Del Mar Bluffs Stabilization Project 2 – Preserving Trackbed Support

Evaluation of Existing Seawalls

Submitted to:
North County Transit District
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By:
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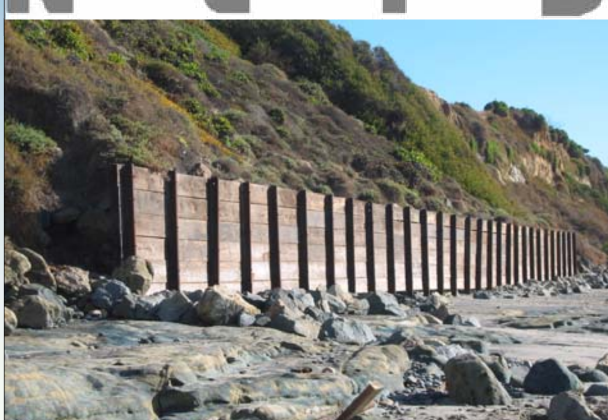
June 18, 2004



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DEL MAR BLUFFS STABILIZATION PROJECT 2 – PRESERVING TRACKBED SUPPORT

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I. INTRODUCTION

This report is written to provide information regarding existing seawalls for *Del Mar Bluffs Stabilization Project 2 - Preserving Trackbed Support*. The maintenance and repair of existing structures and drainage systems is an integral part of preserving trackbed support. Existing seawalls along the toe of the coastal bluffs in Del Mar are intended to provide some degree of protection against bluff toe erosion from wave action. In some cases, taller seawalls contribute to slope stability, and the loss of these walls could jeopardize trackbed support. This document will evaluate the present condition and provide an anticipated life expectancy of the existing seawalls along the Del Mar Bluffs.

II. SITE HISTORY

The NCTD railroad right-of-way is situated on the western edge of the City of Del Mar atop the 50- to 70-foot high coastal bluffs. The coastal bluffs supporting the railroad alignment are subject to ongoing erosion, landslides and surficial failures. As such, *Del Mar Bluffs Stabilization Project 2 – Preserving Trackbed Support* includes the design and installation of stabilization measures intended to preserve trackbed support and maintain the viability of rail operations for a minimum of 20 years.

In past attempts to maintain trackbed support, various mitigative measures have been implemented. Primarily in response to local landslides, several wooden and concrete seawalls have been installed at the toe of the bluff. The timber walls consist, in general, of vertical track rails embedded in caissons to form soldier piles; the piling is used to brace horizontal timber lagging. These walls were installed around 1964 and 1978. The concrete walls are the oldest walls, dating back to 1910; they were designed as gravity walls. Concrete walls often also serve as headwalls for drainage structures (these are designated with "BR *milepost*" on railroad plans).

In general, seawalls installed at the toe of bluff serve primarily to arrest erosion due to wave action. Where walls are taller and larger, the walls can also serve to passively retain the upper portions of the bluff. These larger walls may prove effective when calculating slope stabilities. Smaller walls would not significantly improve bluff stability, but would rather affect bluff retreat rates by slowing the deterioration at the toe of slope. But in all cases, the security of the bluff provided by the walls can be no greater than the condition of the wall itself. Located in the splash zone of the ocean, these walls are subject to aggressive wave attack. Wave action tends to undermine wall stability, and saltwater quickly corrodes steel members. The following sections will examine each wall and evaluate its present condition and its future effectiveness.

III. GRAVITY WALLS

A. GENERAL

Gravity walls are one of the most basic types of retaining walls. Gravity walls rely on their mass and cross-section geometry to resist the earth pressure that is attempting to move the structure in a lateral direction. The wall is designed such that its own weight resists overturning, and friction along the bottom of the wall resists sliding. Concrete gravity walls (Figure 1) may have little to no reinforcement, and thus may exhibit vertical cracking if the wall is long or contains bends; these cracks do not usually threaten overall stability.

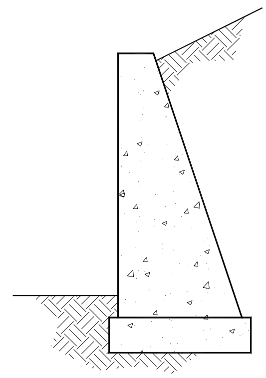

Figure 1. Typical Concrete Gravity Retaining Wall

Adequate bearing capacity is required below a gravity wall. The ground pressure is highest below the face of the wall, and if suitable bearing material is not utilized the wall may rotate as settlement occurs. For this reason, a footing may be placed below the wall to reach deeper and stronger soil material, or loose material may be replaced with a concrete or aggregate slurry.

Gravity walls are typically found in older construction projects, before economical plywood forms and higher-strength materials were common and/or available. Today the more common type of concrete wall consists of a relatively slender steel-reinforced cantilevered stem on a spread footing. This type of wall uses minimal material and utilizes the tensile capacity of reinforcing bars to resist bending.

Due to their simple construction and large mass, concrete gravity walls tend to perform well over long periods of time. They do not rely upon reinforcing steel for structural support, which can be advantageous in highly corrosive environments. Failure of these walls would usually be attributable to loss of bearing material or to a substantial change in its loading condition, such as the unanticipated addition of a slope behind the wall or a buildup of groundwater.

B. ANDERSON CANYON (BR 245.4)

The seawall at Anderson Canyon is a 150-foot-long concrete gravity wall with a 14-foot return wall. The thickness at the top of wall is 2 feet with an approximate back face batter of 2.75V:1H. Wall face heights vary from 18 feet at the south end to 12.5 feet at the north end, with the bottom of wall elevation remaining constant. The wall is located approximately between track stations 1481+93 and 1483+55.

Historical records indicate that some type of retaining structure was originally placed at this location in 1910. Thirty years later, in 1941, a landslide occurred which resulted in the need for a replacement structure. The records are not clear whether the new structure was an addition to the old one or was completely redesigned.

Figure 2. Gravity Wall at Anderson Canyon (BR 245.4)

Though there are no conspicuous details that give away the specific wall type, there are subtle clues that suggest this wall is a gravity wall and not a steel reinforced cantilevered stem wall. First, a very similar wall exists at the Stratford Court location which is known to be a gravity wall. Because the age of these two walls is similar it is reasonable to assume they are the same type of wall. Second, a construction joint exists inside the drainage outlet 9 feet from the wall face. This dimension is consistent with a typical backwall batter that would be required on a gravity wall of this height.

The overall condition of the wall appears very good. No major cracks exist except for those due to thermal effects at vertical control joints. Calculations show the wall to be adequately stable for both sliding and overturning. Bearing pressure under the toe of the wall is high, but it is assumed that an aggregate slurry exists beneath the wall which is much stronger than typical soil. The assumed presence of an aggregate slurry is based on the fact that aggregate slurry is visible beneath the wall at BR 245.16 and construction methods are likely similar for both walls. Since the backfill soil rests at a steep angle it is unlikely that an increase in soil loading will occur.

Loss of soil foundation material is not evident along the face of the main wall, but some loss is evident along the base of the return wall. However, as it stands now, the soil loss at the return wall is not undermining the structural integrity. Because of this, along with its short length and short average height, it is reasonable to assume that loss of the return wall is not a highly probable event in the near future.

Based on the overall condition and projected time-based structural changes, calculations show that the wall will remain intact and will provide support to the bluff for the next 20 years.

C. STRATFORD COURT (BR 245.16)

The Stratford Court seawall consists of a 60-foot length of wall with a 20-foot ascending return wall. The wall thickness is 2 feet at the wall top and slopes to 5.75 feet at the wall base, giving the back face a 3.75V:1H slope. The face height is a constant 14 feet to top of foundation material. The wall is located approximately between track stations 1493+36 and 1493+96.

Historical records indicate the construction date of the wall to be 1910, which corresponds to that of the first Anderson Canyon wall. The original drainage outlet has been abandoned and the wall opening has been filled with concrete. An exposed section at the south end of the wall reveals the sloping back face which characterizes a gravity wall. The return wall also has a sloped back face, which demonstrates its function as a gravity wall retaining soil in a direction parallel to the beach. Roughly six inches of aggregate slurry is exposed at the base of the wall.

Figure 3. Gravity Wall below Stratford Court (BR 245.16)

Calculations for overturning and sliding show the Stratford Court wall to be currently stable. Like the Anderson Canyon wall this wall has high footing toe pressures, but the aggregate slurry is providing the required strength to resists these large pressures.

The overall condition of the wall is reasonable considering its age; however, there is a major crack (approximately 0.6” wide) at the interface between the main wall and return (Figure 4). This crack may be attributed to movement of the wall, thermal loads or a combination of both effects. A micrometer gauge was installed on the crack in 2002 by Leighton & Associates, and no displacement has been observed in the past year indicating that the wall is stable.

Figure 4. Crack monitor on Stratford Court Wall

An additional concern is soil loss at the foundation. Because the main portion of wall is founded on an aggregate slurry, soil loss around the foundation should not be a problem. Erosion is evident at the foot of the return wall and if this progresses it could result in undermining of this portion of the wall. However, the bulk of the erosion seems to be directed out from the footing and not under it, so realistically the return wall foundation should not be compromised.

