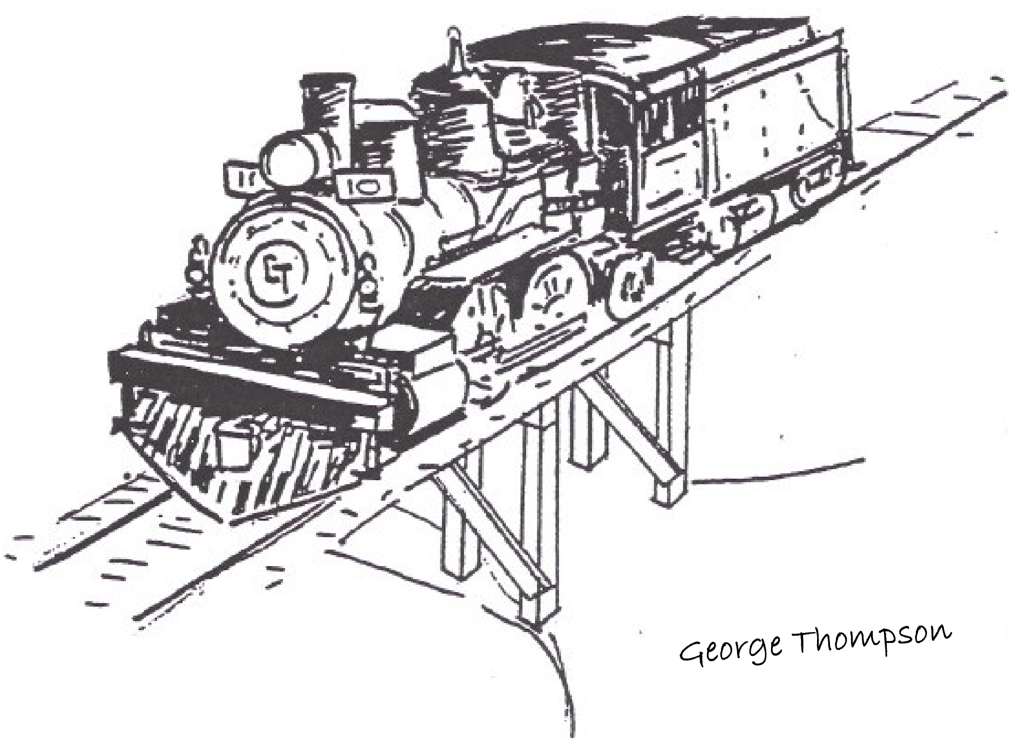
JULY 2025

CHAPTER 9

RAILROADS



Chapter 9: Railroads

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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.*

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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.

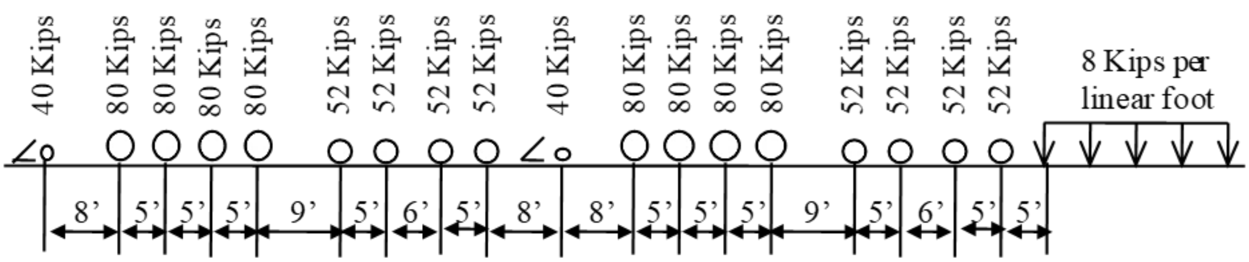

Figure 9-1. Cooper E80 Loading from the Appendix of the GUIDELINES

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.

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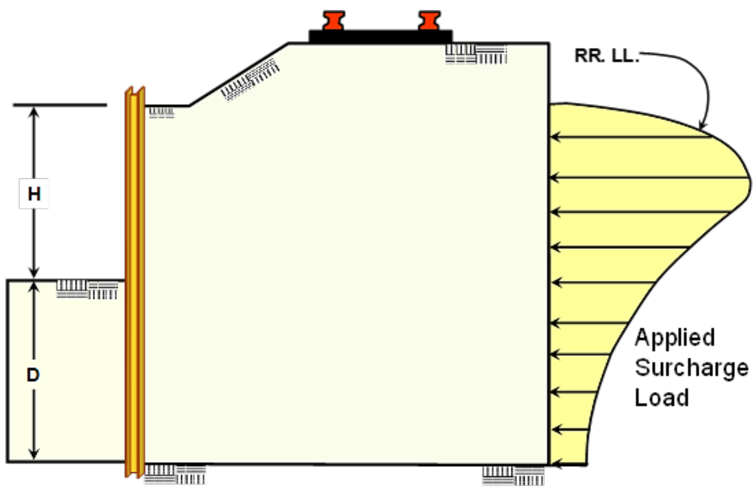

Figure 9-2. Pressure diagram for Boussinesq Strip Load

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.

