2.05	1.45 1.45
Hoesch 5007	27.56 700	20.00 508	0.965 24.5	0.610 15.5	110.20 164.0	47.98 234.3	92.3 4959	106.6 5729	922.5 125,966	13.67 289.4	6.72 2.05	1.45 1.45
Hoesch 5207	27.56 700	20.00 508	1.004 25.5	0.650 16.5	115.44 171.8	50.26 245.4	96.0 5158	111.3 5979	961.3 131,266	14.76 312.6	6.72 2.05	1.45 1.45

Figure D-1. Hoesch Steel Sheet Piling

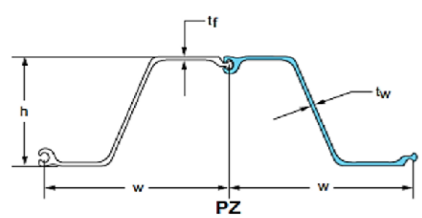
NUCOR® SKYLINE

HOT ROLLED STEEL SHEET PILE NZ



SECTION	Width (w) in mm	Height (h) in mm	THICKNESS Flange (t_f) in mm	THICKNESS Web (t_w) in mm	Cross Sectional Area in²/ft cm²/m	WEIGHT Single Pile lb/ft kg/m	WEIGHT Wall Area lb/ft² kg/m²	SECTION MODULUS Elastic in³/ft cm³/m	SECTION MODULUS Plastic in³/ft cm³/m	Moment of Inertia in⁴/ft cm⁴/m	COATING AREA Both Sides ft²/ft of single m²/m	COATING AREA Wall Surface ft²/ft² m²/m²
NZ 14	30.31 770	13.39 340	0.375 9.5	0.375 9.5	6.40 135.4	55 81.26	21.77 106.30	25.65 1379	30.50 1640	171.7 23447	6.10 1.86	1.20 1.20
NZ 19	27.56 700	16.14 410	0.375 9.5	0.375 9.5	7.07 149.6	55 81.85	24.05 117.40	35.08 1886	41.33 2222	283.1 38659	6.18 1.88	1.35 1.35
NZ 20	27.56 700	16.16 411	0.394 10.0	0.394 10.0	7.34 155.4	57 85.37	24.82 122.00	36.24 1948	42.80 2301	292.8 39984	6.18 1.88	1.35 1.35
NZ 21	27.56 700	16.20 412	0.433 11.0	0.433 11.0	7.80 165.2	61 90.78	26.56 129.70	38.69 2080	45.85 2465	313.4 42797	6.18 1.88	1.35 1.35
NZ 22	27.56 700.0	16.25 413.0	0.480 12.20	0.480 12.20	8.57 181.4	67 99.71	29.20 142.44	41.47 2230	49.34 2653	336.9 46006	6.18 1.88	1.35 1.35
NZ 26	27.56 700	17.32 440	0.500 12.7	0.500 12.7	9.08 192.2	71 105.66	30.99 151.30	48.50 2608	57.01 3065	419.9 57340	6.49 1.98	1.41 1.41
NZ 28	27.56 700	17.38 441	0.560 14.2	0.560 14.2	9.98 211.2	78 116.08	33.96 165.82	52.62 2829	62.16 3342	457.4 62461	6.49 1.98	1.41 1.41
NZ 38	27.56 700	19.69 500	0.689 17.5	0.500 12.7	11.00 232.9	86 127.99	37.45 182.83	70.84 3809	81.57 4386	697.3 95214	6.58 2.01	1.43 1.43
NZ 40	27.56 700.0	19.73 501.0	0.735 18.70	0.551 14.00	11.77 249.1	92 136.91	40.06 195.59	74.97 4031	86.75 4664	739.6 100997	6.58 2.01	1.43 1.43
NZ 42	27.56 700.0	19.77 502.0	0.769 19.50	0.589 15.0	12.41 262.7	97 144.36	42.24 206.23	78.17 4203	90.80 4881	772.5 105490	6.58 2.01	1.43 1.43