In conclusion, as far as the projected condition of the Stratford Court seawall is concerned, it appears that it will adequately and reliably retain soil for the next 20 years.

D. STATION 1541

A 30-foot-long concrete wall exists at the far north end of the bluffs. Little historical information exists about this wall, but due to its similarity to the other concrete walls at the bluffs it is most likely a gravity wall dating back to around 1910.

A large amount of soil exists in front of the wall, which requires an educated guess as to the wall height. Based on the distance from top of wall to the base of competent foundation material it appears that the wall height is 15 feet. This is within the same range as heights at Stratford Court and Anderson Canyon.

Figure 5. Gravity Wall at Station 1541

The soil in front of the wall will tend to protect the foundation from scour. The back slope has a fairly steep angle with a significant amount of vegetation so the possibility of additional soil loading is not high. Thus, this wall should last an additional 20 years without major problems.

IV. <u>TIMBER WALLS WITH PILING</u>

A. GENERAL

Timber walls with piling (Figure 6) are basically a modification of a system known as soldier piles. The vertical shaft of a soldier pile will provide lateral support to a retained portion of earth by transferring the lateral force through the pile and deep into competent foundation material. There is no need for a wide base or spread footing to support the upper portion of the wall.

The timber walls use a track rail embedded in concrete to function as a soldier pile. Timber lagging is provided along the back of these rails to support the earth between rails. This allows for a wide rail spacing. Connection between the rail and timber is made with "U" bolts, which will primarily function as restraint while the wall is being backfilled.

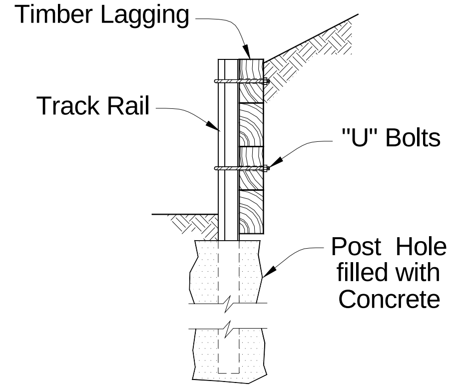

Figure 6. Typical Timber Retaining Wall with Piling

Timber walls are an economical way of providing protection to the bluffs because of their use of widespread railroad materials such as track rails. Thus, one would typically encounter this type of wall around railroad facilities.

Because of their use of materials that are vulnerable to deterioration and corrosion, timber walls have a relatively short life span. The matter is made worse when these types of materials are in contact with marine environments. Steel is highly susceptible to corrosion, especially in sea water. The timber walls at the bluffs are located in what is known as the splash zone which tends to be the harshest type of environment for corrosion. These are areas that tend to go through cycles of wet and dry periods as the result of wave and tide action. An average corrosion rate was determined from field measurements and confirmed with theoretical values. These corrosion rates where used to project the strength of the rails over time and thus determine an anticipated lifespan.

Figure 7. Corrosive scaling on rails

Timber, if treated properly, can withstand very harsh environments, but if left untreated the durability is marginal. It is not certain whether or not the timber in these seawalls is pressure treated. In projecting the lifespan of the timber at the Del Mar bluffs visual inspection was used to compare the walls of different age. A qualitative deterioration rate could then be used to determine the condition of the timber after 20 years.

B. STATION 1514

Between 7th and 8th Street a 70-foot timber wall stands at the base of a pedestrian trail. The height is a constant 5.5 feet from top of wall to top of concrete post hole. From historical records the build date on the wall appears to be 1977. Landslides during the winter months necessitated a seawall at this location to keep surf activity from attacking the toe of slope and destabilizing the slide area further.

About one foot of sand exists above the top of the concrete post hole at present, however this height varies seasonally. One concern is the possibility of soil eroding and compromising the foundation. However, formational material underlies the base of this wall; therefore the foundation should remain intact though the top sand may shift. The depth of the concrete piles is estimated to be at least four feet deep, similar to the timber wall at Twelfth Street.

Figure 8. Timber Wall at Station 1514

As the rails have corroded their strength capacity has decreased. Assuming that the load demand on the rails will be constant for the next few decades, a chart of safety factor versus time has been created (see Appendix B). From this analysis a failure date in 2049 is projected for the rails, which demonstrates they will likely remain intact for at least the next 20 years.

The condition of the wood timber is good when compared to the older timber wall built in 1964 between 11th and 12th Street. Based on the 13-year difference between these walls, the projected condition of the timber in 20 years will probably be poor, but intact.

Because the wall is not very tall the soil loading is low, which indirectly benefits the longevity of this wall. The assumption that the load is not expected to change over time is based on the backfill soil's steep angle which won't allow much additional soil to build up. Thus, based on the expected rate of deterioration of the wall over time it should remain sound for the next 20 years.

C. TWELFTH STREET (BR 244.4 & BR 244.5)

The Twelfth Street wall extends 388 feet between 11th and 12th Street from station 1530+16 to 1534+04. The exposed height from sand level is roughly 3.2 feet, but the total height from top of concrete post hole is closer to 5.3 feet. The build date is 1964, which makes it the oldest timber wall at the bluffs. An additional 16-inch timber plank was added to the wall in 1981 as a way of extending the height of the wall. Design plans were discovered for this wall which provided a great benefit in evaluating its current and anticipated state.

At present the wall has 2 feet of cover in the form of sand and rip-rap in front. The presence of rip-rap will help maintain this cover over time, but it can't be completely counted on. One concern is that the as-built plans illustrate the concrete post hole extending only 4 feet deep. Calculations showed this to be an inadequate length if the cover is removed, which immediately lends doubt to the integrity of the wall over the next 20 years.

Because of their age the rails have already seen a high amount of corrosion and diminished strength capacity. Calculations project the rail to fail in 2022 or 18 years from now, which further demonstrates the inadequecy of this wall over time.

Figure 9. Timber Wall at Twelfth Street

The condition of the wood is poor. The timber height extension which was placed 20 years ago has already failed in one section at the south end. Moreover, these extensions are held to the main wall with timber vertical posts that are in very poor condition. These extensions are allowing large deflections which are especially noticeable near the north end of the wall.

Figure 10. Failure of Timber Wall Extension

From the standpoint of foundation integrity, rail corrosion, and wood deterioration this wall will not provide the required reliability to last the next 20 years. More than likely the first mode of failure will be the loss of the timber extension over an extended period of time. The failure of the foundation is linked to the rather erratic loss of beach sand, while corrosion of the rail is a steady process that will eventually compromise the entire system.

D. THIRTEENTH STREET

The timber wall at Thirteenth Street is the tallest of the timber seawalls in Del Mar. It stretches 200 feet along the bluff from Station 1536+88 to 1538+88 and stands 8.2 feet from top of wall to top of post hole. This wall was built just after the Station 1514 wall in 1978, thus the two walls are markedly similar. The rains of the winter months started landslides, which required a seawall for additional protection.

Though the current amount of sand cover at the front of the wall is negligible, it appears that formational material begins at a very shallow depth. This is a benefit to the foundation's longevity. An existing historical construction inspection diary dated 3/28/78 indicates that the piles extend 20 feet, but confirmation is difficult to obtain. Calculations show that a depth of around 15 feet is required for stability. Based on the assumed pile depth and the fact that the wall has lasted over 20 years already, it is reasonable to assume that the foundation is performing satisfactorily. There is a small gap between the formational base and the bottom horizontal timber which could allow sea water to scour the backfill during seasons where the level of beach sand is low enough to expose this opening.

Figure 11. Timber Wall at Thirteenth Street

The materials are showing signs of aging. As would be expected corrosion is slowly degrading the strength capacity of the rails. However, the projected failure date of the rails is 2051, which suggests they will perform well through the next 20 years. The timber is holding up adequately. Though the wood is likely to degrade to a poor state over the next 20 years, it should not have any significant problems.

The one concern for this wall is a significant lateral deflection of 5 inches at the top of wall that occurs over about 40 feet of the wall length. Calculations do not support that the deflection is a result of a typical amount of soil loading deflecting the rail. One possible cause may be that a small landslide has occurred since the wall has been standing. The initial loading from the slide may have been much higher than a typical soil loading because of dynamic effects. The other possibility is that the foundation is poorly designed and is allowing a rotation at the base.

Though the deflection is a concern, it is not grounds for rejection. So based on its overall condition, as well as the low probability of higher future soil loading, this wall should perform well for the next 20 years.

V. <u>MAINTENANCE ISSUES</u>

The preceding discussion of remaining life span of existing seawalls assumes that no significant maintenance is performed on the walls. This section will discuss options for seawall preventive maintenance and repair. Constraints on structural improvements are likely to be imposed from an environmental standpoint. These must be determined on a site-specific basis if preventive maintenance and repair is determined to be required. Maintenance of seawalls may need to comply with the California Coastal Act, Department of Transportation 4(f) requirements, Del Mar's Beach Preservation Initiative and/or other regulatory acts and agencies.

A. GRAVITY WALLS

Because the existing gravity walls are unreinforced and retain soil on the basis of their mass alone, there is little structural maintenance that would be practical for these walls. The most visible deterioration of these walls is of a cosmetic nature in the form of cracks and localized spalling. If desired, these can treated with patches and pressure-injection grouting. If the concrete itself is corroding, then painting or other corrosion-inhibiting coating may be appropriate for the wall surfaces.

