BALLARD BRIDGE SIDEWALK WIDENING ALTERNATIVE STUDY

Seattle Department of Transportation

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Appendix A
Structural Analysis

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Technical Memorandum

Date: June 21, 2013

Subject: Ballard Bridge Sidewalk Widening – Structural Analysis [Appendix A]

From: Greg Banks, BergerABAM

To: Seattle Department of Transportation, SDOT

Route to: James Corney

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1. Introduction

BergerABAM was hired by the Seattle Department of Transportation (SDOT) to do a conceptual study to assess the feasibility of widening the existing bridge pedestrian sidewalks on the Ballard Bridge approach structures. The feasibility study focused on alternatives increasing the width of the sidewalks to 6 feet and/or 10 feet. This document serves to provide the results of the structural analyses conducted by BergerABAM as part of the study. The supporting structural calculations are included in Appendix H. Sketches of the sidewalk widening concepts analyzed are included in Appendix B.

2. Existing Conditions

Bridge Type/Layout

The Ballard Bridge is composed of eight segments; six elevated bridge structure segments and a fill approach segment at each the north and south ends of the elevated bridge structure. Segment 4 is the main bascule span and is located in Salmon Bay spanning the Ship Canal. Segments 1 to 3 are to the north of the bascule span, and Segments 5 to 8 are south of the bascule span. General details of each segment are provided in Table 1. It is important to note that each of these segments have a distinct structural system which required a unique design concept for the structural support of the sidewalk widening.

Table 1 – Segment Details

Location	Segment No.	Type
North Approach	1	Fill Approach Structure with CIP Cantilever Retaining Walls
	2	CIP T-Beam Girders Structure & CIP Box Girder Ramps
	3	Built-Up Riveted Steel Girders with Transverse Floor Beams
Bascule Span – Not Evaluated		
South Approach	5	Built-Up Riveted Steel Girders with Transverse Floor Beams
	6	Skewed Rolled Steel Beams
	7	CIP T-Beam Girder Structure
	8	Fill Approach Structure with Counterfort Gravity Retaining Wall

Pedestrian Sidewalks

Pedestrian sidewalks currently exist on each the west and east sides of the structure, see details in Figure 1. The existing sidewalk conditions are substandard in width and safety as noted below:

> *Sidewalk Width*
>
> The sidewalks are typically 4 feet wide between the 18-inch high traffic curb and the metal railing. The sidewalk width reduces to approximately 3.5 feet at the light post pedestal locations.
>
> *Safety*
>
> There is an 18-inch curb separating vehicular traffic from pedestrian traffic, which does not provide adequate pedestrian safety under vehicular crash conditions or for bicyclist containment. Also, the 42-inch high exterior railing similarly does not meet the current height requirements for bicyclist safety and likely will not satisfy AASHTO requirements for vehicular impact.

These deficiencies illustrate the opportunity to improve the safety conditions of the existing Ballard Bridge while providing a better pedestrian/bicyclist shared-use facility.

The existing sidewalk utilizes a walkway support system which is similar on all of the elevated bridge structure segments except Segment 2. The system consists of a concrete edge beam which either spans between bridge transverse cantilever elements (Segments 3, 5, and 7) or is continuously supported by a bridge girder (Segment 6). This concrete edge beam is cast together with the concrete elements of the pedestrian railing, and is otherwise disconnected from the main bridge deck. A 3.5-inch thick concrete slab element is supported at the edge beam and the bridge deck and spans the space between. Segment 2 uses a simpler system where the sidewalk is simply the top slab of the bridge deck.

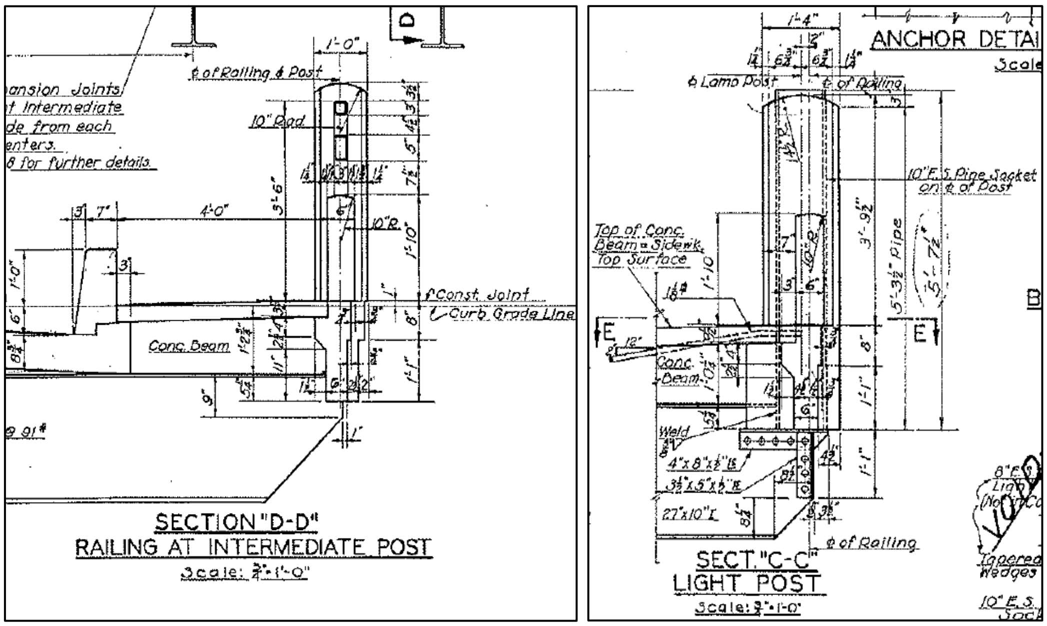


Figure 1 – Existing Sidewalk Conditions

The existing sidewalks at the fill approaches, Segments 1 and 8, are unique. Segment 1 incorporates the sidewalk surface with the adjacent curb and vehicular slab and connects the pedestrian barrier to the top of the retaining wall. Segment 8 connects the pedestrian barrier to the top of the retaining wall, but uses a more typical slab on grade for the sidewalk separate from the curb.

Demolition Details

The sidewalk widening alternatives will require the existing curb and pedestrian railing to be removed without significant damage to the supporting structure; removing the curb without damaging the bridge deck and removing the pedestrian railing without damaging the edge beam.

For the fill approach segments, Segments 1 and 8, removal of the top of the existing retaining wall will be required along with removal of a portion of the existing adjacent roadway slab. The existing steel within the roadway slab extents being removed will not need to be preserved.

Demolition extents are shown in the sketches included in Appendix B.

3. Design Criteria

The below criteria was used for the structural calculations associated with the sidewalk widening feasibility study. The criteria are structural in nature and are not intended to be the overall general criteria used for the project.

Project Specific Assumptions:

- Combined pedestrian/bicycle railing shall be a minimum of 54 inches from the surface of the deck to the top of horizontal rail.
- Metal railing shall be fabricated from steel.
- The curb separating the vehicular lanes and the sidewalk shall be removed and replaced with a WSDOT 34-inch single slope barrier, conforming to WSDOT *Bridge Design Manual* (BDM), §10.2.1
- The vehicular barrier per WSDOT BDM §10.2.1 shall have railing meeting the bicycle minimum height of 54 inches, designed to sit on top the 34-inch traffic barrier, per WSDOT BDM §10.5.2(B)
- The design clear width of the new sidewalk shall be taken as 10 feet or 6 feet.

The following manuals were used for analysis and design of the proposed alternatives:

- *AASHTO LRFD Bridge Design Specifications*, 6E (AASHTO) ["LRFD"]
- *WSDOT Bridge Design Manual*, M 23-50.12, 2012E (WSDOT) ["BDM"]

The structural design criteria from the AASHTO LRFD and the WSDOT BDM can be grouped into three sub-categories:

1. Structural criteria for material – e.g. concrete, steel, aluminum or wood
2. Structural criteria based on component – e.g. railing and barrier requirements, decking, expansion joints, attachments and cantilever, removal of existing structures, etc.
3. Structural criteria for evaluating the structure (existing or new) – e.g. service, strength and seismic analysis and capacity

Material Design Criteria
The AASHTO LRFD provides minimum requirements for the proportioning and detailing of various materials by chapter: concrete (LRFD Ch. 5), steel (LRFD Ch. 6), aluminum (LRFD Ch. 7) and wood (LRFD Ch. 8). WSDOT BDM similarly addresses these same materials, providing further information and guidance: concrete (BDM Ch. 5), steel (BDM Ch. 6), aluminum (BDM Ch. 7) and wood (BDM Ch. 8).

Component Design Criteria
AASHTO provides both geometric and structural design criteria for pedestrian railings (LRFD §13.8), bicycle railings (LRFD §13.9), traffic barriers (LRFD §13.7), combined traffic-ped-bicycle barrier-rails (LRFD §13.10). WSDOT BDM, §§10.2 & 10.5, address bridge traffic barriers and railing, respectively. SDOT has more stringent spacing criteria than both AASHTO and WSDOT in terms of clear spacing between rails. SDOT uses a 4-inch maximum spacing.

Structural Evaluation Criteria
The existing structure (including the Phase II seismic retrofits) was evaluated for increased dead and live loads, as well as increased wind resistance. The design loads, load factors and appropriate combinations are discussed below. Existing structure demands was checked against the Approach Structure LFR Load Rating Report. The existing structure was assumed not strengthened to accommodate the widening.

Recommended design loads and load factors are provided in AASHTO LRFD Chapter 3 and further developed per WSDOT BDM Chapter 3. For the design of traffic barriers and railing, loads applied to such elements are provided in LRFD Chapter 13.

Applicable service and strength load combinations are determined per AASHTO LRFD Chapter 3 and modified in some cases by WSDOT BDM Chapter 3.

The mass increase of the sidewalk widening concepts was compared to the provisions of WSDOT BDM §4.3 which discuss the condition wherein a seismic analysis would be invoked.

4. Widening Concepts

Structural concepts were developed that would allow the pedestrian sidewalk to be widened to either 6 feet or 10 feet for each segment of the bridge; except for Segment 4, the bascule span. As noted, it was assumed that the existing curb would be replaced with a concrete TL-4 crash tested WSDOT single slope barrier to separate the vehicular traffic from the pedestrian and bicyclist traffic. In addition, it was assumed that the existing pedestrian railing would be replaced with SDOT's pedestrian railing working plan. Further assumptions included concrete densities of 155 pcf for the reinforced concrete weight, and concrete densities of 150 pcf for calculation of the modulus of elasticity.

Calculations have been performed assuming pedestrian loads of 75psf as designated in the AASHTO LRFD Bridge Design Specifications. A unique structural support system was developed for the various structure types along the length of the bridge. Schematic drawings have been provided to show the type of construction required for each segment of the bridge, see Appendix B. Detailed structural calculations can be found in Appendix H. It should be noted that the calculations conducted as part of this study assumed a widened sidewalk on one side of the structure.

Segment 1: Cantilevered Gravity Retaining Wall

The Segment 1 widening concept consists of a slab that cantilevers out beyond the face of the existing retaining wall. The cantilevered slab has been preliminarily estimated to be 6 inches thick in order to provide strength and stability for the pedestrian loads. The cantilevered slab will be cast integrally with a thicker anchor slab that will be supported vertically on the existing retaining wall stem and the approach fill material. No dowels or reinforcing will be used to positively attach the anchor slab to the wall. The retaining wall's vertical stem design is very strong for vertical loads, so this connection will be used for vertical load; however, without a steel connection, the anchor slab will not impart bending moments or significant lateral loads to the top of the existing wall.

As the widening increases, the size of the anchor slab must also increase. The depth of the anchor slab is estimated to be 1 foot, 3 inches and 2 feet, 3 inches thick for the 6- and 10-foot sidewalks, respectively. The anchor slab is estimated to extend to the existing curb line for the 6-foot sidewalk and to 1 foot, 6 inches beyond the existing curb line for the 10-foot sidewalk. Extending the anchor slab beyond the existing curb line for the 10-foot sidewalk was not preferred; however, the existing concrete vehicular slab in Segment 1 contains longitudinal joints, so the joint nearest to the existing concrete curb was used. In both cases (i.e., the 6- and 10-foot widening), it is recommended that expandable material be located between the inside face of the retaining wall and the side of the anchor slab. This material is intended to prevent vehicle impacts on the traffic barrier to impart load directly into the top of the retaining wall.

Segment 2: Conventionally Reinforced Concrete

The Segment 2 widening concept will attach directly to the existing cast-in-place concrete box girder structures that were added as a prior vehicular widening in the 1950s. Segment 2 is the only portion of the structure with the existing sidewalk integral to the bridge deck, and is 6.5 inches thick in-lieu of 3.5 inches thick. The thicker slab allows for a doweled bar to be installed in the edge of the existing deck. The slab extension will not be cantilevered, but will sit on a longitudinal steel edge beam that is in-turn supported by a new steel transverse beam. The proposed transverse support beam will be through-bolted to the existing concrete cantilever ribs/beams that extend off of the existing concrete box girder structures. This system is similar for both the 6-foot and 10-foot widening conditions.

Segment 3 and 5: Non-Redundant Structural Steel Girder Bridge

Segments 3 and 5 are similar in terms of structure type. The widening concept for Segments 3 and 5 widening differs for the 6- and 10-foot conditions. For the 6-foot condition, a new sidewalk slab would be cast starting at the existing curb line and cantilever over the top of the existing concrete edge beam. The construction contract documents should draw attention to the care that will be required for removing the existing pedestrian rail in order to preserve the integrity of the edge beam. It has been assumed that the new slab will be connected to the existing edge beam through doweled anchors.

For the 10-foot condition, a new concrete sidewalk slab would be cast similar to the 6 foot condition. A new steel longitudinal edge beam would be required, along with transverse floor beam extensions at each floor beam location. The transverse floor beam extensions will support the new steel sidewalk longitudinal edge beam. The existing pedestrian railing and edge beam would be completely removed, along with a portion of the deck to allow for the splicing of the new transverse floor beam extensions. The construction contract documents should note that it will be necessary to preserve the existing deck steel where the new steel transverse floor beams are to be installed.

Segment 6: Structural Steel Wide Flange Bridge

Segment 6 is a single span of structure that spans the BNSF right-of-way. The Segment 6 widening concept differs between the 6-foot and 10-foot conditions. The concept for the 6-foot condition is similar to that for Segments 3 and 5 (i.e., a new sidewalk slab would be cast, starting at the existing curb line and cantilevering over the top of the existing concrete edge beam). Again, the construction contract documents should draw attention to the care that will be required for removing the existing pedestrian rail in order to preserve the integrity of the edge beam.

The Segment 6 widening concept for the 10-foot condition is unique to the entire project. An entirely new girder is shown to support the widening, due to the capacity limitations of the existing exterior beam, the complicated geometry of the skewed interior beams, and the relatively short span. The addition of this proposed girder will require that a seat extension be provided at each supporting pier wall (i.e., Piers 22 and 23). Vertical clearance over the BNSF right-of-way will also need to be maintained. The widening itself will consist of a new sidewalk slab that would be cast starting at the existing curb line and cantilevering over the top of the proposed girder.

Segment 7: Conventionally Reinforced Concrete T-beams
The Segment 7 widening concept for the 6-foot condition is similar to that for Segments 3 and 5 (i.e., a new sidewalk slab would be cast, starting at the existing curb line and cantilevering over the top of the existing concrete edge beam). Again, the construction contract documents should draw attention to the care that will be required for removing the existing pedestrian rail in order to preserve the integrity of the edge beam.

For the 10-foot condition, a new sidewalk slab would be cast, starting at the existing curb line. The slab will not be cantilevered but will sit on a new longitudinal steel edge beam that is in-turn supported by a series of new transverse steel truss systems. The proposed truss systems will be through-bolted to the existing concrete cantilever ribs/beams that extend off of the exterior face of the existing concrete T-beam girders to form a tension tie, and will be connected to the existing concrete tee beam girders with resin bonded anchors to form a compression strut.

Segment 8: Counterfort Gravity Retaining Wall
The Segment 8 widening concept is similar to Segment 1; a thin concrete slab will cantilever out beyond the face of the retaining wall, and a thick anchor slab will bear on the retained fill material behind the retaining wall. The existing retaining wall system in Segment 8 differs from Segment 1 in that Segment 8 uses a counterfort retaining wall system, whereby individual footings support large vertical triangular-shaped concrete structural members oriented perpendicular to a concrete fascia that retains the soil. In this system, the fascia collects the load from the soil and carries it horizontally to the large, concrete, triangular-shaped counterforts. The vertical reinforcement in the concrete fascia is minimal because the facing is not intended to carry load in that direction.

The counterfort wall does not lend itself well to carrying vertical loads or bending moments about the top of the facing wall. Therefore, the conceptual design shows the anchor slab fully isolated from the concrete wall with expandable material. The anchor slab was set to extend to the existing curb line for the 6-foot condition, which resulted in an anchor slab depth of 3 feet. For the 10-foot condition, the anchor slab was set to extend 10 feet out from the curb line as an attempt to locate the edge of the anchor slab at the adjacent driving lane. The associated depth of the anchor slab in this condition was 1 foot, 10 inches.

5. Assessment

For the Ballard Bridge Sidewalk Widening concept study, the objective was to ensure that a widening concept was feasible. It was assumed the concepts would be further optimized in final design efforts. The structural feasibility of the widening concepts were evaluated based on the impacts to the LFR load rating operating rate factors and impacts to the overall mass of the structure. Each is described further below:

Load Rating

As part of the seismic retrofit project, a bridge load rating analysis was conducted. Bridge load rating is a procedure to determine the adequacy of the structural bridge components to carry live loads. Two load rating methods exist: the Load and Resistance Factor Rating (LRFR), and the Load Factor Rating (LFR). Per the AASHTO Manual of Bridge Evaluation (MBE), new bridges designed after October 1st, 2010 shall be rated based on LRFR methods, and bridges designed prior to October 1st, 2010, existing bridges, or partially rehabilitated or reconstructed bridges where part of the existing structure was designed by the allowable stress method, can be rated by either the LRFR or LFR methods. The Ballard Bridge approach structures fall into the latter category. The LFR load ratings for the modified portions of the Ballard Bridge were investigated as part of this feasibility study.

LFR load rating equation 13.1.2-1 from the WSDOT Bridge Design Manual is presented below.

$$RF = \frac{(\phi C - \gamma_{DL} D \pm S)}{\gamma_{LL} LL(1+IM)}$$

Note that the equation shown is used for the determination of load rating factors for legal trucks, which are the trucks used for assessing load restricting a structure. A different equation is available in the bridge design manual for overload vehicles. No equation has been provided in the bridge design manual for the determination of the effects of pedestrian loading on load ratings. Therefore pedestrian loading has not been applied to the calculation of load rating modifiers provided in this document.

A full load rating analysis/report to provide rating factors including the widening concepts was not conducted as part of this study. Instead, modification factors that act as multipliers to the rating factors presented in the load rating report were developed. The basis of the development of this modification factor is as follows:

> The only variable in the rating factor equation at a given location is the live load under investigation. The structural element capacity, dead loads, and prestress contribution are constant values. A new rating factor was calculated by adding in the dead load contribution from the sidewalk widening to the existing structure dead load demands. The modification factor was taken as the ratio of this revised rating factor to the component rating factor provided in the Load Rating report, see equation below:

$$RF_{modified} = (Modification\ Factor)\ x\ RF$$

The calculations for the determination of these modification factors have not been provided in the Appendix H of this report as the data provided is not intended to give load rating output for the Ballard Bridge structure. The modification values shown in the Tables 2 and 3 below are intended to be used in conjunction with the Load Rating report.

Table 2 – Load Rating Modification Factors – 6-Foot Sidewalk

Schematic Modifications to Load Rating Factors - 6ft Wide Sidewalk				
Segment	**Member**	**Load Condition**	**Modification Factor**	
			Inventory	Operating
2	Widening Box	Moment	0.98	0.97
		Shear	0.94	0.94
3 & 5	91# Floor Beam	Moment	0.88	0.88
		Shear	0.99	0.99
	98# Floor Beam	Moment	0.91	0.91
		Shear	0.99	0.99
	Girder	Moment	0.96	0.96
		Shear	0.96	0.96
6	Girder 3 or 4	Moment	0.95	0.95
		Shear	0.98	0.98
7	Short Box between Piers 23 & 24	Moment	0.99	0.99
		Shear	1.00	1.00
	Bridge Past Pier 24	Moment	0.96	0.95
		Shear	0.95	0.95

Table 3 – Load Rating Modification Factors – 10-Foot Sidewalk

Schematic Modifications to Load Rating Factors - 10ft Wide Sidewalk				
Segment	**Member**	**Load Condition**	**Modification Factor**	
			Inventory	Operating
2	Widening Box	Moment	0.96	0.96
		Shear	0.90	0.90
3 & 5	91# Floor Beam	Moment	0.75	0.75
		Shear	0.98	0.98
	98# Floor Beam	Moment	0.80	0.80
		Shear	0.98	0.98
	Girder	Moment	0.91	0.91
		Shear	0.89	0.91
6	Girder 3 or 4	Moment	0.91	0.91
		Shear	0.97	0.97
7	Short Box between Piers 23 & 24	Moment	0.99	0.99
		Shear	0.99	1.00
	Bridge Past Pier 24	Moment	0.90	0.90
		Shear	0.88	0.88

Segments 3 and 5 show the highest sensitivity to the sidewalk widening. This is due to the existing structural system of cantilevered floor beams supporting the bridge deck. Additional load from the widening occurs beyond the current extents of the cantilevered floor beams. The load, although relatively small, applied to the limits of the cantilever creates large bending moments in the floor beams. Note, the 6 foot path alternative only connects to the 91# floor beam locations; therefore a modification factor of 1.00 is shown for the 98# beams. Similarly, in Segment 6, Girder 4 shows modification factors of 1.00 as an additional beam line is recommended for that condition.

Tables 2 and 3 were developed showing the impacts of a single sidewalk widening (ie. widening on only one side of the structure). However, the modification factors will not change when considering two sidewalks, or a sidewalk widening on each side of the bridge, with the exception of Segment 7. Segments 2, 3, 5, and 6 all have rating modification factors relevant to adjacent supporting members. Sidewalk widening on the other side of the bridge would affect similar members on the opposite side of the structure. Segment 7 was analyzed as a whole bridge; therefore the modification would be doubled for that structure.

The load rating analysis did not reveal any operating live load deficiencies that would result in needing to restrict live loading on the bridge. Based on the reported modification factors and the magnitude of the operating rating factors, it was determined that the sidewalk widening concepts were feasible without the need to restrict live loading on the bridge. However, it should be noted that the addition of the sidewalk widening does cut into the live load carrying capacity of the bridge structure. To minimize these impacts, alternate materials/details could be investigated in final design (see Section 7 of this memorandum), or modifications to the widening concepts could be evaluated. One such potential modification noted in the development of the widening concepts is as follows:

Segment 3 and 5 Floor Beams

- Adding bracing members at each connected floor beam.
    - Benefit – This modification has potential to change the structural system of the existing floor beam from a cantilevered beam to a more beneficial simply supported beam.
    - Concern – Changing the structural system may have consequences not easily determined without analysis. E.g.:
        - Increasing axial loads to existing bracing.
        - Ensuring dead load conditions are modified by new framing ("rerouting" existing dead load forces is challenging).
        - Connection issues to existing beams.

- Adding new floor beams between all existing floor beams.
    - Benefit – Adding new floor beams will effectively halve the loads to existing floor beams.
    - Concern – Adding floor beams will significantly increase the amount of material added to the structure. Significantly increasing added mass and cost.

Structural Mass

The widening concepts will add mass to the existing bridge structure which will impact the seismic demands on the structure. The widening concepts of the bridge have not yet been fully reviewed for seismic performance. For this study, the increase in mass was recorded and compared to the mass of the existing structure. Per Section 4.3 of the WSDOT *Bridge Design Manual*, a seismic analysis of a bridge widening without new substructure may be waived with the owner's approval if the added mass from the widening is 10 percent or less of the original structure weight. The results of the preliminary analysis indicate that the added mass due to the bridge widening would not exceed the 10 percent threshold for Alternative 2 with all combinations of widths (see Table 4 for details). It should be noted that Table compares the superstructure mass increases only. For the option with 10-foot width on both sides, the mass increase exceeds 10 percent of the superstructure mass, but it remains below 10 percent of the total structure mass. Based on precedent experience, the retrofitted capacity/demand ratios are high enough to accommodate the additional mass; however, further analysis should be performed if the City chooses to construct the 10-foot additional width on both sides. Details of the structural calculations conducted as part of this study can be found in Appendix H.

Table 4 – Structure Mass Increase Percentages for Sidewalk Widening Alternatives

Bridge Segment	Alternative 2 - Widening Options				
	6 ft	10 ft	6 ft & 6 ft	6 ft & 10 ft	10 ft & 10 ft
1*	-	-	-	-	-
2	0.7%	1.0%	1.4%	1.7%	2.1%
3	1.4%	2.7%	2.7%	4.1%	5.5%
4**	-	-	-	-	-
5	1.4%	2.7%	2.7%	4.1%	5.5%
6	0.5%	0.9%	1.0%	1.4%	1.8%
7	1.3%	2.9%	2.6%	4.2%	5.7%
8*	-	-	-	-	-

*Segments 1 and 8 are fill approaches and do not include the same seismic restrictions as the elevated bridge structure approach segments (i.e. Segments 2 through 7). The widening system for Segments 1 and 8 have been designed as self-equilibrating moment-resisting anchor slabs in order to reduce the interaction between the new sidewalk widening and the existing walls.
**Segment 4 bascule span was not evaluated as part of this contract.

6. Alternate Materials/Details

Typical materials and details were used in developing the widening concepts for the Ballard Bridge. Typical design materials provide a sound baseline for preliminary engineering design, and are usually less expensive than more specialized alternatives. For the Ballard Bridge Sidewalk Widening concept study, the objective was to ensure that a widening concept was feasible and then leave the optimization to final design efforts.

There could be significant advantages to using alternate details and materials. Based on the load rating report, the Ballard Bridge has little reserve live load capacity. Utilizing alternative materials/details could help minimize the impacts to the reserve live load capacity. Alternate materials and details would also minimize the added mass, which would minimize the increase in lateral seismic inertial loads. Some beneficial alternate materials/details are (1) lightweight concrete, (2) steel traffic barrier, and (3) aluminum railings. Each are described further below:

Lightweight concrete
The most influential component of mass increase in the sidewalk widening design is the concrete. For example in the Segments 3 and 5, 10-foot widening option, the concrete

represents about 85 percent of the total mass increase; using normal weight concrete with a density of 155 pcf. Lightweight aggregate concrete would be analyzed with a density of 125 pcf, per the WSDOT Bridge Design Manual. If an additional 5 pcf is added in for reinforcing, the use of lightweight concrete would reduce the mass increase associated with concrete by about 16 percent. Relating back to Segments 3 and 5 this would represent about a 13.5 percent mass savings overall.

Lightweight concrete is penalized, from a design standpoint, with lower shear capacity versus normal weight concrete; however, this penalty can be accommodated without significant changes to the provided proposed alternatives.

Steel Traffic Barrier

Concrete is the largest mass contributor to the sidewalk widening, and a major component of the additional concrete is present in the new single slope concrete barrier. For example, the area of concrete in a typical 10-foot sidewalk widening is approximately 6.2 feet^2, and of that area, about 2.4 feet^2 is concrete traffic barrier weighing 372 plf (based on normal weight concrete). A steel traffic barrier would weigh significantly less, potentially resulting in a mass savings greater than an overall change to lightweight concrete.

A steel traffic barrier option would not need to be limited to typical bent plate guardrail designs (although they may be the most cost effective). AASHTO provides guidance for the design of steel traffic barriers. A design could be created which looks more aesthetic than a typical thrie beam.

There are drawbacks to the use of a steel traffic barrier. The traffic barrier would preferably not be designed to act as a bicyclist fence. Steel barriers are relatively flexible compared to concrete barriers, even a small impact could cause pedestrian alarm if the barrier and fence moved together due to a vehicle impact. Additionally, should an impact cause deformation in the steel railing, it could impact the visual appeal of the sidewalk. Constructed separately, the pedestrian fence would not necessarily be impacted by a vehicle impact on the traffic barrier.

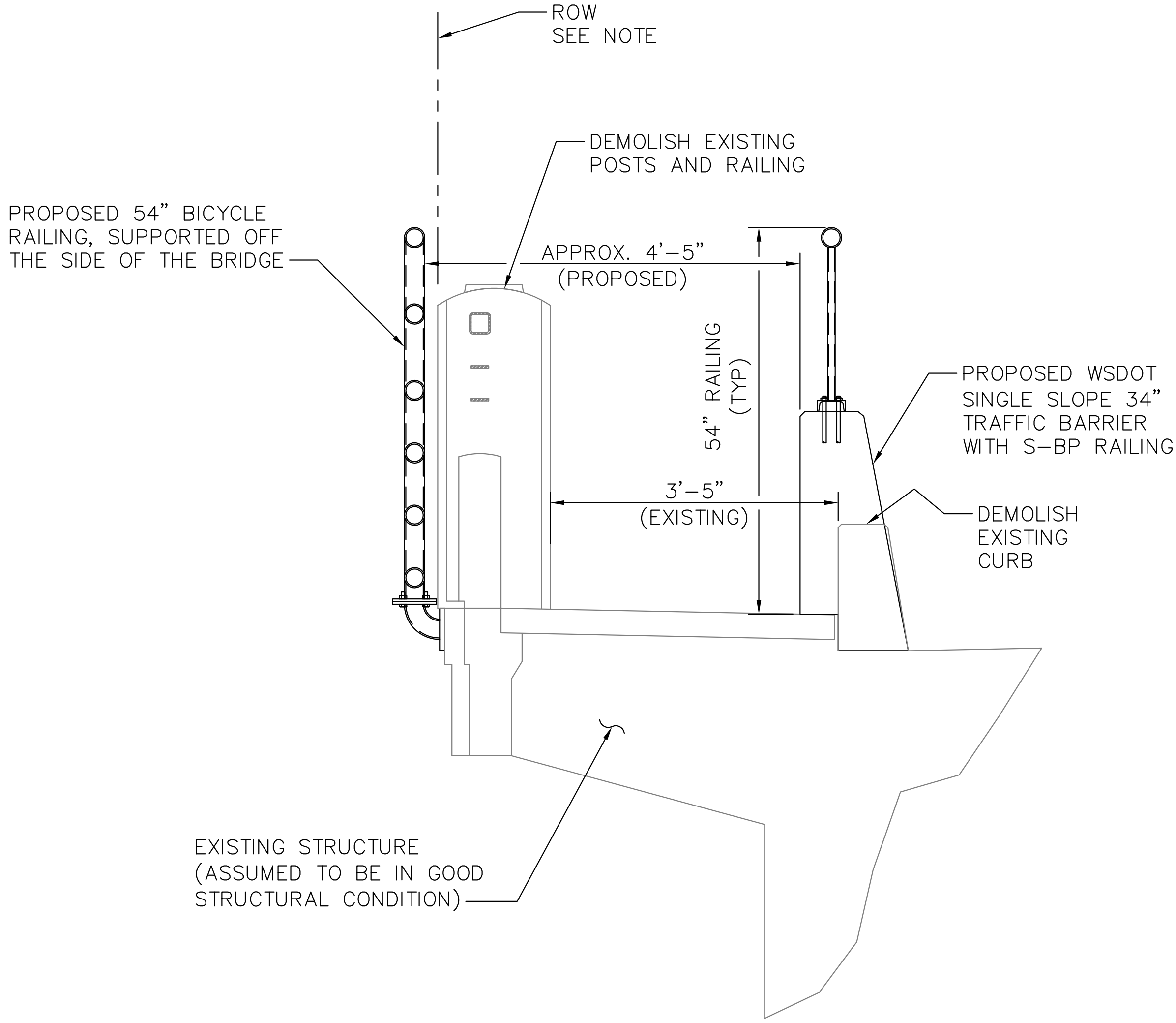
Aluminum Railings

Aluminum railings are a lightweight alternative to structural steel railings. Aluminum alloys are designed with a density of 175 pcf versus the 490 pcf density of steel. Although this weight difference is significant, the design strength of aluminum is approximately 85 percent to that of steel (due to lower AASHTO resistance factors), which would lead to larger member sizes which counter the weight savings. Additionally, the railings themselves are a minor part of the overall mass increase. Even significant mass savings from the railings would only provide minor savings to the overall mass increase.

7. Conclusion

Widening of the sidewalks within the Ballard Bridge elevated bridge structure segments is structurally feasible at a mass increase of less than 10 percent and without live load restrictions. The sidewalk widening for Segments 1 and 8 is feasible with limited impact to the existing retaining wall structures. The sketches provided in Appendix B show a typical sidewalk design, but will require significant additional detailing beyond what is shown for a complete understanding of what needs to be constructed.

Appendix B
Ballard Bridge Sections



ALT. 1 - TYPICAL SECTION

SCALE: 1/2" = 1'-0"

NOTE:
ROW LIMITS TYPICAL FOR SEGMENT 5 AND SOUTH END OF SEGMENT 3. ROW LIMITS ARE OUTSIDE OF BRIDGE EXTENTS FOR SEGMENTS 1, 2, THE NORTHERN END OF SEGMENT 3, AND SEGMENTS 6 TO 8.

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City of Seattle
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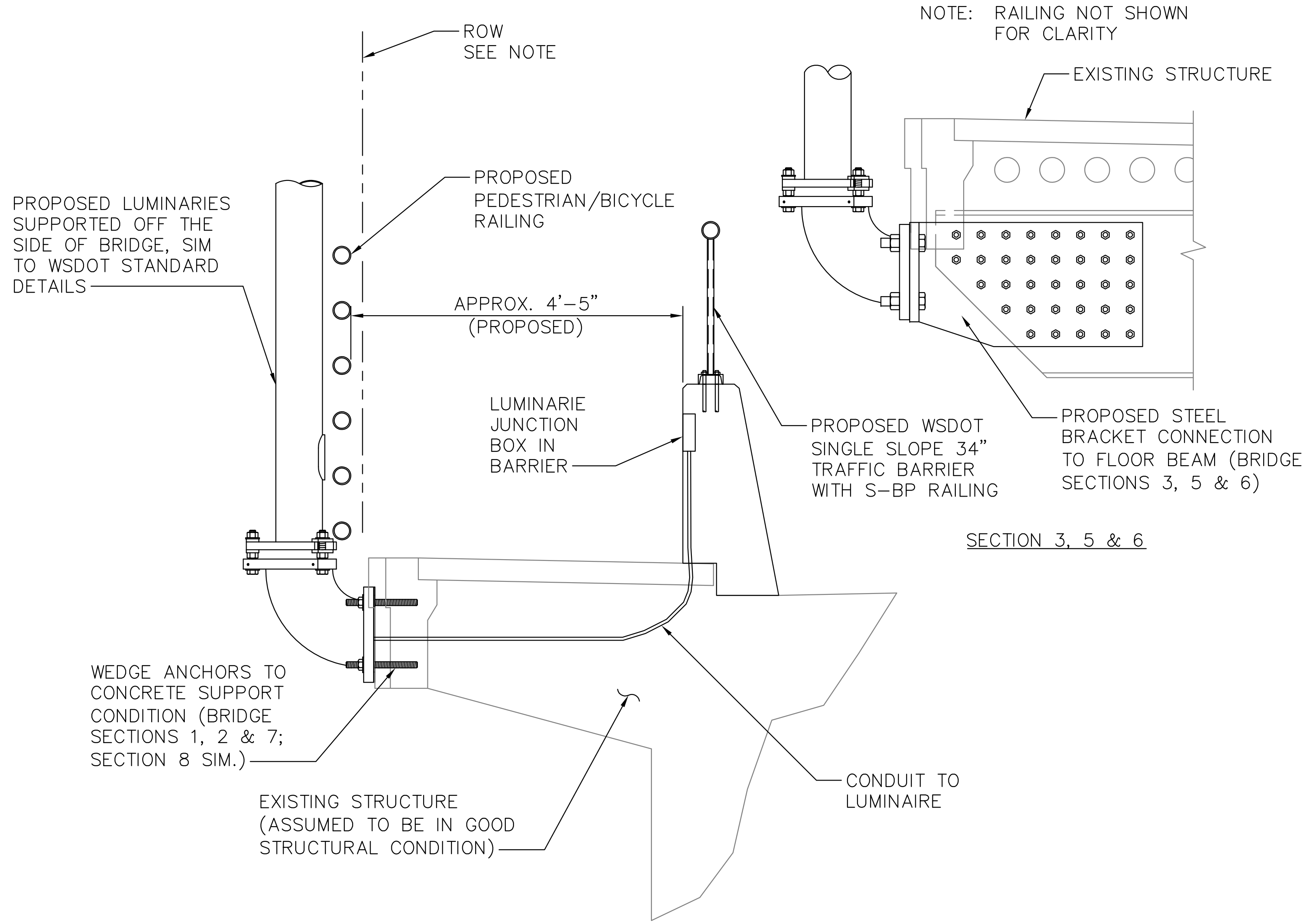
of Transportation

BALLARD BRIDGE
ALTERNATE 1
(MODIFY EXISTING SIDEWALK)



NOTE: ITEMS TO BE DEMOLISHED NOT SHOWN FOR CLARITY

NOTE:
ROW LIMITS TYPICAL FOR SEGMENT 5 AND SOUTH END OF SEGMENT 3. ROW LIMITS ARE OUTSIDE OF BRIDGE EXTENTS FOR SEGMENTS 1, 2, THE NORTHERN END OF SEGMENT 3, AND SEGMENTS 6 TO 8.

ALT. 1 - LIGHT POST LOCATION

SCALE: 1/2" = 1'-0"

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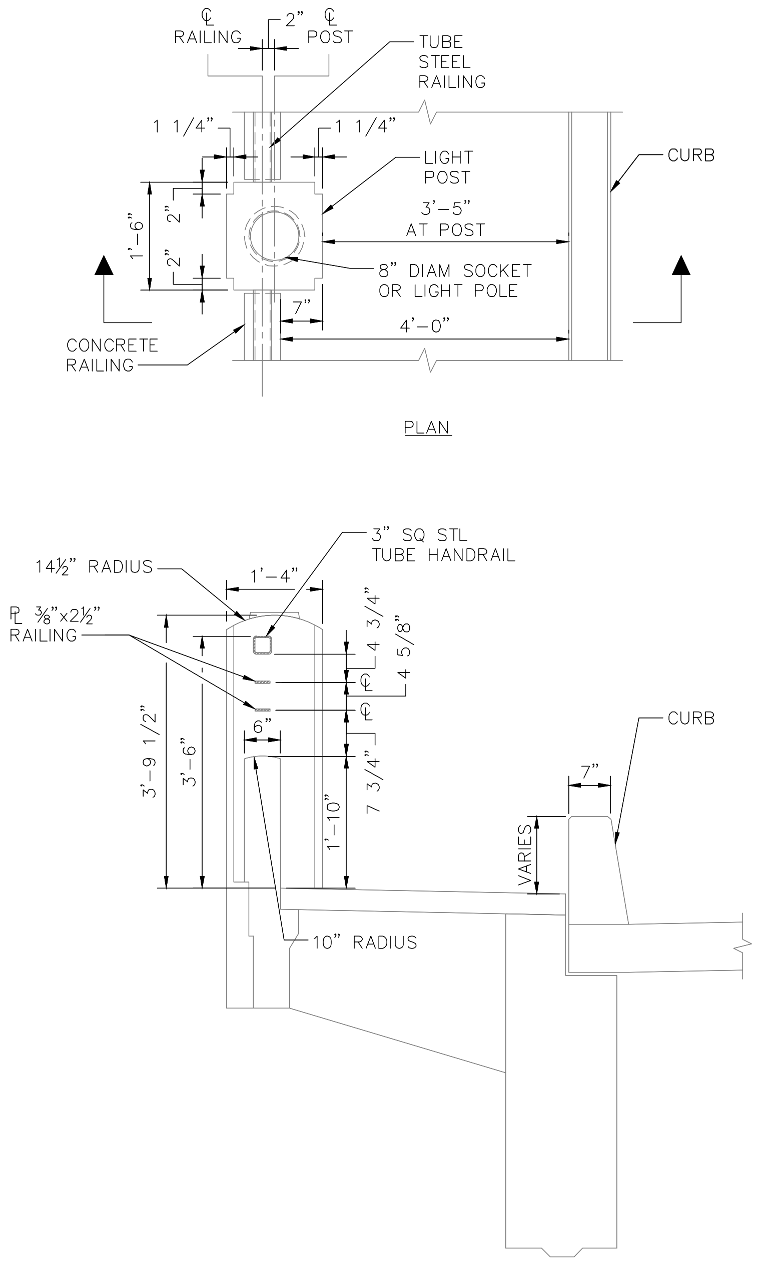
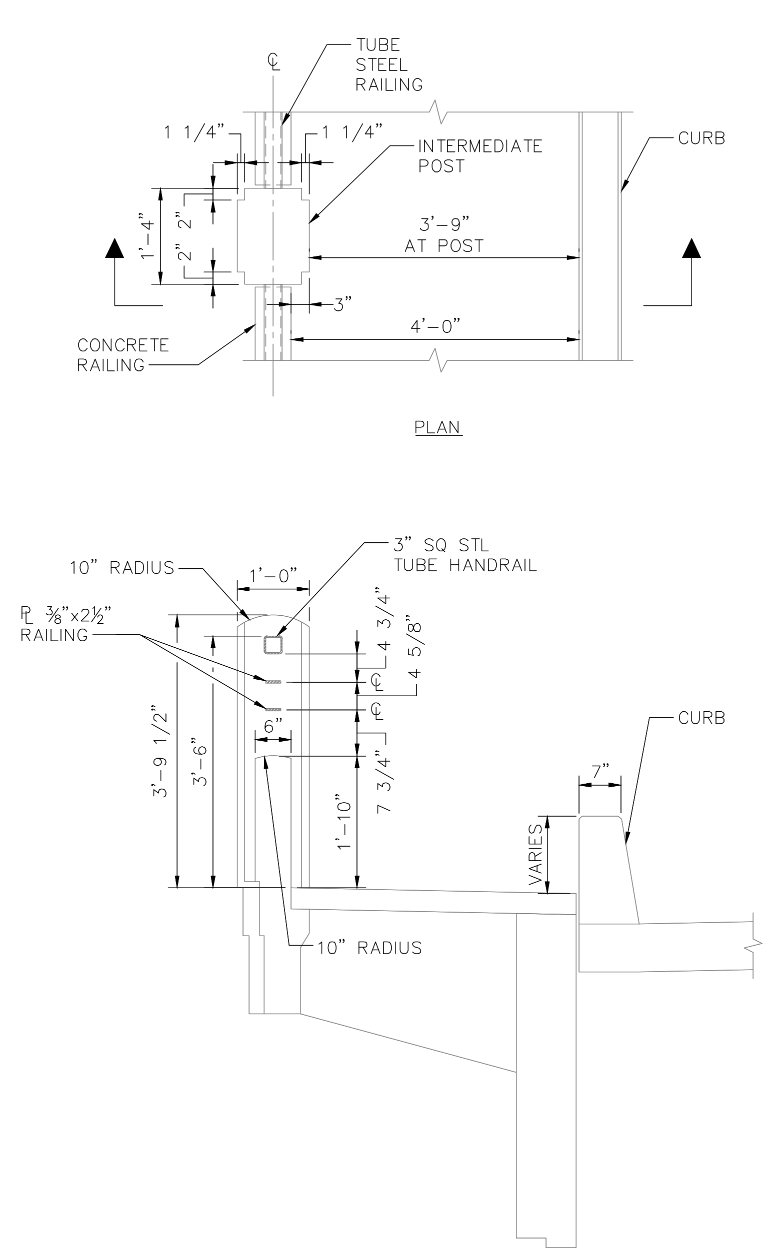
City of Seattle
Seattle Department of Transportation

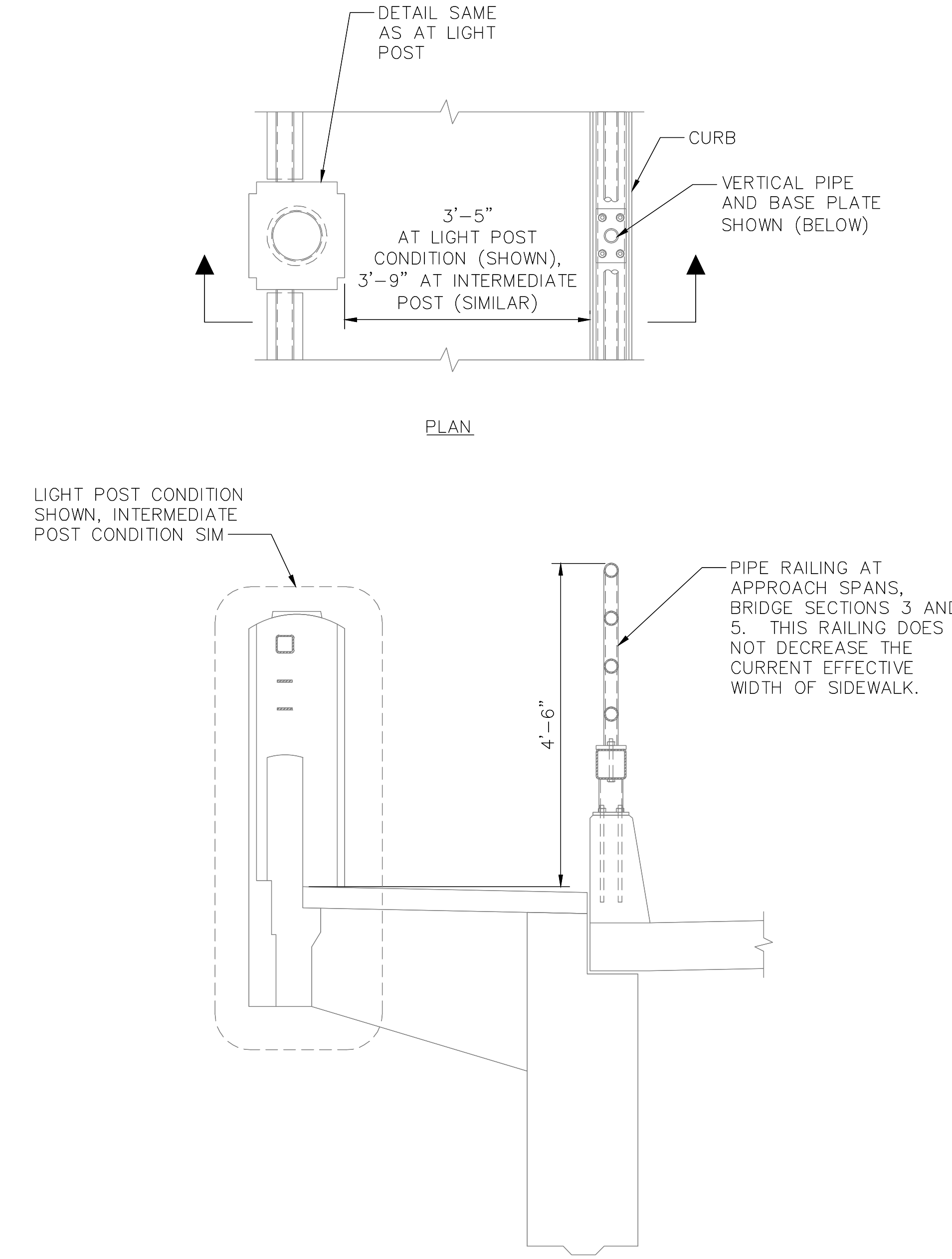
BALLARD BRIDGE
ALTERNATE 1
(MODIFY EXISTING SIDEWALK)



LIGHT POST (AS-BUILT)
SCALE: 3/4" = 1'-0"



INTERMEDIATE POST (AS-BUILT)
SCALE: 3/4" = 1'-0"



APPROACH SPAN RAILING (AS-BUILT)
SCALE: 3/4" = 1'-0"

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHK'D	REV'D

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DEPARTMENT OF FINANCE & ADMINISTRATIVE SERVICES
SEATTLE, WASHINGTON . . . . 20 . .
BY: . . . .
PURCHASING AND CONTRACTING SERVICES DIRECTOR

NAME OR INITIALS AND DATE
DESIGNED . . . .
CHECKED . . . .
DRAWN . . . .
CHECKED . . . .

INITIALS AND DATE
REVIEWED:
PE . . . . CONST. . . . . .
PROJ. MGR. . . . .
RECEIVED . . . .
REVISED AS BUILT . . . .

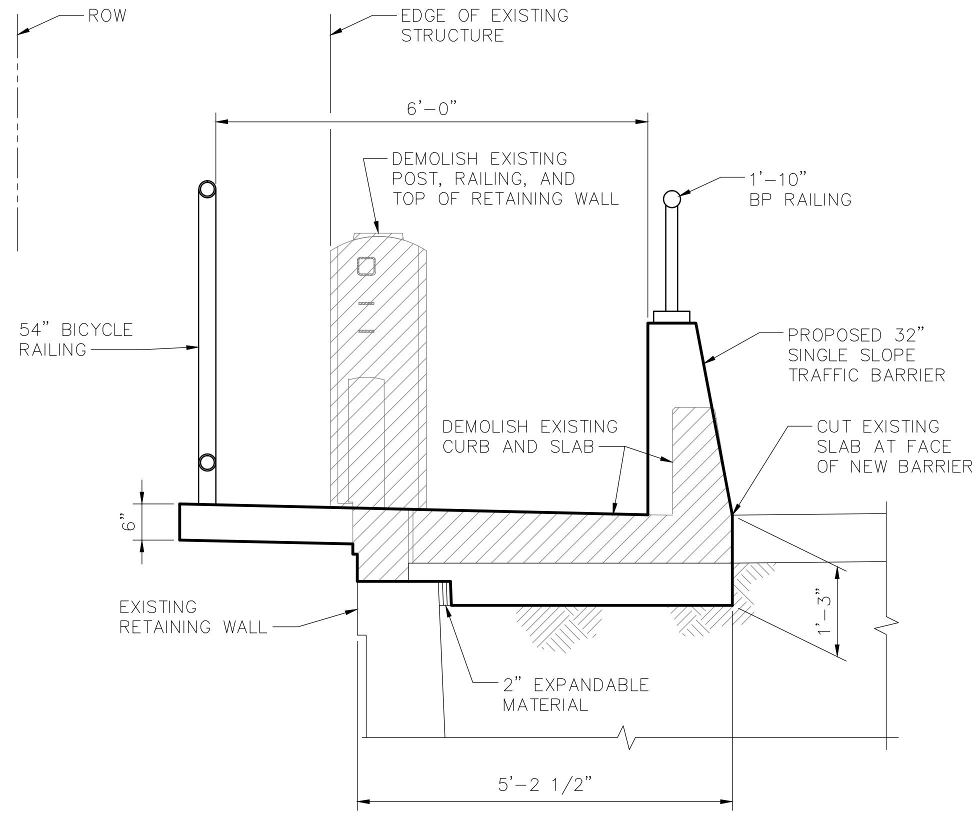
ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

City of Seattle
Seattle Department of Transportation
ORDINANCE NO. . . . . APPROVED. . . . .
FUND: . . . .
INSPECTOR'S BOOK . . . . . .

BALLARD BRIDGE
SIDEWALK SHARED PATH STUDY

JOB NO. PC
R/W
CO
VAULT PLAN NO.
SHEET OF

Last Saved by: Childress on: Jul 22, 2013 8:36 AM File: Q:\Seattle\2010\SAPWT-10-057\CADD\Design\BSP\Existing Cross-Sections.dwg



TYPICAL SECTION 1 -

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

Last Saved by: Childress on: Jul 22, 2013 8:17 AM File: Q:\Seattle\2010\SAPWT-10-057\CADD\Design\TRL\S6.00.dwg

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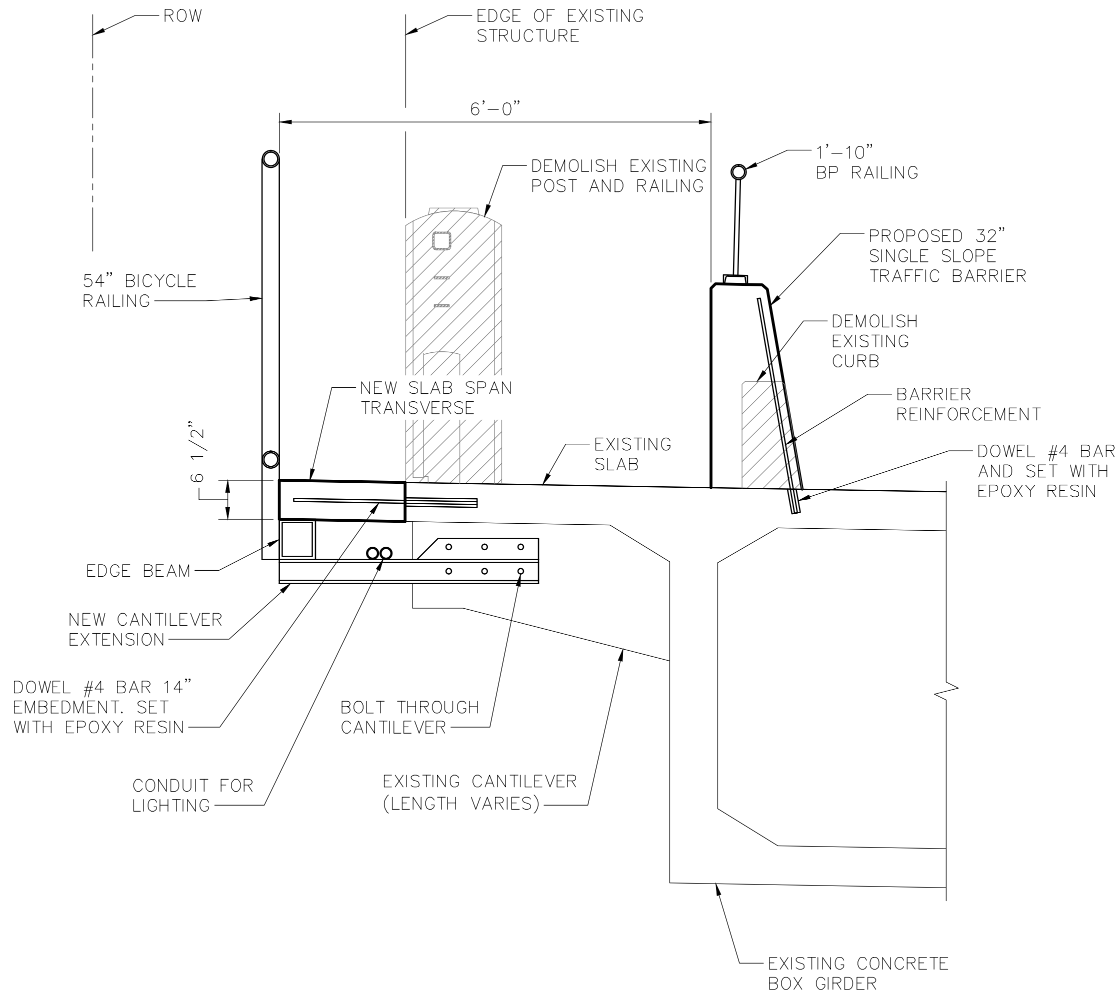
BALLARD BRIDGE
6' WALKWAY
SEGMENT 1

DRAWING NO.: S6.00
JOB NO.: SAPWT-10-057
DATE: 2/7/2013



TYPICAL SECTION 1/-
SCALE: 3/4" = 1'-0"

LEGEND:

(hatch) = DEMOLITION EXTENTS

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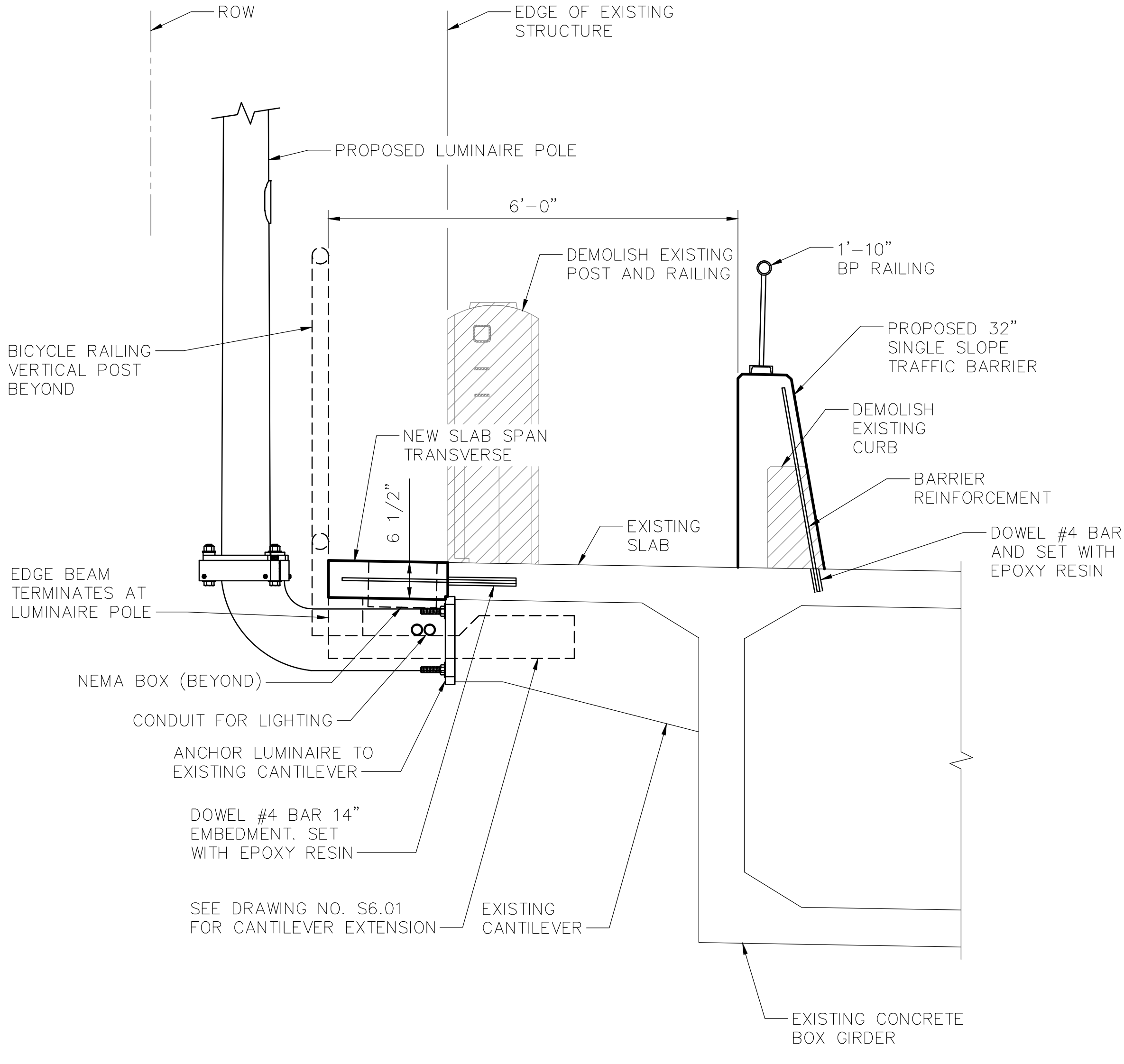
BALLARD BRIDGE
6' WALKWAY
SEGMENT 2

DRAWING NO.: S6.01
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE 2 / -

SCALE: 3/4" = 1'-0"

LEGEND:

(hatched) = DEMOLITION EXTENTS

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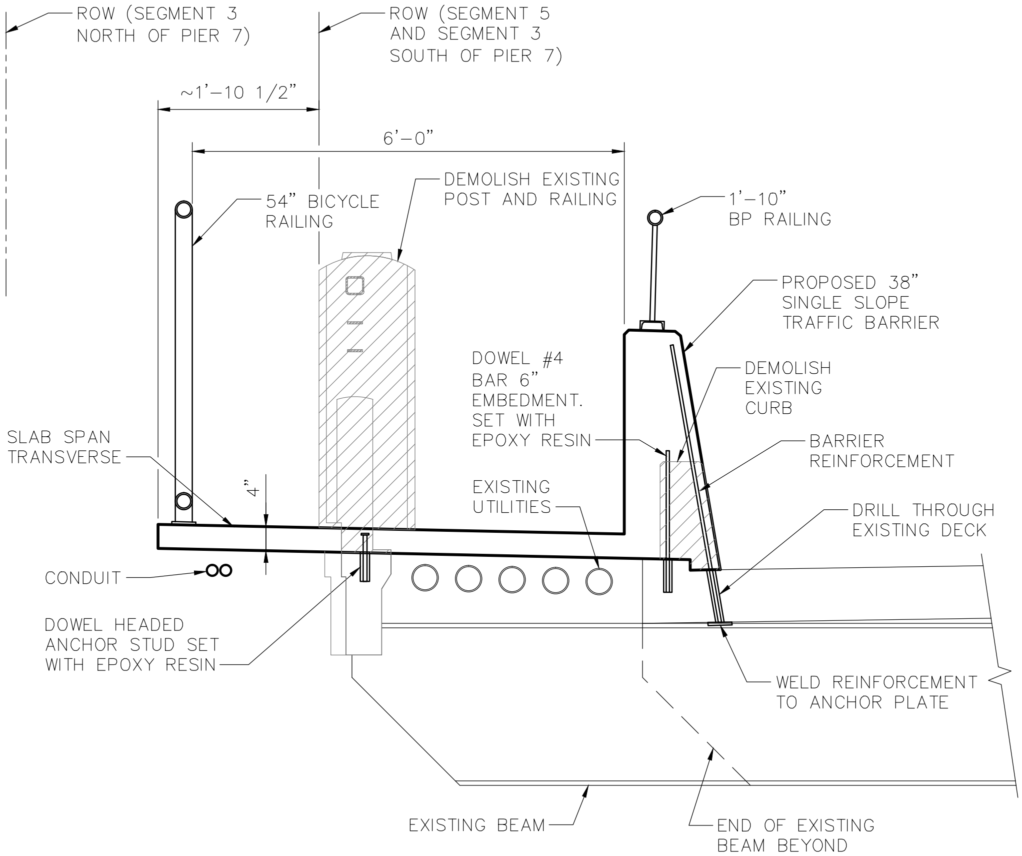
BALLARD BRIDGE
6' WALKWAY
SEGMENT 2

DRAWING NO.: S6.02
JOB NO.: SAPWT–10–057
DATE: 9/18/2012



TYPICAL SECTION 1/-

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE AND EDGE BEAM NOT SHOWN FOR CLARITY.

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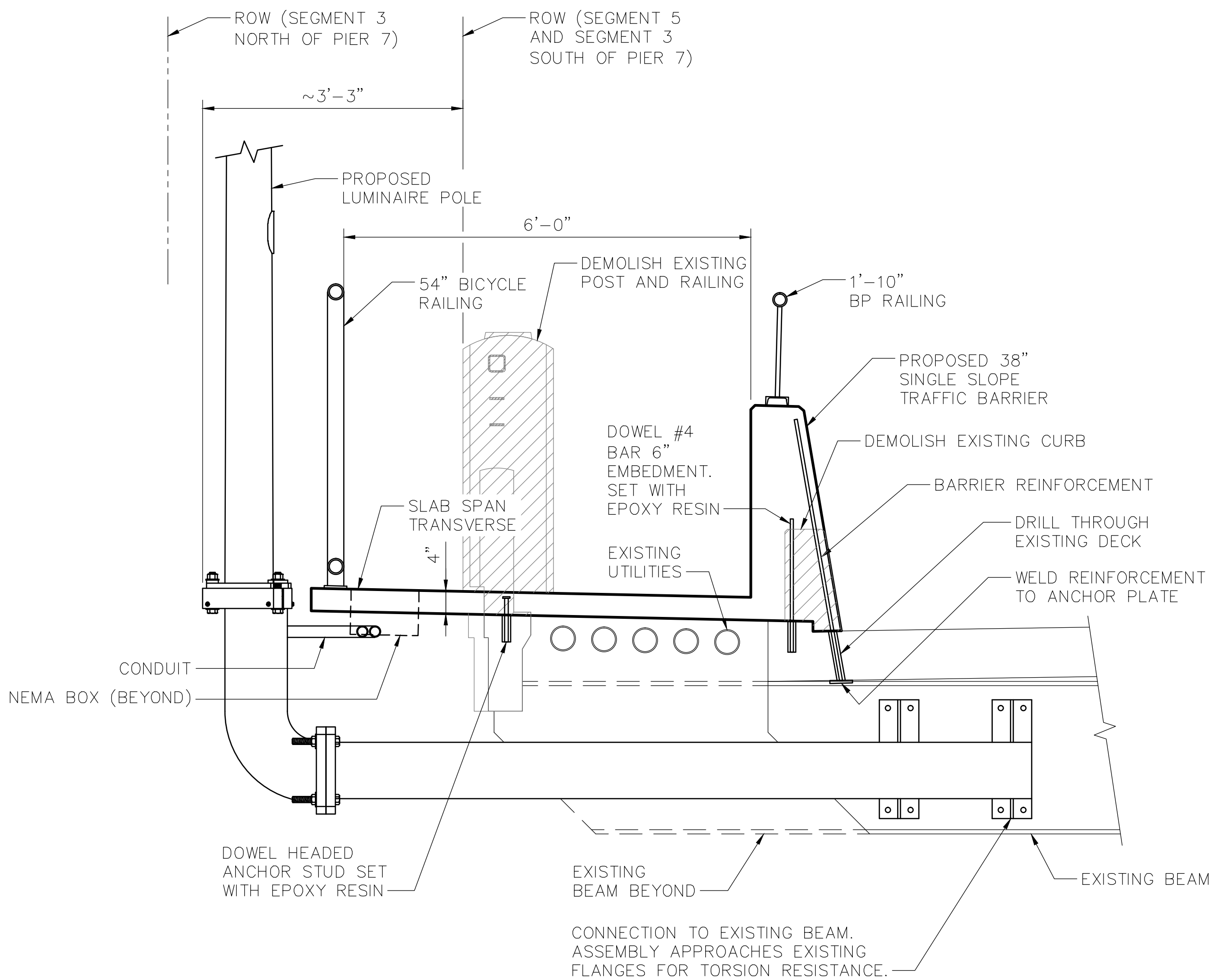
BALLARD BRIDGE
6' WALKWAY
SEGMENTS 3 & 5

DRAWING NO.: S6.03
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE (2 / -)

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE AND EDGE BEAM NOT SHOWN FOR CLARITY.

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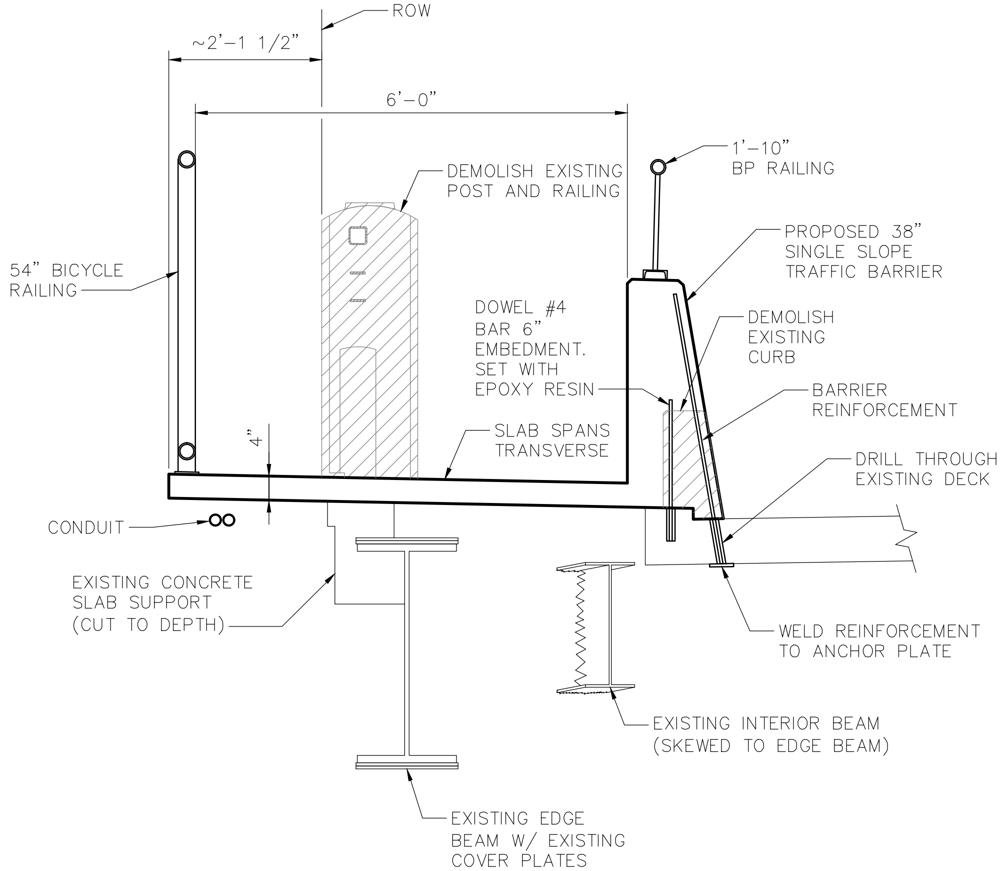
BALLARD BRIDGE
6' WALKWAY
SEGMENTS 3 & 5

DRAWING NO.:	S6.04
JOB NO.:	SAPWT-10-057
DATE:	9/18/2012



TYPICAL SECTION 1/-

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE AND EDGE BEAM NOT SHOWN FOR CLARITY.