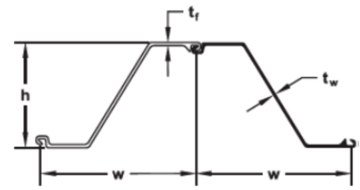
HOT ROLLED STEEL SHEET PILE PZ



SECTION	Width (w) in mm	Height (h) in mm	THICKNESS Flange (t_f) in mm	THICKNESS Web (t_w) in mm	Cross Sectional Area in²/ft cm²/m	WEIGHT Single Pile lb/ft kg/m	WEIGHT Wall Area lb/ft² kg/m²	SECTION MODULUS Elastic in³/ft cm³/m	SECTION MODULUS Plastic in³/ft cm³/m	Moment of Inertia in⁴/ft cm⁴/m	COATING AREA Both Sides ft²/ft of single m²/m	COATING AREA Wall Surface ft²/ft² m²/m²
PZ 22	22.00 559	9.0 229	0.375 9.50	0.375 9.50	6.47 136.9	40.3 60.0	22.0 107.4	18.1 973	21.79 1171.4	84.38 11500	4.48 1.37	1.22 1.22
PZ 27	18.00 457	12.0 305	0.375 9.50	0.375 9.50	7.94 168.1	40.5 60.3	27.0 131.8	30.2 1620	36.49 1961.9	184.20 25200	4.48 1.37	1.49 1.49
PZ 35	22.64 575	14.9 378	0.600 15.21	0.500 12.67	10.29 217.8	66.0 98.2	35.0 170.9	48.5 2608	57.17 3073.5	361.22 49300	5.37 1.64	1.42 1.42
PZ 40	19.69 500	16.1 409	0.600 15.21	0.500 12.67	11.77 249.1	65.6 97.6	40.0 195.3	60.7 3263	71.92 3866.7	490.85 67000	5.37 1.64	1.64 1.64

