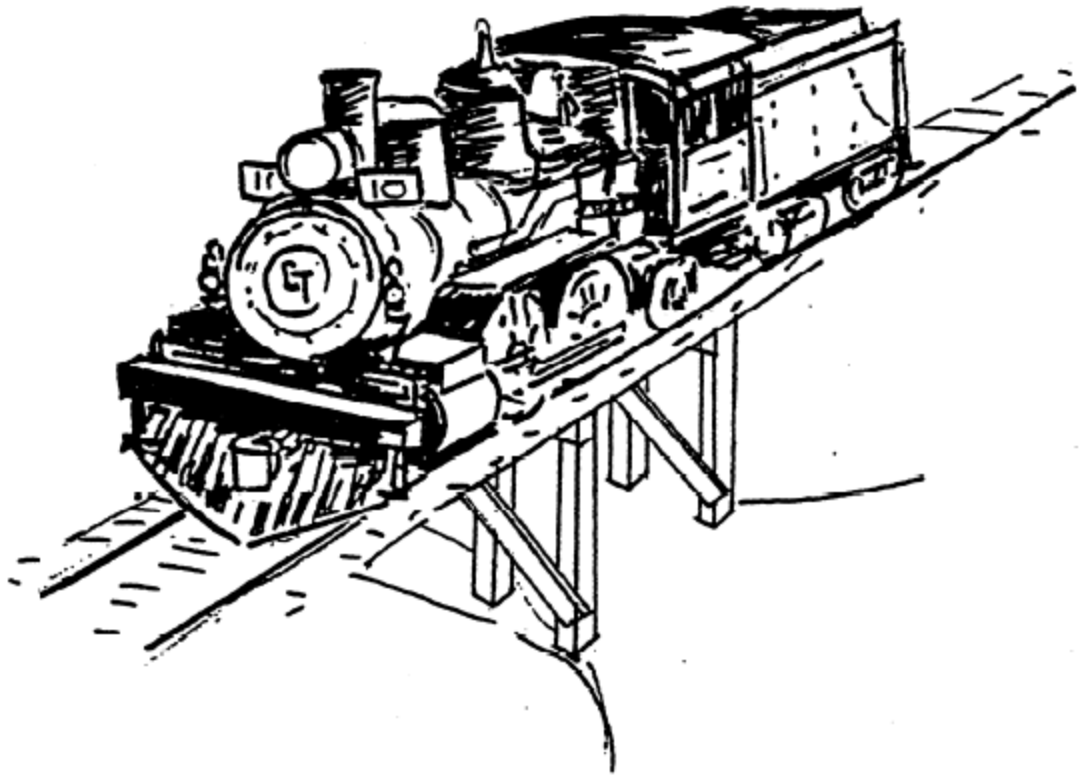


CHAPTER 8

RAILROAD



8.0 INTRODUCTION

Shoring adjacent to railroads present 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, SC engineers may encounter other railways such as light rail and commuter trains like Bay Area Rapid Transit (BART). For these other railways, it is acceptable to use the same guidelines presented here unless there are specific instructions from the concerned railway.

This chapter is developed using the UPRR General Shoring Requirements and the Guidelines for Temporary Shoring published by BNSF and UPRR in 2004. The Guidelines were designed as a supplement to the 2002 American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual of Recommended Practice. When reviewing shoring that encroaches on railroad right-of-way, always ensure that the most current editions of both documents are being used. When the railroad requirements conflict with Caltrans or OSHA specifications, always use the more conservative specification.

Standard Specifications Section 19-1.02, “Preservation of Property,” requires excavation plans to be submitted at least 9 weeks prior to beginning of construction. As well as meeting the requirements of the Standard Specifications, contracts with Railroad involvement will include a section in the Special Provisions titled “Railroad Relations and Insurance,” typically Section 13. Section 13 will include general requirements for the design and construction of temporary shoring and provide reference to additional information and requirements.

The 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 for Temporary Shoring.” Submissions of the Plans and Calculations to the Railroad are to be routed through the Offices of Structure Construction Headquarters in Sacramento (OSC HQ) in accordance with BCM 122-1.0. The OSC HQ will be the Engineer’s single point of contact with the Railroad through the submittal phase. The Railroad may take 6 weeks or more to review the shoring plans and calculations. 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 2 locomotives with 80 kips per axle pulling an infinite train of 8 kips per foot. The lateral pressure from the loading will be determined using the Boussinesq Strip Loading procedure. Since the live loading is considered to be dynamic, use of wall friction in the earth pressure calculations will not be allowed above

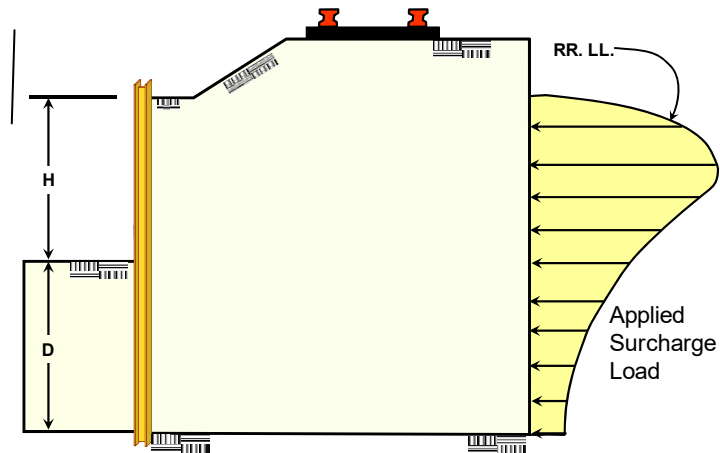


Figure 8-1. Railroad Boussinesq Strip Load

the dredge line. When using the railroad (RR) live load (LL) curves, the plot of the curve always starts at the elevation of the top of the shoring system as shown in Figure 8-1.

8.1 **SELECTED EXCERPTS FROM “Guidelines for Temporary Shoring, Published October 25, 2004, BNSF/UPRR” (GTS)**

8.1.1 **Scope (GTS section 1, p1)**

These guidelines are developed to inform public agencies, design engineers, contractors and inspectors of current Railroad standards and requirements concerning the design and construction of temporary shoring. The temporary shoring addressed below can be used for all locations where the Railroad operates regardless of track ownership. For any items not covered in this CT Shoring Manual, please refer to the Guidelines for Temporary Shoring as published by BNSF and/or UPRR and the AREMA Manual. Throughout the entire construction, all personnel, railroad tracks, and property need to be protected to ensure the safety and economy of the project.

8.1.2 **General Criteria (GTS section 2, p1 - 2)**

The contractor must not begin construction of any component of the shoring system affecting the Railroad right-of-way until written Railroad approval has been received.

1. All excavations shall be in compliance with applicable OSHA regulations and shall be shored where there is any danger to tracks, structures or personnel regardless of depth.

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2. Contractor is responsible for planning and executing all procedures necessary to construct, maintain and remove the temporary shoring system in a safe and controlled manner.
3. Emergency Railroad phone numbers are to be obtained from the Railroad representative in charge of the project prior to the start of any work and shall be posted at the job site.
4. Contractor must obtain a valid right of entry permit from the Railroad and comply with all railroad requirements when working on Railroad property.
5. The Contractor is required to meet minimum safety standards as defined by the Railroad.
6. All temporary shoring systems that support or impact the Railroad's tracks or operations shall be designed and constructed to provide safe and adequate rigidity.
7. The Railroad requirements, construction submittal review times and review criteria should be discussed at the pre-construction meeting with the Contractor.
8. A flagman is required when any work is performed within 25 feet of track centerline. If the Railroad provides flagging or other services, the Contractor shall not be relieved of any responsibilities or liabilities as set forth in any document authorizing the work. No work is allowed within 50 feet of track centerline when a train passes the work site and all personnel must clear the area within 25 feet of track centerline and secure all equipment when trains are present.
9. Appropriate measures for the installation and protection of fiber optic cables shall be addressed in the plans and contract documents. For specific Railroad requirements and additional information refer to:
www.bnsf.com or call 1-800-533-2891.
www.uprr.com, call 1-800-336-9163 or refer to UPRR Fiber Optic Engineering, Construction and Maintenance Standards.
- 10 Relocation of utilities or communication lines not owned by the Railroad shall be coordinated with the utility owners. The utility relocation plans must then be submitted to the Railroad utility representative for approval. The shoring plans must include the correct contact for the Railroad, State or Local utility locating service

provider. The Railroad will not be responsible for costs associated with any utility, signal or communication line relocation or adjustments.

8.1.3 Types of Temporary Shoring (GTS section 5, p5)

8.1.3.1 Shoring Box

A shoring box is considered a prefabricated system and is not accepted by the Railroad. The shoring system is installed as the excavation progresses. The system can be used, however, only in special applications when the Railroad live load surcharge is not present. During excavation, the shoring box is moved down by gravity or by applying vertical loading from excavation equipment.

8.1.3.2 Restrained Systems

Restrained systems are comprised of vertical elements, (continuous sheet piles or discrete soldier piles with lagging) and horizontal elements (braces or tiebacks). Restrained systems are designed to provide lateral support for the soil mass supporting the Railroad and derives their stability from the passive resistance of the vertical structural element against soil below the excavation line and the horizontal components of the anchored or braced elements.

Restrained systems with tiebacks are discouraged by the Railroad. The tiebacks become an obstruction to future utility installations and may also damage existing utilities. All tiebacks must be removed per Railroad requirements. Tiebacks must be designed, furnished, installed, tested and stressed in accordance with AREMA requirements.

8.1.3.3 Unrestrained Systems

Unrestrained systems are comprised of only vertical elements, (continuous sheet piles or discrete soldier piles with lagging). Unrestrained systems are designed to provide lateral support for the soil mass supporting the Railroad and derive their stability solely from the passive resistance of the vertical structural element against soil below the excavation line.

8.1.3.4 Cofferdam

A cofferdam is designed to keep water and soil out of an excavation. This enclosed temporary structure helps with the construction of a permanent structure, such as a bridge

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pier or abutment or similar structure. Cofferdams are usually constructed out of timber, steel, concrete, or a combination of any of these materials. In most cases, the guidelines designate cofferdams to be constructed with steel sheet piles.