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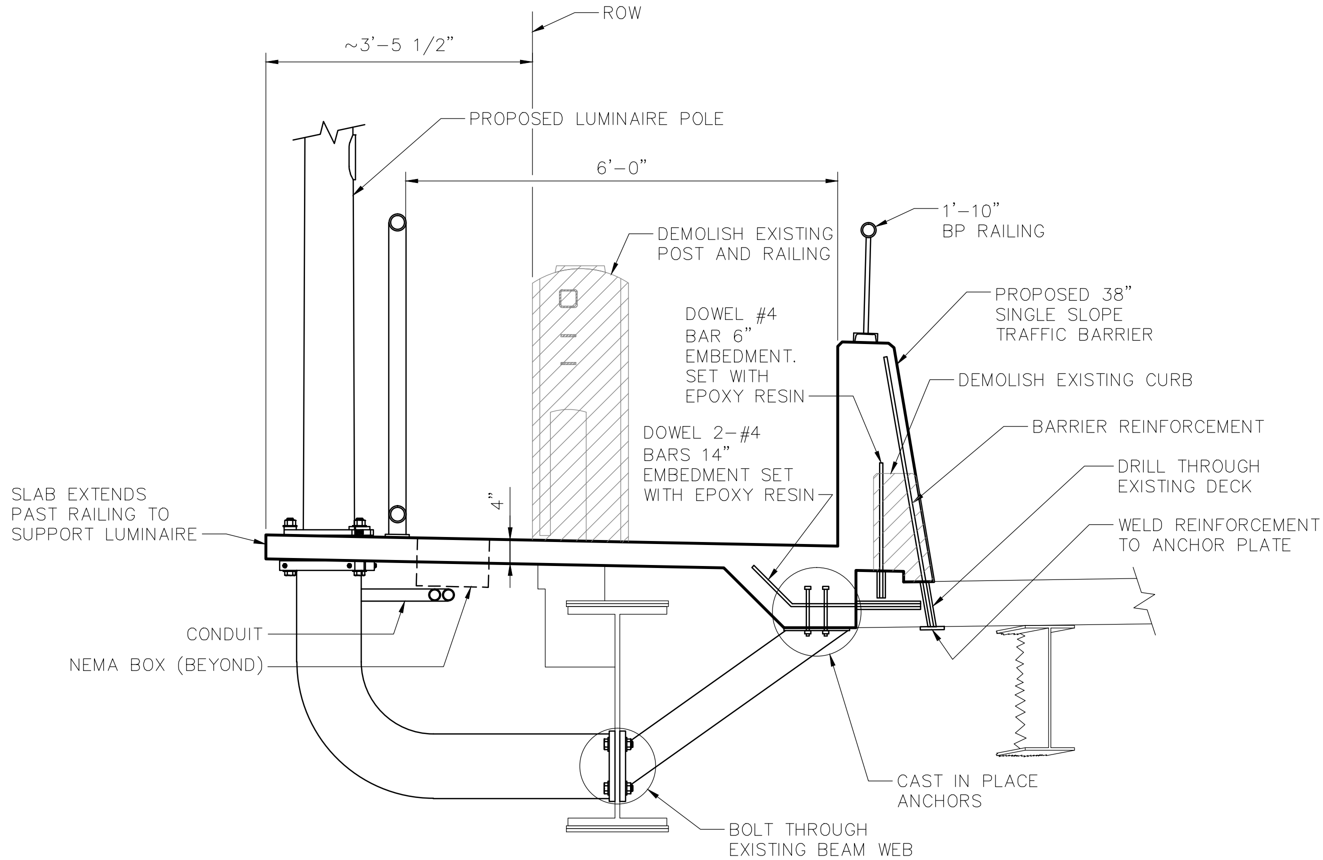
Seattle Department of Transportation

BALLARD BRIDGE
6' WALKWAY
SEGMENT 6

DRAWING NO.: S6.05
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE (2 / -)

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE
AND EDGE BEAM NOT SHOWN FOR CLARITY.

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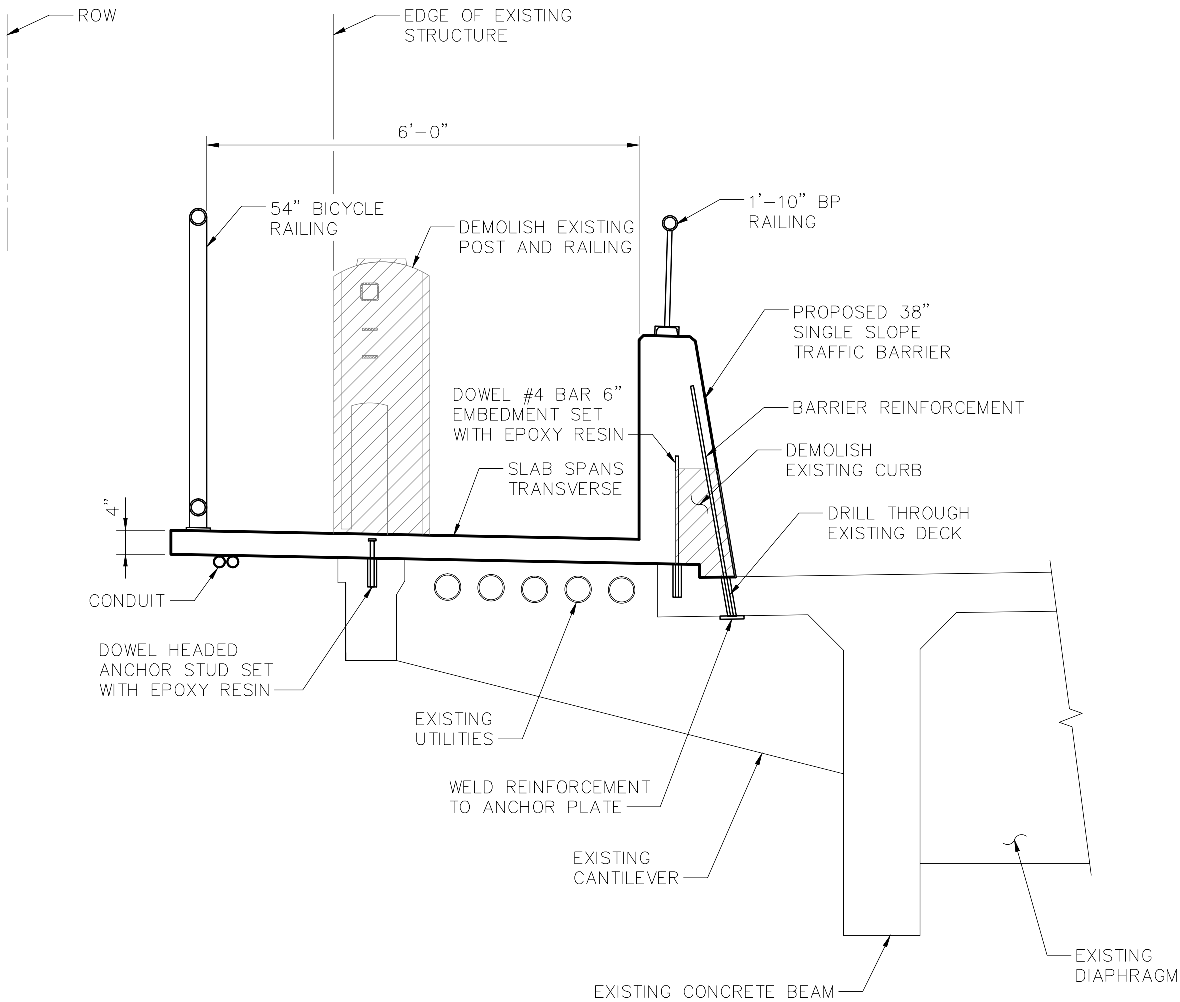
BALLARD BRIDGE
6' WALKWAY
SEGMENT 6

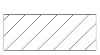
DRAWING NO.: S6.06
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



TYPICAL SECTION (1/-)

SCALE: 3/4" = 1'-0"

LEGEND:

 = DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE AND EDGE BEAM NOT SHOWN FOR CLARITY.

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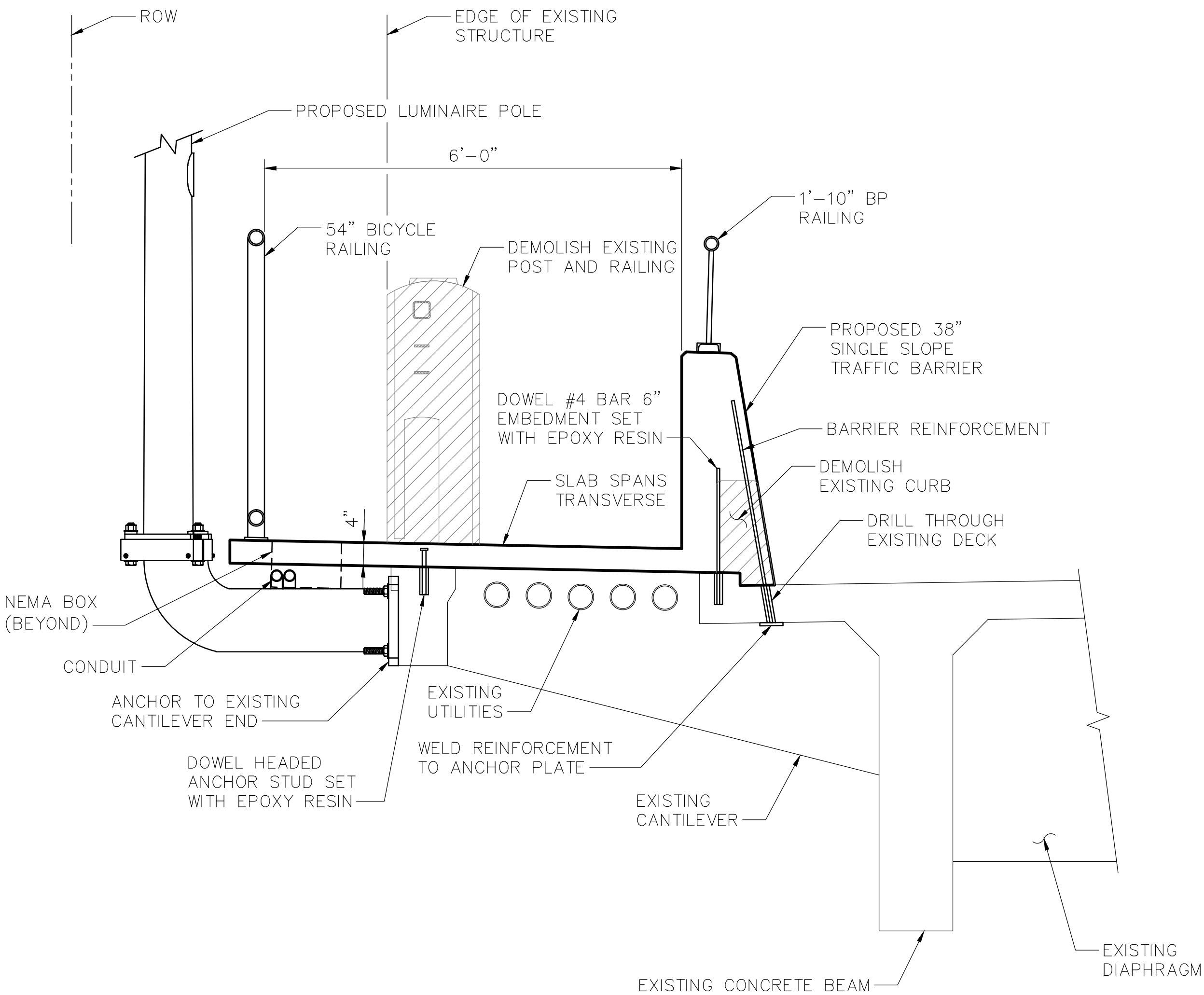
BALLARD BRIDGE
6' WALKWAY
SEGMENT 7

DRAWING NO.: S6.07
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE 2 / -

SCALE: 3/4" = 1'-0"

LEGEND:

(hatched) = DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE AND EDGE BEAM NOT SHOWN FOR CLARITY.

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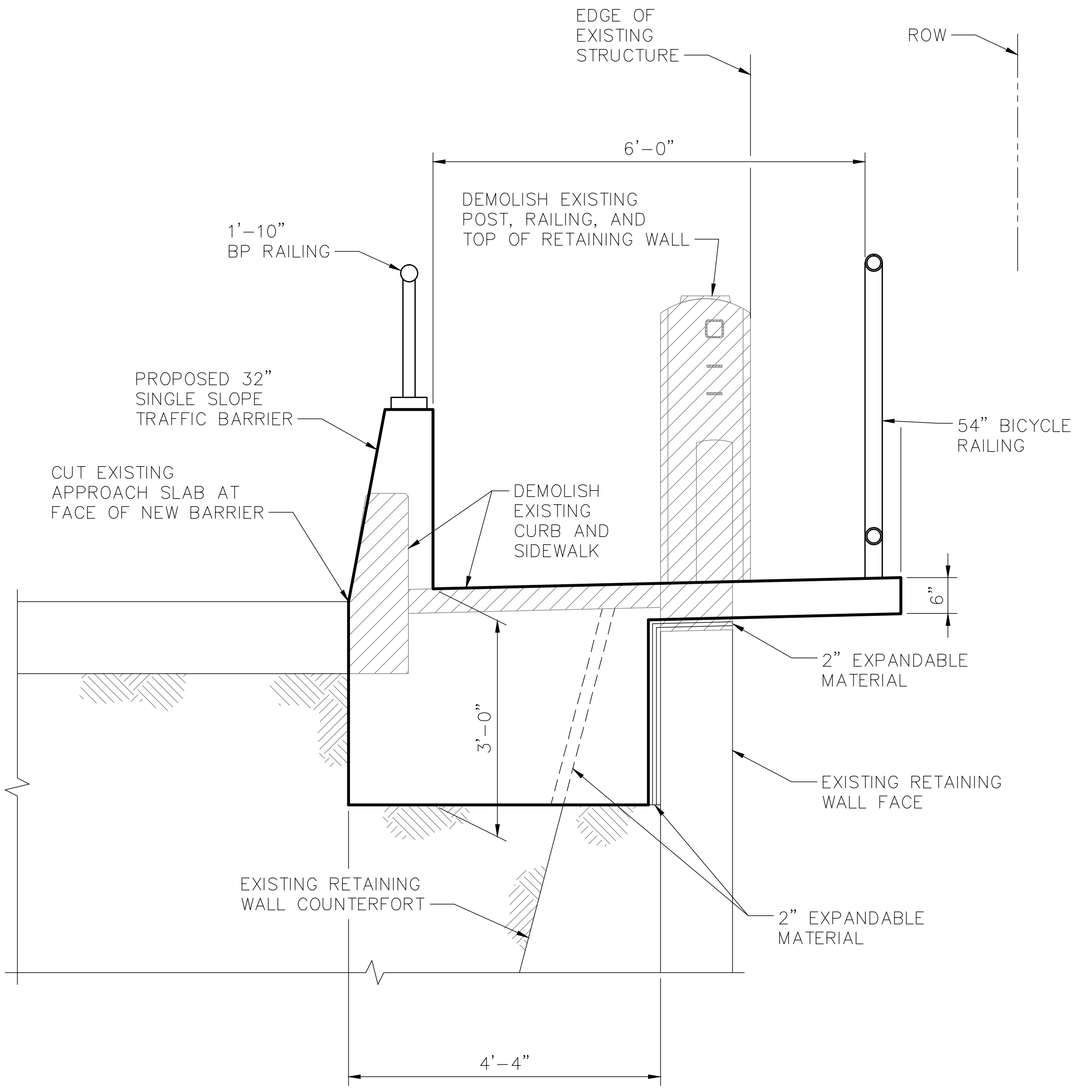
BALLARD BRIDGE
6' WALKWAY
SEGMENT 7

DRAWING NO.:	S6.08
JOB NO.:	SAPWT-10-057
DATE:	9/18/2012



TYPICAL SECTION 1 / -

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

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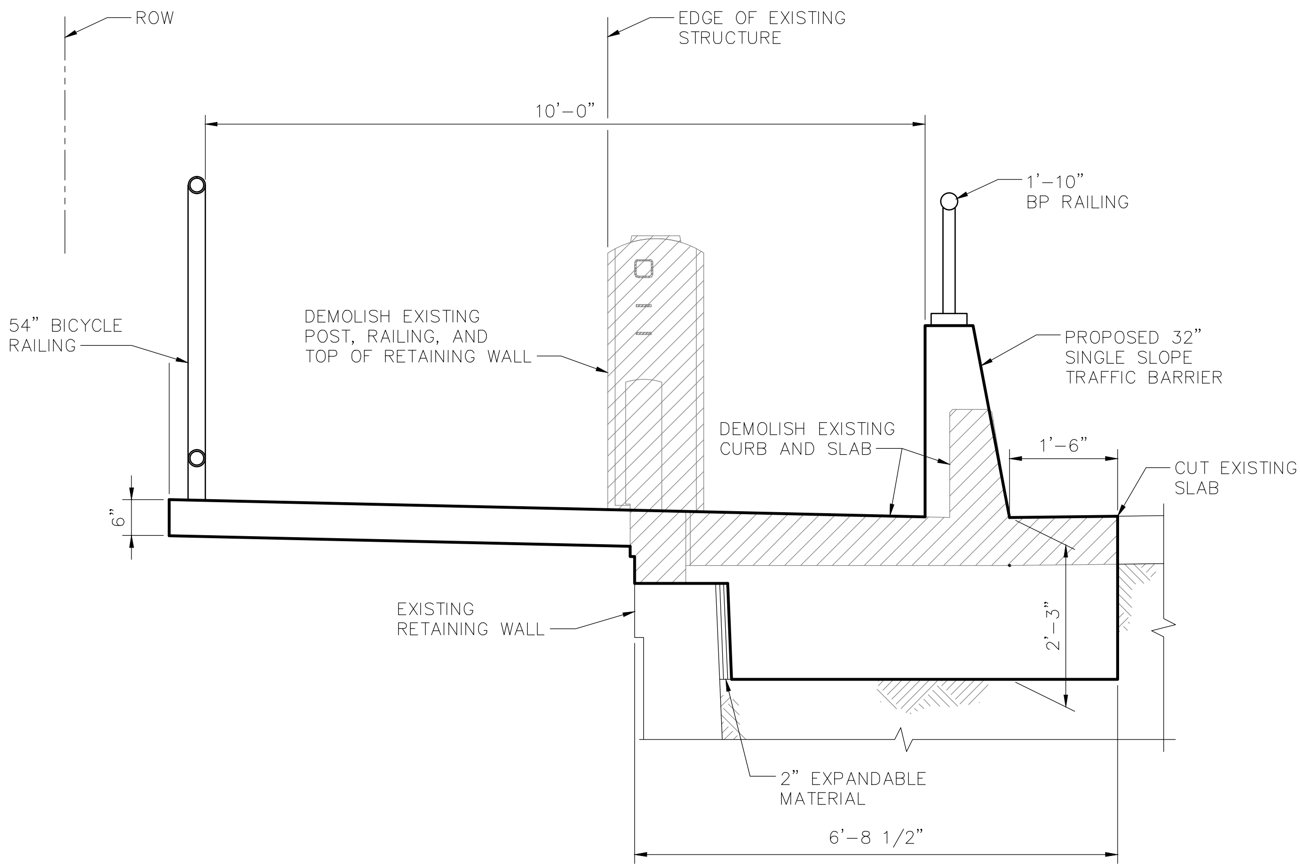
BALLARD BRIDGE
6' WALKWAY
SEGMENT 8

DRAWING NO.: S6.09
JOB NO.: SAPWT–10–057
DATE: 2/7/2013



TYPICAL SECTION 1 / -

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

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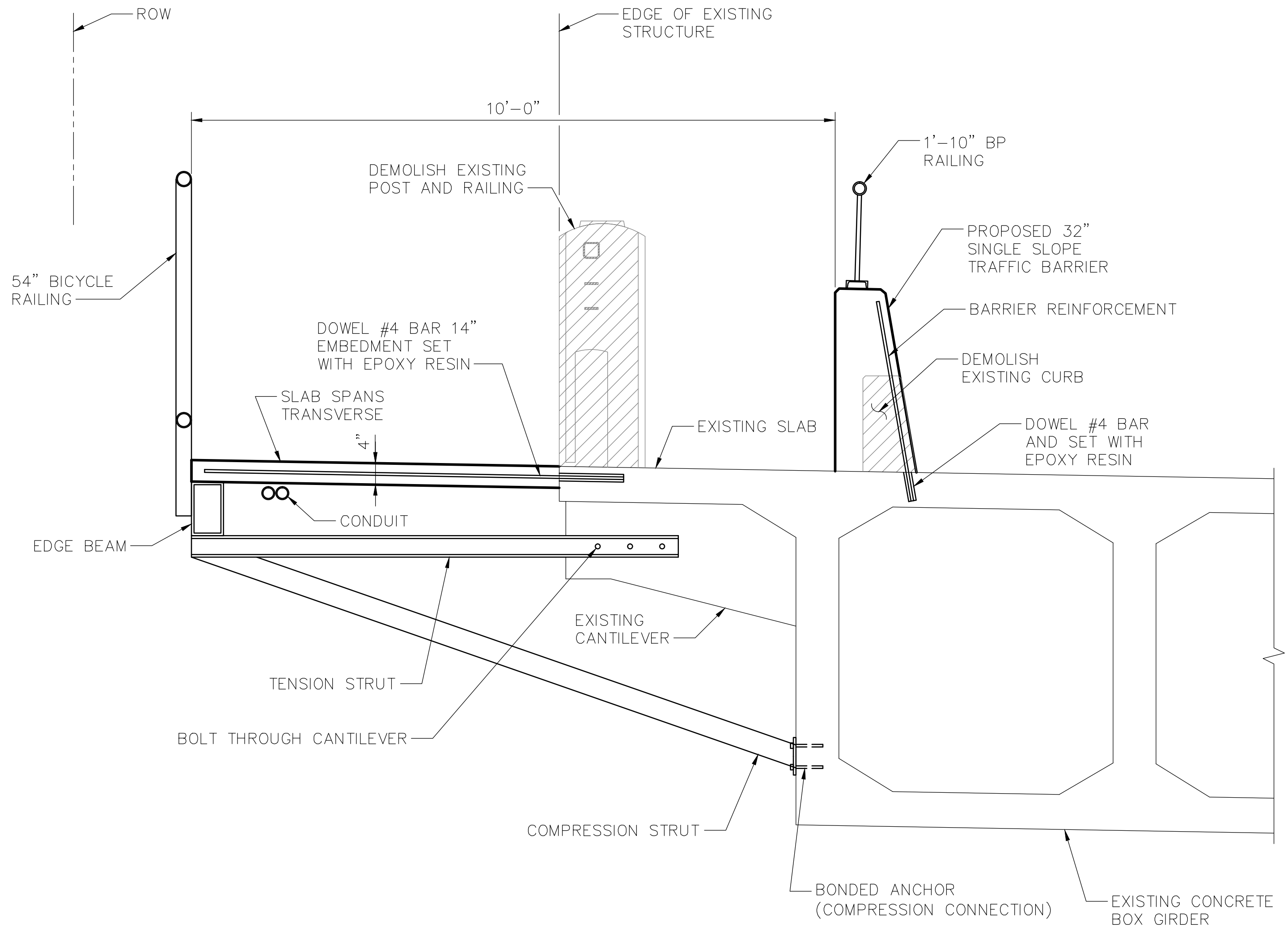
Seattle Department of Transportation

BALLARD BRIDGE
10' WALKWAY
SEGMENT 1

DRAWING NO.: S10.00

JOB NO.: SAPWT–10–057

DATE: 2/7/2013



TYPICAL SECTION 1 -

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

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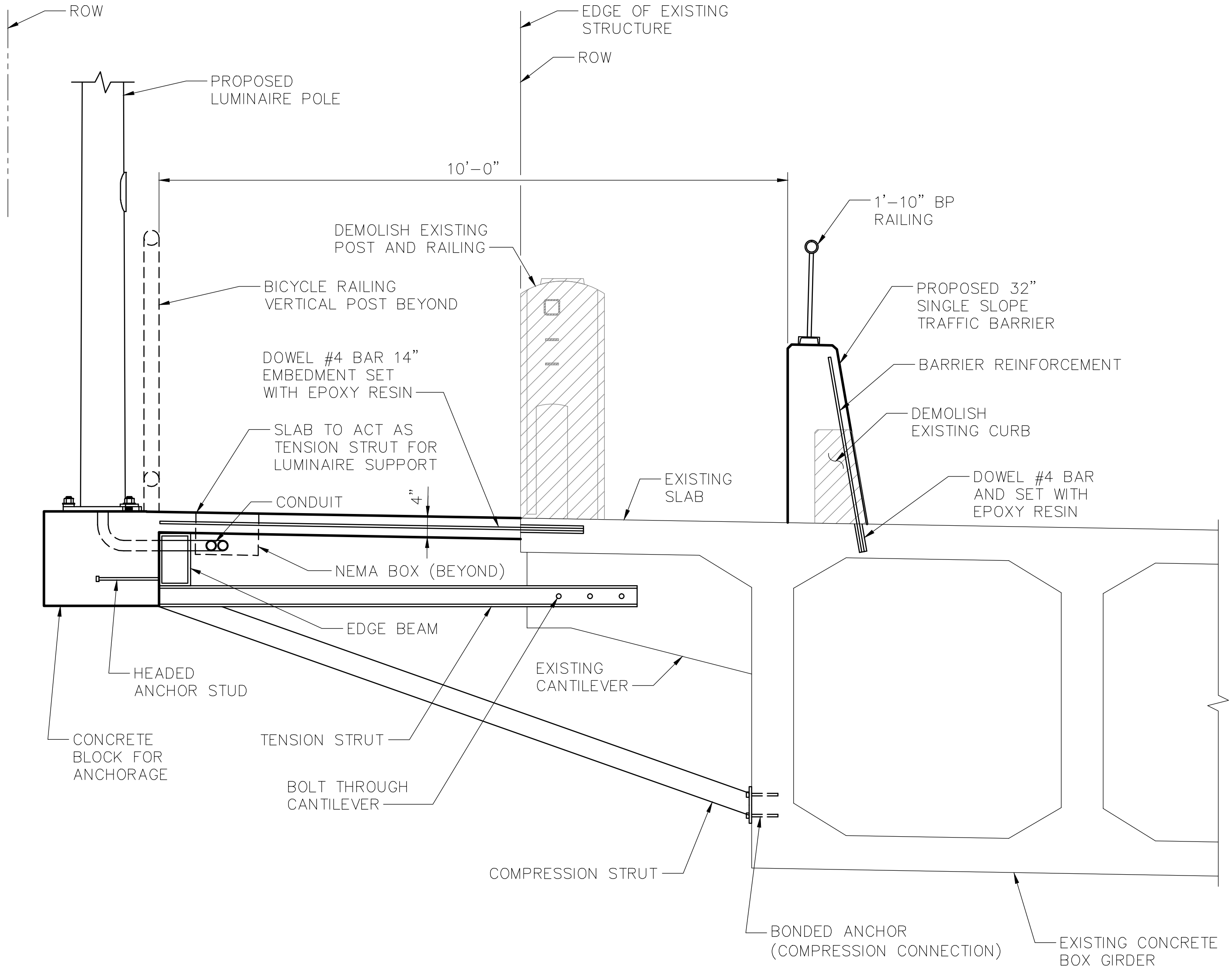
BALLARD BRIDGE
10' WALKWAY
SEGMENT 2

DRAWING NO.: S10.01

JOB NO.: SAPWT–10–057

DATE: 9/18/2012



SECTION AT LUMINAIRE 2 -

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

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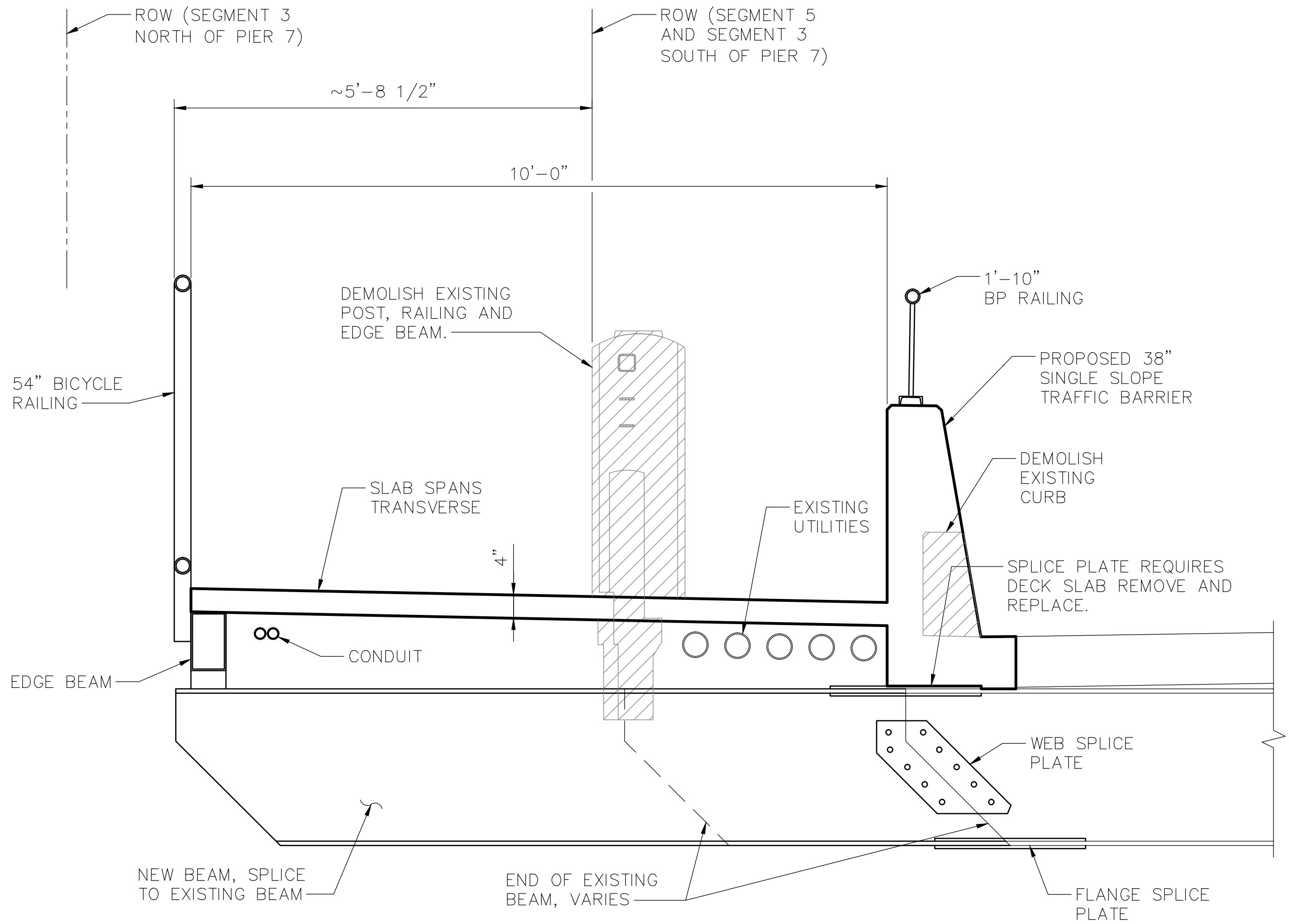
BALLARD BRIDGE
10' WALKWAY
SEGMENT 2

DRAWING NO.:	S10.02
JOB NO.:	SAPWT-10-057
DATE:	9/18/2012



TYPICAL SECTION (1/-)

SCALE: 3/4" = 1'-0"

LEGEND:

[hatched box] = DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE NOT SHOWN FOR CLARITY.

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BALLARD BRIDGE
10' WALKWAY
SEGMENTS 3 & 5

DRAWING NO.: S10.03
JOB NO.: SAPWT-10-057
DATE: 9/18/2012

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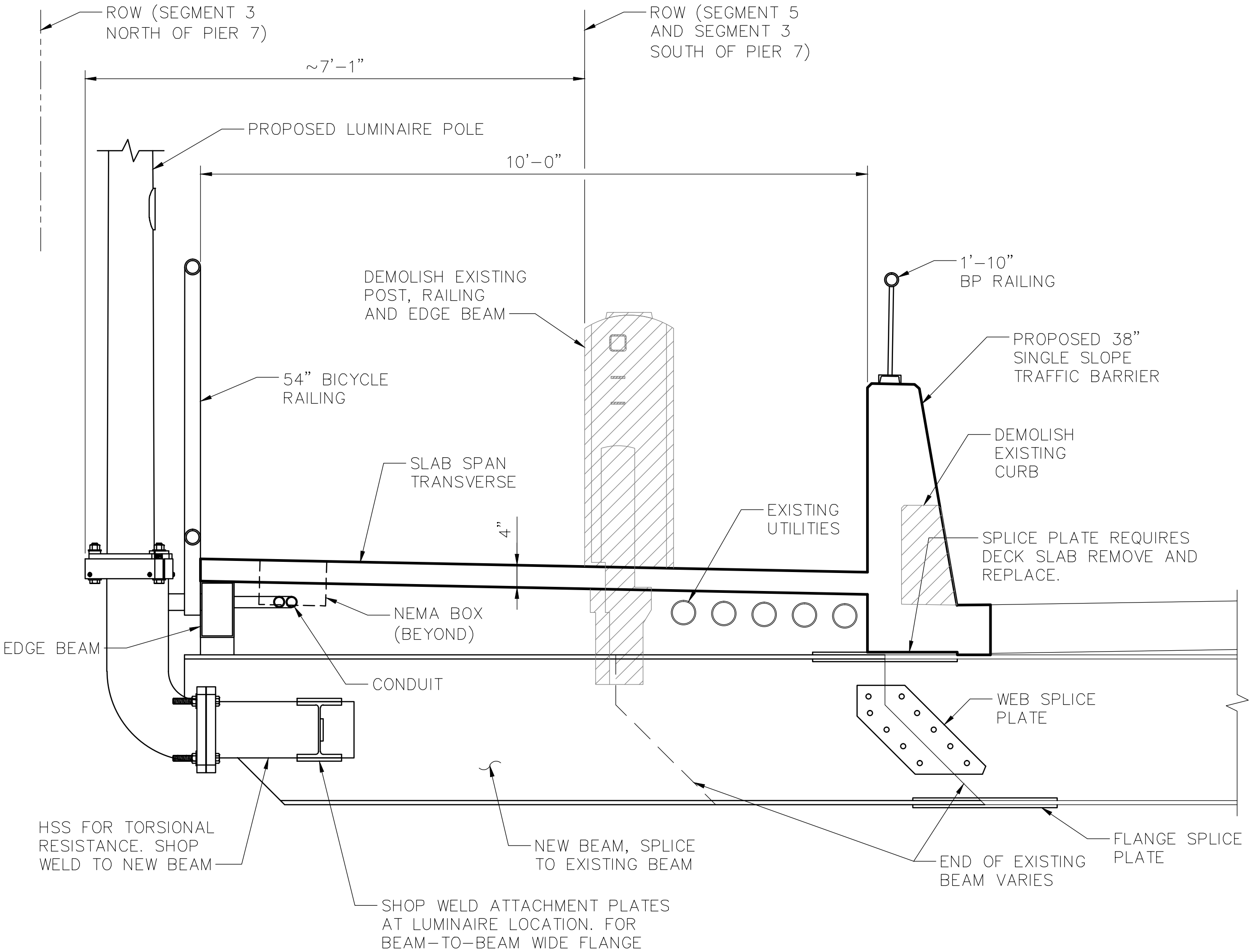

SECTION AT LUMINAIRE (2 / -)

SCALE: 3/4" = 1'-0"

LEGEND:

[hatched] = DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE NOT SHOWN FOR CLARITY.

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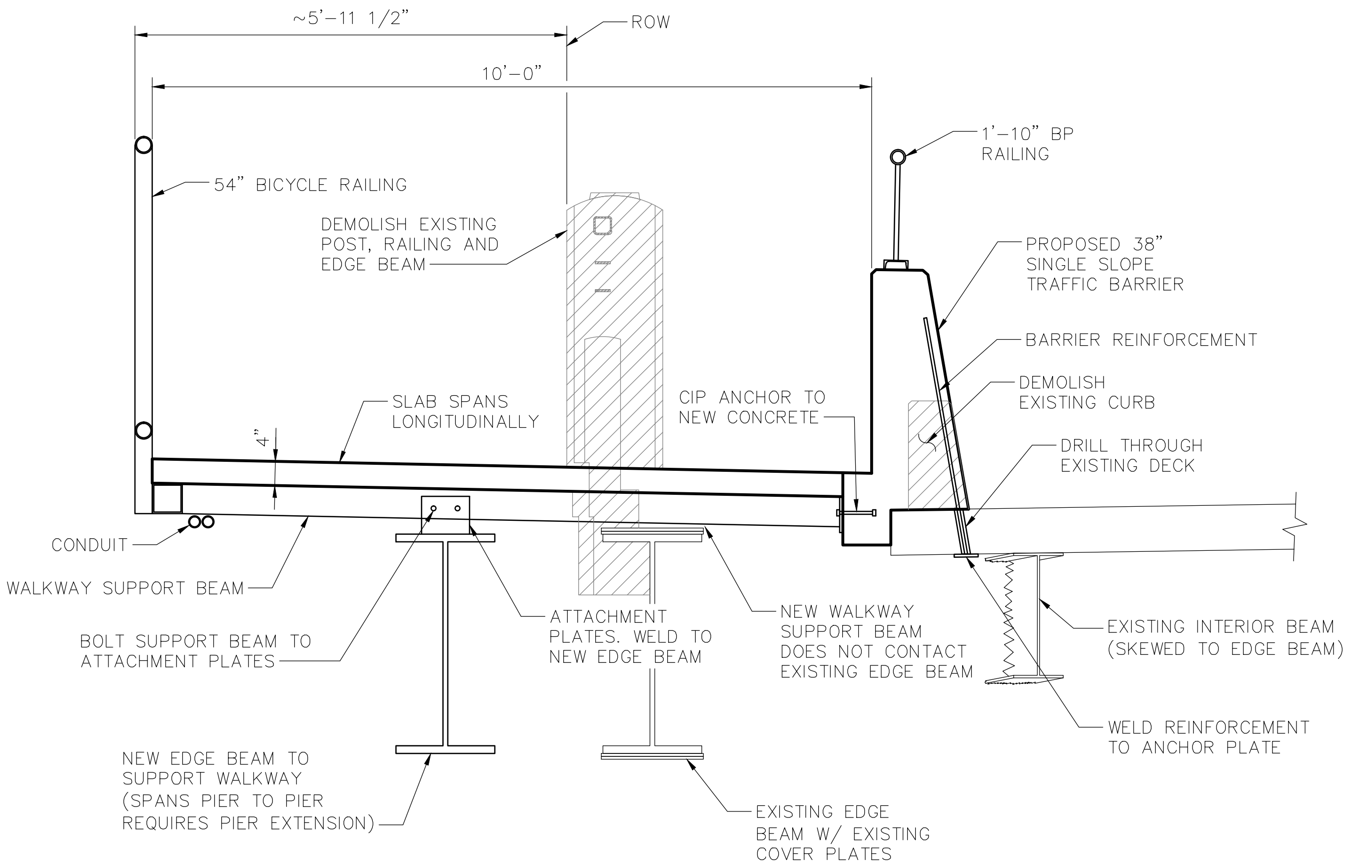
BALLARD BRIDGE
10' WALKWAY
SEGMENTS 3 & 5

DRAWING NO.: S10.04
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



TYPICAL SECTION (1/-)

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY
SURFACE NOT SHOWN FOR CLARITY.

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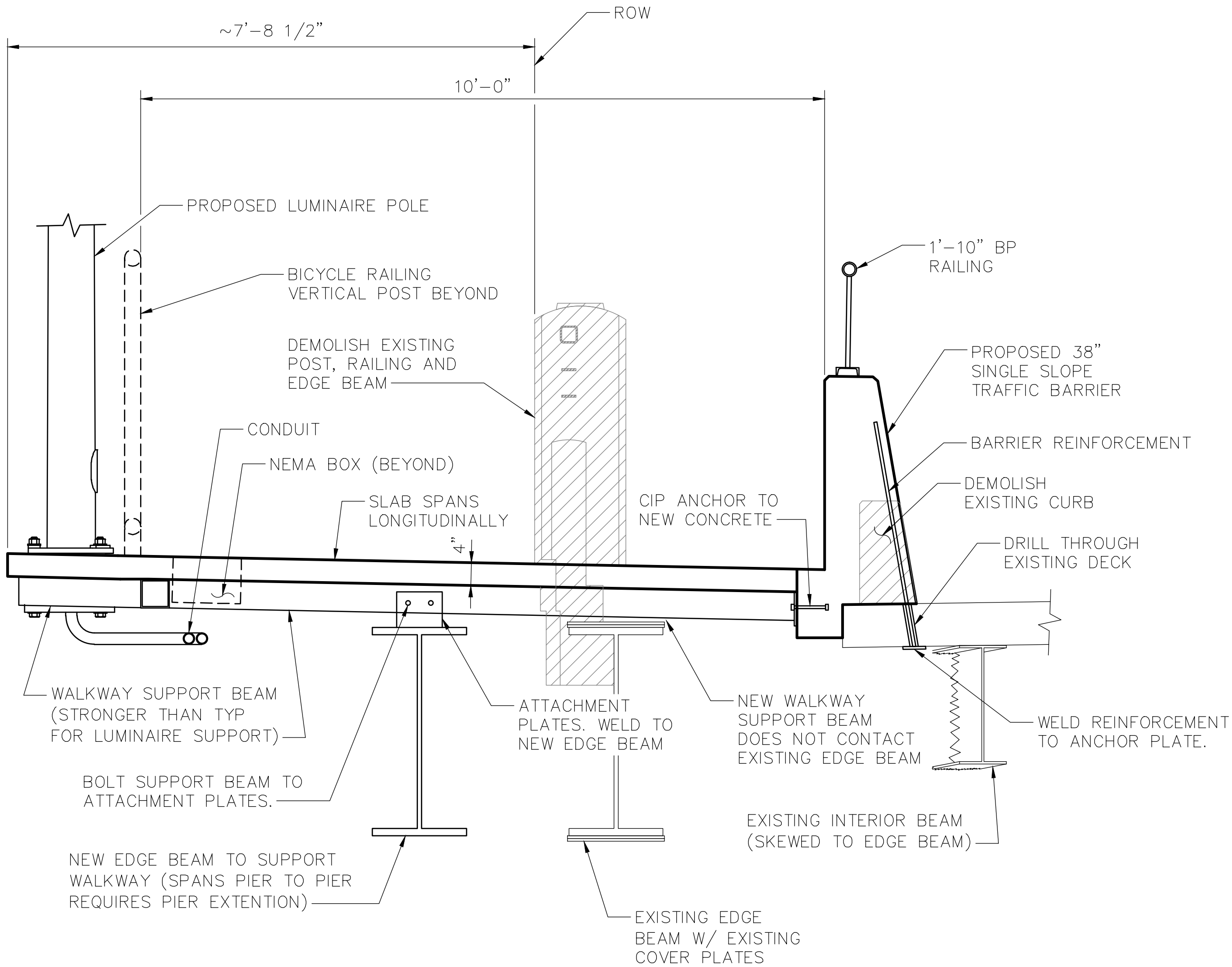
BALLARD BRIDGE
10' WALKWAY
SEGMENT 6

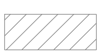
DRAWING NO.: S10.05
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE 2 / -

SCALE: 3/4" = 1'-0"

LEGEND:


= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE NOT SHOWN FOR CLARITY.

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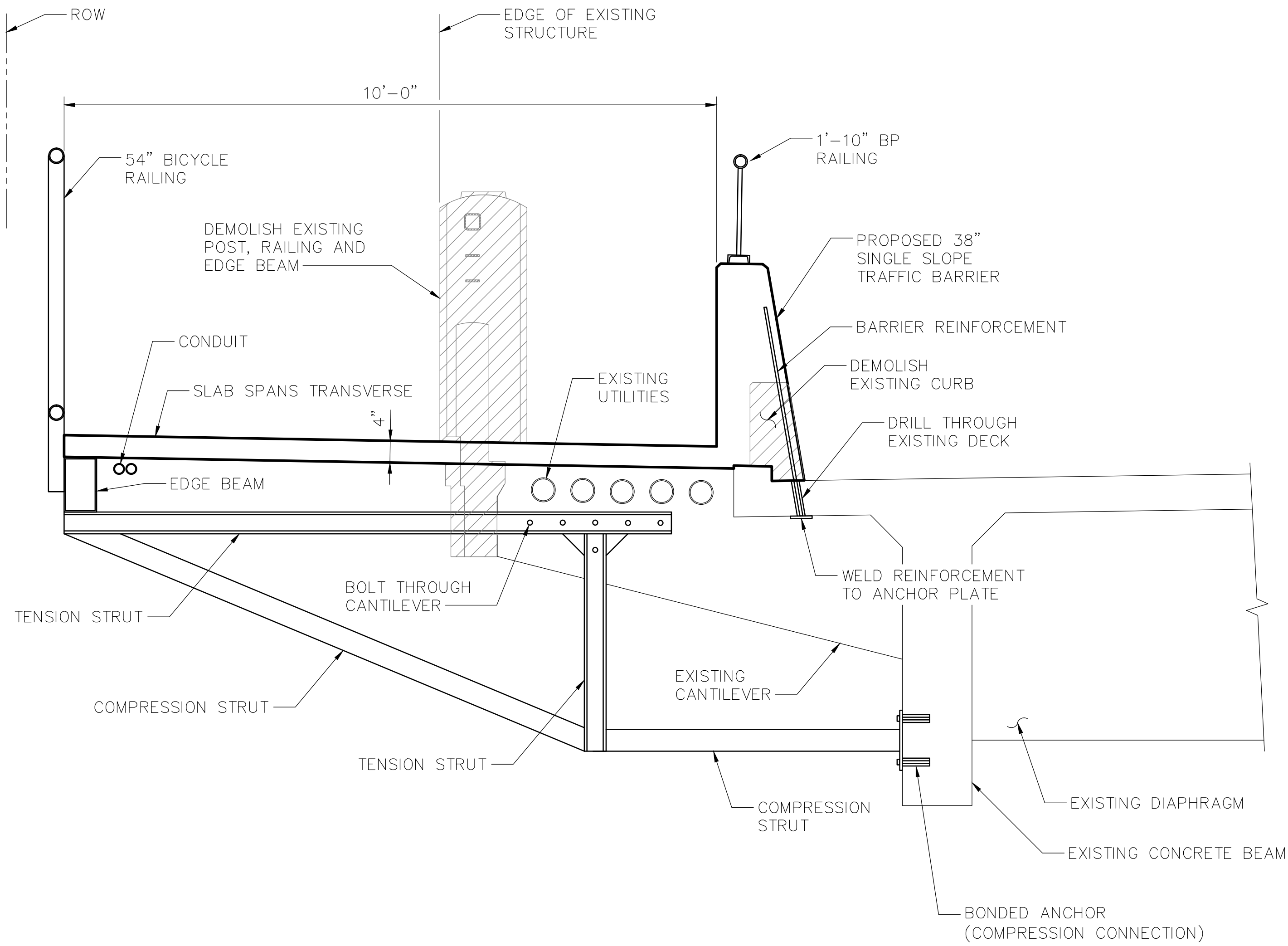
BALLARD BRIDGE
10' WALKWAY
SEGMENT 6

DRAWING NO.: S10.06
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



TYPICAL SECTION 1/-

SCALE: 3/4" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE NOT SHOWN FOR CLARITY.

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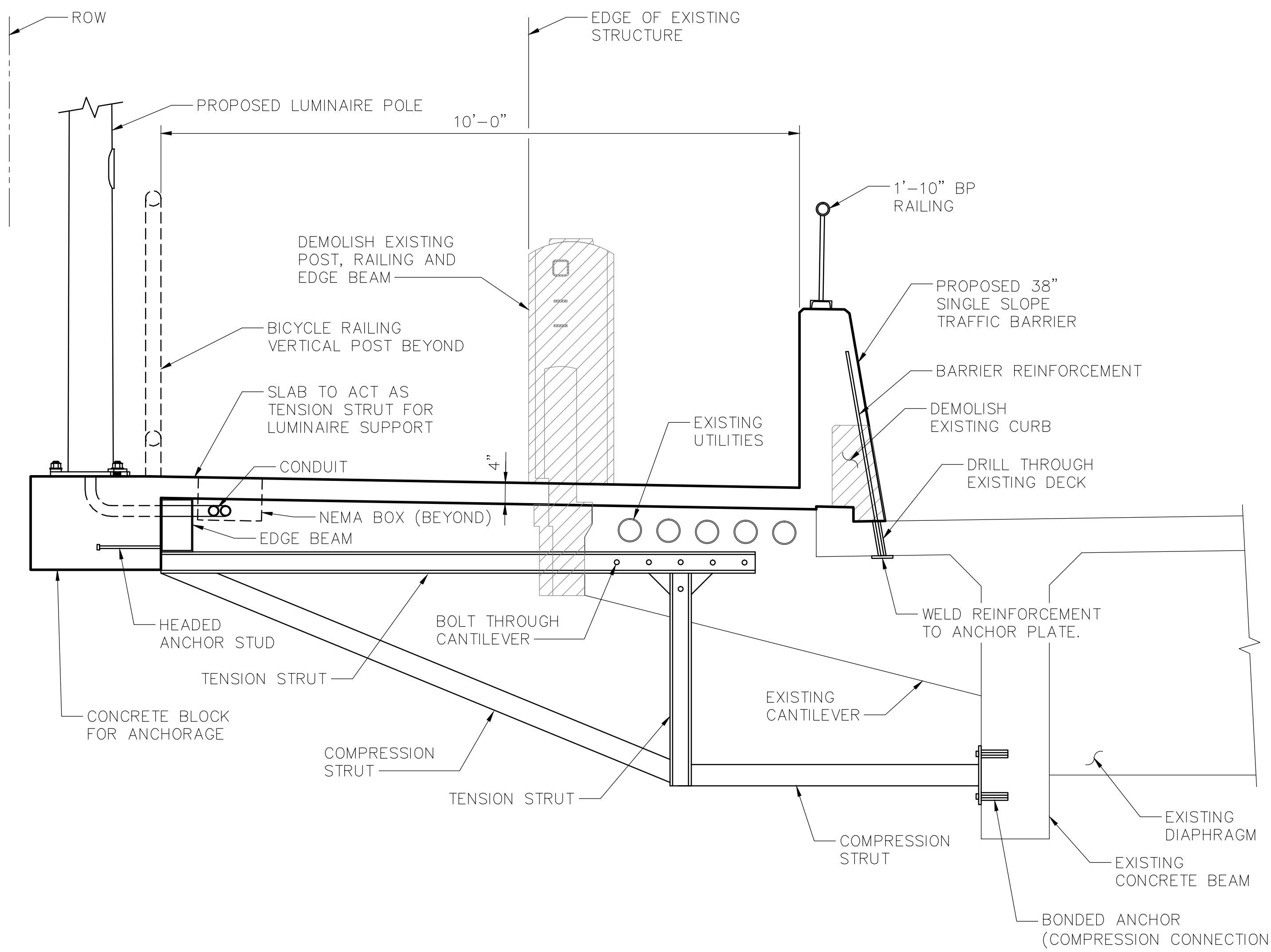
Seattle Department of Transportation

BALLARD BRIDGE
10' WALKWAY
SEGMENT 7

DRAWING NO.: S10.07
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



SECTION AT LUMINAIRE 2 / -

SCALE: 3/4" = 1'-0"

LEGEND:

[hatched box] = DEMOLITION EXTENTS

NOTE:

DEMOLITION OF EXISTING WALKWAY SURFACE NOT SHOWN FOR CLARITY.

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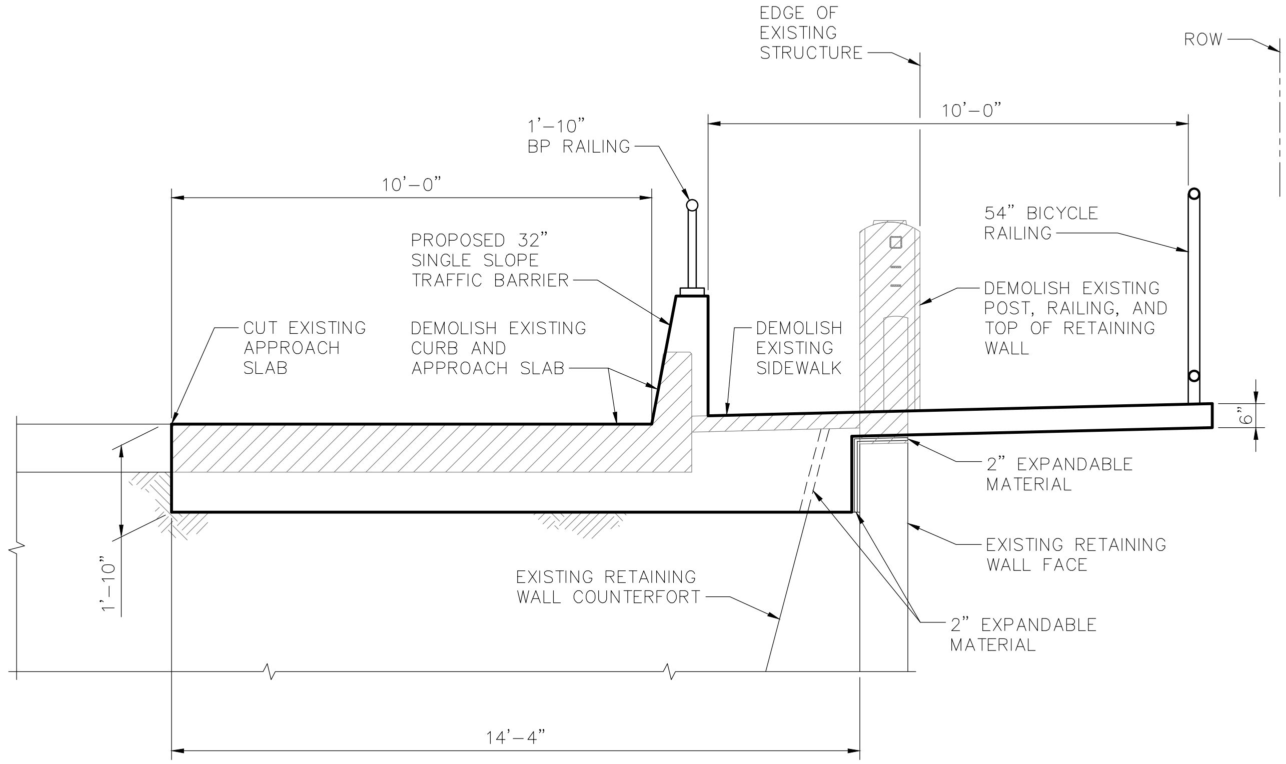
Seattle Department of Transportation

BALLARD BRIDGE
10' WALKWAY
SEGMENT 7

DRAWING NO.: S10.08
JOB NO.: SAPWT-10-057
DATE: 9/18/2012



TYPICAL SECTION 1 / -
SCALE: 1/2" = 1'-0"

LEGEND:

= DEMOLITION EXTENTS

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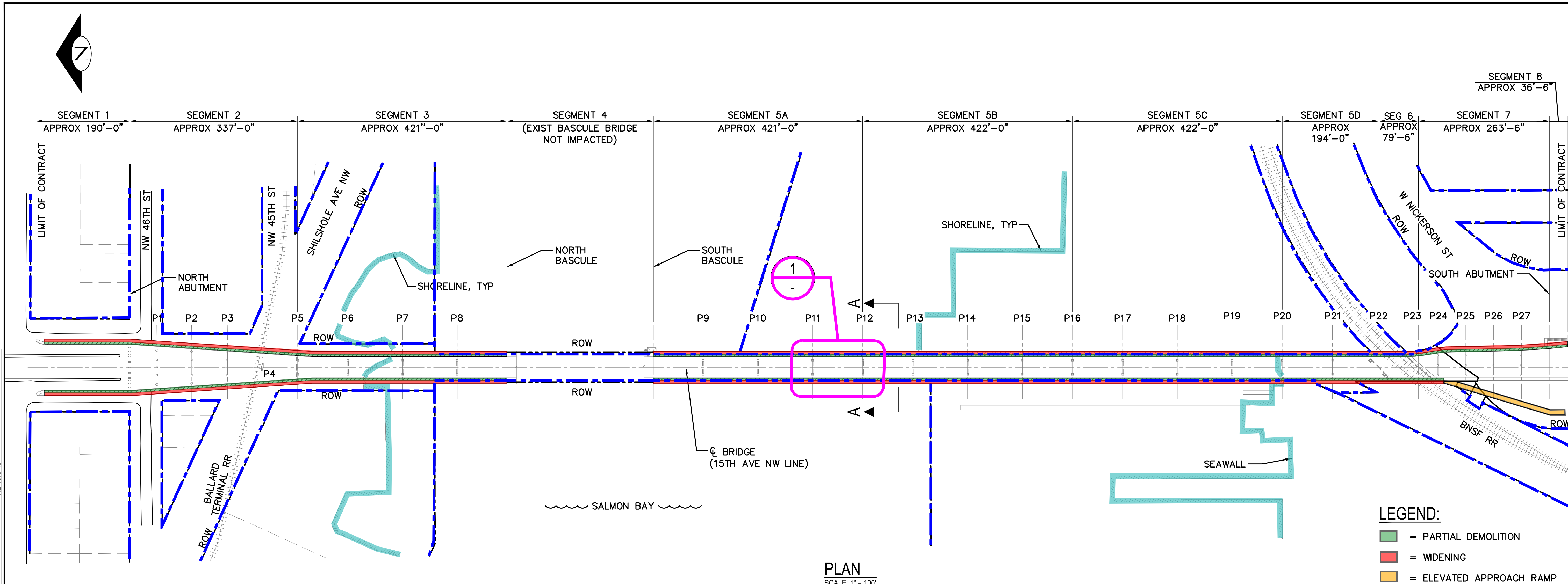
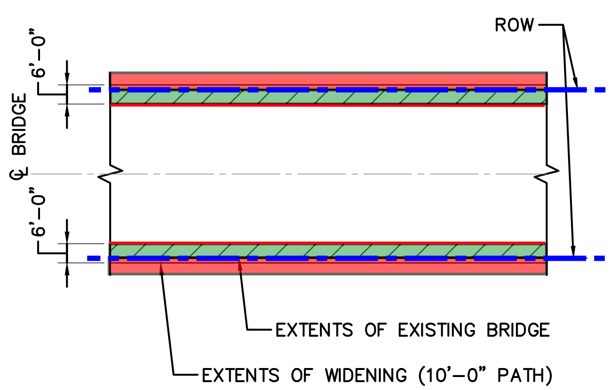
City of Seattle
Seattle Department of Transportation

BALLARD BRIDGE
10' WALKWAY
SEGMENT 8

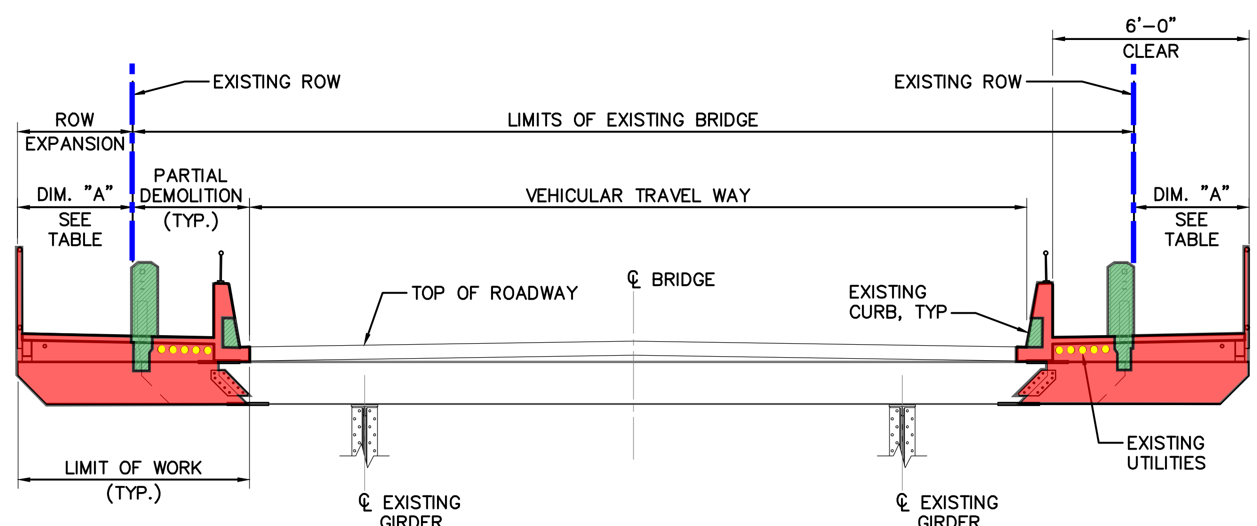
DRAWING NO.: S10.09
JOB NO.: SAPWT-10-057
DATE: 2/7/2013

Appendix C
Exhibits of Bridge and Miscellaneous Graphics





DETAIL 1 -
SCALE: NTS



SECTION A -
SCALE: NTS

SEGMENT	ESTIMATED DIMENSION "A"		APPROX LENGTH		SQUARE FEET		TOTAL SQUARE FEET *
	TYPICAL	@ LIGHT POLE	WEST	EAST	WEST	EAST	
1	1'-10"	3'-3"	190'-0"	190'-0"	0	0	0
2	1'-10"	3'-3"	337'-0"	337'-0"	0	0	0
3	1'-10 1/2"	3'-3"	421'-0"	421'-0"	470	470	940
4	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING
5A	1'-10 1/2"	3'-3"	421'-0"	421'-0"	1,370	1,370	2,740
5B	1'-10 1/2"	3'-3"	422'-0"	422'-0"	1,375	1,375	2,750
5C	1'-10 1/2"	3'-3"	422'-0"	422'-0"	1,375	1,375	2,750
5D	1'-10 1/2"	3'-3"	194'-0"	194'-0"	630	630	1,260
6	2'-1 1/2"	3'-5 1/2"	79'-6"	79'-6"	275	275	550
7	2'-2 1/2"	3'-7"	51'-0"	263'-6"	0.00	0.00	0.00
8	2'-2 1/2"	3'-7"	0'-0"	36'-6"	0.00	0.00	0.00

* AREAS ARE APPROXIMATE AND REPORTED FOR ESTIMATING PURPOSES ONLY.

KEY PLAN

MARK	DATE	NATURE REVISIONS	MADE	CHK'D	REV'D

VAULT SERIAL NO. 35370

Last Saved by: Childress on: Feb 12, 2013 8:40 AM File: Q:\Seattle\2010\SAPWT-10-057\Engineering\Bridges\Sidewalk Widening\Task 15 - Report\Widening Exhibit\Sidewalk 6ft widening ROW with segment 1 and 8.dwg

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BY: . . . .
PURCHASING AND CONTRACTING SERVICES DIRECTOR

NAME OR INITIALS AND DATE
DESIGNED . . . GAB . . . .
CHECKED . . . .
DRAWN . . . . LWC . . . .
CHECKED . . . GAB . . . .

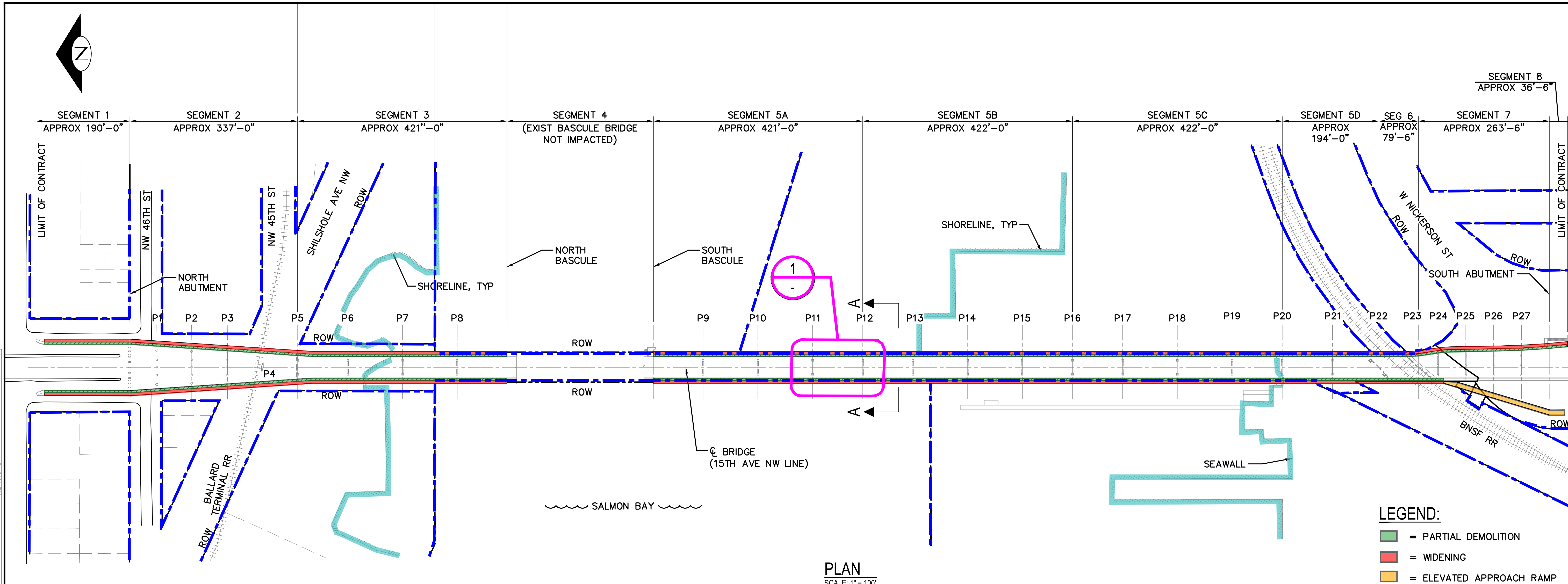
INITIALS AND DATE
REVIEWED:
PE . . . . CONST. . . . . .
PROJ. MGR. . . . .
RECEIVED . . . .
REVISED AS BUILT . . . .

ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

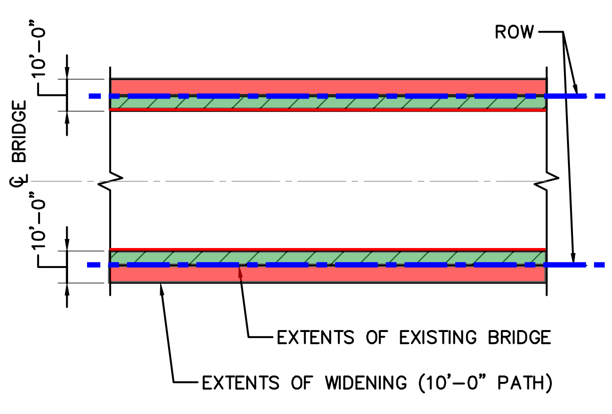
City of Seattle
Seattle Department of Transportation
ORDINANCE NO. 123177 . . . . APPROVED . . . .
FUND: BRIDGE THE GAP . . . .
INSPECTOR'S BOOK . . . . . .