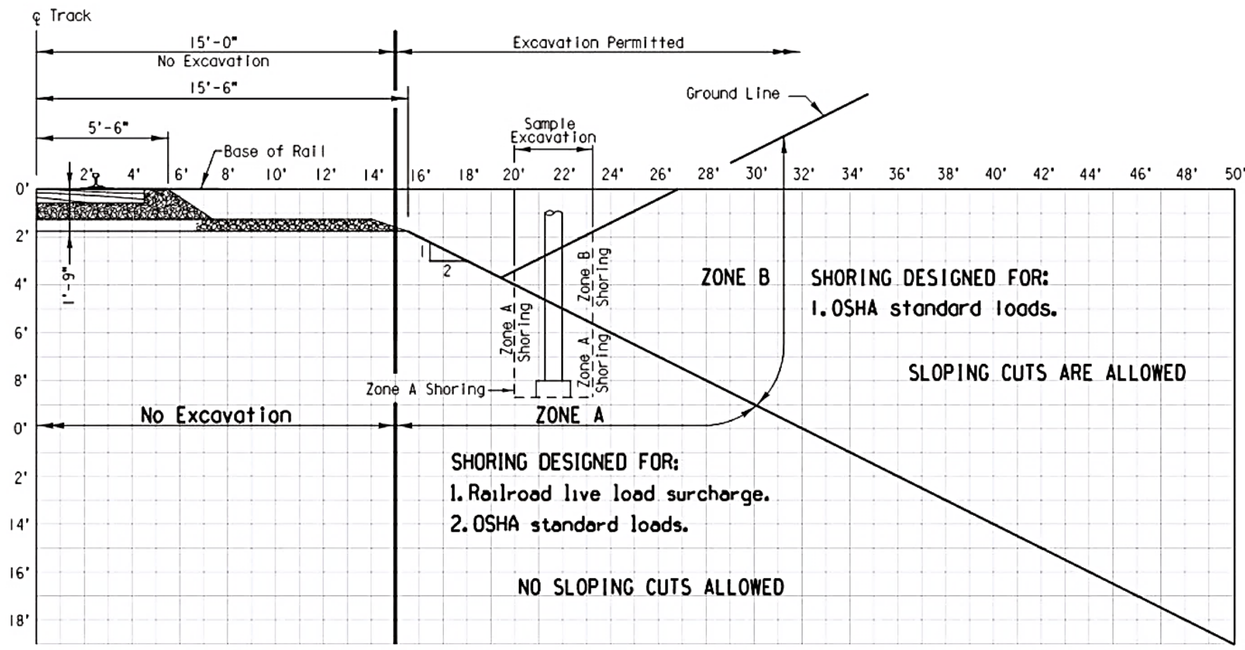

Figure 9-3. Shoring Zones from the GUIDELINES, Section 3, Figure 1

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.

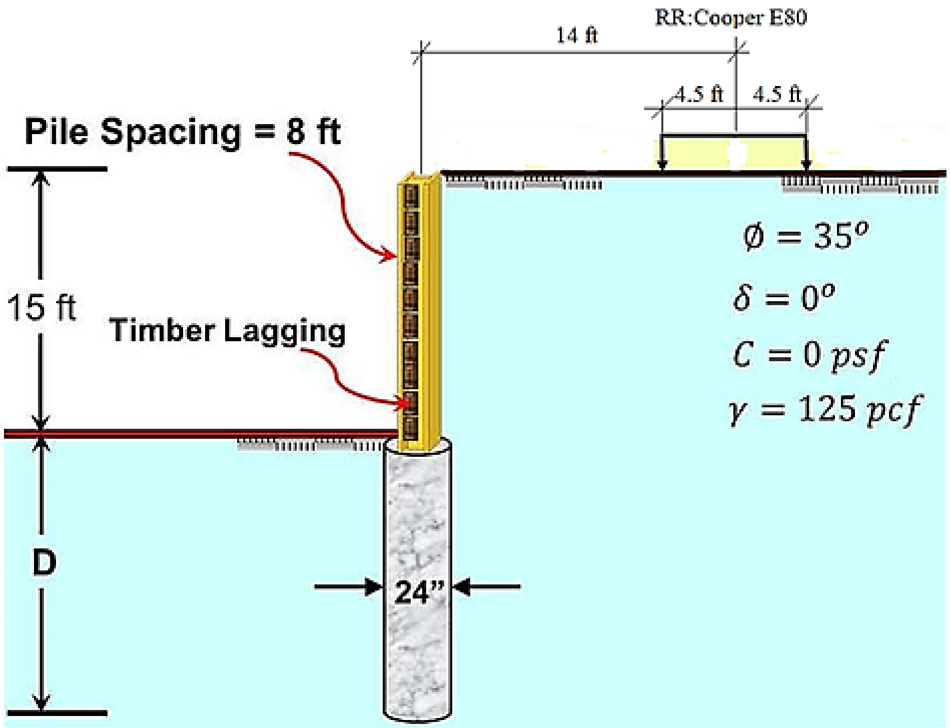

Figure 9-5. Cantilevered Soldier-Pile-Lagging Wall Example Properties

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:

$$\mathbf{q_s} = \frac{\mathbf{Axle\ Load}}{(\mathbf{Axle\ Spacing})(\mathbf{Track} + \mathbf{H_1})};\ \mathrm{H_1} = 0\ \therefore\ \mathrm{q_s} = \frac{80{,}000}{(5)(9+0)} = 1{,}778\ \mathrm{psf}$$