The primary stability concern for these walls would likely be the undermining of the wall foundation. This could be mitigated by the placement of additional soil or concrete around the base of the wall if undermining has not yet occurred. If foundation material has already been removed from below the wall, then more aggressive repairs would be required such as using soil-compaction grouting to re-establish bearing capacity below the wall footing.

B. TIMBER WALLS WITH PILING

The existing timber walls with steel rail piling have a shorter expected life span than the concrete gravity walls due to the rapid deterioration of timber and steel under harsh environmental conditions. The wood lagging undergoes regular wet-dry cycles and tends to progressively deflect under constant loading with time. The steel rail piling experiences corrosion which slowly reduces its effective capacity to withstand loading demands. Maintenance and repair of these walls is much more likely to be required than for concrete gravity walls.

The timber wall at Twelfth Street is showing significant signs of distress and will require maintenance to survive an additional two decades. The wall extension built in 1981 has already failed in some locations and is deflecting considerably along the majority of its length. The primary weakness of the extension is the timber vertical support members. The likely minimum corrective measure for this wall would be the replacement of these members with new pressure-treated timber or steel sections. This would stabilize the uppermost portion of the wall.

Other maintenance and repair measures are relevant to all of the timber walls. Of special concern is the deterioration of the vertical steel rails due to corrosion. The corrosion rate of the surface of these rails is estimated at 0.004 in/yr. The simplest method of treating steel beams for corrosion is to clean the existing surfaces by blasting and then paint the beam surfaces with a protective organic or metallic coating. Protective painting would slow the corrosion process, and would require ongoing maintenance to reapply the coating. Cathodic protection (connecting the rails to sacrificial magnesium anodes) is also a viable option to arrest the corrosion process, although the installation of this type of system can be expensive. Installed properly, a cathodic protection system could effectively halt corrosion of the steel members.

VI. CONCLUSIONS

Table 1. Summary

Wall Type	Location	Remaining 20-year design life
Gravity Wall	Anderson Canyon	Yes
Gravity Wall	Stratford Court	Yes
Gravity Wall	Station 1541	Yes
Timber Wall	Station 1514	Yes
Timber Wall	Twelfth Street	No
Timber Wall	Thirteenth Street	Yes

Table 1 summarizes the results for all walls analyzed in this report. Of the six seawalls at the Del Mar Bluffs, only one, the Twelfth Street timber wall, is <u>not</u> expected to last through the next 20 years. Although some assumptions were required to estimate the remaining service life of each wall, these calculations also draw upon engineering judgment as a way to interpret analysis results and to arrive at a conclusion for each wall.

APPENDIX A

REFERENCES

Leighton, 2001, *Del Mar Bluffs Geotechnical Study. Part 1: Geotechnical Evaluation, Vol. I*, Project No. 040151-001, January 11, 2001.

Leighton, 2003, *Supplemental Geotechnical Evaluation and Determination of Site Specific Conceptual Repair Alternatives*, Project No. 040151-009, June 2, 2003.

APPENDIX B

CALCULATIONS

STRUCTURAL CALCULATIONS

Del Mar Seawalls

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Project: Del Mar Sea Walls	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: B-1
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Checked:	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Date: 4/03
		Revised:

Observed Rail Corrosion

HW - head width

HT - height

o Station 1514 MP 244.8 (1977)

Rail 132RE

	Original	Corroded	Change
HW	3"	2 3/4"	-1/4
Per Side			-.125
Per year			$\boxed{-.00481}$

o 12th Street MP 244.47 (1964)

Rail 90RE

	Original	Corroded	Change
HW	2 9/16"	2 1/4"	-5/16
Per Side			-.156
Per year			$\boxed{-.00400}$

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o 13th Street MP 244.33 (1978)

Confirmed by phone conversation with Leighton + Assoc.

Rail 132 RE

	Original	Corroded	Change
HW	3"	2 13/16"	3/16
Per Side			-.0937
Per Year			$\boxed{-.00375}$

Average corrosion rate per year per side:

$$\frac{.00481 + .00400 + .00375}{3} = \boxed{-.00419}$$

Predicted Corrosion Over Time

	Year Built	Corrosion Rate (in/year)	Amount of Corrosion at 10 Year Intervals (in) 2003	2013	2023	2033	2043	2053	2063
Station 1514 MP 244.8 Rail 132 RE	1977	0.00419	0.125	0.167	**0.209**	0.251	0.293	0.335	0.376
12th Street MP 244.47 Rail 90 RE	1964	0.00419	0.156	0.198	**0.240**	0.282	0.324	0.366	0.407
13th Street MP 244.33 Rail 132 RE	1978	0.00419	0.094	0.136	**0.178**	0.219	0.261	0.303	0.345

Note: Value at 2003 is actual corrosion value

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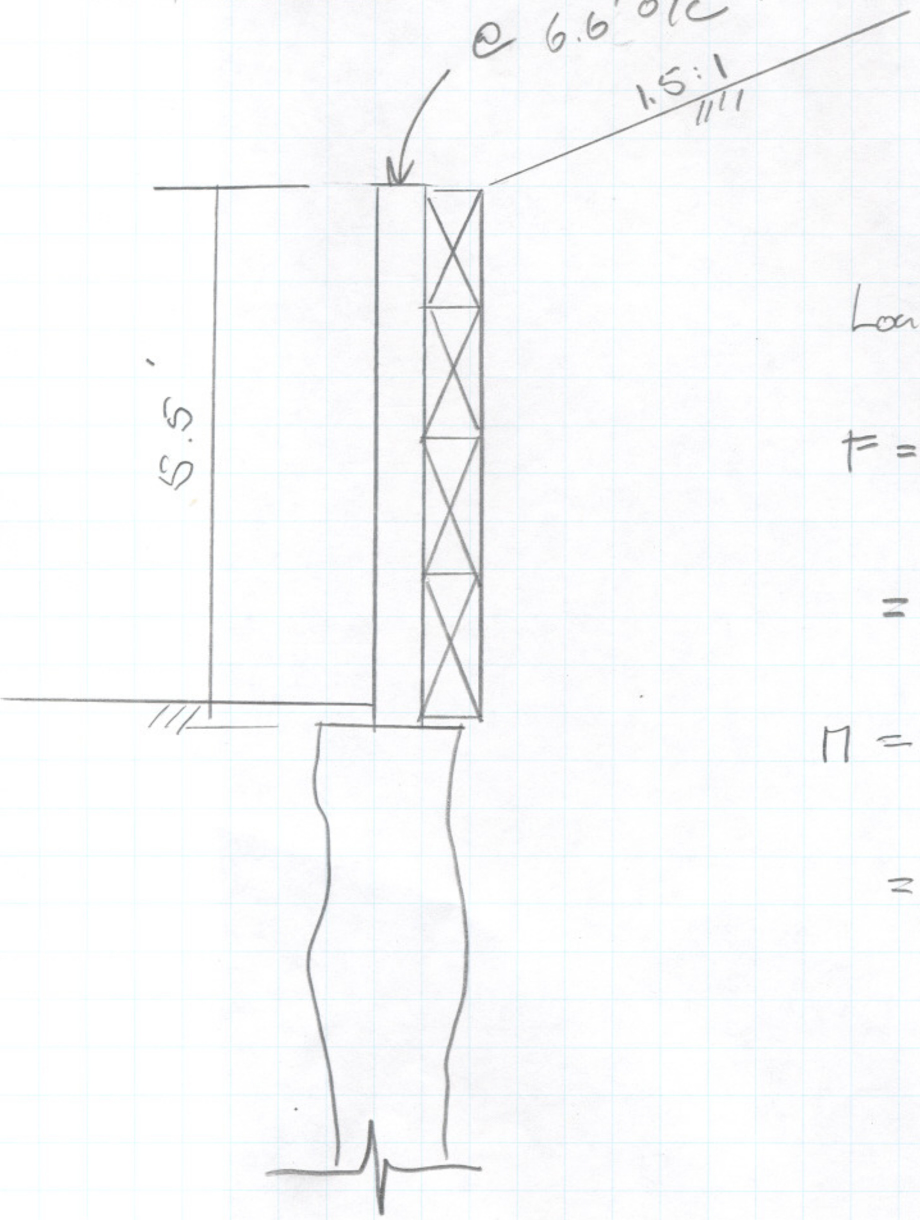
Station 15th Street