Figure D-2. Nucor Skyline

AZ HOT ROLLED STEEL SHEET PILE SERIES



J.D. FIELDS & COMPANY, INC.	Width (w)	Height (h)	THICKNESS Flange (t_f)	THICKNESS Web (t_w)	Cross Sec Area (A)	WEIGHT Single Pile	WEIGHT Wall Area	SECTION MODULUS Elastic	SECTION MODULUS Plastic	Moment of Inertia	COATING AREA Both Sides	COATING AREA Wall Surface
	in	in	in	in	in²/ft	lb/ft	lb/ft²	in³/ft	in³/ft	in⁴/ ft	ft²/ft of single	ft²/ft²
SECTION	mm	mm	mm	mm	cm²/m	kg/m	kg/m²	cm³/m	cm³/m	cm⁴/m	m²/m	m²/m²
AZ 12-770	**30.31** 770	**13.52** 344	**0.335** 8.5	**0.335** 8.5	**5.67** 120.1	**48.78** 72.6	**19.31** 94.3	**23.2** 1245	**27.5** 1480	**156.9** 21430	**6.07** 1.85	**1.20** 1.20
AZ 13-770	**30.31** 770	**13.54** 344	**0.354** 9.0	**0.354** 9.0	**5.94** 125.8	**51.14** 76.1	**20.24** 98.8	**24.2** 1300	**28.8** 1546	**163.7** 22360	**6.07** 1.85	**1.20** 1.20
*AZ 14-770	**30.31** 770	**13.56** 345	**0.375** 9.5	**0.375** 9.5	**6.21** 131.5	**53.42** 79.5	**21.14** 103.2	**25.2** 1355	**30.0** 1611	**170.6** 23300	**6.07** 1.85	**1.20** 1.20
AZ 17-700	**27.56** 700	**16.52** 420	**0.335** 8.5	**0.335** 8.5	**6.28** 133.0	**49.12** 73.1	**21.38** 104.4	**32.2** 1730	**37.7** 2027	**265.3** 36230	**6.10** 1.86	**1.33** 1.33
AZ 18-700	**27.56** 700	**16.54** 420	**0.354** 9.0	**0.354** 9.0	**6.58** 139.2	**51.41** 76.5	**22.39** 109.3	**33.5** 1800	**39.4** 2116	**276.8** 37800	**6.10** 1.86	**1.33** 1.33
AZ 19-700	**27.56** 700	**16.56** 421	**0.375** 9.5	**0.375** 9.5	**6.88** 145.6	**53.76** 80.0	**23.35** 114.3	**34.8** 1870	**41.0** 2206	**288.4** 39380	**6.10** 1.86	**1.33** 1.33
AZ 20-700	**27.56** 700	**16.57** 421	**0.394** 10.0	**0.394** 10.0	**7.18** 152.0	**56.11** 83.5	**24.43** 119.3	**36.2** 1945	**42.7** 2296	**300.0** 40960	**6.10** 1.86	**1.33** 1.33
AZ 18-800	**31.5** 800	**17.68** 449	**0.335** 8.5	**0.335** 8.5	**6.07** 128.6	**54.26** 80.7	**20.67** 100.9	**34.2** 1840	**39.7** 2135	**302.6** 41320	**6.82** 2.08	**1.30** 1.30
*AZ 20-800	**31.5** 800	**17.72** 450	**0.375** 9.5	**0.375** 9.5	**6.66** 141.0	**59.50** 88.6	**22.67** 110.7	**37.2** 2000	**43.3** 2330	**329.9** 45050	**6.82** 2.08	**1.30** 1.30
AZ 22-800	**31.5** 800	**17.76** 451	**0.413** 10.5	**0.413** 10.5	**7.25** 153.5	**64.77** 96.4	**24.68** 120.5	**40.3** 2165	**47.0** 2525	**357.3** 48790	**6.82** 2.08	**1.30** 1.30
AZ 23-800	**31.50** 800	**18.66** 474	**0.453** 11.5	**0.354** 9.0	**7.12** 150.6	**63.56** 94.6	**24.22** 118.2	**43.3** 2330	**49.9** 2680	**404.6** 55260	**6.94** 2.11	**1.32** 1.32
*AZ 25-800	**31.50** 800	**18.70** 475	**0.492** 12.5	**0.394** 10.0	**7.71** 163.3	**68.91** 102.6	**26.26** 128.2	**46.5** 2500	**53.8** 2890	**435.1** 59410	**6.94** 2.11	**1.32** 1.32
AZ 27-800	**31.50** 800	**18.74** 476	**0.531** 13.5	**0.433** 11.0	**8.31** 176.0	**74.26** 110.5	**28.29** 138.1	**49.7** 2670	**57.6** 3100	**465.5** 63570	**6.94** 2.11	**1.32** 1.32