(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:

$$\boldsymbol{\sigma_h} = 2\mathbf{q_s}\frac{\boldsymbol{\beta_R} - \mathbf{sin\beta cos2\alpha}}{\boldsymbol{\pi}} \quad (9\text{-}4\text{-}2)$$

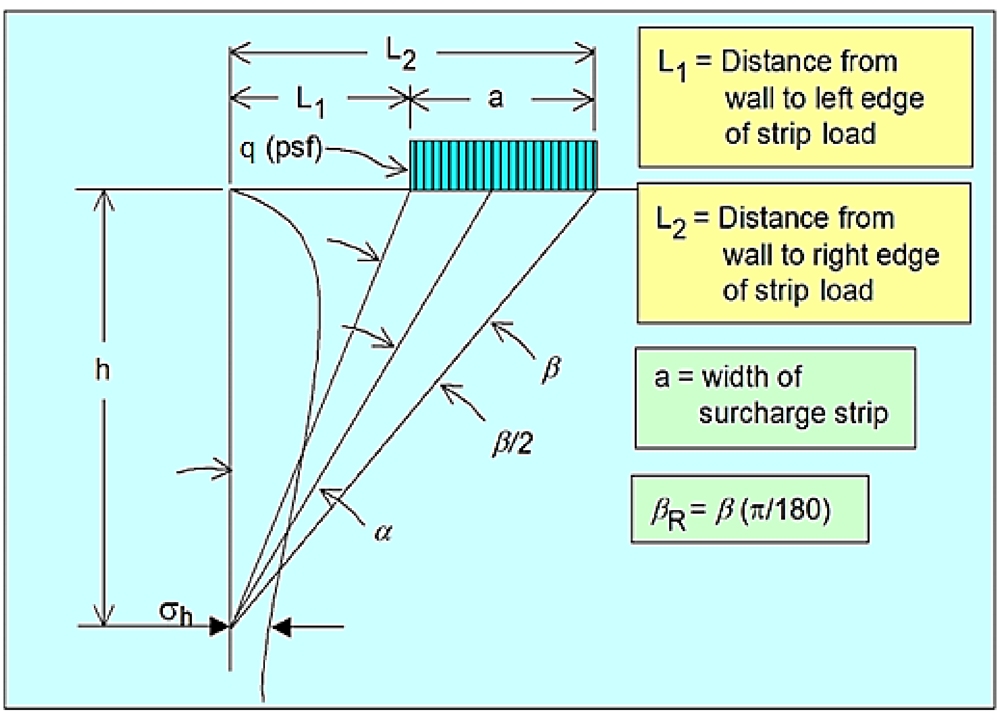

Figure 9-6. Boussinesq Type Strip Load

$$q_S = 1{,}778 \text{ psf}; \;\; L_1 = 9.5 \text{ ft}; \; L_2 = 18.5 \text{ ft}; \;\; H = 5\text{ft} \tag{9-4-3}$$

$$\beta = \sin^{-1}\left(\frac{L_2}{\sqrt{L_2^2 + h^2}}\right) - \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) = \sin^{-1}\left(\frac{18.5}{19.164}\right) - \sin^{-1}\left(\frac{9.5}{10.735}\right) = 12.627° \tag{9-4-4}$$

$$\alpha = \sin^{-1}\left(\frac{L_1}{\sqrt{L_1^2 + h^2}}\right) + \frac{1}{2}\beta = \sin^{-1}\left(\frac{9.5}{10.735}\right) + \frac{1}{2}(12.627°) = 68.560° \tag{9-4-5}$$

$$\beta_R = \beta\left(\frac{\pi}{180}\right) = 12.627°\left(\frac{\pi}{180}\right) = 0.2204 \text{ Rad.} \tag{9-4-6}$$

$$\sigma_h = 2(1{,}778)\frac{0.2204 - \sin(12.627°)\cos(2 \times 68.560°)}{\pi} = 430.79 \approx \mathbf{431\ psf} \text{ (horizontal pressure)} \tag{9-4-7}$$

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.