Load @ Bottom of Rail:

$$F = (5.5)^2 (65\,pcf)\left(\frac{6.6'}{2}\right)$$

$$= 6.49\,k$$

$$M = (6.49\,k)\left(\frac{5.5'}{3}\right)$$

$$= 11.9\,k\,ft$$

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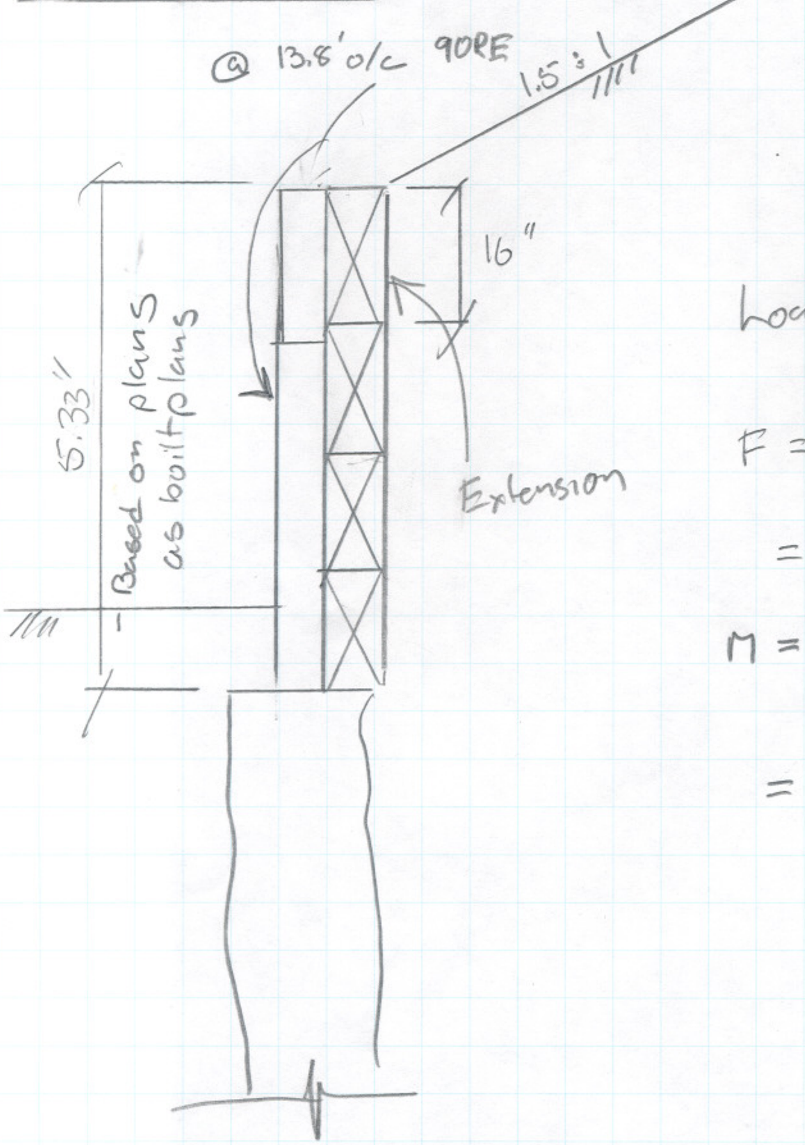
12th Street



Load @ Bottom of Rail:

$$F = (5.33)^2 (65\,pcf)\left(\frac{13.8}{2}\right)'$$

$$= 12.7\ K$$

$$M = (12.7\ K)\left(\frac{4.67'}{3}\right)$$

$$= 19.8\ K\,ft$$

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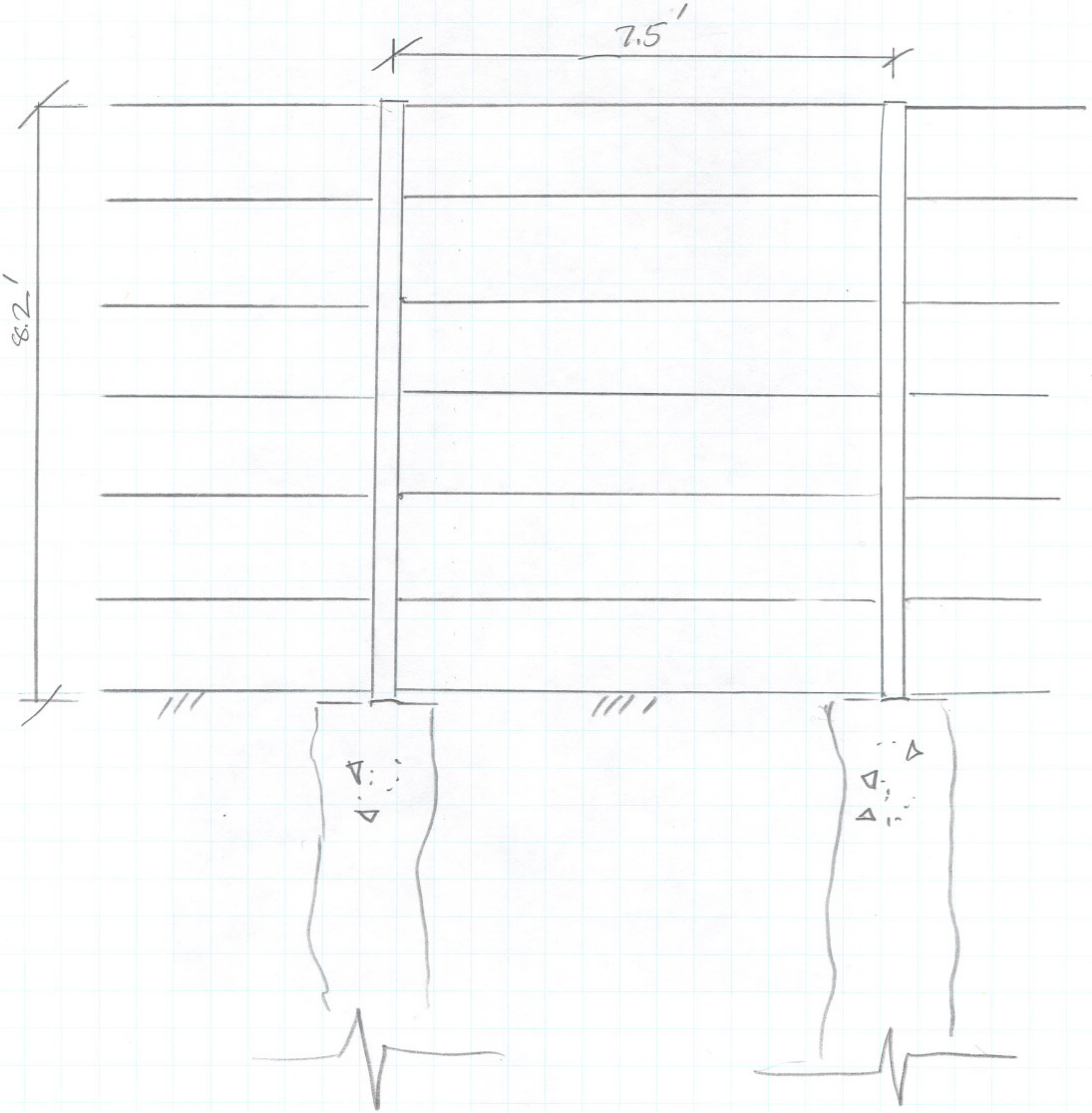
13th Street



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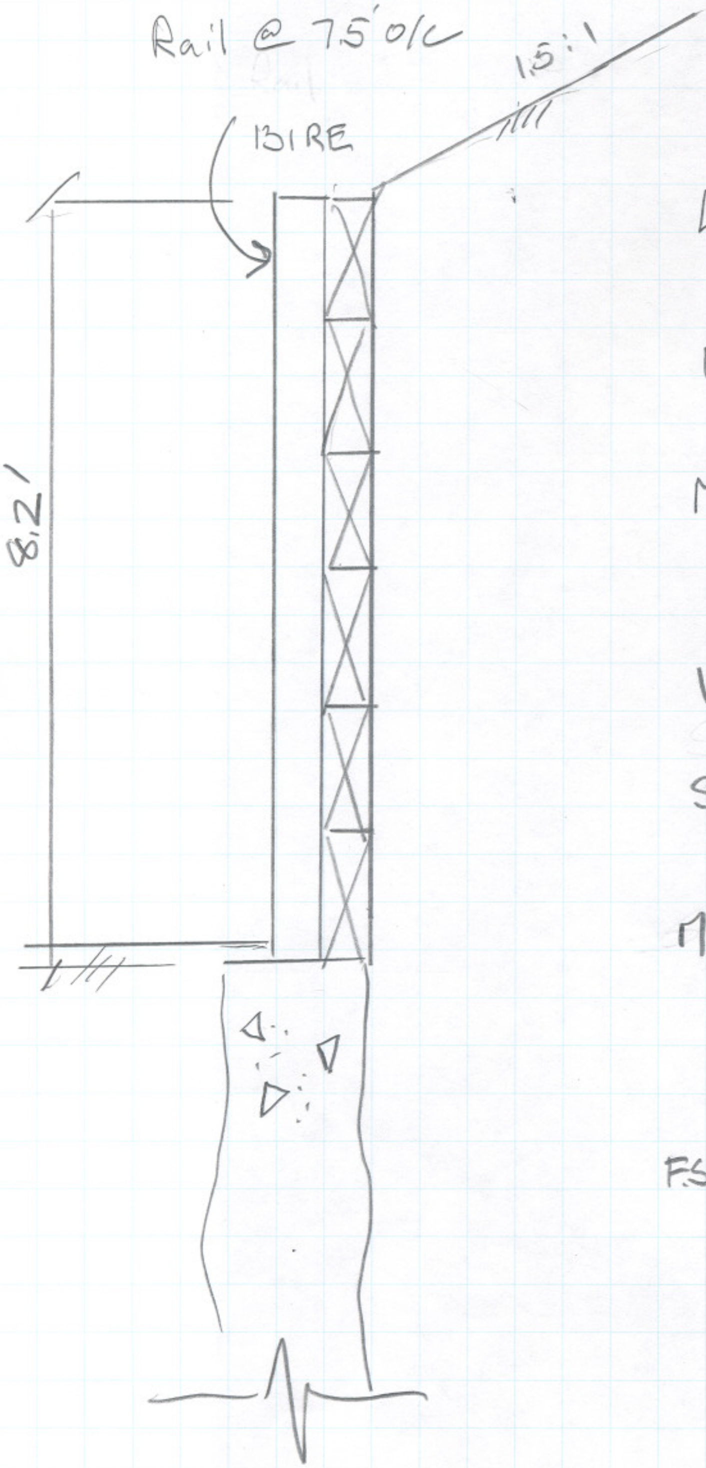