AZ 24-700	**27.56** 700	**18.07** 459	**0.441** 11.2	**0.441** 11.2	**8.23** 174.1	**64.30** 95.7	**28.00** 136.7	**45.2** 2430	**53.5** 2867	**408.8** 55820	**6.33** 1.93	**1.38** 1.38
AZ 26-700	**27.56** 700	**18.11** 460	**0.480** 12.2	**0.480** 12.2	**8.84** 187.2	**69.12** 102.9	**30.10** 146.9	**48.4** 2600	**57.1** 3070	**437.3** 59720	**6.33** 1.93	**1.38** 1.38
AZ 28-700	**27.56** 700	**18.15** 461	**0.520** 13.2	**0.520** 13.2	**9.46** 200.2	**73.93** 110.0	**32.19** 157.2	**51.3** 2760	**60.9** 3273	**465.9** 63620	**6.33** 1.93	**1.38** 1.38
AZ 28-750	**29.53** 750.0	**20.04** 509.0	**0.472** 12.00	**0.394** 10.00	**8.09** 171.2	**67.73** 100.80	**27.53** 134.40	**52.3** 2810	**60.3** 3245	**523.9** 71540	**6.93** 2.11	**1.41** 1.41
AZ 30-750	**29.53** 750.0	**20.08** 510.0	**0.512** 13.00	**0.433** 11.00	**8.73** 184.7	**73.08** 108.80	**29.70** 145.00	**55.9** 3005	**64.8** 3485	**561.5** 76670	**6.93** 2.11	**1.41** 1.41
AZ 32-750	**29.53** 750.0	**20.12** 511.0	**0.551** 14.00	**0.472** 12.00	**9.37** 198.3	**78.44** 116.70	**31.88** 155.60	**59.5** 3200	**69.2** 3720	**599.0** 81800	**6.93** 2.11	**1.41** 1.41
AZ 36-700N	**27.56** 700	**19.65** 499	**0.591** 15.0	**0.441** 11.2	**10.20** 215.9	**79.72** 118.6	**34.71** 169.5	**66.8** 3590	**76.4** 4110	**656.2** 89610	**6.73** 2.05	**1.47** 1.47
*AZ 38-700N	**27.56** 700	**19.69** 500	**0.630** 16.0	**0.480** 12.2	**10.87** 230.0	**84.94** 126.4	**36.98** 180.6	**70.6** 3795	**81.1** 4360	**694.5** 94840	**6.73** 2.05	**1.47** 1.47
AZ 40-700N	**27.56** 700	**19.72** 501	**0.669** 17.0	**0.520** 13.2	**11.54** 244.2	**90.16** 134.2	**39.26** 191.7	**74.3** 3995	**85.7** 4605	**732.9** 100080	**6.73** 2.05	**1.47** 1.47
AZ 42-700N	**27.56** 700	**19.65** 499	**0.709** 18.0	**0.551** 14.0	**12.22** 258.7	**95.51** 142.1	**41.59** 203.1	**78.2** 4205	**90.3** 4855	**768.4** 104930	**6.75** 2.06	**1.47** 1.47
AZ 44-700N	**27.56** 700	**19.69** 500	**0.748** 19.0	**0.591** 15.0	**12.89** 272.8	**100.74** 149.9	**43.87** 214.2	**81.9** 4405	**95.0** 5105	**806.6** 110150	**6.75** 2.06	**1.47** 1.47
AZ 46-700N	**27.56** 700	**19.72** 501	**0.787** 20.0	**0.630** 16.0	**13.56** 287.0	**105.97** 157.7	**46.14** 225.3	**85.7** 4605	**99.5** 5350	**844.9** 115370	**6.75** 2.06	**1.47** 1.47
AZ 48-700	**27.56** 700.0	**19.80** 503.0	**0.866** 22.00	**0.591** 15.00	**13.63** 288.4	**106.49** 158.50	**46.37** 226.40	**88.4** 4755	**102.1** 5490	**876.2** 119650	**6.70** 2.04	**1.46** 1.46
AZ 50-700	**27.56** 700.0	**19.84** 504.0	**0.906** 23.00	**0.630** 16.00	**14.30** 302.6	**111.73** 166.30	**48.65** 237.50	**92.2** 4955	**106.7** 5735	**914.6** 124890	**6.70** 2.04	**1.46** 1.46
AZ 52-700	**27.56** 700.0	**19.88** 505.0	**0.945** 24.00	**0.669** 17.00	**14.97** 317.0	**116.97** 174.10	**50.93** 248.70	**95.9** 5155	**111.3** 5985	**953.0** 130140	**6.70** 2.04	**1.46** 1.46