8.1.4 General Shoring Requirements (GTS section 6, p5 - 7)

For general shoring requirements and specific applications of the following items refer to Figure 8-2. The general requirements per the Guidelines for Temporary Shoring are described below:

1. No excavation shall be permitted closer than 12'-0" measured at a right angle from the centerline of track to the trackside of shoring system. If existing conditions preclude the installation of shoring at the required minimum distance, the shifting of tracks or temporary removal of tracks shall be investigated prior to any approval. All costs associated with track shifting or traffic interruption shall be at Contractor's expense.
2. Evaluate slope and stability conditions to ensure the Railroad embankment will not be adversely affected. Local and global stability conditions must also be evaluated.
3. All shoring within the limits of Zone A or Zone B must be placed prior to the start of excavation.
4. Lateral clearances must provide sufficient space for construction of the required ditches parallel to the standard roadbed section. The size of ditches will vary depending upon the flow and terrain and should be designed accordingly.
5. The shoring system must be designed to support the theoretical embankment shown in zones A and B.
6. Any excavation, holes, or trenches on the Railroad property shall be covered, guarded and/or protected. Handrails, fence, or other barrier methods must meet OSHA and Federal Railroad Administration (FRA) requirements. Temporary lighting may also be required by the Railroad to identify tripping hazards to train crewmen and other Railroad personnel.
7. The most stringent project specifications of the Public Utilities Commission Orders, Department of Industrial Safety, OSHA, FRA, AREMA, BNSF, UPRR or other governmental agencies shall be used.

8. Secondhand material is not acceptable unless the Engineer of Record submits a full inspection report that verifies the material properties and condition of the secondhand material. The report must be signed and sealed by the Engineer of Record.
9. All components of the shoring system are to be removed when the shoring is no longer needed. All voids must be filled and drainage facilities restored.
10. Slurry type materials are not acceptable as fill for soldier piles in drilled holes. Concrete and flowable backfill may prevent removal of the shoring system. Use compacted pea gravel material.

8.1.5 Information Required (GTS section 4, p3 - 4)

Plans and calculations shall be submitted signed and stamped by a Registered Professional Engineer familiar with Railroad loadings and who is licensed in the state where the shoring system is intended for use. Information shall be assembled concerning right-of-way boundary, clearances, proposed grades of tracks and roads, and all other factors that may influence the controlling dimensions of the proposed shoring system.

8.1.5.1 Field Survey

Sufficient information shall be shown on the plans in the form of profiles, cross sections and topographical maps to determine general design and structural requirements. Field survey information of critical or key dimensions shall be referenced to the centerline of track(s) and top of rail elevations. Existing and proposed grades and alignment of tracks and roads shall be indicated together with a record of controlling elevation of water surfaces or ground water. Show the location of existing/proposed utilities and construction history of the area that might hamper proper installation of the piling, soldier beams, or ground anchors.

8.1.5.2 Geotechnical Report

- a. Elevation and location of soil boring in reference to the track(s) centerline and top of rail elevations.
- b. Classification of all soils encountered.
- c. Internal angle of soil friction
- d. Dry and wet unit weights of soil.

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- e. Active and passive soil coefficients, pressure diagram for multiple soil strata.
- f. Bearing capacity and unconfined compression strength of soil.
- g. Backfill and compaction recommendations.
- h. Optimum moisture content of fill material.
- i. Maximum density of fill material.
- j. Minimum recommended factor of safety.
- k. Water table elevation on both sides of the shoring system.
- l. Dewatering wells proposed flownets or zones of influence.
- m. In seismic areas, evaluation of liquefaction potential of various soil strata.

8.1.5.3 Loads

All design criteria, temporary and permanent loading must be clearly stated in the design calculations and on the contract and record plans. Temporary loads include, but are not limited to: construction equipment, construction materials and lower water levels adjoining the bulkhead causing unbalanced hydrostatic pressure. Permanent loads include, but are not limited to: future grading and paving, Railroads or highways, structures, material storage piles, snow and earthquake. The allowable live load after construction should be clearly shown in the plans and painted on the pavements behind the bulkheads or shown on signs at the site and also recorded on the record plans. Some of the loads are:

- a. Live load pressure due to E80 loading for track parallel to shoring system.
- b. Live load pressure due to E80 loading for track at right angle to shoring system.
- c. Other live loads.
- d. Active earth pressure due to soil.
- e. Passive earth pressure due to soil.
- f. Active earth pressure due to surcharge loads.
- g. Active pressure due to sloped embankment.
- h. Dead load.
- i. Buoyancy.
- j. Longitudinal force from live load.
- k. Centrifugal forces.
- l. Shrinkage.
- m. Temperature.

- n. Earthquake.
- o. Stream flow pressure.
- p. Ice pressure.

8.1.5.4 Drainage (AREMA 8.20.2.4)

- a. The drainage pattern on the site before and after construction should be analyzed and adequate drainage provisions should be incorporated into the plans and specifications. Consideration should be given to groundwater as well as surface drainage.
- b. Drainage provisions for backfill should be compatible with the assumed water conditions in design.

8.1.5.5 Structural Design Calculations

- a. List all assumptions used to design the temporary shoring system.
- b. Determine E80 live load lateral pressure using the Boussinesq strip load equation.
- c. Computerized calculations and programs must clearly indicated the input and output data. List all equations used in determining the output.
- d. Example calculations with values must be provided to support computerized output and match the calculated computer result.
- e. Provide a simple free body diagram showing all controlling dimensions and applied loads on the temporary shoring system.
- f. Calculated lateral deflections of the shoring and effects to the rail system must be included. Include the elastic deflection of the wall as well as the deflection due to the passive deflection of the resisting soil mass.
- g. Documents and manufacturer's recommendations that support the design assumptions must be included with the calculations.

8.1.5.6 Computation of Applied Forces (GTS section 7, p7 - 8)

Below are all the applied forces that need to be taken into consideration when designing for a Railroad system.

- 1. Railroad live and lateral forces.

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- a. For specific applications of the Coopers E80 live load refer to Figure 8-3 and Figure 8-4.
2. Dead Load.
 - a. Spoil pile: must be included assuming a minimum height of two feet of soil adjacent to the excavation.
 - b. Track: use 200 lbs/linear ft for rails, inside guardrails and fasteners.
 - c. Roadbed: ballast, including track ties, use 120 lb per cubic foot.
3. Active and passive earth pressures.
 - a. The active and passive earth pressures may be computed by any approved method.
4. Active earth pressure due to unbalanced water pressure.
 - a. When bulkheads are used for waterfront construction, the bulkhead is subjected to a maximum earth pressure at the low water stage. During a rainstorm or a rapidly receding high water, the water level behind the bulkhead may be several feet higher than in front of the bulkhead.
 - b. Drained conditions in backfill apply when clean sand or clean sand and gravel are used and adequate permanent drainage outlets are provided. Where drained conditions exist, the design water level may be assumed at the drainage outlet elevation.
5. Pressure due to embankment surcharges.
 - a. Conventional analysis should be used to determine the additional surcharge from embankment slope.
6. Additional analysis for centrifugal force calculations as described in the AREMA Manual is required where track curvature exceeds three degrees.
7. Include and compute all other loads that are impacting the shoring system such as a typical Railroad service vehicle.

8.1.5.7 Structural Integrity (GTS section 8, p9 - 10)

Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the loads and forces in such combinations as stipulated in the AREMA Manual, which represents various combinations of loads and forces to which a structure may be subjected. Each part of the

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structure shall be proportioned for the group loads that are applicable, and the maximum design required shall be used.

1. Embedment depth.
 - a. Calculated depth of embedment is the embedment depth required to maintain static equilibrium.
 - b. Minimum depth of embedment is the total depth of embedment required to provide static equilibrium plus additional embedment due to the minimum factor of safety.
 1. Embedment depth factor of safety for well-defined loading conditions and thoroughly determined soil parameters is generally 1.3 for most temporary shoring systems.
 2. All anchored shoring systems require a minimum embedment depth of 1.5 times the calculated depth of embedment. Shallow penetration into strong soil layers is not acceptable.
2. The allowable stresses based on AREMA requirements are as follows:

Structural Steel:

 - 0.55F_y for compression in the extreme fiber. (AREMA Ch.15 Table 1-11)
 - 0.35F_y for shear. (AREMA Ch.15 Table 1-11)

Sheet Pile Sections: 2/3 of yield strength for steel. (AREMA 8.20.5.7)

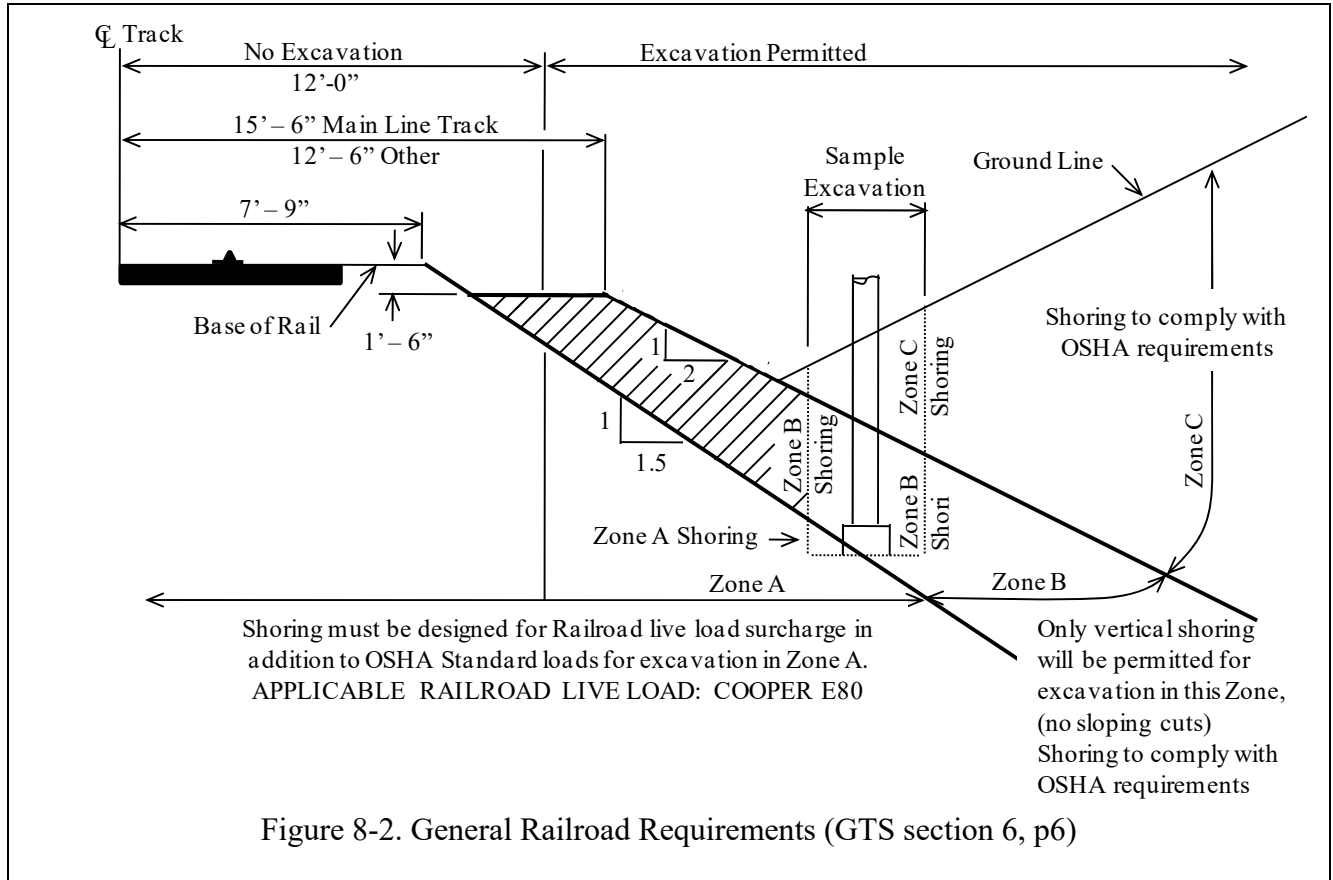
Concrete: 1/3 of compressive strength. (AREMA 8.20.5.7)

Anchor Rods: ½ of yield strength for steel. (AREMA 8.20.5.7)
3. AISC allowances for increasing allowable stress due to temporary loading conditions are not acceptable.
4. Gravity type temporary shoring systems must also be analyzed for overturning, sliding and global stability.
5. Calculated deflections of temporary shoring system and top of rail elevation shall not exceed the criteria outlined in Table 8-1 Deflection Criteria.

