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January 4, 2011

Eagles Nest Board of Directors
c/o Benjamin White Architecture
148 Elcho Avenue, Unit 3
Crested Butte, Colorado 81224

Attention: Mr. Ben White

SGM Project # 2010-246.001

Subject: Retaining Wall Evaluation

At your request, representatives of SGM conducted site visits to the Eagles Nest Townhouses in Mount Crested Butte on September 27 and November 22, 2010. The purpose of the site observations was to observe a retaining wall supporting a parking lot at lots B & C. In preparation of this report we reviewed the original design plans prepared by Pickett Henthorn Architects (dated May 30, 1980) as well as construction pictures. A copy of the original design plan is attached to this report.

**Observations:**

The retaining walls observed are a two level tied back soldier beam and lagging wall. An access road follows a bench between the two walls. A parking lot is located above the upper wall. In general, the terrain in the vicinity of the wall slopes down to the east. The soldier beams consist of double channels placed back to back with a two inch gap between them. The size of the channels varied with wall height. In general the majority of the soldier beams consisted of C12x20.7 channels, but we also observed C8x11.5, C6x8.2 and C4x4.5 on the shorter portions of the wall. These sizes are based upon measurements we made and do not match the sizes specified on the plans provided to us. The soldier beams are founded in drilled concrete piers and are tied back by one to three layers of anchor rods depending upon the height of the wall. Below the wall the ground slopes steeply away to the road.

The upper anchor rods angle down and the lower anchor rods generally angle upward. Most of the lower anchors rods are double 1 ½" diameter rods and the upper anchors are a single 1 ½" diameter rod. Smaller 1 1/8" and ¾" diameter rods were used on the shorter walls. According to the original design plans the rods are anchored to a continuous concrete deadman but in the construction photographs they appear to be anchored to individual concrete deadmen rather than one continuous unit. Multiple anchors are attached to each deadman. In the original design plans the deadmen are socketed into undisturbed bedrock approximately twenty five feet behind the face of the wall. The original plans only specified a single row of anchors.

The lagging consists of standard 6"x8" railroad ties bearing against the back of the soldier beams. Below ten feet the ties are doubled. The ties have weathered significantly and it appears that water frequently drains through the face of the wall.

We visually observed the condition of the exposed portions of the walls. During our site visits we observed thirteen anchor rods on the upper wall and one anchor on the lower wall that had broken at the anchor plate. Many of the anchors rods were not accessible due to location or were obscured by dirt and it is possible that more have broken that we were not able to observe. In addition to the broken anchor rods we observed two anchors which were loose. The loose anchors were located in the bottom row at soldier beams with three rows of tie backs that had broken anchors in the top row and had rotated outwards. With the exception of one soldier beam, all of the broken anchor rods were in the top row. One soldier beam had broken anchors at the top and middle rows. The wall has rotated outward as much as a foot at the soldier beams with broken anchor rods. At these locations subsidence of the backfill was observed. At the inside corner of the middle portion of the upper wall, a soldier beam has twisted considerably. This soldier beam is loaded at an angle by the wall to the south of the corner.

**Discussion:**

After performing our site observations we reviewed the material provided and analyzed the wall for the actual as built condition. For our analysis we assumed the backfill was a granular material with a moist unit weight of 110 pounds per cubic foot, an internal angle of friction of 30 degrees and no cohesion. We based these parameters upon soils observed in the vicinity of the site. Because this wall has multiple levels of support we used a Peck's apparent pressure distribution for the lateral earth pressures on the wall. This pressure distribution assumes a constant pressure over the height of the wall rather than a linearly increasing pressure with depth. This distribution results in higher loads on the upper row of tie backs. We believe this is applicable to this situation and the failure of the top row of anchor rods confirms this. A uniform surcharge load of 200 pounds per square foot was assumed to account for construction loading. We have assumed all of the steel used in the wall conforms to ASTM A36 with a yield strength of 36 ksi. For our analysis of the timber ties we assumed they consisted of ponderosa pine which is the weakest species typically used for timber ties. Allowable Stress Design methodology was used for our analysis. The load distribution upon the lagging was determined by an equation published in an article for the Deep Foundation Institute Journal (November 2008) which was coauthored by one of the engineers involved in this project. Our calculations have been attached to this report.