*Indicates standard stocking sections. Please check with your local sales representative for material availability.

Figure D-3. J.D. Fields & Co Sheet Piling

Section	Width b	Height h	Thickness t_f	Thickness t_w	Sectional area	Mass single pile	Mass wall	Moment of inertia	Elastic section modulus	Static moment	Plastic section modulus	Class[1] S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 GP	S 500 GP
	mm	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^4/m	cm^3/m	cm^3/m	cm^3/m								
AZ®-800																			
AZ 18-800	800	449	8.5	8.5	129	80.7	**101**	41320	**1840**	1065	2135	3	3	3	3	3	4	4	4
AZ 20-800	800	450	9.5	9.5	141	88.6	**111**	45050	**2000**	1165	2330	3	3	3	3	3	3	3	4
AZ 22-800	800	451	10.5	10.5	153	96.4	**120**	48790	**2165**	1260	2525	2	2	3	3	3	3	3	3
AZ 23-800	800	474	11.5	9.0	151	94.6	**118**	55260	**2330**	1340	2680	2	2	2	3	3	3	3	3
AZ 25-800	800	475	12.5	10.0	163	102.6	**128**	59410	**2500**	1445	2890	2	2	2	2	2	3	3	3
AZ 27-800	800	476	13.5	11.0	176	110.5	**138**	63570	**2670**	1550	3100	2	2	2	2	2	2	2	3
AZ®-750																			
AZ 28-750	750	509	12.0	10.0	171	100.8	**134**	71540	**2810**	1620	3245	2	2	2	2	3	3	3	3
AZ 30-750	750	510	13.0	11.0	185	108.8	**145**	76670	**3005**	1740	3485	2	2	2	2	2	2	3	3
AZ 32-750	750	511	14.0	12.0	198	116.7	**156**	81800	**3200**	1860	3720	2	2	2	2	2	2	2	2
AZ®-700 and AZ®-770																			
AZ 12-770	770	344	8.5	8.5	120	72.6	**94**	21430	**1245**	740	1480	2	2	3	3	3	3	3	3
AZ 13-770	770	344	9.0	9.0	126	76.1	**99**	22360	**1300**	775	1546	2	2	3	3	3	3	3	3
AZ 14-770	770	345	9.5	9.5	132	79.5	**103**	23300	**1355**	805	1611	2	2	2	2	3	3	3	3
AZ 14-770-10/10	770	345	10.0	10.0	137	82.9	**108**	24240	**1405**	840	1677	2	2	2	2	2	3	3	3
AZ 12-700	700	314	8.5	8.5	123	67.7	**97**	18880	**1205**	710	1415	2	2	3	3	3	3	3	3
AZ 13-700	700	315	9.5	9.5	135	74.0	**106**	20540	**1305**	770	1540	2	2	2	3	3	3	3	3
AZ 13-700-10/10	700	316	10.0	10.0	140	77.2	**110**	21370	**1355**	800	1600	2	2	2	2	3	3	3	3
AZ 14-700	700	316	10.5	10.5	146	80.3	**115**	22190	**1405**	835	1665	2	2	2	2	2	3	3	3
AZ 17-700	700	420	8.5	8.5	133	73.1	**104**	36230	**1730**	1015	2027	2	2	3	3	3	3	3	3
AZ 18-700	700	420	9.0	9.0	139	76.5	**109**	37800	**1800**	1060	2116	2	2	3	3	3	3	3	3
AZ 19-700	700	421	9.5	9.5	146	80.0	**114**	39380	**1870**	1105	2206	2	2	2	3	3	3	3	3
AZ 20-700	700	421	10.0	10.0	152	83.5	**119**	40960	**1945**	1150	2296	2	2	2	2	2	3	3	3
AZ 24-700	700	459	11.2	11.2	174	95.7	**137**	55820	**2430**	1435	2867	2	2	2	2	2	2	3	3
AZ 26-700	700	460	12.2	12.2	187	102.9	**147**	59720	**2600**	1535	3070	2	2	2	2	2	2	2	2
AZ 28-700	700	461	13.2	13.2	200	110.0	**157**	63620	**2760**	1635	3273	2	2	2	2	2	2	2	2
AZ 36-700N	700	499	15.0	11.2	216	118.6	**169**	89610	**3590**	2055	4110	2	2	2	2	2	2	2	2
AZ 38-700N	700	500	16.0	12.2	230	126.4	**181**	94840	**3795**	2180	4360	2	2	2	2	2	2	2	2
AZ 40-700N	700	501	17.0	13.2	244	134.2	**192**	100080	**3995**	2305	4605	2	2	2	2	2	2	2	2
AZ 42-700N	700	499	18.0	14.0	259	142.1	**203**	104930	**4205**	2425	4855	2	2	2	2	2	2	2	2
AZ 44-700N	700	500	19.0	15.0	273	149.9	**214**	110150	**4405**	2550	5105	2	2	2	2	2	2	2	2
AZ 46-700N	700	501	20.0	16.0	287	157.7	**225**	115370	**4605**	2675	5350	2	2	2	2	2	2	2	2
AZ 48-700	700	503	22.0	15.0	288	158.5	**226**	119650	**4755**	2745	5490	2	2	2	2	2	2	2	2
AZ 50-700	700	504	23.0	16.0	303	166.3	**238**	124890	**4955**	2870	5735	2	2	2	2	2	2	2	2
AZ 52-700	700	505	24.0	17.0	317	174.1	**249**	130140	**5155**	2990	5985	2	2	2	2	2	2	2	2
AZ®																			
AZ 18[2]	630	380	9.5	9.5	150	74.4	**118**	34200	**1800**	1050	2104	2	2	2	3	3	3	3	3
AZ 18-10/10	630	381	10.0	10.0	157	77.8	**123**	35540	**1870**	1095	2189	2	2	2	2	3	3	3	3
AZ 26[2]	630	427	13.0	12.2	198	97.8	**155**	55510	**2600**	1530	3059	2	2	2	2	2	2	2	2