BALLARD BRIDGE
6FT SIDEWALK WIDENING
DRAFT ROW EXHIBIT

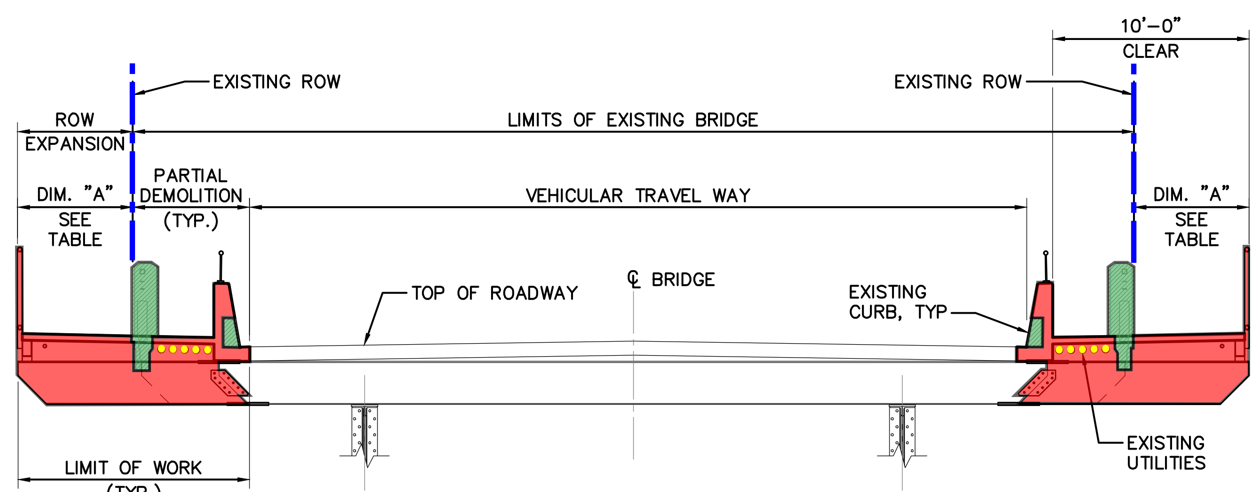
JOB NO. PC R/W CO
VAULT PLAN NO. 744-705
SHEET OF



PLAN
SCALE: 1" = 100'



DETAIL 1
SCALE: NTS



SECTION A
SCALE: NTS

SEGMENT	ESTIMATED DIMENSION "A"		APPROX LENGTH		SQUARE FEET		TOTAL SQUARE FEET *
	TYPICAL	@ LIGHT POLE	WEST	EAST	WEST	EAST	
1	5'–10"	7'–7"	190'–0"	190'–0"	0	0	0
2	5'–10"	7'–7"	337'–0"	337'–0"	0	0	0
3	5'–8 1/2"	7'–1"	421'–0"	421'–0"	1,030	1,030	2,060
4	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING	NO WIDENING
5A	5'–8 1/2"	7'–1"	421'–0"	421'–0"	2,980	2,980	5960
5B	5'–8 1/2"	7'–1"	422'–0"	422'–0"	2,990	2,990	5,980
5C	5'–8 1/2"	7'–1"	422'–0"	422'–0"	2,990	2,990	5,980
5D	5'–8 1/2"	7'–1"	194'–0"	194'–0"	1,375	1,375	2,750
6	5'–11 1/2"	7'–8 1/2"	79'–6"	79'–6"	615	615	1,230
7	6'–0 1/2"	7'–9 1/2"	51'–0"	263'–6"	0.00	0.00	0.00
8	6'–0 1/2"	7'–9 1/2"	0'–0"	36'–6"	0.00	0.00	0.00

* AREAS ARE APPROXIMATE AND REPORTED FOR ESTIMATING PURPOSES ONLY.

KEY PLAN

VAULT SERIAL NO. 35370 | DATE | MARK | NATURE REVISIONS | MADE | CHK'D | REV'D

Last Saved by: Childress on: Feb 12, 2013 8:43 AM File: Q:\Seattle\2010\SAPWT-10-057\Engineering\Bridges\Sidewalk Widening\Task 15 - Report\Widening Exhibit\Sidewalk 10ft widening ROW with segment 1 and 8.dwg

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DESIGNED . . . GAB . . . .
CHECKED . . . .
DRAWN . . . . LWC . . . .
CHECKED . . . . GAB . . . .

INITIALS AND DATE
REVIEWED:
PE . . . . CONST. . . . . .
PROJ. MGR. . . . .
RECEIVED . . . .
REVISED AS BUILT . . . .

ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0–02.3 OF THE PROJECT MANUAL.

City of Seattle
Seattle Department of Transportation
ORDINANCE NO. 123177 . . . . APPROVED . . . .
FUND: BRIDGE THE GAP . . . .
INSPECTOR'S BOOK . . . . . .

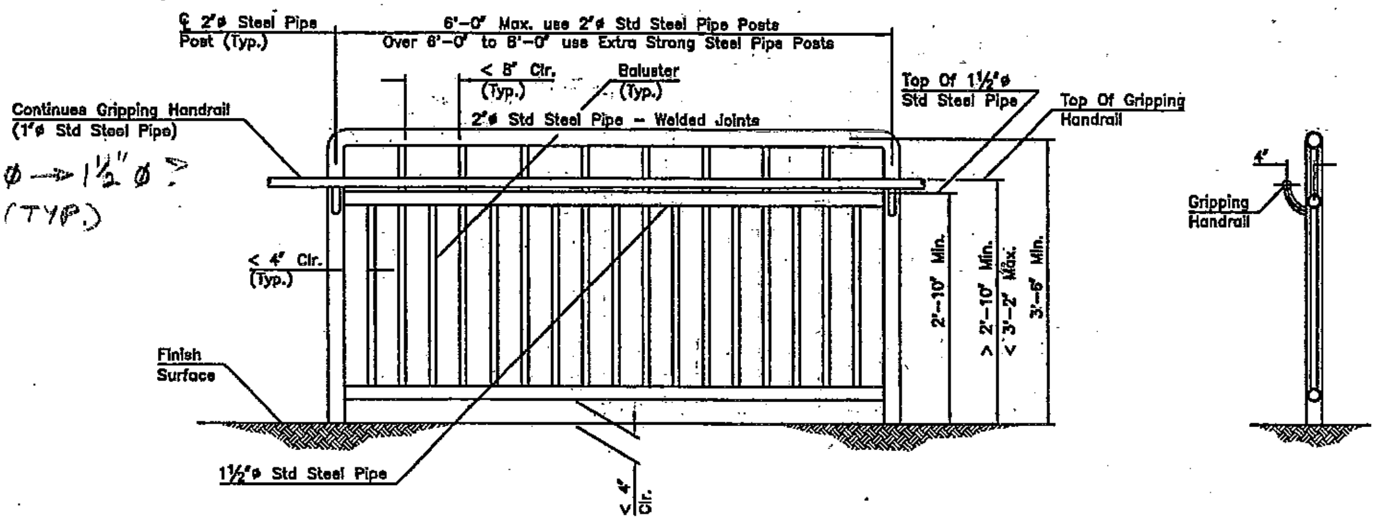
BALLARD BRIDGE
10FT SIDEWALK WIDENING
DRAFT ROW EXHIBIT

JOB NO. PC R/W CO
VAULT PLAN NO. 744-705
SHEET OF

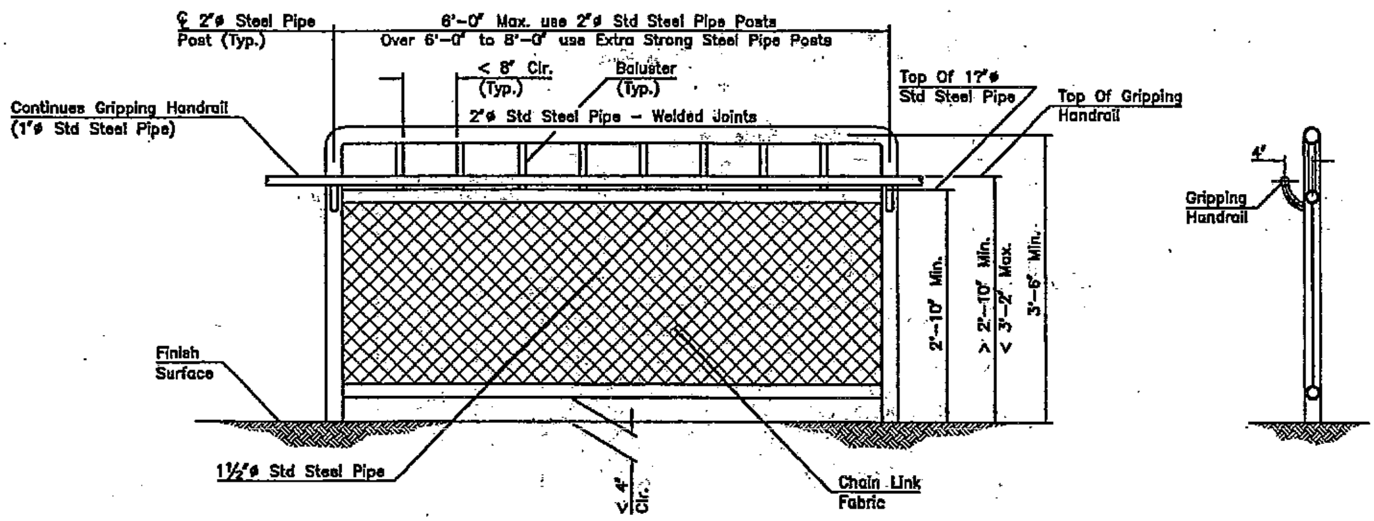
CITY OF SEATTLE STANDARD PLAN
(Proposal for Steel Pipe Handrail)
– According to 2000 IBC Codes –

PEDESTRIAN RAILING:
(See Standard Plan No.442)

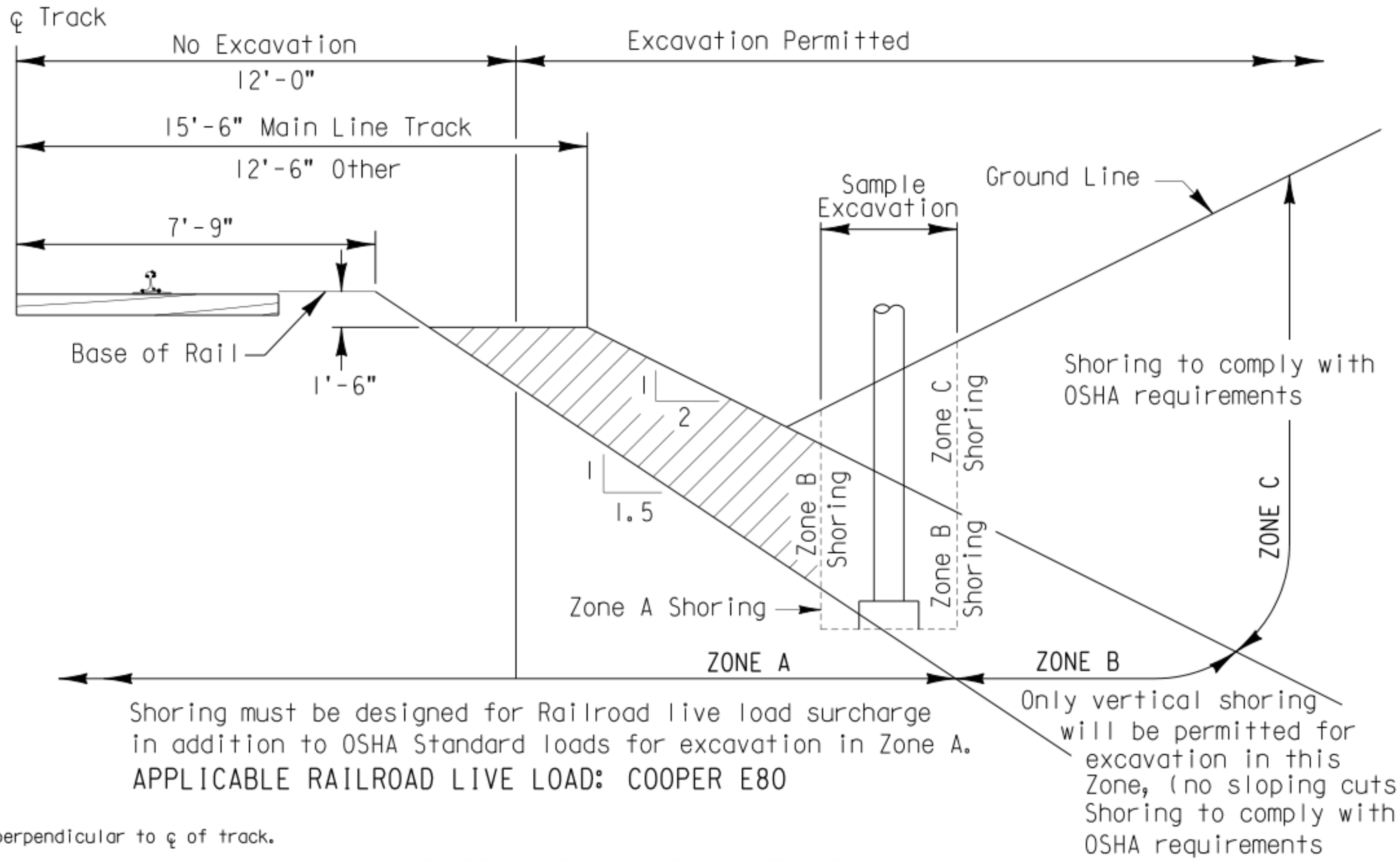
PEDESTRIAN GUARD RAILING:



OR



P:\Struct\Temp\Handrail Drawings\CityofSeattle.dwg
6-21-04



GENERAL EXCAVATION ZONES

SCALE: (NOT TO SCALE)

GENERAL NOTES:

All dimensions are measured perpendicular to ℄ of track.

Prior to commencing any work, the contractor shall submit for approval by the Railroad detailed plans indicating the nature and extent of the track protection shoring proposed. The contractor shall install the temporary shoring system per the approved plans. Design of the temporary shoring system to comply with **GUIDELINES FOR TEMPORARY SHORING.**

For excavations which encroach into zone A or B, shoring plans shall be accompanied by design calculations. Plans and calculations must be signed and stamped by a Professional Engineer registered in the state where the work will be performed.

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DESIGN BY: PGP | DRAWN BY: JFS | CHECKED BY: AA

APPROVED:

K.H. Jennison
BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

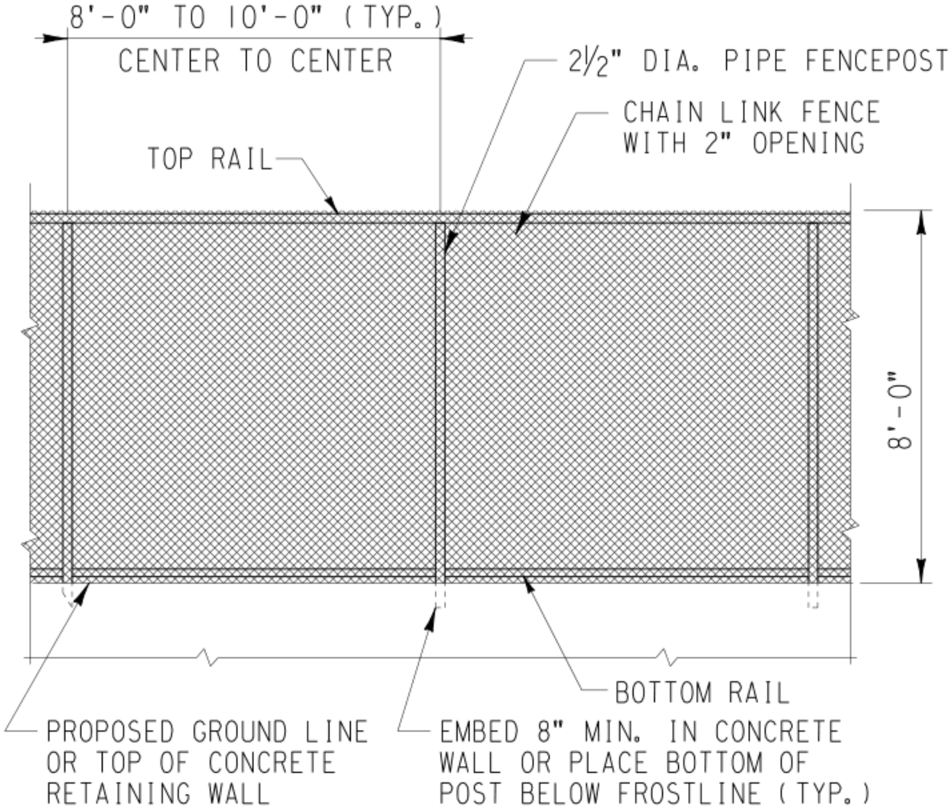
George J. Meyer 9-1-04
UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BRIDGE STANDARDS

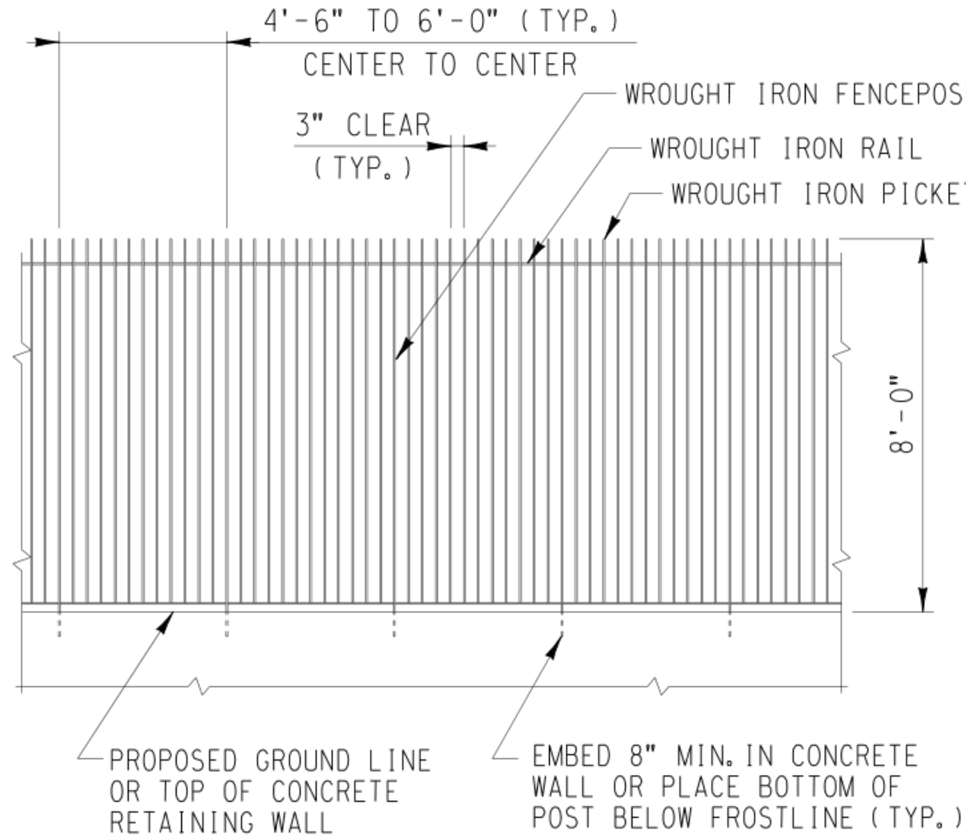
GENERAL SHORING REQUIREMENTS

FILE OWNER: UPRR | DATE: 5-6-03
PLAN NO.: 710000 1 | SHEET: 1 OF 1

PLOTTED: $$$$DATE$$$ $TIME



CHAIN LINK FENCE



WROUGHT IRON PICKET FENCE

FENCE ELEVATION

SCALE: 3/16" = 1'-0"

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DESIGN BY: RAF | DRAWN BY: FJS | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY — UNION PACIFIC

BRIDGE STANDARDS

RIGHT-OF-WAY FENCING

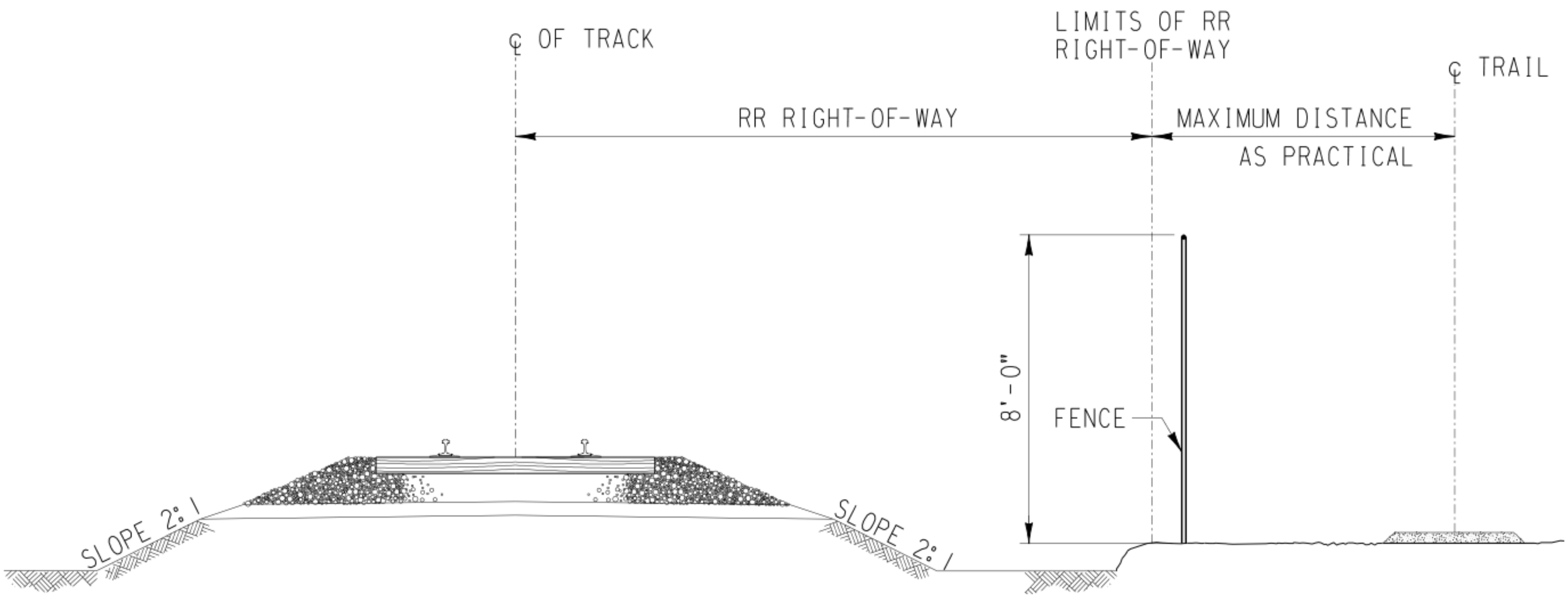
FENCE DETAILS

FILE OWNER: UPRR | DATE: 1/24/07

PLAN NO.: 711000 | SHEET: 1

PLOTTED: 3/30/2007 2:53:20 PM

FILE NAME: P:\ustation\dgn\std\pp_guidelines.dgn



TYPICAL SECTION WITH STANDARD FLAT BOTTOM DITCH

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DESIGN BY: RAF | DRAWN BY: FJS | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

UNION PACIFIC

BRIDGE STANDARDS

RIGHT-OF-WAY FENCING

FENCE REQUIREMENTS FOR ADJACENT TRAIL OUTSIDE RAILROAD RIGHT-OF-WAY

FILE OWNER: UPRR | DATE: 1/24/07

PLAN NO.: 711000 | SHEET: 2

PLOTTED: 3/30/2007 2:53:35 PM

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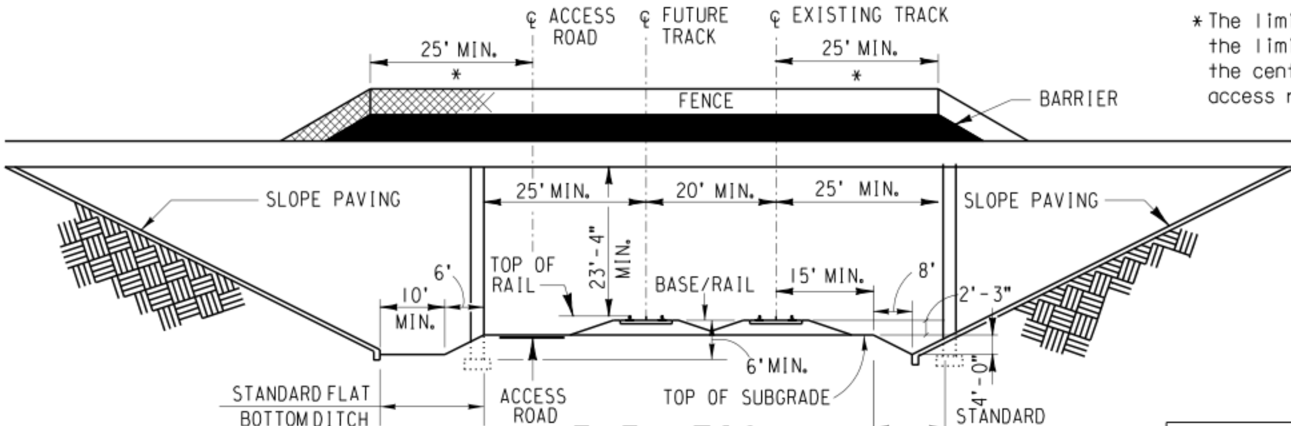
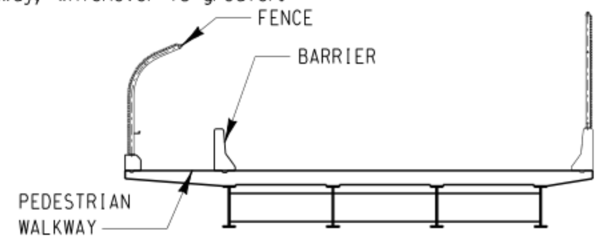


ELEVATION

PERPENDICULAR TO TRACKS

* The limits of the fence with barrier rail shall extend to the limits of Railroad Right-of-Way or a minimum of 25 ft. beyond the centerline of the outermost existing track, future track or access roadway, whichever is greater.



OVERHEAD STRUCTURE CROSS SECTION WITH FENCE

NOTE: WIDTH AND HEIGHT SUBJECT TO HYDRAULIC REQUIREMENTS. SEE PLAN NO. 711100, SHEET 5.

GENERAL

Fence shall be provided as indicated on the cross sections and elevation view on both sides of the Overhead Structure in ALL new or modified structures.

Barrier rail for Overhead Structures, without walkways, that may be subject to snow removal shall be a minimum of 42 inches in height with a 4 foot wide shoulder or 30 inches in height with a 6 foot wide shoulder. See Plan No. 711100, Sheet 4.

Lights are to be installed on the underside of the Overhead Structure where shadows cast by the structure would interfere with Railroad operations.

Slope paving shall be provided where end slopes are equal to or exceed 2 horizontal to 1 vertical.

Falsework for construction of overhead structures shall comply with Railroad Requirements.

Demolition of existing Overhead Structures shall comply with Railroad Demolition Requirements. Temporary shoring shall be designed in accordance with Railroad Guidelines for Temporary Shoring.

Applicant shall be responsible for identification, location and protection of existing utilities.

Call the following numbers at least 48 hours prior to commencing work to determine location of fiber optics:
UPRR "Call Before You Dig", 1-800-336-9193
BNSF "Call Before You Dig", 1-800-533-2891.

CLEARANCES

Minimum vertical clearance shall be 23'-4" above the top of high rail within 25' of centerline of track. Additional clearance may be required for construction purposes or if sag of vertical curve must be adjusted or if future track raise for flood considerations or maintenance is probable.

Minimum horizontal clearances, measured at right angle from centerline of track, shall be as shown in elevation view.

For minimum construction clearances, see Plan No. 711100, Sheet 3.

PIERS

Piers shall be located outside Railroad Right-of-Way.

Pier protection walls shall be provided in accordance with AREMA Chapter 8, Part 2.1.5 for piers within 25 feet of the centerline of track.

Top of footings located within 25 feet from centerline of track shall be a minimum of 6 feet below base of rail and a minimum of 1 foot below flowline of ditch.

DRAINAGE

Drainage from the Overhead Structure shall be diverted away from and not discharged onto the tracks, roadbed and Railroad Right-of-Way.

At minimum, a standard "V"-shaped or flat-bottom ditch shall be provided on each side of the tracks as necessary.

Culverts may be installed in lieu of standard Railroad ditches when approved by Railroad Central Engineering. Maintenance of culverts will be at Applicant's expense.

FUTURE TRACKS AND ACCESS ROAD

Space is to be provided for one or more future tracks as required for long range planning or other operating requirements. Where provision is made for more than two tracks, space is to be provided for an access road on both sides of tracks.

REVISIONS		
DATE	LTR.	DESCRIPTION
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DESIGN BY: RAF | DRAWN BY: KDM | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY | UNION PACIFIC

BRIDGE STANDARDS

GRADE SEPARATION GUIDELINES (OVERHEADS)

GENERAL OVERHEAD STRUCTURE DRAWING

FILE OWNER: UPRR	DATE: 1/24/07
PLAN NO.: 711100	SHEET: 1

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PLAN

1. North Arrow
2. Centerline of bridge and/or centerline of project.
3. Track layout and limits of Railroad right-of-way with respect to centerline of main lines.
4. Footprint of proposed superstructure and substructure including existing structure if applicable.
5. Show and label future tracks, access roadways and existing tracks as main line, siding, spur, etc.
6. Indicate point of minumum vertical clearance and distance, measured perpendicular, from the centerline of nearest track.
7. Horizontal clearance at right angle from centerline of nearest existing or future track to the face of obstruction such as substructure above grade.
8. Horizontal clearance at right angle from centerline of nearest existing or future track to the face of nearest foundation below grade.
9. Indicate horizontal spacing at right angle between centerlines of existing and/or future tracks.
10. Limits of shoring and minimum distance at right angle from centerline of nearest track.
11. Locate and show all existing facilities and utilities and their proposed relocation, if required.
12. Toe of slope and/or limits of retaining wall.
13. Existing and proposed contours.
14. Limits of barrier rail and fence.
15. Indicate minimum structure separation for adjacent structures.
16. Indicate Railroad Milepost and direction of increasing Milepost.
17. Direction of flow for all drainage systems within project limits.
18. Timetable direction arrows, nearest Railroad station and end station of Railroad Subdivision.

ELEVATION

1. Individual span length and total bridge length.
2. Limits of barrier rail and fence with respect to centerline of track.
3. Depth of foundation below bottom of tie.
4. Horizontal clearance at right angle from centerline of nearest existing or future track to the face of obstruction such as substructure above grade.
5. Indicate horizontal spacing at right angle between centerlines of existing and/or future tracks.
6. Minimum horizontal clearance at right angle from centerline of nearest existing or future track to the face of foundation below grade.
7. Indicate top and bottom of pier protection wall elevation relative to top of rail elevation.
8. Controlling dimensions of drainage ditches and/or drainage structures.
9. Top of rail elevations for all tracks.
10. Minimum permanent vertical clearance above top of high rail to the lowest point under the bridge.
11. Existing and proposed goundline & roadway profile.
12. Show elevation of existing or relocated utilities.
13. Show slope and specify type of slope paving. Toe of slope shall be shown relative to drainage ditch and top of subgrade.

14. Show and label future tracks, access roads and existing tracks as main line, siding spur, etc.
15. Show location of deck joints.
16. Location of deck drains.

TYPICAL SECTION

1. Total width of superstructure.
2. Width of shoulder and/or sidewalk.
3. Type of barrier rail, fence and their heights.
4. Depth of superstructure.

TITLE BLOCK

1. The name & logo of engineering firm or project owner.
2. Drawing title.
3. Railroad milepost number and subdivision.
4. City, county and state.
5. Project name and location.
6. Date.
7. Latitude and longitude.

RAILROAD PROFILE GRADE DIAGRAM

1. Show existing and proposed track profile at the bridge location and a minimum of 1,000 feet past each edge of the bridge.

Note: The Railroad Milepost is calculated at the intersection of centerlines of the Overhead Structure and Existing Track. All separate Overhead Structures shall have individual Milepost designations.

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DESIGN BY: RAF | DRAWN BY: MMD | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY

UNION PACIFIC

BRIDGE STANDARDS

GRADE SEPARATION GUIDELINES (OVERHEADS)

MINIMUM LAYOUT REQUIREMENTS FOR OVERHEAD STRUCTURES

FILE OWNER: UPRR | DATE: 1/24/07

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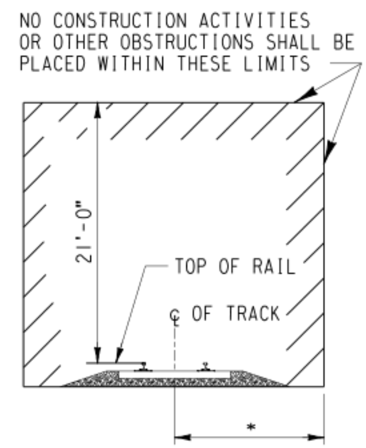
CONSTRUCTION NOTES:

1. Any shoring system that impacts the Railroad's operation and/or supports the Railroad's embankment shall be designed and constructed per Railroad Guidelines for Temporary Shoring.
2. All demolition within the Railroad's right-of-way and/or demolition that may impact the Railroad's tracks or operations shall comply with the Railroad's Demolition requirements.
3. Erection over the Railroad's track shall be planned such that it enables the track(s) to remain open to traffic per Railroad requirements.
4. The elevation of the existing top-of-rail profile shall be verified before beginning construction. All discrepancies shall be brought to the attention of the Railroad prior to construction.
5. The proposed grade separation project shall not change the quantity and/or characteristics of the flow in the Railroad ditches and/or drainage structures.
6. The contractor must submit a proposed method of erosion and sediment control and have the method approved by the Railroad prior to beginning any grading on the project site.
7. For Railroad coordination please refer to the Railroad's Coordination Requirements as part of the Specifications or Special Provisions of the project.
8. Temporary Construction Clearances, including falsework clearances, shall comply with Figure I.
9. All permanent clearances shall be verified before project closeout.



MINIMUM CONSTRUCTION CLEARANCE ENVELOPE

(NORMAL TO RAILROAD)

FIGURE I

* 15'-0" for BNSF and 12'-0" for UPRR

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DESIGN BY: RAF | DRAWN BY: KDM | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

BNSF - ASSISTANT DIRECTOR STRUCTURES DESIGN

George J. Meyn

UPRR - MGR SPECIAL PROJECTS STRUCTURES DESIGN

BNSF RAILWAY

UNION PACIFIC

BRIDGE STANDARDS

GRADE SEPARATION GUIDELINES (OVERHEADS)

CONSTRUCTION NOTES

FILE OWNER: UPRR | DATE: 1/24/07

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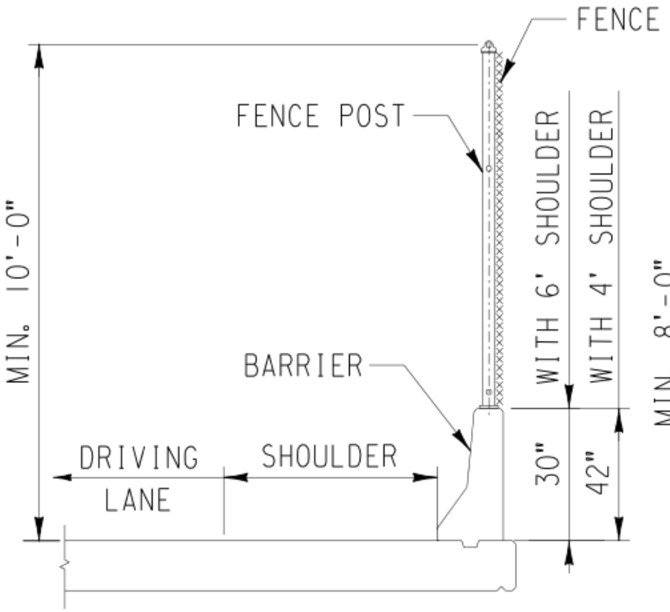
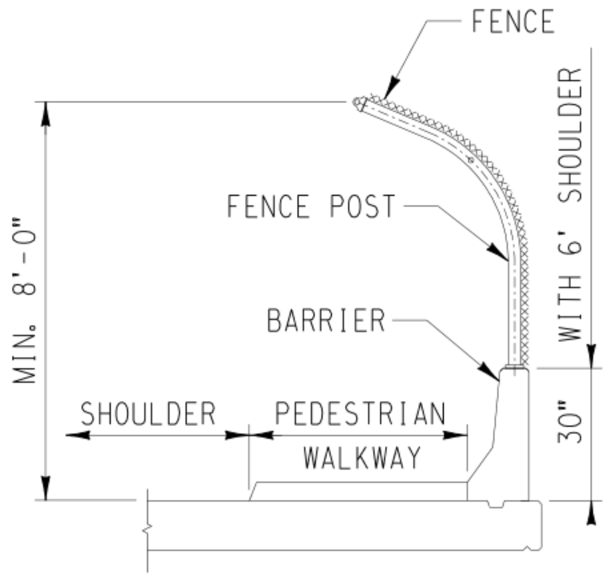
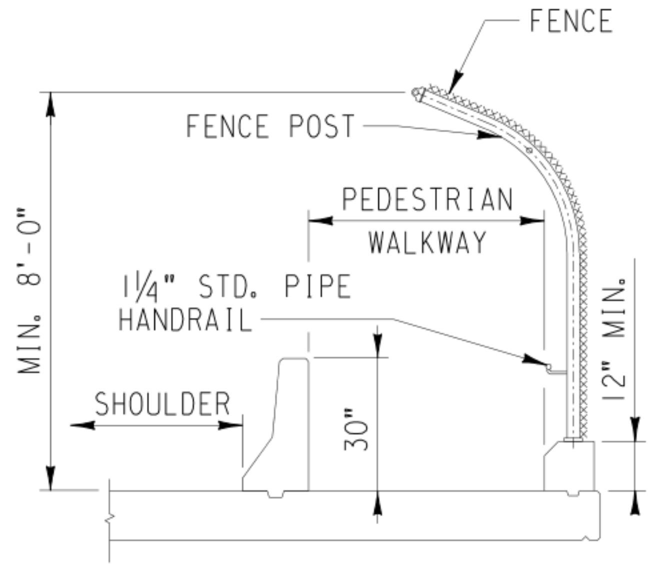


FENCE ON BARRIER



WALKWAY FENCE ON BARRIER



WALKWAY FENCE ON CURB WITH BARRIER

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DESIGN BY: RAF | DRAWN BY: MMD | CHECKED BY: KHJ

APPROVED:

K.H. Jennison

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George J. Meyn

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BNSF RAILWAY — UNION PACIFIC

BRIDGE STANDARDS

GRADE SEPARATION GUIDELINES (OVERHEADS)

OVERHEAD STRUCTURE BARRIERS AND FENCES

FILE OWNER: UPRR | DATE: 1/24/07

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Appendix D
Photos and Field Measurements

Appendix Narrative

Introduction

The following appendix summarizes the data gathered on a site visit made on 17 July 2012. There were two primary objectives for this site visit. Firstly, as-built dimensions at “choke-points” were measured for comparison with extant engineering drawings. Secondly, photographs of the pedestrian paths along the east and west sidewalks, as well as the pedestrian underpass and trail area at the south end were collected for future reference.

The contents of this section can be divided into three components. First is a key plan of the bridge noting the approximate locations and directions of view of each photos Second are the photographs, and third is a single sketch showing the typically measurements recorded at critical points along the bridge.

Summary of Findings

Prior to visiting the site, City of Seattle was to provide us with the majority of extant drawings for the entire bridge, from Section 1 (north) to Section 8 (south). We noted that the critical “choke points” would most likely be at locations such as the luminaire poles, intermediate posts, approach span railing, and bascule span transition areas. From the information in hand, we decided to measure any other anomalies encountered in the field as well as verify the acquired information from the City.

We took a number of measurements at the locations noted above all along the bridge, and additionally at points where the railing/sidewalk appeared to taper or change direction. What we discovered was that the sidewalks appear to generally be in conformance with the information we had previously obtained, and that the dimensions were fairly consistent along the length of the bridge.

With the exception of the south east transition area of the bascule span, the sidewalk at section 4 was generally wider than the other sections. At all the other sections, both east and west, the cross-sections at the luminaire poles were the most spatially constricted. Please refer to the sketch at the end of this section for specific dimensions. These dimensions are approximate, average and were not collected for every single instance.

BergerABAM

Project: Ballard Bridge

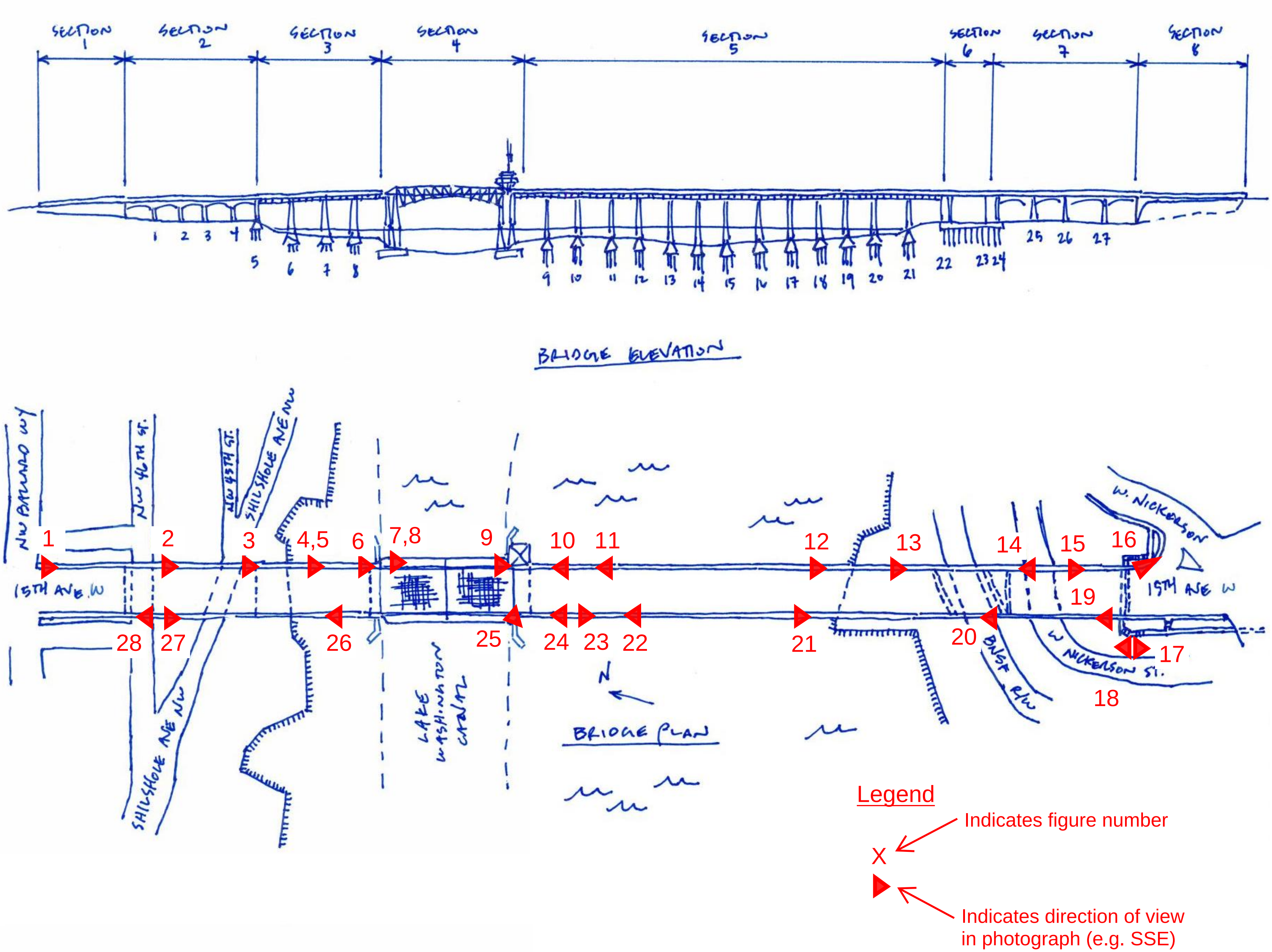
Subject: Bridge Overview

Sheet ___ of ___

Job Number: SA1wt-10-097

Designer: BGF

Date: 18 July 2012



Photographs

Fig. 1 – NE end of Section 1

Fig. 2 – NE Section 2 over NW 46th St.

Fig. 3 – NE Section 2 over Shilshole Ave.

Fig. 4 – NE Section 3, north of Lk. Wa Channel

Fig. 5 – SE end of Section 3, prior to bascule span (Section 4)

Fig. 6 – End of Section 3, near guard rail

Fig. 7 – NE transition to bascule span (Section 4)

Fig. 8 - NE transition to bascule span (Section 4)

Fig. 9 – SE transition at bascule span (Section 4)

Fig. 10 - SE transition to bascule span (Section 4)

Fig. 11 – NE end of Section 5

Fig. 12 – SE end of Section 5

Fig. 13 – View towards E side of Section 6

Fig. 14 – Bicyclists passing at east side of Section 7

Fig. 15 - Southerly view of Section 7 east sidewalk

Fig. 16 – View of Section 8 east sidewalk to pedestrian underpass

Fig. 17 – View of pedestrian underpass to west sidewalk at Section 8

Fig. 18 – View of SW side of Section 8 from proposed pedestrian path

Fig. 19 – Northward view at Section 8 SW sidewalk

Fig. 20 – Section 7 sidewalk

Fig. 21 – Southward view from Section 5 towards Section 6, 7 and 8

Fig. 22 – Northward view at Section 5

Fig. 23 – Southward view of bicyclist at Section 5 guardrail

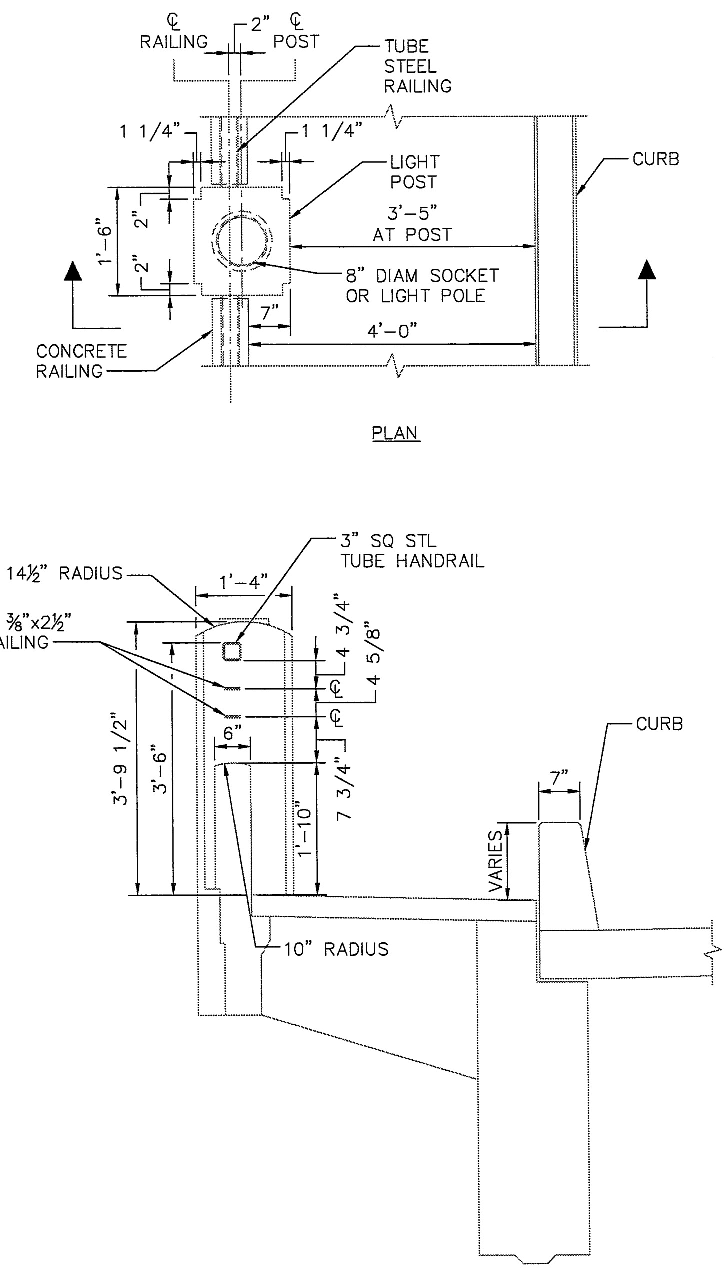
Fig. 24 – Northward view at Section 5 guardrail

Fig. 25 – SW transition at bascule span (Section 4)

Fig. 26 – Northerly view of Section 3 and 2 at west sidewalk

Fig. 27 – Southward wiew of Section 2 west over NW 46th St.

Fig. 28 – Northward view of Section 2 west over NW 46th St.

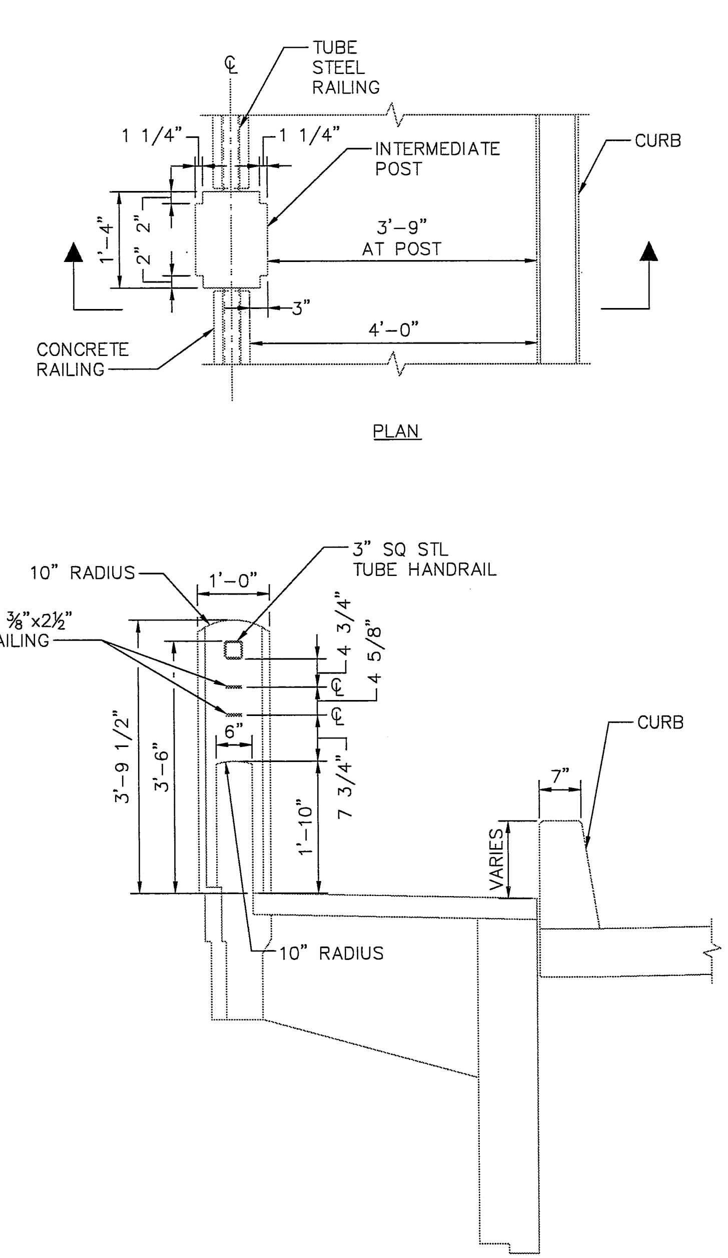


PLAN

SECTION

LIGHT POST (AS-BUILT)

SCALE: 3/4" = 1'-0"

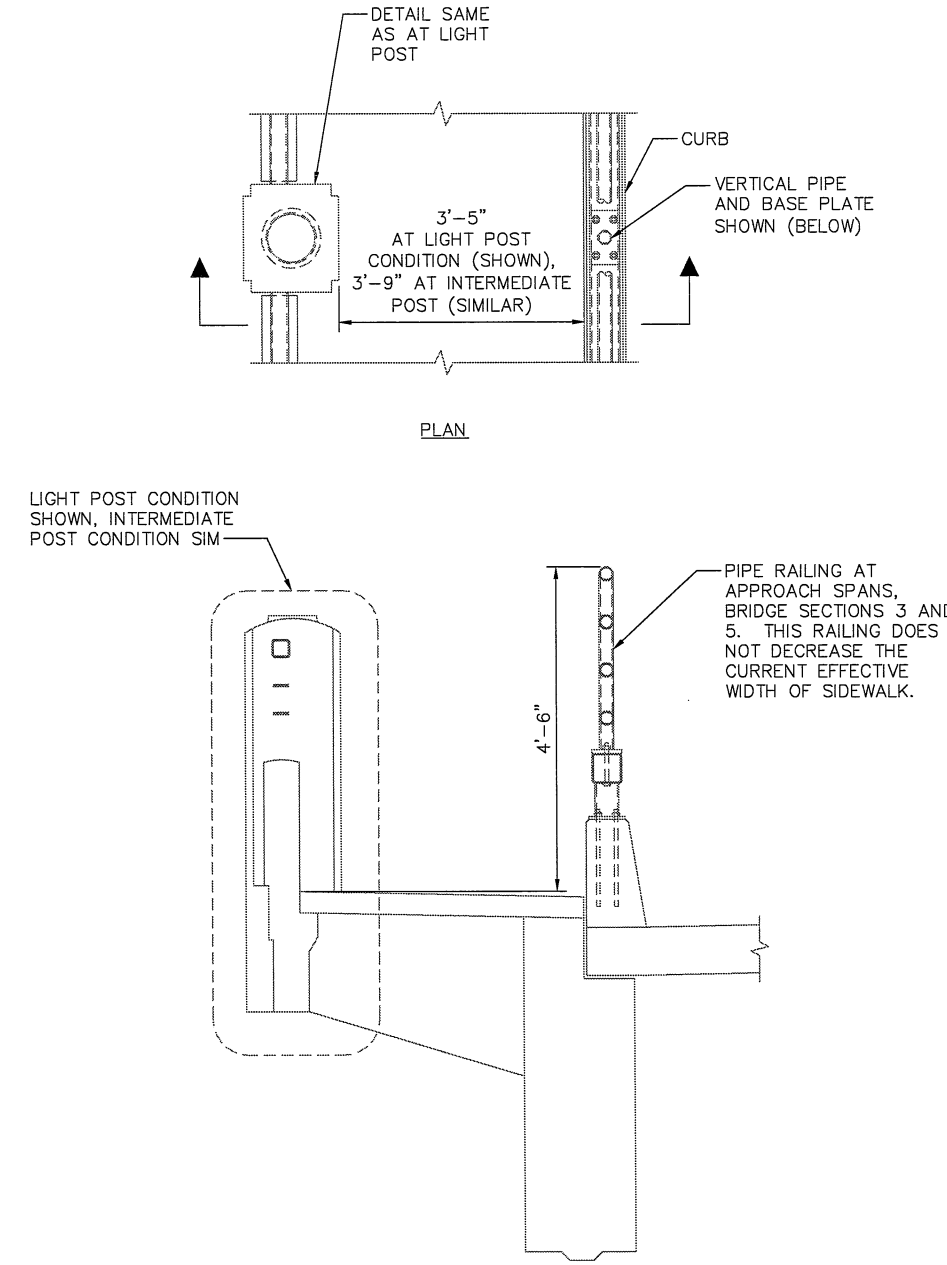


PLAN

SECTION

INTERMEDIATE POST (AS-BUILT)

SCALE: 3/4" = 1'-0"



PLAN

SECTION

APPROACH SPAN RAILING (AS-BUILT)

SCALE: 3/4" = 1'-0"

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHK'D	REV'D

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ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

City of Seattle
Seattle Department of Transportation
ORDINANCE NO. . . . . APPROVED. . . . .
FUND: . . . .
INSPECTOR'S BOOK. . . . .

BALLARD BRIDGE
SIDEWALK SHARED PATH STUDY
TASK 15

JOB NO. PC R/W CO
VAULT PLAN NO.
SHEET OF

Last Saved by: Ben.fisher on: Jul 19, 2012 3:36 PM File: Q:\Seattle\2010\SAPWT-10-057\CADD\Design\BSP\Existing Cross-Sections.dwg

Ballard Bridge Sidewalk Widening Alternative Study
Seattle Department of Transportation, Seattle, Washington

Appendix E
Stormwater Analysis

PRELIMINARY STORMWATER TECHNICAL MEMORANDUM

BALLARD BRIDGE SIDEWALK WIDENING CONCEPT STUDY

Seattle Department of Transportation

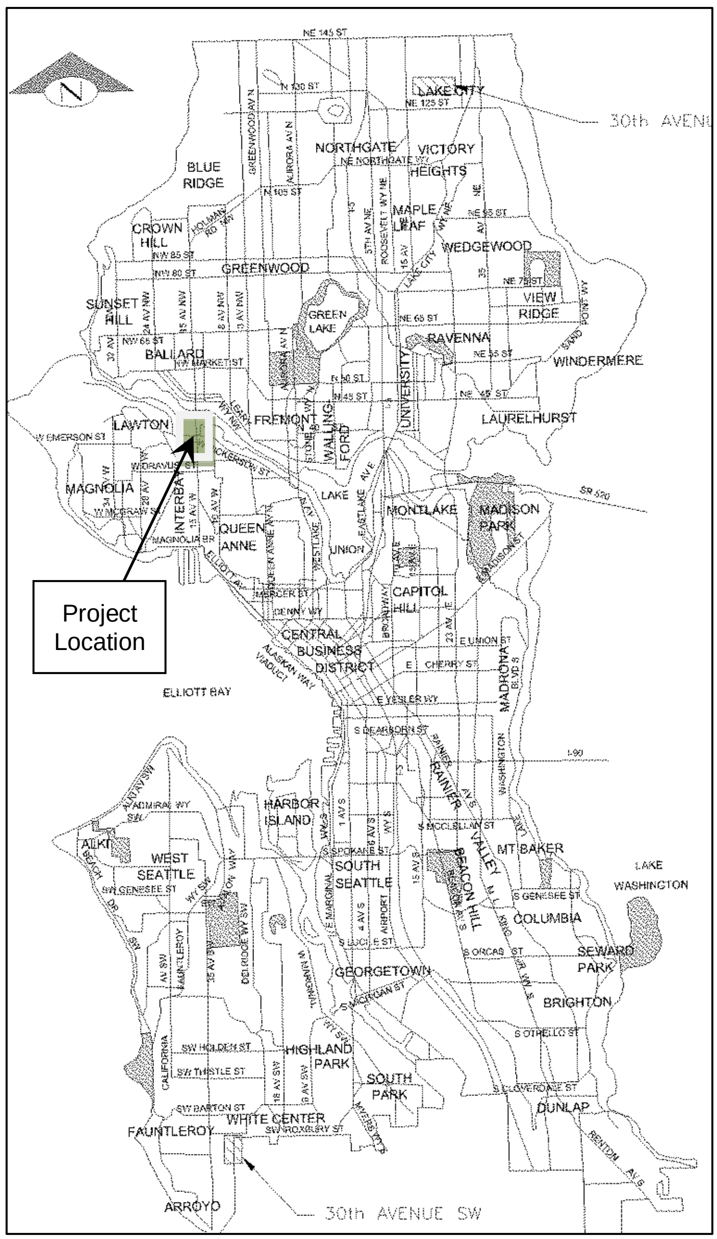
May 6, 2013

Prepared By: Russell D. Bauder, PE

1.0 STUDY DESCRIPTION

The Ballard Bridge Sidewalk Widening Concept Study being prepared by Berger/ABAM, Inc., analyzes the options for improving the pedestrian facilities of the Ballard Bridge, or 15th Avenue NW Bridge as it is also known. The bridge crosses Salmon Bay, which is part of the Lake Washington Ship Canal. There are four lanes of vehicular traffic and two pedestrian walkways, one on each side of the bridge.

There are a number of concepts currently under consideration. This report documents the potential City of Seattle Stormwater Code Requirements of the 6' and 10' sidewalk options. The 6' concept will remove the existing sidewalks and construct a 6' wide sidewalk on both sides of the bridge, not including the bascule portion of the bridge. The largest concept will remove the existing sidewalk and construct 10' sidewalks on both sides of the bridge, except for the bascule portion.



Both concepts include the partial demolition of the bridge deck and removal of the traffic curbs, sidewalks and bridge rails. The proposed work for both concepts includes bridge deck widening, traffic barriers, sidewalks, and new bridge rails.

In addition, the concepts include a new trail connection at the south end of the bridge. The connection will provide a pedestrian/bike link to the Ship Canal Trail.

2.0 BASIN DELINEATION

The bridge is located in the Lake Union/Ship Canal drainage basin as determined from the available City GIS data, survey, side sewer cards, and record documents from the City's Record Vault.

The runoff from the sidewalk and roadway on the bridge is collected in bridge drains located at the face of the existing traffic curbs. The collected runoff discharges from stand pipes attached to the bridge columns. Most of the water discharges directly to Salmon Bay. The portion of the northern approach from NW Shilshole Avenue to NW Ballard Way is collected in a storm sewer and discharged to Salmon Bay at 20th Avenue NW. A portion of the southern approach between Nickerson Street & Emerson Street discharges to the Metro/King County Sewer Treatment Plant at West Point. There are aerial pictures of the bridge and neighboring sewer systems at the end of this report.

3.0 MINIMUM REQUIREMENTS

Per the 2009 Stormwater Code, the conceptual improvements would be classified as a sidewalk/trail project. The new and replaced non-pollution generating areas of the bridge widening and the southern trail connection are presented in Table 1.

BRIDGE OPTION	NORTH END	SOUTH END	TOTAL AREA
6' Sidewalk	12,000 SF	35,000 SF	47,000 SF
10' Sidewalk	19,000 SF	47,000 SF	66,000 SF

Table 1 – Non–Pollution Generating Impervious Surface Areas

- Based on the requirements the Stormwater Manual, Volume 3, Section 2.4.2 - Flow Control Requirement 1, Green Stormwater Infrastructure (GSI), the concepts will be required to incorporate GSI to the maximum Extent Feasible since the total impervious area exceeds 2000 SF.
- Based on Section 2.6.4, Construction Site Stormwater Pollution Prevention Control will be required on all concepts.
- Based on Section 2.6.5, Amended Soils will be required in disturbed areas on all concepts.

3.1 Green Stormwater Infrastructure

There are a number of approved methods used to meet GSI requirements. These methods are divided into four Categories as follows:

- Runoff Reduction
- Runoff Infiltration and Reuse
- Impervious Surface Reduction
- Non-Infiltrating Retention.

A very common means of reducing runoff is the preservation and/or planting of trees. The northern approach to the bridge offers limited opportunity for tree planting. New trees may be planted around the southern bridge approach. There are at least 14 trees within the area at the time of this report. Ten new trees are assumed for this report.

Most of the bridge deck runoff is currently dispersed to Salmon Bay or the ground under the bridge. The use of point discharge dispersion, such as rock pockets, is under consideration for the areas which currently discharge under the approach spans. Sheet flow dispersion is not an option as there are no flat vegetated areas in the ROW, which are large enough to receive the runoff.

Infiltration facilities and reuse facilities are possible in the rain shaded areas under each bridge approach. The use of these facilities will need further study during design and coordination with SDOT Bridge Operations and SPU. The site soils at the north end of the bridge have an average infiltration rate. The soils near the southern end of the bridge have a high water table. If infiltration is proposed, soils tests will be performed at specific locations.

Permeable sidewalk concrete is not suitable on the bridge. The trail connection at the southern end of the bridge should be considered in future studies. Trail surface smoothness will be a concern. As will the potential for contaminated soils in the area.

Bioretention planters or cisterns may fit in the area around the southern bridge approach.

BRIDGE OPTION	NEW AND REPLACED IMPERVIOUS AREA	AREA MITIGATED BY EX. AND NEW TREES	AREA MITIGATED BY DIRECT DISCHARGE	PERCENTAGE
6’ Sidewalks	47,000 SF	900 SF	15,200 SF	34.3%
10’ Sidewalks	66,000 SF	900 SF	25,300 SF	39.7%

Table 2 – Existing Mitigation

The GSI calculations, used n the report, are attached to the end of this report.

3.2 Stormwater Pollution Control Plan

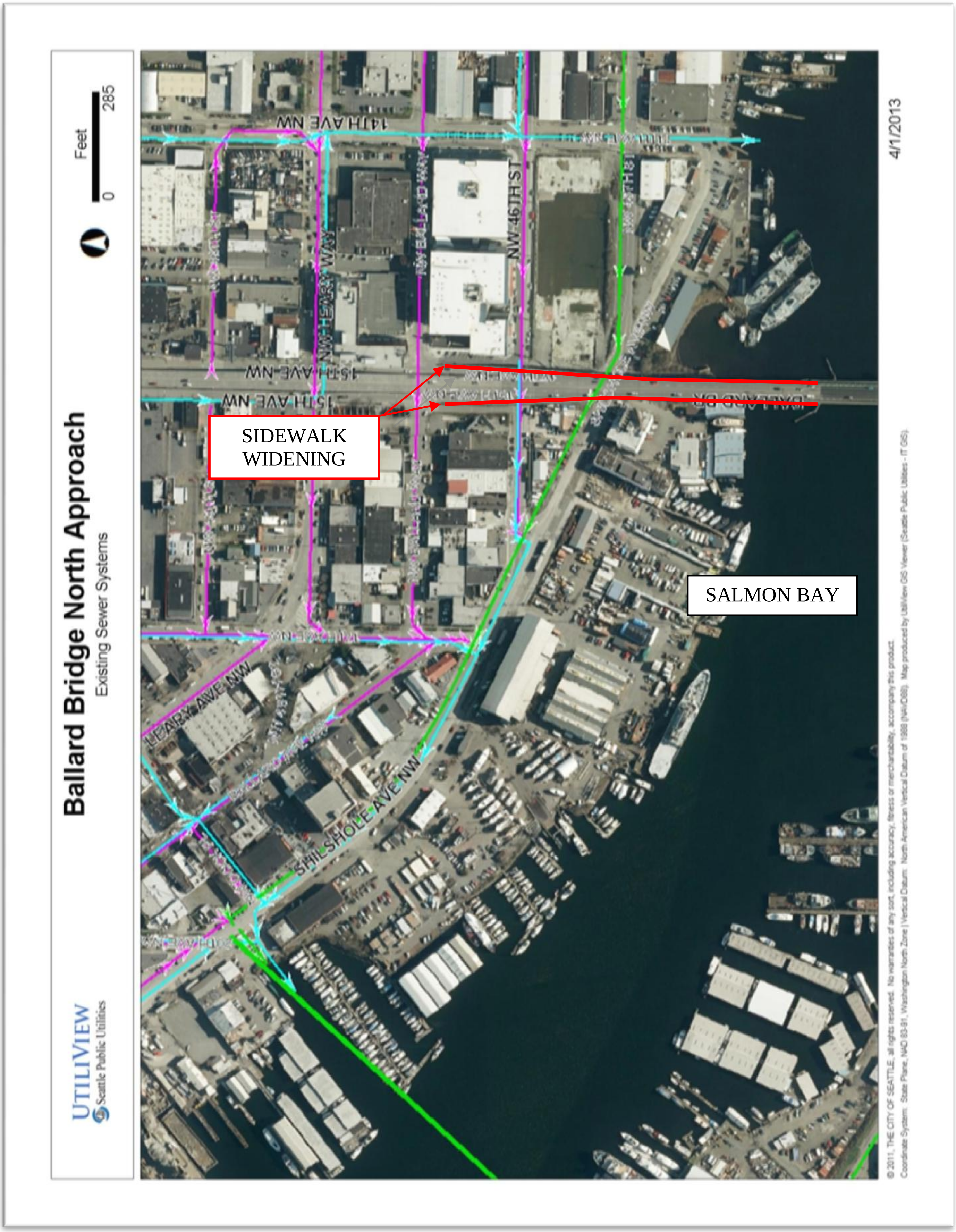
The Contractor will be required to provide a written Stormwater Pollution Control Plan for his operations. The report will identify specific control measures and best management practices.