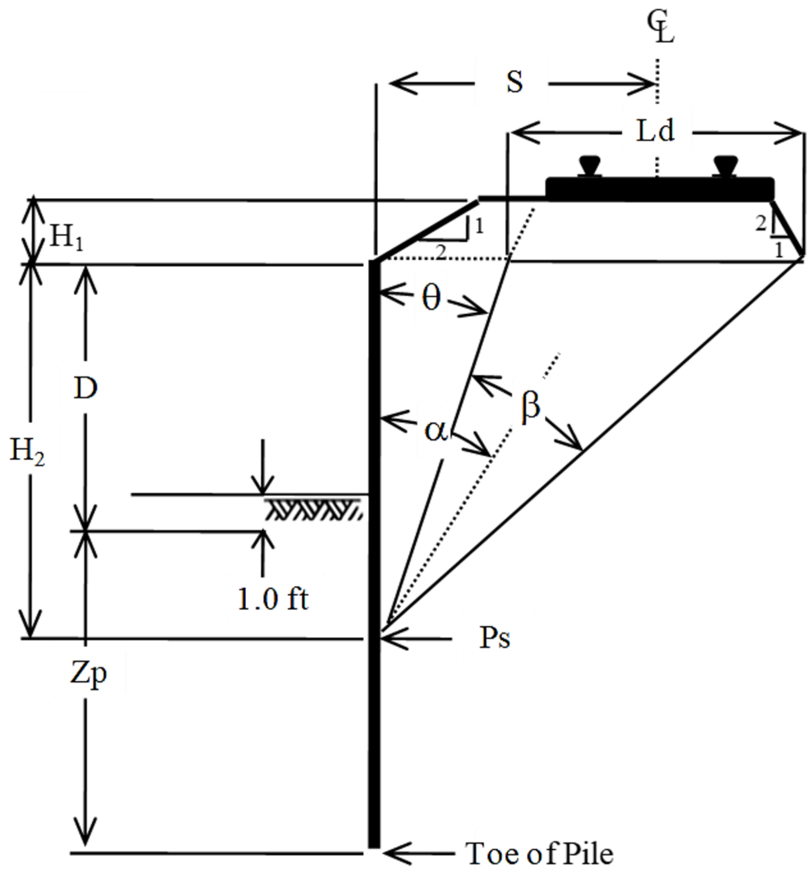

Figure 9-7, Railroad Adjacent to Soldier Pile Wall

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:

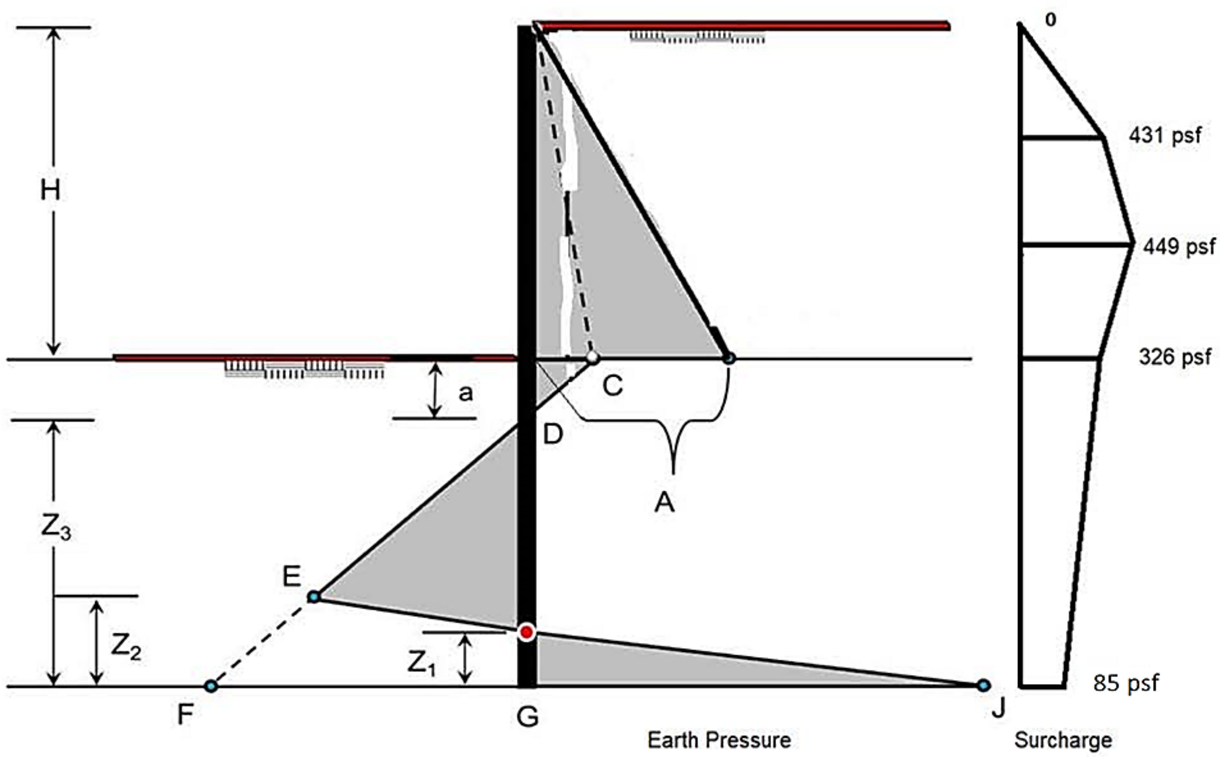

Figure 9-8. Rigorous Pressure Diagram and Horizontal Load from Surcharge

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)*.

$$K_a = \tan^2\left(45 - \frac{\emptyset}{2}\right) = \tan^2\left(45 - \frac{35}{2}\right) = 0.271 \qquad (9\text{-}4\text{-}8)$$

$$K_p = \tan^2\left(45 + \frac{\emptyset}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \qquad (9\text{-}4\text{-}9)$$

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.

$$0 = \sigma(\text{at C})(\text{in ksf}) - \text{a}\left[\gamma(\text{kcf}) \times \left(\text{K}_\text{p} f - \text{K}_\text{a}\right)\right] \qquad (9\text{-}4\text{-}10)$$

$$\text{a} = \frac{\sigma(\text{at excavation line}, \gamma \text{H} \text{K}_\text{a})(\text{ksf})}{\gamma(\text{kcf}) \times \left(\text{K}_\text{p} f - \text{K}_\text{a}\right)}$$

$$= \frac{0.508 \text{ ksf}}{(0.125 \text{ kcf})(3.69 \times 2.8 - 0.271)} = 0.404 \text{ ft} \qquad (9\text{-}4\text{-}11)$$