Figure D-4. ArcelorMittal Steel Sheet Piling

Table D-1. Larssen Steel Sheet Piling

Type	Width (in)	Weight		S per ft of wall (in^3/LF)	I (in^4/LF)	r (in)
		per ft of pile (Lb)	per sq ft of wall (Lb)			
SL1	14.4	13.8	11.5	2.83	4.5	1.15
SL2	17.7	21.8	14.7	5.58	14.3	1.81
SL3	17.7	25.5	17.3	10.20	40.3	2.80
SL4	17,7	31.5	21.3	15.80	77.6	3.52
31	17.7	30.2	20.5	8.55	25.3	2.05
I	15.8	26.9	20.5	9.3	27.1	2.13
II	15.8	32.8	25.0	15.8	62.2	2.91
III	15.8	41.7	31.8	25.3	123.0	3.62
IV	15.8	50.3	38.3	37.9	231.0	4.53
V	16.5	67.2	48.7	55.I	372.0	5.12
VI	16.5	82.0	59.4	78.I	673.0	6.22
IIn	15.8	32.8	25.0	20.5	109.0	3.84
IIIn	15.8	41.7	31.8	29.8	170.0	4.27
IIs	19.7	46.8	28.5	29.8	201.0	4.90
IIIs	19.7	52.1	32.4	37.2	278.0	5.41
IVs	19.7	59.I	36.1	46.5	401.0	6.16
Vs	19.7	71.2	43.4	59.5	527.0	6.43

Table D-2. Casteel USA Steel Sheet Piling

Type	Width (in)	Weight per ft of pile (Lb)	Weight per sq ft of wall (Lb)	**S** per ft of wall (in^3/LF)	**I** (in^4)	**r** (in)
CL42	21.7	15.5	8.6	2.55	4.5	1.33
CL47	21.7	17.4	9.6	2.88	5.1	1.38
CL57*	21.7	21.1	11.7	3.53	6.2	1.38
CS60*	27.6	28.2	12.3	6.98	20.6	2.36
CS76*	27.6	35.6	15.6	8.89	26.3	2.36
CU94*	23.6	37.9	19.3	10.19	39.7	2.71
CU81*	19.7	27.2	16.6	11.16	52.7	3.31
CU104	23.6	41.9	21.3	11.39	44.4	2.71
CU99	19.7	33.3	20.3	13.28	62.8	3.31
CU118	19.7	39.7	24.2	15.80	74.5	3.31
CU110*	22.7	42.5	22.5	21.39	149.0	4.76
CU116*	22.7	44.9	23.8	22.32	158.2	4.76
CU122	22.7	47.2	25.0	23.25	164.8	4.76
CZ84*	21.7	31.1	17.2	13.62	53.6	3.27
CZ95	21.7	35.2	19.5	15.53	61.2	3.27
CZ107*	21.7	39.6	21.9	17.48	68.8	3.27
CZ113	21.7	41.7	23.1	18.40	72.7	3.27
CZ114	24.0	46.8	23.4	31.62	211.6	5.55
CZ128	24.0	52.3	26.2	35.34	236.5	5.55
CZ141	24.0	57.9	28.9	39.06	261.4	5.55
CZ148	24.0	60.7	30.3	40.92	273.9	5.55

* Note: Supplied only by special arrangement with the mill.

D-3 L.B. Foster Piling Products

D-3.01 Domestically produced by Gerdau Ameristeel and delivered by L.B. Foster

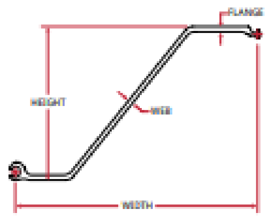
L.B. Foster Piling and Gerdau Ameristeel work together as strategic partners to provide the cost savings of innovative PZC sheet piling to customers throughout North America. (Note: it appears that L.B. Foster's steel Piling Products line of business was sold to J.D. Fields & Company, Inc. in September, 2021). Some of these innovations include:

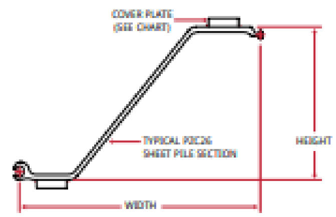
1. Wider sections maximize production.
2. Lighter sections minimize required steel.
3. Stronger sections increase strength per pound.
4. Superior ball & socket interlock.
5. PZC™ is a registered trademark of Gerdau Ameristeel.

Figures D-5 through D-7 illustrate various PZC sheet piling properties; Table D-3 illustrates their high-efficiency products, and Table D-4 illustrates their heavy-duty sheet pile products.