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Table 8-1. 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
$12' < S < 18'$	3/8"	1/4"
$18' < S < 24'$	1/2"	1/4"



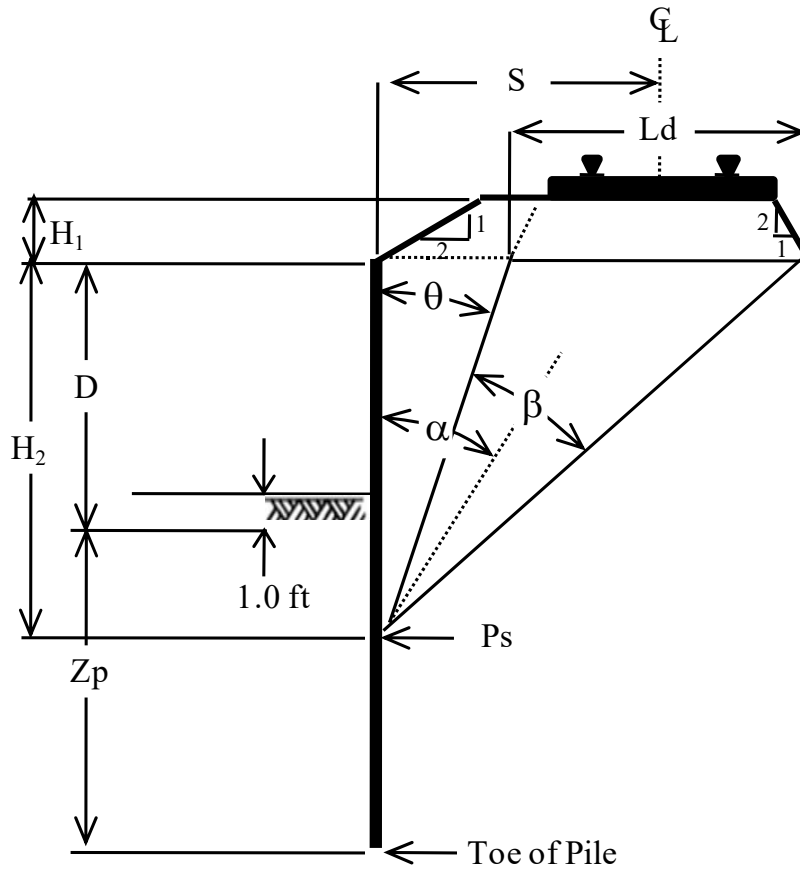


Figure 8-3. Live Load Pressure due to Cooper E80 (GTS section 7, p8)

Vertical Pressure, q , shall be based on distribution width L_d .

Where:

L_d = Length of the tie plus H_1 .

H_1 = Height from the bottom of tie to the top of shoring

H_2 = Depth of point being evaluated with Boussinesq equation

S = The distance perpendicular from centerline of track to the face of shoring

D = The distance from top of shoring to one foot below dredge line.

Z_p = The minimum embedment depth

q = The intensity of strip load due to E80 Railroad live load and can be calculated as follows:

For $H_1 = 0$, L_d = Length of Tie or

For $H_1 > 0$, L_d = Length of Tie + H_1

$$q = \frac{80,000}{5 \text{ ft}(L_d)}$$

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Case 1: Lateral live load pressure P_s , due to E80 loading for track parallel to shoring system is calculated using the Boussinesq Strip Load Equation

$$P_s = \frac{2q}{\pi} (\beta + \sin \beta \sin^2 \alpha - \sin \beta \cos^2 \alpha) = \frac{2q}{\pi} (\beta - \sin \beta \cos(2\alpha))$$

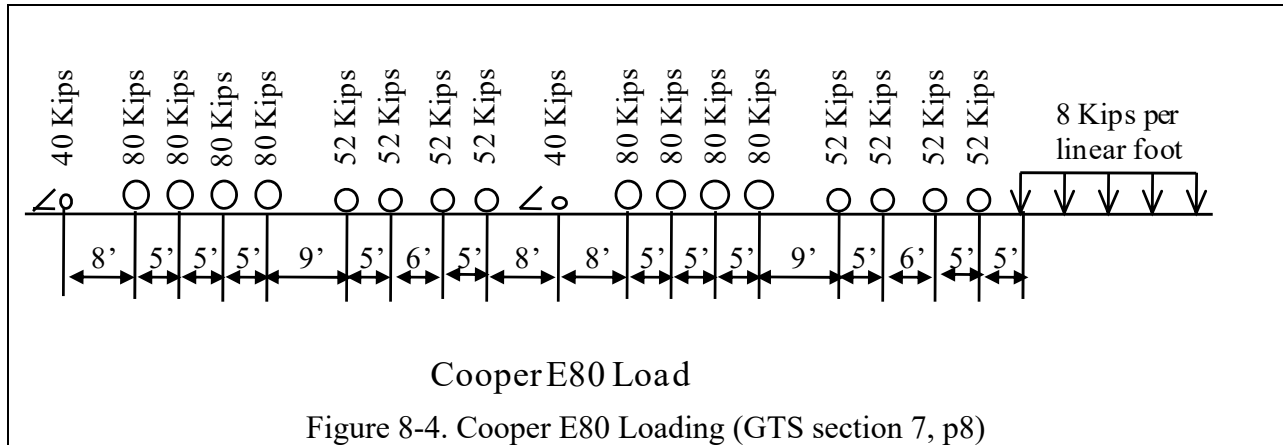
Where α and β are angles measured in radians,

$$\alpha = \theta + \frac{\beta}{2}$$

Case 2: Live load pressure due to E80 loading for track at a right angle to the shoring system can be calculated using the following equation:

$$P_s = K_a q$$

Where K_a is the active earth pressure coefficient.



8.2 EXAMPLE 8-1 (Railroad Example)

Check a temporary shoring system adjacent to the railroad shown below.

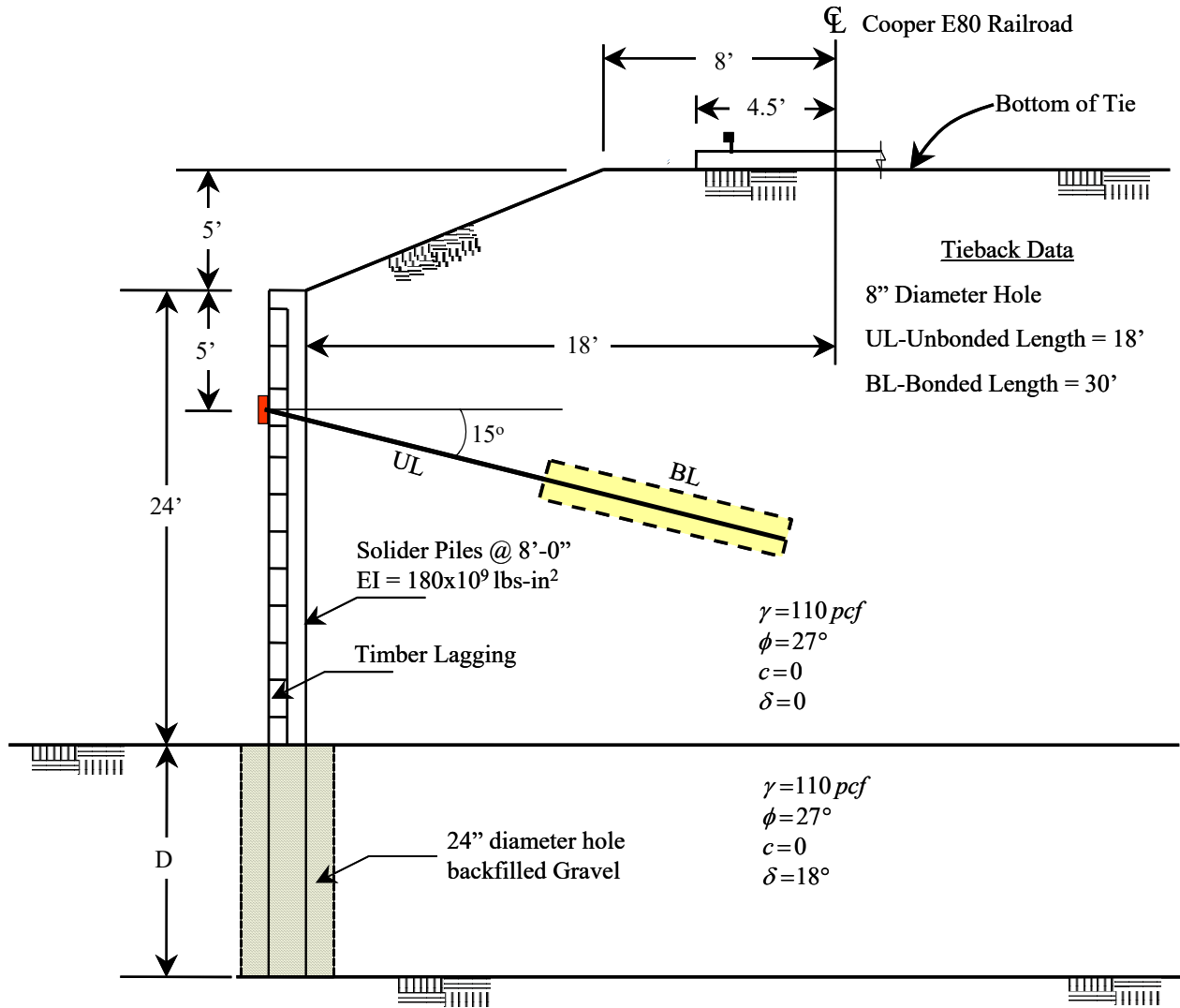


Figure 8-5. EXAMPLE 8-1

Determine:

1. Active & Passive Earth Pressures.
2. Pile Embedment D per Section 8.1.5.7 of this chapter.
3. Tieback Load with FS = 1.0.
4. Check the deflection of the shoring system per Railroad requirements.