Based upon our analysis we believe the soldier beams and lagging can adequately resist the loads applied provided the soil is adequately drained. It has come to our attention that several dry wells exist behind the wall that allow surface run-off to saturate the backfill soils. The soldier beams are not strong enough to resist the lateral earth pressures created by saturated soils. Although we can not visually observe the deadmen, we did not see any evidence of poor performance.

Railroad ties typically have an expected life span of thirty to forty years[1]. We expect that within the next five to ten years the ties used in this wall will begin to deteriorate to the point that re-facing the wall will become necessary. Considering the amount of moisture this wall is exposed to on a routine basis it is remarkable that the ties are in as good of condition as they are in.

[1] *The Tie Guide, Handbook for Commercial Timbers Used by the Crosstie Industry*, Prepared by the Railway Tie Association

We do not believe the anchor rods used to tie back the wall are adequate. This conclusion is substantiated by failures already observed in the upper row of tie backs. Based upon our analysis we believe the factor of safety in these anchor rods is 1.2 which is substantially below accepted industry standards of 1.5 to 2.0 depending upon the application. As the anchor rods fail the wall rotates outward which relieves some of the pressure on the wall temporarily but adds a significant amount of additional load to the surrounding anchors. Over time we expect that the wall will progressively fail as more anchors are overstressed. This type of failure will typically occur over an extended period of time unless an unusual condition such as a water line break occurs. The presence of utilities in the backfill could cause this condition to occur if a pipe is broken due to subsidence of the soils caused by wall movement. Another situation which could accelerate the rate of failure is compaction of the backfill behind the wall. It is our understanding that the Board would like to repair the parking lot above the wall. We assume that any repair work will require the use of vibratory compactors and other heavy machinery. This type of work will increase the loads applied to the wall.

**Recommendations:**

We recommend retaining a specialty contractor to install a new top row of tie backs to the portions of the wall that are taller than twelve feet. The design of this type of work is heavily influenced by the means and methods of the contractor. Typically this type of repair is designed by the contractors engineer and reviewed by the owner's engineer. We have requested proposals from three contractors we believe are reputable in this type of work. We anticipate the repair method proposed will consist of installing new tiebacks in between the soldier beams with walers (horizontal beams against the existing lagging) connecting the new tie backs to the existing soldier beams. The tie backs will likely be drilled through the backfill and into the hard bedrock. The hole is then filled with grout, anchoring it into the bedrock, and a metal rod is installed in the center to attach to the walers. <u>We do not recommend working on the parking lot until the wall has been repaired.</u>

We also believe it would be prudent to consider re-facing options at this time. Many contractors will not apply a new facing to a wall they did not install the tie backs in. This is due to the risk that if the facing fails due to improperly installed anchors they could be liable for the entire repair. Facing options could include precast concrete panels attached to the existing soldier beams or shotcrete. Shotcrete has the advantage that it is more adaptable to variable field conditions. Either method can have architectural features applied to make the wall more aesthetically pleasing. We have asked the contractors to include a separate proposal for re-facing the wall so that the board can consider the total cost of the repair. The work could be performed in phases so that the entire cost of the project could be spread over several years. Our suggestion for phasing is to perform all of the tie back work in the first phase and re-facing in the second phase. This will limit the additional mobilization costs associated with splitting up each phase of the work.

Another possible repair method is to install soil nails through the face of the existing wall and apply a shotcrete facing. A soil nail is similar to the tie back described above but more of them would be installed and they would not be attached to the existing soldier beams. They work by stabilizing the whole soil mass. This repair method would have to be performed all at once but it could be potentially cheaper overall. The existing wall would be left in place behind the new soil nail wall.

We also recommend removing the dry wells from behind the wall and replacing them either below the wall or finding another method to dispose of the surface run-off. At this time surface run-off is directed to the dry wells which allow it to soak into the backfill. It is also our understanding that leaks have been observed in the sewer mains. Saturating the backfill behind the wall can more than double the amount of pressure on the wall. Any leaking utilities should be repaired. This work should be performed after repairing the wall to avoid over stressing the existing wall.