3.3 Amended Soils

The contract will include amending the soil in areas disturbed by construction operations.

4.0 POTENTIAL COSTS

As a minimum the planting of new trees, the pollution control plan and amended soils will cost approximately $5,000. The construction of rock pockets at bridge drain outfalls and the use of permeable concrete may add as much as $20,000 to the cost of the bridge widening project.

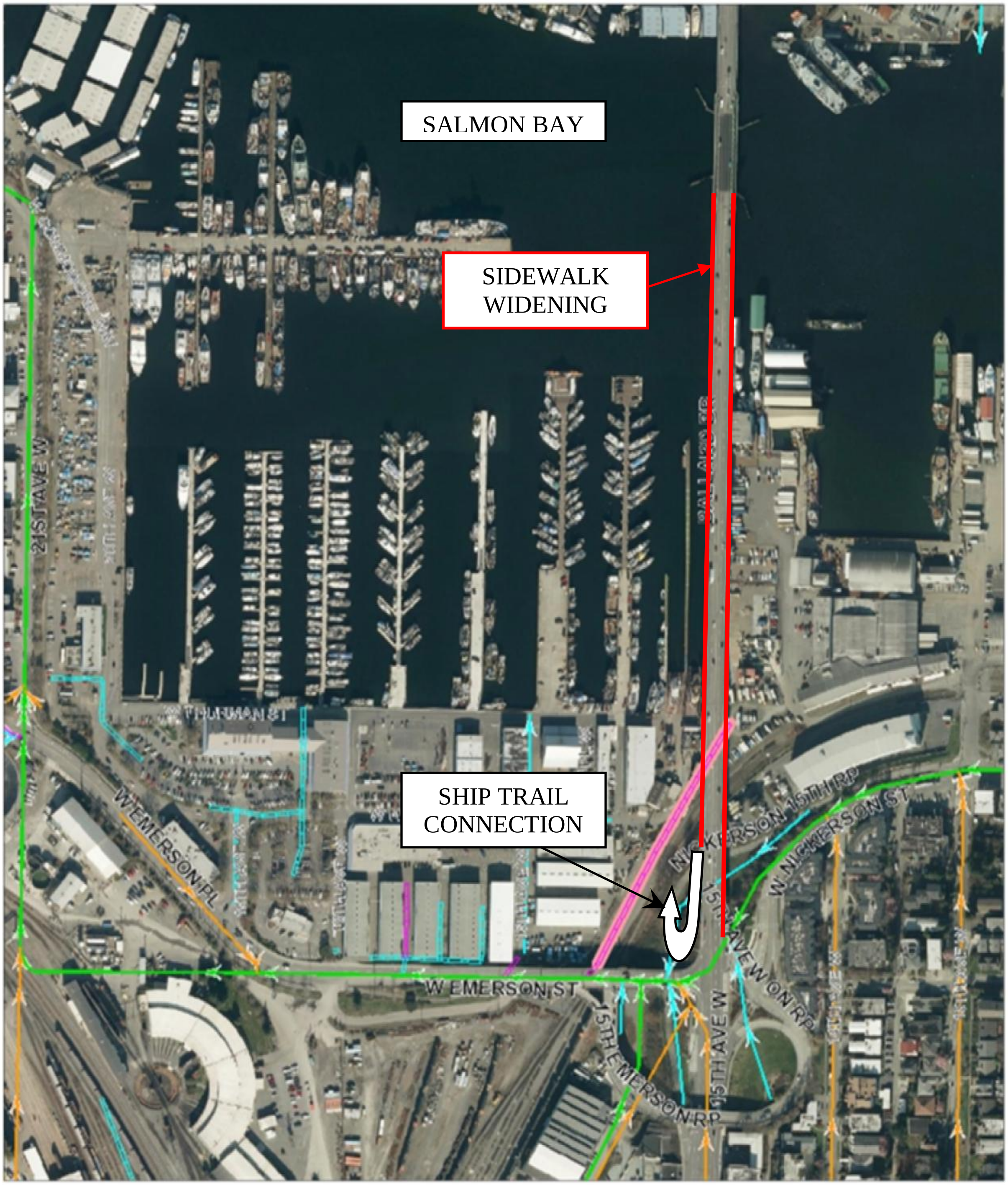

Ballard Bridge North Approach
Existing Sewer Systems
Feet
0
285
4/1/2013
UTILIVIEW
Seattle Public Utilities
14TH AVE NW
15TH AVE NW
NW LEARY WAY
NW 46TH ST
LEARY AVE NW
SHILSHOLE AVE NW
SIDEWALK WIDENING
SALMON BAY

UTILIVIEW
Seattle Public Utilities

Ballard Bridge South Approach

Existing Sewer Systems

Feet
0 326



Coordinate System: State Plane, NAD 83-91, Washington North Zone | Vertical Datum: North American Vertical Datum of 1988 (NAVD88). Map produced by UtiliView GIS viewer (Seattle Public Utilities - IT GIS).

4/1/2013

City of Seattle GSI to MEF Requirement Calculator (2012-05-01)

Building Permit No. →		Project Type →	Sidewalk
Project Address →	Ballard Bridge Sidewalk Widening - 6' Sidewalks	Project Area →	50,000 sf
		New plus Replaced Impervious Area →	47,000 sf
		Area Requiring Mitigation →	47,000 sf

Runoff Reduction Methods

Runoff Reduction Methods			Facility Size			Credit		Area Mitigated
Retained Trees								
Existing Evergreen	# Trees		Total Canopy Area of Trees		sf x	20% Canopy (or min 100 sf/tree) =		
Existing Deciduous	# Trees	14	Total Canopy Area of Trees		sf x	10% Canopy (or min 50 sf/tree) =	700	
New Trees								
New Evergreen			# Trees		x	50 sf/tree =		
New Deciduous			# Trees	10	x	20 sf/tree =	200	
						Total Area Mitigated by Trees	=	900 sf
Dispersion [1]								
Downspout or Sheet Flow Dispersion			Dispersed Impervious Area	15,200	sf x	100.0%	=	15,200 sf
						Note: Confirm flow paths can be achieved		

Infiltration and Reuse Facilities

Infiltration and Reuse Facilities			Facility Size			Sizing Factor		Area Mitigated
Infiltrating Facilities								
Bioretention Cell (without Underdrain)								
1 Contributing Area		sf	Bioretention Bottom Area		sf ÷	Enter Contributing Area	=	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
2 Contributing Area		sf	Bioretention Bottom Area		sf ÷	Enter Contributing Area	=	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
3 Contributing Area		sf	Bioretention Bottom Area		sf ÷	Enter Contributing Area	=	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
Detention Cistern to Bioretention Cell (BC) (without Underdrain) [2]								
Contributing Area		sf	Bioretention Bottom Area		sf ÷	Only for SFR	=	sf
Number Cisterns								
BC Ponding Depth		in						
BC Design Infilt Rate		in/hr						
Permeable Pavement Facility (may receive run-on) [3]								
Contributing Area		sf	Permeable Pavement Area		sf ÷	Enter Contributing Area	=	sf
Ponding Depth [4]		in				Plus Permeable Pavement Facility Area	=	sf
Design Infiltration Rate		in/hr						
Reuse Facilities [1]								
Rainwater Harvesting			Applicant must provide documentation of area mitigated by rainwater harvesting					sf

Impervious Surface Reduction Methods

Impervious Surface Reduction Methods			Facility Size			Credit		Area Mitigated
Alternative Pavement Surfaces								
Permeable Pavement Surface (Subgrade Slope ≤2%)			Permeable Pavement Area		sf x	100.0%	=	sf
Permeable Pavement Surface (Subgrade Slope >2-5%)			Permeable Pavement Area		sf x	55.0%	=	sf
Alternative Roof Surfaces [1]								
Green Roof (Single/Multi-Course / 4" Growth Medium)			Green Roof Area		sf x	55.0%	=	sf
Green Roof (Multi-Course / 8" Growth Medium)			Green Roof Area		sf x	84.0%	=	sf
Partial Infiltration [1]								
Bioretention Cell with Detention (without Underdrain)								
Contributing Area		sf						
Ponding Depth		in	Bioretention Bottom Area		sf	#VALUE!	=	sf
Design Infiltration Rate		in/hr				**#VALUE!**		
						#VALUE!		

Non-Infiltrating Facilities

Non-Infiltrating Facilities			Facility Size			Credit		Area Mitigated
Non Infiltrating Facilities								
Bioretention Planter (with underdrain)								
Contributing Area		sf						
Ponding Depth		in	Bioretention Bottom Area		sf ÷	Enter Contributing Area	=	sf
Detention Cistern with Harvesting Capacity [5, 6]								
Contributing Area		sf	Min Cistern Area		sf	Only Applicable for SFR	=	sf
			Min Live Cistern Volume		gal			

Total Area Mitigated →	16,100 sf
Area Requiring Mitigation →	47,000 sf
% Impervious Area Mitigated →	34.3 %
GSI to MEF Target Achieved? →	**NO**
	Mitigate More Area

Notes:

GSI - Green Stormwater Infrastructure; sf - square feet; in - inch; eqn - equation; BC - bioretention cell
min - minimum; ft - feet; in/hr - inch per hour; gal - gallons; infilt - infiltration

1. Single family residential projects and trail/sidewalk projects are not required to evaluate this BMP.
2. Each above ground cistern must have 6.68 sf minimum bottom area, a 0.25 inch orifice and a minimum of 3 feet of live storage above the orifice. If using two cisterns they must be connected and have only one orifice. Flow from cistern orifice must be routed to bioretention cell.
3. The area contributing runoff to a facility shall be no larger than 3 times the permeable pavement facility area corresponding to a minimum sizing factor of 33.3%).
4. Average subsurface ponding depth in aggregate storage reservoir.
5. Cistern must be above ground. Cistern area must be rounded up to next commercially available product. Cistern need not have more than 3 feet of live storage volume above orifice.
6. Water collected using the detention cistern may be used for non-potable uses only (e.g., irrigation). For additional uses of harvested water consider the "Rainwater Harvesting" BMP.

This calculator does not provide conveyance flow calculations.
Applicant is responsible to ensure system overflow conveyance is provided per Section 4.2.5 of the Stormwater Manual Volume 3.

City of Seattle GSI to MEF Requirement Calculator (2012-05-01)

Building Permit No. →		Project Type →	Sidewalk
Project Address →	Ballard Bridge Sidewalk Widening - 10' Sidewalks	Project Area →	70,000 sf
		New plus Replaced Impervious Area →	66,000 sf
		Area Requiring Mitigation →	66,000 sf

Runoff Reduction Methods

Runoff Reduction Methods			Facility Size			Credit		Area Mitigated
Retained Trees								
Existing Evergreen	# Trees		Total Canopy Area of Trees		sf	x	20% Canopy (or min 100 sf/tree) =	
Existing Deciduous	# Trees	14	Total Canopy Area of Trees		sf	x	10% Canopy (or min 50 sf/tree) = 700	
New Trees								
New Evergreen			# Trees			x	50 sf/tree =	
New Deciduous			# Trees	10		x	20 sf/tree = 200	
							Total Area Mitigated by Trees =	900 sf
Dispersion [1]								
Downspout or Sheet Flow Dispersion			Dispersed Impervious Area	25,300	sf	x	100.0% =	25,300 sf
							Note: Confirm flow paths can be achieved	

Infiltration and Reuse Facilities

Infiltration and Reuse Facilities			Facility Size			Sizing Factor		Area Mitigated
Infiltrating Facilities								
Bioretention Cell (without Underdrain)								
1 Contributing Area		sf	Bioretention Bottom Area		sf	÷	Enter Contributing Area =	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
2 Contributing Area		sf	Bioretention Bottom Area		sf	÷	Enter Contributing Area =	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
3 Contributing Area		sf	Bioretention Bottom Area		sf	÷	Enter Contributing Area =	sf
Ponding Depth		in						
Design Infiltration Rate		in/hr						
Detention Cistern to Bioretention Cell (BC) (without Underdrain) [2]								
Contributing Area		sf	Bioretention Bottom Area		sf	÷	Only for SFR =	sf
Number Cisterns								
BC Ponding Depth		in						
BC Design Infilt Rate		in/hr						
Permeable Pavement Facility (may receive run-on) [3]								
Contributing Area		sf	Permeable Pavement Area		sf	÷	Enter Contributing Area =	sf
Ponding Depth [4]		in					Plus Permeable Pavement Facility Area =	sf
Design Infiltration Rate		in/hr						
Reuse Facilities [1]								
Rainwater Harvesting			Applicant must provide documentation of area mitigated by rainwater harvesting					sf

Impervious Surface Reduction Methods

Impervious Surface Reduction Methods			Facility Size			Credit		Area Mitigated
Alternative Pavement Surfaces								
Permeable Pavement Surface (Subgrade Slope ≤2%)			Permeable Pavement Area		sf	x	100.0% =	sf
Permeable Pavement Surface (Subgrade Slope >2-5%)			Permeable Pavement Area		sf	x	55.0% =	sf
Alternative Roof Surfaces [1]								
Green Roof (Single/Multi-Course / 4" Growth Medium)			Green Roof Area		sf	x	55.0% =	sf
Green Roof (Multi-Course / 8" Growth Medium)			Green Roof Area		sf	x	84.0% =	sf
Partial Infiltration [1]								
Bioretention Cell with Detention (without Underdrain)								
Contributing Area		sf						
Ponding Depth		in	Bioretention Bottom Area		sf		#VALUE! =	sf
Design Infiltration Rate		in/hr					**#VALUE!**	
							#VALUE!	

Non-Infiltrating Facilities

Non-Infiltrating Facilities			Facility Size			Credit		Area Mitigated
Non Infiltrating Facilities								
Bioretention Planter (with underdrain)								
Contributing Area		sf						
Ponding Depth		in	Bioretention Bottom Area		sf	÷	Enter Contributing Area =	sf
Detention Cistern with Harvesting Capacity [5, 6]								
Contributing Area		sf	Min Cistern Area		sf		Only Applicable for SFR =	sf
			Min Live Cistern Volume		gal			

Total Area Mitigated →	26,200 sf
Area Requiring Mitigation →	66,000 sf
% Impervious Area Mitigated →	39.7 %
GSI to MEF Target Achieved? →	**NO**
	Mitigate More Area

Notes:

GSI - Green Stormwater Infrastructure	sf - square feet	in - inch	eqn - equation	BC - bioretention cell
min - minimum	ft - feet	in/hr - inch per hour	gal - gallons	infilt - infiltration

1. Single family residential projects and trail/sidewalk projects are not required to evaluate this BMP.
2. Each above ground cistern must have 6.68 sf minimum bottom area, a 0.25 inch orifice and a minimum of 3 feet of live storage above the orifice. If using two cisterns they must be connected and have only one orifice. Flow from cistern orifice must be routed to bioretention cell.
3. The area contributing runoff to a facility shall be no larger than 3 times the permeable pavement facility area corresponding to a minimum sizing factor of 33.3%).
4. Average subsurface ponding depth in aggregate storage reservoir.
5. Cistern must be above ground. Cistern area must be rounded up to next commercially available product. Cistern need not have more than 3 feet of live storage volume above orifice.
6. Water collected using the detention cistern may be used for non-potable uses only (e.g., irrigation). For additional uses of harvested water consider the "Rainwater Harvesting" BMP.

This calculator does not provide conveyance flow calculations.
Applicant is responsible to ensure system overflow conveyance is provided per Section 4.2.5 of the Stormwater Manual Volume 3.

Appendix F
Lighting/Photometric Analysis

Illumination Study - Bridge Summary
By DKS

June 2013

Bridge Illumination
An illumination study was conducted to evaluate the required pole spacing and layout with the new sidewalk widening. The conceptual illumination study was conducted by DKS and Associates and was conducted using AGI32 Lighting Analysis Software. The lighting analysis was completed using the calculation criteria identified in the *SDOT Right-of-Way Lighting Level Design Guidelines*. It was assumed for this analysis that all roadway lighting will use LED cobra head style fixtures with the following characteristics:

Roadway Luminaire on the Bridge

Style:	120LED
Distribution Type:	Type II Cuttoff
Wattage:	270W

Based on the roadway and fixture characteristics the following lighting criteria were used for the analysis:

Light Levels

Roadway Class:	Roadway	Sidewalk
Average Design Illuminance:	2.0 fc	1.2 fc
Uniformity Ratio (avg/min):	3:1	4:1

Based on the new sidewalk widening conceptual layout, it was recommended that the light poles be mounted on the outside of the bridge to replace the existing lighting that is currently mounted on the existing bridge barrier. This will allow for the streetlights to provide adequate lighting to the sidewalk as well as the roadway without the need for separate pedestrian level lighting. Standard SDOT street light poles where assumed for the analysis with 12 foot bracket arms in a staggered configuration. Luminaires were assumed to be mounted at a 35 foot mounting height. A summary of the study is included in the Appendix, including conceptual pole layout and calculations results. Also included in the study was a cost analysis. The recommended illumination system upgrade costs have been included in the cost estimate.

Illumination Study - Trail Summary
By DKS

June 2013

Bike Trail Illumination

An illumination study was conducted to evaluate the required pole spacing and layout for the proposed Ballard Bridge to Ship Canal bike trail. The conceptual illumination study was conducted by DKS and Associates and was conducted using AGI32 Lighting Analysis Software. The lighting analysis was completed using the calculation criteria identified in the *SDOT Right-of-Way Lighting Level Design Guidelines*. It was assumed for this analysis that the pedestrian scale lighting pole and LED fixtures with the following characteristics:

Pedestrian Luminaire on the Bike Trail

Style:	49LED Post Top
Distribution Type:	Type III Medium Cuttoff
Wattage:	100W
Mounting height	14' City Standard

The existing pedestrian path under East Emerson Place does not currently have illumination. Within the *SDOT Right-of-Way Lighting Level Design Guidelines* there are not clear guidelines for the lighting requirements for this facility, but given that it will be a grade separated bike facility and will transition under the existing E Emerson Place overpass, the pedestrian illumination has been conceptually designed to meet the requirements assuming obstructions are present within the facility:

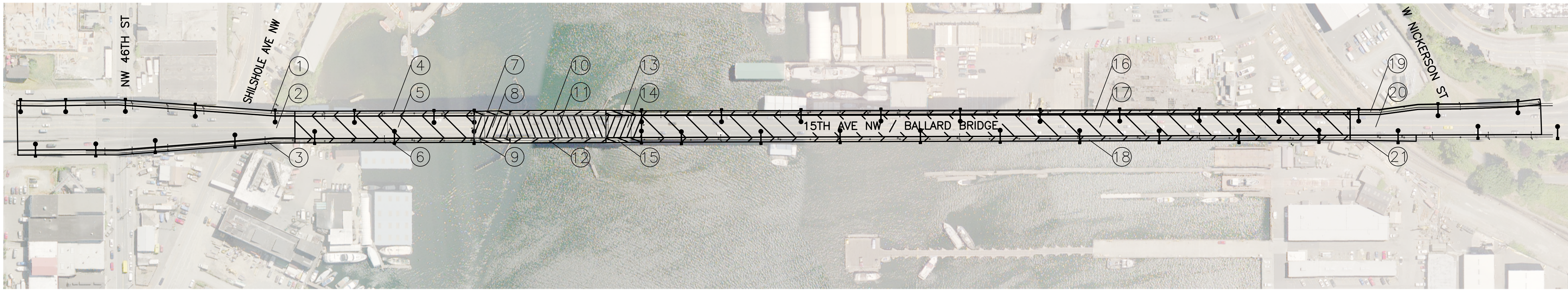
Light Levels

Classification:	Pedestrian & Bicycle Facility
Average Design Illuminance:	0.7 fc
Uniformity Ratio (avg/min):	4:1

Luminaires have been placed on the west side of the bike trail only to minimize the need for conduit installation on both sides of the trail between the connection to the Ballard Bridge and the E Emerson Underpass. The existing roadway lighting with 250 watt HPS luminaires along the W Nickerson Street ramp have been included in the analysis which are contributing to the illumination along the bike trail. This minimized the need for additional lights along the bike trail on the west side of W Nickerson Street. One additional pedestrian light was assumed to provide adequate lighting under the W Emerson Pl overpass. A summary of the study is included in the Appendix, including conceptual pole layout and calculations results. Also included in the study was a cost analysis. The recommended illumination system upgrade costs have been included in the cost estimate.

REGION NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.

50 0 100 200ft



DESIGN CRITERION

AREA TYPE	DESIGNED AVG. MAINTAINED ILLUMINATION LEVEL, fc	UNIFORMITY (AVG/MIN)
ROADWAY	2.0	3.0
PEDESTRIAN WAY	1.2	4.0

LIGHTING CALCULATION RESULTS

			ROADWAY		EAST SIDEWALK		WEST SIDEWALK	
DESCRIPTION	PATTERN	SPACING (FEET)	AVERAGE MAINTAINED ILLUMINANCE (FC)	UNIFORMITY (AVG/MIN)	AVERAGE MAINTAINED ILLUMINANCE (FC)	UNIFORMITY (AVG/MIN)	AVERAGE MAINTAINED ILLUMINANCE (FC)	UNIFORMITY (AVG/MIN)
NORTH MERGE LANE	Staggered	160'/120'	2.11	2.64	1.50	3.00	1.40	2.80
NORTH OF BASCULE	Staggered	160'	2.90	2.27	1.73	2.16	1.64	2.05
BASCULE NORTH TRANSITION	N/A	N/A	2.59	3.24	1.69	2.68	1.54	2.52
BASCULE	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
BASCULE SOUTH TRANSITION	N/A	N/A	2.60	3.33	1.53	2.55	1.77	2.77
SOUTH OF BASCULE	Staggered	160'	2.69	2.13	1.58	1.98	1.66	2.08
SOUTH MERGE LANE	Staggered	160'	2.39	2.60	1.59	3.18	1.69	2.11

LEGEND:

	NORTH MERGE LANE	NORTH OF BASCULE	BASCULE NORTH TRANSITION	BASCULE	BASCULE SOUTH TRANSITION	SOUTH OF BASCULE	SOUTH MERGE LANE
EAST SIDEWALK	1	4	7	10	13	16	19
ROADWAY	2	5	8	11	14	17	20
WEST SIDEWALK	3	6	9	12	15	18	21

NEW STREETLIGHT POLE, 12' BRACKET ARM AND LUMINAIRE

VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHK'D	REV'D

LT1
BALLARD BRIDGE

DKS Associates
TRANSPORTATION SOLUTIONS
719 Second Avenue, Suite 1250
Seattle, WA 98104-1706
(206) 382-9800

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BY: . . . .
DIRECTOR, CONTRACTING SERVICES

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DESIGNED . SZR 06/05/13 . . . .
CHECKED . JMH 06/05/13 . . . .
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Seattle Department of Transportation
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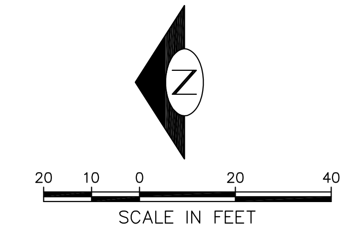
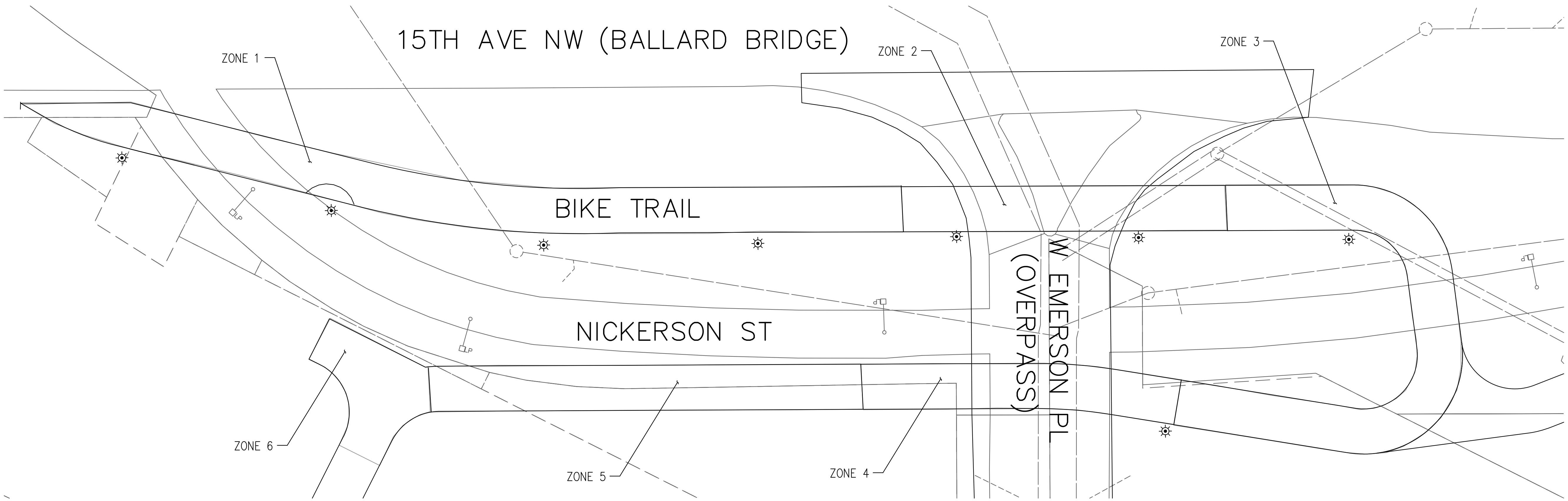
FUND: . . . .
INSPECTOR'S BOOK. . . . . .

BALLARD BRIDGE SIDEWALK WIDENING
BALLARD BRIDGE
CONCEPTUAL LIGHTING ANALYSIS

JOB NO. PC
R/W –
CO
VAULT PLAN NO.
SHEET 1 OF 2

REGION NO.	STATE	FEDERAL AID PROJECT NO.	SHEET NO.
			2





VAULT SERIAL NO.	DATE	MARK	NATURE REVISIONS	MADE	CHK'D	REV'D

DESIGN CRITERION

AREA TYPE	DESIGNED AVG. MAINTAINED ILLUMINATION LEVEL, fc	UNIFORMITY (AVG/MIN)
BIKE TRAIL / PEDESTRIAN WAY	0.7	4.0

LIGHTING CALCULATION RESULTS

ZONE NO.	AVERAGE ILLUMINATION LEVEL, fc	UNIFORMITY (AVG/MIN)
ZONE 1	1.01	3.37
ZONE 2	1.21	3.46
ZONE 3	0.94	3.24
ZONE 4	1.15	2.35
ZONE 5	1.15	1.85
ZONE 6	0.70	2.80

LEGEND

PEDESTRIAN LIGHT POLE, FOUNDATION AND LUMINAIRE

EXISTING LIGHT POLE

LT2
BIKE TRAIL

DKS *Associates*
TRANSPORTATION SOLUTIONS
719 Second Avenue, Suite 1250
Seattle, WA 98104-1706
(206) 382-9800

APPROVED FOR ADVERTISING
FRED PODESTA
DEPARTMENT OF EXECUTIVE ADMINISTRATION
SEATTLE, WASHINGTON . . . . 20 . . .

BY: . . . .
DIRECTOR, CONTRACTING SERVICES

NAME OR INITIALS AND DATE
DESIGNED . SZR, 06/05/13. . . . .
CHECKED . JMH ,06/05/13. . . . .
DRAWN . SZR, 06/05/13 . . . .
CHECKED . JMH ,06/05/13. . . . .

INITIALS AND DATE
REVIEWED:
PE. . . . . CONST. . . . . .
PROJ. MGR. . . . .
RECEIVED . . . .
REVISED AS BUILT . . . .

ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

City of Seattle
Seattle Department of Transportation
ORDINANCE NO. . . . . APPROVED. . . . .
FUND: . . . .
INSPECTOR'S BOOK . . . . . .

BALLARD BRIDGE SIDEWALK WIDENING
BIKE TRAIL
CONCEPTUAL LIGHTING ANALYSIS

JOB NO. PC
R/W –
CO

VAULT PLAN NO.

SHEET 2 OF 2

Appendix G
Bridge Widening Cost Estimate
(Planning Level)

BALLARD BRIDGE SIDEWALK WIDENING
COST ESTIMATE - ALTERNATIVE NO. 1
01 NOVEMBER 2013
UPDATED: 8/20/2014

Schematic Quantites and Cost Estimate - Alternative No. 1 (1-ft widening)				
ITEM DESCRIPTION	**UNIT**	**UNIT COST**	**QUANTITY**	**TOTAL**
REMOVE CEM CONC SIDEWALK	SY	$ 20	0	$ -
REMOVE CONC TRAFFIC BARRIER	LF	$ 15	5600	$ 84,000
REMOVE LUMINAIRE	EA	$ 1,500	28	$ 42,000
SURFACE PREPARATION	SF	$ 5	4800	$ 24,000
CONCRETE CL 4000 FOR BARRIER CURB	CY	$ 900	285.867135	$ 257,300
EPOXY COATED STEEL REINFORCING BAR	LB	$ 2	35396.8275	$ 70,800
REINFORCING BAR (DOWELED)	LB	$ 1	9030.66731	$ 9,100
DRILL HOLE 5/8" DIAMETER	LF	$ 30	282.5	$ 8,500
DRILL HOLE 3/4" DIAMETER	LF	$ 30	198	$ 6,000
EPOXY BONDING COMPOUND	EA HOLE	$ 20	856	$ 17,200
STRUCTURAL HIGH STRENGTH STEEL	LB	$ 4	128954.968	$ 515,900
ILLUMINATION	LS	$ 950,000	1	$ 950,000

Note: costs for widening one side of bridge.

	Subtotal	$ 1,984,800
BOTH SIDES OF BRIDGE	TOTAL	$ 3,969,600
MOBILIZATION	10%	$ 396,960
TOTAL ESTIMATED ALT 1 COSTS (2013)	**TOTAL**	**$ 4,366,560**

Ballard Bridge Sidewalk Widening Concept Study

Estimated Construction Costs (Alternative 3)

Total Costs are 2014 dollars, and rounded to the nearest $100.

ITEM NO.	STD ITEM NO.	DESCRIPTION	DESIGN CONTINGENCY	UNIT PRICE	UNIT	QUANTITY	COST
1		STEEL RAILING	1.00	$ 4	LBS.	175,638	$ 702,600
2		PAINTING STEEL RAILING	1.00	$ 0.25	LBS.	175,638	$ 44,000
3		DRILL HOLE FOR POST CONNECTION	1.00	$ 30	LF.	2,168	$ 65,100
4		EPOXY RESIN BOND COMPOUND	1.00	$ 20	EA.	3,252	$ 65,100
5		ANCHOR RODS	1.00	$ 4	LBS.	3,961	$ 15,900
ESTIMATED TOTAL COSTS (2014$)							$ 892,700
MOBILIZATION			10.0%				$ 89,300
ESTIMATED TOTAL COST (2014$)							$ 982,000

Note: costs for widening one side of bridge.

Schematic Quantites and Cost Estimate - 6ft Wide Sidewalk

ITEM DESCRIPTION	UNIT	UNIT COST	QUANTITY	TOTAL	*Exact Quantities*
REMOVE CEM CONC SIDEWALK	SY	$ 20	1100	$ 22,000	*1010*
REMOVE CONC TRAFFIC BARRIER	LF	$ 15	5600	$ 84,000	*5596*
REMOVE LUMINAIRE	EA	$ 1,500	28	$ 42,000	*28*
SURFACE PREPARATION	SF	$ 5	4800	$ 24,000	*4743*
CONCRETE CL 4000 FOR BARRIER CURB	CY	$ 900	570	$ 513,000	*560*
EPOXY COATED STEEL REINFORCING BAR	LB	$ 2	67000	$ 100,500	*66767*
REINFORCING BAR (DOWELED)	LB	$ 1	10000	$ 10,000	*9620*
DRILL HOLE 5/8" DIAMETER	LF	$ 30	680	$ 20,400	*678*
DRILL HOLE 3/4" DIAMETER	LF	$ 30	890	$ 26,700	*883*
EPOXY BONDING COMPOUND	EA HOLE	$ 20	2300	$ 46,000	*2277*
STRUCTURAL HIGH STRENGTH STEEL	LB	$ 4	157000	$ 628,000	*156172*
ILLUMINATION	LS	---	40	$ 950,000	*28*

Subtotal	$	2,466,600
40% Contingency	$	986,640
TOTAL	**$**	**3,453,240**

Schematic Quantites and Cost Estimate - 10ft Wide Sidewalk

ITEM DESCRIPTION	UNIT	UNIT COST	QUANTITY	TOTAL	*Exact Quantities*
REMOVE CEM CONC SIDEWALK	SY	$ 20	1200	$ 24,000	*1180*
REMOVE CONC TRAFFIC BARRIER	LF	$ 15	5600	$ 84,000	*5596*
REMOVE CONC DECK - PROTECT EX. STEEL	CY	$ 250	110	$ 27,500	*104*
REMOVE LUMINAIRE	EA	$ 1,500	28	$ 42,000	*28*
SURFACE PREPARATION	SF	$ 5	2400	$ 12,000	*2324*
CONCRETE CL 4000 FOR BARRIER CURB	CY	$ 900	850	$ 765,000	*850*
EPOXY COATED STEEL REINFORCING BAR	LB	$ 2	138000	$ 207,000	*137393*
REINFORCING BAR (DOWELED)	LB	$ 1	3700	$ 3,700	*3626*
DRILL HOLE 5/8" DIAMETER	LF	$ 30	400	$ 12,000	*393*
DRILL HOLE 3/4" DIAMETER	LF	$ 30	700	$ 21,000	*691*
EPOXY BONDING COMPOUND	EA HOLE	$ 20	710	$ 14,200	*708*
STRUCTURAL HIGH STRENGTH STEEL	LB	$ 4	339000	$ 1,356,000	*338934*
LAMINATED ELASTOMERIC BEARING PADS	EA	$ 2,500	2	$ 5,000	*2*
ILLUMINATION	LS	---	40	$ 950,000	*28*

Subtotal	$	3,523,400
40% Contingency	$	1,409,360
TOTAL	**$**	**4,932,760**

BergerABAM

Segment 2 - 6ft		336.12 ft		5 span	24 Cantilevers	
Concrete Slab	6.5" x 1'-7"				0.86 SF	11 CY
Slab Reinforcing		2.63 ft	#4 Transverse @12" w/ dowel		1.75 LB/FT	589 LB
		3.00 ft	#4 Longitundinal		2.00 LB/FT	674 LB
			Number of Epoxy Bars			337
		1.17 ft	5/8" Hole @12" spacing			393.18 ft
	Surface Prep		6.5 in Face			182.07 SF
Edge Beam	HSS 6x6x3/16				14.51 LB/FT	4877 LB
New Cantilever	MC8x8.5 2@18ft			6.67 ft	56.67 LB/Cantilever	1360 LB
New Cantilever Plate	PL0.25x4x1'9" 2@18ft			.29 CF	142.92 LB/Cantilever	3430 LB
Bolts Through Cantilever	6 - 5/8" x 13" @18ft			.01 CF	7.31 LB/Cantilever	175 LB
		1.00 ft	6 - 3/4" Holes @ Cantilevers			144.00 ft
Pedestrian Rail	See Calculation				36.6 LB/FT	12302 LB
BP Rail	See Calculation				6.7 LB/FT	2252 LB
Concrete Barrier	32" Single Slope (8" top)				2.46 SF	31 CY
	Surface Prep					394.81 SF
Barrier Reinforcing		2.96 ft	#5 w/ dowel		3.09 LB @ 9" in 10' @18" else	898 LB
			Number of Epoxy Bars			291
		.50 ft	3/4" Hole @12" spacing			145.50 ft
		6.82 ft	#4 Hoop @ 12"		4.69 LB/FT	1577 LB
		8.00 ft	4 - #5 Horiz EF		8.34 LB/FT	2805 LB
Standard Luminarie Poles	at interior piers					4 EA
Pole offset pipe	HSS10.75x0.375			3.00 ft	41.59 plf 4 Locations	499 LB
Top Connection Plate	1.625" thick x 181SQIN			.17 CF	83 LB 4 Locations	334 LB
Base Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB 4 Locations	415 LB
Removal - Curb						336.12 LF
Removal - Barrier						336.12 LF

Concrete	41 CY
Reinforcing Steel	5055 LB
Reinforcing Steel (Doweled)	1487 LB
Light Poles	4 EA
Structural Steel	25644 LB
Removal LF	672.24 LF
Removal SF	0.00 SF
Remove Light Poles	4 EA
Drill Hole 5/8"	393.18 LF
Drill Hole 3/4"	289.50 LF
Epoxy Bonding Compound	628.00 EA
Surface Prep.	576.87 SF

Segments 3 & 5 - 6ft	1875.95 ft	18 span

Concrete Slab	4" x 6'-5"				2.14 SF		149 CY
Slab Reinforcing		6.50 ft	#4 Transverse		4.34 LB/FT		8145 LB
		7.00 ft	#4 Longitundinal		4.68 LB/FT		8772 LB
Headed Anchor Studs	5/8" x 8" @ 24"				0.56 LB/Stud		525 LB
			Number of Epoxy Bars				938
		.50 ft	3/4" Hole @24" spacing				469.00 ft
	Surface Prep		9" on Edge Beam				1406.96 SF
Pedestrian Rail	See Calculation				36.6 LB/FT		68660 LB
BP Rail	See Calculation				6.7 LB/FT		12569 LB
Concrete Barrier	38" Single Slope (8" top)				3.07 SF		213 CY
	Surface Prep						2069.50 SF
Barrier Reinforcing		3.72 ft	#5 drilled through		3.88 LB	@ 9" in 10' @18" else	5785 LB
		.75 ft	3/4" Hole				1117.98 ft
		7.92 ft	#4 Hoop @ 12"		5.45 LB/FT		10227 LB
		8.00 ft	4 - #5 Horiz EF		8.34 LB/FT		15653 LB
		3.00 ft	#4 Vertical w/ dowel @ 4ft		2.00 LB/FT		940 LB
			Number of Epoxy Bars				469
		.50 ft	5/8" Hole @48" spacing				234.50 ft
	PL 3/8"x4"x0'4"			.0035 CF	1.70 LB	@ 9" in 10' @18" else	2536 LB
Standard Luminarie Poles	One per span						18 EA
Pole offset pipe	HSS10.75x0.375			3.00 ft	41.59 plf	18 Locations	2246 LB
Top Connection Plate	1.625" thick x 181SQIN			.17 CF	83 LB	18 Locations	1501 LB
Base Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB	18 Locations	1866 LB
Lateral Offset Tube	HSS 10x10x0.375			10.00 ft	47.82 plf	18 Locations	8608 LB
Outside Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB	18 Locations	1866 LB
Beam Attachment Plates	2 x 0.625" thick x 6"x1'-4"			.07 CF	34 LB	18 Locations	612 LB
Removal - Curb							1875.95 LF
Removal - Barrier							1875.95 LF
Removal - Sidewalk							7503.80 SF
						Concrete	362 CY
						Reinforcing Steel	42797 LB
						Reinforcing Steel (Doweled)	6724 LB
						Light Poles	18 EA
						Structural Steel	100990 LB
						Removal LF	3751.90 LF
						Removal SF	7503.80 SF
						Remove Light Poles	18 EA
						Drill Hole 5/8"	234.50 LF
						Drill Hole 3/4"	469.00 LF
						Epoxy Bonding Compound	1407.00 EA
						Surface Prep.	3476.46 SF
Segment 6 - 6ft		84.91 ft		1 span			
Concrete Slab	4" x 6'-5"				2.14 SF		7 CY

Slab Reinforcing		6.50 ft	#4 Transverse		4.34 LB/FT		369 LB
		7.00 ft	#4 Longitundinal		4.68 LB/FT		397 LB
	Surface Prep		9" on Edge Beam				63.68 SF
Pedestrian Rail	See Calculation				36.6 LB/FT		3108 LB
BP Rail	See Calculation				6.7 LB/FT		569 LB
Concrete Barrier	38" Single Slope (8" top)				3.07 SF		10 CY
	Surface Prep						93.67 SF
Barrier Reinforcing		3.72 ft	#5 drilled through		3.88 LB	@ 9" in 10' @18" else	271 LB
		.75 ft	3/4" Hole				52.50 ft
		7.92 ft	#4 Hoop @ 12"		5.45 LB/FT		463 LB
		8.00 ft	4 - #5 Horiz EF		8.34 LB/FT		708 LB
		3.00 ft	#4 Vertical w/ dowel @ 4ft		2.00 LB/FT		43 LB
			Number of Epoxy Bars				23
		.50 ft	5/8" Hole @48" spacing				11.50 ft
	PL 3/8"x4"x0'4"			.0035 CF	1.70 LB	@ 9" in 10' @18" else	119 LB
Standard Luminarie Poles	One						1 EA
Outside Pole Elbow pipe	HSS8.625x0.375			5.00 ft	33.07 plf	1 Location	165 LB
Top Connection Plate	1.625" thick x 181SQIN			.17 CF	83 LB	1 Location	83 LB
Base Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB	1 Location	104 LB
Kicker Tube	HSS 6x6x0.375			3.00 ft	27.41 plf	1 Location	82 LB
Outside Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB	1 Location	104 LB
Inside Reaction Plate	0.625" x 14" x 14"			.07 CF	35 LB	1 Location	35 LB
Headed Anchor Studs	4- 5/8" x 8"				0.56 LB/Stud		2 LB
		2.83 ft	#4 Transverse w/ dowel		1.89 LB/Bar		4 LB
			Number of Epoxy Bars				2
		1.17 ft	5/8" Hole @48" spacing				2.33 ft
Removal - Curb							84.91 LF
Removal - Barrier							84.91 LF
Removal - Sidewalk							339.64 SF
						Concrete	16 CY
						Reinforcing Steel	1941 LB
						Reinforcing Steel (Doweled)	314 LB
						Light Poles	1 EA
						Structural Steel	4371 LB
						Removal LF	169.82 LF
						Removal SF	339.64 SF
						Remove Light Poles	1 EA
						Drill Hole 5/8"	13.83 LF
						Drill Hole 3/4"	52.50 LF
						Epoxy Bonding Compound	25.00 EA
						Surface Prep.	157.35 SF
Segment 7 - 6ft		287.32 ft		4 span			

Concrete Slab	4" x 6'-5"				2.14 SF		23 CY
Slab Reinforcing		6.50 ft	#4 Transverse		4.34	LB/FT	1248 LB
		7.00 ft	#4 Longitundinal		4.68	LB/FT	1344 LB
Headed Anchor Studs	5/8" x 8" @ 24"				0.56	LB/Stud	80 LB
			Number of Epoxy Bars @2ft				144
		.50 ft	3/4" Hole @24" spacing				72.00 ft
	Surface Prep		9" on Edge Beam				215.49 SF
Pedestrian Rail	See Calculation				36.6	LB/FT	10516 LB
BP Rail	See Calculation				6.7	LB/FT	1925 LB
Concrete Barrier	38" Single Slope (8" top)				3.07 SF		33 CY
	Surface Prep						316.96 SF
Barrier Reinforcing		3.72 ft	#5 drilled through		3.88	LB @ 9" in 10' @18" else	950 LB
		.75 ft	3/4" Hole				183.75 ft
		7.92 ft	#4 Hoop @ 12"		5.45	LB/FT	1566 LB
		8.00 ft	4 - #5 Horiz EF		8.34	LB/FT	2397 LB
		3.00 ft	#4 Vertical w/ dowel @ 4ft		2.00	LB/FT	144 LB
			Number of Epoxy Bars				73
		.50 ft	5/8" Hole @48" spacing				36.50 ft
	PL 3/8"x4"x0'4"			.0035 CF	1.70	LB @ 9" in 10' @18" else	417 LB
Standard Luminarie Poles	One per span						4 EA
Pole offset pipe	HSS10.75x0.375			3.50 ft	41.59 plf	4 Locations	582 LB
Top Connection Plate	1.625" thick x 181SQIN			.17 CF	83 LB	4 Locations	334 LB
Base Connection Plate	1.625" thick x 225SQIN			.21 CF	104 LB	4 Locations	415 LB
Removal - Curb							287.32 LF
Removal - Barrier							287.32 LF
Removal - Sidewalk							1149.28 SF

Concrete	55 CY
Reinforcing Steel	6555 LB
Reinforcing Steel (Doweled)	1094 LB
Light Poles	4 EA
Structural Steel	15912 LB
Removal LF	574.64 LF
Removal SF	1149.28 SF
Remove Light Poles	4 EA
Drill Hole 5/8"	36.50 LF
Drill Hole 3/4"	72.00 LF
Epoxy Bonding Compound	217.00 EA
Surface Prep.	532.45 SF

Segment 1 - 6ft 187.09 ft Approx

Concrete CAD Projected area			10 SQFT		69 CY
Slab Reinforcing	#4 @ 6" Top Bar	14.67 ft	10.09 LB/FT		1888 LB
	#4 @ 12" Bot Bar	5.33 ft	3.67 LB/FT		686 LB
	#4 @ 12" TS bot	4.25 ft	2.92 LB/FT		547 LB
	#4 @ 12" T&B Trans	20.00 ft	13.76 LB/FT		2574 LB
	#4 @ 12" Barrier Back	3.50 ft	2.41 LB/FT		451 LB
	#5 @ 9" Barrier Front	5.67 ft	5.91 LB/FT		1106 LB
	8 - #5 horiz bars	8.00 ft	8.34 LB/FT		1561 LB
Pedestrian Rail	See Calculation		36.6 LB/FT		6847 LB
BP Rail	See Calculation		6.7 LB/FT		1254 LB
Removal - Curb					187.09 LF
Removal - Barrier					187.09 LF
Removal - Sidewalk			5.21 ft	width of removal	974.43 SF
				Concrete	69 CY
				Reinforcing Steel	8813 LB
				Light Poles	1
				Structural Steel	8101 LB
				Removal LF	374.18 LF
				Removal SF	974.43 SF
				Remove Light Poles	1

Segment 8 - 6ft 26.67 ft Approx

Concrete CAD Projected area			16.6 SQFT		16 CY
Slab Reinforcing	#4 @ 6" Top Bar	14.67 ft	10.09 LB/FT		269 LB
	#4 @ 12" Bot Bar	6.00 ft	4.13 LB/FT		110 LB
	#4 @ 6" TS bot	11.67 ft	8.03 LB/FT		214 LB
	#4 @ 12" & 6" T&B Trans	29.00 ft	19.95 LB/FT		532 LB
	#4 @ 12" Barrier Back	3.58 ft	2.47 LB/FT		66 LB
	#5 @ 9" Barrier Front	6.89 ft	7.19 LB/FT		192 LB
	8 - #5 horiz bars	8.00 ft	8.34 LB/FT		223 LB
Pedestrian Rail	See Calculation		36.6 LB/FT		976 LB
BP Rail	See Calculation		6.7 LB/FT		179 LB
Removal - Curb					26.67 LF
Removal - Barrier					26.67 LF
Removal - Sidewalk			3.50 ft	width of removal	93.33 SF
				Concrete	16 CY
				Reinforcing Steel	1605 LB
				Structural Steel	1155 LB
				Removal LF	53.33 LF

Removal SF	93.33 SF

Concrete	560 CY
Reinforcing Steel	66767 LB
Reinforcing Steel (Doweled)	9620 LB
Light Poles	28
Structural Steel	156172 LB
Removal of Barrier/Curb	5596.11 LF
Removal of Sidewalk	9086.05 SF
Remove Light Poles	28
Drill Hole 5/8"	678.01 ft
Drill Hole 3/4"	883.00 ft
Epoxy Bonding Compound	2277.00 EA
Surface Prep.	4743.14 SF

Segment 2 - 10ft 336.12 ft 5 span 24 Cantilevers

Concrete Slab	6.5" x 5'-8"			3.30 SF	41 CY
Slab Reinforcing		6.71 ft	#4 Transverse w/ dowel	4.48 LB/FT	1506 LB
			Number of Epoxy Bars		337.00
		1.17 ft	5/8" Hole @12" spacing		393.18 ft
	Surface Prep		4 in Face		112.04 SF
		7.00 ft	#4 Longitundinal	4.68 LB/FT	1572 LB
Edge Beam	HSS 8x6x3/16			17.1 LB/FT	5748 LB
Tension Strut @ Canti.	MC8x8.5 2@18ft		14.83 ft	126.08 LB/Cantilever	3026 LB
Compression Strut @ Canti.	HSS 8x4x3/16		14.54 plf	151.80 LB/Cantilever	3643 LB
Bolts Through Cantilever	3 - 5/8" x 13" @18ft		.01 CF	3.65 LB/Cantilever	88 LB
		1.00 ft	3 - 3/4" Holes @ Cantilevers		72.00 ft
Compression Anchor Plate	1"x16"x6"		.06 CF	27.22 LB/Cantilever	653 LB
Pedestrian Rail	See Calculation			36.6 LB/FT	12302 LB
BP Rail	See Calculation			6.7 LB/FT	2252 LB
Concrete Barrier	32" Single Slope (8" top)			2.46 SF	31 CY
	Surface Prep				394.81 SF
Barrier Reinforcing		2.96 ft	#5 w/ dowel	3.09 LB @ 9" in 10' @18" else	898 LB
			Number of Epoxy Bars		291
		.50 ft	3/4" Hole @12" spacing		145.50 ft
		6.82 ft	#4 Hoop @ 12"	4.69 LB/FT	1577 LB
		8.00 ft	4 - #5 Horiz EF	8.34 LB/FT	2805 LB
Standard Luminarie Poles	at interior piers				4 EA
Concrete Block		4.67 CF		4 Locations	1 CY
Transverse Reinf		5.33 ft	4 - #4 bars	4 Locations	59 LB
Longitudinal Reinf		6.00 ft	4 - #4 bars	4 Locations	66 LB
Headed Anchor Studs	4- 5/8" x 8"			0.56 LB/Stud	9 LB
		.67 ft	4 - 3/4" Holes @ Cantilevers		64.03 ft
Removal - Curb					336.12 LF
Removal - Barrier					336.12 LF
				Concrete	72 CY

Reinforcing Steel	6918 LB
Reinforcing Steel (Doweled)	2404 LB
Light Poles	4 EA
Structural Steel	27722 LB
Removal LF	672.24 LF
Removal SF	0.00 SF
Remove Light Poles	4 EA
Drill Hole 5/8"	393.18 LF
Drill Hole 3/4"	281.53 LF
Epoxy Bonding Compound	628.00 EA
Surface Prep.	506.85 SF

Segments 3 & 5 - 10ft 1875.95 ft 18 span

Concrete Slab	4" x10'-0"			3.33 SF		232 CY
Slab Reinforcing		10.50 ft	#4 Transverse @ 4"	21.04	LB/FT	39474 LB
		11.00 ft	#4 Longitundinal	7.35	LB/FT	13784 LB
Edge Beam	HSS 8x6x1/4			22.39	LB/FT	42003 LB
Pedestrian Rail	See Calculation			36.6	LB/FT	68660 LB
BP Rail	See Calculation			6.7	LB/FT	12569 LB
Concrete Barrier	38" Single Slope (8" top)			3.07 SF		213 CY
Concrete Barrier Slab	8.75" x 18"			1.09 SF		76 CY
	Surface Prep		face of cut deck			1406.96 SF
Barrier Reinforcing		9.03 ft	#5 Hoop	9.42	LB @ 9" in 10' @18" else	14036 LB
		14.00 ft	4 - #5 Horiz EF + 6 in slab	14.60	LB/FT	27393 LB
Beam Extension	W27x84		15.67 ft @ 20 ft			123440 LB
Flange Splice Plates	0.625" plates for 5 bolt line		.04 CF @ 20 ft			1773 LB
Flange Web Plates	0.625" plates for 5 bolt line		.01 CF @ 20 ft			443 LB
Standard Luminarie Poles	One per span					18 EA
Pole offset pipe	HSS10.75x0.375		3.00 ft	41.59 plf	18 Locations	2246 LB
Top Connection Plate	1.625" thick x 181SQIN		.17 CF	83 LB	18 Locations	1501 LB
Base Connection Plate	1.625" thick x 225SQIN		.21 CF	104 LB	18 Locations	1866 LB
Lateral Offset Tube	HSS 10x10x0.375		3.00 ft	47.82 plf	18 Locations	2582 LB
Outside Connection Plate	1.625" thick x 225SQIN		.21 CF	104 LB	18 Locations	1866 LB

Lateral Wide Flange	W10x19		19.00 plf	380 LB	18 Locations	6840 LB
Removal - Curb						1875.95 LF
Removal - Barrier						1875.95 LF
Removal - Sidewalk						7503.80 SF
Removal - Deck protect Steel						2813.93 CF
					Concrete	521 CY
					Reinforcing Steel	94687 LB
					Reinforcing Steel (Doweled)	0 LB
					Light Poles	18 EA
					Structural Steel	265790 LB
					Removal LF	3751.90 LF
					Removal SF	7503.80 SF
					Removal CF	2813.93 CF
					Remove Light Poles 18 EA	
					Drill Hole 5/8"	.00 LF
					Drill Hole 3/4"	.00 LF
					Epoxy Bonding Compound	0.00 EA
					Surface Prep.	1406.96 SF
Segment 6 - 6ft		84.91 ft	1 span			
Concrete Slab	4" x 10"			3.33 SF		10 CY
Slab Reinforcing		10.50 ft #4 Transverse @ 4"		21.04	LB/FT	1787 LB
		11.00 ft #4 Longitundinal		7.35	LB/FT	624 LB
Pedestrian Rail	See Calculation			36.6	LB/FT	3108 LB
BP Rail	See Calculation			6.7	LB/FT	569 LB
Concrete Barrier	38" Single Slope (8" top)			3.07 SF		10 CY
	Surface Prep					93.67 SF
Barrier Reinforcing		3.72 ft #5 drilled through		3.88	LB @ 9" in 10' @18" else	271 LB
		.75 ft 3/4" Hole				52.50 ft
		7.92 ft #4 Hoop @ 12"		5.45	LB/FT	463 LB
		8.00 ft 4 - #5 Horiz EF		8.34	LB/FT	708 LB
	PL 3/8"x4"x0'4"		.0035 CF	1.70	LB @ 9" in 10' @18" else	119 LB

New Bridge Beam	W36x150		150.00 plf			12737 LB
Walkway Support Beam	HSS 6x6x3/16		14.51 plf		9'-6" @ 10ft	1170 LB
Walkway Edge Beam	HSS 6x6x3/16		14.51 plf			1232 LB
Plates - Walkway to Beam	2-PL 0.625x8x0'10"		.06 CF	Per support beam		241 LB
Plates - Walkway to Concrete	PL 0.625x6x1'2"		.03 CF	Per support beam		126 LB
Headed Anchor Studs	2-5/8" x 8"			0.56	LB/Stud	10 LB
Standard Luminarie Poles	One					1 EA
Strong HSS support Beam	HSS10x6x1/4		11.67 ft	25.79 plf	1 Location	301 LB
Base Connection Plate	1.625" thick x 225SQIN		.21 CF	104 LB	1 Location	104 LB
Pier 22 Extension Top		10.33SQFT x 2FT		20.7 CF	@ Pier 22	1 CY
Pier 22 Extension Base	38" Max Depth - Tapers	11.8SQFT x 1ft2in		13.8 CF	@ Pier 22	1 CY
Main Reinforcing	3 - #7 Bars		7.75 ft		@ Pier 22	48 LB
Secondary Reinforcing	6 - #5 Bars		6.08 ft		@ Pier 22	55 LB
Vertical Hoops	5 - #5 Hoops		8.33 ft		@ Pier 22	63 LB
Horiz. Hoops	4 - #5 Hoops		14.33 ft		@ Pier 22	86 LB
Pier 23 Extension		8.1SQFT x 2ft		16.2 CF	@Pier 23	1 CY
Doweled Reinforcing	10 - #5 Bars		2.58 ft		@Pier 23	39 LB
Doweled Reinforcing	6 - #5 Bars		3.58 ft		@Pier 23	32 LB
Horiz. Hoops	3 - #5 bars		15.00 ft		@Pier 23	68 LB
Elastomeric Bearing Pads						2
Removal - Curb						84.91 LF
Removal - Barrier						84.91 LF
Removal - Sidewalk						339.64 SF

Concrete		22 CY
Reinforcing Steel		3972 LB
Reinforcing Steel (Doweled)		271 LB
Light Poles		1 EA
Structural Steel		19716 LB
Bearing Pads		2
Removal LF		169.82 LF
Removal SF		339.64 SF
Remove Light Poles	1 EA	
Drill Hole 5/8"		.00 LF
Drill Hole 3/4"		52.50 LF
Epoxy Bonding Compound		0.00 EA

					Surface Prep.	93.67 SF
Segment 7 - 10ft		287.32 ft		4 span	20 Cantilevers	
Concrete Slab	4" x 10'			3.33 SF		35 CY
Slab Reinforcing		10.50 ft	#4 Transverse @ 4"	21.04	LB/FT	6046 LB
		11.00 ft	#4 Longitundinal	7.35	LB/FT	2111 LB
Pedestrian Rail	See Calculation			36.6	LB/FT	10516 LB
BP Rail	See Calculation			6.7	LB/FT	1925 LB
Concrete Barrier	38" Single Slope (8" top)			3.07 SF		33 CY
	Surface Prep					316.96 SF
Barrier Reinforcing		3.72 ft	#5 drilled through	3.88	LB @ 9" in 10' @18" else	950 LB
		.75 ft	3/4" Hole			183.75 ft
		7.92 ft	#4 Hoop @ 12"	5.45	LB/FT	1566 LB
		8.00 ft	4 - #5 Horiz EF	8.34	LB/FT	2397 LB
	PL 3/8"x4"x0'4"		.0035 CF	1.70	LB @ 9" in 10' @18" else	417 LB
Edge Beam	HSS 8x6x3/16		17.10 plf			4913 LB
Horiz. Tension Strut @ Canti.	MC8x8.5 2@18ft		18.00 ft	153.00	LB/Cantilever	3060 LB
Vert. Tension Strut @ Canti.	MC8x8.5 2@18ft		3.00 ft	25.50	LB/Cantilever	510 LB
Diag. Compr. Strut @ Canti.	HSS 8x4x3/16		14.54 plf	124.23	LB/Cantilever	2485 LB
Horiz. Compr. Strut @ Canti.	HSS 8x4x3/16		14.54 plf	5.00	LB/Cantilever	100 LB
Bolts Through Cantilever	3 - 5/8" x 13" @18ft		.01 CF	3.65	LB/Cantilever	73 LB
		1.00 ft	6 - 3/4" Holes @ Cantilevers			120.00 ft
Compression Anchor Plate	1"x16"x6"		.06 CF	27.22	LB/Cantilever	544 LB
Standard Luminarie Poles	One per span					4 EA
Concrete Block		4.67 CF			4 Locations	1 CY
Transverse Reinf		5.33 ft	4 - #4 bars		4 Locations	59 LB
Longitudinal Reinf		6.00 ft	4 - #4 bars		4 Locations	66 LB
Headed Anchor Studs	4- 5/8" x 8"			0.56	LB/Stud	9 LB
			Number of Epoxy Anchors			80
		.67 ft	4 - 3/4" Holes @ Cantilevers			53.36 ft
Removal - Curb						287.32 LF
Removal - Barrier						287.32 LF
Removal - Sidewalk						1149.28 SF

Concrete	69 CY
Reinforcing Steel	12246 LB
Reinforcing Steel (Doweled)	950 LB
Light Poles	4 EA
Structural Steel	24552 LB
Removal LF	574.64 LF
Removal SF	1149.28 SF
Remove Light Poles	4 EA
Drill Hole 5/8"	.00 LF
Drill Hole 3/4"	357.11 LF
Epoxy Bonding Compound	80.00 EA
Surface Prep.	316.96 SF

Segment 1 - 10ft 187.09 ft Approx

Concrete CAD Projected area			19.3	SQFT	134 CY
Slab Reinforcing	#5 @ 6" Top Bar	25.67 ft	17.66	LB/FT	3304 LB
	#4 @ 6" Bot Bar	18.67 ft	12.84	LB/FT	2403 LB
	#4 @ 6" TS bot	21.50 ft	14.79	LB/FT	2767 LB
	#4 @ T&B Trans	42.00 ft	28.90	LB/FT	5406 LB
	#4 @ 12" Barrier Back	3.75 ft	2.58	LB/FT	483 LB
	#5 @ 9" Barrier Front	6.11 ft	6.37	LB/FT	1192 LB
	8 - #5 horiz bars	8.00 ft	8.34	LB/FT	1561 LB
Pedestrian Rail	See Calculation		36.6	LB/FT	6847 LB
BP Rail	See Calculation		6.7	LB/FT	1254 LB
Removal - Curb					187.09 LF
Removal - Barrier					187.09 LF
Removal - Sidewalk			6.67 ft	width of removal	1247.27 SF

Concrete	134 CY
Reinforcing Steel	17116 LB
Light Poles	1
Structural Steel	8101 LB
Removal LF	374.18 LF
Removal SF	1247.27 SF
Remove Light Poles	1

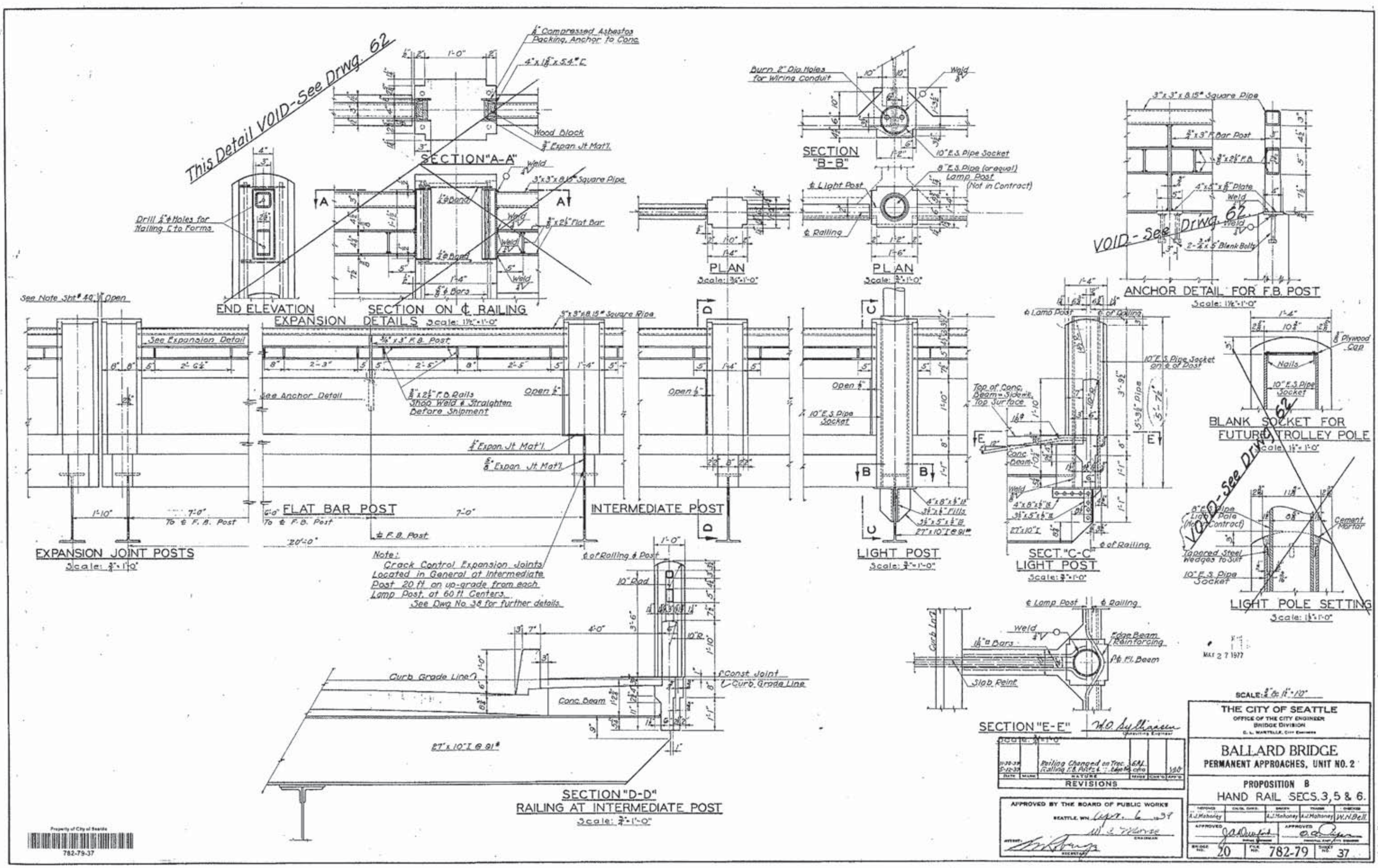
Segment 8 - 10ft 26.67 ft Approx

Concrete CAD Projected area			32.8	SQFT	32 CY
Slab Reinforcing	#6 @ 6" Top Bar	22.67 ft	15.59	LB/FT	416 LB
	#4 @ 12" Bot Bar	9.00 ft	6.19	LB/FT	165 LB
	#5 @ 12" TS bot	16.75 ft	11.52	LB/FT	307 LB
	#5 @ 12" TS top	13.25 ft	9.12	LB/FT	243 LB
	#4 @ 12" T&B Trans	16.00 ft	11.01	LB/FT	294 LB
	#5 @ 12" T&B Trans	32.00 ft	22.02	LB/FT	587 LB
	#4 @ 12" Barrier Back	3.58 ft	2.47	LB/FT	66 LB
	#5 @ 9" Barrier Front	5.56 ft	5.79	LB/FT	155 LB
	8 - #5 horiz bars	8.00 ft	8.34	LB/FT	223 LB
Pedestrian Rail	See Calculation		36.6	LB/FT	976 LB
BP Rail	See Calculation		6.7	LB/FT	179 LB
Removal - Curb					26.67 LF
Removal - Barrier					26.67 LF
Removal - Sidewalk			14.17 ft	width of removal	377.78 SF
				Concrete	32 CY
				Reinforcing Steel	2455 LB
				Structural Steel	1155 LB
				Removal LF	53.33 LF
				Removal SF	377.78 SF

Appendix H
Structural Calculations

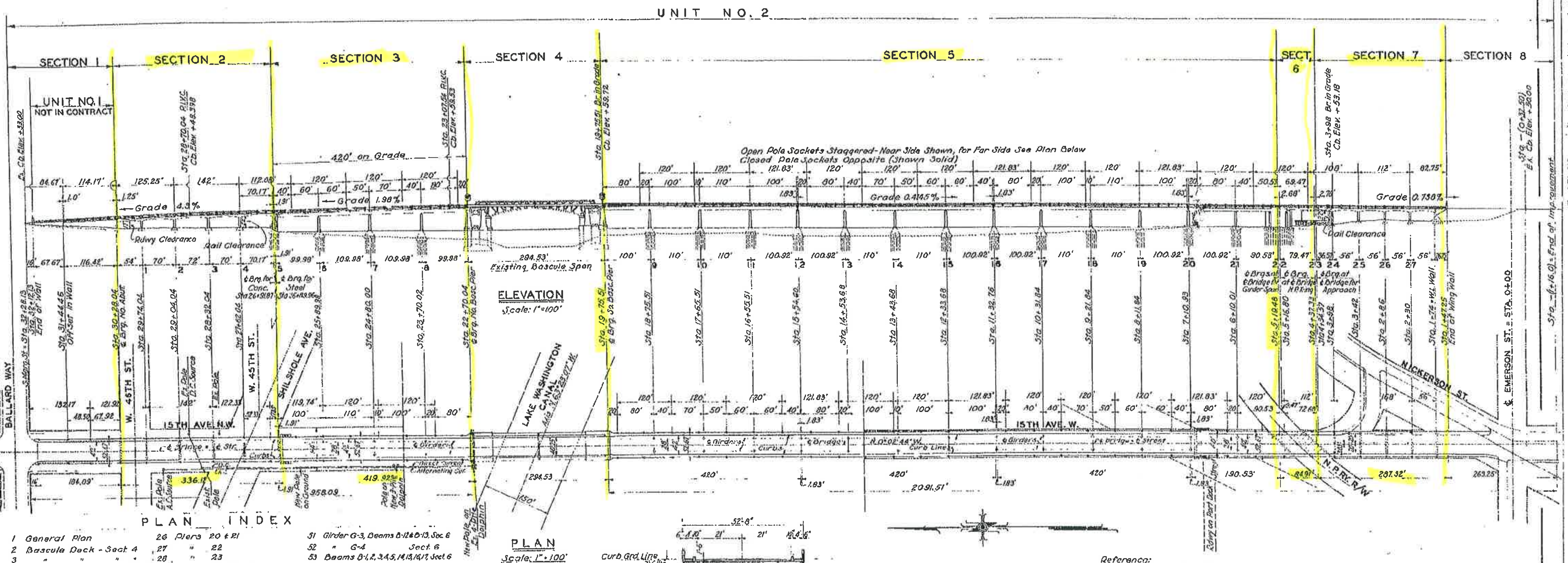
Appendix H – Ballard Path Widening Calculations

<u>General</u>	
1	Existing Sidewalk
2 – 3	Segment Limits and Mass
4	Existing Material Properties
5 – 18	Barrier & Railing
19	WSDOT Epoxy Resin Table
20	WSDOT Steel Light Standard Elbow
<u>6' Alternative</u>	
21 – 28	Segment 1
29 – 48	Segment 2
49 – 83	Segments 3 & 5
84 – 111	Segment 6
112 – 134	Segment 7
135 – 141	Segment 8
<u>10' Alternatives</u>	
142 – 147	Segment 1
148 – 167	Segment 2
168 – 189	Segments 3 & 5
190 – 247	Segment 6
248 – 265	Segment 7
266 – 272	Segment 8



This Detail VOID-See Drwg. 62
VOID-See Drwg. 62
SECTION "A-A"
END ELEVATION
SECTION ON ¢ RAILING
EXPANSION DETAILS Scale: 1½"=1'-0"
¼" Compressed Asbestos Packing, Anchor to Conc.
4" x 1⅝" x 5.4# C
Wood Block
¾" Expan. Jt Mat'l.
3" x 3" x 8.15# Square Pipe
⅜" x 2½" Flat Bar
Drill ½" ϕ Holes for Railing ¢ to Forms
Burn 2" Dia. Holes for Wiring Conduit
SECTION "B-B"
10" E.S. Pipe Socket
8" E.S. Pipe (or equal) Lamp Post (Not in Contract)
¢ Light Post
¢ Railing
PLAN
3" x 3" x 8.15# Square Pipe
⅜" x 3" F.Bar Post
4" x 5" x ⅝" Plate
2-¾" x 5" Blank Bolts
ANCHOR DETAIL FOR F.B. POST Scale: 1½"=1'-0"
See Expansion Detail
See Anchor Detail
⅜" x 2½" F.B. Rails Shop Weld & Straighten Before Shipment
Open ⅛"
¾" Expan. Jt. Mat'l.
⅝" Expan. Jt. Mat'l.
10" E.S. Pipe Socket
EXPANSION JOINT POSTS Scale: ¾"=1'-0"
FLAT BAR POST
INTERMEDIATE POST
LIGHT POST Scale: ¾"=1'-0"
SECT. "C-C" LIGHT POST Scale: ¾"=1'-0"
10" E.S. Pipe Socket on ¢ of Post
Top of Conc. Beam = Sidewalk Top Surface
27" x 10" I @ 91#
Plywood Cap
Nails
10" E.S. Pipe Socket
BLANK SOCKET FOR FUTURE TROLLEY POLE Scale: 1½"=1'-0"
8" E.S. Pipe Light Pole (Not in Contract)
Cement Mortar
Tapered Steel Wedges to Suit
10" E.S. Pipe Socket
LIGHT POLE SETTING Scale: 1½"=1'-0"
Note:
Crack Control Expansion Joints Located in General at Intermediate Post 20 ft. on up-grade from each Lamp Post, at 60 ft Centers.
See Dwg. No. 38 for further details.
Curb Grade Line
Conc. Beam
Const. Joint
Curb Grade Line
27" x 10" I @ 91#
SECTION "D-D"
RAILING AT INTERMEDIATE POST
Scale: ¾"=1'-0"
¢ Lamp Post
¢ Railing
Edge Beam Reinforcing
Pt. Fl. Beam
Slab Reinf.
SECTION "E-E" Scale: 1½"=1'-0"
MAY 27 1977
SCALE: ¾" & 1½" = 1'0"
THE CITY OF SEATTLE
OFFICE OF THE CITY ENGINEER
BRIDGE DIVISION
BALLARD BRIDGE
PERMANENT APPROACHES, UNIT NO. 2
PROPOSITION B
HAND RAIL SECS. 3, 5 & 6.
REVISIONS
APPROVED BY THE BOARD OF PUBLIC WORKS
SEATTLE, WN. Apr. 6 1939
A.J. Mahoney
W.H. Bell
BRIDGE NO. 20
FILE NO. 782-79
SHEET NO. 37
782-79-37

61 Sheets
+11 Sheets
72



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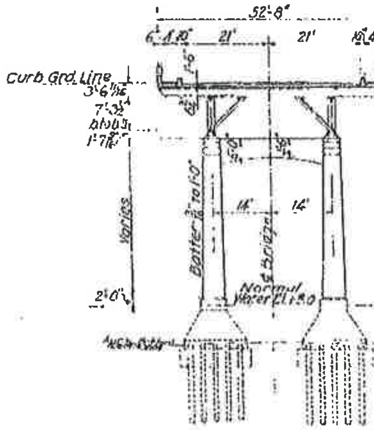

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782-79-1

W.O. Sylliaasen
Bridge Engineer

Plat & Engs. RR Xing Water Elevation

REVISIONS

SCALE: 1" = 20' & 100'

THE CITY OF SEATTLE
OFFICE OF THE CITY ENGINEER
BRIDGE DIVISION

BALLARD BRIDGE
PERMANENT APPROACHES, UNIT NO. 2
PROPOSITION B
GENERAL PLAN

20 782-79

24041

Corney, James

From: Banks, Greg
Sent: Friday, December 14, 2012 11:20 AM
To: Corney, James
Subject: FW: Superstructure mass

See below.