STEP 1: Develop the Pressure Diagram

The appropriate pressure diagram should be broken down into diagrams: above the excavation line, below the excavation line, and the Railroad surcharge load.

For pressure Diagram above the excavation line ($H = 24$ feet and $\delta = 0^\circ$ due to vibrations from the RR in which case wall friction is ignored):

Calculate active earth pressure above excavation line using Trial Wedge Method formulation shown below.

$$P_A = \frac{W[\tan(\alpha - \phi)]}{[1 + \tan \delta \tan(\alpha - \phi)] \cos \delta} \quad (\text{Eq. 4-42})$$

The final wedge is shown below with the wedge angle of 55.92 degrees.

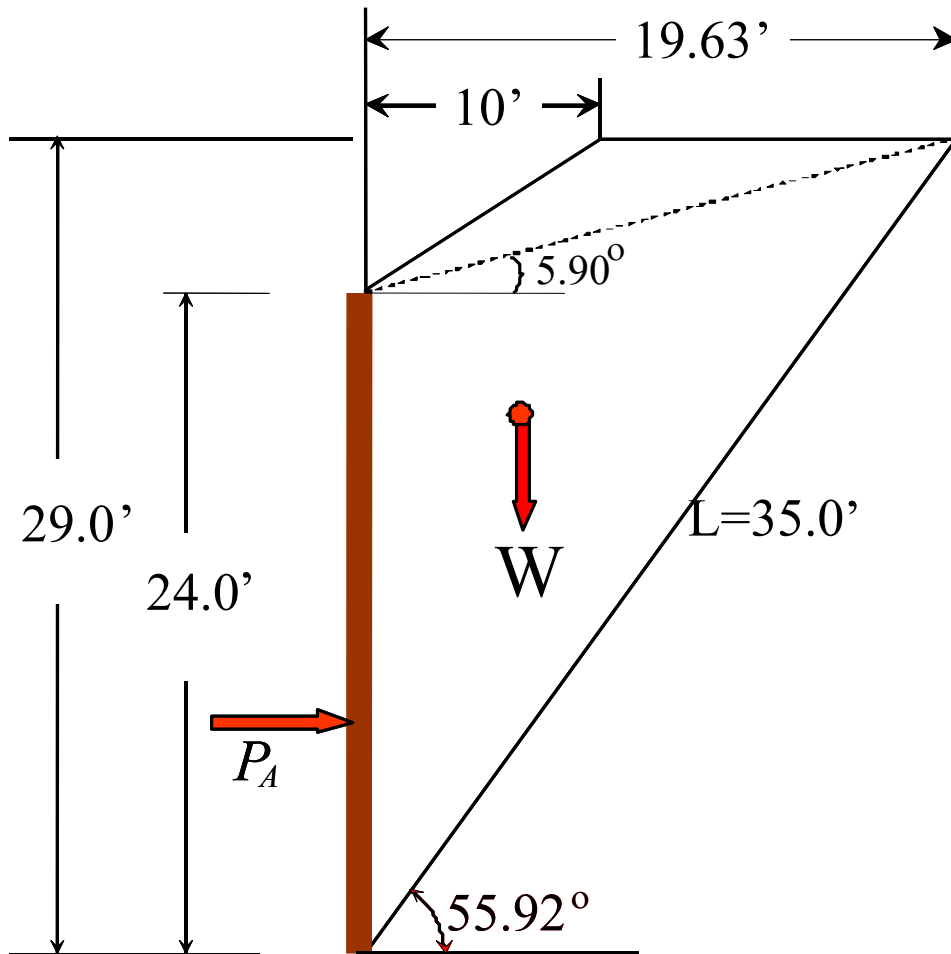


Figure 8-6. Final Trial Wedge For EXAMPLE 8-1

$$y_1 = 29.0 \text{ ft}$$

$$y_2 = 24.0 \text{ ft}$$

$$x_1 = 10.0 \text{ ft}$$

$$x_2 = 19.6 \text{ ft}$$

$$L = 35.0 \text{ ft}$$

$$Area = \frac{y_1(x_2 - x_1) + y_2x_1}{2} = \frac{29.0(19.63 - 10.0) + 24.0(10.0)}{2} = 259.64 \text{ ft}^2 / \text{ft}$$

$$W = A\gamma = \frac{259.64(110)}{1000} = 28.56 \text{ klf}$$

$$P_A = \frac{W[\tan(\alpha - \phi)]}{[1 + \tan \delta \tan(\alpha - \phi)] \cos \delta} = \frac{28.56 \tan(55.92 - 27)}{[1 + \tan(0)\tan(55.92 - 27)] \cos(0)} \approx 15.8 \text{ klf}$$

The P_A developed in the above equation will be used to determine the pressure diagram above excavation. (Use this P_A to determine trapezoidal load.)

$$\sigma_{Trapezoid} = \frac{1.3P_A}{\frac{2}{3}H} = \frac{1.3(15,800)}{\frac{2}{3}(24)} = 1,283.75 \text{ psf} \quad (\text{Eq. 7-1 and Eq. 7-2})$$

For pressure diagram below the excavation line ($H > 24$ feet):

The horizontal active earth pressure coefficient (Eq. 4-20) and horizontal passive earth pressure coefficient (Eq. 4-22) are determined using Coulomb's Earth Pressure theory. Please note that the earth pressure coefficient using the Coulomb method or the log spiral methods alluded to in CHAPTER 4 is similar since the soil friction angle is low.

$$k_{ah} = \frac{\cos^2(\phi)}{\cos(\delta) \left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi)}{\cos(\delta)}} \right]^2} \cos(\delta) = \frac{\cos^2(27)}{\cos(18) \left[1 + \sqrt{\frac{\sin(45)\sin(27)}{\cos(18)}} \right]^2} \cos(18) = 0.318$$

$$k_{ph} = \frac{\cos^2(\phi)}{\cos(\delta) \left[1 - \sqrt{\frac{\sin(\delta + \phi)\sin(\phi)}{\cos(\delta)}} \right]^2} \cos(\delta) = \frac{\cos^2(27)}{\cos(18) \left[1 - \sqrt{\frac{\sin(45)\sin(27)}{\cos(18)}} \right]^2} \cos(18) = 4.521$$

Lateral load distribution at excavation line:

$$\sigma_{a1} = \gamma(H = 24 \text{ ft})k_a = 110(24)(0.318) = 839.52 \text{ psf}$$

Lateral load distribution at D ft below excavation line:

$$\sigma_{a2} = \sigma_{a1} + \gamma(D)k_a = 839.52 + 110(D)(0.318) = (839.52 + 34.98D) \text{ psf}$$

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Calculate passive earth pressure at D ft below excavation line:

$$\sigma_{p1} = \gamma (D) k_p = 110 (D)(4.521) = 497.31D \text{ psf}$$

Surcharge Load: See Section 4.8.1 of CHAPTER 4 for a discussion on the minimum surcharge load. In this example, a minimum surcharge of 72 psf must be applied at the top the shoring system. The Engineer is reminded to verify the governing surcharge for all scenarios. In addition, the Boussinesq load will be applied to the entire depth of the shoring system. The application of the surcharge load also begins at the top of the shoring system.

Surcharge based on E80 Cooper Load =

$$q_s = \frac{\text{Axle Load}}{(\text{Axle Spacing})(\text{Track} + H1)} = \frac{80,000}{(5)(9 + 5)} = 1142.86 \text{ psf}$$

Axle Load: Maximum load per Railroad Axle in lbs. (See Cooper E80 Load Figure 8-4)

Axle Spacing: Minimum distance of spacing between Railroad Axles in feet. (See Cooper E80 Load Figure 8-4)

Track: Length of Railroad Tie in feet. (See problem statement)

H1: Height of backfill slope between bottom of tie and top of retaining system in feet. Per code 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. Below shows a sample calculation to determine the Boussinesq Load at a depth of 5 ft:

$$\sigma_h = 2Q_s \frac{\beta_R - \sin \beta \cos 2\alpha}{\pi}$$

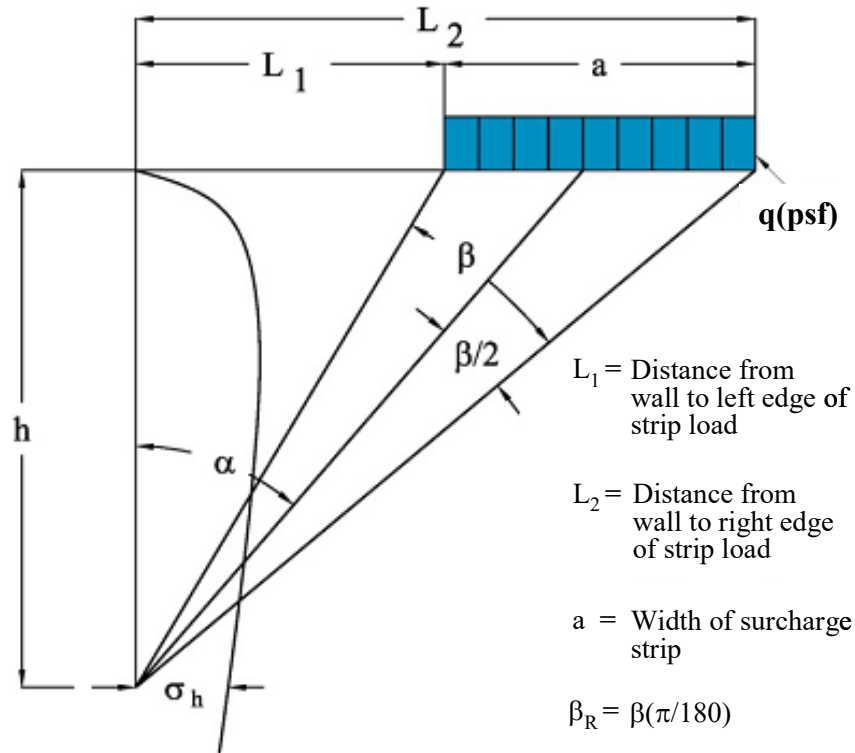


Figure 8-7. Boussinesq Type Strip Load for Railroad

$$q_s = 1,142.86 \text{ psf}$$

$$\beta = \sin^{-1}\left(\frac{L_2}{\sqrt{L_2^2 + h^2}}\right) - \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) = \sin^{-1}\left(\frac{22.5}{23.05}\right) - \sin^{-1}\left(\frac{13.5}{14.40}\right) = 7.79^\circ$$

$$\alpha = \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) + \frac{1}{2}\beta = \sin^{-1}\left(\frac{13.5}{14.40}\right) + \frac{1}{2}(7.79^\circ) = 73.57^\circ$$

$$\beta_R = \beta\left(\frac{\pi}{180}\right) = 7.79^\circ\left(\frac{\pi}{180}\right) = 0.14$$

$$\sigma_h = 2(1,142.86) \frac{0.14 - \sin(7.79^\circ)\cos(2 * 73.57^\circ)}{\pi} = 181.87 \approx 182 \text{ psf}$$