Load @ Bottom of Rail

$$F = \frac{(8.2')^2(65\,pcf)(7.5')}{2} = 16.3K$$

$$M = (16.3\,K)\frac{(8.2')}{3} = 44.5\,Kft$$

When Built:

$S_x = 22.5\,in^3$

$$M_y = 70ksi\frac{(22.5in^3)}{12} = 131\,Kft$$

$$F.S. = \frac{131\,kft}{44.6} = 2.94$$

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Check deflection:

$$I = 57.4 in^4$$

$$\Delta_{max} = \frac{Wl^3}{15EI} = \frac{(16.3\,K)(8.2' \times 12)^3}{15(29000ksi)(57.4 in^4)}$$

$$= .62 in$$

Check Capacity in 2023:

Corrosion = .178 in per side $\Rightarrow S_x = 14.7 in^3$

$$M_n = f_y S_x = 70ksi\left(\frac{15.8 in^3}{12}\right) = 92.2\,kft$$

$$\text{Safety Factor} = \frac{92.2\,kft}{44.5\,kft} = \underline{2.07}$$

Shear: $d = 7.125 - 2(.178 in) = 6.77 in$

$t = .66 - 2(.178 in) = .304 in$

$f_{yv} = .6(70ksi) = 42ksi$

$$V_n = 42ksi(.304 in)(6.77 in) = 86\,K$$

$$\text{Safety Factor} = \frac{86K}{16.3K} = \underline{5.28}$$

Rail Section Properties

Station 1514

Corrosion	I	N/A	Depth	N/A 2	Sx
0.125	65.1	3.21	6.875	3.665	17.76
0.209	50.5	3.27	6.707	3.437	14.69
0.293	36.5	3.45	6.539	3.089	10.58

12th

Corrosion	I	N/A	Depth	N/A 2	Sx
0.156	22.8	2.6	5.313	2.713	8.40
0.24	15.6	2.76	5.145	2.385	5.65
0.261	13.5	2.84	5.103	2.263	4.75

13th

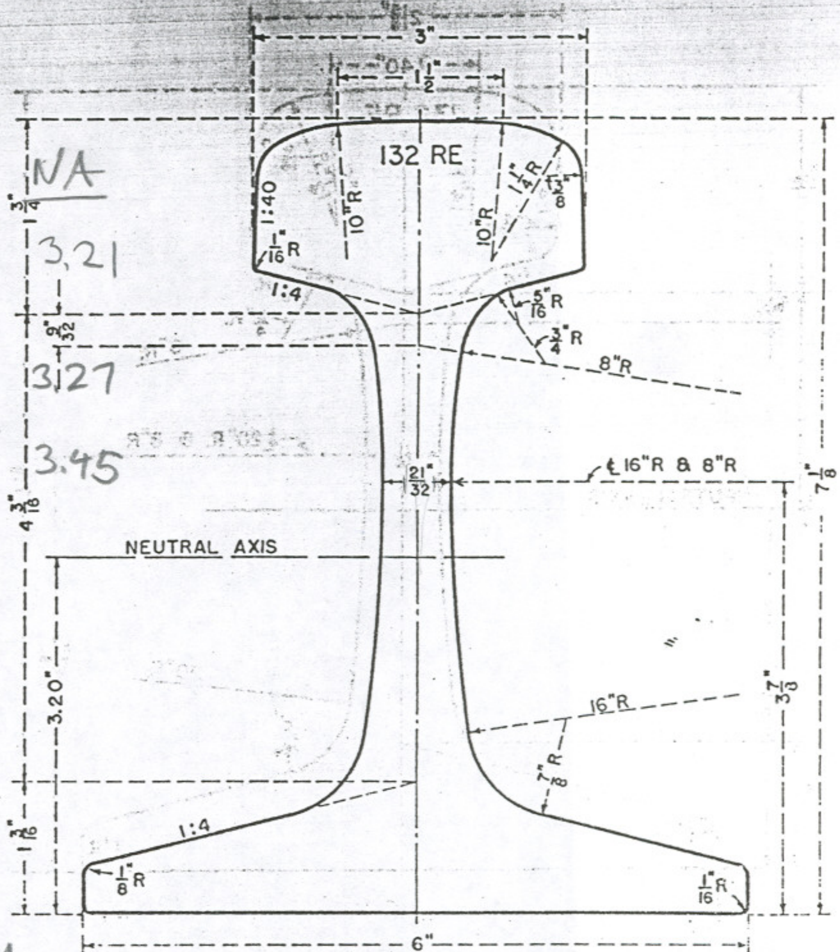
Corrosion	I	N/A	Depth	N/A 2	Sx
0.094	70.6	3.21	6.937	3.727	18.94
0.178	55.8	3.24	6.769	3.529	15.81
0.303	35	3.47	6.519	3.049	10.09

(1962 A)

(Reapproved without change 1962)

Station 1614

7.5th Street	Offset	I	NA
2003	.125	65.1	3.21
2023	.209	50.5	3.27
2043	.293	36.5	3.45



13th Street	Offset	I	NA
2003	.094	70.6	3.21
2023	.178	55.8	3.24
2053	.303	35.0	3.47

	Area Sq In	Percent
Head	4.42	34.1
Web	3.66	28.3
Base	4.87	37.6
Total	12.95	100.0

Moment of inertia	88.2
Section modulus, head	22.5
Section modulus, base	27.6
Ratio m.i. to area	6.81
Ratio s.m. head to area	1.74
Ratio height to base	1.19
Calculated weight, lb per yd	132.1

Fig. 5—132 RE rail section.

References, Vol. 21, 1920, p. 1455; Vol. 34, 1933, pp. 640, 824; Vol. 39, 1938, pp. 397, 802 Vol. 48, 1947, pp. 662, 908; Vol. 54, 1953, pp. 1177, 1413; Vol. 63, 1962, pp. 498, 768.

(1971)

Note: Offsets drawn in AutoCad, section properties determined in AutoCAD

B 11

12th Street	Offset	I	N/A
2003	.156	22.8	2.60
2023	.240	15.6	2.76
2028	.261	13.5	2.84

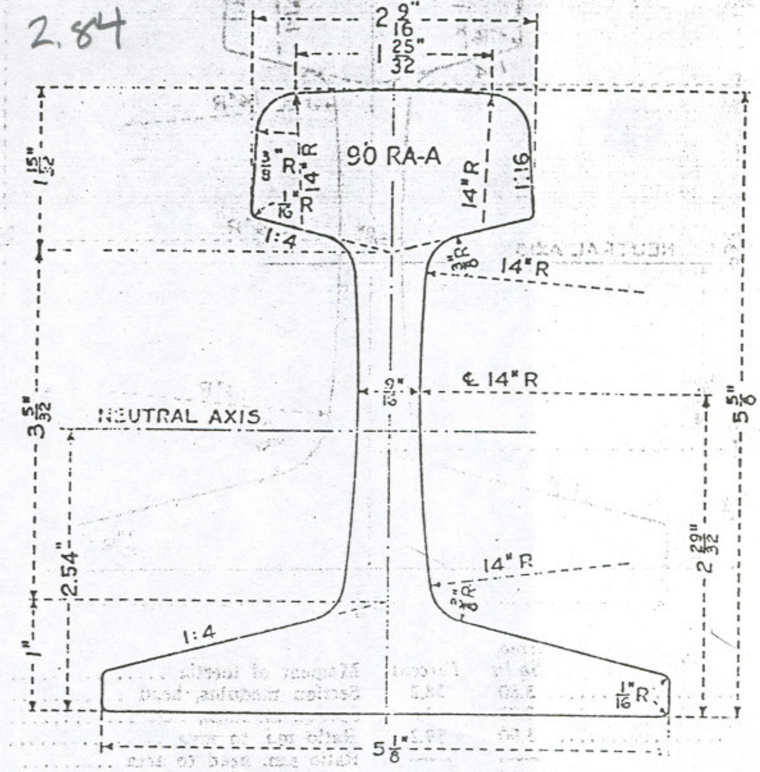
Part 1

Design

RECOMMENDED RAIL SECTIONS

1962

(Reapproved without change 1962)



	Area Sq In	Percent
Head	3.19	36.3
Web	2.12	24.0
Base	3.51	39.7
Total	8.82	100.0

Moment of inertia	38.7
Section modulus, head	12.6
Section modulus, base	15.2
Ratio m.i. to area	4.39
Ratio s.m. head to area	1.43
Ratio height to base	1.10
Calculated weight, lb per yd	90.0

Fig. 1—90 RA-A rail section.