From: Stringer, Stuart
Sent: Friday, December 14, 2012 11:04 AM
To: Banks, Greg
Subject: RE: Superstructure mass

Segment 2:	9,000 kips
Segment 3:	3,863 kips
Segment 5:	13,395 kips
Segment 6:	645 kips
Segment 7:	5,350 kips

Stuart J. Stringer, PE
Engineer
Voice 206-214-1026
Fax 206-431-2250
Email stuart.stringer@abam.com

BergerABAM
33301 Ninth Avenue South, Suite 300
Federal Way, Washington 98003-2600
http://www.abam.com

From: Banks, Greg
Sent: Thursday, December 13, 2012 2:25 PM
To: Stringer, Stuart
Subject: Superstructure mass

Stuart:

Would you be able to tell me what the weight of the superstructure is for each segment of the Ballard Bridge Approach Structures?

Regards-
Greg

Greg Banks, P.E.
Senior Engineer
Voice 206/431-2253
Fax 206/431-2250
Email greg.banks@abam.com

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Federal Way, Washington 98003-2600
http://www.abam.com

- Segment 2 – Original structure: f’c = 8 ksi, fy = 33 ksi
- Segment 2 – Box girder widening: f’c = 5 ksi, fy = 40 ksi
- Segment 3 and 5: fy = 45 ksi, fu = 70 ksi (A94 silicone steel)
- Segment 6: fy = 45 ksi, fu = 70 ksi (A94 silicone steel)
- Segment 7 – Original structure: f’c = 8 ksi, fy = 33 ksi
- Segment 7 – Widening: f’c = 5 ksi, fy = 40 ksi

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LRFD Barrier Rail Design

Weight of the Barrier: $W_b := 490\text{plf}$

Strength of Reinforcing Steel $F_y := 60\text{ksi}$

Thickness of the Deck: $t_d := 6.5\text{in}$
(REF SHT 60)

Compressive Strength of Concrete $f'_c := 4\text{ksi}$

- *A combination barrier in conjunction with a raised curb and sidewalk is used only on low-speed highways (* $\text{speed} \le 45\text{mph}$*)*
- *On high-speed highways (* $\text{speed} \ge 50\text{mph}$*), the pedestrian or bicycle path should have both an outboard pedestrian or bicycle railing and an inboard combination railing.* *(AASHTO C13.7.1.1)*

Designing the bridge for Test Level 4 (**TL4**) - Recommended for most bridges (AASHTO 13.7.2)

Table A13.2-1 Design Forces for Traffic Railings.

Design Forces and Designations	Railing Test Levels					
	TL-1	TL-2	TL-3	TL-4	TL-5	TL-6
F_t Transverse (kips)	13.5	27.0	54.0	54.0	124.0	175.0
F_L Longitudinal (kips)	4.5	9.0	18.0	18.0	41.0	58.0
F_v Vertical (kips) Down	4.5	4.5	4.5	18.0	80.0	80.0
L_t and L_L (ft.)	4.0	4.0	4.0	3.5	8.0	8.0
L_v (ft.)	18.0	18.0	18.0	18.0	40.0	40.0
H_e (min) (in.)	18.0	20.0	24.0	32.0	42.0	56.0
Minimum H Height of Rail (in.)	27.0	27.0	27.0	32.0	42.0	90.0

Transverse Load: $F_t := 54\text{kips}$

Longitudinal Load: $F_L := 18\text{kips}$

Vertical Force (Downward): $F_v := 18\text{kips}$

Minimum effective height of vehicle rollover: $H_{e.min} := 32\text{in}$

Longitudinal length of distribution of friction force: $L_L := 3.5\text{ft}$

Longitudinal length of distribution of impact force: $L_t := 3.5\text{ft}$

Longitudinal distribution of vertical force on top of railing: $L_v := 18\text{ft}$

Design Height of Barrier: $H := 32\text{in}$

Design Procedure for Concrete Railings (A13.3.1)

Two tests are required: Test for impacts within wall segments and tests for impacts at the end of wall or joint

Within a wall segment (A13.3.1-1, 2)

Total transverse resistance of the railing

$$R_w = \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(8 \times M_b + 8 \times M_w + \frac{M_c \times L_c^{\,2}}{H}\right)$$

M_w *is the flexural resistance of the wall* ***about its vertical axis (kip-ft)***

Critical Wall Length over which the yield line mechanism occurs

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + 8\,H\left(\frac{M_b + M_w}{M_c}\right)}$$

M_b *is the additional flexural resistance of the beam due to wall thickening at the top, in addition to* M_w*, if any (kip-ft)*

At end of wall or at joint (A13.3.1-3, 4)

Total transverse resistance of the railing

$$R_w = \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(M_b + M_w + \frac{M_c \times L_c^{\,2}}{H}\right)$$

M_c *is the flexural resistance of cantilevered walls* ***about an axis parallel to the longitudinal axis*** *of the bridge (kip-ft/ft)*

Critical Wall Length over which the yield line mechanism occurs

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H\left(\frac{M_b + M_w}{M_c}\right)}$$

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Flexural Resistance about the Vertical Axis

Two moment capacity values are accepted in the AASHTO equation. The first, M_w, is a baseline value. This represents a barrier which is uniform in shape and reinforcement. Because most barriers are not uniform in shape and reinforcement, a second value , M_b, has been created to represent a thickening or extra reinforcing at the top of the

Baseline Moment about vertical axis M_w

Horiz. bar (Typical size) on one face of wall: $H_{no} := 5$ $\boxed{d_{no_{H_{no}}} = 0.625 \times in}$ $d_{no_{H_{no}}} = 0.625 \times in$ $A_{no_{H_{no}}} = 0.31 \times in^2$

Vertical Bar number: $V_{no} := 4$ $\boxed{d_{no_{V_{no}}} = 0.5 \times in}$ $d_{no_{V_{no}}} = 0.5 \times in$ $A_{no_{V_{no}}} = 0.2 \times in^2$

Number of bars along face (total including strong section): $n_{bars} := 4$ $\boxed{\beta_1 = 0.85}$

Clear Cover to bar: $clear := 2.5in + d_{no_{V_{no}}} = 3 \times in$ Height of Barrier Rail: $H = 32 \times in$

Average Thickness of Barrier Rail not including any discrete width change at top of rail: $t_{avg} := 8in + \frac{4}{21} \times \frac{H}{2} = 11.048 \times in$ $h_{mem} := t_{avg}$ $spacing_{bv} := \frac{H}{n_{bars}} = 8 \times in$

Area of tension steel: $A_{s.bv} := \frac{A_{no_{H_{no}}}}{spacing_{bv}}$ $A_{s.bv} \times H = 1.24 \times in^2$ depth to rebar: $d := h_{mem} - clear - \frac{diam_{deform_{H_{no}}}}{2} = 7.7 \times in$

Moment Capacity for full yielding of steel

$a_{bv} := \frac{A_{s.bv} \times F_y}{0.85 \times f'_c} = 0.684 \times in$ $M_w := 1.0 \times A_{s.bv} \times F_y \times \left(d - \frac{a_{bv}}{2}\right) \times H$ $\boxed{M_w = 45.644 \times kip\ ft}$

Additional Flexural Resistance at the top of the barrier (M_b) $\boxed{\beta_1 = 0.85}$

Analyze the strong top portion of the barrier for additional moment capacity

Additional Resistance: $\boxed{M_b := 0}$ $\boxed{M_w + M_b = 45.644\ ft \times kips}$

Flexural Resistance about the axis parallel to the longitudinal axis of the bridge
"Where the width of the concrete railing varies along the height, Mc used in Eqs. 1 through 4 for wall resistance should be taken as the average of its value along the height of the railing." (AASHTO CA13.3.1)

This statement should also apply to variations wall resistance due to reinforcing along the height of the railing. Weighted average for resistances due to reinforcing changes should be used.

The lower portion of the barrier reinforcement anchored into deck slab

The basic development length for a hooked bar of yield strength of 60ksi is determined by 5.11.2.4 (Applies to all bar sizes)

$\boxed{V_{no} = 4}$ $\qquad d_{no_{V_{no}}} = 0.5 \times in$

$$l_d := \frac{38.0 \times d_{no_{V_{no}}}}{\sqrt{f'_c \left(\frac{1}{ksi}\right)}} \qquad l_d = 9.5 \times in \qquad l_{d.min} := \max\left(8 \times d_{no_{V_{no}}}, 6in\right) \qquad l_{d.min} = 6 \times in$$

$$l_d := \frac{\max(F_y, 60ksi)}{60ksi} \times \max(l_d, l_{d.min}) \qquad \boxed{l_d = 9.5 \times in}$$

For No. 11 bar and smaller, side cover (normal to plane of hook) not less than 2.5 in., and for 90-deg hook, cover on bar extension beyond hook not less than 2in:

$$\boxed{l_{d.lower} := \max(0.7 \times l_d, l_{d.min}) = 6.65 \times in}$$

Lower Section Moment Capacity

$A_{no_{V_{no}}} = 0.2 \times in^2$

Spacing of bars Near edges: $\boxed{spacing_{near} := 9in}$

Away from Edges: $\boxed{spacing_{away} := 24in}$ $\qquad \boxed{d_{no_{V_{no}}} = 0.5 \times in}$ $\qquad diam_{deform_{V_{no}}} = 0.563 \times in$

Clear Cover to bar: $\boxed{clear := 2in}$

Thickness of lower section of Barrier: $\boxed{h_{mem} := 8in + \frac{4}{21} \times H = 14.095 \times in}$ $\qquad \boxed{\beta_1 = 0.85}$

Area of tension steel: $A_{s.near} := \frac{A_{no_{V_{no}}}}{spacing_{near}} = 0.267 \times \frac{in^2}{ft}$

$$A_{s.away} := \frac{A_{no_{V_{no}}}}{spacing_{away}} = 0.1 \times \frac{in^2}{ft}$$

depth to rebar: $d := h_{mem} - clear - \frac{diam_{deform_{V_{no}}}}{2} = 11.81 \times in$

Moment Capacity for full capacity of steel

$$a_{near} := \frac{A_{s.near} \times F_y}{0.85 \times f'_c} = 0.392 \times in \qquad M_{c.near} := 1.0 \times A_{s.near} \times F_y \times \left(d - \frac{a_{near}}{2}\right) \qquad \boxed{M_{c.near} = 15.491 \times \frac{kip\ ft}{ft}}$$

$$a_{away} := \frac{A_{s.away} \times F_y}{0.85 \times f'_c} = 0.147 \times in \qquad M_{c.away} := 1.0 \times A_{s.away} \times F_y \times \left(d - \frac{a_{away}}{2}\right) \qquad \boxed{M_{c.away} = 5.87 \times \frac{kip\ ft}{ft}}$$

Total transverse resistance of the railing

Impact within a wall segment

Critical Wall Length over which the yield line mechanism occurs

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + 8\,H\left(\frac{M_b + M_w}{M_{c.away}}\right)}$$

$\boxed{L_c = 14.748\,\text{ft}}$

$L_{c.center} := L_c$

$$R_w := \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(8 \times M_b + 8 \times M_w + \frac{M_{c.away} \times L_c^{\,2}}{H}\right)$$

$\boxed{R_w = 64.929 \times \text{kips}}$

$R_{w.center} := R_w$

The demand is: $\boxed{F_t = 54 \times \text{kips}}$

$$\frac{F_t}{R_w} = 0.832$$

$\boxed{Check_{C.D}(R_w, F_t) = \text{"SATISFACTORY"}}$

At end of wall or at joint

Critical Wall Length over which the yield line mechanism occurs

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + H\left(\frac{M_b + M_w}{M_{c.near}}\right)}$$

$\boxed{L_c = 5.055\,\text{ft}}$

$L_{c.joint} := L_c$

$$R_w := \left(\frac{2}{2 \times L_c - L_t}\right) \times \left(M_b + M_w + \frac{M_{c.near} \times L_c^{\,2}}{H}\right)$$

$\boxed{R_w = 58.7 \times \text{kips}}$

$R_{w.joint} := R_w$

The demand is: $\boxed{F_t = 54 \times \text{kips}}$

$$\frac{F_t}{R_w} = 0.92$$

$\boxed{Check_{C.D}(R_w, F_t) = \text{"SATISFACTORY"}}$

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Flexure and Shear Resistance of Round HSS Members (AASHTO 6.12)

$\varphi_f := 1.0$ $E := 29000ksi$

$\varphi_v := 1.0$ $F_y := 35ksi$

2.5in STD Steel Pipe

Flexure Check

$A_g := 1.59in^2$ Gross Area of HSS

$D := 2.88in$ Outside Diameter of Tube

$t := 0.189in$ Thickness of Tube

$Z := 1.37in^3$ Plastic Modulus

$S := 1.01in^3$ Section Modulus

Limitation: $DT := \frac{D}{t} = 15.238$ shall not exceed $0.45 \times \frac{E}{F_y} = 373$ $\text{Check}_{C.D}\left(0.45 \times \frac{E}{F_y}, \frac{D}{t}\right) = \text{"SATISFACTORY"}$

$$M_{n.yield} := F_y \times Z = 47.95\,kip \times in$$

$$M_{n.lb} := \left(\frac{0.021 \times E}{\frac{D}{t}} + F_y\right) \times S = 75.715\,kip \times in$$

$$M_{n.lb.thin} := \left(\frac{0.33 \times E}{\frac{D}{t}}\right) \times S = 634.312\,kip \times in$$

$$M_n := \begin{cases} M_{n.yield} \\ M_{n.lb} & \text{if } DT > 0.07 \times \frac{E}{F_y} \\ M_{n.lb.thin} & \text{if } DT > 0.31 \times \frac{E}{F_y} \\ \text{"NA"} & \text{if } DT > 0.45 \times \frac{E}{F_y} \end{cases} = 47.95\,kip \times in$$

$$\varphi Mn_{2.5STD} := \varphi_f \times M_n = 47.95\,kip \times in$$

Shear Check

$L_v := 54in$ Distance between Maximum to Zero Shear

$$F_{v.n} := 0.58 \times F_y = 20.3\,ksi$$

$$F_{cr_1} := \min\left[\frac{1.60 \times E}{\sqrt{\frac{L_v}{D}} \times \left(\frac{D}{t}\right)^{\frac{5}{4}}}, 0.58 \times F_y\right] = 20.3\,ksi$$

$$F_{cr_2} := \min\left[\frac{0.78 \times E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}, 0.58 \times F_y\right] = 20.3\,ksi$$

$$F_{cr} := \min(F_{cr}) = 20.3\,ksi$$

$$V_n := 0.5 \times F_{cr} \times A_g = 16.138\,kips$$

$$\varphi Vn_{2.5STD} := \varphi_v \times V_n = 16.138\,kips$$

Flexure and Shear Resistance of Round HSS Members (AASHTO 6.12)

$\varphi_f := 1.0$ $\quad E := 29000ksi$

$\varphi_v := 1.0$ $\quad F_y := 42ksi$

HSS 2.875x0.25

Flexure Check

$A_g := 1.93in^2$ Gross Area of HSS

$D := 2.875in$ Outside Diameter of Tube

$t := 0.233in$ Thickness of Tube

$Z := 1.63in^3$ Plastic Modulus

$S := 1.18in^3$ Section Modulus

Limitation: $DT := \frac{D}{t} = 12.339$ shall not exceed $0.45 \times \frac{E}{F_y} = 311$ $\quad Check_{C.D}\left(0.45 \times \frac{E}{F_y}, \frac{D}{t}\right) = \text{"SATISFACTORY"}$

$$M_{n.yield} := F_y \times Z = 68.46\,kip \times in$$

$$M_{n.lb} := \left(\frac{0.021 \times E}{\frac{D}{t}} + F_y\right) \times S = 107.799\,kip \times in$$

$$M_{n.lb.thin} := \left(\frac{0.33 \times E}{\frac{D}{t}}\right) \times S = 915.192\,kip \times in$$

$$M_n := \begin{vmatrix} M_{n.yield} \\ M_{n.lb} \text{ if } DT > 0.07 \times \frac{E}{F_y} \\ M_{n.lb.thin} \text{ if } DT > 0.31 \times \frac{E}{F_y} \\ \text{"NA" if } DT > 0.45 \times \frac{E}{F_y} \end{vmatrix} = 68.46\,kip \times in$$

$$\varphi Mn_{HSS2.875x0.25} := \varphi_f \times M_n = 68.46\,kip \times in$$

Shear Check

$L_v := 54in$ Distance between Maximum to Zero Shear

$$F_{v.n} := 0.58 \times F_y = 24.36\,ksi$$

$$F_{cr_1} := \min\left[\frac{1.60 \times E}{\sqrt{\frac{L_v}{D}} \times \left(\frac{D}{t}\right)^{\frac{5}{4}}}, 0.58 \times F_y\right] = 24.36\,ksi$$

$$F_{cr_2} := \min\left[\frac{0.78 \times E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}, 0.58 \times F_y\right] = 24.36\,ksi$$

$$F_{cr} := \min(F_{cr}) = 24.36\,ksi$$

$$V_n := 0.5 \times F_{cr} \times A_g = 23.507\,kips$$

$$\varphi Vn_{HSS2.875x0.25} := \varphi_v \times V_n = 23.507\,kips$$

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Flexure and Shear Resistance of Round HSS Members (AASHTO 6.12)

$\varphi_f := 1.0$ $E := 29000\text{ksi}$

$\varphi_v := 1.0$ $F_y := 42\text{ksi}$

HSS 2.875x0.203

Flexure Check

$A_g := 1.59\text{in}^2$ Gross Area of HSS

$D := 2.875\text{in}$ Outside Diameter of Tube

$t := 0.189\text{in}$ Thickness of Tube

$Z := 1.37\text{in}^3$ Plastic Modulus

$S := 1.01\text{in}^3$ Section Modulus

Limitation: $DT := \frac{D}{t} = 15.212$ shall not exceed $0.45 \times \frac{E}{F_y} = 311$ $\text{Check}_{C.D}\left(0.45 \times \frac{E}{F_y}, \frac{D}{t}\right) = \text{"SATISFACTORY"}$

$M_{n.yield} := F_y \times Z = 57.54\,\text{kip} \times \text{in}$

$$M_{n.lb} := \left(\frac{0.021 \times E}{\frac{D}{t}} + F_y\right) \times S = 82.855\,\text{kip} \times \text{in}$$

$$M_{n.lb.thin} := \left(\frac{0.33 \times E}{\frac{D}{t}}\right) \times S = 635.415\,\text{kip} \times \text{in}$$

$$M_n := \begin{cases} M_{n.yield} & \\ M_{n.lb} & \text{if } DT > 0.07 \times \frac{E}{F_y} \\ M_{n.lb.thin} & \text{if } DT > 0.31 \times \frac{E}{F_y} \\ \text{"NA"} & \text{if } DT > 0.45 \times \frac{E}{F_y} \end{cases} = 57.54\,\text{kip} \times \text{in}$$

$\varphi Mn_{HSS2.875x0.203} := \varphi_f \times M_n = 57.54\,\text{kip} \times \text{in}$

Shear Check

$L_v := 54\text{in}$ Distance between Maximum to Zero Shear

$F_{v.n} := 0.58 \times F_y = 24.36\,\text{ksi}$

$$F_{cr_1} := \min\left[\frac{1.60 \times E}{\sqrt{\frac{L_v}{D}} \times \left(\frac{D}{t}\right)^{\frac{5}{4}}}, 0.58 \times F_y\right] = 24.36\,\text{ksi}$$

$$F_{cr_2} := \min\left[\frac{0.78 \times E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}, 0.58 \times F_y\right] = 24.36\,\text{ksi}$$

$F_{cr} := \min(F_{cr}) = 24.36\,\text{ksi}$

$V_n := 0.5 \times F_{cr} \times A_g = 19.366\,\text{kips}$

$\varphi Vn_{HSS2.875x0.203} := \varphi_v \times V_n = 19.366\,\text{kips}$

Flexure and Shear Resistance of Round HSS Members (AASHTO 6.12)

$\varphi_f := 1.0$ $\quad E := 29000ksi$

$\varphi_v := 1.0$ $\quad F_y := 42ksi$

HSS 1.900x0.145

Flexure Check

$A_g := 0.749in^2$ Gross Area of HSS

$D := 1.900in$ Outside Diameter of Tube

$t := 0.135in$ Thickness of Tube

$Z := 0.421in^3$ Plastic Modulus

$S := 0.309in^3$ Section Modulus

Limitation: $DT := \frac{D}{t} = 14.074$ shall not exceed $0.45 \times \frac{E}{F_y} = 311$ $\quad Check_{C.D}\left(0.45 \times \frac{E}{F_y}, \frac{D}{t}\right) = \text{"SATISFACTORY"}$

$$M_{n.yield} := F_y \times Z = 17.682\,kip \times in$$

$$M_{n.lb} := \left(\frac{0.021 \times E}{\frac{D}{t}} + F_y\right) \times S = 26.349\,kip \times in$$

$$M_{n.lb.thin} := \left(\frac{0.33 \times E}{\frac{D}{t}}\right) \times S = 210.112\,kip \times in$$

$$M_n := \begin{vmatrix} M_{n.yield} \\ M_{n.lb} \text{ if } DT > 0.07 \times \frac{E}{F_y} \\ M_{n.lb.thin} \text{ if } DT > 0.31 \times \frac{E}{F_y} \\ \text{"NA" if } DT > 0.45 \times \frac{E}{F_y} \end{vmatrix} = 17.682\,kip \times in$$

$$\varphi Mn_{HSS1.900x0.145} := \varphi_f \times M_n = 17.682\,kip \times in$$

Shear Check

$L_v := 54in$ Distance between Maximum to Zero Shear

$$F_{v.n} := 0.58 \times F_y = 24.36\,ksi$$

$$F_{cr_1} := \min\left[\frac{1.60 \times E}{\sqrt{\frac{L_v}{D}} \times \left(\frac{D}{t}\right)^{\frac{5}{4}}}, 0.58 \times F_y\right] = 24.36\,ksi$$

$$F_{cr_2} := \min\left[\frac{0.78 \times E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}}, 0.58 \times F_y\right] = 24.36\,ksi$$

$$F_{cr} := \min(F_{cr}) = 24.36\,ksi$$

$$V_n := 0.5 \times F_{cr} \times A_g = 9.123\,kips$$

$$\varphi Vn_{HSS1.900x0.145} := \varphi_v \times V_n = 9.123\,kips$$

Ballard Pedestrian Rail Design (AASHTO 13.8.2)

Loads: $w := 50\text{plf}$ Vertically and Laterally acting Simultaneously

$P := 200\text{lb}$ Point Load laterally at the top lateral member

Post (Interior Post Governs)

Post Spacing: $L := 7\text{ft}$

Design Reaction Force: $R := 0.20\text{kips} + 0.050\text{klf} \times L = 550\,\text{lb}$ (EQ 13.8.2-1)

Maximum design height of post: (where post is welded below 4" walkway slab) $h := 54\text{in} + 4\text{in} = 58\,\text{in}$

Moment in Post: $M_{post} := R \times h = 31.9\,\text{kip} \times \text{in}$

Shear in Post: $V_{post} := R = 550\,\text{lb}$

Factored Moment: $\boxed{M_{u.post} := 1.75 \times M_{post} = 55.825\,\text{kip} \times \text{in}}$

Factored Shear: $\boxed{V_{u.post} := 1.75 \times V_{post} = 0.963\,\text{kips}}$

$\varphi Mn_{HSS2.875x0.25} = 68.46\,\text{kip} \times \text{in}$

$\varphi Vn_{HSS2.875x0.25} = 23.507\,\text{kips}$

HSS 2.875 x 0.25 is Satisfactory

Lateral Beam

Design Load: (Vertical and lateral distributed load acting simultaneously) $distr := \sqrt{w^2 + w^2} = 70.711\,\frac{\text{lb}}{\text{ft}}$

Moment in Beam: $M_{beam} := \frac{distr \times L^2}{8} = 5.197\,\text{kip} \times \text{in}$ (distributed load governs) $\frac{P \times L}{4} = 4.2\,\text{kip} \times \text{in}$

Factored Moment: $\boxed{M_{u.beam} := 1.75 \times M_{beam} = 9.095\,\text{kip} \times \text{in}}$ $\varphi Mn_{HSS2.875x0.203} = 57.54\,\text{kip} \times \text{in}$

Shear in Beam: $V_{beam} := \frac{distr \times L}{2} = 0.247\,\text{kips}$ (distributed load governs)

Factored Shear: $\boxed{V_{u.beam} := 1.75 \times V_{beam} = 0.433\,\text{kips}}$ $\varphi Vn_{HSS2.875x0.203} = 19.366\,\text{kip}$

HSS 2.875 x 0.203 is Satisfactory

Vertical Interior Beam

Design Load: $w = 50\,\text{plf}$

Moment in Beam: $M_{beam} := \frac{w \times L^2}{8} = 3.675\,\text{kip} \times \text{in}$ (distributed load governs) $\frac{P \times L}{4} = 4.2\,\text{kip} \times \text{in}$

Factored Moment: $\boxed{M_{u.beam} := 1.75 \times M_{beam} = 6.431\,\text{kip} \times \text{in}}$ $\varphi Mn_{HSS1.900x0.145} = 17.682\,\text{kip} \times \text{in}$

Shear in Beam: $V_{beam} := \frac{w \times L}{2} = 0.175\,\text{kips}$ (distributed load governs)

Factored Shear: $\boxed{V_{u.beam} := 1.75 \times V_{beam} = 0.306\,\text{kips}}$ $\varphi Vn_{HSS1.900x0.145} = 9.123\,\text{kip}$

HSS 1.900 x 0.145 is Satisfactory

Ballard Pedestrian Rail

Number of Verticals

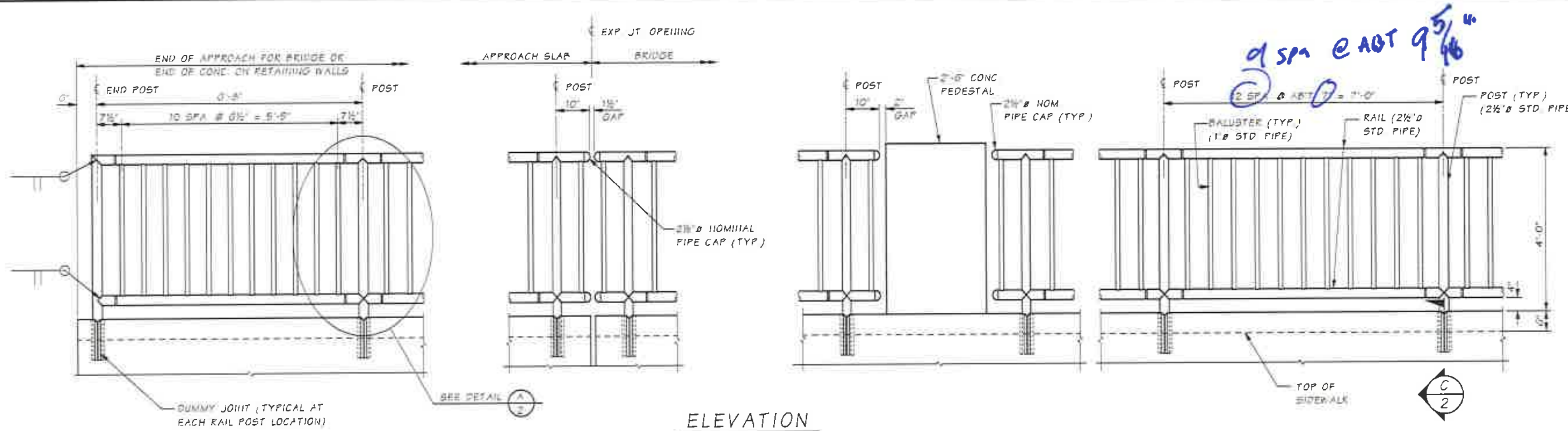
Spacing of Posts	84	in
Max vertical space 6"+Vert Diam:	7.9	in
Minimum # of Spaces	10.6	
Actual # of Spaces	11.0	
Calculated Spacing	7.64	in
Actual Spacing	7.625	
Actual Clear Opening - @ spacing	5.73	in
Actual Clear Opening - worst case	5.85	in
	7.625in Spacing is Satisfactory	

Weight of Pedestrian Rail

HSS 2.875x0.250	7.02	plf
HSS 2.875x0.203	5.8	plf
HSS 1.900x0.145	2.72	plf
<u>Lengths</u>		
Post	65.25	in
Top Rail	84	in
Bottom Rail	84	in
Verticals	500.5	in
<u>Weight</u>		
Post	38.2	lb
Top Rail	40.6	lb
Bottom Rail	40.6	lb
Verticals	113.4	lb
Per Foot Weight	33.3	plf
Add 10% for misc components	**36.6**	**plf**

Weight of Type BP Rail on Barrier

Density of Aluminum	175	pcf
Rail - 2.5" STD pipe	2.07	plf
Vertical - 1" STD pipe	0.60	plf
Extruded Channel (approx by geometry)	2.42	plf
<u>Lengths</u>		
Rail	8	in
Vertical	20.75	in
Extruded Channel	8	in
<u>Weight</u>		
Rail	1.4	lb
Vertical	1.0	lb
Extruded Channel	1.6	lb
Per Foot Weight	6.0	plf
Add 10% for misc components	**6.7**	**plf**



ELEVATION

BALUSTERS NORMAL TO GRADE
TOP & BOTTOM RAILS PARALLEL TO GRADE

NOTES

1. PIPE RAILING, AND PIPE RAILING SPLICES, SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200 FEET.
2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION AND INCLUDING AN ERECTION DIAGRAM. MATERIAL BEING USED SHALL BE SPECIFIED IN THE SHOP DRAWINGS.
3. PIPE RAILING, AND PIPE RAILING SPLICES, MAY BE HEATED TO NOT MORE THAN 400°F FOR A PERIOD NOT TO EXCEED 30 MINUTES TO FACILITATE FORMING OR BENDING HORIZONTAL CURVATURE.
4. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH. FLAME CUTTING WILL NOT BE PERMITTED.
5. WELDING OF ALUMINUM SHALL CONFORM TO STD. SPEC. SECTION 9-28.14(3).
6. AFTER FABRICATION, POSTS SHALL BE HEAT TREATED IN ACCORDANCE WITH SECTION 6.5 OF THE AASHTO STANDARD SPECIFICATIONS FOR STRUCTURAL SUPPORTS FOR HIGHWAY SIGNS, LUMINAIRES, AND TRAFFIC SIGNALS DATED 2001 AND INTEREMS THROUGH 2003.
7. ALL ALUMINUM PARTS SHALL BE GIVEN A (CLEAR OR BRONZE)* ANODIC COATING OF AT LEAST 0.0006 INCHES THICK AND SEALED TO MEET THE REQUIREMENTS OF ASTM B 580 WITH A UNIFORM FINISH.
8. PIPE RAILING, PIPE BALUSTERS, PIPE RAILING SPLICES, SHALL BE ADEQUATELY WRAPPED TO INSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.

* NOTE TO DESIGNER:
Designer to choose color for their project in consultation with the Bridge Architect.

MAT'L	PART	MATERIAL SPECIFICATION
ALUMINUM	PIPES	ASTM B 221-6005-T5 SCHEDULE 40 (STD. PIPE) ASTM B 241 OR B 429 6061-T6
	BAR	ASTM B 221-6005-T5
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL

Bridge Design Engr.		REGION NO.	STATE	FED. AID PROJ. NO.
Supervisor		10	WASH.	
Designed By		JOB NUMBER		
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE / REVISION / BY / APP'D			

THU Sep 02 07:39:46 2010

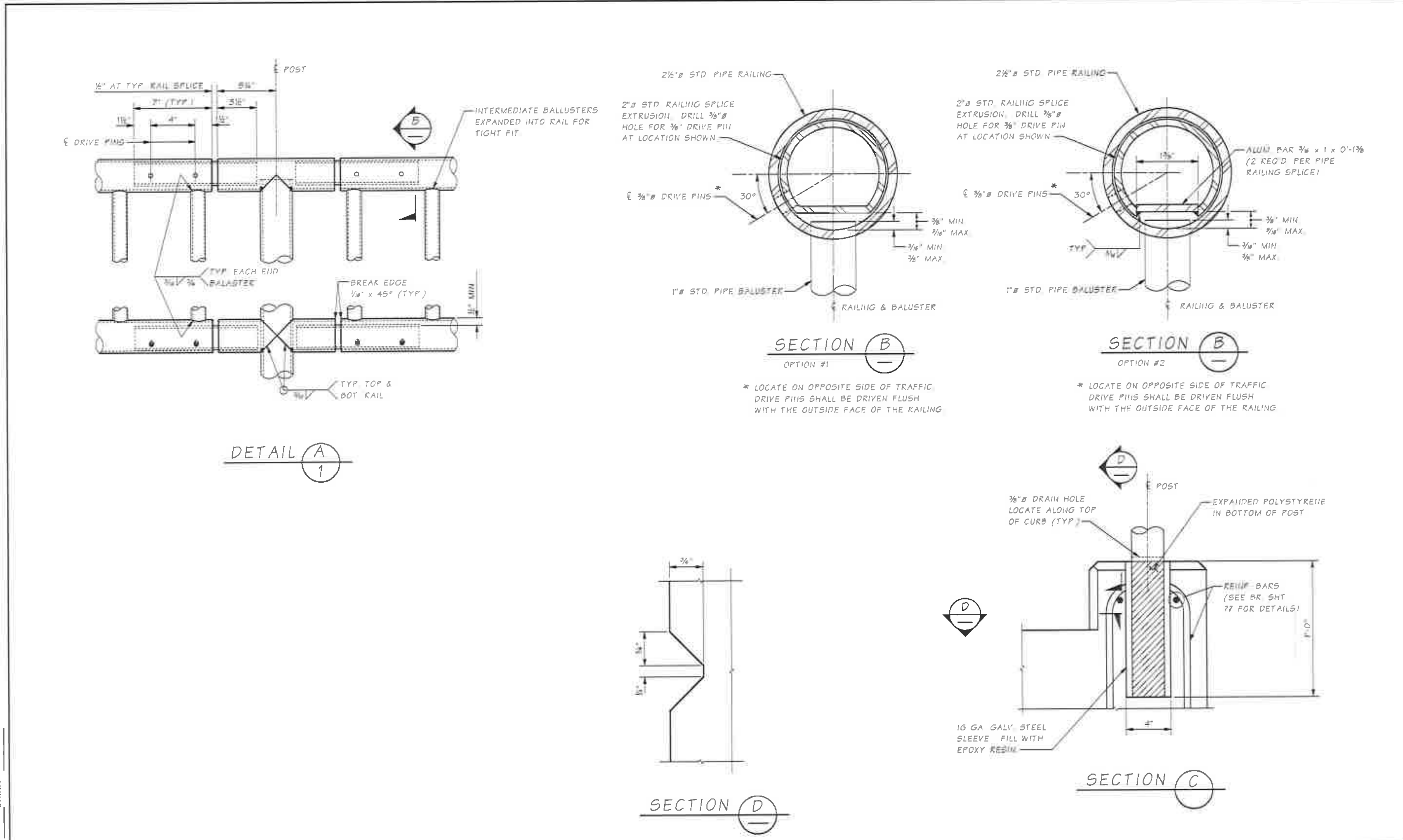
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

PEDESTRIAN RAILING DETAILS 1 OF 2

JOB NO. SHEET SR



Bridge Design Engr.	M:\STANDARDS\BP Rails\PedRail_Alumin 2.MAN	STATE: WASH		
Supervisor		10		
Designed By		JOB NUMBER		
Checked By				
Detailed By				
Bridge Projects Engr.				
Prelim. Plan By				
Architect/Specialist	DATE / REVISION / BY / APP'D	FED. AID PROJ. NO.		

BRIDGE AND STRUCTURES OFFICE

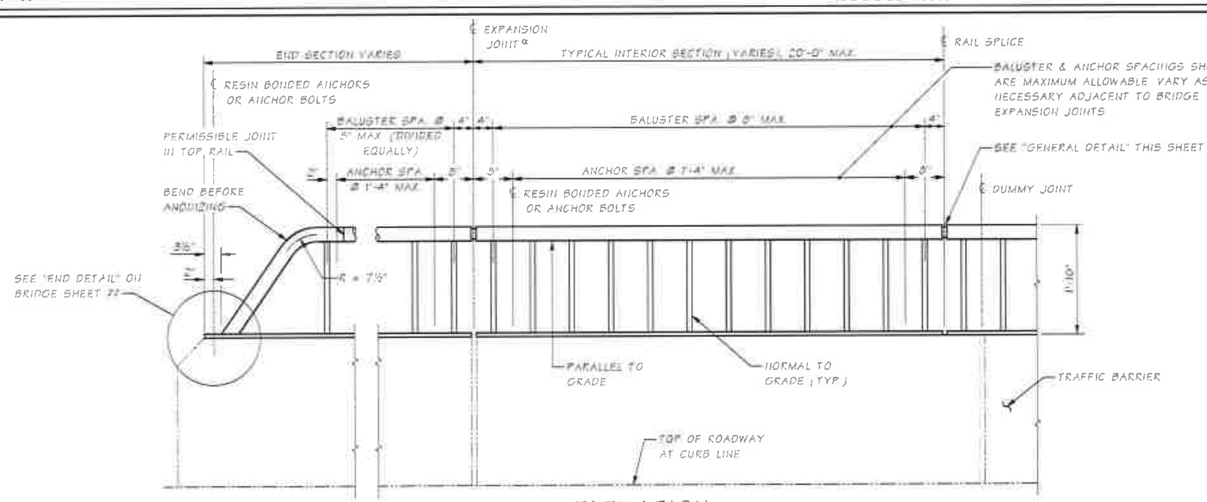
Washington State Department of Transportation

STANDARD RAILINGS

PEDESTRIAN RAILING DETAILS 2 OF 2

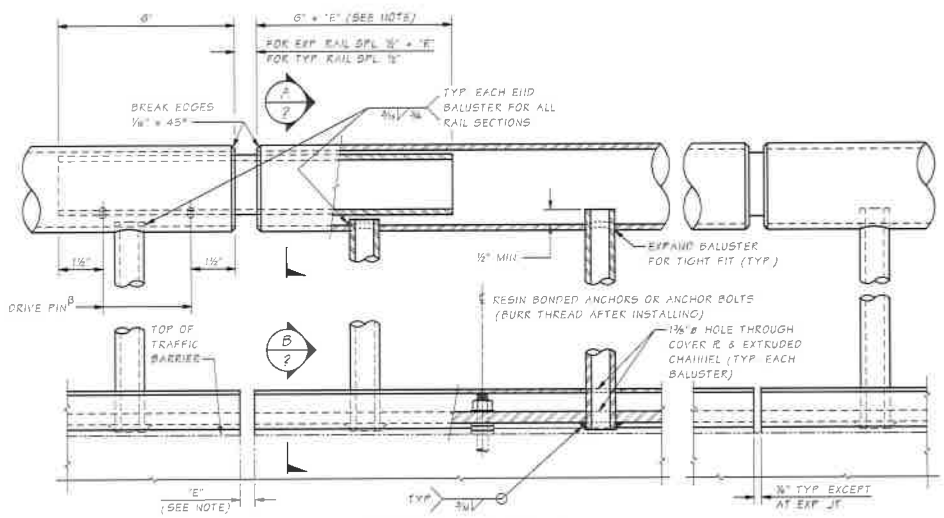
Thu Sep 02 07:38:52 2010

SR
JOB NO.
SHEET



ELEVATION

BALUSTER AND GUARDRAIL SECTION ATTACHMENT DETAILS NOT SHOWN



GENERAL DETAIL

NOTES

1. PIPE RAILING, PIPE RAILING SPLICES, COVER PLATES AND BOTTOM EXTRUDED CHANNELS SHALL BE BENT TO THE HORIZONTAL CURVE WHERE THE RADIUS OF CURVATURE IS LESS THAN 200'. THESE ITEMS MAY BE HEATED TO NOT MORE THAN 400°F FOR A PERIOD NOT TO EXCEED 30 MINUTES TO FACILITATE FORMING OR BENDING TO HORIZONTAL CURVATURE.

2. SHOP DRAWINGS OF RAILING SHALL BE SUBMITTED FOR APPROVAL SHOWING COMPLETE DIMENSIONS AND DETAILS OF FABRICATION AND INCLUDING AN ERECTION DIAGRAM. MATERIAL SPECIFICATIONS SHALL BE PROVIDED IN THE SHOP DRAWINGS FOR ALL COMPONENTS.

3. CUTTING SHALL BE DONE BY SAWING OR MILLING AND ALL CUTS SHALL BE TRUE AND SMOOTH. FLAME CUTTING WILL NOT BE PERMITTED.

4. WELDING OF ALUMINUM SHALL CONFORM TO STD. SPEC. SECTION 9-28.14(3).

5. ALL ALUMINUM PARTS SHALL BE GIVEN A *(CLEAR OR BRONZE) ANODIC COATING OF AT LEAST 0.0006" THICK AND SEALED TO MEET THE REQUIREMENTS OF ASTM B 580 WITH A UNIFORM FINISH.

6. PIPE RAILING, PIPE BALUSTERS AND PIPE RAILING SPLICES SHALL BE ADEQUATELY WRAPPED TO INSURE SURFACE PROTECTION DURING HANDLING AND TRANSPORTATION TO THE JOB SITE.

* NOTE TO DESIGNER:
Designer to choose color for their project in consultation with the Bridge Architect.

α PROVIDE EXPANSION RAIL SPLICE AT TRAFFIC BARRIER EXPANSION JOINTS. RAIL SPLICE JOINTS ARE NO REQUIRED AT TRAFFIC BARRIER DUMMY JOINTS.

β LOCATE ON OPPOSITE SIDE OF TRAFFIC. DRIVE PINS SHALL BE DRIVEN FLUSH WITH THE OUTSIDE FACE OF THE RAILING.

NOTE:
"E" DIMENSION SHALL BE EQUAL TO OPENING OF TRAFFIC BARRIER EXPANSION JOINTS.

Bridge Design Engr.	M:\STANDARDS\BP_Rails\BP1.man	REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Supervisor		10	WASH.			
Designed By		JOB NUMBER				
Checked By						
Detailed By						
Bridge Projects Engr.						
Prelim. Plan By						
Architect/Specialist	DATE	REVISION	BY	APP'D		

Thu Sep 02 07:40:24 2010

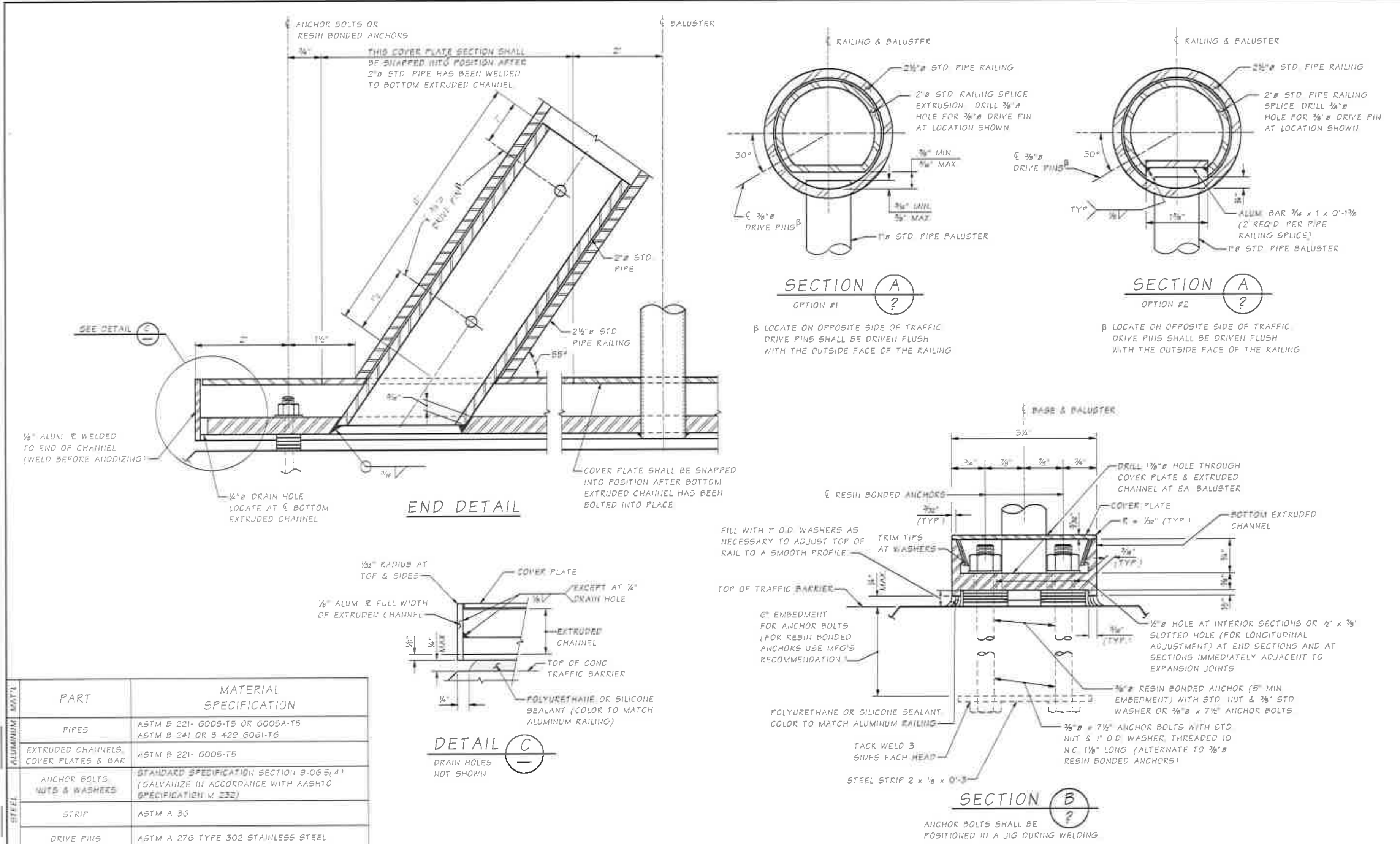
BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE BP DETAILS 1 OF 2

JOB NO. SK SHEET



MAT'L	PART	MATERIAL SPECIFICATION
ALUMINUM	PIPES	ASTM B 221- 6005-T5 OR 6005A-T5 ASTM B 241 OR B 429 6061-T6
ALUMINUM	EXTRUDED CHANNELS, COVER PLATES & BAR	ASTM B 221- 6005-T5
STEEL	ANCHOR BOLTS, NUTS & WASHERS	STANDARD SPECIFICATION SECTION 9-06.5(4) (GALVANIZE IN ACCORDANCE WITH AASHTO SPECIFICATION M 232)
STEEL	STRIP	ASTM A 36
STEEL	DRIVE PINS	ASTM A 276 TYPE 302 STAINLESS STEEL

BRIDGE AND STRUCTURES OFFICE

Washington State Department of Transportation

STANDARD RAILINGS

BRIDGE RAILING TYPE BP DETAILS 2 OF 2

Core drilled holes shall have a minimum clearance of 3″ from the edge of the concrete and 1″ clearance from existing reinforcing bars in the existing structure. These clearances shall be noted in the plans.

4. **Dowelling Reinforcing Bars Into the Existing Structure**

   a. Dowel bars shall be set with an approved epoxy resin. The existing structural element shall be checked for its adequacy to transmit the load transferred to it from the dowel bars.

   b. Dowel spacing and edge distance affect the allowable tensile dowel loads. Allowable tensile loads, dowel bar embedment, and drilled hole sizes for reinforcing bars (Grade 60) used as dowels and set with an approved epoxy resin are shown in Table 5.5.4-1. These values are based on an edge clearance greater than 3″, a dowel spacing greater than 6″, and are shown for both uncoated and epoxy coated dowels. Table 5.5.4-2 lists dowel embedment lengths when the dowel spacing is less than 6″. Note that in Table 5.5.4-2 the edge clearance is equal to or greater than 3″, because this is the minimum edge clearance for a drilled hole from a concrete edge.

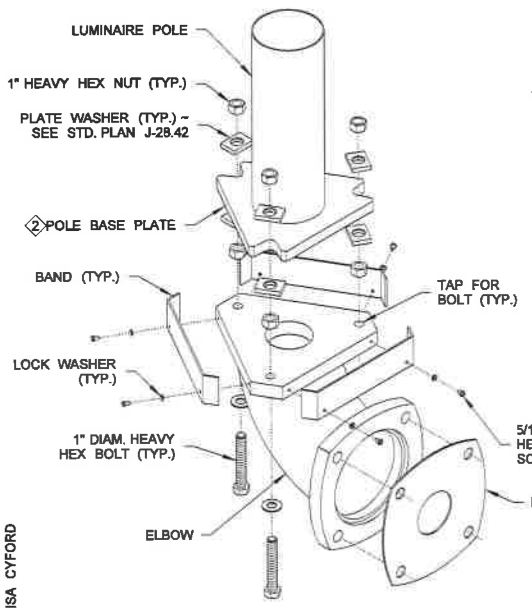
   If it is not possible to obtain these embedments, such as for traffic railing dowels into existing deck slabs, the allowable load on the dowel shall be reduced by the ratio of the actual embedment divided by the required embedment.

   c. The embedments shown in Table 5.5.4-1 and Table 5.5.4-2 are based on dowels embedded in concrete with f'_c=4,000 psi.

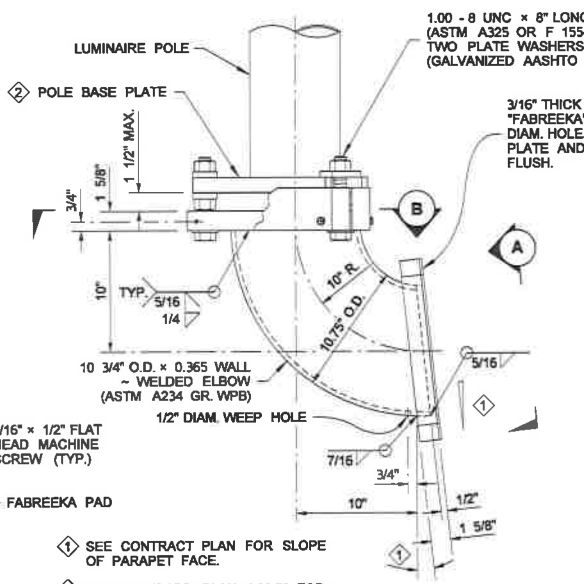
Bar Size	**Allowable Design Tensile Load, T*** (kips)	**Drill Hole Size** (in)	**Required Embedment, L_e**	
			Uncoated (in)	**Epoxy Coated** (in)
#4	12.0	⅝	7	8
#5	18.6	¾	8	9
#6	26.4	1	9	10
#7	36.0	1⅛	11	12
#8	47.4	1¼	13	14.5
#9	60.0	1⅜	16	17½
#10	73.6	1½	20	22
#11	89.0	1⅝	25	28

* Allowable Tensile Load (Strength Design) = $(f_y)(A_s)$.

Allowable Tensile Load for Dowels Set With Epoxy Resin f'_c = 4,000 psi, Grade 60 Reinforcing Bars, Edge Clearance ≥ 3″, and Spacing ≥ 6″
Table 5.5.4-1

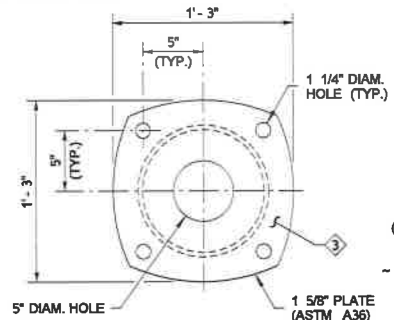


EXPLODED ISOMETRIC VIEW



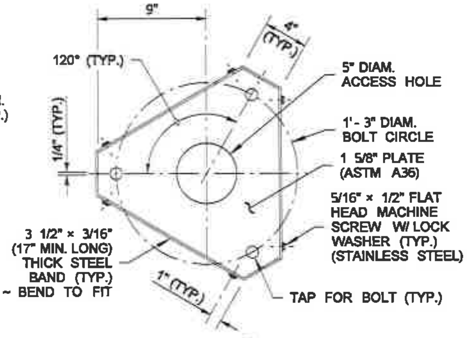
① SEE CONTRACT PLAN FOR SLOPE OF PARAPET FACE.

② SEE STANDARD PLAN J-28.50 FOR POLE BASE PLATE REQUIREMENTS.



VIEW A

③ THE FACE SHALL BE PLANE AFTER FABRICATION, TO PROVIDE A SEAL BETWEEN THE BARRIER AND THE ELBOW.



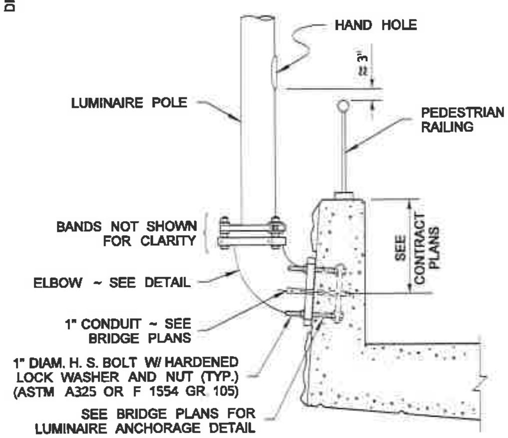
SECTION B

NOTES

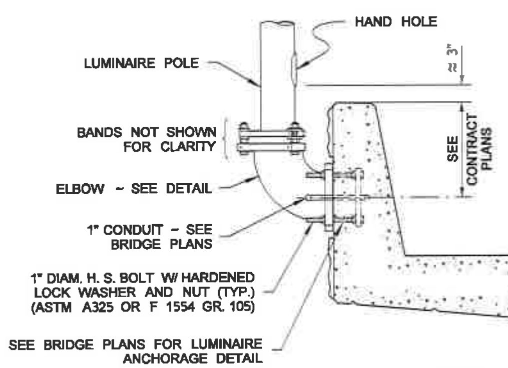
1. Galvanize the Elbow Assembly after fabrication according to AASHTO M 111.
2. See **Standard Plan J-28.50** for Pole Base and Hand Hole details.

STEEL LIGHT STANDARD ELBOW DETAIL

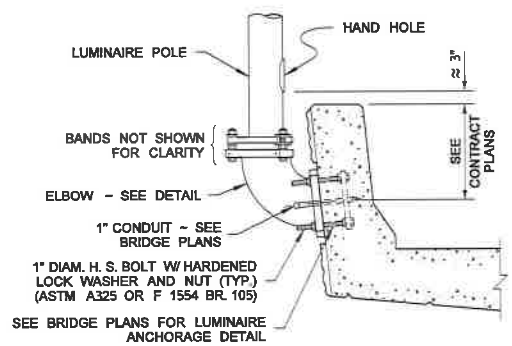
FOR LUMINAIRE POLES WITH SINGLE MAST ARM 12'- 0" OR LESS AND DOUBLE MAST ARMS 8'- 0" OR LESS, MOUNTED ON BRIDGE OR RETAINING WALLS.



BRIDGE PEDESTRIAN BARRIER



SINGLE-SLOPE BRIDGE TRAFFIC BARRIER



F-SHAPE BRIDGE TRAFFIC BARRIER

WHEN TRAFFIC BARRIER HEIGHT IS 42", MAINTAIN APPROX. HEIGHT FROM TOP OF BARRIER TO HAND HOLE SHOWN.

TYPICAL SECTIONS

NOTE: THIS PLAN IS NOT A LEGAL ENGINEERING DOCUMENT BUT AN ELECTRONIC DUPLICATE. THE ORIGINAL, SIGNED BY THE ENGINEER AND APPROVED FOR PUBLICATION IS KEPT ON FILE AT THE WASHINGTON STATE DEPARTMENT OF TRANSPORTATION. A COPY MAY BE OBTAINED UPON REQUEST.

STEEL LIGHT STANDARD ELBOW MOUNTING ON BRIDGE 7 RETAINING WALL
STANDARD PLAN J-28.45-01

SHEET 1 OF 1 SHEET

APPROVED FOR PUBLICATION

Pasco Bakotich III — 06-27-11

STATE DESIGN ENGINEER — DATE

Washington State Department of Transportation

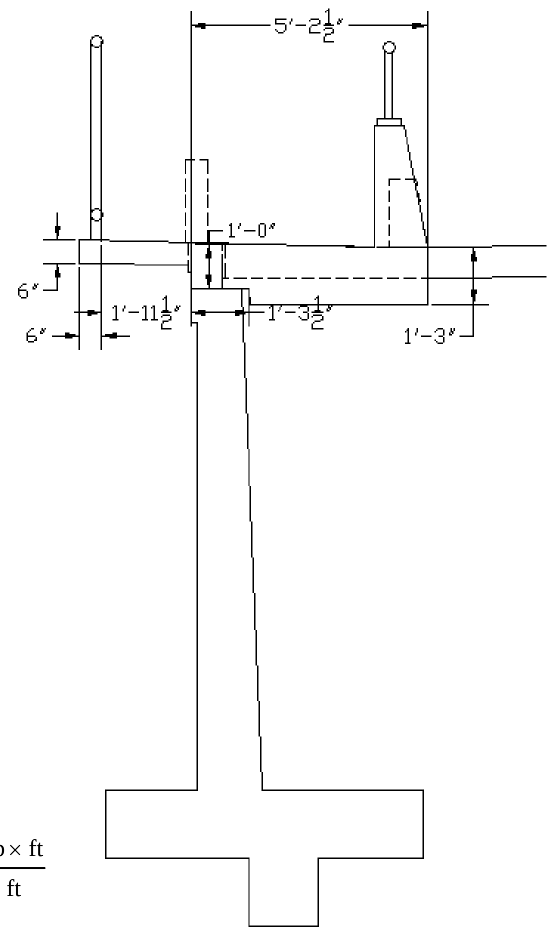
BALLARD Widening - Segment 1 - 6t Sidewalk Extension

The sidewalk extension in Segment 1 shall be made in the form of an anchor slab with cantilevered sidewalk extension. An anchor slab is preferred in order to isolate the existing retaining wall structure from new loads introduced by the sidewalk extension and TL-4 traffic barrier.

In Segment 1, the anchor slab shall be cast against the top of the existing retaining wall. This will allow for transfer of some vertical loads into the existing stem wall (additional axial loading is minor) while not transferring bending moments (more significant loading condition) or potential lateral loads from the new TL-4 traffic barrier.