**Limitations:**

This report is based upon our site observations, a review of the documents provided, and our experience with projects of this type. Our observations were limited by the presence of backfill, and other wall elements. Other unseen defects or conditions may exist that could affect the structural integrity of the wall observed. We believe this work was conducted to the standard of care ordinarily practiced by other engineers in this area at this time. No warranty is made, express or implied.

Thank you for the opportunity to work with you on this project, we appreciate your business. Please do not hesitate to call if you have any questions comments.

Respectfully,

**SCHMUESER GORDON MEYER**

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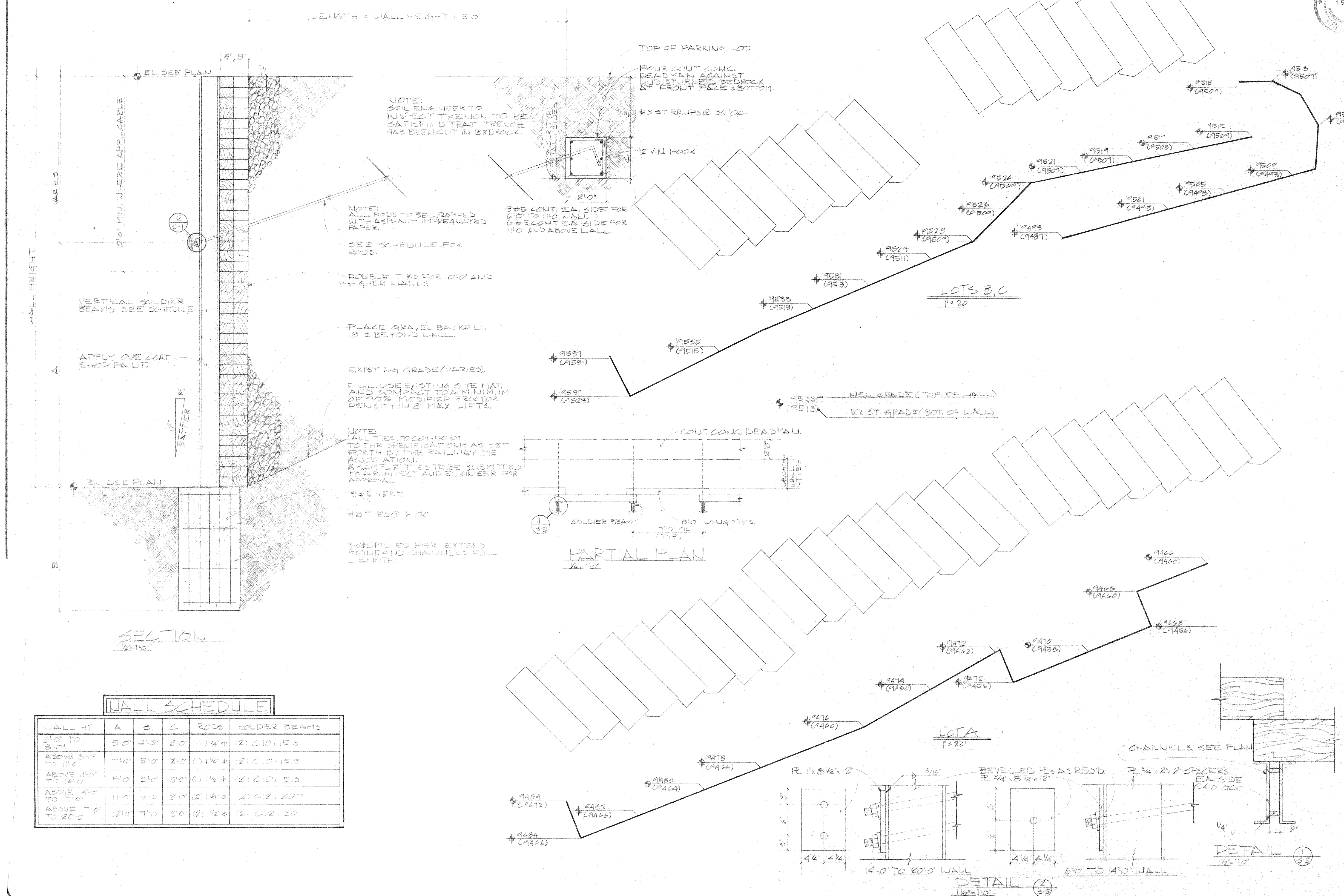
PROFESSIONAL ENGINEER