References, Vol. 16, 1915, pp. 397, 1117; Vol. 49, 1948, pp. 375, 614; Vol. 54, 1953, pp. 1177, 1413; Vol. 63, 1962, pp. 498, 768.

Rail Capacity Over Time - Station 1514

M demand (K)	11.9
fy (ksi)	70

Year	1977	2003	2013	2023	2033	2043	2053
Sx (in^3)	22.5	17.8	**16.2**	14.7	**12.6**	10.6	**8.6**
Mn (K-ft)	131	104	95	86	74	62	50
SF	11.03	8.73	7.96	7.20	6.20	5.20	4.19

V demand (K)	6.49
fyv (ksi)	42

Year	1977	2003	2013	2023	2033	2043	2053
t (in)	0.66	0.41	0.32	0.24	0.15	0.07	-0.01
d (in)	7.13	6.88	6.79	6.71	6.62	6.54	6.46
Vn (K)	196	117	92	67	43	20	-3
SF	30.26	18.07	14.17	10.36	6.64	3.01	-0.53

Min SF	11.03	8.73	7.96	7.20	6.20	3.01	-0.53

Rail Capacity Over Time - 12th Street

M demand (K)	19.8
fy (ksi)	70

Year	1964	2003	2008	2013	2018	2023	2028
Sx (in^3)	12.6	8.4	**7.7**	**7**	**6.3**	5.6	4.8
Mn (K-ft)	74	49	45	41	37	33	28
SF	3.71	2.47	2.27	2.06	1.86	1.65	1.40

V demand (K)	12.7
fyv (ksi)	42

Year	1964	2003	2008	2013	2018	2023	2028	2033
t (in)	0.56	0.25	**0.21**	0.17	**0.13**	0.08	**0.04**	0.00
d (in)	5.63	5.31	**5.27**	5.23	**5.19**	5.15	**5.10**	5.06
Vn (K)	133	56	46	37	27	18	9	0
SF	6.72	2.83	2.34	1.85	1.38	0.91	0.45	0.00

Min SF	3.71	2.47	2.27	1.85	1.38	0.91	0.45

Rail Capacity Over Time - 13th Street

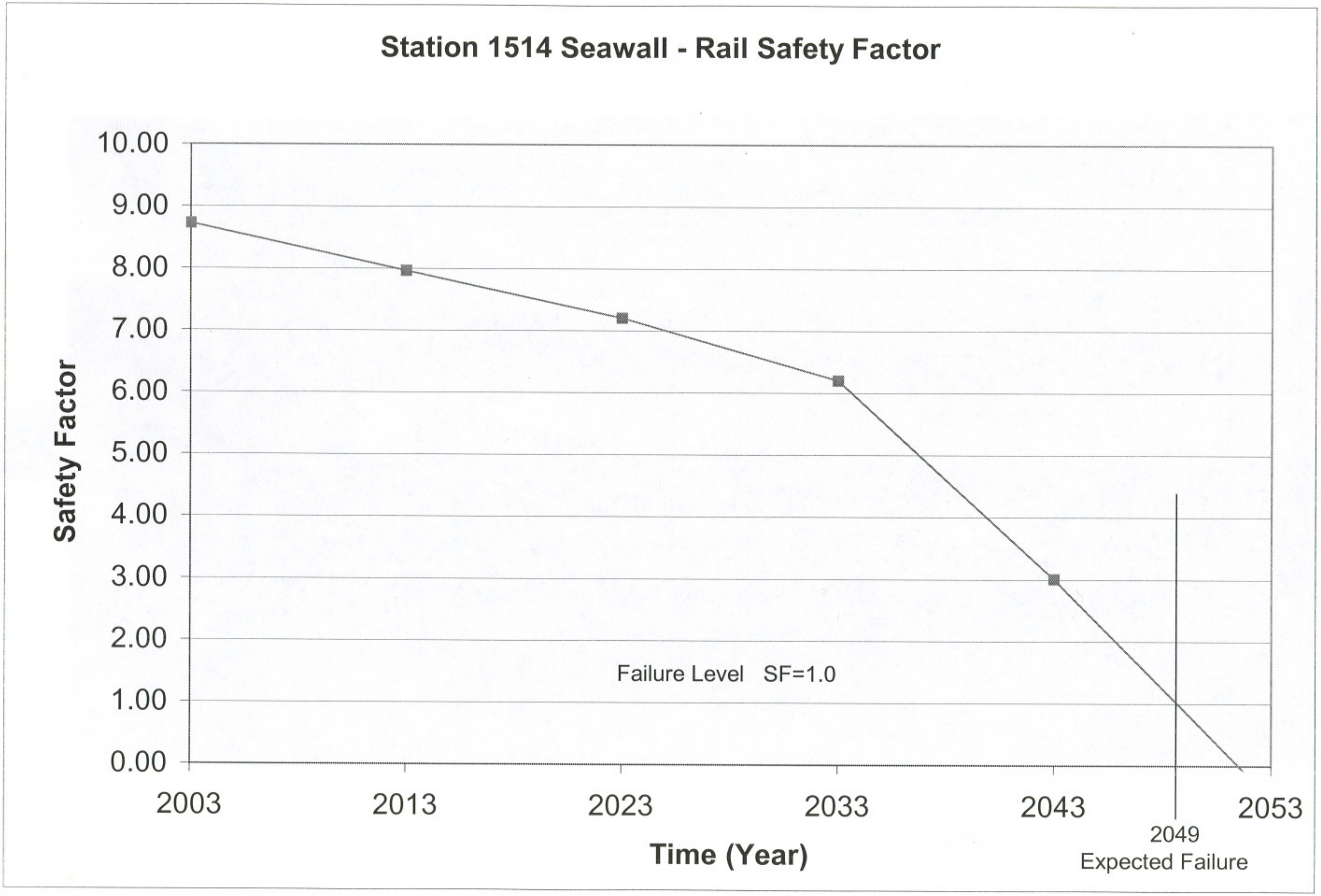
M demand (K)	44.5
fy (ksi)	70

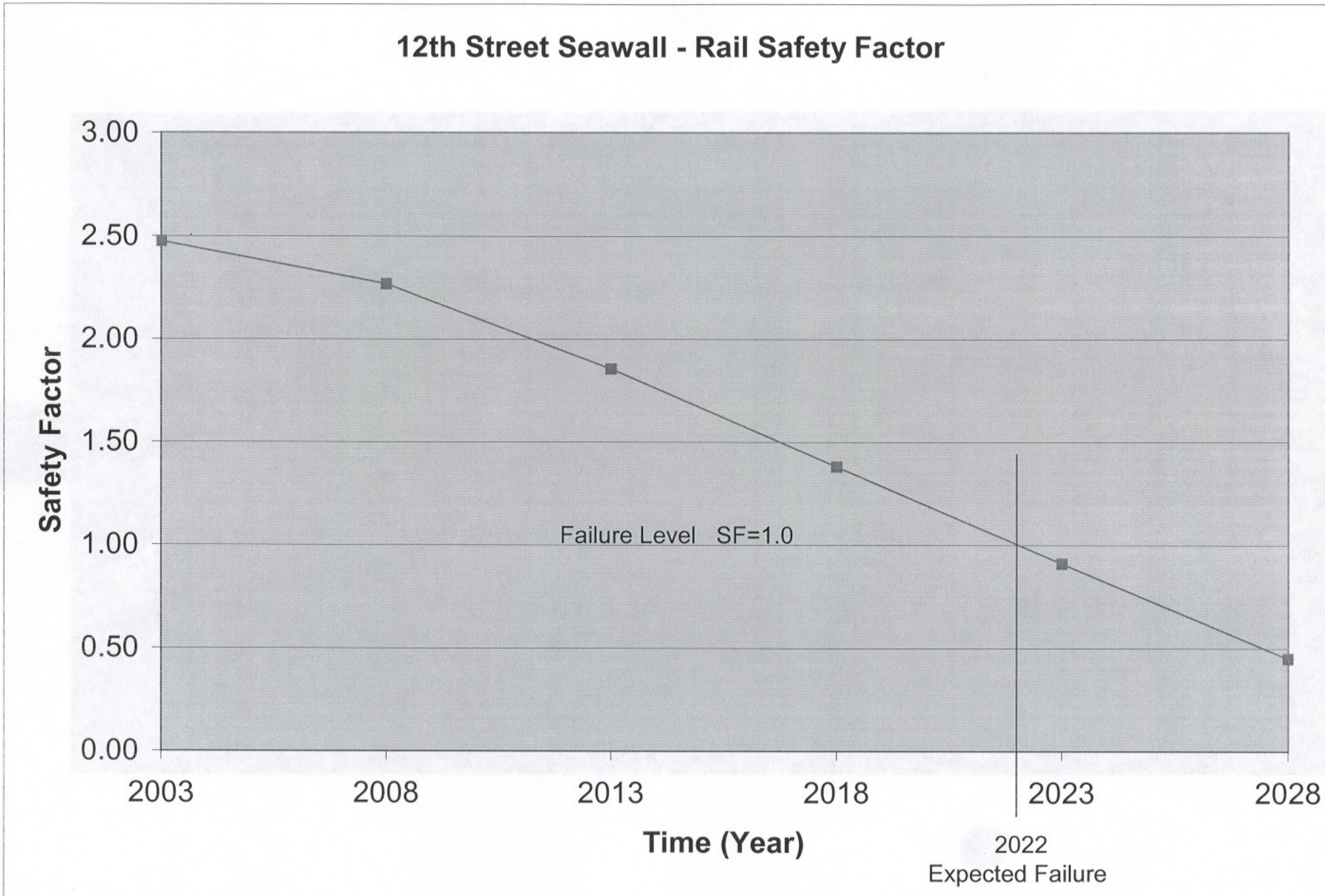
Year	1978	2003	2013	2023	2033	2043	2053
Sx (in^3)	22.5	18.9	**17.4**	15.8	**13.9**	**12.0**	10.1
Mn (K-ft)	131	110	101	92	81	70	59
SF	2.95	2.48	2.27	2.07	1.82	1.57	1.32