The slab has been designed for stability under the following conditions:

1) Unbalanced pedestrian live load overturning using strength factors to check LRFD eccentricity limitations
2.) Extreme limit state 10 kip vehicle impact stability load applied to the top of the traffic barrier



$L_{Overhang} := 1ft + 11.5in$ $\quad w_{LL} := 75psf$ $\quad \gamma_c := 155pcf$

Overturning Moment

$$OM_{LL} := w_{LL} \times \frac{L_{Overhang}^{\ 2}}{2} = 143.815 \times \frac{lb \times ft}{ft}$$

$$OM_{DC} := 6in \times \gamma_c \times \frac{\left(L_{Overhang} + 6in\right)^2}{2} = 234.182 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Lat.Railing} := 50plf \times 54in = 225 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Vert.Railing} := 50plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 103.906 \times \frac{lb \times ft}{ft}$$

$$OM_{DC.Railing} := 36.6plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 76.059 \times \frac{lb \times ft}{ft}$$

$$OM_{service} := \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \dots + \left(OM_{DC} + OM_{DC.Railing}\right) = 782.963 \times \frac{lb \times ft}{ft}$$

$$OM_{factored} := 1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \dots + 0.9 \times \left(OM_{DC} + OM_{DC.Railing}\right) = 1.106 \times 10^3 \times \frac{lb \times ft}{ft}$$

Continuous Anchor Slab Length (minimum): $\quad L_{AS} := 20ft$

$$OM_{veh} := 10kips \times (32in + 11in) = 3.583 \times 10^4 ft \times \frac{lb \times ft}{ft}$$

$$OM_{Extreme} := OM_{veh} + L_{AS} \times \left(OM_{DC} + OM_{DC.Railing}\right) = 4.204 \times 10^4 \times lb \times ft$$

Resisting Moment

$D_{slab} := 15in$ $Width := 5ft + 2.5in$

$$Res_{DC.slab} := 12in \times \gamma_c \times \frac{(Width)^2}{2} \ldots$$
$$+ \left(D_{slab} - 12in\right) \times \gamma_c \times [Width - (1ft + 1.5in) - 2in] \times \left[\frac{[Width - (1ft + 3.5in)]}{2} + (1ft + 1.5in) + 2in\right]$$

$$Res_{DC.slab} = 2.596 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{DC.barrier} := 32\,in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times \left[4ft + 2in - \left[\frac{8in}{2} + \left(32in \times \frac{4}{21}\right)\right]\right] = 1.265 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{service} := Res_{DC.slab} + Res_{DC.barrier} = 3.861 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{factored} := 0.9 \times \left(Res_{DC.slab} + Res_{DC.barrier}\right) = 3.475 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{Extreme} := Res_{service} \times L_{AS} = 7.722 \times 10^4 \times lb \times ft$$

Strength Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf\right] \ldots$$
$$+ 12in \times \gamma_c \times Width + \left(D_{slab} - 12in\right) \times \gamma_c \times [Width - (1ft + 1.5in) - 2in] + 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c$$

$$V_{DL} = 1.567 \times 10^3 \times plf$$

$$V_{LL} := \left(w_{LL} \times L_{Overhang} + 50plf\right) = 196.875 \times plf$$

$$M_{abt.toe} := OM_{factored} - Res_{factored} = -2.368 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL} + 1.75 \times V_{LL}\right)}\right] = 1.254\,ft$$

$$ecc_{limit} := \frac{3}{8} \times Width = 1.953\,ft$$ (10.6.3.3) (Foundation on Rock)

Bearing Pressure Check

$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$

$$V_{DL} + V_{LL} = 1.764 \times 10^3 \times plf$$

Bearing Pressure is reasonable as axial load on retaining wall stem

Extreme Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf\right] \ldots$$
$$+ D_{slab} \times \gamma_c \times (Width) + in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c$$
$$= 1.248 \times 10^3 \times plf$$

$$M_{abt.toe} := OM_{Extreme} - Res_{Extreme} = -3.518 \times 10^4 \times lb \times ft$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL}\right) \times L_{AS}}\right] = 1.038\,ft$$

$$ecc_{limit} := \frac{Width}{3} = 1.736\,ft$$ (10.6.4.2)

$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$

Sliding Resistance

$\varphi_\tau := 0.80$ (Table 10.5.5.2.2-1)

$\varphi_f := 28deg$ Assuming a reasonably shallow angle of internal friction

$\mu_{R.t} := \tan(\varphi_f) = 0.532$ (EQ 10.6.3.4-2)

$$Sliding_{res.DC.slab.at.wall} := ft \times \gamma_c \times (1ft + 1.5in + 2in - 1in) \times L_{AS} = 3.746 \times 10^3 ft \times plf$$

$$Sliding_{res.DC.slab.past} := D_{slab} \times \gamma_c \times [5ft + 2.5in - (1ft + 1.5in) - 2in] \times L_{AS} = 1.518 \times 10^4 ft \times plf$$

$$Sliding_{res.DC.barrier} := 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times L_{AS} = 7.611 \times 10^3 ft \times plf$$

$$Sliding_{resistance} := \varphi_\tau \times \mu_{R.t} \times \left(Sliding_{res.DC.slab.at.wall} + Sliding_{res.DC.slab.past} + Sliding_{res.DC.barrier}\right) = 1.129 \times 10^4 ft \times plf$$

$Check_{C.D}\left(Sliding_{resistance}, 10kips\right) = $ "SATISFACTORY"

Cantievered Sidewalk Design

$E_s := 29000ksi$ $F_y := 60ksi$ $f'_c := 4ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2.5in = 0.208\,ft$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.438 \times in$

Design Moment: $M_{des} := \left[1.75 \times (OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}) + 1.25 \times (OM_{DC} + OM_{DC.Railing})\right] \times w_{mem} = 1.215 \times 10^3 \times lb \times ft$

Service Moment: $M_{service} := OM_{service} \times w_{mem} = 782.963 \times lb \times ft$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 3.25 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.125 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.085 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 1.215 \times kip \times ft$

$\phi M_n = 5.321 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 437.9 \times \%$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.24 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 480 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 72 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 2.88 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.5\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 3.456 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 3834 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 7.563$$

<u>Determine strain compatible neutral axis</u>

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 1.053 \times \text{in}$$

Moment Arm: $arm := d - \frac{x}{3} = 2.899 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 3.241 \times \text{kips}$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 8.102 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2.5 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.02$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(\text{ksi})\ (\text{in}) - 2 \times d_c = 27.072 \times \text{in}}$$

$spacing \times (1 + bundles) = 6 \times \text{in}$

$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$V_u := \left[\left(1.75 \times w_{LL} + 1.25 \times \gamma_c \times 6\text{in}\right) \times L_{Overhang} + 1.75 \times 50\text{plf} + 1.25 \times 36.6\text{plf}\right] \times w_{mem} = 0.58 \times \text{kips}$$

Use Simplified Calc Values

Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv $Simplified := \text{"yes"}$

$\beta_w := 2$ $\theta := 45\text{deg}$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (\text{ksi})} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_c = 3.993 \times \text{kips}$ $\text{Check}_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

Slab edge deflection

Moment of Inertia: $I := \frac{(6in)^3}{12} = 216 \times \frac{in^4}{ft}$

Modulus of Elasticity: $E_c = 5.521 \times 10^8 \frac{lb}{ft^2}$

Edge Deflection due to ped load: (Cantilever) $\delta := \frac{w_{LL} \times L_{Overhang}^{\ 4}}{8 \times E_c \times I} = 2.877 \times 10^{-4} \times in$

$\frac{L_{Overhang}}{\delta} = 81685$ Exceeds 300 per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges. section 5

SATISFACTORY


MILLER PAINT

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BALLARD - 6FT - SEGMENT 2

New construction

$\gamma_c := 155pcf$

Existing overhang = 4ft 1in max to outside of existing pedestal (REF SHT 36)
2ft 0in min to outside of existing pedestal (REF SHT 35)

Distance from outside of existing pedestal to inside of existing curb = 5ft 7in (REF SHT 42)

Additional walkway length (6ft): $Addl_L := \left[6ft - \left[(5ft + 7in) - \left(8in + 32in \times \frac{4}{21}\right)\right]\right] = 1.591\ ft$

Total unsupported walkway length: $Unsupp_L := Addl_L + (4ft + 1in) = 5.675\ ft$

Walkway will be supported along outside edge and connect to existing cantilevered slab (PIN-FIX)

Thickness of Slab (Ex and New): $t := 6.5in$

$w_{DC} := t \times \gamma_c = 84.0 \times psf$

$w_{LL} := 75psf$

$M2_{DC.pos} := w_{DC} \times \frac{9}{128} \times Unsupp_L^{\,2} = 190.1 \times \frac{ft \times lb}{ft}$ $\quad M2_{DC.neg} := w_{DC} \times \frac{Unsupp_L^{\,2}}{8} = 337.9 \times \frac{ft \times lb}{ft}$

$M2_{LL.pos} := w_{LL} \times \frac{9}{128} \times Unsupp_L^{\,2} = 169.8 \times \frac{ft \times lb}{ft}$ $\quad M2_{LL.neg} := w_{LL} \times \frac{Unsupp_L^{\,2}}{8} = 301.9 \times \frac{ft \times lb}{ft}$

Design New Bonded Reinforcing for Maximum Positive/Negative Moment (Core drilled hole with roughened surface BDM 5.5.4)

$E_s := 29000ksi$

Concrete Strength of Widening: $f'_c := 5ksi$

Bar Strength adjusted to Allowable Design Tensile Load of #4 bar :
(12 kips BDM 5.5.4-1)

$F_y := \frac{12kips}{0.2in^2} = 60 \times ksi$

Bar size (US number): $bar_{no} := 4$ $\quad d_{no_{bar_{no}}} = 0.5 \times in$ $\quad diam_{deform_{bar_{no}}} = 0.563 \times in$ $\quad A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 3in$

Number of layers: $full_{layers} := 1$ $\quad bundled := \text{"no"}$ $\quad bundles := yes_{no}(bundled) = 0$ $\quad \beta_1 = 0.8$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 1$ $\quad spacing := \frac{w_{mem}}{n_{bars}} = 12 \times in$

Height of member $h_{mem} := 6.5in$ $\quad A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $\quad A_s \times w_{mem} = 0.2 \times in^2$ $\quad clear_{spacing} = 11.438 \times in$

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Reinforcing Bar is centered in slab. Negative moment governs concrete design.

Design Moment: $M_{des} := (1.25 \times M2_{DC.neg} + 1.75 \times M2_{LL.neg}) \times w_{mem} = 0.951\,ft \times kips$

Service Moment: $M_{service} := (M2_{DC.neg} + M2_{LL.neg}) \times w_{mem} = 0.64\,ft \times kips$

depth to rebar $$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 3.25 \times in$

Rectangular Compression Depth: $$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$ $a_{req} = 0.077 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.066 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 0.951 \times kip \times ft$

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.235 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$ $\phi M_n = 2.819 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) =$ "SATISFACTORY"

$\dfrac{\phi M_n}{M_{des}} = 296.5 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} :=$ "yes"

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 827.3 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^{\ 2}}{6}$ $S = 84.5 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 5.826 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} =$ "Exception in effect. Moment capacity exceeds 4/3 Moment demand"

xXxXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 12 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.542\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 6.991 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)
The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 4287 \times ksi$$

Determine strain compatible neutral axis

$$n := \frac{E_s}{E_c} = 6.765$$

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 0.751 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 3 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 2.56 \times kips$

Stress in the steel at service limit state

$$\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 12.798 \times ksi}$$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 3 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.224$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 12.442 \times in}$$

$$spacing \times (1 + bundles) = 12 \times in$$

$$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$$\boxed{\phi_v := 0.9}$$

$$V_u := \left[\frac{5}{8} \times \left(1.25 \times w_{DC} + 1.75 \times w_{LL}\right) \times Unsupp_L\right] \times w_{mem} = 0.838 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := \text{"yes"}$

$\beta_w := 2$ $\theta := 45deg$

$$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$$

$$\phi_v \times V_c = 4.464 \times kips$$

$$Check_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$$

Development Lengths (AASHTO LRFD)

The basic development length for a #11 bar and smaller is determined by 5.11.2.1.1

$bar_{no} := 5$ $\quad A_b := A_{no_{bar_{no}}} = 0.31 \times in^2$ $\quad d_b := d_{no_{bar_{no}}} = 0.625 \times in$ $\quad F_y := 40ksi$ $\quad f'_c := 5ksi$

$f_{ct} := \text{"NA"}$

$$l_{d'} := \frac{1.25 \times A_b \left(\frac{1}{in^2}\right) \times F_y \left(\frac{1}{ksi}\right)}{\sqrt{f'_c \left(\frac{1}{ksi}\right)}} \bullet (in) \qquad l_{d'} = 6.932 \times in$$

But the development length can not be less than

$$l_{d.min} := 0.4 \times d_b \left(\frac{1}{in}\right) \times F_y \left(\frac{1}{ksi}\right) \bullet (in) \qquad l_{d.min} = 10 \times in$$

or 12 inches

Adjustment factors (LRFD 5.11.2.1.2 & 3):

Bars coated with epoxy with cover less than 3db or clear spacing between bars less than 6 db:

Top bars placed over 12 inches of concrete coated with epoxy with cover less than 3db or clear spacing between bars less than 6 db:

All other epoxy bar cases: $\boxed{1.2 \times l_{d'} = 8.318 \times in}$

Reinforcement being developed in the length under consideration is spaced laterally at least 6 inches on center with at least 3 inches clear cover measured in the direction of spacing: $\boxed{0.8 \times l_{d'} = 5.545 \times in}$

Lap Splice (LRFD 5.11.5.3.1)

$\boxed{\max(1.3 \times 0.8 \times 1.2 \times l_{d'}, 12in) = 12 \times in}$

Lap new bar with existing steel a minimum of 12" - Assuming 2" cover to bar - use 14"

Existing steel is spaced at 6" on center, new steel therefore can not be located more than 6" from existing steel

Min embedment for new #4 bar is 8" per WSDOT BDM 5.5-6
Increasing by 1.3 for lap splice results in 10.4". Provide 12" embedment.

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Check Existing Reinforcing for Maximum Negative Moment $E_s := 29000ksi$ $F_y := 40ksi$

Concrete Strength of Widening: $f'_c := 5ksi$

Bar size (US number): $bar_{no} := 5$ $d_{no_{bar_{no}}} = 0.625 \times in$ $diam_{deform_{bar_{no}}} = 0.688 \times in$ $A_{no_{bar_{no}}} = 0.31 \times in^2$

$clear := 1.5in$ (REF Sheet 42)

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.8$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6.5in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.62 \times in^2$ $clear_{spacing} = 5.313 \times in$

Design Moment: $M_{des} := (1.25 \times M2_{DC.neg} + 1.75 \times M2_{LL.neg}) \times w_{mem} = 0.951\ ft \times kips$

Service Moment: $M_{service} := (M2_{DC.neg} + M2_{LL.neg}) \times w_{mem} = 0.64\ ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 4.688 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.053 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.068 \times in^2$

Check compression for full yielding of actual steel $M_{des} = 0.951 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.486 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 8.267 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$ $\frac{\phi M_n}{M_{des}} = 869.5 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 827.3 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 84.5 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 5.826 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 5$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\ ft$ x $h_{mem} = 0.542\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 6.991 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 4287 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 6.765$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 1.494 \times \text{in}$$

Moment Arm: $arm := d - \frac{x}{3} = 4.189 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 1.833 \times \text{kips}$

Stress in the steel at service limit state

$$\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 2.956 \times \text{ksi}}$$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 1.5 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.429$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(\text{ksi})\ (\text{in}) - 2 \times d_c = 121.326 \times \text{in}}$$

$$spacing \times (1 + bundles) = 6 \times \text{in}$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

Load to Edge Beam

Dead Load of Slab
Toeboard (OSHA Req CFR 1910.28 (b)(7))

$$R2_{DC.Slab} := \frac{3 \times w_{DC} \times Unsupp_L}{8} = 178.7 \times plf$$

Dead Load of 54 inch Bicycle Railing:

$$R2_{DC.Railing} := 36.6plf$$

Live Load Reaction

$$R2_{LL} := \frac{3 \times w_{LL} \times Unsupp_L}{8} = 159.6 \times plf$$

Edge Beam Length = 18ft 0in max (REF SHT 35)

$$L_{edge} := 18ft$$

Edge beam is simply supported:

$$V2_{edge} := 1.25 \times \left(R2_{DC.Slab} + R2_{DC.Railing}\right) + 1.75 \times R2_{LL} = 548.374 \times plf$$

$$M2_{edge} := \frac{\left[1.25 \times \left(R2_{DC.Slab} + R2_{DC.Railing}\right) + 1.75 \times R2_{LL}\right] \times L_{edge}^{\ 2}}{8} = 22.2\,ft \times kips$$

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit
(AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges
(AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges
(AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange
(AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 6x6x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 27.8kip \times ft$$

$$\varphi M_{n.AASHTO.HSS6x6x3} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 30.9 \times kip \times ft$$

$$\frac{M2_{edge}}{\varphi M_{n.AASHTO.HSS6x6x3}} = 0.719$$

$$Check_{C.D}\left(\varphi M_{n.AASHTO.HSS6x6x3}, M2_{edge}\right) = \text{"SATISFACTORY"}$$

Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

"For square and rectangular HSS, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side of the area of both webs shall be considered effective in resisting the shear" (6.12.1.2.3b)

$V_n = V_{cr} = C \times V_p$

C = if $\frac{D}{t_w} \le 1.12 \times \sqrt{\frac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := 5\frac{3}{16}in$ $\quad E := 29000ksi$ $\quad F_{yw} := 46ksi$

$t_w := \frac{3}{16}in$

$\frac{D}{t_w} = 27.7$ $\qquad 1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}} = 62.9$

$$C := \text{if}\left[\frac{D}{t_w} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$$

$$V_p := 0.58 \times F_{yw} \times D \times t_w = 26 \times kips$$

$\varphi_v \times V_p = 26 \times kips$

Torsion from Pedestrian Rail:

$$Tors_{Ped} := 50plf \times \left(54in + t + \frac{6in}{2}\right) \times \frac{L_{edge}}{2} + 200lb \times \left(54in + t + \frac{6in}{2}\right) = 3.44\,ft \times kips$$

$$V2_{tors.Ped} := 1.75 \times \frac{Tors_{Ped}}{6in - \frac{3}{16}in} = 12.427 \times kips$$

$$Shear_{edge.beam} := V2_{edge} \times \frac{L_{edge}}{2} + V2_{tors.Ped} = 17.362 \times kips$$

$Check_{C.D}(\varphi_v \times V_p, Shear_{edge.beam}) =$ "SATISFACTORY"

Edge Beam Load transfers to New Channel Cantilever Extension

Length of Cantilever extension:
(Additional cantilever length - edge beam width {HSS6} / 2 + 6 inches to first anchor to existing cantilever)

$$\boxed{Canti_{new.L} := Addl_L - \frac{6in}{2} + 6in = 1.841\ ft}$$

Reaction from edge beam to new channel cantilver extension:
(Each simple span edge beam has its own channel supports ~ two channels per existing cantilever)

$$V2_{LL.ped.rail} := 1.75 \times \left(50plf \times \frac{L_{edge}}{2} + 200lb\right) = 1.138 \times kips$$

$$R_{Canti.new} := V2_{edge} \times \frac{L_{edge}}{2} + V2_{LL.ped.rail} = 6.1 \times kips$$

$$M2_{Canti.new} := R_{Canti.new} \times Canti_{new.L} = 11.182\ ft \times kips$$

AASHTO LRFD Channel Flexure (6.12.2.2.5)

The provisions for channels in flexure about their strong or x-axis are taken from AISC (2005). (C6.12.2.2.5)

However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-11) may be used for the development of AASHTO capacities.

MC 8x8.5 ~ Fy=36ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 12kip \times ft + 3kip \times ft \times \frac{4}{5} = 14.4 \times kip \times ft$$

$$\varphi M_{n.AASHTO.MC8x8.5} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 16.0 \times kip \times ft$$

$$\frac{M2_{Canti.new}}{\varphi M_{n.AASHTO.MC8x8.5}} = 0.699$$

$$\boxed{Check_{C.D}\left(\varphi M_{n.AASHTO.MC8x8.5}, M2_{Canti.new}\right) = \text{"SATISFACTORY"}}$$

New Channel Cantilever Load applies an eccentric load to the anchor connections

Eccentrically Loaded Irregular Bolt Group - AASHTO LRFD

Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): $A325 := \text{"yes"}$ $A490 := \text{"No"}$

$ASTM = \text{"A325"}$

Nominal Bolt Diameter: $d := \frac{5}{8} in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.31 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$$F_{ub} := \begin{array}{|l} FUB \leftarrow 0ksi \\ FUB \leftarrow 150ksi \ \text{ if } \ ASTM = \text{"A490"} \\ \text{if } \ ASTM = \text{"A325"} \\ \quad \begin{array}{|l} FUB \leftarrow 120ksi \ \text{ if } \ d \le 1.0in \\ FUB \leftarrow 105ksi \ \text{ if } \ d > 1.0in \end{array} \end{array}$$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 1$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 14 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9)

For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter

Thickness of the connected material: $t := 0.179in$

Tensile strength of the connected material specified in Table 6.4.1-1:

Grade 36: $F_u := 58ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 15.6 \times kips$

Strength Resistance minimum of bolt shear and bearing

$\boxed{R_n := \min(R_{n.shear}, R_{n.holes}) = 13.99 \times kips}$

$\varphi := 0.80$ For A325 and A490 Bolts in Shear and Tension

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Irregular, eccentrically loaded bolt group shear check - Instantaneous Rotation Method (AISC 7-6)

Requirements: ***All bolts must be the same size and strength***
Bolt CG must be right of load (farther positive)
Main load must be oriented vertically (May require "rotation" of bolt group)

Bolt Pattern:

$Bolt_1 := (0in \;\; 0in)$ $Bolt_2 := (6in \;\; 0in)$ $Bolt_3 := (12in \;\; 0in)$ $Bolt_4 := (0in \;\; 6in)$ $Bolt_5 := (6in \;\; 6in)$ $Bolt_6 := (12in \;\; 6in)$

Shear capacity of one bolt: $R_n = 13.99 \times kips$

Resistance Factor for Strength design of bolts in bearing: $\varphi_s := 0.80$

Define Factored Loading:

Load Location: $Load_X := -Canti_{new.L}$ $Load_Y := 0in$

Shear

Vertical Load Component (Up is positive) $V := -R_{Canti.new}$ $V = -6.073 \times kips$

Horizontal Load Component (Right is positive) $H := 0.000000001kips$ $H = 1 \times 10^{-9} \times kips$

$X^T = (0 \;\; 6 \;\; 12 \;\; 0 \;\; 6 \;\; 12) \times in$

$Y^T = (0 \;\; 0 \;\; 0 \;\; 6 \;\; 6 \;\; 6) \times in$

Geometry Calculations:

Bolt CG: $X_{CG} := \frac{\sum X}{rows(X)}$ $X_{CG} = 6 \times in$ $Y_{CG} := \frac{\sum Y}{rows(Y)}$ $Y_{CG} = 3 \times in$

Load Angle (relative to vertical): $\Theta := atan\left(\frac{H}{-V}\right)$ $\Theta = 9.435 \times 10^{-9} \times deg$

$$int_x := \frac{\left[\frac{V}{-H + (H = 0) \times kip} \times Load_X\right] + Load_Y - \left(\frac{H}{V} \times X_{CG}\right) + Y_{CG}}{\frac{V}{-H + (H = 0) \times kip} + \frac{-H}{V}}$$

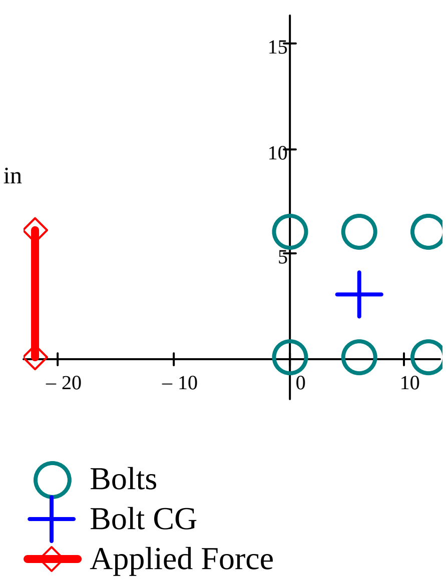
$int_x = -22.10 \times in$

$$int_y := \frac{V}{-H + (H = 0) \times kip} \times int_x - \left[\frac{V}{-H + (H = 0) \times kip} \times Load_X\right] + Load_Y$$

$int_y = 3.00 \times in$

$$ecc := \sqrt{\left(X_{CG} - int_x\right)^2 + \left(Y_{CG} - int_y\right)^2}$$

$ecc = 28.095 \times in$



Coordinates must be rotated to orient load vertically for evalutaion

$V := -\sqrt{V^2 + H^2}$ $\quad H := 0$

$X' := X \quad Y' := Y \quad int_{x'} := int_x \quad int_{y'} := int_y \quad Load_{X'} := Load_X \quad Load_{Y'} := Load_Y$

$X_{CG'} := X_{CG} \quad Y_{CG'} := Y_{CG} \quad X := Coordinate_{rotation}(X', Y', -\Theta)^{\langle 1 \rangle} \quad Y := Coordinate_{rotation}(X', Y', -\Theta)^{\langle 2 \rangle}$

$$\begin{pmatrix} X_{CG} \\ int_x \\ Load_X \end{pmatrix} := Coordinate_{rotation}\left[\begin{pmatrix} X_{CG'} \\ int_{x'} \\ Load_{X'} \end{pmatrix}, \begin{pmatrix} Y_{CG'} \\ int_{y'} \\ Load_{Y'} \end{pmatrix}, -\Theta\right]^{\langle 1 \rangle} \qquad \begin{pmatrix} Y_{CG} \\ int_y \\ Load_Y \end{pmatrix} := Coordinate_{rotation}\left[\begin{pmatrix} X_{CG'} \\ int_{x'} \\ Load_{X'} \end{pmatrix}, \begin{pmatrix} Y_{CG'} \\ int_{y'} \\ Load_{Y'} \end{pmatrix}, -\Theta\right]^{\langle 2 \rangle}$$

$Load_X = -22.095 \times in \quad X_{CG} = 6 \times in$

$Load_Y = 3.638 \times 10^{-9} \times \quad Y_{CG} = 3 \times in$

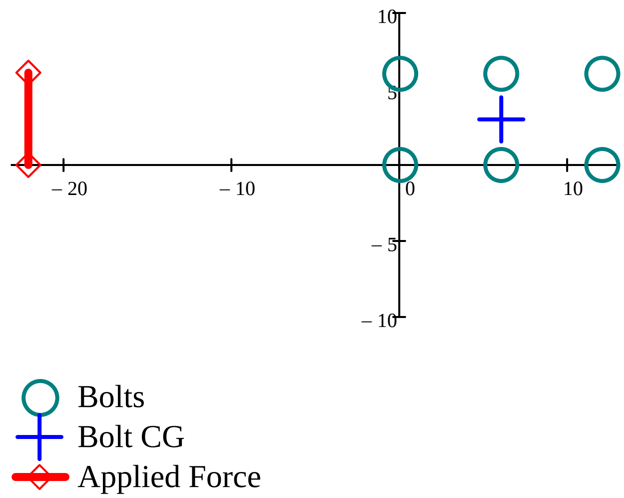
$int_x = -22.095 \times in$

$int_y = 3 \times in \quad V = -6.073 \times kips$

$ecc := \sqrt{(X_{CG} - int_x)^2 + (Y_{CG} - int_y)^2}$

$ecc = 28.095 \times in$

$input_{check}$ = "input is valid"



Bolt shear due to eccentric load:

Plastic deformation limit: $\Delta := 0.34in$

$R_{unit} = 1 \times kip$

$$P := \begin{vmatrix} r_{oX} \leftarrow 0.1in \\ r_{oY} \leftarrow 0.1in \\ yfactor \leftarrow 0.1 \\ Y_{count} \leftarrow 1 \end{vmatrix}$$

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$$
\begin{aligned}
&\text{while } \ yfactor < -0.0000001 \vee yfactor > 0.00000001\\
&\quad r_{oY} \leftarrow r_{oY} + yfactor \times in\\
&\quad factor \leftarrow 1.25\\
&\quad \text{if } \ Y_{count} = 10000\\
&\quad\quad r_{oY} \leftarrow 0\\
&\quad\quad yfactor \leftarrow 0\\
&\quad \text{while } \ factor < 0.999999 \vee factor > 1.000001\\
&\quad\quad r_{oX} \leftarrow r_{oX} - \frac{factor - 1}{1.5} \times r_{oX}\\
&\quad\quad IC_x \leftarrow X_{CG} + r_{oX}\\
&\quad\quad IC_y \leftarrow \frac{-H}{V} \times IC_x - \left(\frac{-H}{V} \times X_{CG}\right) + Y_{CG} + r_{oY}\\
&\quad\quad \text{for } \ i \in 1 .. rows(X)\\
&\quad\quad\quad l_{r_i} \leftarrow \sqrt{\left(X_i - IC_x\right)^2 + \left(Y_i - IC_y\right)^2}\\
&\quad\quad f_{lr} \leftarrow \frac{l_r}{\max\left(l_r\right)}\\
&\quad\quad \text{for } \ i \in 1 .. rows(X)\\
&\quad\quad\quad f_{R_i} \leftarrow \left(1 - e^{-10 \times \frac{\Delta}{in} \times f_{lr_i}}\right)^{0.55}\\
&\quad\quad \text{for } \ i \in 1 .. rows(X)\\
&\quad\quad\quad v_i \leftarrow f_{R_i} \times R_{unit} \times \left(IC_x - X_i\right) \times \left(l_{r_i}\right)^{-1}\\
&\quad\quad\quad h_i \leftarrow f_{R_i} \times R_{unit} \times \left(IC_y - Y_i\right) \times \left(l_{r_i}\right)^{-1}\\
&\quad\quad\quad al_i \leftarrow v_i\\
&\quad\quad\quad m_i \leftarrow f_{R_i} \times R_{unit} \times l_{r_i}\\
&\quad\quad factor \leftarrow \left[\sum al \times \left(-Load_X + IC_x\right)\right] \times \left(\sum m\right)^{-1}\\
&\quad Y_{count} \leftarrow Y_{count} + 1\\
&\quad yfactor \leftarrow -\left(\frac{\sum h}{kips} \times 1\right)\\
&\begin{bmatrix} al \\ v \times \cos(\Theta) + h \times \sin(\Theta) \\ -(v \times \sin(\Theta) + h \times \cos(\Theta)) \end{bmatrix}
\end{aligned}
$$

Number of bolts actively resisting load:

$$\frac{P_1}{kips} = \begin{pmatrix} 0.913 \\ 0.401 \\ -0.777 \\ 0.913 \\ 0.401 \\ -0.777 \end{pmatrix}$$

$$\boxed{\frac{\sum P_1}{kips} = 1.073}$$

$input_{check}$ = "input is valid"

<u>Actual Bolt Shear:</u>

Vertical

$$Shear_V := \frac{V}{\sum P_1} \times P_2$$

Horizontal

$$Shear_H := \frac{V}{\sum P_1} \times P_3$$

Break shear into Horizontal and Vertical components:

Total Shear

Vertical

$$Shear_V = \begin{pmatrix} -5.164 \\ -2.267 \\ 4.394 \\ -5.164 \\ -2.267 \\ 4.394 \end{pmatrix} \times kips$$

Horizontal

$$Shear_H = \begin{pmatrix} 2.042 \\ 4.292 \\ 2.986 \\ -2.042 \\ -4.292 \\ -2.986 \end{pmatrix} \times kips$$

$$Shear_{Bolts} := \sqrt{Shear_V^{\,2} + Shear_H^{\,2}}$$

$$\boxed{Shear_{Bolts} = \begin{pmatrix} 5.553 \\ 4.854 \\ 5.313 \\ 5.553 \\ 4.854 \\ 5.313 \end{pmatrix} \times kips}$$

Plastic Reduction to shear capacity

$$R_{n.plastic} := R_n \times \frac{\max\left(\frac{Shear_{Bolts}}{R_{unit}}\right)}{\left|\frac{V}{\sum P_1}\right|}$$

$$\boxed{R_{n.plastic} = 13.731 \times kips}$$

$$\boxed{\varphi_s \times R_{n.plastic} = 10.985 \times kips}$$

The limiting demand / capacity ratio is: $\boxed{\max\left(\frac{Shear_{Bolts}}{\varphi_s \times R_{n.plastic}}\right) = 0.506}$

occuring at: $\boxed{Bolt_{num} = 1}$

$$\boxed{Check_{C.D}\left[1, \left(\max\left(\frac{Shear_{Bolts}}{R_{n.plastic}}\right)\right)\right] = \text{"SATISFACTORY"}}$$

#4 stirrups are located in existing Cantilever at 6" on center. Stirrups will provide restraining force for anchored connection to resist concrete blowout. Anchorage is acceptable.

Additional dead load to the structure from walkway widening.

Dead Load of Additional Slab: $plf_{slab} := w_{DC} \times Addl_L + [4in \times (4in + t)] \times \gamma_c = 151.593 \times plf$

Dead Load of Railing: $plf_{railing} := R2_{DC.Railing} = 36.6 \times plf$

Dead Load of Edge Beam: $plf_{edge.beam} := 14.51plf$

Dead Load of New Cantiliever Extensions: $plf_{new.canti} := \frac{8.5plf \times \left(Canti_{new.L} + 6in \times 2 + 2in\right) \times 2}{L_{edge}} \times 115\% = 3.267 \times plf$

Dead Load of New Traffic Barrier + BP rail: $plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$

Total weight of new work: $plf_{new} := plf_{slab} + plf_{railing} + plf_{edge.beam} + plf_{new.canti} + plf_{barrier} = 593.2 \times plf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedistal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 58): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedistal weight: $plf_{pedistal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{L_{edge}} = 39.9 \times plf$

Dead Load of Existing intermediate Conc Rail: $plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{L_{edge}} = 127 \times plf$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 59)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{L_{edge}} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{L_{edge}}\right]$$

$plf_{metal.rail} = 16.6 \times plf$

Dead Load of Existing Edge Beam: $plf_{ex.edge.beam} := \frac{(1ft + 1.5in) \times 6in \times \left[L_{edge} - (1ft + 4in)\right] \times \gamma_{c.ex}}{L_{edge}} = 78.1 \times plf$

Dead Load of Existing Curb:
(REF SHT 56) $plf_{curb} := 9.5in \times \frac{(9.5in + 8in)}{2} \times \gamma_{c.ex} = 86.589 \times plf$

Total weight of removals: $plf_{rem} := plf_{pedistal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{curb} = 348.3 \times plf$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 244.9 \times plf$

Weight of Segment 2 calculated for seismic analysis: $Segment2_{weight} := 9000kips$

Length of Segment 2: $Segment2_{length} := 336.17ft$

Average Per foot weight of Segment 2: $\dfrac{Segment2_{weight}}{Segment2_{length}} = 2.677 \times 10^{4} \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment2_{weight}}{Segment2_{length}}} = 0.915 \times \%$ Less than 10% OKAY

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: $H_{mast} := 30ft + (3ft + 7in) = 33.583\ ft$
(REF SHT E-6/91/4 : 782-95)

Length of Luminaire Mast arm: $L_{arm} := 12ft$
(12' max REF WSDOT STD J-28.10-01)

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d$ (psf)

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\ ft$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head (REF WSDOT BDM 10.1(B)) $A := 3.3ft^2$

$V_{wind.lum} := P_z \times A = 44.876\ lb$

Height to point of connection: $H := H_{mast} + 6.5in + \frac{10in}{2} = 34.542\ ft$

$M_{wind.lum} := V_{wind.lum} \times H = 1.55\ ft \times kips$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

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Wind on Mast Arm

$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$

$K_z := K_{z.eq}\left(H_{arm}, 900ft, 9.5\right) = 1.151$

$$C_{d.cylinder}(V,d) := \left| \begin{array}{l} \omega \leftarrow 1.105 \times \dfrac{V}{mph} \times \dfrac{d}{ft} \\ CD \leftarrow \dfrac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{ if } \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{ if } \ \omega \ge 78 \\ CD \end{array} \right.$$

Cylinders (3in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 3in) = 1.1$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$V_{wind.arm} := P_z \times A = 89.752\,lb$

Height to point of connection: $H = 34.542\,ft$

$M_{wind.arm} := V_{wind.arm} \times H = 3.1\,ft \times kips$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$K_z := K_{z.eq}\left(H_{pole}, 900ft, 9.5\right) = 1.079$

Cylinders (8in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 8in) = 0.553$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$V_{wind.arm} := P_z \times A = 315.609\,lb$

Height to point of connection: $H := \dfrac{H_{mast}}{2} + 6.5in + \dfrac{10in}{2} = 17.75\,ft$

$M_{wind.pole} := V_{wind.arm} \times H = 5.602\,ft \times kips$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$	
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		
Single Member or Truss Member	**$C_vVd \le 5.33$ (39)**	**$5.33(39) < C_vVd < 10.66(78)$**	**$C_vVd \ge 10.66(78)$**
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r - \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

Total Moment from wind (omnidirectional):

$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 10.252\,ft \times kips$

Dead Loads

Distance from Pole location to Point of connection: $Offset := Addl_L + 2.5in + 3in + 6in = 2.55\,ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$W_{lum} := 60lb$

$M_{DC.lum} := W_{lum} \times (L_{arm} - Offset) = 0.567\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$

$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.159\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\,lb$

$M_{DC.pole} := W_{pole} \times (-Offset) = -0.876\,ft \times kips$

Total Moment from Dead Load:
(+15% for electrical etc.)

$M_{DC} := 115\% \times (M_{DC.lum} + M_{DC.arm} + M_{DC.pole}) = -0.172\,ft \times kips$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$

$M_{Ice.lum} := Ice_{lum} \times (L_{arm} - Offset) = 0.561\,ft \times kips$

Weight of Arm:

$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$

$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.098\,ft \times kips$

Weight of Pole:

$Ice_{pole} := \pi \times 8in \times 3psf \times H_{pole} = 294.001\,lb$

$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -0.75\,ft \times kips$

Total Moment from Dead Load:

$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -0.091\,ft \times kips$

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Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left|M_{DC}\right| + M_{wind} \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.172 \\ 7.881 \\ 4.052 \end{pmatrix} ft \times kips$$

Governing Load $M_{design} := \max\left(M_{design'}\right) = 7.881\ ft \times kips$

Light pole attaches to 12" concrete cantilever . Cantilever is reinforced with 2-#11 bars at the base and 8-#5 bars at the top. (REF SHT 44)

Tension / Compression moment couple:

$$Demand_T := \frac{M_{design}}{10in} = 9.457 \times kips$$

$$Capacity_{Top.T} := \frac{\left(2 \times A_{no.11}\right) \times 40ksi}{1.67} = 74.731 \times kips$$

$$Capacity_{Bottom.T} := \frac{\left(8 \times A_{no.5}\right) \times 40ksi}{1.67} = 59.401 \times kips$$

$$Check_{C.D}\left(\min\left(Capacity_{Top.T}, Capacity_{Bottom.T}\right), Demand_T\right) = \text{"SATISFACTORY"}$$

Tension couple would acceptably transfer load to existing steel.

Required diameter of anchor

$$A_{req} := \frac{Demand_T \times 1.67}{2 \times 36ksi} = 0.219 \times in^2$$

$$diameter := \sqrt{4 \times \frac{A_{req}}{\pi}} = 0.528 \times in$$

Use 2 - 3/4" diameter F1554 Gr36 Anchors (Tension)
Use 2 top and bottom (4 total)

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$\gamma_c := 155\text{pcf}$

New construction

Existing Edge of curb to edge of deck distance = 13in (REF SHT 31)

Distance from back of curb to face of existing concrete barrier / 6" edge beam = 4ft 0in

Width of new Traffic Barrier: $\text{width}_{\text{barrier}} := (32\text{in} + 6\text{in}) \times \frac{4}{21} + 8\text{in} = 15.238 \times \text{in}$

Distance from back of new Traffic Barrier to centerline of existing edge beam: $\text{span} := 4\text{ft} - \left(\text{width}_{\text{barrier}} - 13\text{in}\right) + \frac{6\text{in}}{2} = 4.063\,\text{ft}$

Distance from centerline of existing edge beam to 6ft walkway limit: $\text{canti} := 6\text{ft} - \text{span} = 1.937\,\text{ft}$

Walkway will be supported by a modified existing edge beam (cantilevering beyond) and monolithically connected to new traffic Barrier (Bounding conditions: FREE-PIN-PIN for design at edge beam ~ PIN-FIX for design at barrier connection)

Thickness of Slab: $t := 4\text{in}$ $\quad w_{DC.slab} := t \times \gamma_c = 51.7 \times \text{psf}$

$w_{LL} := 75\text{psf}$

Dead Load of 54 inch Bicycle Railing: $w_{DC.Railing} := 36.6\text{plf}$

FREE-PIN-PIN Reactions for edge beam design

$$R_{edge.beam.railing} := \frac{w_{DC.Railing} \times (1.5\text{in} + \text{canti} + \text{span} + 4\text{in})}{\text{span} + 4\text{in}} = 53.8 \times \text{plf}$$

(assume 1.5in to centerline of railing & 4" from edge of barrier to vertical restraint doweled bar)

$$R_{edge.beam.slab} := \frac{w_{DC.slab} \times \frac{(6\text{in} + \text{canti} + \text{span} + 4\text{in})^2}{2}}{\text{span} + 4\text{in}} = 274.4 \times \text{plf}$$

(assume 6in to free edge of slab)

$$R_{edge.beam.DC} := R_{edge.beam.railing} + R_{edge.beam.slab} = 328.111 \times \text{plf}$$

PIN-FIX Reactions for barrier side design (Neglecting uplift from cantilevered load

$$R_{barrier.DC} := \frac{5}{8} \times w_{DC.slab} \times (\text{span} + 4\text{in}) = 141.981 \times \text{plf}$$

Cantilever Negative Moment at Edge Beam

$$M35_{canti.serv.neg} := w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2} + w_{LL} \times \frac{canti^2}{2} = 369.4 \times \frac{lb \times ft}{ft}$$

$$M35_{canti.neg} := 1.25 \times \left[w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2} \right] + 1.75 \times \left(w_{LL} \times \frac{canti^2}{2} \right) = 532.1 \times \frac{lb \times ft}{ft}$$

Midspan Positive Bending (PIN-PIN - with partial Live Load contribution for maximum moment)

$$M35_{DC.slab.pos} := \frac{w_{DC.slab}}{8 \times (span + 4in)^2} \times (span + 4in + canti + 6in)^2 \times [span + 4in - (canti + 6in)]^2 = 59.946 \times \frac{lb \times ft}{ft}$$

$$M35_{LL.slab.pos} := \frac{w_{LL} \times span^2}{8} = 154.8 \times \frac{lb \times ft}{ft}$$

$$M35_{slab.serv.pos} := 1.25 \times M35_{DC.slab.pos} + 1.75 \times M35_{LL.slab.pos} = 345.8 \times \frac{lb \times ft}{ft}$$

$$M35_{slab.pos} := 1.25 \times M35_{DC.slab.pos} + 1.75 \times M35_{LL.slab.pos} = 345.8 \times \frac{lb \times ft}{ft}$$

Barrier End Negative Bending (Calculate as PIN-FIX span plus pure bending from cantilever)

$$M35_{slab.serv.neg} := \left(w_{DC.slab} + w_{LL} \right) \times \frac{(span + 4in)^2}{8} + M35_{canti.serv.neg} = 675.5 \times \frac{lb \times ft}{ft}$$

$$M35_{slab.neg} := \left(1.25 \times w_{DC.slab} + 1.75 \times w_{LL} \right) \times \frac{(span + 4in)^2}{8} + M35_{canti.neg} = 1 \times 10^3 \times \frac{lb \times ft}{ft}$$

Cantilever Shear at Edge Beam

$$V35_{canti} := 1.25 \times \left[w_{DC.Railing} + w_{DC.slab} \times (canti + 6in) \right] + 1.75 \times \left(w_{LL} \times canti \right) = 457.274 \times plf$$

$$V35_{slab} := 1.25 \times w_{DC.slab} \times \frac{(span + 4in)}{2} + 1.75 \times w_{LL} \times \frac{(span + 4in)}{2} = 430.522 \times plf$$

$$V_{35} := \max\left(V35_{canti}, V35_{slab} \right) = 457.274 \times plf$$

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Design Slab Reinforcing for Maximum Negative Moment

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M35_{slab.neg} \times w_{mem} = 1.005\,ft \times kips$

Service Moment: $M_{service} := M35_{slab.serv.neg} \times w_{mem} = 0.676\,ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 1.75 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.199 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.135 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 1.005 \times kip \times ft$

$\phi M_n = 2.621 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 260.7 \times \%$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.333\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)
The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf} \right) \right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

Determine strain compatible neutral axis

$$n := \frac{E_s}{E_c} = 7.563$$

$$w_{mem} \times x \times \left(\frac{x}{2} \right) = (A_s \times w_{mem}) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2} \right) \times x^2 + (A_s \times w_{mem} \times n) \times x - (A_s \times w_{mem} \times n \times d) = 0$$

$$x := \max\left[Quadratic_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2} \right)}{ft}, \frac{(A_s \times w_{mem} \times n)}{ft^2}, \frac{-(A_s \times w_{mem} \times n \times d)}{ft^3} \right] \times 12in \right] = 0.72 \times in$$

Moment Arm: $$arm := d - \frac{x}{3} = 1.51 \times in$$

Service Steel Tension: $$T := \frac{M_{service}}{arm} = 5.369 \times kips$$

Stress in the steel at service limit state $$f_s := \frac{T}{A_s \times w_{mem}} = 13.423 \times ksi$$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 2 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times (h_{mem} - d_c)} = 2.429$$

$$s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 12.105 \times in$$

$$spacing \times (1 + bundles) = 6 \times in$$

$$Check_{C.D}[s_{max}, spacing \times (1 + bundles)] = \text{"SATISFACTORY"}$$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\phi_v := 0.9$

$$V_u := V_{35} \times w_{mem} = 0.457 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := \text{"yes"}$

$\beta_w := 2$ $\theta := 45deg$

$$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$$

$$\phi_v \times V_c = 2.15 \times kips$$

$$Check_{C.D}(\phi_v \times V_c, V_u) = \text{"SATISFACTORY"}$$

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Design Slab Reinforcing for Maximum Positive Moment

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 1.5in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 6in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.2 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M35_{slab.pos} \times w_{mem} = 0.173\,ft \times kips$

Service Moment: $M_{service} := M35_{slab.serv.pos} \times w_{mem} = 0.173\,ft \times kips$

depth to rebar
$$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$
$d = 2.25 \times in$

Rectangular Compression Depth:
$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$
$a_{req} = 0.051 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.017 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 0.173 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 1.76 \times kip\,ft$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$ $\frac{\phi M_n}{M_{des}} = 1 \times 10^3 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$
in $full_{layers} = 1$ (total bars $n = 1$)
spaced at $spacing = 6 \times in$
$w_{mem} = 0.5\,ft$ x $h_{mem} = 0.333\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 1.184 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf} \right) \right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2} \right) = \left(A_s \times w_{mem} \right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2} \right) \times x^2 + \left(A_s \times w_{mem} \times n \right) \times x - \left(A_s \times w_{mem} \times n \times d \right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2} \right)}{ft}, \frac{\left(A_s \times w_{mem} \times n \right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d \right)}{ft^3} \right] \times 12in \right] = 0.842 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.969 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 1.054 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 5.269 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.5 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c \right)} = 1.857$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 50.655 \times in}$$

$$spacing \times (1 + bundles) = 6 \times in$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles) \right] = \text{"SATISFACTORY"}$$

Load to Edge Beam

Dead Load Reaction (FREE-PIN-PIN)

$$R_{edge.beam.DC} = 328.111 \times plf$$

Live Load Reaction

$$R_{edge.beam.LL} := \frac{w_{LL} \times 6ft \times \left(\frac{6ft}{2} + 4in\right)}{span + 4in} = 341.155 \times plf$$

Edge Beam Length: (REF SHT 38)

$$L_{edge} := 20ft$$

Edge beam is PIN-FIX: (REF SHT 38)

$$V35_{edge} := 1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL} = 1.007 \times 10^{3} \times plf$$

$$M35_{edge.pos} := \frac{\left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge}^{2}}{24} = 16.8\,ft \times kips$$

$$M35_{edge.neg} := \frac{\left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge}^{2}}{12} = 33.6\,ft \times kips$$

$$M35_{edge.serv.pos} := \frac{\left(R_{edge.beam.DC} + R_{edge.beam.LL}\right) \times L_{edge}^{2}}{24} = 11.2\,ft \times kips$$

$$M35_{edge.serv.neg} := \frac{\left(R_{edge.beam.DC} + R_{edge.beam.LL}\right) \times L_{edge}^{2}}{12} = 22.3\,ft \times kips$$

Check Existing Edge Beam for Capacity

$$E_s := 29000ksi$$

Original structure concrete strength:

$$f'_c := 2.5ksi$$

Original structure Steel Strength:

$$F_y := 33ksi$$

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Check Existing Edge Beam for Positive Capacity (Neglect compression in new slab)

Bar info (REF SHT 38): $bar_{area} := 1\frac{1}{8}in^2$ $bar_{diam} := \sqrt{bar_{area}} = 1.061 \times in$

Assumed Clear: $clear := 2in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member: $w_{mem} := 6in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member (minus new slab depth): $h_{mem} := (1ft + 1in) + 8in - 4in = 1.417\,ft$

$A_s := \frac{bar_{area}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 1.125 \times in^2$ $clear_{spacing} = 4.939 \times in$

Design Moment: $M_{des} := M35_{edge.pos} = 16.786\,ft \times kips$

Service Moment: $M_{service} := M35_{edge.serv.pos} = 11.154\,ft \times kips$

depth to rebar: $d := h_{mem} - clear - \left[\frac{bar_{diam}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max(1 \times in, bar_{diam}) + bar_{diam}\right)\right]$

$d = 14.47 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 1.269 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.49 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 2.912 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 16.786 \times kip \times ft$

$\phi M_n = 36.235 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 215.9 \times \%$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 585 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 578 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 28.179 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 6 \times in$

$w_{mem} = 0.5\,ft$ x $h_{mem} = 1.417\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 16.907 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3031 \times ksi$$

$$n := \frac{E_s}{E_c} = 9.567$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 5.631 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 12.593 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 10.629 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 9.448 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 2 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.19$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi) \ (in) - 2 \times d_c = 42.674 \times in}$$

$$spacing \times (1 + bundles) = 6 \times in$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$a_g := 0.75in$ (Aggregate Size) $\phi_v := 0.9$

$V_u := \frac{5}{8} \times V35_{edge} \times L_{edge} = 12.59 \times kips$

Use Simplified Calc Values?
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := "no"$

Strain in tension reinforcement:

$$\varepsilon_s := \frac{\left[\frac{|M_{des}|}{(0.9 \times d)} + |V_u|\right]}{E_s \times A_s \times w_{mem}} = 8.6 \times 10^{-4}$$

Without min Transverse Steel: $s_x := 0.9 \times d = 13.02 \times in$ (approximate distance between crack control steel layers) $s_{xe} := s_x \times \frac{1.38}{a_g + 0.63in} = 13.023$

Angle of inclination of diagonal compressive strut

$$\beta_{wo} := \frac{4.8}{(1 + 750 \times \varepsilon_s)} \times \frac{51}{(39 + s_{xe})} = 2.861$$

$V_{c.wo} := 0.0316 \times \beta_{wo} \times \sqrt{f'_c\,(ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_{c.wo} = 10.051 \times kips$

$ShearReinf_{Req} = "Factored Capacity is less than 2x demand - Reinforcing Required"$

2x requirement does not apply to slabs, footings, or culverts (5.8.2.4)

With min Transverse Steel:

$$\beta_w := if\left[yes_{no}(Simplified) = 1, 2.0, \frac{4.8}{(1 + 750 \times \varepsilon_s)}\right] = 2.918$$

$$\theta := if\left[yes_{no}(Simplified) = 1, 45, (29 + 3500 \times \varepsilon_s)\right] deg = 32.01 \times deg$$

$V_{c.w} := 0.0316 \times \beta_w \times \sqrt{f'_c\,(ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_{c.w} = 10.252 \times kips$

$$v_u := \frac{|V_u|}{(0.9 \times d) \times w_{mem}} = 0.161 \times ksi$$

$s_{max} := if(v_u < 0.125 \times f'_c, min(0.8 \times d, 24in), min(0.4 \times d, 12in)) = 11.576 \times in$

$s_{stirrup} := 12in$ $A_{v.min} := 0.0316 \times \sqrt{f'_c\,(ksi)} \times \frac{w_{mem} \times s_{stirrup}}{F_y}$ $A_{v.min} = 0.109 \times in^2$

Minimum area applies only to sections where 2x requirement applies (5.8.2.5)

$A_{stirrup} := 2 \times A_{no.5}$ $A_{stirrup} = 0.62 \times in^2$

(REF SHT 38)

$minimum = "Minimum Provided"$

$\varphi V_c := \phi_v \times if(A_{stirrup} \geq A_{v.min}, V_{c.w}, V_{c.wo}) = 10.252 \times kips$

angle between transverse reinforcing and longitudinal axis $\alpha := 90deg$

$$V_s := \frac{A_{stirrup} \times F_y \times (0.9 \times d) \times (cot(\theta) + cot(\alpha)) \times sin(\alpha)}{s_{stirrup}}$$

$\varphi V_s := \phi_v \times V_s = 31.968 \times kips$

$\phi V_{n_1} := \varphi V_c + \varphi V_s$ $\phi V_{n_1} = 42.22 \times kips$

Governing Shear Capacity:

$\phi V_{n_2} := \phi_v \times 0.25 \times f'_c \times w_{mem} \times (0.9 \times d)$ $\phi V_{n_2} = 43.952 \times kips$

$\phi V_n := min(\phi V_n)$ $\phi V_n = 42.22 \times kips$

$Check_{C.D}(\phi V_n, V_u) = "SATISFACTORY"$

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Deflection of Edge Beam

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$f'_c := 4ksi$ Specified compressive strength of concrete (ksi)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

Live Load on edge beam (service): $R_{edge.beam.LL} = 341.155 \times plf$

$$CG_{bot} := \frac{[6in \times [(1ft + 1in) + 8in]] \times \frac{[(1ft + 1in) + 8in]}{2} + [(4ft - 6in) \times 4in] \times \left[(1ft + 1in) + 8in - \frac{4in}{2}\right]}{[6in \times [(1ft + 1in) + 8in]] + [(4ft - 6in) \times 4in]} = 15.357 \times in$$

$$I_x := \frac{6in \times [(1ft + 1in) + 8in]^3}{12} + [6in \times [(1ft + 1in) + 8in]] \times \left[CG_{bot} - \frac{[(1ft + 1in) + 8in]}{2}\right]^2 \ldots = 1.006 \times 10^4 \times in^4$$
$$+ \frac{(4ft - 6in) \times (4in)^3}{12} + [(4ft - 6in) \times 4in] \times \left[CG_{bot} - \left[(1ft + 1in) + 8in - \frac{4in}{2}\right]\right]^2$$

PIN - FIX Deflection: $\delta := \frac{R_{edge.beam.LL} \times L_{edge}{}^4}{185 \times E_c \times I_x} = 0.013 \times in$ $\qquad \frac{L_{edge}}{\delta} = 18151$

Live Load Deflection on sidewalk supporting members is not significant.

Live Load Deflection is satisfactory

Check Existing Edge Beam for Negative Capacity (Including tension in new slab)

$f'_c = 4 \times ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2.5in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 12 \times in$

Height of member (minus new slab depth) $h_{mem} := (1ft + 1in) + 8in = 1.75\,ft$

$$A_s := \frac{bar_{area}}{spacing} \times full_{layers}$$ $A_s \times w_{mem} = 1.125 \times in^2$ $clear_{spacing} = 10.939 \times in$

Design Moment: $M_{des} := M35_{edge.neg} = 33.572\,ft \times kips$

Service Moment: $M_{service} := M35_{edge.serv.neg} = 22.309\,ft \times kips$

depth to rebar
$$d := h_{mem} - clear - \left[\frac{bar_{diam}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max(1 \times in, bar_{diam}) + bar_{diam}\right)\right]$$

$d = 17.97 \times in$

Rectangular Compression Depth: $$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$ $a_{req} = 0.621 \times in$

Required Area of Steel: $$A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$$ $A_{s.req} \times w_{mem} = 0.422 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 33.572 \times kip \times ft$

$$a := \frac{A_s \times F_y}{0.85 \times f'_c}$$ $a = 1.654 \times in$ $$\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$$ $\phi M_n = 86.784 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$ Compression block is less than 4" OK $\dfrac{\phi M_n}{M_{des}} = 258.5 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $S = 882 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 54.39 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 12 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 1.75\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 65.268 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 3834 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 4.389 \times \text{in}$$

Moment Arm: $\text{arm} := d - \frac{x}{3} = 16.507 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{\text{arm}} = 16.218 \times \text{kips}$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 14.416 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := \text{clear} = 2.5 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.193$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(\text{ksi})\ (\text{in}) - 2 \times d_c = 25.525 \times \text{in}}$$

$$\text{spacing} \times (1 + \text{bundles}) = 12 \times \text{in}$$

$$\text{Check}_{C.D}\left[s_{max}, \text{spacing} \times (1 + \text{bundles})\right] = \text{"SATISFACTORY"}$$

Tension load to new slab must travel though shear friction at the existing concrete to new concrete interface

Use Galvanized Headed Anchor Stud ILO rebar $f'_c = 4 \times ksi$ $F_y := 36ksi$ For shear $\phi := 0.9$ (5.5.4.2.1)

Interface Shear Transfer (5.8.4)

Slab on clean concrete

$c := 0.075ksi$

$\mu := 0.6$

$K_1 := 0.2$

$K_2 := 0.8ksi$

For cast-in-place concrete slab on clean concrete girder surfaces, intentionally roughened to 0.25in
c = 0.28 ksi μ = 1.0 K1 = 0.3 K2 = 1.8 ksi (NW) or 1.3 ksi (LW)
For normal weight concrete placed monolithically
c = 0.40 ksi μ = 1.4 K1 = 0.25 K2 = 1.5 ksi
For normal weight concrete placed monolithically, or against surface intentionally roughened to 0.25in
c = 0.24 ksi μ = 1.0 K1 = 0.25 K2 = 1.0 ksi
For normal weight concrete against surface intentionally roughened to 0.25in
c = 0.24 ksi μ = 1.0 K1 = 0.25 K2 = 1.5 ksi
For normal weight concrete against surface not intentionally roughened
c = 0.075 ksi μ = 0.6 K1 = 0.2 K2 = 0.8 ksi

$A_{cv} := 6in = 72 \times \frac{in^2}{ft}$ Area of Concrete Engaged in Shear Transfer

$A_{vf} := \pi \times \frac{(0.5in)^2}{4} \times \frac{1}{18in} = 0.131 \times \frac{in^2}{ft}$ Area of Shear Reinforcement Crossing the Shear Plane

$P_c := R_{edge.beam.DC} = 328.111 \times plf$

Permanent net compressive force normal to the shear plane;
if force is tensile $P_c = 0.0kips$ (conservative)

$$V_{n'_1} := c \times A_{cv} + \mu \left(A_{vf} \times F_y + P_c\right) = 8.424 \times klf$$

Must be less than: $V_{n'_2} := K_1 \times f'_c \times A_{cv} = 57.6 \times klf$ $V_{n'_3} := K_2 \times A_{cv} = 57.6 \times klf$

$$V_n := \min\left(V_{n'}\right) = 8.424 \times klf$$

$$\phi \times V_n = 7.582 \times klf$$

Shear Demand:
(Zero moment to maximum moment = L/4 REF AISC Table 3-23)

$$Demand := \frac{A_{no.4} \times 60ksi}{\frac{L_{edge}}{4}} = 2.4 \times klf$$

$Check_{C.D}\left(\phi \times V_n, Demand\right) = \text{"SATISFACTORY"}$

Use 1/2" diameter stud at 18" to connect existing edge beam

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Tension dowel requirement at Barrier

Dead Load reaction at barrier: $R_{barrier.DC} = 141.981 \times plf$

Partial Live Load reaction at barrier: $R_{barrier.LL} := \dfrac{-w_{LL} \times \dfrac{canti^2}{2}}{span + 4in} = -31.984 \times plf$

Uplift at Barrier: $-1.75 \times R_{barrier.LL} - 0.9 \times R_{barrier.DC} = -71.811 \dfrac{lb}{ft}$

No Uplift

Partial Wind Uplift:

AASHTO specifies a superstructre wind uplift load of 20psf times the width of the bridge applied at the windward quarter point. To create an equivalent distributed force and resultant, a triangular distribution of wind load will be established across the windward 3/4 of the bridge width. The Highest pressure will occur under the proposed sidewalk.

$$p_{wind} := \frac{20psf \times W}{\dfrac{3}{4} \times W} \times 2 = 53.333 \times psf$$

$w_{DC.slab} = 51.667 \times psf$

53.3psf is nearly equivalent to the 51.7psf slab dead load. The slab is acceptable for wind uplift loads

Conservatively apply maximum wind to area tributary to barrier reaction.

$$R_{barrier.WL} := -p_{wind} \times \frac{span}{2} = -108.36 \times plf$$

Uplift at Barrier:

$$Uplift_I := -\left(1.75 \times R_{barrier.LL} + 0.9 \times R_{barrier.DC}\right) = -71.811 \times plf$$

$$Uplift_{III} := -\left(1.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = 23.921 \times plf$$

$$Uplift_V := -\left(1.35 \times R_{barrier.LL} + 0.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = -41.261 \times plf$$

$$\max\left(Uplift_I, Uplift_{III}, Uplift_V\right) = 23.921 \times plf$$

$$Tension := \frac{0.9 \times A_{no.4} \times 60ksi}{4ft} = 2.7 \times 10^3 \times plf$$

Uplift force is minor Install #4 bars vertically into slab at 4ft on center.

WSDOT BDM 5.5.4 requires 8 inches of epoxy embedment for full development.

Thickness of the deck is 6.5 inches
Thickness at the curb is 9 inches (REF SHT 30)

Full Development is not required. Provide 6" embed

$$\frac{6in}{8in} \times Tension = 2.025 \times 10^3 \times plf$$

$$\boxed{Check_{C.D}\left(\frac{6in}{8in} \times Tension, \max\left(Uplift_I, Uplift_{III}, Uplift_V\right)\right) = \text{"SATISFACTORY"}}$$

Additional dead load to the structure from walkway widening.