LBFoster
Piling

Z Pile Profile





Cover Plated Z Profile

Section	Width+	Height+	Web Thick-ness+	Flange Thick-ness+	Weight		Moment of Inertia		Section Modulus		Nominal Coating Area*
	in.	in.	in.	in.	lb / lft	lb / ft2	in4	in4 / wft	in3	in3 / wft	ft2 / lft
	mm	mm	mm	mm	kg / lm	kg / m2	cm4	cm4 / wm	cm3	cm3 / wm	m2 / lm
PZC 13	**27.88**	**12.56**	**0.375**	**0.375**	**50.4**	**21.7**	**353.0**	**152.0**	**56.2**	**24.2**	**5.60**
	708	**319**	**9.5**	**9.5**	**75.1**	**106.0**	**14,690**	**20,760**	**920**	**1,300**	**1.71**
PZC 14	27.88	12.60	0.420	0.420	55.0	23.7	381.6	164.3	60.5	26.0	5.60
	708	320	10.7	10.7	81.8	115.5	15,890	22,440	990	1,400	1.71
PZC 18	**25.00**	**15.25**	**0.375**	**0.375**	**50.4**	**24.2**	**532.2**	**255.5**	**69.8**	**33.5**	**5.60**
	635	**387**	**9.5**	**9.5**	**75.1**	**118.2**	**22,150**	**34,890**	**1,145**	**1,800**	**1.71**
PZC 19	25.00	15.30	0.420	0.420	55.0	26.4	576.3	276.6	75.3	36.1	5.60
	635	388	10.7	10.7	81.8	128.8	23,990	37,780	1,235	1,945	1.71
PZC 25	27.88	17.66	0.485	0.560	69.4	29.9	938.7	404.1	106.3	45.7	6.15
	708	449	12.3	14.2	103.3	145.9	39,070	55,190	1,740	2,455	1.87
PZC 26	**27.88**	**17.70**	**0.525**	**0.600**	**73.9**	**31.8**	**994.3**	**428.1**	**112.4**	**48.4**	**6.15**
	708	**450**	**13.3**	**15.2**	**110.0**	**155.4**	**41,390**	**58,460**	**1,840**	**2,600**	**1.87**
PZC 28	27.88	17.75	0.570	0.645	79.0	34.0	1,057	455.1	119.1	51.3	6.15
	708	451	14.5	16.4	117.6	166.1	44,000	62,150	1,950	2,755	1.87

Available Grades: ASTM A572 Gr. 50 and 60, A588 and A690
+Values stated are nominal
*Both sides of sheet: excludes socket interior and ball interlock

PZC™ is a trademark of Gerdau

Section	Normal Width	Plate Size	Per Single Section				Per Unit of Wall			
			Area	Weight	Total Surface Area	Nominal Coating Area*	Weight		Moment of Inertia	Section Modulus
							Full Length Plates	Half Length Plates		
	in.	in.	in2	lb / ft	ft2 / lin ft	ft2 / lin ft	lb / ft2	lb / ft2	in4 / ft	in3 / ft
	mm	mm	mm2	kg / m	m2 / m	m2 / m	kg / m2	kg / m2	cm4 / m	cm3 / m
PZC 38-CP (PZC26)	**27.88**	**3.5 x 1.00**	**28.72**	**97.7**	**6.98**	**5.48**	**42.1**	**36.9**	**691.4**	**70.2**
	708		185.3	145.3	2.13	1.67	205.6	180.2	94,420	3,770
PZC 39-CP (PZC26)	**27.88**	**3.5 x 1.125**	**29.60**	**100.6**	**7.03**	**6.53**	**43.3**	**37.6**	**728.3**	**73.0**
	708		191.0	149.7	2.14	1.99	211.4	183.6	99,460	3,930
PZC 41-CP (PZC26)	**27.88**	**3.5 x 1.25**	**30.47**	**103.6**	**7.07**	**6.57**	**44.6**	**38.2**	**766.1**	**75.8**
	708		196.6	154.2	2.15	2.00	217.8	186.5	104,600	4,080

*Excludes socket interior and ball interlock
- Filet weld should be sized to adequately resist design loads and should be continuous and all around.
- Cover plate length depends upon bending moment diagram. Best economy is obtained when plate length is limited to area of high moment.