John J Boulden, PE
Structural Engineer

Reviewed By:

William B. Swigert, PE, SE
Sector Leader

Attachments: Original Design Plans
Calculations




# WALL SCHEDULE

| WALL HT. | A | B | C | RODS | SOLDIER BEAMS |
|---|---|---|---|---|---|
| 6'-0" TO 8'-0" | 5'-0" | 4'-0" | 2'-0" | (1) 1 1/4" φ | (2) C10 x 15.3 |
| ABOVE 8'-0" TO 11'-0" | 7'-0" | 5'-0" | 2'-0" | (1) 1 1/4" φ | (2) C10 x 15.3 |
| ABOVE 11'-0" TO 14'-0" | 9'-0" | 5'-0" | 3'-0" | (1) 1 1/2" φ | (2) C10 x 15.3 |
| ABOVE 14'-0" TO 17'-0" | 11'-0" | 6'-0" | 3'-0" | (2) 1 1/4" φ | (2) C12 x 20.7 |
| ABOVE 17'-0" TO 20'-0" | 12'-0" | 7'-0" | 3'-0" | (2) 1 1/2" φ | (2) C12 x 30 |

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MT. CRESTED BUTTE, COLORADO.

Phase
Job No.
Drawings RETAINING WALLS PLAN & DETAIL
Drawn
Date
Revised ⚠ ADDED 3/30/80
⚠ SHEET S-5
OF 5

## Determine Soil Parameters:

Based upon soils observed in the area assume a granular soil with the following properties:

$$\gamma_{moist} = 110\,pcf$$
$$\phi = 30^\circ$$
$$c = 0$$
$$\gamma_{sat} = 140\,pcf$$

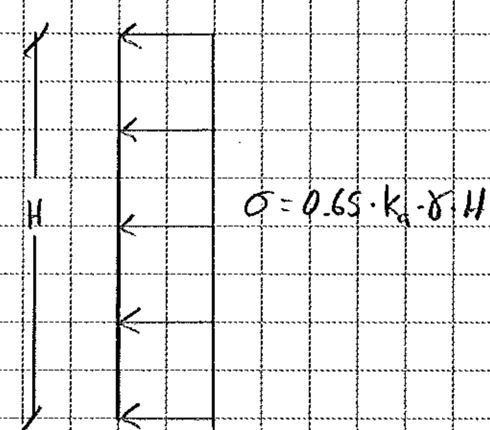
Based upon these parameters:

$$K_a = \tan^2(45 - 30/2) = 0.33$$
$$K_p = 1/0.33 = 3$$

Use Peck's Apparent Pressure diagrams due to tie backs.

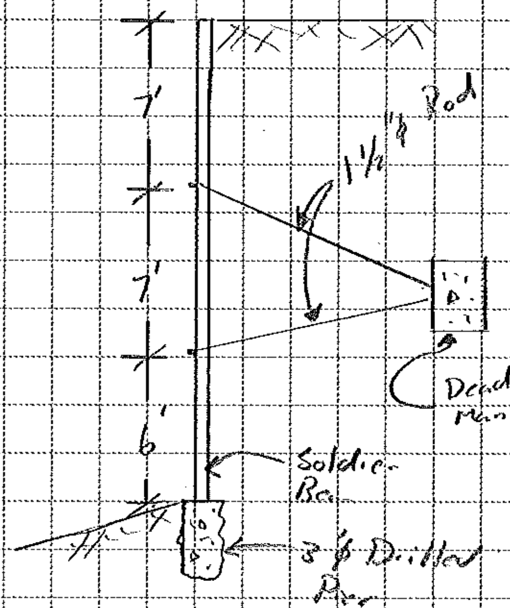
Worst case $H = 20'$

$$\therefore \sigma = 0.65 \cdot 0.33 \cdot 110\,pcf \cdot 20'$$
$$= 477\,psf$$




GLENWOOD SPRINGS 118 W. 6TH, SUITE 200 GLENWOOD SPRINGS, CO 81601 970-945-1004 FX: 970-945-5948
ASPEN P.O. BOX 2155 ASPEN, CO 81612 970-925-6727 FX: 970-925-4157
CRESTED BUTTE P.O. BOX 3088 CRESTED BUTTE, CO 81224 970-349-5355 FX: 970-349-5358
MEEKER 320 3RD ST. MEEKER, CO 81641 970-878-5180 FX: 970-878-4181