The above procedure is used to determine Boussinesq loads at specific intervals, keep in mind that for the upper 10 ft of the shoring system the minimum surcharge load is 72 psf. For the moment arms, each is assumed to be in the middle of the trapezoids. Table 8-2 below displays Boussinesq loads at various intervals below the top of temporary retaining system (not below the railroad tie):

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Table 8-2. Boussinesq loads at various depths

Depth (ft)	Load (psf)	Location
0	72	Top of shoring
5	182	
10	238	
15	208	
24	123	Dredge line
34	65*	Bottom of shoring

The General Pressure Diagram is shown below in Figure 8-8:

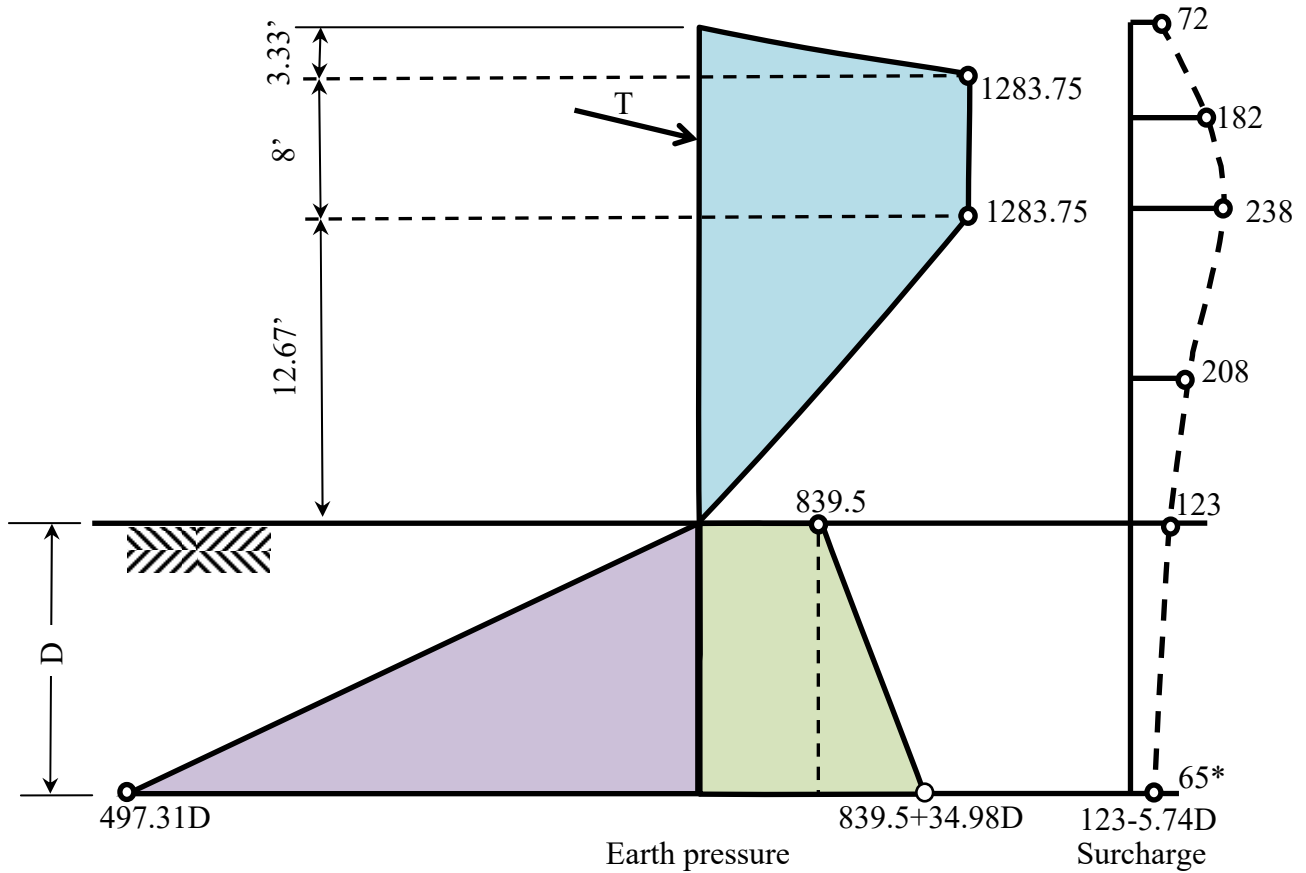


Figure 8-8. General Pressure Diagram

* The surcharge load of 65 psf is shown for illustrative purposes only. The actual load is dependent on depth, D, shown in the equation above.

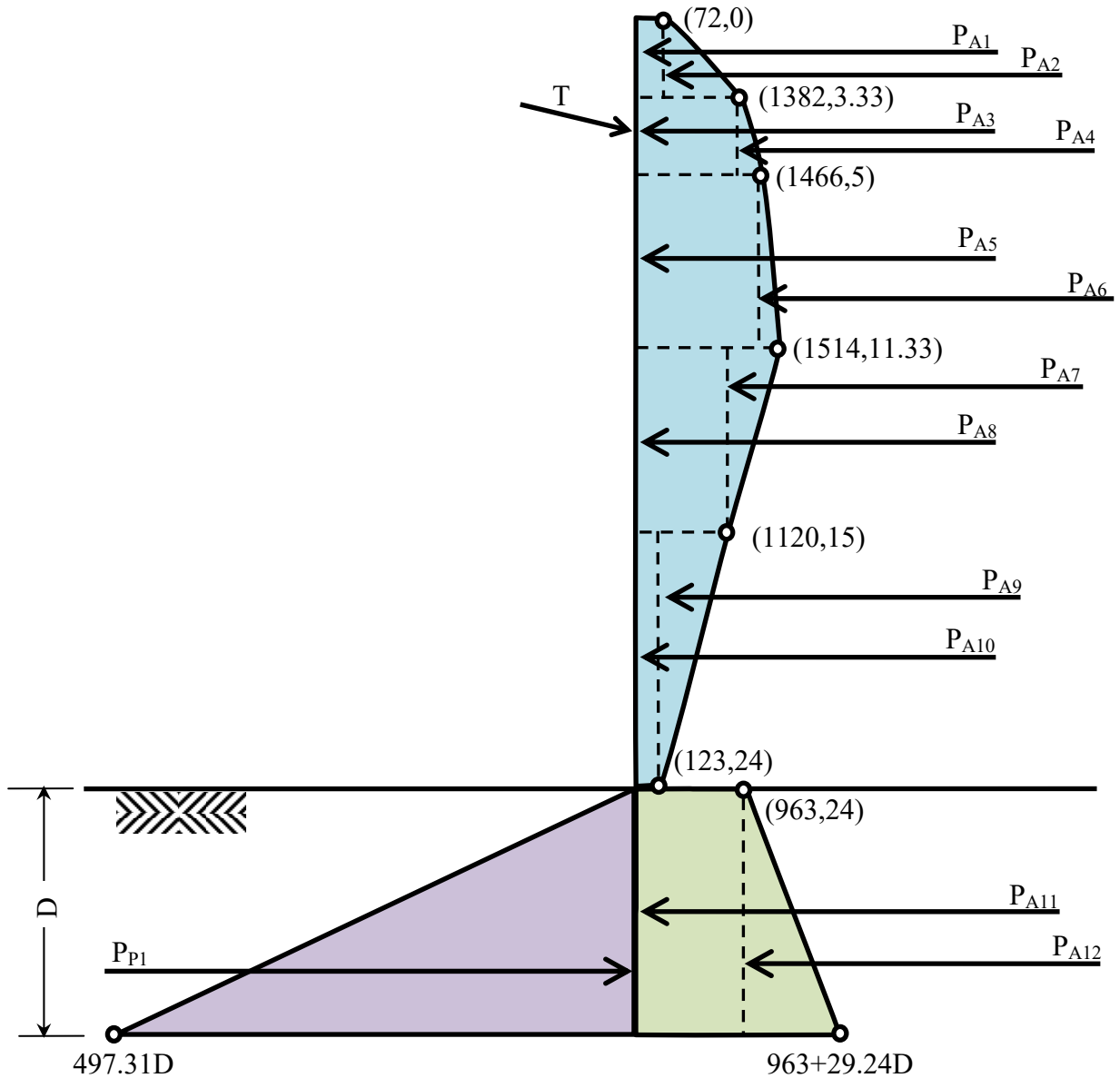


Figure 8-9. Combined Pressure Diagram (No Scale)

The loads coordinates from the Boussinesq load are added to the trapezoidal pressure diagram to calculate the total load acting on the shoring system as shown in Figure 8-9.