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.

$$\boldsymbol{f} = 0.08 \times \emptyset = (0.08 \times 35) = 2.8 \qquad (9\text{-}4\text{-}12)$$

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:

$$\text{A} = 0.125 \times 15 \times 0.271 \times 8 = 4.065 \frac{\text{kip}}{\text{ft}} \qquad (9\text{-}4\text{-}13)$$

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

$$\mathrm{C} = 0.125 \times 15 \times 0.271 \times 2 = 1.01625 \frac{\mathrm{kip}}{\mathrm{ft}} \quad (9\text{-}4\text{-}14)$$

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

$$\mathrm{F} = \left(0.125 \times \mathrm{Z}_3 \times \left((3.69 \times 2.8) - 0.271\right) \times 2\right) = 2.51525\mathrm{Z}_3 \frac{\mathrm{kip}}{\mathrm{ft}} \quad (9\text{-}4\text{-}15)$$

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

$$\mathrm{J} = \left(0.125 \times (\mathrm{Z}_3 + 0.404) \times \left((3.69 \times 2.8) - 0.271\right) \times 2\right) + (0.125 \times 15 \times 3.69 \times 2.8 \times 2) = 2.51525\ \mathrm{Z}_3 + 39.7612 \frac{\mathrm{kip}}{\mathrm{ft}}$$

(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:

$$\mathrm{P}_{s1} = \frac{1}{2}(431\mathrm{psf} \times 5\ \mathrm{ft}) \times 8 = 8.62\ \mathrm{kips, at\ 3.33\ ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}17)$$

$$\mathrm{P}_{s2} = (431\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 17.24\ \mathrm{kips, at\ 7.5\ ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}18)$$

$$\mathrm{P}_{s3} = \frac{1}{2}(18\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 0.36\ \mathrm{kips, at\ 8.33\ ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}19)$$

$$\mathrm{P}_{s4} = \frac{1}{2}(123\ \mathrm{psf}\times 5\ \mathrm{ft}) \times 8 = 2.46\ \mathrm{kips, at\ 11.67ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}20)$$

$$\mathrm{P}_{s5} = (326\ \mathrm{psf}\ \times 5\ \mathrm{ft}) \times 8 = 13.04\ \mathrm{kips, at\ 12.5\ ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}21)$$

$$\mathrm{P}_{s6} = \frac{1}{2}(241\ \mathrm{psf}\times 17\mathrm{ft}) \times 2.0 = 4.10\mathrm{kips, at\ 20.67ft\ from\ top\ of\ wall.} \quad 9\text{-}4\text{-}22)$$

$$\mathrm{P}_{s7} = (85\ \mathrm{psf} \times 17\ \mathrm{ft}) \times 2.0 = 2.89\ \mathrm{kips, at\ 23.5\ ft\ from\ top\ of\ wall.} \quad (9\text{-}4\text{-}23)$$

2. Calculate active earth force above dredge line, $\mathbf{P_1}$:

$$\mathrm{P}_1 = \frac{1}{2} \times 4.065 \frac{\mathrm{kip}}{\mathrm{ft}} \times 15\ \mathrm{ft} = 30.4875\ \mathrm{kips} \quad (9\text{-}4\text{-}24)$$

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

$$P_2 = \frac{1}{2} \times 1.01625 \frac{\text{kip}}{\text{ft}} \times 0.404 = 0.2053 \text{ kips} \quad (9\text{-}4\text{-}25)$$

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

$$\text{Area FEJ} = P_3 = \frac{1}{2} \times \left(2.51525\, Z_3 \frac{\text{kip}}{\text{ft}} + \left(2.51525\, Z_3 + 39.7612 \frac{\text{kip}}{\text{ft}}\right)\right) \times Z_2$$
$$= 2.51525\, Z_3 Z_2 + 19.881\, Z_2 \text{ kips}$$

(9-4-26)

$$\text{Area FDG} = P_4 = \frac{1}{2} \times 2.51525\, Z_3 \frac{\text{kip}}{\text{ft}} \times Z_3 \text{ ft} = 1.257625\, Z_3^2 \text{ kips} \quad (9\text{-}4\text{-}27)$$