## Soldier Beams




Assume a uniform surcharge behind the wall of 200 psf to account for construction loads

Surcharge Pressure $= 0.33 \cdot 200\,psf = 66\,psf$

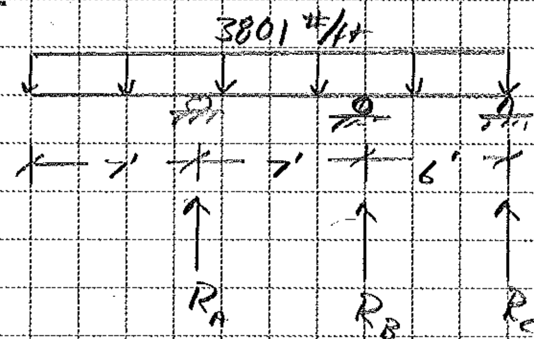
Total Pressure $= 66\,psf + 477\,psf = 543\,psf$

Distributed Load on soldier beam $w$

$w = 7' \cdot 543\,psf = 3801\ \#/ft$

Assume drilled pier resists lateral load but not moment because of slope @ toe of wall.

Free Body Diagram of Soldier Beam:




Beams appear to be (2) C12x20.7 with a 2" gap

$I = 258\,in^4$
$S = 43\,in^3$
$Z = 51.2\,in^3$

Assume A36 steel
$f_y = 36\,ksi$
$E = 29e^3\,ksi$

Model $R_A$ & $R_B$ as springs, assume 20' long

From $\Delta = \frac{PL}{AE} \Rightarrow P = \frac{\Delta AE}{L} \Rightarrow \frac{P}{1"} = \frac{1.77\,in^2 \cdot 29e^6\,psi}{240"} = 214\ k/in$

Spring Constant

Soldier Beams (cont.)

Beam was modeled by computer & the following results were obtained:

$R_A = 51.9$ kips
$R_B = 14.3$ kips
$R_C = 9.9$ kips
$M_{max} = 153.2$ k-in @ 17'-4" from top
$M_{min} = 1117.5$ k-in @ 7' from top

$$\text{Bending Stress} = \frac{1117.5\text{ k-in}}{51.2\text{ in}^3} = 21.8\text{ ksi}$$

$$\text{Allowable Stress} = 36\text{ ksi} \cdot 0.6 = 21.6\text{ ksi}$$ ← See Attached for more detailed analysis.

$$\text{Stress Ratio} = \frac{21.8}{21.6} = 1.01 \quad \text{Ok}$$

Check Lagging:

Because of soil arching the pressure upon the lagging is less than it is on the soldier beams.

$\sigma_{max} = K_A \cdot (w + 1.2 \cdot l \cdot \gamma)$ ← From Pirko/Boulden 2008

$\sigma_{max} = 0.33 \cdot (200\text{psf} + 1.2 \cdot 7' \cdot 110\text{pcf}) = 370$ psf

$M_{max} = 1.5 \cdot 370\text{psf} \cdot 7^2 = 27.2$ k-in $\quad S = \frac{12 \cdot 8^2}{12} = 64\text{ in}^3$

$$\sigma = \frac{27.2\text{ k-in}}{64\text{ in}^3} = 425\text{ psi}$$

Lagging (cont.)

The lowest strength material typically used for railroad ties is Ponderosa Pine.