STEP II: Determine Depth, D

For Soldier piles an arching factor needs to be calculated and applied to both the Active and Passive forces below the dredge line only. Assume that the effective width of the piles is 1.27 ft.

$$\text{Arching Factor} = 0.08\phi = 0.08(27) = 2.16$$

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Calculating Driving and Resisting Moments taken about the Tieback Force:

Table 8-3. Calculated Driving and Resisting Moments

Driving Force (plf)	Arm (ft)	Driving Moment M_{DR} (lb)
$P_{A1} = (3.33)(72)(8) = 1,918.08$	$\frac{1}{2}(3.33) + 1.67 = 3.33$	-6,387.2
$P_{A2} = \frac{1}{2}(3.33)(1310)(8) = 17,449.2$	$\frac{1}{3}(3.33) + 1.67 = 2.78$	-48,508.8
$P_{A3} = (1.67)(1382)(8) = 18,463.52$	$\frac{1}{2}(1.67) = 0.84$	-15,509.4
$P_{A4} = \frac{1}{2}(1.67)(84)(8) = 561.12$	$\frac{1}{3}(1.67) = 0.56$	-314.2
$P_{A5} = (6.33)(1466)(8) = 74,238.2$	$\frac{1}{2}(6.33) = 3.16$	234,593
$P_{A6} = \frac{1}{2}(6.33)(48)(8) = 1,215.40$	$\frac{2}{3}(6.33) = 4.22$	5,128.8
$P_{A7} = \frac{1}{2}(3.67)(394)(8) = 5,783.9$	$6.33 + \frac{1}{3}(3.67) = 7.55$	43,668.6
$P_{A8} = (3.67)(1120)(8) = 32,883.2$	$6.33 + \frac{1}{2}(3.67) = 8.17$	268,656
$P_{A9} = \frac{1}{2}(9)(997)(8) = 35,892$	$10 + \frac{1}{3}(9) = 13$	466,596
$P_{A10} = (9)(123)(8) = 8,856$	$10 + \frac{1}{2}(9) = 14.5$	128,412
$P_{A11} = (963)(D)(1.27)(2.16) = 2,641.7 D$	$19 + \frac{1}{2}(D)$	$1,320.85 D^2 + 50,192.3 D$
$P_{A12} = \frac{1}{2}(29.24 D)(D)(1.27)(2.16) = 40.11 D^2$	$19 + \frac{2}{3}(D)$	$26.74 D^3 + 762.09 D^2$
Resisting Force (plf)	Arm (ft)	Resisting Moment M_{RS} (lb)
$P_{P1} = \frac{1}{2}(D)(497.31 D)(1.27)(2.16) = 682.11 D^2$	$19 + \frac{2}{3}(D)$	$454.74 D^3 + 12,960.09 D^2$

$$M_{DR} = 26.74 D^3 + 2,082.94 D^2 + 50,192.3 D + 1,076,334.5$$

$$M_{RS} = 454.74 D^3 + 12,960.09 D^2$$

Per AREMA, for restrained temporary shoring systems the minimum embedment length is 1.5 times the calculated depth. See Section 8.1.5.7. For equilibrium status ($FS = 1$), set the resisting moment equal to the driving moment as shown below and solve for D :

$$M_{RS} = M_{DR}$$

$$26.74 D^3 + 2,082.94 D^2 + 50,192.3 D + 1,076,334.5 = 454.74 D^3 + 12,960.1 D^2$$

$$428 D^3 + 10,877.16 D^2 - 50,192.3 D - 1,076,334.5 = 0$$

$$D^3 + 25.41 D^2 - 117.27 D - 2,514.8 = 0$$

$$D = 10.2 \text{ ft}$$

$$\text{Minimum required Depth, } D = 10.2 \text{ ft} * 1.5 = 15.3 \text{ ft}$$

STEP III: Calculate Tieback Load

Sum forces in the horizontal direction and set to zero:

$$\sum F_x = 0$$

$$\{T_H + 682.11(10.2)^2\} = \left\{ \begin{array}{l} 1,918.08 + 17,449.2 + 18,463.5 + 561.12 + 74,238.2 + 1,215.4 \\ + 5,783.9 + 32,883.2 + 35,892 + 8,856 + 2,641.7(10.2) + 40.11(10.2)^2 \end{array} \right\}$$

$$T_H = 228,379 - 70,967 = 157.41 \text{ Kips}$$

$$T = \frac{157.41}{\cos(15^\circ)} = 162.97 \text{ Kips (along } 15^\circ \text{ angle)}$$

Calculated Maximum Moment = 529.41 K-ft.

Calculated Maximum Shear = 119.02 Kips.

Graphical solution for determining maximum shear and moment for Railroad Problem EXAMPLE 8-1 follows. The graphical solution is necessary in this instance when calculating deflections. Note that in the following analysis, for simplicity, the active and passive loads in the embedded zone have been combined.

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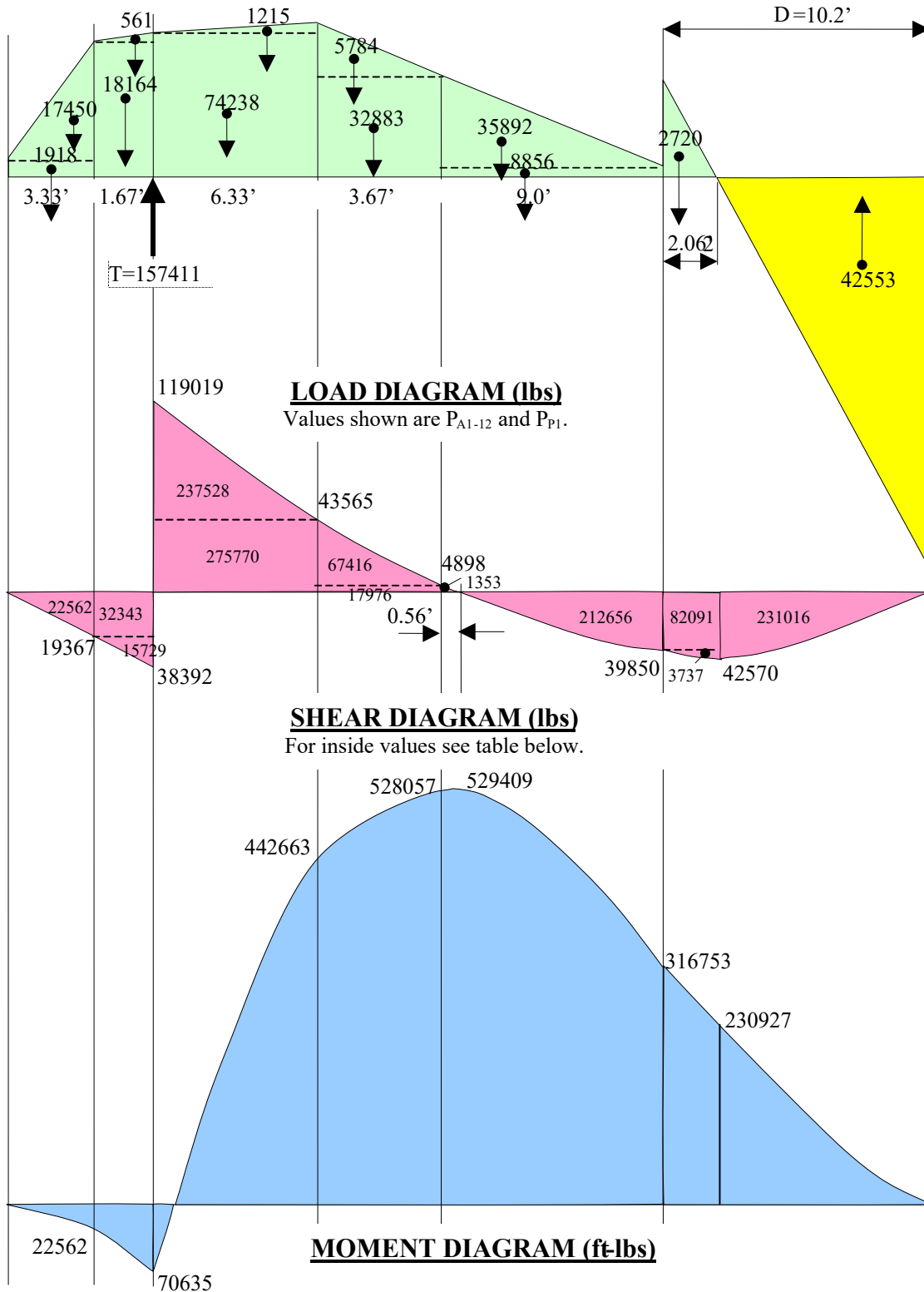


Figure 8-10. Final Load, Shear, and Moment Diagrams for EXAMPLE 8-1

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NOTE: By geometry the point of zero shear was determined to be 10.56' below the tieback and F_{A9} and F_{A10} have been adjusted accordingly. The following table is provided to show how the various areas from the load and shear diagrams above were used to determine the values for the moment diagram.

Table 8-4. Determining Moment Diagram Values

<i>Area Under the Shear Diagram (sf)</i>	<i>Segment Area (sf)</i>	<i>Moment (ft-lbs)</i>
$F_{A1} = \frac{1}{2} (3.33)(1,918.08) = 3,193.6$	$3,193.6 + 19,368.6 = 22,562$	22,562
$F_{A2} = \frac{1}{3} (3.33)(17,449.2) = 19,368.6$		
$F_{A3} = \frac{1}{2} (1.67)(18,463.5) = 15,417.0$	$15,417.0 + 312.36 + 19,367(1.67) = 15,728.36 + 32,343 = 48,073$	$22,562 + 48,073 = 70,635$
$F_{A4} = \frac{1}{3} (1.67)(561.12) = 312.36$		
$F_{A5} = \frac{1}{2} (6.33)(74,238.2) = 234,964.0$	$234,964.0 + 2,564.41 + 6.33(43,565.5) = 237,528 + 275,770 = 513,298$	$-70,635 + 513,298 = 442,663$
$F_{A6} = \frac{1}{3} (6.33)(1,215.36) = 2,564.41$		
$F_{A7} = \frac{1}{3} (3.67)(5,783.92) = 7,075.66$	$7,075.66 + 60,340.67 + 3.67(4898.36) = 67,416.33 + 17,976.98 = 85,393$	$442,663 + 85,393 = 528,056$
$F_{A8} = \frac{1}{2} (3.67)(32,883.2) = 60,340.67$		
$F_{A9a} = (0.56)(1,058.0)(8) = 4,739.84$ $F_{A9a} = \frac{1}{2} (0.56)(4,739.84) = 1,327.16$	$1,327.16 + 25.92 = 1,353$	$528,056 + 1,353 = 529,409$
$F_{A10a} = \frac{1}{2} (0.56)(1,120 - 1,058.0)(8) = 138.88$ $F_{A10a} = \frac{1}{3} (0.56)(138.88) = 25.92$		
$F_{A9b} = \frac{1}{2} (8.44)(1,058.0 - 123)(8) = 31,565.6$ $F_{A9b} = \frac{2}{3} (8.44)(31,565.6) = 177,609.11$		
$F_{A10b} = (8.44)(123)(8) = 8,304.96$ $F_{A10b} = \frac{1}{2} (8.44)(8,304.96) = 35,046.93$		
Areas Below Excavation		
$F_{A11a} = \frac{2}{3} (2.06)(2,720.95) = 3,736.77$	$3,736.77 + 2.06(39,849.64) = 3,736.77 + 82,090.26 = 85,827$	$316,753 - 85,827 = 230,926$
$F_{P1a} = \frac{2}{3} (8.14)(42,570.59) = 231,016$		$230,926 - 231,016 = -90$

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Determine lagging needs:

By inspection, the maximum load on the lagging is 1,521 psf acting 10 ft below the top of the shoring system (see Figure 8-8). Per CHAPTER 5, maximum lagging load may be limited to 400 psf without surcharges and assume that the design load on the lagging may taken as 0.6 times the calculated pressure based on a simple span. In this example the Railroad surcharge voids the 400 psf limitation. Also, the Railroad nullifies the use of the 1.33 load duration factor as discussed in CHAPTER 5. Therefore:

$$M_{\max} = \frac{wL^2}{8} = \frac{(1,521)(8')^2}{8} = 12,168 \text{ ft} \cdot \text{lb}$$

$$S \text{ Required} = \frac{M_{\max} * 12 * 0.6}{F_b} = \frac{12,168 \text{ ft} \cdot \text{lb} * 12 \text{ in/ft} * 0.6}{1,500 \text{ psi}} = 58.41 \text{ in}^3$$

Use 6 x 12' s (rough lumber): $S = 72 \text{ in}^3$ (Note that no lagging size was specified in the example problem statement)

Note that if the 400 psf limitation had been used, the required S would have been 15.36 in^3 and the minimum required rough lumber size would have been 3 x 12.