Available Grades: ASTM A572 Gr. 50

Figure D-5. PZC Sheet Pile Section Properties

D-3.02 High Efficiency "Z" Section Steel Sheet Pile

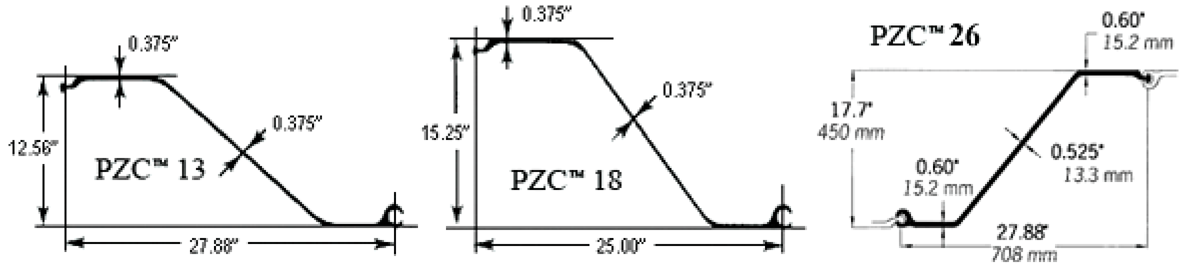


Figure D-6. Cross Sections of Various PZC Sheet Piles

Table D-3. Tabulated Data for Various PZC Sheet Piles

					Per Foot of Wall			
Section Designation	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight Per Foot	Moment of Inertia	Section Modulus
	in.	in.	in.	in.	in^2/ft	lbs/ft^2	in^4/ft	in^3/ft
PZC™12	27.88	12.52	0.335	0.335	5.87	20.0	140.6	22.4
PZC™13	27.88	12.56	0.375	0.375	6.38	21.7	152.0	24.2
PZC™14	27.88	12.60	0.420	0.420	6.95	23.7	164.8	26.0
PZC™17	25.00	15.21	0.335	0.335	6.55	22.3	236.6	31.0
PZC™18	25.00	15.25	0.375	0.375	7.12	24.2	255.5	33.5
PZC™19	25.00	15.30	0.420	0.420	7.76	26.4	276.7	36.1
PZC™26	27.88	17.70	0.525	0.600	9.35	31.8	428.1	48.4
All Dimensions Are Nominal								

D-3.03 Heavy Duty "Z" Section Sheet Pile

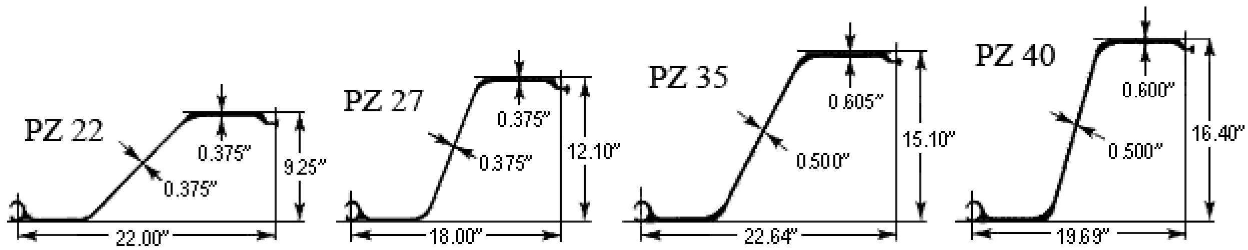


Figure D-7. Cross Sections of Various Heavy Duty PZ Sheet Piles

Table D-4. Tabulated Data for Various Heavy Duty PZ Sheet Piles

					Per Foot of Wall			
Section Designation	Nominal Width	Wall Depth (Height)	Web Thickness	Flange Thickness	Area	Weight Per Foot	Moment Of Inertia	Section Modulus
	in.	in.	in.	in.	in.^{2}/ft	lbs/ft^2	in.^{4}/ft	in.^{3}/ft
PZ 22	22	9.25	0.375	0.375	6.65	22.6	85.1	18.4
PZ 27	18	12.1	0.375	0.375	8.13	27.7	187.3	31
PZ 35	22.64	15.1	0.5	0.605	10.28	35	369.4	48.9
PZ 40	19.69	16.4	0.5	0.6	11.75	40	502.7	61.3
All Dimensions Are Nominal								