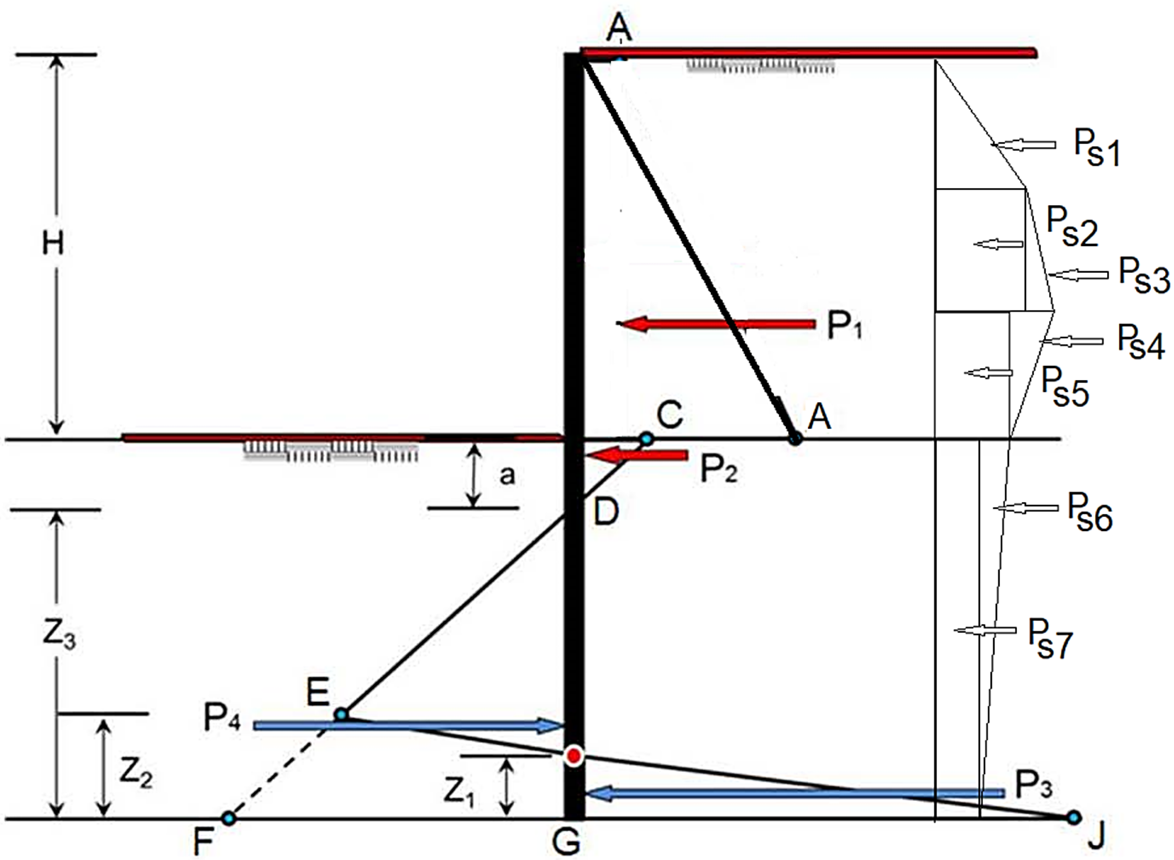

Figure 9-9. Force Diagram for Rigorous Method

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**:

$$\Sigma F = 0 \tag{9-4-28}$$

$$P_1 + P_2 + P_3 + P_{S1} + P_{S2} + P_{S3} + P_{S4} + P_{S5} + P_{S6} + P_{S7} - P_4 = 0$$
$$30.4875 + 0.2053 + (2.51525\ Z_3 Z_2 + 19.881\ Z_2) + 8.62 + 17.24 + 0.36 + 2.46 + 13.04 + 4.10 + 2.89 - 1.257625\ Z_3^2 = 0$$

(9-4-29)

Simplify and solve for **Z2**:

$$Z_2 = \frac{1.257625 Z_3^2 - 79.3998}{2.51525 Z_3 + 19.881} \tag{9-4-30}$$

$$\Sigma M_G = 0 \tag{9-4-31}$$

$$\begin{aligned}
&\big(30.4875 \times (Z_3 + 0.404 + 5)\big) + \left(0.2053 \times \left(Z_3 + \frac{2(0.404)}{3}\right)\right) \\
&+ \left((2.51525 Z_3 Z_2 + 19.881 Z_2) \times \frac{Z_2}{3}\right) \\
&+ \big(8.62 \times (Z_3 + 0.404 + 11.67)\big) + \big(17.24 \times (Z_3 + 0.404 + 7.5)\big) \\
&+ \big(0.36 \times (Z_3 + 0.404 + 6.67)\big) + \big(2.46 \times (Z_3 + 0.404 + 3.33)\big) \\
&+ \big(13.04 \times (Z_3 + 0.404 + 2.5)\big) + \big(4.097 \times (Z_3 + 0.404 - 5.67)\big) \\
&+ \big(2.89 \times (Z_3 + 0.404 - 8.5)\big) - \left(1.257625\ Z_3^2 \times \left(\frac{Z_3}{3}\right)\right) = 0
\end{aligned}$$

(9-4-32)

Simplify and collect like terms:

$$79.3998\ Z_3 + 409.7808 + 0.83842\ Z_3 Z_2^2 + 6.627\ Z_2^2 - 0.41921\ Z_3^3 = 0$$

(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 **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.