Check shear in the lagging at distance d from the face of support:

$$V = \left(\frac{L}{2} - d \right) (w)(0.6) = \left(\frac{8'}{2} - \frac{4''}{12} \right) (1,521)(0.6) = 3,349 \text{ lb}$$

$$f_v = \frac{3V}{2A} = \frac{3(3,349)}{2(6'')(12'')} = 69.8 \text{ psi} < 140 \text{ psi} \quad \therefore \text{OK}$$

In the above example, the actual pile spacing was used as the span length for the lagging. However, if further refinement is necessary, the span length could to taken as the clear distance between supports plus half the required bearing length at each support. For 12" high lagging with the required bearing length of a , the revised span length would be:

$$\frac{\left(\frac{wL}{2} \right)}{a * 12} = 450 \text{ psi}$$

$$a = \frac{wL}{2(12)(450)} = \frac{(1,521)(8)}{10,800} = 1.13 \text{ in}$$

$$\text{Span Length } L = 8' - \frac{12''}{12} + \frac{1.13''}{12} = 7.09 \text{ ft}$$

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A common substitute for wood lagging is a steel plate. The analysis for steel plate lagging is similar to that shown above for wood lagging:

$$F_b = 36,000 \text{ psi} * 0.75 = 27,000 \text{ psi}$$

$$M_{\max} = \frac{wL^2}{8} = \frac{(1,521)(8')^2}{8} = 12,168 \text{ ft} \cdot \text{lb}$$

$$S \text{ Required} = \frac{M_{\max} * 12 * 0.6}{F_b} = \frac{12,168 \text{ ft} \cdot \text{lb} * 12 \text{ in/ft} * 0.6}{27,000 \text{ psi}} = 3.25 \text{ in}^3$$

$$\text{Required plate thickness} = \sqrt{\frac{(6)(3.25)}{12}} = 1.275 \text{ in}$$

By inspection shear for steel lagging is OK.

8.3 DEFLECTION CALCULATION

Horizontal movement, or deflection, of shoring systems as described in CHAPTER 6 and CHAPTER 7 of this Manual can only be roughly approximated because soils do not apply pressures as true equivalent fluid, even in the totally active state. A deflection calculation can be made by structural mechanics procedures (moment area – M/EI) and then some engineering judgment should be used. Soil type, stage construction and the time that the shoring is in place will affect the movement. Monitoring or performance testing is important also.

The following is an example of a deflection calculation for EXAMPLE 8-1, a soldier pile with a single tieback. It is assumed that the lock-off load of the tieback is sufficient to preclude any movement at the tieback support. Additionally, the Point of Fixity of the pile will be assumed at $0.25D$ below the excavation line. For simplicity, the point of maximum deflection is assumed to occur at the location of maximum moment. The moment-area method will be used to calculate the deflections.

Determine the depth to the Point of Fixity (PoF) below excavation line.

$$\text{PoF} = (0.25)(D) = (0.25)(10.2') = 2.55'$$

Determine the deflection δ_p as shown in Figure 8-11.

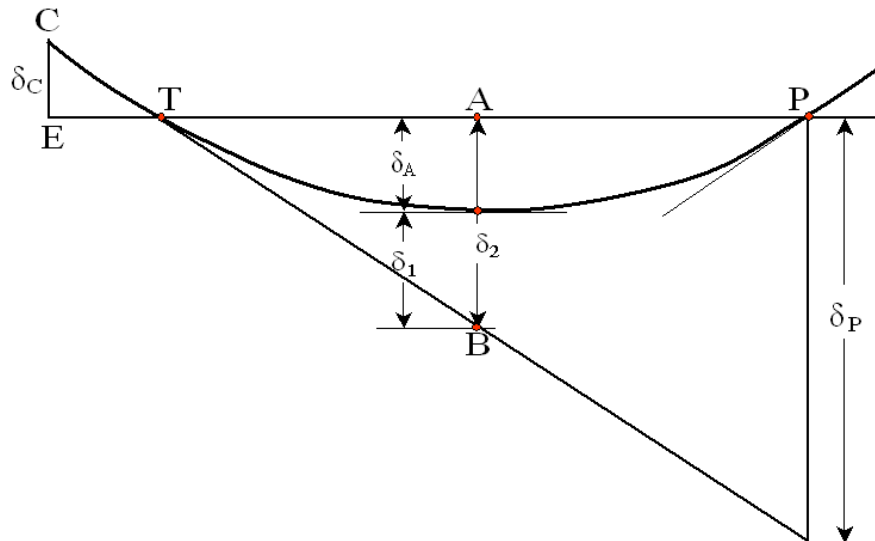


Figure 8-11. Deflected Shape of Shoring System

$$\delta_2 = (\delta_p) \left(\frac{10.56'}{21.55'} \right)$$

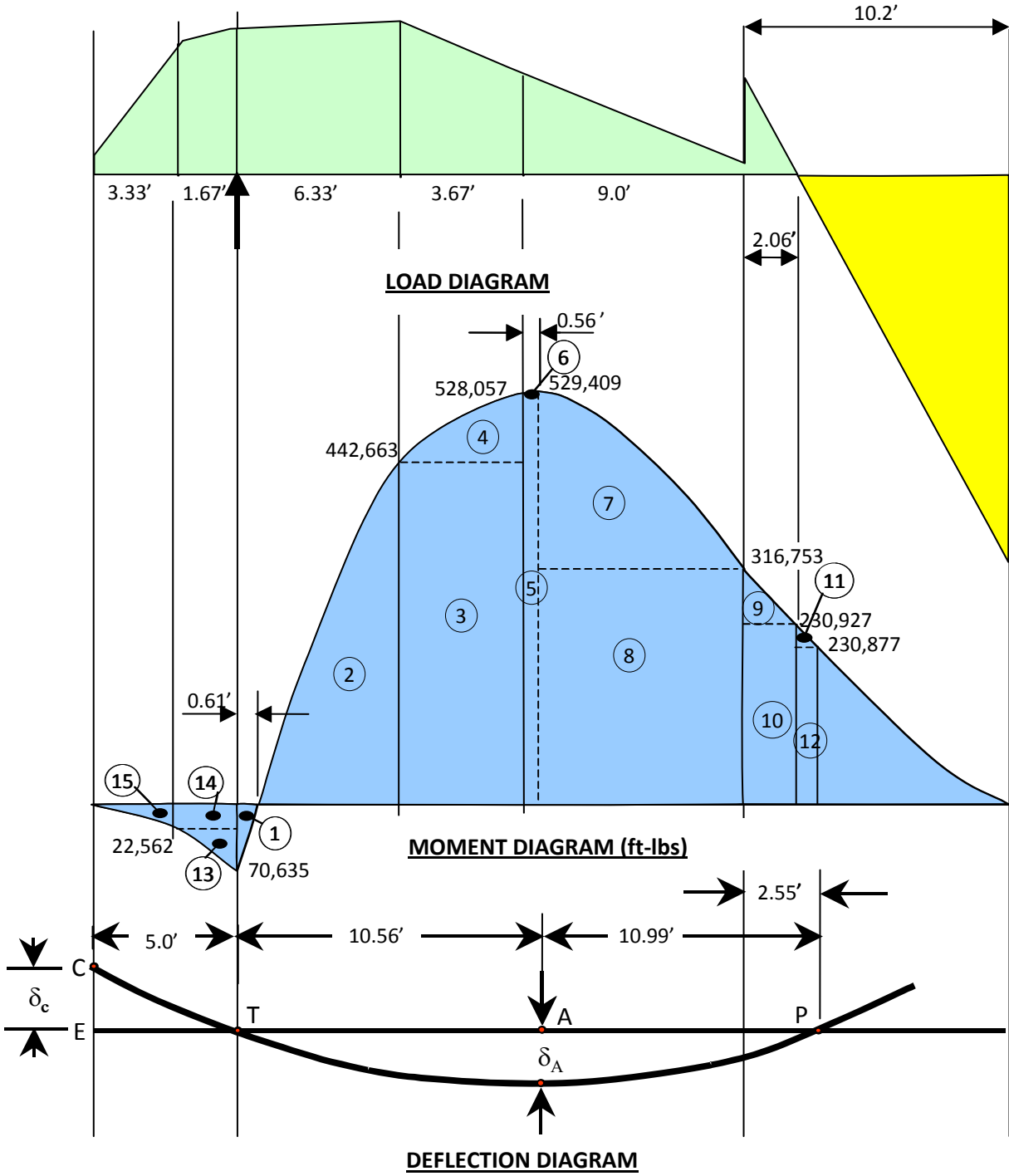


Figure 8-12. Schematic of Load, Moment and Deflection Diagrams for EXAMPLE 8-1

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Determine the deflection tangent to the elastic curve at the point of assumed maximum deflection from the tangent at T (δ_1).

The true deflection at A: $\delta_A = \delta_2 - \delta_1$. For the following calculations see Figure 8-12, Schematic of Load, Moment and Deflection Diagrams for EXAMPLE 8-1 for additional details. The moments below are taken about point P, the PoF.