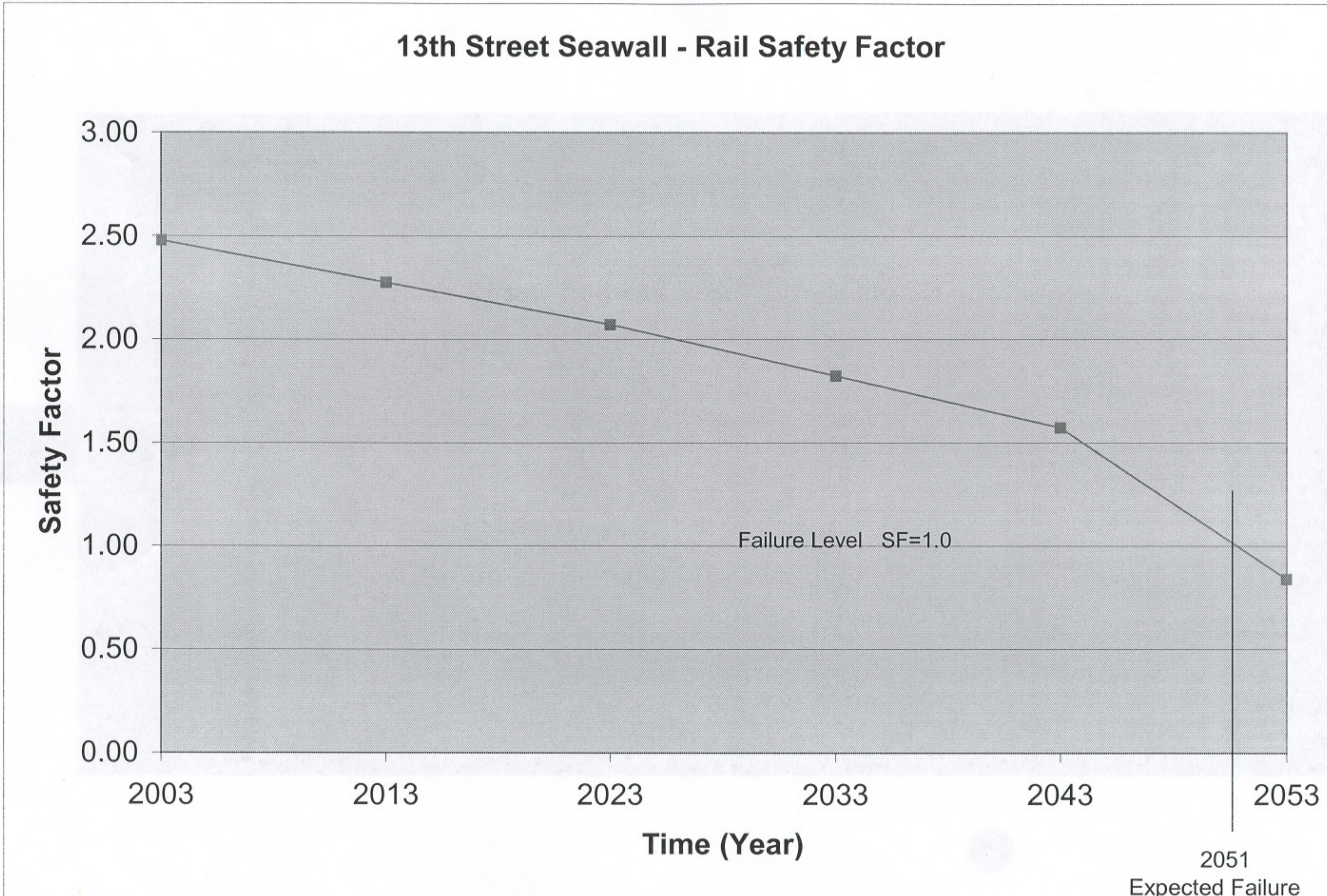
V demand (K)	16.3
fyv (ksi)	42

Year	1978	2003	2013	2023	2033	2043	2053
t (in)	0.66	0.47	0.39	0.30	0.22	0.13	0.05
d (in)	7.13	6.94	6.85	6.77	6.69	6.60	6.52
Vn (K)	196	137	111	86	61	37	14
SF	12.05	8.38	6.80	5.26	3.75	2.27	0.84

Min SF	2.95	2.48	2.27	2.07	1.82	1.57	0.84


Station 1514 Seawall - Rail Safety Factor
Safety Factor
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Failure Level SF=1.0
2003
2013
2023
2033
2043
2053
2049
Expected Failure
Time (Year)


12th Street Seawall - Rail Safety Factor
Safety Factor
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Failure Level SF=1.0
2003
2008
2013
2018
2023
2028
Time (Year)
2022
Expected Failure


13th Street Seawall - Rail Safety Factor
Safety Factor
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Failure Level SF=1.0
2003
2013
2023
2033
2043
2053
Time (Year)
2051
Expected Failure

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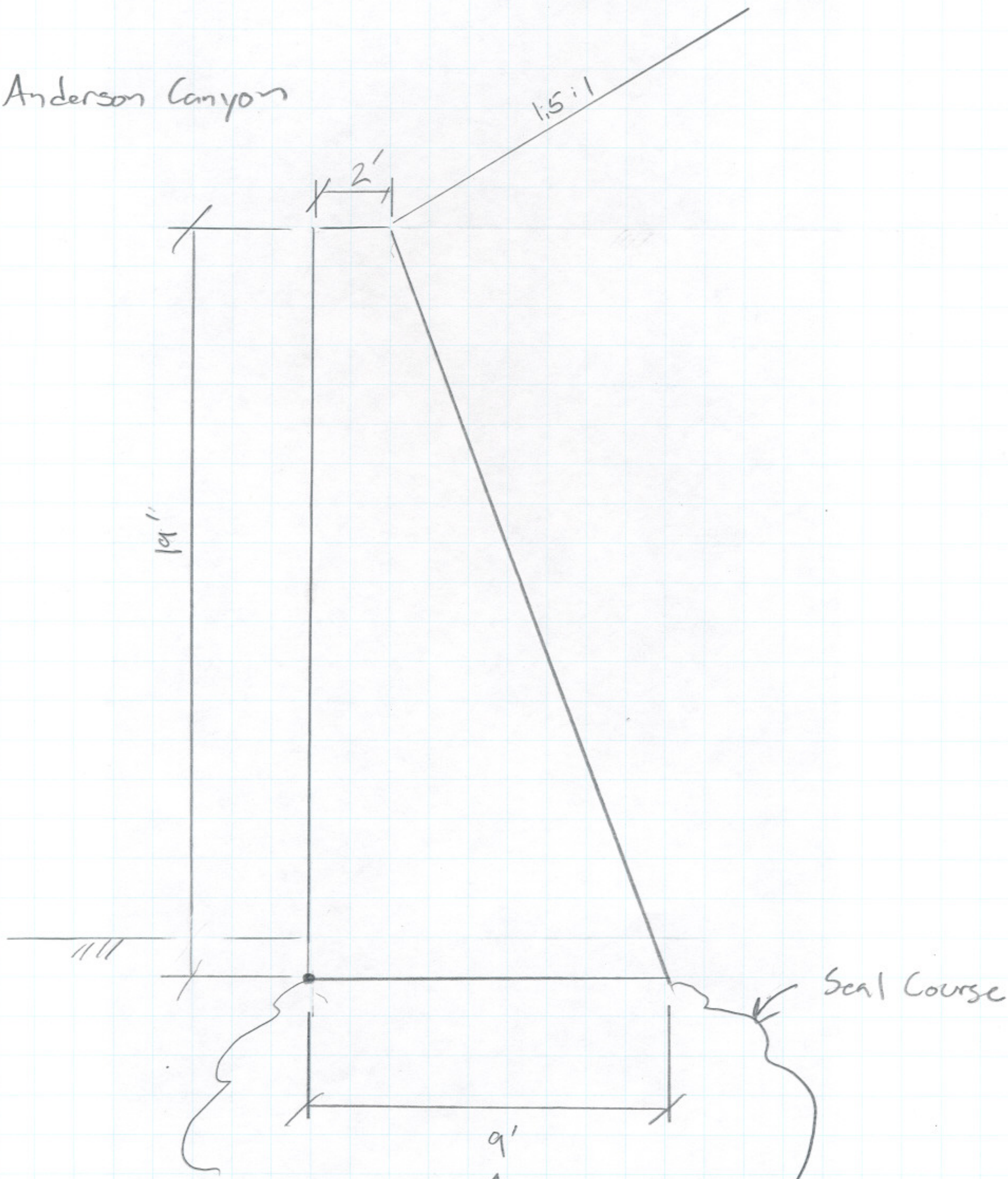
Project: Del Mar Sea Walls	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: B-18
Designed: JH		Proj. #: 518-109
		Date: 4/03
Checked:	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Revised:

Anderson Canyon



assumed based on crack in
drainage outlet

Project: Del Mar Sea Walls	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: B-19
		Proj. #: 518-109
Designed: JH		Date: 4/03
Checked:	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Revised:

Overturning Moment:

Assume EFP = 65pcf for 2:1 slope

$$M_0 = 65pcf \frac{(19')^3}{6} = 74.3 \text{ Kft}$$

Resisting Moment: (assume soil weight 110pcf)

$$M_{resist} = \frac{(2.0 \times 19) \times 150pcf \times 2}{2} + \frac{(7.0 \times 19)(2 + 7/3)(150pcf)}{2}$$

$$+ (110pcf)\frac{(7')(19')}{2} \cdot \left(2 + 2\left(\frac{7}{3}\right)\right)$$

$$= 5.7 + 43.2 + 48.8 = 97.7 \text{ Kft}$$

$$FS = \frac{97.7}{74.3} = \underline{1.31}$$

Bearing Pressure:

$M = 97.7 - 74.3 = 23.4 \text{ kft}$

$Q = (2.0 \times 19 \times 150) + (7.0 \times 19 \times \frac{150}{2}) + (7 \times 19 \times \frac{110}{2})$

$= 23.0 \text{K}$

$e' = \frac{23.4\text{K}}{23.0\text{K}} = 1.02'$

$e = \frac{9'}{2} - 1.02' = 3.48' > \frac{L}{6}$

$q_{max} = \frac{2Q}{2(\frac{L}{2} - e)} = \frac{2(23.0\text{K})}{2(\frac{9}{2} - 3.48)} = 22.5 \text{ ksf}$

Sliding:

Lateral Force: $F = 65\text{pcf}\,\frac{(19')^2}{2} = 11.7\text{ k}$

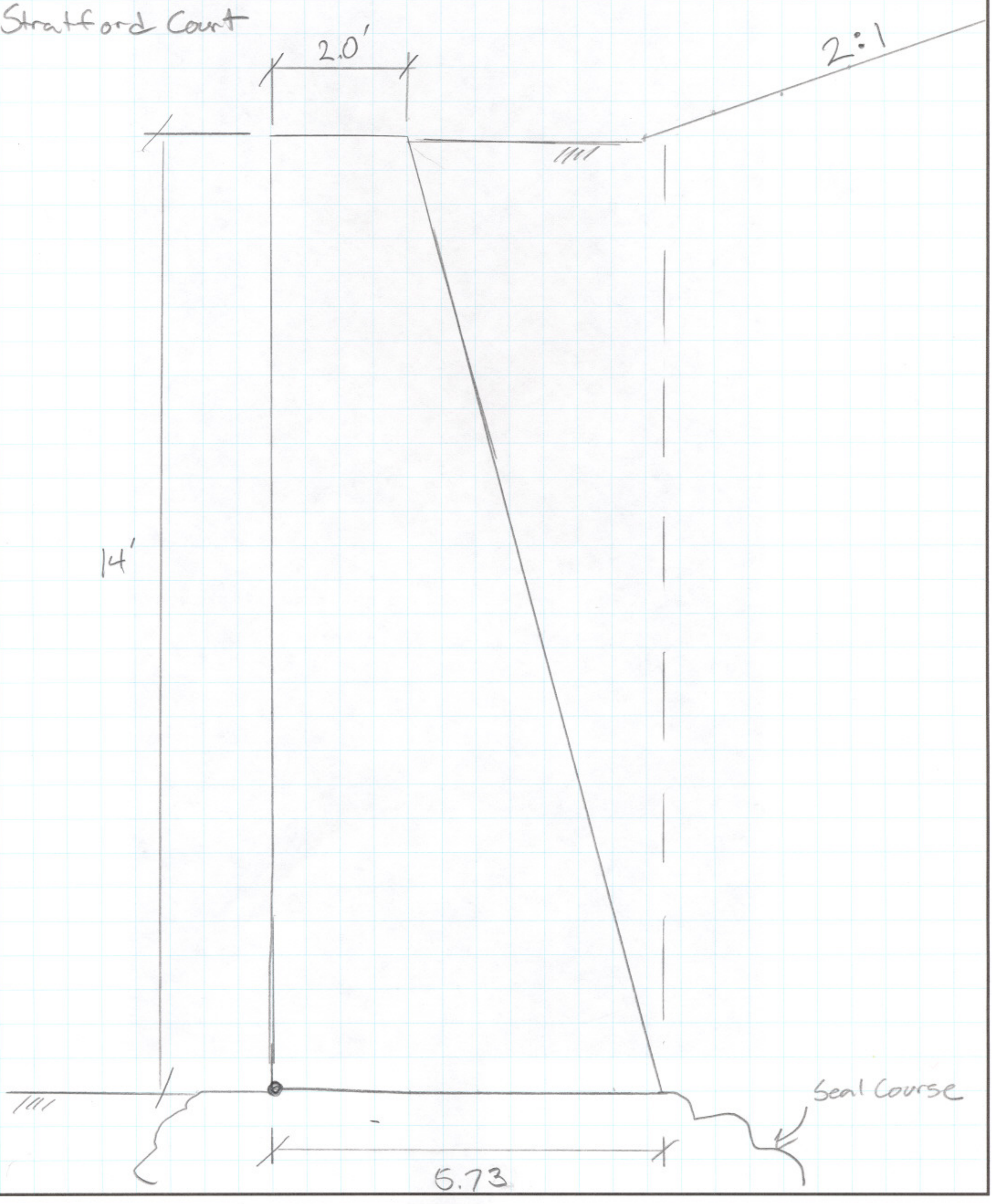
Lateral Resistance: Assume $\mu = .55$

$F_R = .55(23.0\text{k}) = 12.7\text{k}$

$FS = \frac{12.7\text{k}}{11.7\text{k}} = 1.10$

Project: Del Mar	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: B-22
Sea Walls		Proj. #: 618-109
Designed: JH		Date: 4/03
Checked:	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Revised:

Stratford Court



Project: Del Mar Sea Walls	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: 3-23
		Proj. #: 518-109
Designed: JH		Date: 4/03
Checked:	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Revised:

Overturning Moment:

Assume EFP = 55 pcf (Since slope is flat to 2:1 slope)

$$M_o = \frac{55pcf\,(14')^3}{6} = 25.2\ k\ ft$$

Resisting Moment: (assume soil weight 110pcf)

$$M_{resist} = \frac{(2.0' \times 14')^{\times 2} \times 150pcf}{2} + \frac{(3.73)(14)}{2}\left(2+\frac{3.73}{3}\right)(150pcf)$$

$$+ (110pcf)\frac{(3.73')(14')}{2}\left(2+\frac{3.73(2)}{3}\right)$$

$$= 29.8\ kft \qquad = 4.2 + 12.7 + 12.9$$

$$F.S. = \frac{29.8}{25.2} = \underline{1.18}$$

Project: Del Mar Sea Walls	SIMON WONG ENGINEERING STRUCTURAL & BRIDGE ENGINEERS	Page: B-24
Designed: JH	9968 Hibert Street, Suite 202 San Diego, CA 92131 (858) 566-3113 FAX (858) 566-6844	Proj. #: 518-109
Checked:		Date: 4/03
		Revised:

Bearing Presure:

$$M = 29.8 - 25.2 = 4.6 \text{K ft}$$

$$Q = (2.0 \times 14 \times 150) + \left(3.73 \times 14 \times \frac{150}{2}\right) + \frac{(3.73 \times 14 \times 110)}{2}$$

$$= 11.0 \text{ K}$$

$$e' = \frac{M}{Q} = \frac{4.6\text{K}}{11.0\text{K}} = .42'$$

$$e = \frac{5.73'}{2} - .42' = 2.44' > \frac{L}{6}$$

$$q_{max} = \frac{2Q}{3\left(\frac{L}{2} - e\right)} = \frac{2(11.0\text{k})}{3\left(\frac{5.73}{2} - 2.44\right)} = 17.3 \text{ ksf}$$

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San Diego, CA 92131
(858) 566-3113
FAX (858) 566-6844

Sliding:

Lateral Force: $F = 55pcf \frac{(14')^2}{2} = 5.39K$

Lateral Resistance: Assume $\mu = .55$

$F_R = .55(11.0K) = 6.05K$

$SF = \frac{6.05K}{5.39K} = \underline{1.12}$