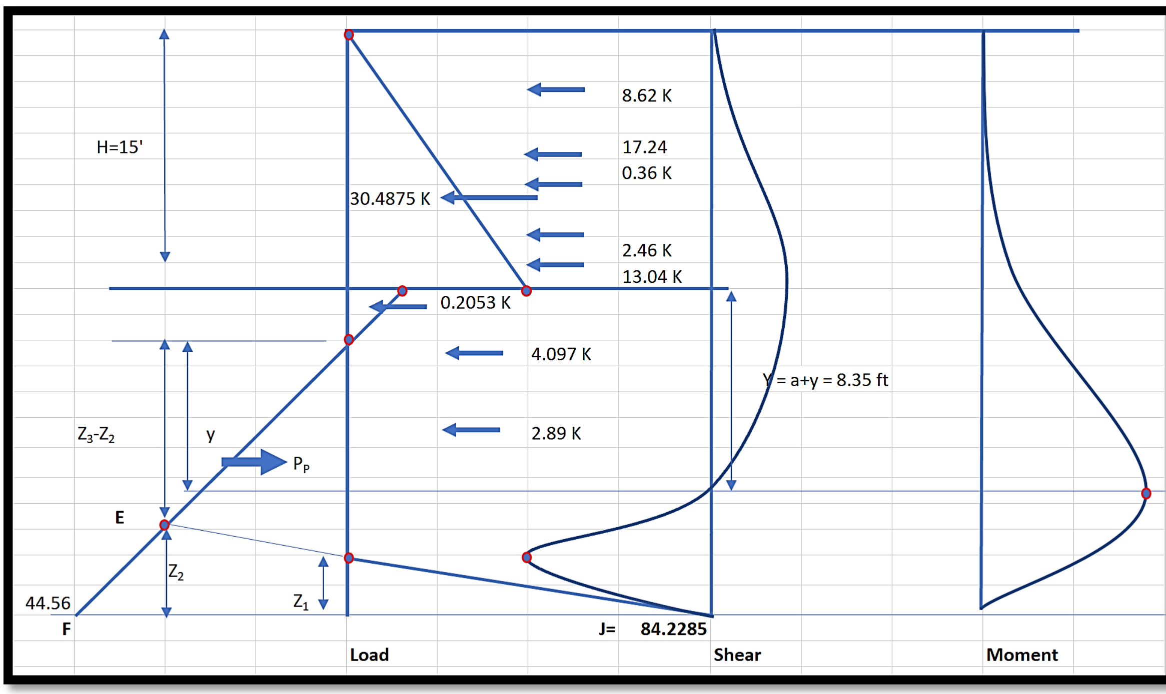

Figure 9-10. Pressure, Shear, and Moment Diagram

As illustrated in Figure 9-10, find pressure (kip/ft) at point **E** using similar triangles:

$$\frac{F}{Z_3} = \frac{E}{(Z_3 - Z_2)} \quad => \quad E = \frac{(Z_3 - Z_2)F}{Z_3} \tag{9-4-36}$$

$$E = \frac{(17.7148 \text{ ft} - 4.8925 \text{ ft})44.56 \frac{\text{kip}}{\text{ft}}}{17.7148 \text{ ft}} = 32.2534 \frac{\text{kip}}{\text{ft}} \tag{9-4-37}$$

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

$$\frac{4.8925\ \mathrm{ft}}{(32.2534+84.2285)\frac{\mathrm{kip}}{\mathrm{ft}}}=\frac{Z_1}{84.2285\frac{\mathrm{kip}}{\mathrm{ft}}} \quad => \quad Z_1=3.5378\ \mathrm{ft} \tag{9-4-38}$$

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

$$V_{max}=\frac{1}{2}\times\left(84.2285\frac{\mathrm{k}}{\mathrm{ft}}\right)\times(3.5378\ \mathrm{ft})=148.992\ \mathrm{kips} \tag{9-4-39}$$

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):

$$P_p=\frac{1}{2}\left[0.125\times y\times\left((3.69\times 2.8)-0.271\right)\times 2\right]\times y=1.257625y^2\ \frac{\mathrm{k}}{\mathrm{ft}} \tag{9-4-40}$$

Set up equation for sum of forces:

$$\Sigma\, F_x=0 \tag{9-4-41}$$

$$\begin{aligned}1.257625y^2&=30.4875+0.2053+8.62+17.24+0.36+2.46+13.04+4.097+2.89\\1.257625y^2&=79.3998 => \quad y=7.946\ \mathrm{ft}\end{aligned} \tag{9-4-42}$$

$Y = 7.946+0.404=8.35\ \mathrm{ft}$ (below the dredge line) (9-4-43)

$$M_{max}=\begin{Bmatrix}M_{+}=30.4875(5+8.35)+0.2053\left(8.35-\frac{0.404}{3}\right)+8.62(11.67+8.35)+17.24(7.5+8.35)\\+0.36(6.67+8.35)+2.46(3.33+8.35)+13.04(2.5+8.35)+5.74(15+8.35-20.67)\\+4.05(15+8.35-23.5)=\ 1044.921\ \mathrm{kips-ft}\\M_{-}=1.257625\times 8.086^2\times\left(\frac{8.086}{3}\right)=221.631\ \mathrm{kips-ft}\end{Bmatrix} \tag{9-4-44}$$

$M_{max}=1044.921\ \mathrm{kips-ft}$ (as illustrated in Figure 9-11) (9-4-45)