Additional Dead Load, Slab and Railing: $plf_{slab.railing} := w_{DC.slab} \times (span + canti + 6in) + w_{DC.Railing} = 372.4 \times plf$

Dead Load of New Traffic Barrier + BP rail: $plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$

Total weight of new work: $plf_{new} := plf_{slab.railing} + plf_{barrier} = 759.7 \times plf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedestal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 37 & 38): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\, ft^2$

Pedestal weight: $plf_{pedestal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{L_{edge}} = 35.9 \times plf$

Dead Load of Existing intermediate Conc Rail: $plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{L_{edge}} = 128 \times plf$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{L_{edge}} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{L_{edge}}\right]$$

$plf_{metal.rail} = 16.4 \times plf$

Dead Load of Existing Edge Beam: $plf_{ex.edge.beam} := 6in \times 4in \times \gamma_{c.ex} = 25.0 \times plf$

Dead Load of Existing sidewalk:
(REF SHT 37)
$plf_{ex.sidewalk} := 3.5in \times 4ft \times \gamma_{c.ex} = 175 \times plf$

Dead Load of Existing Curb:
(REF SHT 38)
$plf_{curb} := \left[18in \times \left(7in + \frac{3in}{2}\right) - 3in \times 2in\right] \times \gamma_{c.ex} = 153.125 \times plf$

Total weight of removals:

$$plf_{rem} := plf_{pedestal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{ex.sidewalk} + plf_{curb} = 533.5 \times plf$$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 226.1 \times plf$

Weight of Segment 3 calculated for seismic analysis: $Segment3_{weight} := 3863kips$

Length of Segment 3: $Segment3_{length} := 419.92ft$

Average Per foot weight of Segment 3: $\dfrac{Segment3_{weight}}{Segment3_{length}} = 9.199 \times 10^{3} \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment3_{weight}}{Segment3_{length}}} = 2.458 \times \%$ Less than 10% OKAY

Weight of Segment 5 calculated for seismic analysis: $Segment5_{weight} := 13395kips$

Length of Segment 5: $Segment5_{length} := 1979.51ft - 519.48ft = 1.46 \times 10^{3}\ ft$

Average Per foot weight of Segment 5: $\dfrac{Segment5_{weight}}{Segment5_{length}} = 9.174 \times 10^{3} \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment5_{weight}}{Segment5_{length}}} = 2.465 \times \%$ Less than 10% OKAY

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Check Existing Steel Transverse Beam

Point Load to Edge Beam

Edge Beam Length:
(REF SHT 38)

$L_{edge} := 20ft$

Edge beam is FIX-FIX:
(REF SHT 38)

Dead Load Reaction:

$P_{edge.slab.rail} := R_{edge.beam.DC} \times L_{edge} = 6.562 \times kips$

Dead Load of Existing edge beam:

$P_{edge.exbeam} := \gamma_{c.ex} \times 6in \times [(1ft + 1in) + 8in - 4in] \times L_{edge} = 2.125 \times kips$

$P_{edge.DC} := P_{edge.slab.rail} + P_{edge.exbeam} = 8.687 \times kips$

Live Load Reaction:

$P_{edge.pedLL} := R_{edge.beam.LL} \times L_{edge} = 6.823 \times kips$

Point Load to Edge of Deck

Edge of deck is supported by transverse beams at 10ft on center:
(REF SHT 31)

$L_{deck} := 10ft$

Dead Load Reaction:

$P_{deck.slab.rail} := \left(plf_{slab.railing} - R_{edge.beam.DC}\right) \times L_{deck} = 0.443 \times kips$

Dead Load of New Traffic Barrier:

$P_{barrier} := plf_{barrier} \times L_{deck} = 3.872 \times kips$

$P_{deck.DC} := P_{deck.slab.rail} + P_{barrier} = 4.316 \times kips$

Live Load Reaction:

$P_{deck.pedLL} := \left(w_{LL} \times 6ft - R_{edge.beam.LL}\right) \times L_{deck} = 1.088 \times kips$

Distributed Loads

Dead Load of existing Slab:
(REF SHT 31)

$w_{ex.slab} := \gamma_{c.ex} \times \left[8\frac{3}{4}in \times L_{deck} + \left(5\frac{7}{8}in - 2\frac{7}{8}in\right) \times 10in\right] = 1.125 \times klf$

Dead Load of existing transverse girder:

$w_{ex.girder} := 91plf$

Longitudinal Support GIrder 14ft from centerline of bridge (REF SHT 32)
Inside of Curb is 21ft from centerline of bridge (REF SHT 31)

$Curb_{to.girder} := 21ft - 14ft = 7\,ft$

$Curb_{to.girder} - 2ft = 5\,ft$

Live Load (HS20):
(Girders spaced at 10ft, axles spaced at 14ft by lever rule only consider one axle ~ AASHTO LRFD 4.6.2.2.2f)

$HS20_{wheel} := \frac{32kips}{2} = 16 \times kips$

Only one wheel will load the girder

<u>Bending at girder support</u>

Service Moment:

$$M_{at.edge} := \left(P_{edge.DC} + P_{edge.pedLL}\right) \times \left(span + width_{barrier} + Curb_{to.girder}\right) = 191.294\,ft \times kips$$

$$M_{at.EOD} := \left(P_{deck.DC} + P_{deck.pedLL}\right) \times \left(width_{barrier} + Curb_{to.girder}\right) = 44.69\,ft \times kips$$

$$M_{distr} := \frac{w_{ex.slab} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2} + \frac{w_{ex.girder} \times \left(span + width_{barrier} + Curb_{to.girder}\right)^2}{2} = 45.391\,ft \times kips$$

$$M_{veh} := HS20_{wheel} \times \left(Curb_{to.girder} - 2ft\right) = 80\,ft \times kips$$

$$M_{Girder.serv} := M_{at.edge} + M_{at.EOD} + M_{distr} + M_{veh} = 361.375\,ft \times kips$$

No dynamic allowance is required on vehicles when combined with pedestrian Load (AASHTO 3.6.1.6)

Factored Moment:

$$M_{at.edge} := \left(1.25 \times P_{edge.DC} + 1.75 \times P_{edge.pedLL}\right) \times \left(span + width_{barrier} + Curb_{to.girder}\right) = 281.193\,ft \times kips$$

$$M_{at.EOD} := \left(1.25 \times P_{deck.DC} + 1.75 \times P_{deck.pedLL}\right) \times \left(width_{barrier} + Curb_{to.girder}\right) = 60.363\,ft \times kips$$

$$M_{distr} := 1.25 \times \left[\frac{w_{ex.slab} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2} + \frac{w_{ex.girder} \times \left(span + width_{barrier} + Curb_{to.girder}\right)^2}{2}\right] = 56.738\,ft \times kips$$

$$M_{veh} := 1.75 \times HS20_{wheel} \times \left(Curb_{to.girder} - 2ft\right) = 140\,ft \times kips$$

$$M_{Girder.factored} := M_{at.edge} + M_{at.EOD} + M_{distr} + M_{veh} = 538.295\,ft \times kips$$

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Moment Capacity at strength limit state in Major Axis for Discretely braced flanges in Compression - AASHTO LRFD 6.10.8.1.1

$\varphi_f := 1.00$ for flexure $\qquad E := 29000ksi$

Member information (REF SHT 41)

Top Flange: $b_{tf} := 10in$ $\quad t_{tf} := 0.745in$ $\quad A_{tf} := b_{tf} \times t_{tf} = 7.45 \times in^2$ $\quad I_{tf} := \dfrac{b_{tf} \times t_{tf}^3}{12} = 0.345 \times in^4$

Web: $h_{web} := 27in$ $\quad t_{web} := 0.46in$ $\quad A_{web} := h_{web} \times t_{web} = 12.42 \times in^2$ $\quad I_{web} := \dfrac{t_{web} \times h_{web}^3}{12} = 754.515 \times in^4$

Bottom Flange: $b_{bf} := 10in$ $\quad t_{bf} := 0.745in$ $\quad A_{bf} := b_{bf} \times t_{bf} = 7.45 \times in^2$ $\quad I_{bf} := \dfrac{b_{bf} \times t_{bf}^3}{12} = 0.345 \times in^4$

Dimensions shown are from the W27x94 shape. The 91# beam weight shown is silicone steel, which is slightly less dense than carbon steel (~97.5%)

$$CG := \frac{A_{tf} \times \frac{t_{tf}}{2} + A_{web} \times \left(t_{tf} + \frac{h_{web}}{2}\right) + A_{bf} \times \left(t_{tf} + h_{web} + \frac{t_{bf}}{2}\right)}{A_{tf} + A_{web} + A_{bf}} = 14.245 \times in$$

$$I_{total} := I_{tf} + A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right)^2 + I_{web} + A_{web} \times \left(CG - t_{tf} - \frac{h_{web}}{2}\right)^2 + I_{bf} + A_{bf} \times \left(CG - t_{tf} - h_{web} - \frac{t_{bf}}{2}\right)^2 = 3.623 \times 10^3 \times in^4$$

$$S_x := \frac{I_{total}}{CG} = 254.311 \times in^3 \qquad Z_x := 2 \times \left[A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right) + t_{web} \times (CG - t_{tf}) \times \frac{(CG - t_{tf})}{2}\right] = 290.535 \times in^3$$

$F_y := 45ksi$ $\qquad L_b := span + width_{barrier} + Curb_{to.girder} = 12.333\ ft$

At the strength limit state, the following requirement shall be satisfied: $f_{bu} + \frac{1}{3} \times f_l \le \varphi_f \times F_{nc}$

The local buckling resistance (6.10.8.2.2) of the compression flange shall be taken as

if $\lambda_f \le \lambda_{pf}$ then $F_{nc} = R_b \times R_h \times F_{yc}$ $\qquad \lambda_f := \dfrac{b_{bf}}{2 \times t_{bf}} = 6.7$ $\qquad \lambda_{pf} := 0.38 \times \sqrt{\dfrac{E}{F_y}} = 9.647$

$less(\lambda_f, \lambda_{pf})$ = "Less than or Equal to"

Web load-shedding factor determined as specified in Article 6.10.1.10.2 $\qquad R_b = 1.0$ if $\dfrac{2 \times D_c}{t_w} \le \lambda_{rw}$

Depth of web in compression in elastic range $\qquad D_c := h_{web} - (CG - t_{tf}) + t_{bf} = 14.245 \times in$

Limiting slenderness ratio for a noncompact web:

$\dfrac{2 \times D_c}{t_{web}} = 61.935$ $\qquad \lambda_{rw} := 5.7 \times \sqrt{\dfrac{E}{F_y}} = 144.7$ $\qquad less\left(\dfrac{2 \times D_c}{t_{web}}, \lambda_{rw}\right)$ = "Less than or Equal to" $\qquad R_b := 1.0$

Hybrid Factor: $\quad R_h = \dfrac{12 + \beta \times (3 \times \rho - \rho^3)}{12 + 2 \times \beta}$ $\qquad D_n := D_c = 14.245 \times in$ For doubly symmetric sections

$A_{fn} := b_{bf} \times t_{bf} = 7.45 \times in^2$ $\qquad \beta := \dfrac{2 \times D_n \times t_{web}}{A_{fn}} = 1.759$

$\rho := 1.0$ $\qquad$ Non Composite with no cover plates on tensile side

$R_h := \dfrac{12 + \beta \times (3 \times \rho - \rho^3)}{12 + 2 \times \beta} = 1$ $\qquad F_{nc.lb} := R_b \times R_h \times F_y = 45 \times ksi$

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The Lateral Torsional Buckling resistance (6.10.8.2.3) of the compression flange shall be taken as

$L_p = 1.0 \times r_t \times \sqrt{\frac{E}{F_{yc}}}$ Limit of Plastic deformation

$b_{fc} := b_{bf} = 10 \times in$

$t_{fc} := t_{bf} = 0.745 \times in$

$$r_t := \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3} \times \frac{D_c \times t_{web}}{b_{fc} \times t_{fc}}\right)}}$$

$r_t = 0.212\,ft$

$L_p := 1.0 \times r_t \times \sqrt{\frac{E}{F_y}} = 5.37\,ft$

$F_{yr} := 0.7 \times F_y = 31.5 \times ksi$

$L_r := \pi \times r_t \times \sqrt{\frac{E}{F_{yr}}} = 20.165\,ft$

$C_b := 1$ Unbraced Cantilevers or simple spans

$$F_{nc.ltb} := C_b \times \left[1 - \left(1 - \frac{F_{yr}}{R_h \times F_y}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \times F_y = 38.6 \times ksi$$

$F_{nc} := \min(F_{nc.lb}, F_{nc.ltb}) = 38.6 \times ksi$

The Maximum Allowable Unbraced Length (6.10.1.6-3) is:

$$L_b \le 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{M_{yc}}{M_u}}$$

$$L_{max} := 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{F_y \times S_x}{M_{Girder.factored}}} = 8.577\,ft$$

$Check_{C.D}(L_{max}, L_b) = $ "Not Satisfactory"

Kickers may be required

No Lateral bending is anticipated at this location $f_l := 0$

$f_{bu} \le \varphi_f \times F_{nc}$

$S_x \times (\varphi_f \times F_{nc}) = 819 \times kip \times ft$

$M_{Girder.factored} = 538.295\,ft \times kips$

$Check_{C.D}[S_x \times (\varphi_f \times F_{nc}), M_{Girder.factored}] = $ "SATISFACTORY"

$$\frac{M_{Girder.factored}}{[S_x \times (\varphi_f \times F_{nc})]} = 0.657$$

Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck:
(REF SHT E-6/91/4 : 782-95) $H_{mast} := 30ft + (3ft + 7in) = 33.583\,ft$

Length of Luminaire Mast arm:
(12' max REF WSDOT STD J-28.10-01) $L_{arm} := 12ft$

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d \qquad (psf)$$

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head
(REF WSDOT BDM 10.1(B)) $A := 3.3ft^2$

$$V_{wind.lum} := P_z \times A = 44.876\,lb$$

Height to point of connection: $H := H_{mast} + 4ft = 37.583\,ft$

$$M_{wind.lum} := V_{wind.lum} \times H = 1.687\,ft \times kips$$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

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Wind on Mast Arm

$$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_z := K_{z.eq}(H_{arm}, 900ft, 9.5) = 1.151$$

$$C_{d.cylinder}(V,d) := \left| \begin{array}{l} \omega \leftarrow 1.105 \times \frac{V}{mph} \times \frac{d}{ft} \\ CD \leftarrow \frac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{ if } \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{ if } \ \omega \ge 78 \\ CD \end{array} \right.$$

Cylinders (3in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 3in) = 1.1$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$$V_{wind.arm} := P_z \times A = 89.752\,lb$$

Height to point of connection: $H := H_{mast} + 4ft = 37.583\,ft$

$$M_{wind.arm} := V_{wind.arm} \times H = 3.373\,ft \times kips$$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$$K_z := K_{z.eq}(H_{pole}, 900ft, 9.5) = 1.079$$

Cylinders (8in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 8in) = 0.553$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$$V_{wind.pole} := P_z \times A = 315.609\,lb$$

Height to point of connection: $H := \frac{H_{mast}}{2} + 4ft = 20.792\,ft$

$$M_{wind.pole} := V_{wind.pole} \times H = 6.562\,ft \times kips$$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$	
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		

Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37+1.08r-\frac{C_vVd}{19.8}-\frac{C_vVdr}{4.94}$ (SI) $1.37+1.08r-\frac{C_vVd}{145}-\frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55+\frac{(10.66-C_vVd)}{9.67}$ (SI) $0.55+\frac{(78.2-C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

$$V_{wind} := V_{wind.lum} + V_{wind.arm} + V_{wind.pole} = 0.45 \times kips$$

Total Moment from wind (omnidirectional):

$$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 11.622\,ft \times kips$$

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Dead Loads

Distance from Pole location to Point of connection: $Offset := 5in + 3in + 6in + 6ft + 3ft = 10.167\,ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$W_{lum} := 60lb$

$M_{DC.lum} := W_{lum} \times (L_{arm} - Offset) = 0.11\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$

$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = -0.192\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\,lb$

$M_{DC.pole} := W_{pole} \times (-Offset) = -3.492\,ft \times kips$

Total Moment from Dead Load:

$M_{DC} := M_{DC.lum} + M_{DC.arm} + M_{DC.pole} = -3.574\,ft \times kips$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$

$M_{Ice.lum} := Ice_{lum} \times (L_{arm} - Offset) = 0.109\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$

$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = -0.118\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$Ice_{pole} := \pi \times 8in \times 3psf \times H_{mast} = 211.01\,lb$

$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -2.145\,ft \times kips$

Total Moment from Dead Load:

$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -2.154\,ft \times kips$

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Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left(\left|M_{DC}\right| + M_{wind}\right) \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 3.574 \\ 11.425 \\ 8.676 \end{pmatrix} ft \times kips$$

Governing Load $\quad M_{design} := \max\left(M_{design'}\right) = 11.425\, ft \times kips$

Connect pole to square tube:

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1) $\quad M_n = M_p = F_y \times Z$

Flange Compact Criteria Buckling Limit
(AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges
(AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges
(AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange
(AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 10x10x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$M_{n.\Omega.AISC} := 42.8 kip \times ft$$

$$M_{n.\Omega.AASHTO.HSS10x10x3} := \frac{1.0}{0.9} \times M_{n.\Omega.AISC} = 47.6 \times kip \times ft$$

$$\frac{M_{design}}{M_{n.\Omega.AASHTO.HSS10x10x3}} = 0.24$$

$$Check_{C.D}\left(M_{n.\Omega.AASHTO.HSS10x10x3}, M_{design}\right) = \text{"SATISFACTORY"}$$

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HSS Supporting Luminaire shall connect to the girder flange with a pair of attachment plates
Space attachment plates at 24" to reduce the magnitude of the moment couple

Design moment output from AASHTO Signs and Luminaires code is Working Stress. LRFD factor conservatively assumed as 1.4 (wind > 1.25 DC)

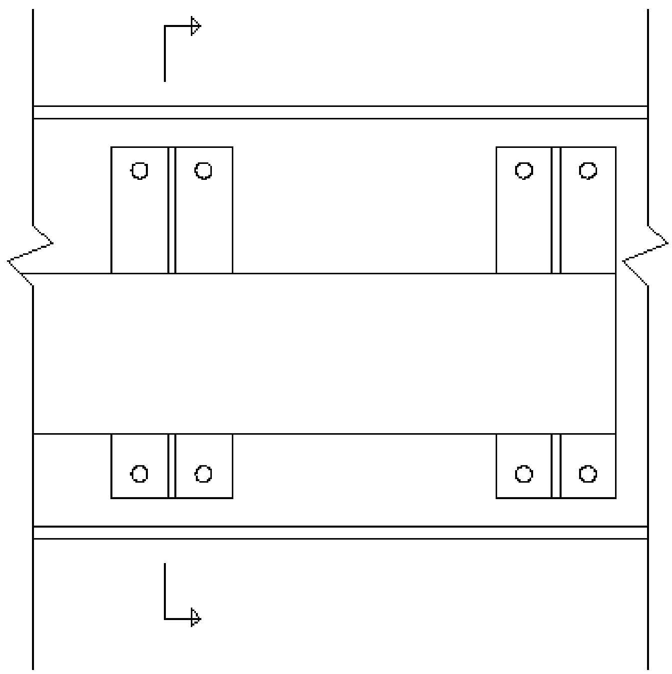
$M_{design} = 11.425\,ft \times kips$

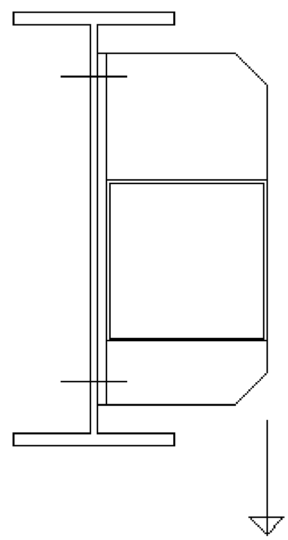
$$Couple := \frac{1.4 \times M_{design}}{2ft} = 7.998 \times kips$$

Load from couple carried by Stiffners to attachment plate:

Small Stiffener is 10" x 4" (10" to connect to outside face of HSS 10 member)
Conservatively place 50% load at top of small stiffener (neglect large stiffener):

$$Moment_{stiffener} := 50\% \times Couple \times 10in = 3.332\,ft \times kips$$





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AASHTO LRFD 5th Edition - Nominal Flexural Resistance of Rectangular Bars and Solid Rounds (6.12.2.2.7)

$L_b := 10in$ $d := 4.5in$ $t := \frac{1}{2} in$ $F_y := 36ksi$ $E := 29000ksi$ $\varphi_f := 1.00$

Moment Demand: $(178.2kips \times \sin(26.7deg)) \times L_b = 66.724\, ft \times kips$

For yielding the nominal flexural resistance shall be taken as:

For rectangular bars with $\frac{L_b \times d}{t^2} \le \frac{0.08 \times E}{F_y}$ in flexure about their major geometric axis, rectangular bars in flexure about their minor geomietric axis or solid rounds:

$M_n = M_p = F_y \times Z \le 1.6 M_y$ $\frac{L_b \times d}{t^2} = 180$ $\frac{0.08 \times E}{F_y} = 64.444$ Does not apply

$S_x := \frac{t \times d^2}{6} = 1.688 \times in^3$

$M_y := S_x \times F_y = 5.062\, ft \times kips$ $1.6 \times M_y = 8.1\, ft \times kips$

$Z := 2 \times \left[\frac{(d \times t)}{2} \times \frac{d}{4}\right] = 2.531 \times in^3$

$M_{p'} := Z \times F_y = 7.594\, ft \times kips$ $M_p := \min(M_{p'}, 1.6 \times M_y) = 7.594\, ft \times kips$

For Lateral Torsional buckling, the nominal flexural rseistance shall be taken as follows for rectangular bars in flexure about their major geometric axis:

If $\frac{0.08 \times E}{F_y} < \frac{L_b \times d}{t^2} \le \frac{1.9 \times E}{F_y}$, then: $M_n = C_b \times \left[1.52 - 0.274 \times \left(\frac{L_b \times d}{t^2}\right) \times \frac{F_y}{E}\right] \times M_y \le M_p$

$C_b := 1.0$ For unbraced Cantilevers (A6.3.3)

$C_b \times \left[1.52 - 0.274 \times \left(\frac{L_b \times d}{t^2}\right) \times \frac{F_y}{E}\right] \times M_y = 7.385\, ft \times kips$

$M_{n.less} := \min\left[C_b \times \left[1.52 - 0.274 \times \left(\frac{L_b \times d}{t^2}\right) \times \frac{F_y}{E}\right] \times M_y, M_p\right] = 7.385\, ft \times kips$

If $\frac{L_b \times d}{t^2} > \frac{1.9 \times E}{F_y}$, then: $M_n = F_{cr} \times S_x \le M_p$

$F_{cr} := \frac{1.9 \times E \times C_b}{\left(\frac{L_b \times d}{t^2}\right)} = 306.111 \times ksi$ $M_{n.greater} := \min(F_{cr} \times S_x, M_p) = 7.594\, ft \times kips$

$\frac{0.08 \times E}{F_y} = 64.444$ $\frac{L_b \times d}{t^2} = 180$ $\frac{1.9 \times E}{F_y} = 1.531 \times 10^3$

$\varphi M_n := \varphi_f \times \mathrm{if}\left(\frac{L_b \times d}{t^2} > \frac{1.9 \times E}{F_y}, M_{n.greater}, \mathrm{if}\left(\frac{0.08 \times E}{F_y} < \frac{L_b \times d}{t^2}, M_{n.less}, M_p\right)\right) = 7.385\, ft \times kips$

$Check_{C.D}(\varphi M_n, Moment_{stiffener}) =$ "SATISFACTORY"

Fillet Weld - AASHTO LRFD 6.13.3.2.4

$\varphi_{e2} := 0.8$

$F_{exx} := 70\text{ksi}$

$R_r := 0.6 \times \varphi_{e2} \times F_{exx} = 33.6 \times \text{ksi}$

$\boxed{Weld_{size} := \frac{5}{16}\text{in}}$

$Moment_{stiffener} = 3.332\,\text{ft} \times \text{kips}$

$$S := \frac{\left(2 \times \frac{Weld_{size}}{\sqrt{2}}\right) \times (4.5\text{in})^2}{6} = 1.492 \times \text{in}^3$$

$$\sigma_m := \frac{Moment_{stiffener}}{S} = 26.81 \times \text{ksi} \qquad \sigma_v := \frac{Couple}{2 \times \frac{Weld_{size}}{\sqrt{2}} \times 4.5\text{in}} = 4.022 \times \text{ksi}$$

$\sigma := \sigma_m + \sigma_v = 30.832 \times \text{ksi}$

$\boxed{Check_{C.D}(R_r, \sigma) = \text{"SATISFACTORY"}}$

Luminaire bolted to Existing Beam

Web Height of existing beam = 27in - 2x0.5in = 26in (REF SHT 37)

Eccentrically Loaded Irregular Bolt Group - AASHTO LRFD

Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): $A325 := \text{"yes"}$ $A490 := \text{"No"}$

$ASTM = \text{"A325"}$

Nominal Bolt Diameter: $d := \frac{5}{8}in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.31 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$$F_{ub} := \left| \begin{array}{l} FUB \leftarrow 0ksi \\ FUB \leftarrow 150ksi \ \text{if} \ ASTM = \text{"A490"} \\ \text{if} \ ASTM = \text{"A325"} \\ \quad \left| \begin{array}{l} FUB \leftarrow 120ksi \ \text{if} \ d \le 1.0in \\ FUB \leftarrow 105ksi \ \text{if} \ d > 1.0in \end{array} \right. \end{array} \right.$$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 1$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 14 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9)

For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter

Thickness of the connected material: $t := 0.460in$

Tensile strength of the connected material specified in Table 6.4.1-1:

Grade 36: $F_u := 70ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 48.3 \times kips$

Strength Resistance minimum of bolt shear and bearing

$R_n := \min(R_{n.shear}, R_{n.holes}) = 13.99 \times kips$

Slip Critical Resistance of a single bolt (6.13.2.8)

Hole Size Factor: $K_h := 1.00$

Surface Condition Factor: $K_s := 0.50$

Minimum Required Bolt Tension: $P_t := 19kips$

$R_{n.slip} := K_h \times K_s \times N_s \times P_t$ $R_{n.slip} = 9.5 \times kips$

Table 6.13.2.8-1 Minimum Required Bolt Tension.

Bolt Diameter, in.	Required Tension-P_t (kip)	
	M 164 (A 325)	M 253 (A 490)
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1-1/8	56	80
1-1/4	71	102
1-3/8	85	121
1-1/2	103	148

Table 6.13.2.8-2 Values of K_h.

for standard holes	1.00
for oversize and short-slotted holes	0.85
for long-slotted holes with the slot perpendicular to the direction of the force	0.70
for long-slotted holes with the slot parallel to the direction of the force	0.60

Table 6.13.2.8-3 Values of K_s.

for Class A surface conditions	0.33
for Class B surface conditions	0.50
for Class C surface conditions	0.33

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Bolt Pattern:

$Bolt_1 := (0in \;\; -7in)$ $Bolt_2 := (6in \;\; -7in)$ $Bolt_3 := (0in \;\; 12in)$ $Bolt_4 := (6in \;\; 12in)$ $Bolt_5 := (24in \;\; -7in)$ $Bolt_6 := (30in \;\; -7in)$

$Bolt_7 := (24in \;\; 12in)$ $Bolt_8 := (30in \;\; 12in)$

"Slip-critical connections shall be proportioned to prevent slip under Load Combination Service II, as specified in Table 3.4.1-1, and to provide bearing, shear and tensile resistance at the applicable strength limit state load combintions." (6.13.2.1.1)

Service Load Combination II (Slip Critical)

Load: Horizontal Component: $H := -(V_{wind.lum} + V_{wind.arm} + V_{wind.pole}) = -450.237\,lb$

Vertical Component: $V := -(W_{lum} + W_{arm} + W_{pole}) = -449.529\,lb$

$CG = (15 \;\; 2.5) \times in$

$m := \frac{V}{H} = 0.998$

Workpoint, or point in line with load path: $WP := \left(-Offset \;\; \frac{M_{wind}}{V_{wind}}\right)$

Resultant Load:

$Load := \sqrt{H^2 + V^2} = 0.636 \times kips$

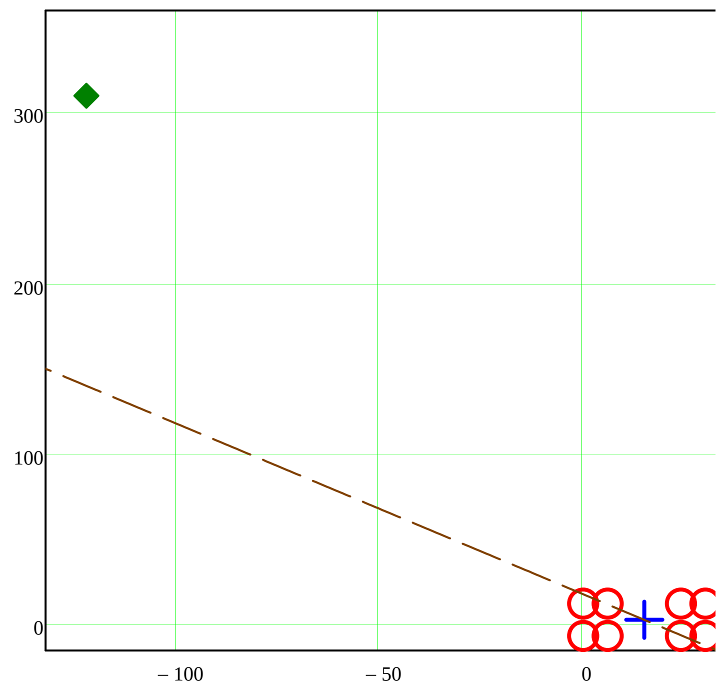
Eccentricity:

$ecc := \sqrt{(CG_{1,1} - INT_{1,1})^2 + (CG_{1,2} - INT_{1,2})^2} = 314.228 \times in$

Eccentric Moment:

$M_{ecc} := Load \times ecc = 199.922 \times kip \times in$

Arm from CG to CL of Bolts and bolt contribution



$$arm = \begin{pmatrix} 17.755 \\ 13.086 \\ 17.755 \\ 13.086 \\ 13.086 \\ 17.755 \\ 13.086 \\ 17.755 \end{pmatrix} \times in \qquad \frac{1kip \times arm}{max(arm)} = \begin{pmatrix} 1 \\ 0.737 \\ 1 \\ 0.737 \\ 0.737 \\ 1 \\ 0.737 \\ 1 \end{pmatrix} \times kips$$

$$Unit_{Rn} := \sum \frac{arm^2}{max(arm)} = 109.6 \times \frac{kip \times in}{kip}$$

Shear from Moment in outermost Bolt(s)

$$\frac{M_{ecc}}{Unit_{Rn}} = 1.824 \times kips$$

Service Load Shear Components

$$V_{ea} := \frac{V}{\text{rows(Bolt)}} = -0.056 \times \text{kips} \qquad H_{ea} := \frac{H}{\text{rows(Bolt)}} = -0.056 \times \text{kips}$$

Shear due to moment in each bolt

$$\frac{M_{ecc}}{Unit_{Rn}} \times \frac{arm}{\max(arm)} = \begin{pmatrix} 1.824 \\ 1.344 \\ 1.824 \\ 1.344 \\ 1.344 \\ 1.824 \\ 1.344 \\ 1.824 \end{pmatrix} \times \text{kips} \qquad V_{ecc} = \begin{pmatrix} -1.541 \\ -0.925 \\ -1.541 \\ -0.925 \\ 0.925 \\ 1.541 \\ 0.925 \\ 1.541 \end{pmatrix} \times \text{kips} \qquad H_{ecc} = \begin{pmatrix} 0.976 \\ 0.976 \\ -0.976 \\ -0.976 \\ 0.976 \\ 0.976 \\ -0.976 \\ -0.976 \end{pmatrix} \times \text{kips}$$

$$V_{combined} := V_{ea} + V_{ecc} = \begin{pmatrix} -1.597 \\ -0.981 \\ -1.597 \\ -0.981 \\ 0.868 \\ 1.485 \\ 0.868 \\ 1.485 \end{pmatrix} \times \text{kips} \qquad H_{combined} := H_{ea} + H_{ecc} = \begin{pmatrix} 0.92 \\ 0.92 \\ -1.032 \\ -1.032 \\ 0.92 \\ 0.92 \\ -1.032 \\ -1.032 \end{pmatrix} \times \text{kips}$$

$$\sqrt{V_{combined}^2 + H_{combined}^2} = \begin{pmatrix} 1.843 \\ 1.345 \\ 1.902 \\ 1.424 \\ 1.265 \\ 1.747 \\ 1.349 \\ 1.808 \end{pmatrix} \times \text{kips}$$

$$V_{bolt.max} := \max\left(\sqrt{V_{combined}^2 + H_{combined}^2}\right) = 1.902 \times \text{kips}$$

$$\boxed{\frac{V_{bolt.max}}{R_{n.slip}} = 0.2}$$

$$\boxed{Check_{C.D}\left(R_{n.slip}, V_{bolt.max}\right) = \text{"SATISFACTORY"}}$$

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Irregular, eccentrically loaded bolt group shear check - Instantaneous Rotation Method (AISC 7-6)

Requirements: ***All bolts must be the same size and strength***
Bolt CG must be right of load (farther positive)
Main load must be oriented vertically (May require "rotation" of bolt group)

Shear capacity of one bolt: $R_n = 13.99 \times \text{kips}$

Resistance Factor for Strength design of bolts in bearing: $\varphi_s := 0.80$

Define Factored Loading:

Load Location: $\text{Load}_X := WP_{1,1}$ $\quad \text{Load}_Y := WP_{1,2}$

Shear

Vertical Load Component (Up is positive) $V := 1.25 \times -\left(W_{lum} + W_{arm} + W_{pole}\right) = -561.911\,\text{lb}$

Horizontal Load Component (Right is positive) $H := 1.4 \times -\left(V_{wind.lum} + V_{wind.arm} + V_{wind.pole}\right) = -630.332\,\text{lb}$

$$X^T = (0 \;\; 6 \;\; 0 \;\; 6 \;\; 24 \;\; 30 \;\; 24 \;\; 30) \times \text{in}$$

$$Y^T = (-7 \;\; -7 \;\; 12 \;\; 12 \;\; -7 \;\; -7 \;\; 12 \;\; 12) \times \text{in}$$

Geometry Calculations:

Bolt CG: $X_{CG} := \dfrac{\sum X}{\text{rows}(X)}$ $\quad X_{CG} = 15 \times \text{in}$ $\qquad Y_{CG} := \dfrac{\sum Y}{\text{rows}(Y)}$ $\quad Y_{CG} = 2.5 \times \text{in}$

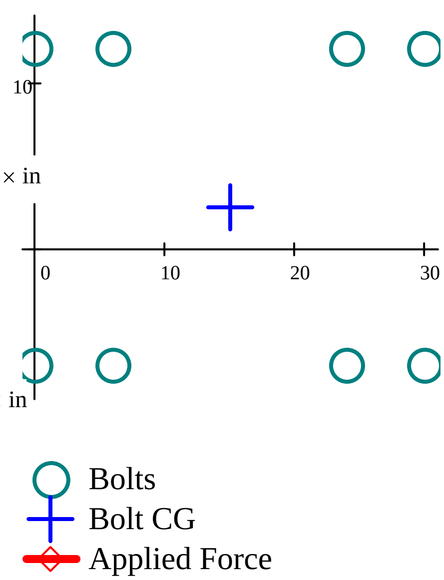
Load Angle (relative to vertical): $\Theta := \text{atan}\left(\dfrac{H}{-V}\right)$ $\quad \Theta = -48.285 \times \text{deg}$

$$\text{int}_x := \frac{\left[\dfrac{V}{-H + (H = 0) \times \text{kip}} \times \text{Load}_X\right] + \text{Load}_Y - \left(\dfrac{H}{V} \times X_{CG}\right) + Y_{CG}}{\dfrac{V}{-H + (H = 0) \times \text{kip}} + \dfrac{-H}{V}}$$

$\text{int}_x = -200.76 \times \text{in}$

$$\text{int}_y := \frac{V}{-H + (H = 0) \times \text{kip}} \times \text{int}_x - \left[\frac{V}{-H + (H = 0) \times \text{kip}} \times \text{Load}_X\right] + \text{Load}_Y$$

$\text{int}_y = 379.97 \times \text{in}$



$$\text{ecc} := \sqrt{\left(X_{CG} - \text{int}_x\right)^2 + \left(Y_{CG} - \text{int}_y\right)^2}$$

$\text{ecc} = 434.782 \times \text{in}$

Coordinates must be rotated to orient load vertically for evalutaion

$V := -\sqrt{V^2 + H^2}$ $H := 0$

$X' := X$ $Y' := Y$ $int_{x'} := int_x$ $int_{y'} := int_y$ $Load_{X'} := Load_X$ $Load_{Y'} := Load_Y$

$X_{CG'} := X_{CG}$ $Y_{CG'} := Y_{CG}$ $X := Coordinate_{rotation}(X', Y', -\Theta)^{\langle 1 \rangle}$ $Y := Coordinate_{rotation}(X', Y', -\Theta)^{\langle 2 \rangle}$

$$\begin{pmatrix} X_{CG} \\ int_x \\ Load_X \end{pmatrix} := Coordinate_{rotation}\left[\begin{pmatrix} X_{CG'} \\ int_{x'} \\ Load_{X'} \end{pmatrix}, \begin{pmatrix} Y_{CG'} \\ int_{y'} \\ Load_{Y'} \end{pmatrix}, -\Theta\right]^{\langle 1 \rangle} \quad \begin{pmatrix} Y_{CG} \\ int_y \\ Load_Y \end{pmatrix} := Coordinate_{rotation}\left[\begin{pmatrix} X_{CG'} \\ int_{x'} \\ Load_{X'} \end{pmatrix}, \begin{pmatrix} Y_{CG'} \\ int_{y'} \\ Load_{Y'} \end{pmatrix}, -\Theta\right]^{\langle 2 \rangle}$$

$Load_X = -312.4 \times in$ $X_{CG} = 8.115 \times in$

$Load_Y = 115.051 \times in$ $Y_{CG} = 12.86 \times in$

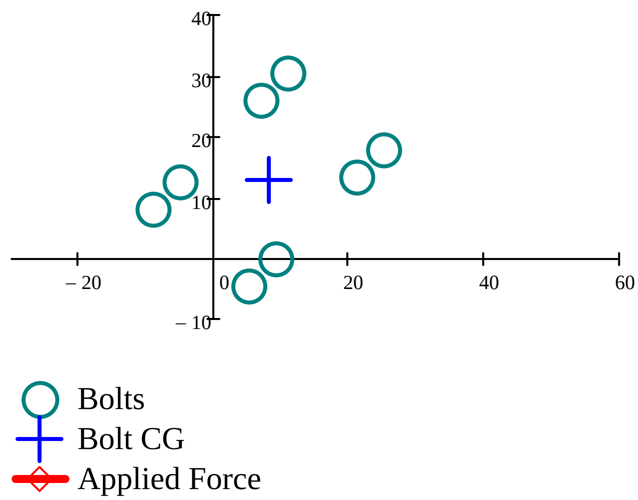
$int_x = -417.224 \times in$

$int_y = 102.98 \times in$ $V = -0.844 \times kips$

$$ecc := \sqrt{(X_{CG} - int_x)^2 + (Y_{CG} - int_y)^2}$$

$ecc = 434.782 \times in$

$input_{check}$ = "input is valid"



Bolt shear due to eccentric load:

$$P := \begin{vmatrix} r_{oX} \leftarrow 0.1in \\ r_{oY} \leftarrow 0.1in \\ yfactor \leftarrow 0.1 \\ Y_{count} \leftarrow 1 \end{vmatrix}$$

Plastic deformation limit: $\Delta := 0.34in$

$R_{unit} = 1 \times kip$

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$$
\begin{aligned}
&\text{while } \ yfactor < -0.0000001 \lor yfactor > 0.00000001\\
&\quad r_{oY} \leftarrow r_{oY} + yfactor \times in\\
&\quad factor \leftarrow 1.25\\
&\quad \text{if } \ Y_{count} = 10000\\
&\quad\quad r_{oY} \leftarrow 0\\
&\quad\quad yfactor \leftarrow 0\\
&\quad \text{while } \ factor < 0.999999 \lor factor > 1.000001\\
&\quad\quad r_{oX} \leftarrow r_{oX} - \frac{factor - 1}{1.5} \times r_{oX}\\
&\quad\quad IC_x \leftarrow X_{CG} + r_{oX}\\
&\quad\quad IC_y \leftarrow \frac{-H}{V} \times IC_x - \left(\frac{-H}{V} \times X_{CG}\right) + Y_{CG} + r_{oY}\\
&\quad\quad \text{for } \ i \in 1..\text{rows}(X)\\
&\quad\quad\quad l_{r_i} \leftarrow \sqrt{\left(X_i - IC_x\right)^2 + \left(Y_i - IC_y\right)^2}\\
&\quad\quad f_{lr} \leftarrow \frac{l_r}{\max\left(l_r\right)}\\
&\quad\quad \text{for } \ i \in 1..\text{rows}(X)\\
&\quad\quad\quad f_{R_i} \leftarrow \left(1 - e^{-10 \times \frac{\Delta}{in} \times f_{lr_i}}\right)^{0.55}\\
&\quad\quad \text{for } \ i \in 1..\text{rows}(X)\\
&\quad\quad\quad v_i \leftarrow f_{R_i} \times R_{unit} \times \left(IC_x - X_i\right) \times \left(l_{r_i}\right)^{-1}\\
&\quad\quad\quad h_i \leftarrow f_{R_i} \times R_{unit} \times \left(IC_y - Y_i\right) \times \left(l_{r_i}\right)^{-1}\\
&\quad\quad\quad al_i \leftarrow v_i\\
&\quad\quad\quad m_i \leftarrow f_{R_i} \times R_{unit} \times l_{r_i}\\
&\quad\quad factor \leftarrow \left[\sum al \times \left(-Load_X + IC_x\right)\right] \times \left(\sum m\right)^{-1}\\
&\quad Y_{count} \leftarrow Y_{count} + 1\\
&\quad yfactor \leftarrow -\left(\frac{\sum h}{kips} \times 1\right)\\
&\begin{bmatrix} al \\ v \times \cos(\Theta) + h \times \sin(\Theta) \\ -(v \times \sin(\Theta) + h \times \cos(\Theta)) \end{bmatrix}
\end{aligned}
$$

Number of bolts actively resisting load:

$$\frac{P_1}{kips} = \begin{pmatrix} 0.226 \\ 0.015 \\ 0.947 \\ 0.956 \\ -0.93 \\ -0.93 \\ 0.174 \\ -0.088 \end{pmatrix}$$

$$\boxed{\frac{\sum P_1}{kips} = 0.37}$$

$input_{check}$ = "input is valid"

Actual Bolt Shear:

Vertical

$$Shear_V := \frac{V}{\sum P_1} \times P_2$$

Horizontal

$$Shear_H := \frac{V}{\sum P_1} \times P_3$$

Break shear into Horizontal and Vertical components:

Total Shear

Vertical

$$Shear_V = \begin{pmatrix} 1.278 \\ 1.59 \\ -0.995 \\ -1.388 \\ 1.383 \\ 0.939 \\ -1.848 \\ -1.52 \end{pmatrix} \times kips$$

Horizontal

$$Shear_H = \begin{pmatrix} 1.06 \\ 1.412 \\ -1.218 \\ -1.572 \\ 1.558 \\ 1.163 \\ -1.709 \\ -1.324 \end{pmatrix} \times kips$$

$$Shear_{Bolts} := \sqrt{{Shear_V}^2 + {Shear_H}^2}$$

$$\boxed{Shear_{Bolts} = \begin{pmatrix} 1.661 \\ 2.126 \\ 1.573 \\ 2.097 \\ 2.083 \\ 1.495 \\ 2.517 \\ 2.016 \end{pmatrix} \times kips}$$

Plastic Reduction to shear capacity

$$R_{n.plastic} := R_n \times \frac{\max\left(\frac{Shear_{Bolts}}{R_{unit}}\right)}{\left|\frac{V}{\sum P_1}\right|}$$

$$\boxed{R_{n.plastic} = 15.428 \times kips}$$

$$\boxed{\varphi_s \times R_{n.plastic} = 12.342 \times kips}$$

The limiting demand / capacity ratio is: $\boxed{\max\left(\frac{Shear_{Bolts}}{\varphi_s \times R_{n.plastic}}\right) = 0.204}$

occuring at: $\boxed{Bolt_{num} = 7}$

$$\boxed{Check_{C.D}\left[1, \left(\max\left(\frac{Shear_{Bolts}}{R_{n.plastic}}\right)\right)\right] = \text{"SATISFACTORY"}}$$

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BALLARD - 6FT - SEGMENTS 6

New construction

$\gamma_c := 155pcf$

Existing overhang = 5ft 4in from inside curb to outside of existing concrete barrier (REF SHT 36)

Width of new Traffic Barrier:

$$width_{barrier} := (32in + 6in) \times \frac{4}{21} + 8in = 15.238 \times in$$

Existing Exterior Girder is shown as B36 x 16 1/2 x 230# SIL (REF SHT 52)

Distance from back of new Traffic Barrier to centerline of existing edge beam:

$$span := (5ft + 4in) - width_{barrier} - \frac{6in}{2} = 3.813\,ft$$

Distance from centerline of existing edge beam to 6ft walkway limit:

$$canti := 6ft - span = 2.187\,ft$$

Walkway will be supported by a modified existing edge beam (cantilevering beyond) and monolithically connected to new traffic Barrier (Free - Pin - Pin for design at edge beam ~ Pin-Fix for design at barrier connection)

Thickness of Slab: $t := 4in$ $\quad w_{DC.slab} := t \times \gamma_c = 51.7 \times psf$

$w_{LL} := 75psf$

Dead Load of 54 inch Bicycle Railing: $w_{DC.Railing} := 36.6plf$

Free - Pin - Pin Reactions

$$R_{barrier.railing} := \frac{w_{DC.Railing} \times (canti + 1.5in)}{-(span + 4in)} = -20.4 \times plf$$

(assume 1.5in to centerline of railing & 4" from edge of barrier to vertical restraint doweled bar)

$$R_{barrier.slab} := \frac{w_{DC.slab} \times \left[\frac{(canti + 6in)^2}{2} - \frac{(span + 4in)^2}{2}\right]}{-(span + 4in)} = 62.2 \times plf$$

(assume 6in to free edge of slab)

$$R_{barrier.DC} := R_{barrier.railing} + R_{barrier.slab} = 41.8 \times plf$$

$$R_{edge.beam.DC} := w_{DC.Railing} + w_{DC.slab} \times [(canti + 6in) + (span + 4in)] - R_{barrier.DC} = 347.892 \times plf$$

Cantilever Negative Moment at Edge Beam

$$M6_{canti.neg} := 1.25 \times \left[w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2} \right] + 1.75 \times \left(w_{LL} \times \frac{canti^2}{2} \right) = 652.6 \times \frac{lb \times ft}{ft}$$

PIN-PIN Positive Bending

$$M6_{slab.pos} := \left(1.25 \times w_{DC.slab} + 1.75 \times w_{LL}\right) \times \frac{(span + 4in)^2}{8} - \frac{M6_{canti.neg}}{2} = 94.7 \times \frac{lb \times ft}{ft}$$

PIN - FIX Negative Bending (Adding negative moment contributors from cantilever)

$$M6_{slab.neg} := \left(1.25 \times w_{DC.slab} + 1.75 \times w_{LL}\right) \times \frac{(span + 4in)^2}{8} + M6_{canti.neg} = 1.1 \times 10^3 \times \frac{lb \times ft}{ft}$$

Service Moments

$$M6_{canti.serv.neg} := \left[w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2} \right] + \left(w_{LL} \times \frac{canti^2}{2} \right) = 450.3 \times \frac{lb \times ft}{ft}$$

$$M6_{slab.serv.neg} := \left(w_{DC.slab} + w_{LL}\right) \times \frac{(span + 4in)^2}{8} + M6_{canti.serv.neg} = 722.6 \times \frac{lb \times ft}{ft}$$

$$M6_{slab.serv.pos} := \left(w_{DC.slab} + w_{LL}\right) \times \frac{(span + 4in)^2}{8} - \frac{M6_{canti.serv.neg}}{2} = 47.1 \times \frac{lb \times ft}{ft}$$

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Design Slab Reinforcing for Maximum Negative Moment

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in - d_{no_{bar_{no}}} \times 0.5 = 1.75 \times in$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 6in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.2 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M6_{slab.neg} \times w_{mem} = 0.537\ ft \times kips$

Service Moment: $M_{service} := M6_{slab.serv.neg} \times w_{mem} = 0.361\ ft \times kips$

depth to rebar
$$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$
$d = 2 \times in$

Rectangular Compression Depth:
$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$
$a_{req} = 0.184 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.063 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 0.537 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 1.535 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = "SATISFACTORY"$ $\frac{\phi M_n}{M_{des}} = 286 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 6 \times in$

$w_{mem} = 0.5\ ft$ x $h_{mem} = 0.333\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 1.184 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = (A_s \times w_{mem}) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + (A_s \times w_{mem} \times n) \times x - (A_s \times w_{mem} \times n \times d) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{(A_s \times w_{mem} \times n)}{ft^2}, \frac{-(A_s \times w_{mem} \times n \times d)}{ft^3}\right] \times 12in\right] = 0.783 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.739 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 2.493 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 12.466 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.75 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times (h_{mem} - d_c)} = 2.111$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi) \ (in) - 2 \times d_c = 16.448 \times in}$$

$spacing \times (1 + bundles) = 6 \times in$

$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$V_u := \left[1.25 \times \left[w_{DC.Railing} + w_{DC.slab} \times (canti + 6in)\right] + 1.75 \times (w_{LL} \times canti)\right] \times w_{mem} = 0.253 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := \text{"yes"}$

$\beta_w := 2 \qquad \theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c \ (ksi)} \times w_{mem} \times (0.9 \times d)$

$\phi_v \times V_c = 1.229 \times kips$

$\text{Check}_{C.D}(\phi_v \times V_c, V_u) = \text{"SATISFACTORY"}$

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Design Slab Reinforcing for Maximum Positive Moment

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in - d_{no_{bar_{no}}} \times 0.5 = 1.75 \times in$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 6in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.2 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M6_{slab.pos} \times w_{mem} = 0.047\ ft \times kips$

Service Moment: $M_{service} := M6_{slab.serv.pos} \times w_{mem} = 0.024\ ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 2 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.016 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 5.28 \times 10^{-3} \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 0.047 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 1.535 \times kip\ ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = "SATISFACTORY"$

$\frac{\phi M_n}{M_{des}} = 3.2 \times 10^3 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^{\ 2}}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 6 \times in$

$w_{mem} = 0.5\ ft$ x $h_{mem} = 0.333\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 1.184 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf} \right) \right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2} \right) = \left(A_s \times w_{mem} \right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2} \right) \times x^2 + \left(A_s \times w_{mem} \times n \right) \times x - \left(A_s \times w_{mem} \times n \times d \right) = 0$$

$$x := \max \left[Quadratic_{a.b.c} \left[\frac{\left(w_{mem} \times \frac{1}{2} \right)}{ft}, \frac{\left(A_s \times w_{mem} \times n \right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d \right)}{ft^3} \right] \times 12in \right] = 0.783 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.739 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 0.163 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 0.813 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.75 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c \right)} = 2.111$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi) \ (in) - 2 \times d_c = 302.493 \times in}$$

$$spacing \times (1 + bundles) = 6 \times in$$

$Check_{C.D} \left[s_{max}, spacing \times (1 + bundles) \right] = "SATISFACTORY"$

Load to Edge Beam

Dead Load Reaction $R_{edge.beam.DC} = 347.892 \times plf$

Live Load Reaction $R_{edge.beam.LL} := \dfrac{w_{LL} \times 6ft \times \left(\dfrac{6ft}{2} + 4in\right)}{span + 4in} = 361.722 \times plf$

Check Existing Edge Beam for Capacity

Existing edge beam (W36x16.5x230#) will fully support walkway, design load to walkway considers the additional load from the new widening and the reduction of load from the component removals.

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Tension dowel requirement at Barrier

Dead Load reaction at barrier: $R_{barrier.DC} = 41.763 \times plf$

Partial Live Load reaction at barrier: $R_{barrier.LL} := \dfrac{-w_{LL} \times \dfrac{canti^2}{2}}{span + 4in} = -43.233 \times plf$

Partial Wind Uplift:

AASHTO specifies a superstructre wind uplift load of 20psf times the width of the bridge applied at the windward quarter point. To create an equivalent distributed force and resultant, a triangular distribution of wind load will be established across the windward 3/4 of the bridge width. The Highest pressure will occur under the proposed sidewalk.

$$p_{wind} := \frac{20psf \times W}{\frac{3}{4} \times W} \times 2 = 53.333 \times psf$$

$w_{DC.slab} = 51.667 \times psf$

53.3psf is nearly equivalent to the 51.7psf slab dead load. The slab design is acceptable for wind uplift loads

Conservatively apply maximum wind to area tributary to barrier reaction.

$$R_{barrier.WL} := -p_{wind} \times \frac{span}{2} = -101.693 \times plf$$

Uplift at Barrier:

$$Uplift_I := -\left(1.75 \times R_{barrier.LL} + 0.9 \times R_{barrier.DC}\right) = 38.071 \times plf$$

$$Uplift_{III} := -\left(1.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = 104.783 \times plf$$

$$Uplift_V := -\left(1.35 \times R_{barrier.LL} + 0.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = 61.455 \times plf$$

$$\max\left(Uplift_I, Uplift_{III}, Uplift_V\right) = 104.783 \times plf$$

$$Tension := \frac{0.9 \times A_{no.4} \times 60ksi}{4ft} = 2.7 \times 10^3 \times plf$$

Uplift force is minor. Install #4 bars vertically into slab at 4ft on center.

WSDOT BDM 5.5.4 requires 8 inches of epoxy embedment for full development.

Thickness of the deck is 7.5 inches
Thickness at the curb is 10 inches (REF SHT 36)

Full Development is not required. Provide 6" embed $\dfrac{6in}{8in} \times Tension = 2.025 \times 10^3 \times plf$

$$Check_{C.D}\left(\frac{6in}{8in} \times Tension, \max\left(Uplift_I, Uplift_{III}, Uplift_V\right)\right) = \text{"SATISFACTORY"}$$

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Additional dead load to the structure from walkway widening.

Additional Dead Load, Slab and Railing: $plf_{slab.railing} := R_{barrier.DC} + R_{edge.beam.DC} = 389.7 \times plf$

Dead Load of New Traffic Barrier + BP rail: $plf_{barrier} := \dfrac{\left(8in + 32in \times \dfrac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$

Total weight of new work: $plf_{new} := plf_{slab.railing} + plf_{barrier} = 776.9 \times plf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedistal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 58 & 37): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedistal weight: $plf_{pedistal} := \dfrac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{20ft} = 35.9 \times plf$

Dead Load of Existing intermediate Conc Rail: $plf_{conc.rail} := \dfrac{(1ft + 10in) \times 6in \times [20ft - (1ft + 4in) - 0.5in] \times \gamma_{c.ex}}{20ft} = 128 \times plf$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{20ft} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{20ft}\right]$$

$plf_{metal.rail} = 16.4 \times plf$

Dead Load of Existing Edge Beam: $plf_{ex.edge.beam} := 6in \times 4in \times \gamma_{c.ex} = 25.0 \times plf$

Dead Load of Existing sidewalk:
(REF SHT 37) $plf_{ex.sidewalk} := 3.5in \times 4ft \times \gamma_{c.ex} = 175 \times plf$

Dead Load of Existing Curb:
(REF SHT 38) $plf_{curb} := \left[18in \times \left(7in + \dfrac{3in}{2}\right) - 3in \times 2in\right] \times \gamma_{c.ex} = 153.125 \times plf$

Total weight of removals:

$$plf_{rem} := plf_{pedistal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{ex.sidewalk} + plf_{curb} = 533.5 \times plf$$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 243.4 \times plf$

Weight of Segment 6 $Segment6_{weight} := 645kips$

$Segment6_{length} := 84.91ft$

Length of Segment 6:

Average Per foot weight of Segment 6: $\dfrac{Segment6_{weight}}{Segment6_{length}} = 7.596 \times 10^3 \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment6_{weight}}{Segment6_{length}}} = 3.204 \times \%$ Less than 10% OKAY

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: (REF SHT E-6/91/4 : 782-95) $H_{mast} := 30ft + (3ft + 9.5in) = 33.792\,ft$

Length of Luminaire Mast arm: (12' max REF WSDOT STD J-28.10-01) $L_{arm} := 12ft$

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d \qquad \text{(psf)}$$

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$$H_{lum} := H_{grade.sep} + H_{mast} = 63.792\,ft$$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\qquad \alpha := 9.5$

$$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$$

$I_r := 1.00$ — Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ — Gust Effect Factor (3.8.5)

$C_d := 0.5$ — Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.608 \times psf$$

Effective Projected Area of Luminaire Head (REF WSDOT BDM 10.1(B)) $A := 3.3ft^2$

$$V_{wind.lum} := P_z \times A = 44.907\,lb$$

Height to point of connection: $H := H_{mast} = 33.792\,ft$

$$M_{wind.lum} := V_{wind.lum} \times H = 1.517\,ft \times kips$$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

Wind on Mast Arm

$$H_{arm} := H_{grade.sep} + H_{mast} = 63.792\ \text{ft}$$

$$K_z := K_{z.eq}\left(H_{arm}, 900\text{ft}, 9.5\right) = 1.151$$

$$C_{d.cylinder}(V, d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \dfrac{V}{mph} \times \dfrac{d}{ft} \\ CD \leftarrow \dfrac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{if} \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{if} \ \omega \ge 78 \\ CD \end{array}$$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30	
Traffic Signals (see note 2)	1.2	
Luminaires (with generally rounded surfaces)	0.5	
Luminaires (with rectangular flat side shapes)	1.2	
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)	

Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r - \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

Cylinders (3in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 3\text{in}) = 1.1$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.938 \times psf$$

Effective Projected Area of mast arm $A := 3\text{in} \times L_{arm} = 3\ \text{ft}^2$

$$V_{wind.arm} := P_z \times A = 89.814\ \text{lb}$$

Height to point of connection: $H := H_{mast} + 4\text{ft} = 37.792\ \text{ft}$

$$M_{wind.arm} := V_{wind.arm} \times H = 3.394\ \text{ft} \times \text{kips}$$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.896\ \text{ft}$$

$$K_z := K_{z.eq}\left(H_{pole}, 900\text{ft}, 9.5\right) = 1.079$$

Cylinders (8in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 8\text{in}) = 0.553$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.103 \times psf$$

Effective Projected Area of mast arm $A := 8\text{in} \times H_{mast} = 22.5\ \text{ft}^2$

$$V_{wind.pole} := P_z \times A = 317.715\ \text{lb}$$

Height to point of connection: $H := \frac{H_{mast}}{2} + 4\text{ft} = 20.896\ \text{ft}$

$$M_{wind.pole} := V_{wind.pole} \times H = 6.639\ \text{ft} \times \text{kips}$$

$$V_{wind} := V_{wind.lum} + V_{wind.arm} + V_{wind.pole} = 0.452 \times \text{kips}$$

Total Moment from wind (omnidirectional):

$$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 11.551\ \text{ft} \times \text{kips}$$

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Dead Loads

Distance from Pole location to Point of connection: $Offset := canti + 1ft = 3.187\,ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$$W_{lum} := 60lb$$

$$M_{DC.lum} := W_{lum} \times \left(L_{arm} - Offset\right) = 0.529\,ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$$

$$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.129\,ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 345.632\,lb$$

$$M_{DC.pole} := W_{pole} \times (-Offset) = -1.101\,ft \times kips$$

Total Moment from Dead Load:

$$M_{DC} := M_{DC.lum} + M_{DC.arm} + M_{DC.pole} = -0.443\,ft \times kips$$

$$W_{lum} + W_{arm} + W_{pole} = 0.452 \times kips$$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$$

$$M_{Ice.lum} := Ice_{lum} \times \left(L_{arm} - Offset\right) = 0.524\,ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$$

$$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.08\,ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$Ice_{pole} := \pi \times 8in \times 3psf \times H_{mast} = 212.319\,lb$$

$$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -0.677\,ft \times kips$$

Total Moment from Dead Load:

$$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -0.073\,ft \times kips$$

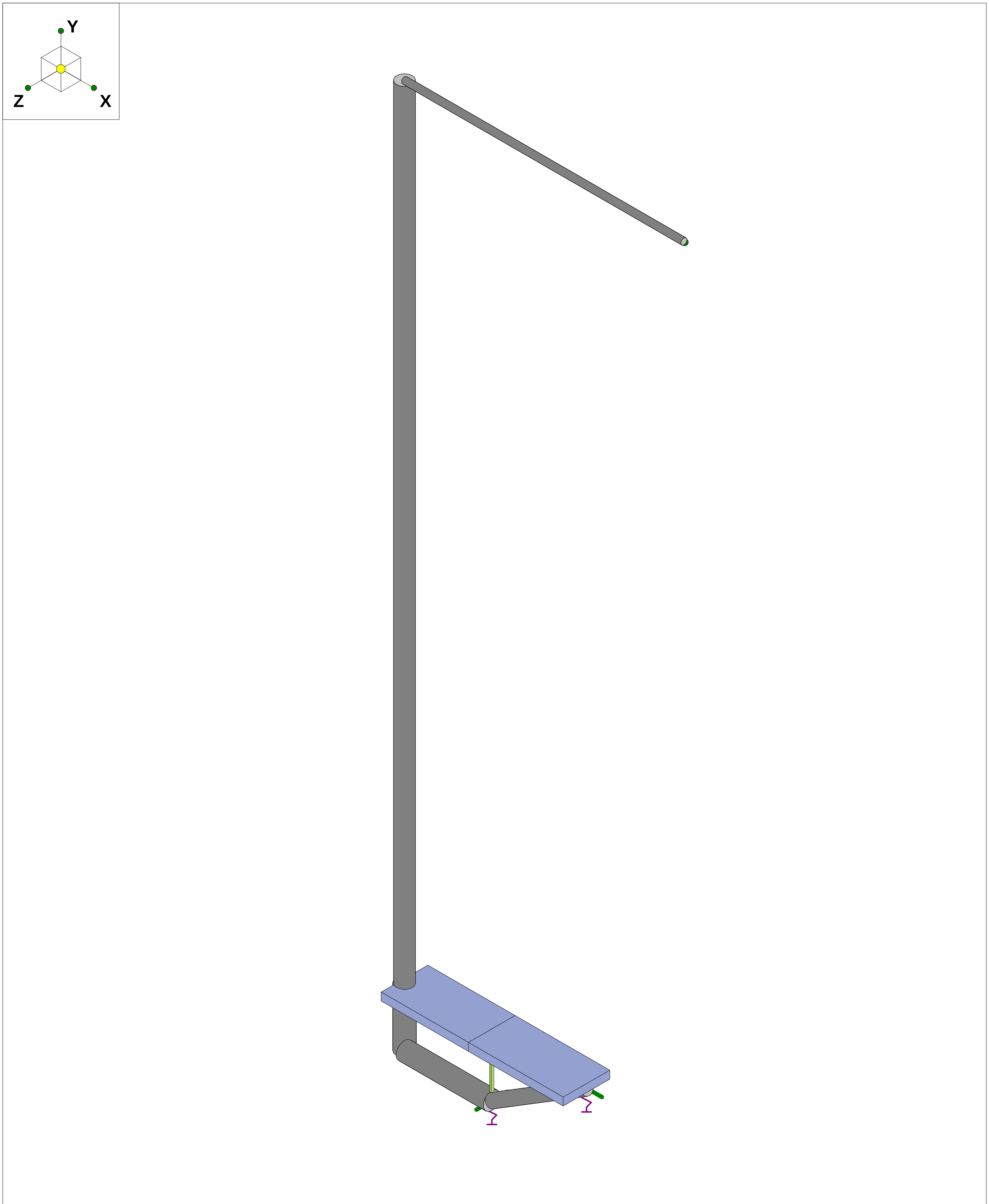
Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left(\left|M_{DC}\right| + M_{wind}\right) \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.443 \\ 9.018 \\ 4.731 \end{pmatrix} ft \times kips$$

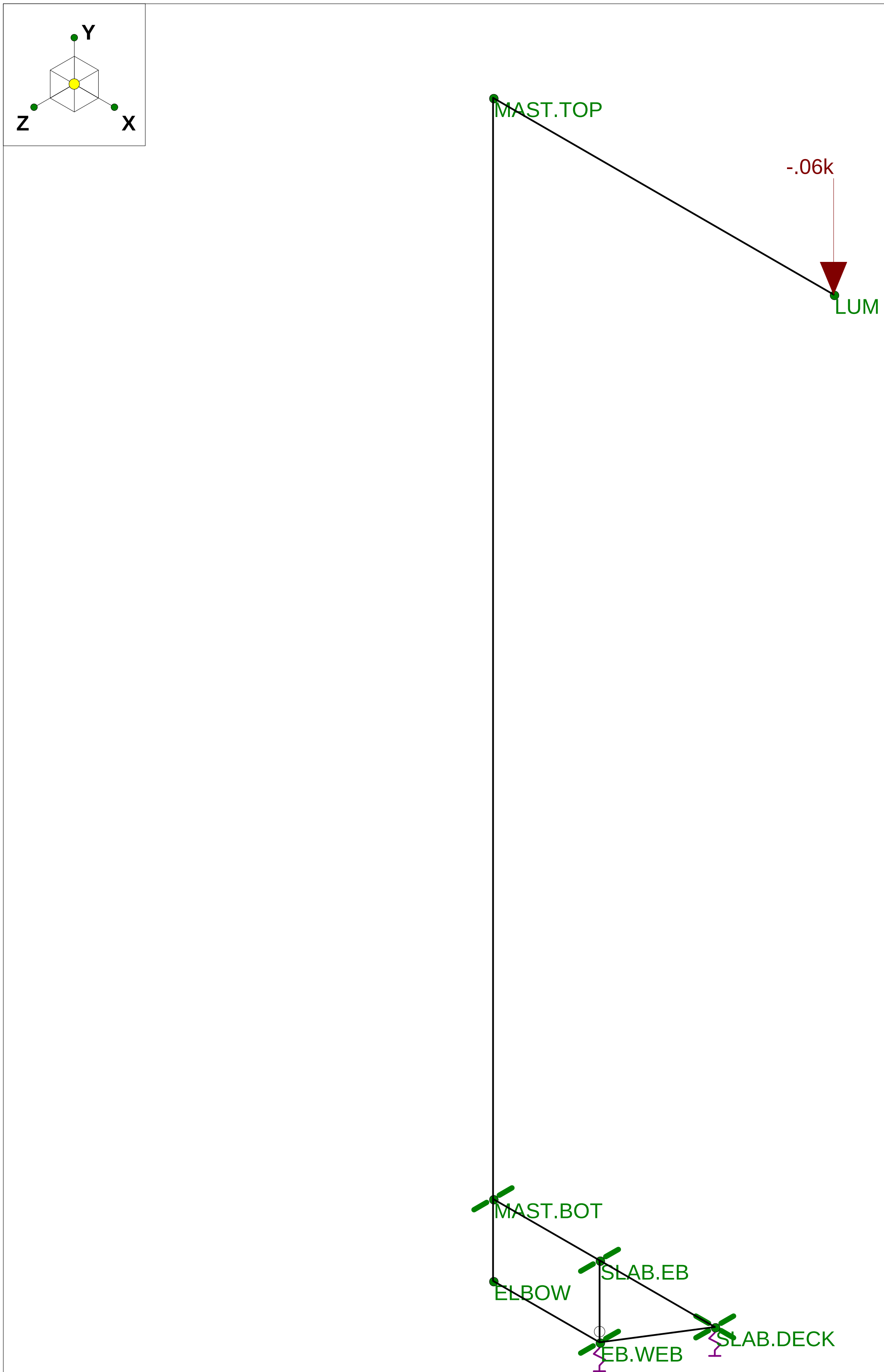
Governing Load $M_{design} := \max\left(M_{design'}\right) = 9.018\, ft \times kips$

Pole will be connected through the top slab into a square tube which will act as a framing member to convert the applied moment from the luminaire pole to component loads which will be carried by the Slab and Tube Below.

This framing system is sufficiently complex as to justify use of an external frame analysis software. See the following pages for general geometry and analysis output.