Table 8-5. Calculations for deflection δ_p

Loc	Area (lb-ft ²)	Moment Arm (ft)	Area Moment (lb-ft ³)
①	$-\frac{1}{4}(70,635)(0.61')$	$20.94' + \frac{4}{5}(0.61')$	-230,818
②	$\frac{3}{4}(442,663)(5.72')$	$15.22' + \frac{2}{5}(5.72')$	33,248,113
③	$(442,663)(3.67')$	$11.55' + \frac{1}{2}(3.67')$	21,744,910
④	$\frac{3}{4}(528,057 - 442,663)(3.67')$	$11.55' + \frac{2}{5}(3.67')$	3,059,817
⑤	$(528,057)(0.56')$	$10.99' + \frac{1}{2}(0.56')$	3,332,673
⑥	$\frac{3}{4}(529,409 - 528,057)(0.56')$	$10.99' + \frac{2}{5}(0.56')$	6,373
⑦	$\frac{3}{4}(529,409 - 316,753)(8.44')$	$2.55' + \frac{3}{5}(8.44')$	10,249,302
⑧	$(316,753)(8.44')$	$2.55' + \frac{1}{2}(8.44')$	18,098,904
⑨	$\frac{1}{4}(316,753 - 230,927)(2.06')$	$0.49' + \frac{3}{5}(2.06')$	76,291
⑩	$(230,927)(2.06')$	$0.49' + \frac{1}{2}(2.06')$	723,076
⑪	$\frac{1}{4}(230,927 - 230,877)(0.49')$	$\frac{3}{5}(0.49')$	2
⑫	$(230,877)(0.49')$	$\frac{1}{2}(0.49')$	27,717
Total			90,336,356

$$\delta_p = 90,336,356 \left(\frac{1728}{180 * 10^9} \right) = 0.867''$$

$$\delta_2 = 0.867'' \left(\frac{10.56'}{21.55'} \right) = 0.426'' \approx 0.43''$$

To determine δ_A calculate δ_I by taking moments about point A.

Table 8-6. Calculations for deflection δ_I

Loc	Area (lb-ft ²)	Moment Arm (ft)	Area Moment (lb-ft ³)
①	$-\frac{1}{4}(70,635)(0.61')$	$9.95' + \frac{4}{5}(0.61')$	-112,436
②	$\frac{3}{4}(442,663)(5.72')$	$4.23' + \frac{2}{5}(5.72')$	12,377,839
③	$(442,663)(3.67')$	$0.56' + \frac{1}{2}(3.67')$	3,890,852
④	$\frac{3}{4}(528,057 - 442,663)(3.67')$	$0.56' + \frac{2}{5}(3.67')$	476,671
⑤	$(528,057)(0.56')$	$\frac{1}{2}(0.56')$	82,799
⑥	$\frac{3}{4}(529,409 - 528,057)(0.56')$	$\frac{2}{5}(0.56')$	127
Total			16,715,853

$$\delta_1 = 16,715,853 \left(\frac{1728}{180 * 10^9} \right) = 0.16''$$

$$\delta_A = \delta_2 - \delta_1 = 0.43'' - 0.16'' = 0.27''$$

Determine the deflection δ_C as shown in Figure 8-13.

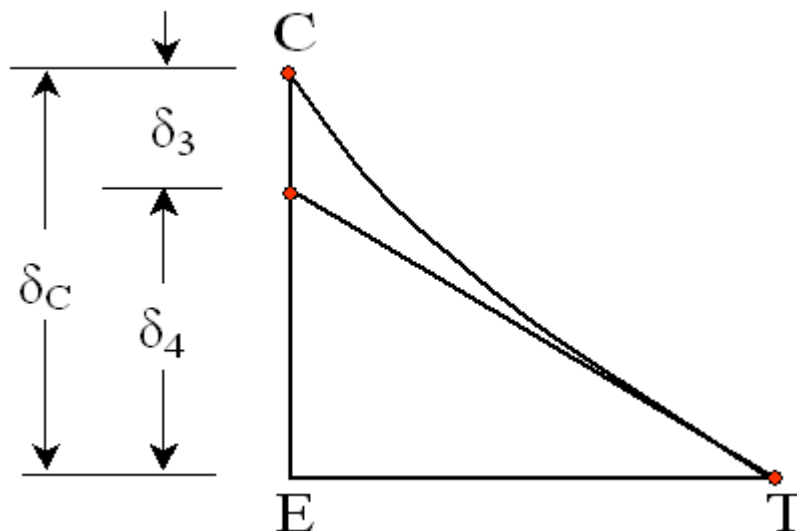


Figure 8-13. Deflected Shape of Shoring System above the Tieback

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$$\delta_C = \delta_3 + \delta_4$$

$$\delta_4 = 0.867'' \left(\frac{5'}{21.55'} \right) = 0.201'' \approx 0.20''$$

Determine δ_3 by taking moments about point E.

Table 8-7. Calculations for deflection δ_3

Loc	Area (lb-ft ²)	Moment Arm (ft)	Area Moment (lb-ft ³)
⑬	$-\frac{1}{4}(70,635 - 22,562)(1.67')$	$3.33 + \frac{4}{5}(1.67')$	-93,648
⑭	$-(22,562)(1.67')$	$3.33 + \frac{1}{2}(1.67')$	-156,933
⑮	$-\frac{1}{4}(22,562)(3.33')$	$\frac{4}{5}(3.33')$	-50,038
Total			-300,619

$$\delta_3 = -300,619 \left(\frac{1728}{180 * 10^9} \right) = -0.0028'' \approx 0.00''$$

$$\delta_C = \delta_3 + \delta_4 = 0.00'' + 0.20'' = 0.20''$$

The final deflection shape of the shoring system using the moment area-M/EI method is shown in Figure 8-14. It is noted that the deflection shown here is only for the vertical element of the shoring system. Deflection of other elements including any lagging must also be considered when determining the maximum deflection on a shoring system.

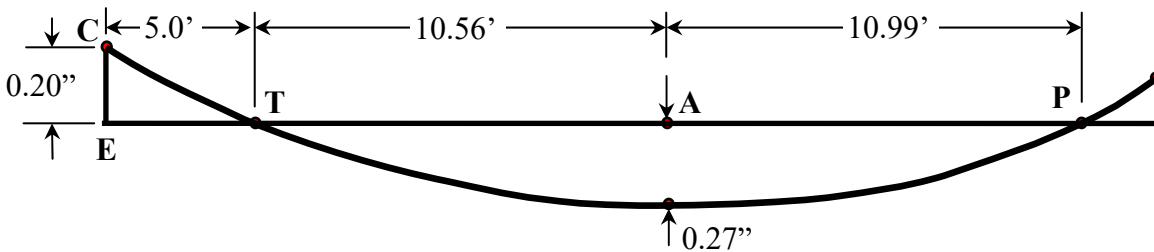


Figure 8-14. Final Deflected Shape of Shoring System

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Caltrans Trenching and Shoring Check Program (CT-TSP), for EXAMPLE 8-1 (Railroad Example)

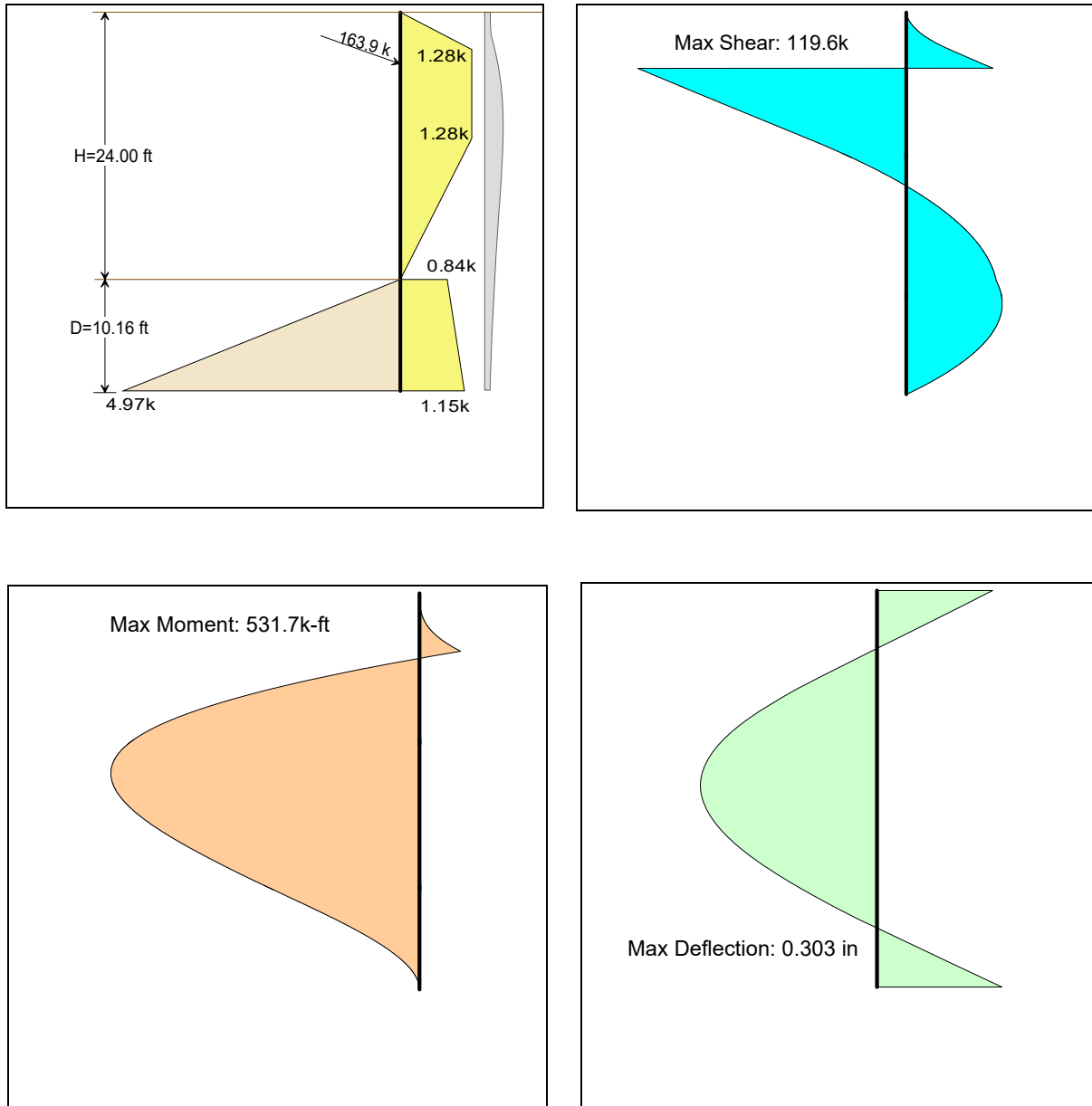


Figure 8-15. Diagrams per CT-TSP