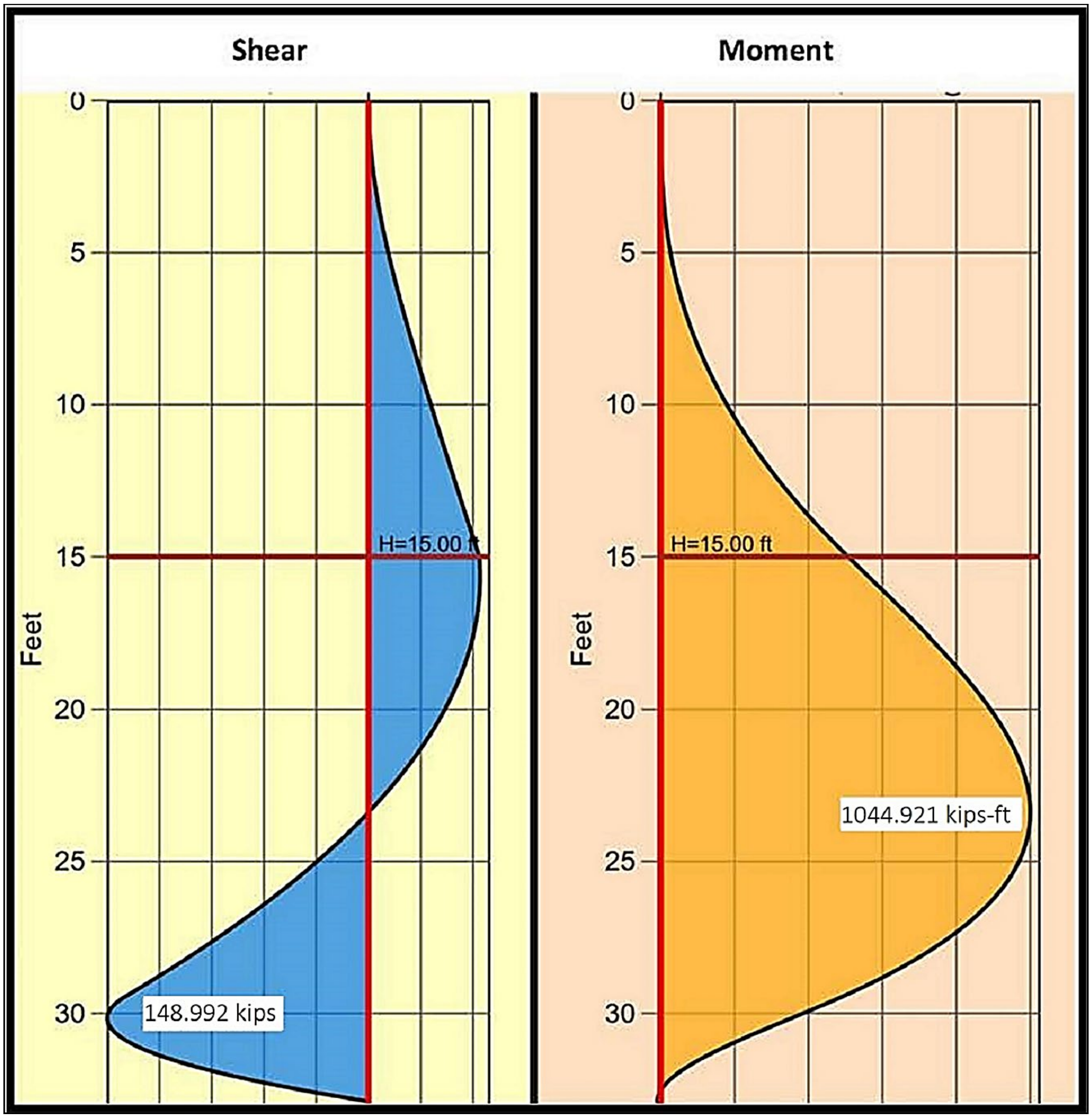

Figure 9-11. Shear and Moment Diagram

W12x336 Beam Shear and Moment:

Shear, **V** = 148.992 Kips.

$$(A = d \times t_w = 16.8 \times 1.78 = 29.904\ in^2, \text{AISC Table } 1-1) \qquad (9\text{-}4\text{-}46)$$

Compare actual versus allowable shear stress:

$$f_V = \frac{V}{A} = \frac{148{,}992\ lb}{29.904\ in^2} = 4{,}982\ psi < 14{,}400\ psi\ \ (0.4\ F_Y)\ \ \textbf{OK.} \qquad (9\text{-}4\text{-}47)$$

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.

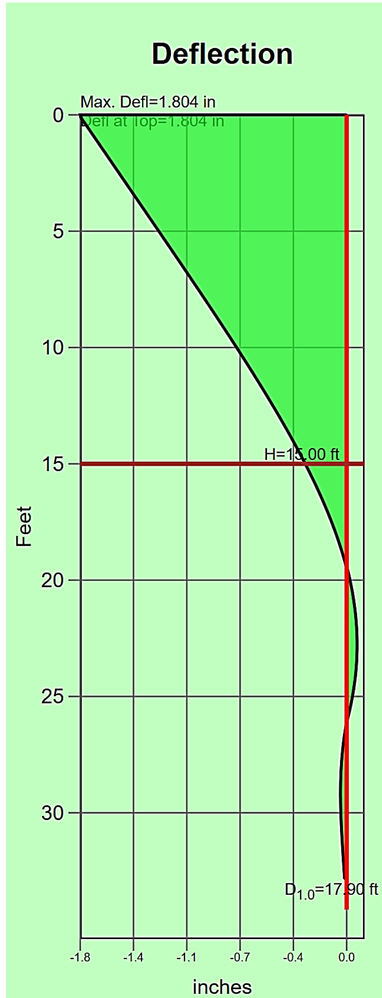

Figure 9-12. Deflection (CT_T&S Program)

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*.