Assume #2 Ponderosa Pine (very conservative)

$F_b = 475\ psi$

$C_D = 1.0$
$C_M = 1.0$
$C_t = 1.0$
$C_L = 1.0$
$C_F = 1.0$
$C_{fu} = 1.0$
$C_i = 1.0$
$C_r = 1.15$
$C_c = 1.0$

$F_b' = 475 psi \cdot 1.15 = 546 psi > 425\ psi$ ok

Check Rods:

Max Tension = 51.9 kips

$\sigma = \frac{51.9\ kips}{1.77\ in^2} = 29.3\ ksi$

The original plans do not specify the rod material

Assume A36 with $F_y = 36\ ksi$ $F.S. = \frac{36}{29.3} = 1.2$ $\boxed{Low}$

**Job Number** 2010-246
**Date** 12/29/2010
**Designer** JJB
**Beam** Soldier Beams

| **Section** | **C12X20.7** | |
|---|---|---|
| **fy** | **36** | ksi |
| **fu** | **58** | ksi |
| **Mu** | **558.75** | **k-in** |
| **Pu** | **0** | **kips** |
| **Vu** | **26.6** | **kips** |

| **Bending** | | |
|---|---|---|
| Φ | 0.9 | LRFD |
| Ω | 1.67 | ASD |
| E | 29000 | ksi |
| Ix | 129 | in^4 |
| Iy | 3.86 | in^4 |
| bf/2tf | 0 | |
| h/tw | 0 | |
| Sx | 21.5 | in^4 |
| J | 0.369 | in^3 |
| Zx | 25.6 | in |
| ry | 0.797 | in |
| Cw | 112 | in^6 |
| d | 12 | in |
| Tf | 0.501 | in |
| Ho | 11.499 | in |
| c | 1 | |
| rts | 0.98 | in |
| Lb | 48 | in |
| Flange λp | 10.8 | |
| Web λp | 106.7 | |
| Lp | 39.8 | in |
| Lr | 143.0 | in |
| Cb | 1 | |
| Fcr | 136 | ksi |
| Mcr | 2919 | k-in |
| Mnp | 891 | k-in |
| Mp | 921.6 | k-in |
| Mn | 891 | k-in |
| ΦMn | 802 | k-in |
| Mn/Ω | 534 | k-in |

| **Tension** | | |
|---|---|---|
| **Tensile Yielding** | | |
| Φ | 0.9 | LRFD |
| Ω | 1.67 | ASD |
| A | 6.08 | in^2 |
| Pn | 219 | kips |
| ΦTn | 197 | kips |
| Tn/Ω | 131 | kips |
| **Tensile Rupture** | | |
| Φ | 0.75 | LRFD |
| Ω | 2 | ASD |
| Ae | 6.08 | in^2 |
| Pn | 353 | kips |
| ΦTn | 264 | kips |
| Tn/Ω | 176 | kips |

**Refer to AISC Section D3 for effective area**

| **Shear** | | |
|---|---|---|
| Φ | 0.9 | LRFD |
| Ω | 1.67 | ASD |
| Web λp | 63.6 | |
| Tw | 0.282 | in |
| Aw | 3.4 | in^2 |
| Cv | 1.0 | |
| Vn | 73.1 | kips |
| ΦVn | 65.8 | kips |
| Vn/Ω | 43.8 | kips |

| **Compression** | | |
|---|---|---|
| Φ | 0.9 | LRFD |
| Ω | 1.67 | ASD |
| K | 0.7 | |
| KL/r | 39.1468 | |
| Fe | 186.8 | ksi |
| Fcr | 33.2 | ksi |
| Pn | 202 | kips |
| ΦPn | 182 | kips |
| Pn/Ω | 121 | kips |

| **Stress Ratios** | | |
|---|---|---|
| **Mode** | **LRFD** | **ASD** |
| Bending | 0.7 | 1.0 |
| Tension | 0.0 | 0.0 |
| Compression | 0.0 | 0.0 |
| Shear | 0.4 | 0.6 |

**Flange is compact for bending**
**Web is compact for bending**
**Web is compact for shear**

| **Compression/Bending Interaction** | |
|---|---|
| **LRFD Ratio** | **0.7** |
| **ASD Ratio** | **1.0** |

| **Tension/Bending Interaction** | |
|---|---|
| **LRFD Ratio** | **0.7** |
| **ASD Ratio** | **1.0** |