		SK - 1
		Jan 17, 2013 at 10:48 AM
	Section 6 Luminaire	Section 6 Luminaire.r3d



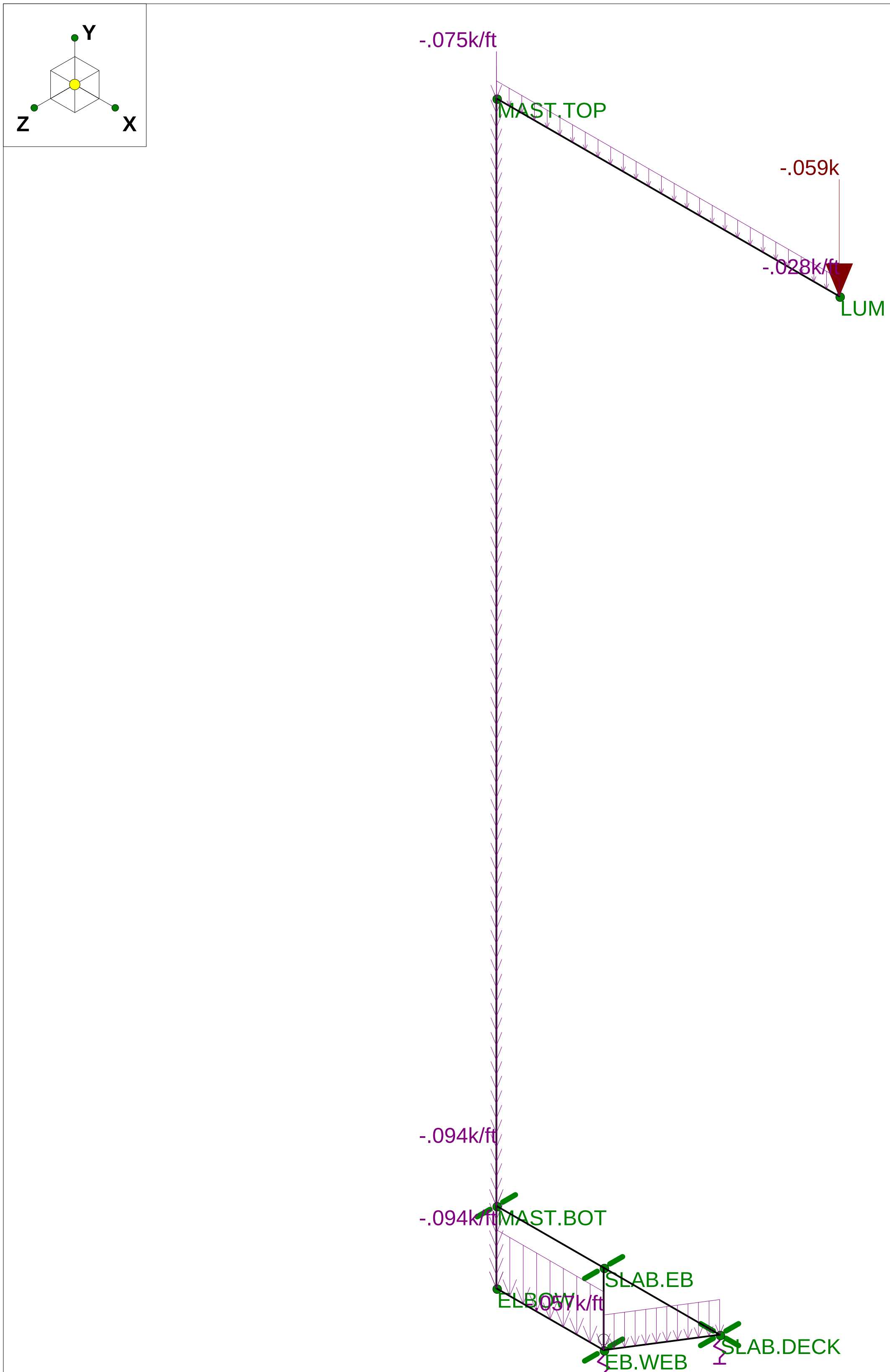
Loads: BLC 1, Dead load

Section 6 Luminaire

SK - 2

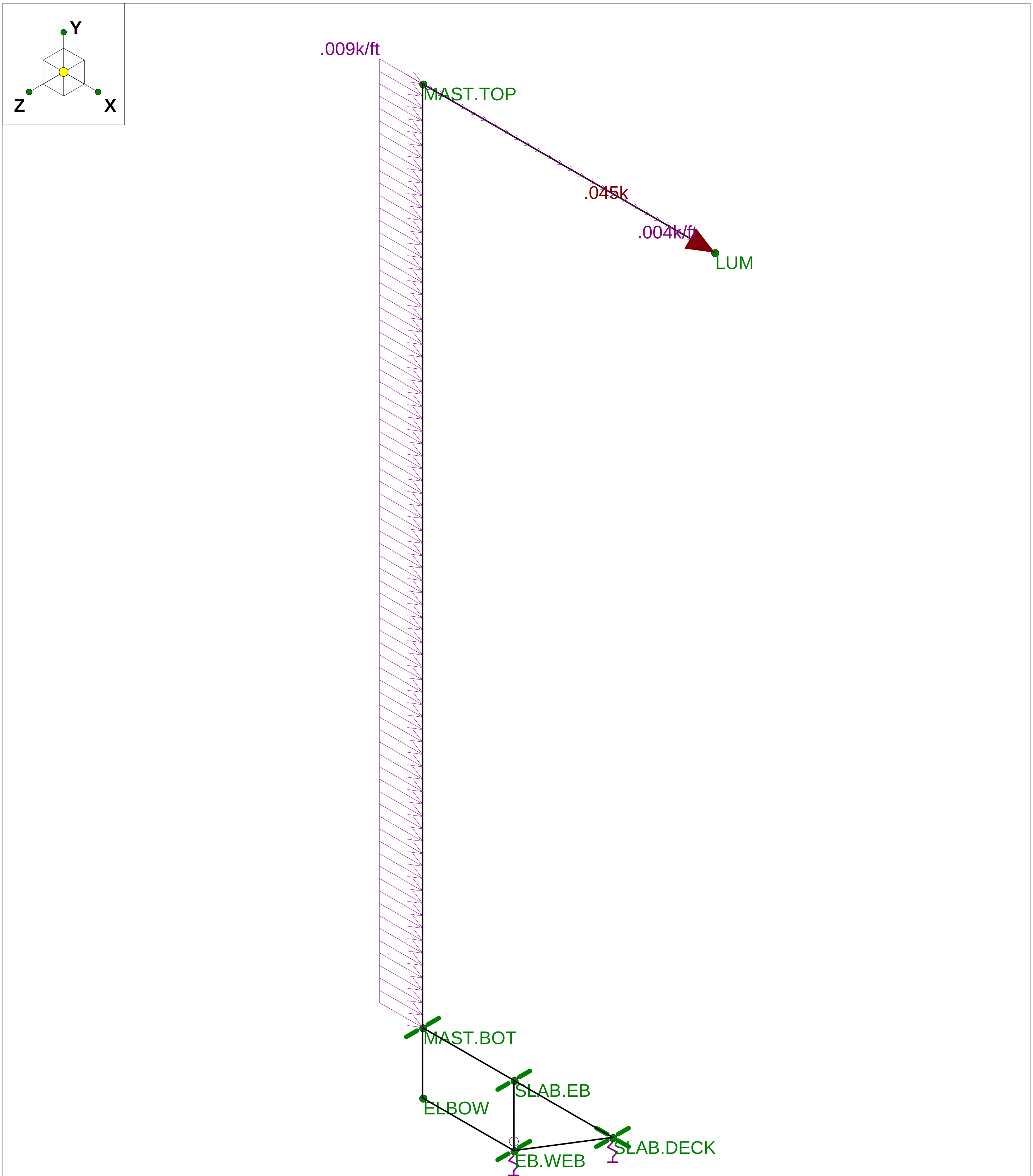
Jan 17, 2013 at 10:48 AM

Section 6 Luminaire.r3d



Loads: BLC 2, Ice Load

		SK - 3
		Jan 17, 2013 at 10:49 AM
	Section 6 Luminaire	Section 6 Luminaire.r3d



Loads: BLC 3, Wind

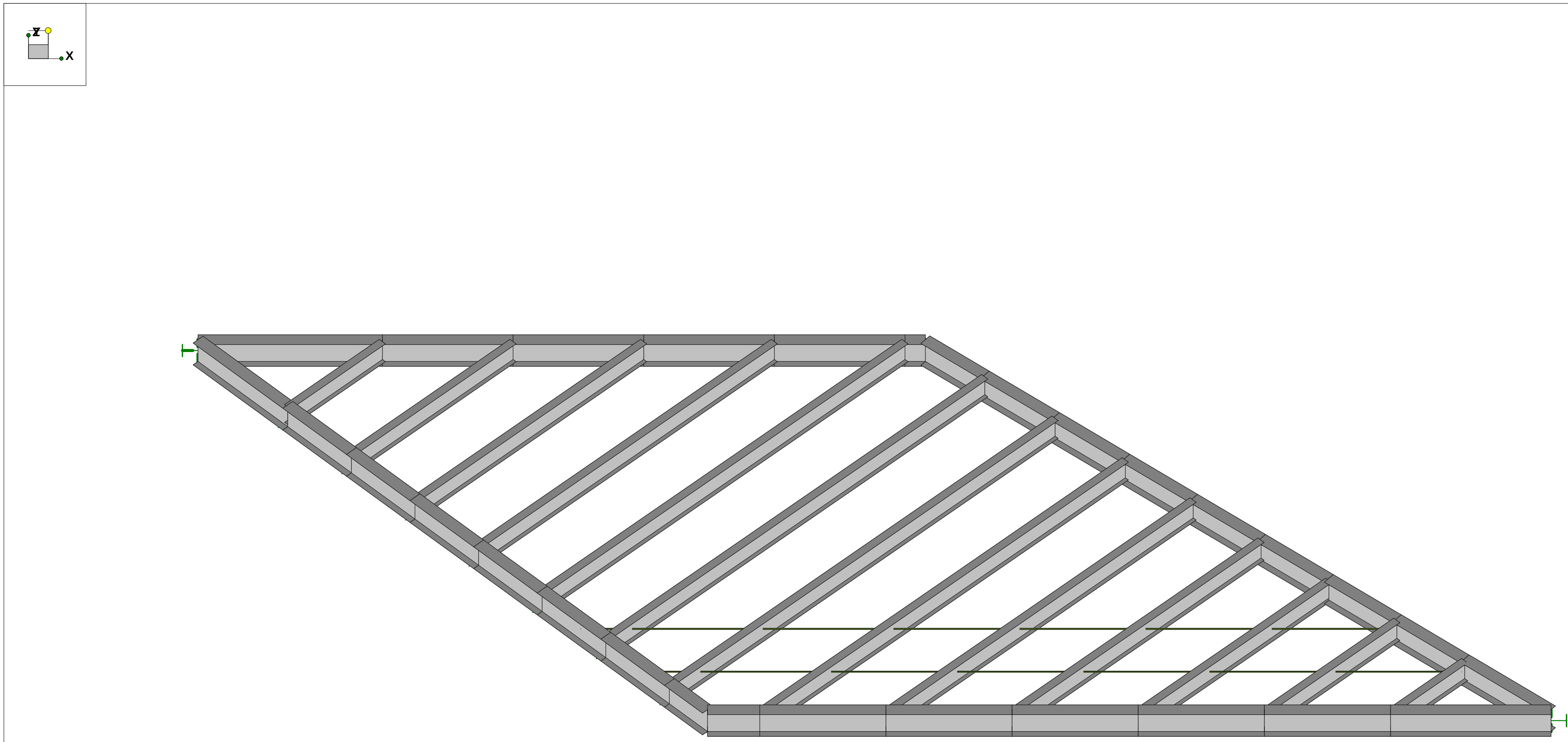
	Section 6 Luminaire	SK - 4
		Jan 17, 2013 at 10:49 AM
		Section 6 Luminaire.r3d

Load Combinations

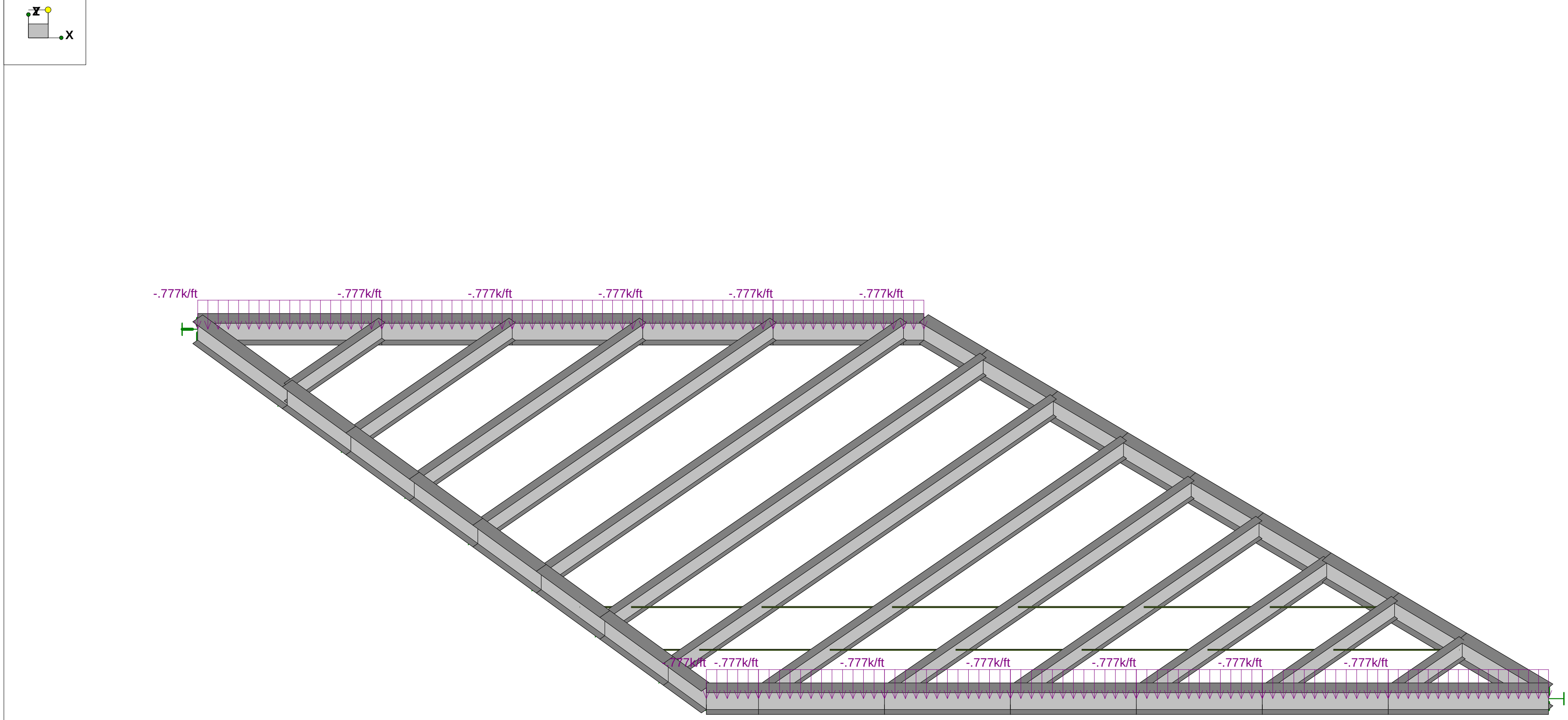
	Description	So...	PDelta	SRSS	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...	BLC	Fac...
1	STR III		Y		DL	1.25	WL	1.4												
2	EX II		Y		DL	1.25	SL	1												
3																				
4	COMBINED EX+	Yes	Y		DL	1.25	WL	1.4	SL	1										
5	COMBINED EX-	Yes	Y		DL	1.25	WL	-1.4	SL	1										

Envelope AISC 14th(360-10): ASD Steel Code Checks

	Member	Shape	Code Check	Loc...	LC	Shea...	Loc...	...	...	Pnc/om [k]	Pnt/om [k]	Mnyy/...	Mnzz/...	...	Eqn
1	M5	HSS8.625x0.25	.472	0	4	.171	0		5	153.453	154.419	34.371	34.371	...	H1-1b
2	M6	HSS8.625x0.25	.271	0	5	.072	3.75		5	152.255	154.419	34.371	34.371	...	H1-1b
3	M7	HSS6x0.188	.327	0	5	.038	0		5	76.273	79.976	12.386	12.386	...	H1-1b

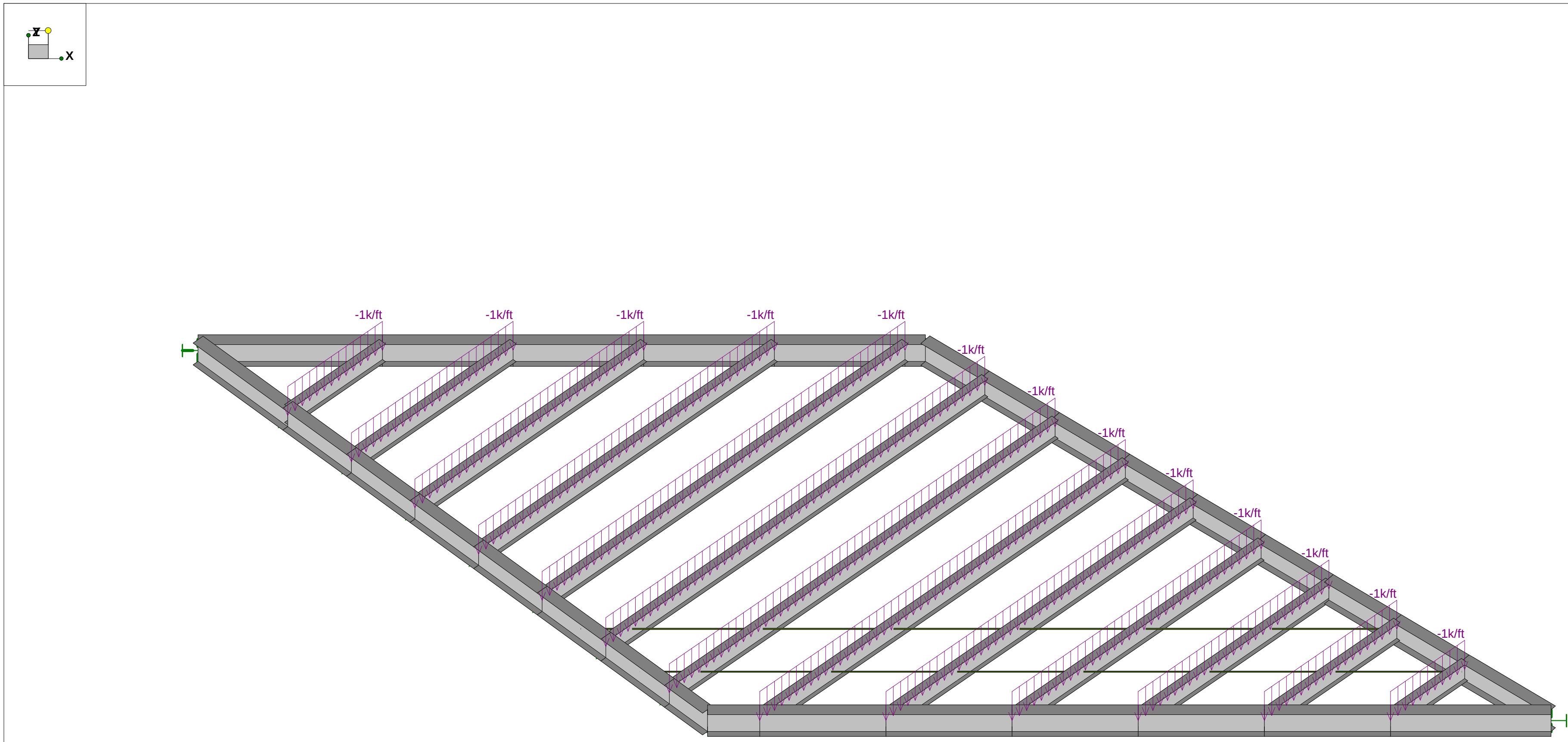


		SK - 1
		Jan 17, 2013 at 10:54 AM
	Section 6 Overall	Section 6 - 6 ft.r3d



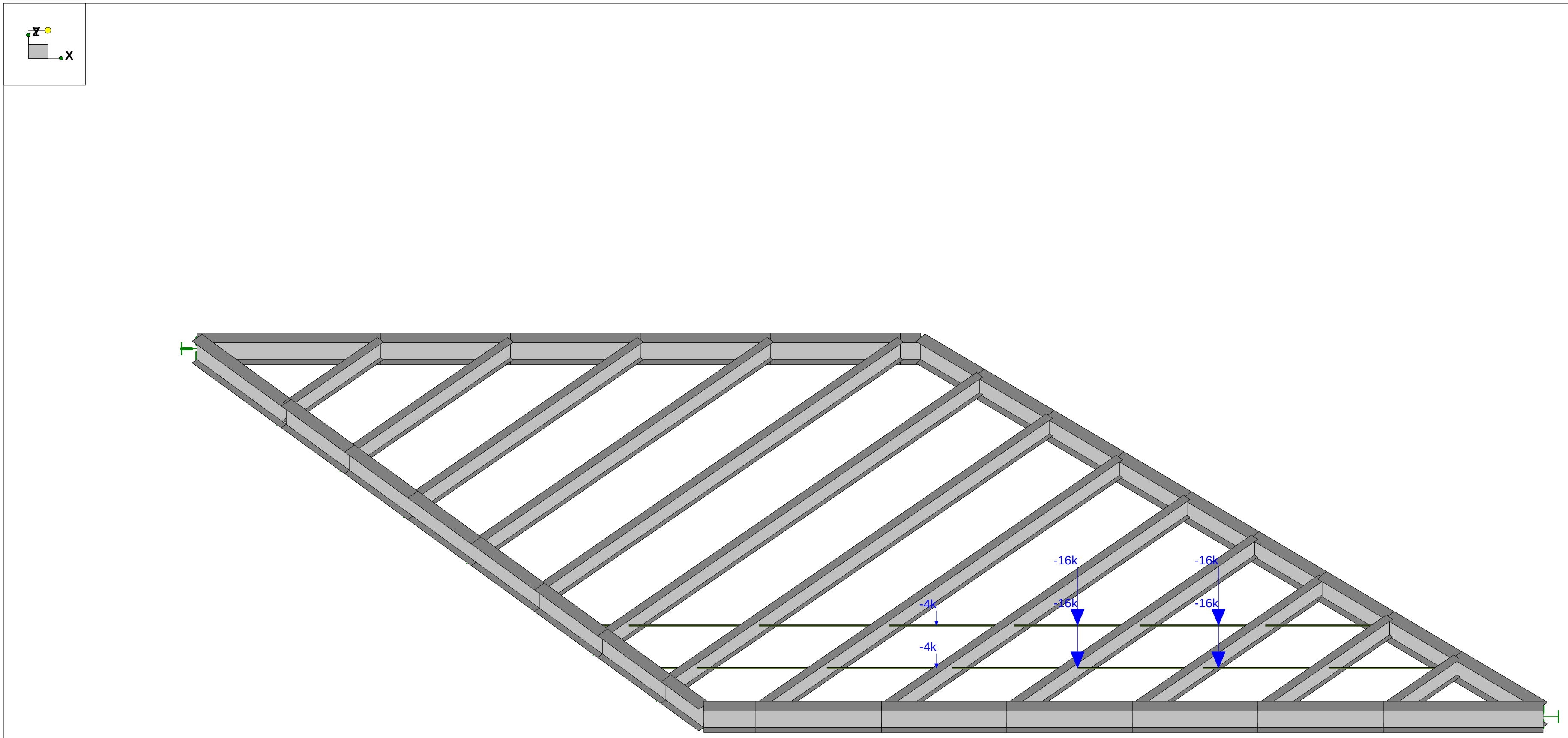
Loads: BLC 2, DC 777 plf

		SK - 2
		Jan 17, 2013 at 11:09 AM
	Section 6 Overall - DC = Total Weight of New Work	Section 6 - 6 ft.r3d



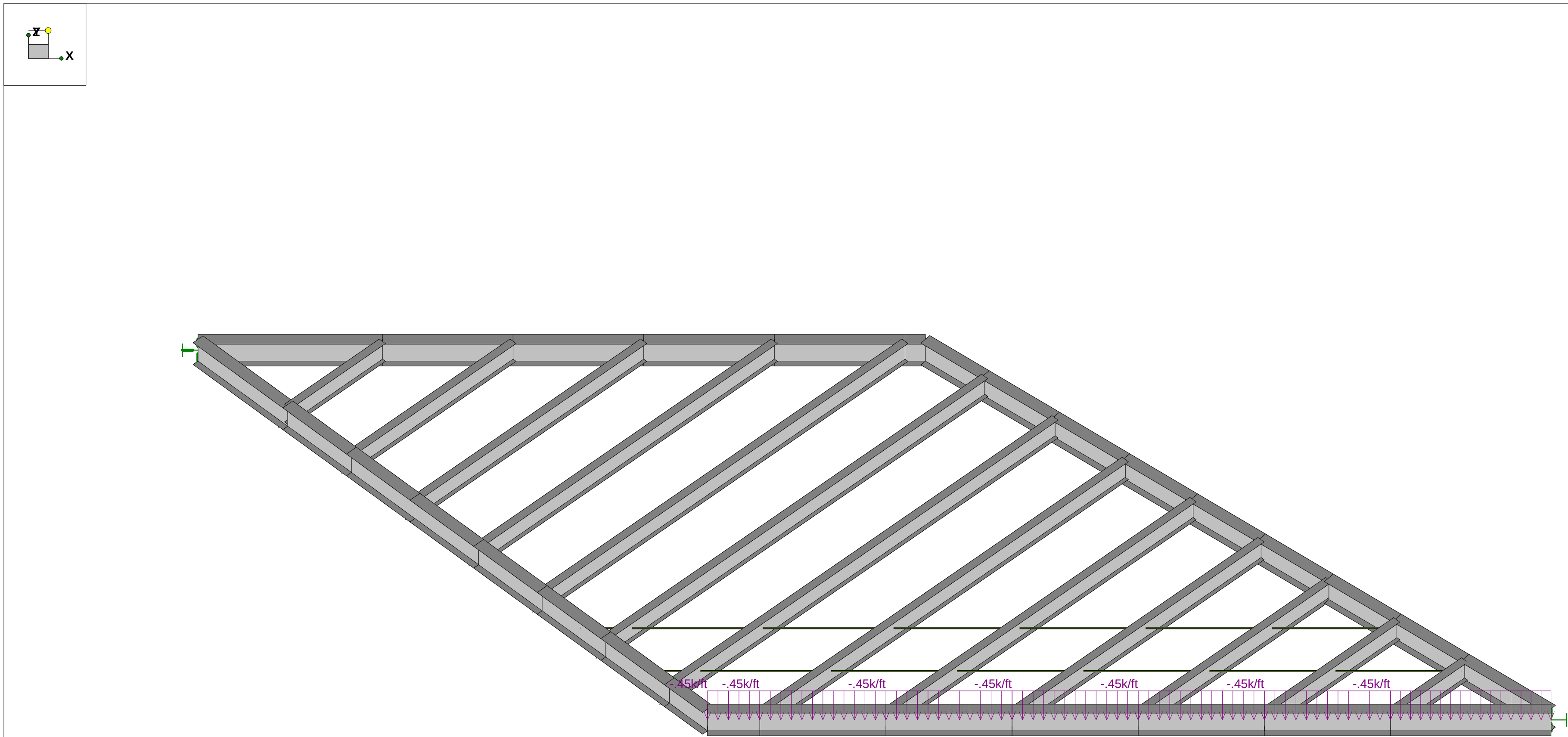
Loads: BLC 3, Slab at 165pcf

		SK - 3
		Jan 17, 2013 at 11:12 AM
	Section 6 - 165pcf x 7.5in x 9ft (round up to 1klf)	Section 6 - 6 ft.r3d



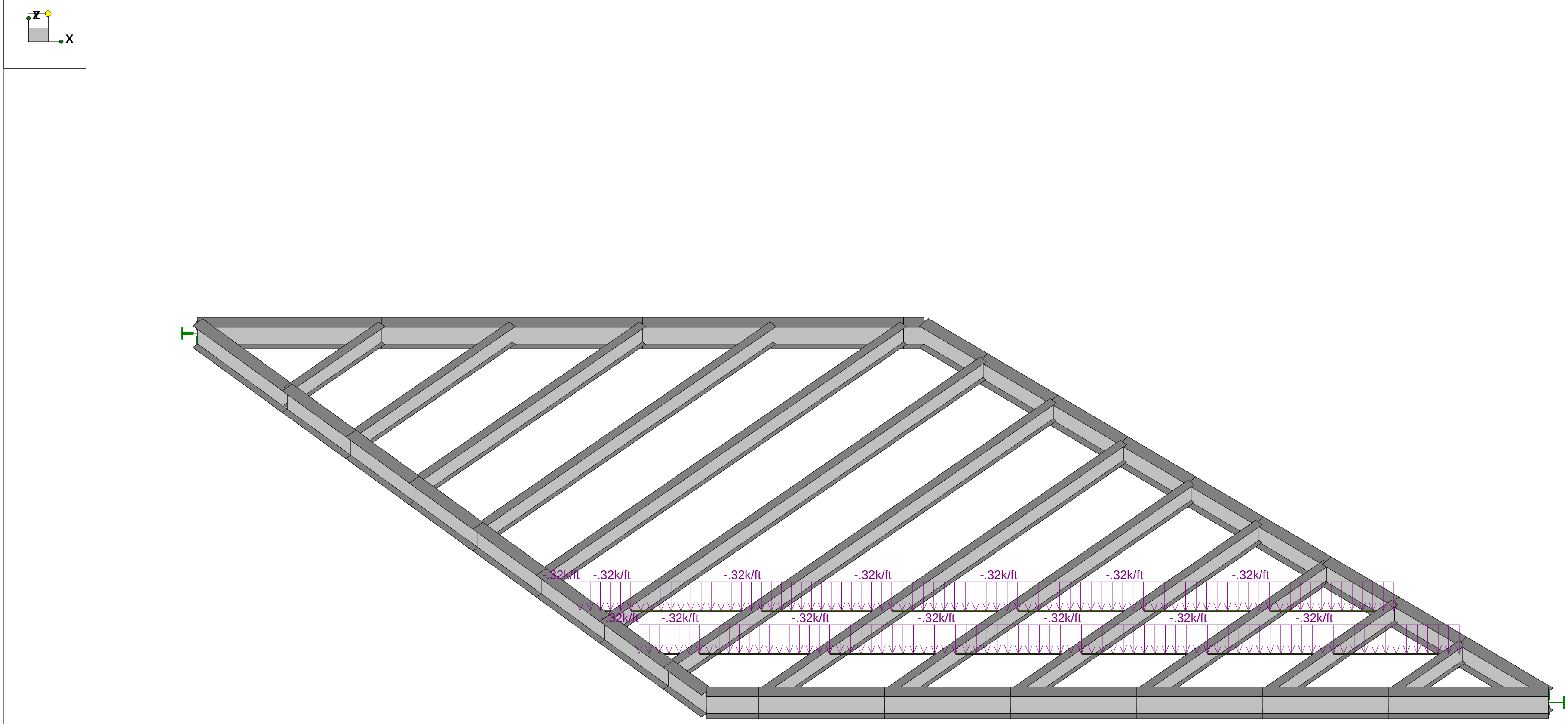
Loads: BLC 4, HS20

		SK - 4
		Jan 17, 2013 at 11:13 AM
	Section 6 HS20 - 1ft from barrier	Section 6 - 6 ft.r3d



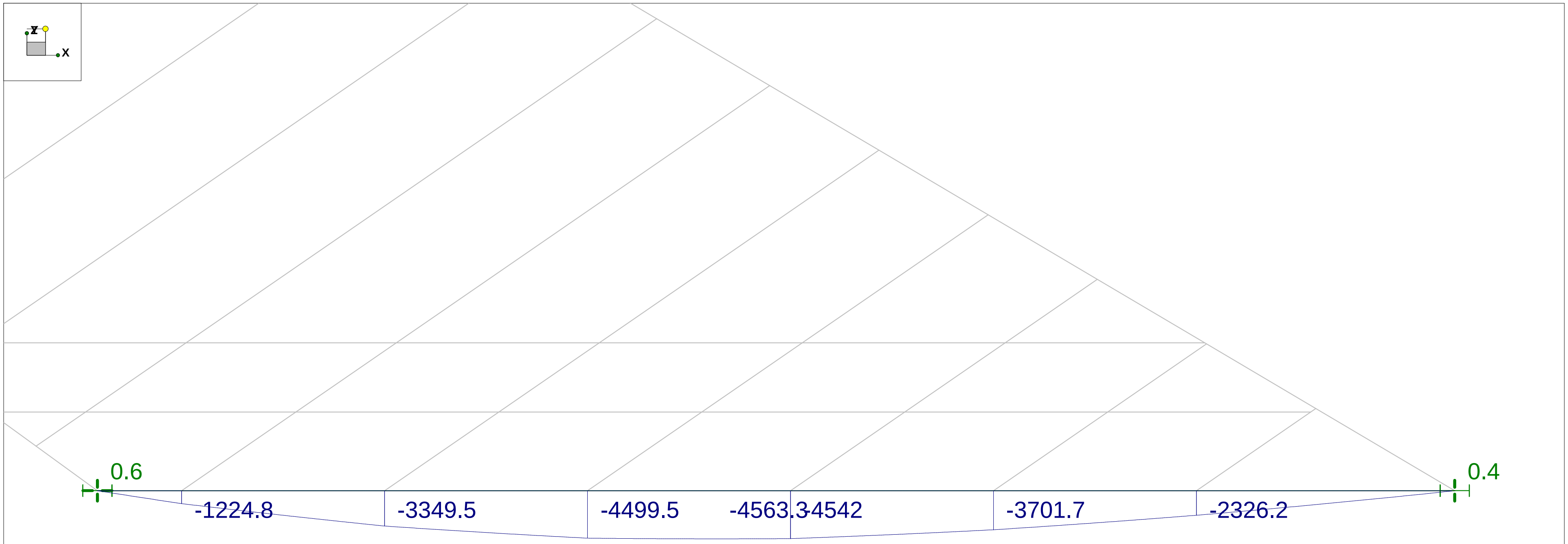
Loads: BLC 5, PED 75psf x 6ft

		SK - 5
		Jan 17, 2013 at 11:13 AM
	Section 6 - Ped load - 75psf x 6ft = 450plf	Section 6 - 6 ft.r3d



Loads: BLC 6, LANE

		SK - 6
		Jan 17, 2013 at 11:14 AM
	Section 6 HL93 Lane Load	Section 6 - 6 ft.r3d



Solution: Envelope
Member z Bending Moments (k-ft)

		SK - 7
		Jan 17, 2013 at 11:20 AM
	Section 6 - Design Moment	Section 6 - 6 ft.r3d

Moment Capacity at strength limit state in Major Axis for Discretely braced flanges in Compression - AASHTO LRFD 6.10.8.1.1

$\varphi_f := 1.00$ for flexure $E := 29000\text{ksi}$

W36 x 230 Member information (Including Flange cover plates)

Top Flange: $b_{tf} := 16.5\text{in}$ $t_{tf} := 1.26\text{in} + \frac{7}{8}\text{in} + \frac{9}{16}\text{in}$ $A_{tf} := b_{tf} \times t_{tf} = 44.509 \times \text{in}^2$ $I_{tf} := \frac{b_{tf} \times t_{tf}^3}{12} = 26.989 \times \text{in}^4$

Web: $h_{web} := 26.5\text{in}$ $t_{web} := 0.760\text{in}$ $A_{web} := h_{web} \times t_{web} = 20.14 \times \text{in}^2$ $I_{web} := \frac{t_{web} \times h_{web}^3}{12} = 1.179 \times 10^3 \times$

Bottom Flange: $b_{bf} := 16.5\text{in}$ $t_{bf} := t_{tf}$ $A_{bf} := b_{bf} \times t_{bf} = 44.509 \times \text{in}^2$ $I_{bf} := \frac{b_{bf} \times t_{bf}^3}{12} = 26.989 \times \text{in}^4$

$$CG := \frac{A_{tf} \times \frac{t_{tf}}{2} + A_{web} \times \left(t_{tf} + \frac{h_{web}}{2}\right) + A_{bf} \times \left(t_{tf} + h_{web} + \frac{t_{bf}}{2}\right)}{A_{tf} + A_{web} + A_{bf}} = 15.947 \times \text{in}$$

$$I_{total} := I_{tf} + A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right)^2 + I_{web} + A_{web} \times \left(CG - t_{tf} - \frac{h_{web}}{2}\right)^2 + I_{bf} + A_{bf} \times \left(CG - t_{tf} - h_{web} - \frac{t_{bf}}{2}\right)^2 = 2.02 \times 10^4 \times \text{in}^4$$

$$S_x := \frac{I_{total}}{CG} = 1.267 \times 10^3 \times \text{in}^3 \qquad Z_x := 2 \times \left[A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right) + t_{web} \times (CG - t_{tf}) \times \frac{(CG - t_{tf})}{2}\right] = 1.433 \times 10^3 \times \text{in}^3$$

$M_u := 4563.3\text{kip} \times \text{ft}$ From RISA-3D structural Analysis $F_y := 45\text{ksi}$ $L_b := 12\text{ft} + 5.5\text{in}$

At the strength limit state, the following requirement shall be satisfied: $f_{bu} + \frac{1}{3} \times f_l \le \varphi_f \times F_{nc}$

The local buckling resistance (6.10.8.2.2) of the compression flange shall be taken as

if $\lambda_f \le \lambda_{pf}$ then $F_{nc} = R_b \times R_h \times F_{yc}$ $\lambda_f := \frac{b_{bf}}{2 \times t_{bf}} = 3.1$ $\lambda_{pf} := 0.38 \times \sqrt{\frac{E}{F_y}} = 9.647$

$\text{less}(\lambda_f, \lambda_{pf}) = \text{"Less than or Equal to"}$

Web load-shedding factor determined as specified in Article 6.10.1.10.2 $R_b = 1.0$ if $\frac{2 \times D_c}{t_w} \le \lambda_{rw}$

Depth of web in compression in elastic range $D_c := h_{web} - (CG - t_{tf}) + t_{bf} = 15.947 \times \text{in}$

Limiting slenderness ratio for a noncompact web:

$\frac{2 \times D_c}{t_{web}} = 41.967$ $\lambda_{rw} := 5.7 \times \sqrt{\frac{E}{F_y}} = 144.7$ $\text{less}\left(\frac{2 \times D_c}{t_{web}}, \lambda_{rw}\right) = \text{"Less than or Equal to"}$ $R_b := 1.0$

Hybrid Factor: $R_h = \frac{12 + \beta \times (3 \times \rho - \rho^3)}{12 + 2 \times \beta}$ $D_n := D_c = 15.947 \times \text{in}$ For doubly symmetric sections

$A_{fn} := b_{bf} \times t_{bf} = 44.509 \times \text{in}^2$ $\beta := \frac{2 \times D_n \times t_{web}}{A_{fn}} = 0.545$

$\rho := 1.0$ Non Composite with no cover plates on tensile side

$R_h := \frac{12 + \beta \times (3 \times \rho - \rho^3)}{12 + 2 \times \beta} = 1$ $F_{nc.lb} := R_b \times R_h \times F_y = 45 \times \text{ksi}$

The Lateral Torsional Buckling resistance (6.10.8.2.3) of the compression flange shall be taken as

$L_p = 1.0 \times r_t \times \sqrt{\frac{E}{F_{yc}}}$ Limit of Plastic deformation

$b_{fc} := b_{bf} = 16.5 \times in$ $t_{fc} := t_{bf} = 2.697 \times in$

$r_t := \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3} \times \frac{D_c \times t_{web}}{b_{fc} \times t_{fc}}\right)}}$ $r_t = 0.38\ ft$

$\boxed{L_p := 1.0 \times r_t \times \sqrt{\frac{E}{F_y}} = 9.648\ ft}$

$F_{yr} := 0.7 \times F_y = 31.5 \times ksi$

$\boxed{L_r := \pi \times r_t \times \sqrt{\frac{E}{F_{yr}}} = 36.228\ ft}$

$C_b := 1$ Unbraced Cantilevers or simple spans

$\boxed{F_{nc.ltb} := C_b \times \left[1 - \left(1 - \frac{F_{yr}}{R_h \times F_y}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \times F_y = 43.6 \times ksi}$

$\boxed{F_{nc} := \min(F_{nc.lb}, F_{nc.ltb}) = 43.6 \times ksi}$

The Maximum Allowable Unbraced Length (6.10.1.6-3) is: $L_b \le 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{M_{yc}}{M_u}}$

$L_{max} := 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{F_y \times S_x}{M_u}} = 11.813\ ft$

$\boxed{Check_{C.D}(L_{max}, L_b) = \text{"Not Satisfactory"}}$ Acceptable

No Lateral bending is anticipated at this location $\boxed{f_l := 0}$

$f_{bu} \le \varphi_f \times F_{nc}$

$\boxed{S_x \times (\varphi_f \times F_{nc}) = 4600 \times kip \times ft}$

$M_u = 4.563 \times 10^3\ ft \times kips$

$\boxed{Check_{C.D}\left[S_x \times (\varphi_f \times F_{nc}), M_u\right] = \text{"SATISFACTORY"}}$

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BALLARD - 6FT - SEGMENT 7

$\gamma_c := 155pcf$

New construction

Existing overhang = 5ft 3in from inside curb to outside of existing concrete barrier (REF SHT 12)

Width of new Traffic Barrier: $width_{barrier} := (32in + 6in) \times \frac{4}{21} + 8in = 15.238 \times in$

Distance from back of new Traffic Barrier to centerline of existing edge beam: $span := (5ft + 3in) - width_{barrier} - \frac{6in}{2} = 3.73\ ft$

Distance from centerline of existing edge beam to 6ft walkway limit: $canti := 6ft - span = 2.27\ ft$

Walkway will be supported by a modified existing edge beam (cantilevering beyond) and monolithically connected to new traffic Barrier (Free - Pin - Pin for design at edge beam ~ Pin-Fix for design at barrier connection)

Thickness of Slab: $t := 4in$ $\quad w_{DC.slab} := t \times \gamma_c = 51.7 \times psf$

$w_{LL} := 75psf$

Dead Load of 54 inch Bicycle Railing: $w_{DC.Railing} := 36.6plf$

Free - Pin - Pin Reactions at Traffic Barrier

$$R_{barrier.railing} := \frac{w_{DC.Railing} \times (canti + 1.5in)}{-(span + 4in)} = -21.6 \times plf$$

(assume 1.5in to centerline of railing & 4" from edge of barrier to vertical restraint doweled bar)

$$R_{barrier.slab} := \frac{w_{DC.slab} \times \left[\frac{(canti + 6in)^2}{2} - \frac{(span + 4in)^2}{2}\right]}{-(span + 4in)} = 56.2 \times plf$$

(assume 6in to free edge of slab to support bicycle railing)

$$R_{barrier.DC} := R_{barrier.railing} + R_{barrier.slab} = 34.6 \times plf$$

$$R_{edge.beam.DC} := w_{DC.Railing} + w_{DC.slab} \times [(canti + 6in) + (span + 4in)] - R_{barrier.DC} = 355.027 \times plf$$

Cantilever Negative Moment at Edge Beam

$$M7_{canti.neg} := 1.25 \times \left[w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2}\right] + 1.75 \times \left(w_{LL} \times \frac{canti^2}{2}\right) = 695.4 \times \frac{lb \times ft}{ft}$$

Free - Fix Positive Bending (Neglecting negative moment contributors from cantilever)

$$M7_{slab.pos} := \left(1.25 \times w_{DC.slab} + 1.75 \times w_{LL}\right) \times \frac{9}{128} \times (span + 4in)^2 = 227.4 \times \frac{lb \times ft}{ft}$$

Free - Fix Negative Bending (Adding negative moment contributors from cantilever)

$$M7_{slab.neg} := \left(1.25 \times w_{DC.slab} + 1.75 \times w_{LL}\right) \times \frac{(span + 4in)^2}{8} + M7_{canti.neg} = 1.1 \times 10^3 \times \frac{lb \times ft}{ft}$$

Service Moments

$$M7_{canti.serv.neg} := \left[w_{DC.Railing} \times (canti + 1.5in) + w_{DC.slab} \times \frac{(canti + 6in)^2}{2}\right] + \left(w_{LL} \times \frac{canti^2}{2}\right) = 479.1 \times \frac{lb \times ft}{ft}$$

$$M7_{slab.serv.neg} := \left(w_{DC.slab} + w_{LL}\right) \times \frac{(span + 4in)^2}{8} + M7_{canti.serv.neg} = 740.5 \times \frac{lb \times ft}{ft}$$

$$M7_{slab.serv.pos} := \left(w_{DC.slab} + w_{LL}\right) \times \frac{9}{128} \times (span + 4in)^2 = 147.1 \times \frac{lb \times ft}{ft}$$

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Design New Slab Reinforcing for Maximum Negative Moment

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M7_{slab.neg} \times w_{mem} = 1.1\,ft \times kips$

Service Moment: $M_{service} := M7_{slab.serv.neg} \times w_{mem} = 0.74\,ft \times kips$

depth to rebar

$$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 1.75 \times in$

Rectangular Compression Depth:

$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$

$a_{req} = 0.219 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.149 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

$M_{des} = 1.1 \times kip \times ft$

$\phi M_n = 2.621 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 238.3 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^{\;2}}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.333\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)
The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 0.72 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.51 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 5.885 \times kips$

Stress in the steel at service limit state $f_s := \frac{T}{A_s \times w_{mem}} = 14.713 \times ksi$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.429$$

$$s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 10.693 \times in$$

$spacing \times (1 + bundles) = 6 \times in$

$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\phi_v := 0.9$

$$V_u := \left[1.25 \times \left[w_{DC.Railing} + w_{DC.slab} \times (canti + 6in)\right] + 1.75 \times \left(w_{LL} \times canti\right)\right] \times w_{mem} = 0.523 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

Simplified := "yes"

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_c = 2.15 \times kips$

$Check_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

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Design New Slab Reinforcing for Maximum Positive Moment $E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 1.5in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \dfrac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.437 \times in$

Design Moment: $M_{des} := M7_{slab.pos} \times w_{mem} = 0.227\,ft \times kips$

Service Moment: $M_{service} := M7_{slab.serv.pos} \times w_{mem} = 0.147\,ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\dfrac{d_{no_{bar_{no}}}}{2} + \dfrac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 2.25 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \dfrac{M_{des}}{f'_c \times w_{mem}} \times \dfrac{2}{0.9 \times 0.85}}$ $a_{req} = 0.033 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.023 \times in^2$

Check compression for full yielding of actual steel $M_{des} = 0.227 \times kip \times ft$

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$ $\phi M_n = 3.521 \times kip\,ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = \text{"SATISFACTORY"}$ $\dfrac{\phi M_n}{M_{des}} = 1.5 \times 10^3 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^{\ 2}}{6}$ $S = 32 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.333\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)
The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 3834 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 0.842 \times \text{in}$$

Moment Arm: $\text{arm} := d - \frac{x}{3} = 1.969 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{\text{arm}} = 0.896 \times \text{kips}$

Stress in the steel at service limit state: $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 2.24 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := \text{clear} = 1.5 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.857$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (\text{ksi})\ (\text{in}) - 2 \times d_c = 123.178 \times \text{in}}$$

$$\text{spacing} \times (1 + \text{bundles}) = 6 \times \text{in}$$

$\text{Check}_{C.D}\left[s_{max}, \text{spacing} \times (1 + \text{bundles})\right] = \text{"SATISFACTORY"}$

At Existing Cantilever Beam Locations, new slab must void around ex. cantilever (at sidewalk grade). Adjacentslab must carry load from beyond end of ex. cantilever (REF SHT 12)

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 18in$ **Bars per Layer:** $n_{bars} := 3$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \dfrac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.6 \times in^2$ $clear_{spacing} = 5.438 \times in$

Design Moment: $M_{des} := M7_{slab.neg} \times \left(w_{mem} + \dfrac{1ft}{2}\right) = 2.199\ ft \times kips$

Service Moment: $M_{service} := M7_{slab.serv.neg} \times \left(w_{mem} + \dfrac{1ft}{2}\right) = 1.481\ ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\dfrac{d_{no_{bar_{no}}}}{2} + \dfrac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 1.75 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \dfrac{M_{des}}{f'_c \times w_{mem}} \times \dfrac{2}{0.9 \times 0.85}}$ $a_{req} = 0.299 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.305 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 2.199 \times kip \times ft$

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$ $\phi M_n = 3.931 \times kip\ ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = \text{"SATISFACTORY"}$ $\dfrac{\phi M_n}{M_{des}} = 178.7 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $S = 32 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 3$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 3$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1.5\ ft$ x $h_{mem} = 0.333\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 3.552 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 3834 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 0.72 \times \text{in}$$

Moment Arm: $\text{arm} := d - \frac{x}{3} = 1.51 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{\text{arm}} = 11.771 \times \text{kips}$

Stress in the steel at service limit state: $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 19.618 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := \text{clear} = 2 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.429$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (\text{ksi}) \; (\text{in}) - 2 \times d_c = 7.019 \times \text{in}}$$

$$\text{spacing} \times (1 + \text{bundles}) = 6 \times \text{in}$$

$$\text{Check}_{C.D}\left[s_{max}, \text{spacing} \times (1 + \text{bundles})\right] = \text{"SATISFACTORY"}$$

Load to Edge Beam

Dead Load Reaction

$$R_{edge.beam.DC} = 355.027 \times plf$$

Live Load Reaction

$$R_{edge.beam.LL} := w_{LL} \times \left(\frac{span + 4in}{2} + canti\right) = 322.619 \times plf$$

Edge Beam Length = 18ft 0in max (REF SHT 59)

$$L_{edge} := 18ft$$

Edge beam is FIX-FIX:

$$V7_{edge} := 1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL} = 1.008 \times 10^{3} \times plf$$

$$M7_{edge.pos} := \frac{\left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge}^{2}}{24} = 13.6\, ft \times kips$$

$$M7_{edge.neg} := \frac{\left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge}^{2}}{12} = 27.2\, ft \times kips$$

$$M7_{edge.serv.pos} := \frac{\left(R_{edge.beam.DC} + R_{edge.beam.LL}\right) \times L_{edge}^{2}}{24} = 9.1\, ft \times kips$$

$$M7_{edge.serv.neg} := \frac{\left(R_{edge.beam.DC} + R_{edge.beam.LL}\right) \times L_{edge}^{2}}{12} = 18.3\, ft \times kips$$

Check Existing Edge Beam for Capacity

$$E_s := 29000ksi$$

Original structure concrete strength: $f'_c := 8ksi$

Original structure Steel Strength: $F_y := 33ksi$

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Check Existing Edge Beam for Positive Capacity (Neglect new slab)

Assume Edge Beam is similar to Sec. 3 & 5 (REF SHT 38)

Bar info (REF SHT 12): $bar_{area} := 1\frac{1}{8}in^2$ $\quad bar_{diam} := \sqrt{bar_{area}} = 1.061 \times in$

Assumed Clear: $clear := 2in$

Number of layers: $full_{layers} := 1$ $\quad bundled := \text{"no"}$ $\quad bundles := yes_{no}(bundled) = 0$ $\quad \beta_1 = 0.85$

Width of member: $w_{mem} := 6in$

Bars per Layer: $n_{bars} := 1$ $\quad spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member (minus new slab depth): $h_{mem} := 1ft + 1in + 8in - 4in = 1.417\,ft$

$$A_s := \frac{bar_{area}}{spacing} \times full_{layers} \qquad A_s \times w_{mem} = 1.125 \times in^2 \qquad clear_{spacing} = 4.939 \times in$$

Design Moment: $M_{des} := M7_{edge.pos} = 13.613\,ft \times kips$

Service Moment: $M_{service} := M7_{edge.serv.pos} = 9.148\,ft \times kips$

depth to rebar

$$d := h_{mem} - clear - \left[\frac{bar_{diam}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max(1 \times in, bar_{diam}) + bar_{diam}\right)\right]$$

$d = 14.47 \times in$

Rectangular Compression Depth:

$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}} \qquad a_{req} = 0.311 \times in$$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $\quad A_{s.req} \times w_{mem} = 0.384 \times in^2$

Check compression for full yielding of actual steel

$$a := \frac{A_s \times F_y}{0.85 \times f'_c} \qquad a = 0.91 \times in \qquad \phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$$

$M_{des} = 13.613 \times kip \times ft$

$\phi M_n = 39.022 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 286.7 \times \%$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $\quad F_r = 1 \times 10^3 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $\quad S = 578 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $\quad M_{cr} = 50.407 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $\quad bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 6 \times in$

$w_{mem} = 0.5\,ft$ x $h_{mem} = 1.417\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 30.244 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 5422 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 5.348$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 4.477 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 12.977 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 8.459 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 7.519 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.19$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 54.649 \times in}$$

$$spacing \times (1 + bundles) = 6 \times in$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

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Check Existing Edge Beam for Negative Capacity (Including new slab - T-section)

$F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $\quad d_{no_{bar_{no}}} = 0.5 \times in$ $\quad diam_{deform_{bar_{no}}} = 0.563 \times in$ $\quad A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2.5in$

Number of layers: $full_{layers} := 1$ $\quad bundled := "no"$ $\quad bundles := yes_{no}(bundled) = 0$ $\quad \beta_1 = 0.85$

Width of member: $w_{mem} := 12in$ $\quad$ **Bars per Layer:** $n_{bars} := 1$ $\quad spacing := \frac{w_{mem}}{n_{bars}} = 12 \times in$

Height of member: $h_{mem} := (1ft + 1in) + 8in = 1.75\,ft$

$A_s := \frac{bar_{area}}{spacing} \times full_{layers}$ $\quad A_s \times w_{mem} = 1.125 \times in^2$ $\quad clear_{spacing} = 10.939 \times in$

Design Moment: $M_{des} := M7_{edge.neg} = 27.226\,ft \times kips$

Service Moment: $M_{service} := M7_{edge.serv.neg} = 18.296\,ft \times kips$

depth to rebar
$$d := h_{mem} - clear - \left[\frac{bar_{diam}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max(1 \times in, bar_{diam}) + bar_{diam}\right)\right]$$

$d = 17.97 \times in$

Rectangular Compression Depth:
$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$
$a_{req} = 0.249 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $\quad A_{s.req} \times w_{mem} = 0.339 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 27.226 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $\quad a = 0.827 \times in$ $\quad \phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\quad \phi M_n = 88.878 \times kip\,ft$

$Check_{C.D}(\phi M_n, M_{des}) = "SATISFACTORY"$

$\frac{\phi M_n}{M_{des}} = 326.4 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $\quad F_r = 1 \times 10^3 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $\quad S = 882 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $\quad M_{cr} = 76.919 \times \frac{kip \times ft}{ft}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $\quad bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 12 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 1.75\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 92.303 \times kip \times ft$

$M_{cr.Check}$ = "Exception in effect. Moment capacity exceeds 4/3 Moment demand"

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 5422 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 5.348$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 3.773 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 16.712 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 13.138 \times kips$

Stress in the steel at service limit state: $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 11.678 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2.5 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.193$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (\text{ksi})\ (\text{in}) - 2 \times d_c = 32.682 \times in}$$

$$spacing \times (1 + bundles) = 12 \times in$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$a_g := 0.75in$ (Aggregate Size) $\phi_v := 0.9$

$V_u := \frac{5}{8} \times V7_{edge} \times L_{edge} = 11.344 \times kips$

Use Simplified Calc Values?
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := "no"$

Strain in tension reinforcement: $\varepsilon_s := \frac{\left[\frac{|M_{des}|}{(0.9 \times d)} + |V_u|\right]}{E_s \times A_s \times w_{mem}} = 9.669 \times 10^{-4}$

Without min Transverse Steel: $s_x := 0.9 \times d = 16.17 \times in$ (approximate distance between crack control steel layers) $s_{xe} := s_x \times \frac{1.38}{a_g + 0.63in} = 16.173$

Angle of inclination of diagonal compressive strut

$\beta_{wo} := \frac{4.8}{(1 + 750 \times \varepsilon_s)} \times \frac{51}{(39 + s_{xe})} = 2.572$

$V_{c.wo} := 0.0316 \times \beta_{wo} \times \sqrt{f'_c\,(ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_{c.wo} = 40.15 \times kips$

$ShearReinf_{Req}$ = "Factored Capacity is greater than 2x demand - No Reinforcing Required"

2x requirement does not apply to slabs, footings, or culverts (5.8.2.4)

With min Transverse Steel: $\beta_w := if\left[yes_{no}(Simplified) = 1, 2.0, \frac{4.8}{(1 + 750 \times \varepsilon_s)}\right] = 2.782$

$\theta := if\left[yes_{no}(Simplified) = 1, 45, (29 + 3500 \times \varepsilon_s)\right] deg = 32.384 \times deg$

$V_{c.w} := 0.0316 \times \beta_w \times \sqrt{f'_c\,(ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_{c.w} = 43.435 \times kips$

$v_u := \frac{|V_u|}{(0.9 \times d) \times w_{mem}} = 0.058 \times ksi$ $s_{max} := if(v_u < 0.125 \times f'_c, min(0.8 \times d, 24in), min(0.4 \times d, 12in)) = 14.376 \times in$

$s_{stirrup} := 12in$ $A_{v.min} := 0.0316 \times \sqrt{f'_c\,(ksi)} \times \frac{w_{mem} \times s_{stirrup}}{F_y}$ $A_{v.min} = 0.215 \times in^2$

Minimum area applies only to sections where 2x requirement applies (5.8.2.5)

$A_{stirrup} := 2 \times A_{no.5}$ $A_{stirrup} = 0.62 \times in^2$

(REF SHTS 12 & 38)

minimum = "Minimum Provided"

$\varphi V_c := \phi_v \times if(A_{stirrup} \geq A_{v.min}, V_{c.w}, V_{c.wo}) = 43.435 \times kips$

angle between transverse reinforcing and longitudinal axis $\alpha := 90deg$

$V_s := \frac{A_{stirrup} \times F_y \times (0.9 \times d) \times (cot(\theta) + cot(\alpha)) \times sin(\alpha)}{s_{stirrup}}$ $\varphi V_s := \phi_v \times V_s = 71.144 \times kips$

$\phi V_{n_1} := \varphi V_c + \varphi V_s$ $\phi V_{n_1} = 114.579 \times kips$

Governing Shear Capacity:

$\phi V_{n_2} := \phi_v \times 0.25 \times f'_c \times w_{mem} \times (0.9 \times d)$ $\phi V_{n_2} = 349.33 \times kips$

$\phi V_n := min(\phi V_n)$ $\phi V_n = 114.579 \times kips$

$Check_{C.D}(\phi V_n, V_u)$ = "SATISFACTORY"

Tension load to new slab must travel though shear friction at the existing concrete to new concrete interface

Use Galvanized Headed Anchor Stud ILO rebar $f'_c := 4ksi$ $F_y := 36ksi$ For shear $\phi := 0.9$ (5.5.4.2.1)

Interface Shear Transfer (5.8.4)

Slab on clean concrete

$c := 0.075ksi$

$\mu := 0.6$

$K_1 := 0.2$

$K_2 := 0.8ksi$

For cast-in-place concrete slab on clean concrete girder surfaces, intentionally roughened to 0.25in
c = 0.28 ksi μ = 1.0 K1 = 0.3 K2 = 1.8 ksi (NW) or 1.3 ksi (LW)
For normal weight concrete placed monolithically
c = 0.40 ksi μ = 1.4 K1 = 0.25 K2 = 1.5 ksi
For normal weight concrete placed monolithically, or against surface intentionally roughened to 0.25in
c = 0.24 ksi μ = 1.0 K1 = 0.25 K2 = 1.0 ksi
For normal weight concrete against surface intentionally roughened to 0.25in
c = 0.24 ksi μ = 1.0 K1 = 0.25 K2 = 1.5 ksi
For normal weight concrete against surface not intentionally roughened
c = 0.075 ksi μ = 0.6 K1 = 0.2 K2 = 0.8 ksi

$A_{cv} := 6in = 72 \times \frac{in^2}{ft}$ Area of Concrete Engaged in Shear Transfer

$A_{vf} := \pi \times \frac{(0.5in)^2}{4} \times \frac{1}{18in} = 0.131 \times \frac{in^2}{ft}$ Area of Shear Reinforcement Crossing the Shear Plane

$P_c := R_{edge.beam.DC} = 355.027 \times plf$ Permanent net compressive force normal to the shear plane; if force is tensile P_c = 0.0kips (conservative)

$$V_{n'_1} := c \times A_{cv} + \mu \left(A_{vf} \times F_y + P_c\right) = 8.44 \times klf$$

Must be less than: $V_{n'_2} := K_1 \times f'_c \times A_{cv} = 57.6 \times klf$ $V_{n'_3} := K_2 \times A_{cv} = 57.6 \times klf$

$$V_n := \min\left(V_{n'}\right) = 8.44 \times klf$$

$$\phi \times V_n = 7.596 \times klf$$

Shear Demand:
(Zero moment to maximum moment = L/4 REF AISC Table 3-23)

$$Demand := \frac{A_{no.4} \times 60ksi}{\frac{L_{edge}}{4}} = 2.667 \times klf$$

$Check_{C.D}\left(\phi \times V_n, Demand\right) = \text{"SATISFACTORY"}$

Tension dowel requirement at Barrier

Dead Load reaction at barrier: $R_{barrier.DC} = 34.629 \times plf$

Partial Live Load reaction at barrier: $R_{barrier.LL} := \dfrac{-w_{LL} \times \dfrac{canti^2}{2}}{span + 4in} = -47.547 \times plf$

Partial Wind Uplift:

AASHTO specifies a superstructre wind uplift load of 20psf times the width of the bridge applied at the windward quarter point. To create an equivalent distributed force and resultant, a triangular distribution of wind load will be established across the windward 3/4 of the bridge width. The Highest pressure will occur under the proposed sidewalk.

$$p_{wind} := \frac{20psf \times W}{\frac{3}{4} \times W} \times 2 = 53.333 \times psf$$

$w_{DC.slab} = 51.667 \times psf$

53.3psf is nearly equivalent to the 51.7psf slab dead load. The slab is acceptable for wind uplift loads

Conservatively apply maximum wind to area tributary to barrier reaction.

$$R_{barrier.WL} := -p_{wind} \times \frac{span}{2} = -99.471 \times plf$$

Uplift at Barrier:

$$Uplift_I := -\left(1.75 \times R_{barrier.LL} + 0.9 \times R_{barrier.DC}\right) = 52.041 \times plf$$

$$Uplift_{III} := -\left(1.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = 108.093 \times plf$$

$$Uplift_V := -\left(1.35 \times R_{barrier.LL} + 0.4 \times R_{barrier.WL} + 0.9 \times R_{barrier.DC}\right) = 72.811 \times plf$$

$$\max\left(Uplift_I, Uplift_{III}, Uplift_V\right) = 108.093 \times plf$$

$$Tension := \frac{0.9 \times A_{no.4} \times 60ksi}{4ft} = 2.7 \times 10^3 \times plf$$

Uplift force is minor Install #4 bars vertically into slab at 4ft on center.

WSDOT BDM 5.5.4 requires 8 inches of epoxy embedment for full development.

Thickness of the deck is 6.5 inches
Thickness at the curb is 9 inches (REF SHT 30)

Full Development is not required. Provide 6" embed

$$\frac{6in}{8in} \times Tension = 2.025 \times 10^3 \times plf$$

$$Check_{C.D}\left(\frac{6in}{8in} \times Tension, \max\left(Uplift_I, Uplift_{III}, Uplift_V\right)\right) = \text{"SATISFACTORY"}$$

Design Moment on Existing Cantilever

$$M_{fact.ex} := \left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge} \times span = 67.705\, ft \times kips$$

$$M_{serv.ex} := \left(R_{edge.beam.DC} + R_{edge.beam.LL}\right) \times L_{edge} \times span = 45.499\, ft \times kips$$

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Check Existing Cantilever Beam for applied moments

$E_s := 29000ksi$

Concrete Strength of Existing Structure: $f'_c := 8ksi$

Rebar Strength in Existing Structure: $F_y := 33ksi$

Bar info: (REF SHT 12) $Area := 1.125in^2$ $d_{bar} := \sqrt{Area} = 1.061 \times in$

Assumed Clear: (REF SHT 12) $clear := 3in$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.65$

Width of member $w_{mem} := 8in$

Bars per Layer: $n_{bars} := 2$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 4 \times in$

Height of member: $h_{mem} := 3ft + 3in - (3.5in + 2.5in)$

$A_s := \dfrac{Area}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 2.25 \times in^2$ $clear_{spacing} = 2.939 \times in$

Design Moment: $M_{des} := M_{fact.ex} = 67.705\,ft \times kips$

Service Moment: $M_{service} := M_{serv.ex} = 45.499\,ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\dfrac{d_{bar}}{2} + \dfrac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{bar}\right) + d_{bar}\right)\right]$

$d = 29.47 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \dfrac{M_{des}}{f'_c \times w_{mem}} \times \dfrac{2}{0.9 \times 0.85}}$ $a_{req} = 0.569 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.937 \times in^2$

Check compression for full yielding of actual steel

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 1.365 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$

$M_{des} = 67.705 \times kip \times ft$

$\phi M_n = 160.309 \times kip\ ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = "SATISFACTORY"$

$\dfrac{\phi M_n}{M_{des}} = 236.8 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 1 \times 10^3 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $S = 2.178 \times 10^3 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 189.943 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 4 \times in$

$w_{mem} = 0.667\,ft$ x $h_{mem} = 2.75\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 151.954 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 5422 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 5.348$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 8.031 \times \text{in}$$

Moment Arm: $\text{arm} := d - \frac{x}{3} = 26.793 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{\text{arm}} = 20.378 \times \text{kips}$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 9.057 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := \text{clear} = 3 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.143$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(\text{ksi})\,(\text{in}) - 2 \times d_c = 44.72 \times \text{in}}$$

$$\text{spacing} \times (1 + \text{bundles}) = 4 \times \text{in}$$

$\text{Check}_{C.D}\left[s_{max}, \text{spacing} \times (1 + \text{bundles})\right] = \text{"SATISFACTORY"}$

Additional dead load to the structure from walkway widening.

Additional Dead Load, Slab and Railing: $plf_{slab.railing} := R_{barrier.DC} + R_{edge.beam.DC} = 389.7 \times plf$

Dead Load of New Traffic Barrier + BP rail: $$plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$$

Total weight of new work: $plf_{new} := plf_{slab.railing} + plf_{barrier} = 776.9 \times plf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedestal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 37 & 38): $A_{ped} := (1ft + 4in) \times 12in - 4 \times (1.25in \times 2in) = 1.264\,ft^2$

Pedestal weight: $$plf_{pedestal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{L_{edge}} = 39.9 \times plf$$

Dead Load of Existing intermediate Conc Rail: $$plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{L_{edge}} = 127 \times plf$$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 59, 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{L_{edge}} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{L_{edge}}\right]$$

$$plf_{metal.rail} = 16.6 \times plf$$

Dead Load of Existing Edge Beam: $plf_{ex.edge.beam} := 6in \times 4in \times \gamma_{c.ex} = 25.0 \times plf$

Dead Load of Existing sidewalk:
(REF SHT 37) $plf_{ex.sidewalk} := 3.5in \times 4ft \times \gamma_{c.ex} = 175 \times plf$

Dead Load of Existing Curb:
(REF SHT 31) $$plf_{curb} := \left[18in \times \left(7in + \frac{3in}{2}\right) - 3in \times 2.5in\right] \times \gamma_{c.ex} = 151.563 \times plf$$

Total weight of removals:

$$plf_{rem} := plf_{pedestal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{ex.sidewalk} + plf_{curb} = 535.1 \times plf$$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 241.8 \times plf$

Weight of Segment 7 calculated for seismic analysis: $Segment7_{weight} := 5350kips$

Length of Segment 7: $Segment7_{length} := 287.32ft = 287.32\,ft$

Average Per foot weight of Segment 7: $$\frac{Segment7_{weight}}{Segment7_{length}} = 1.862 \times 10^4 \times plf$$

Percent Increase due to extended sidewalk load: $$\frac{plf_{increase}}{\frac{Segment7_{weight}}{Segment7_{length}}} = 1.298 \times \%$$ Less than 10% OKAY

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck:
(REF SHT E-6/91/4 : 782-95)

$H_{mast} := 30ft + (3ft + 7in) = 33.583\,ft$

Length of Luminaire Mast arm:
(12' max REF WSDOT STD J-28.10-01)

$L_{arm} := 12ft$

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d$ (psf)

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\,ft$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$

Table 3–5. Height and Exposure Factors, K_z	
Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head
(REF WSDOT BDM 10.1(B))

$A := 3.3ft^2$

$V_{wind.lum} := P_z \times A = 44.876\,lb$

Height to point of connection: $H := H_{mast} + [(1ft + 9in) - 6in] = 34.833\,ft$

$M_{wind.lum} := V_{wind.lum} \times H = 1.563\,ft \times kips$

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Wind on Mast Arm

$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\ ft$

$K_z := K_{z.eq}\left(H_{arm}, 900ft, 9.5\right) = 1.151$

$$C_{d.cylinder}(V, d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \dfrac{V}{mph} \times \dfrac{d}{ft} \\ CD \leftarrow \dfrac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{ if } \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{ if } \ \omega \ge 78 \\ CD \end{array}$$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Cylinders (3in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 3in) = 1.1$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\ ft^2$

$V_{wind.arm} := P_z \times A = 89.752\ lb$

Height to point of connection:

$H := H_{mast} + [(1ft + 9in) - 6in] = 34.833\ ft$

$M_{wind.arm} := V_{wind.arm} \times H = 3.126\ ft \times kips$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\ ft$$

$K_z := K_{z.eq}\left(H_{pole}, 900ft, 9.5\right) = 1.079$

Cylinders (8in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 8in) = 0.553$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\ ft^2$

$V_{wind.pole} := P_z \times A = 315.609\ lb$

Height to point of connection: $H := \dfrac{H_{mast}}{2} + [(1ft + 9in) - 6in] = 18.042\ ft$

$M_{wind.pole} := V_{wind.pole} \times H = 5.694\ ft \times kips$

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$	
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		
Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r - \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

$V_{wind} := V_{wind.lum} + V_{wind.arm} + V_{wind.pole} = 0.45 \times kips$

Total Moment from wind (omnidirectional):

$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 10.384\ ft \times kips$

Dead Loads

Distance from Pole location to Point of connection: $Offset := canti + 6in + 3in + 5in = 3.437\,ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$W_{lum} := 60lb$

$M_{DC.lum} := W_{lum} \times \left(L_{arm} - Offset\right) = 0.514\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$

$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.118\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\,lb$

$M_{DC.pole} := W_{pole} \times (-Offset) = -1.18\,ft \times kips$

Total Moment from Dead Load: $M_{DC} := M_{DC.lum} + M_{DC.arm} + M_{DC.pole} = -0.549\,ft \times kips$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$

$M_{Ice.lum} := Ice_{lum} \times \left(L_{arm} - Offset\right) = 0.509\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$

$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.072\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$Ice_{pole} := \pi \times 8in \times 3psf \times H_{mast} = 211.01\,lb$

$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -0.725\,ft \times kips$

Total Moment from Dead Load: $M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -0.144\,ft \times kips$

Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} |M_{DC}| \times \dfrac{1}{100\%} \\ \left(|M_{DC}| + M_{wind}\right) \times \dfrac{1}{133\%} \\ \left(|M_{DC} + M_{Ice}| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.549 \\ 8.22 \\ 4.424 \end{pmatrix} ft \times kips$$

Governing Load $\quad M_{design} := \max\left(M_{design'}\right) = 8.22\ ft \times kips$

Connect pole to square tube:

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1) $\quad M_n = M_p = F_y \times Z$

Flange Compact Criteria Buckling Limit (AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1) $\quad \lambda_{pf} = 1.12\sqrt{\dfrac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\dfrac{E}{F_y}}$

Flange Local Buckling Limit For Compact Flanges (AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges (AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3) $\quad M_n = F_y \times S_{eff}$

Effective width of compression flange (AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 10x10x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$M_{n.\Omega.AISC} := 42.8 kip \times ft$$

$$M_{n.\Omega.AASHTO.HSS10x10x3} := \frac{1.0}{0.9} \times M_{n.\Omega.AISC} = 47.6 \times kip \times ft$$

$$\frac{M_{design}}{M_{n.\Omega.AASHTO.HSS10x10x3}} = 0.173$$

$$Check_{C.D}\left(M_{n.\Omega.AASHTO.HSS10x10x3}, M_{design}\right) = \text{"SATISFACTORY"}$$

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Light pole attaches to 12" concrete cantilever . Cantilever is reinforced with 2 - 1.125 sq in bars top and bottom. Reinforcing steel in the existing structure is 33 ksi.
(REF SHT 44)

Tension / Compression moment couple:

$$TC := \frac{M_{design}}{10in} = 9.864 \times kips$$

$$\frac{\left(2 \times 1.125in^2\right) \times 33ksi}{1.67} = 44.461 \times kips$$

Tension couple would acceptably transfer load to existing steel.

Required diameter of anchor

$$A_{req} := \frac{TC \times 1.67}{2 \times 36ksi} = 0.229 \times in^2$$

$$diameter := \sqrt{4 \times \frac{A_{req}}{\pi}} = 0.54 \times in$$

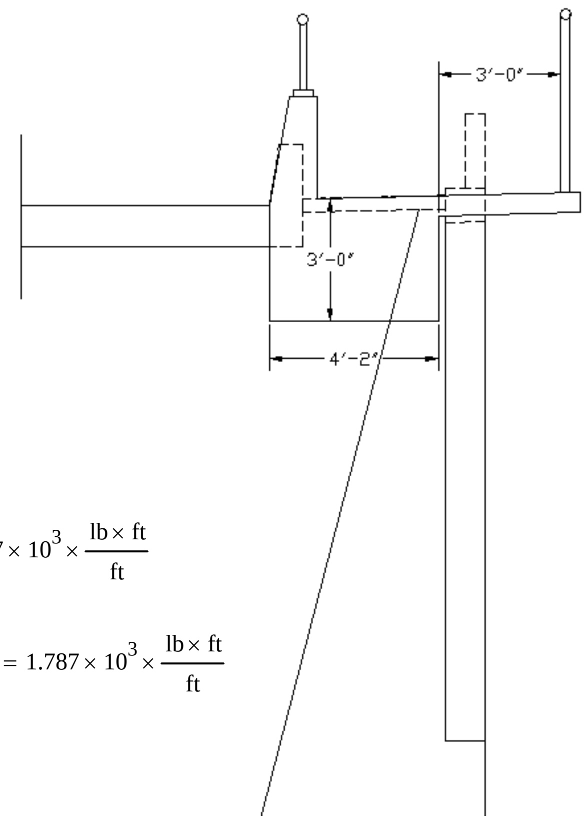
BALLARD Widening - Segment 8 - 6t Sidewalk Extension

The sidewalk extension in Segment 8 shall be made in the form of an anchor slab with cantilevered sidewalk extension. An anchor slab is preferred in order to isolate the existing retaining wall structure from new loads introduced by the sidewalk extension and TL-4 traffic barrier.

The slab has been designed for stability under the following conditions:

1) Unbalanced pedestrian live load overturning using strength factors to check LRFD eccentricity limitations
2.) Extreme limit state 10 kip vehicle impact stability load applied to the top of the traffic barrier

$L_{Overhang} := 3ft + 0in$ $\qquad w_{LL} := 75psf$ $\qquad \gamma_c := 155pcf$



Overturning Moment

$$OM_{LL} := w_{LL} \times \frac{L_{Overhang}^2}{2} = 337.5 \times \frac{lb \times ft}{ft}$$

$$OM_{DC} := 6in \times \gamma_c \times \frac{\left(L_{Overhang} + 6in\right)^2}{2} = 474.688 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Lat.Railing} := 50plf \times 54in = 225 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Vert.Railing} := 50plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 155.99 \times \frac{lb \times ft}{ft}$$

$$OM_{DC.Railing} := 36.6plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 114.184 \times \frac{lb \times ft}{ft}$$

$$OM_{service} := \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + \left(OM_{DC} + OM_{DC.Railing}\right) = 1.307 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$OM_{factored} := 1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + 0.9 \times \left(OM_{DC} + OM_{DC.Railing}\right) = 1.787 \times 10^3 \times \frac{lb \times ft}{ft}$$

Continuous Anchor Slab Length (minimum): $\qquad L_{AS} := 20ft$

$$OM_{veh} := 10kips \times (32in + 11in) = 3.583 \times 10^4 ft \times \frac{lb \times ft}{ft}$$

$$OM_{Extreme} := OM_{veh} + L_{AS} \times \left(OM_{DC} + OM_{DC.Railing}\right) = 4.761 \times 10^4 \times lb \times ft$$

Resisting Moment $\qquad D_{slab} := 36in \qquad Width := 4ft + 2in$

$$Res_{DC.slab} := D_{slab} \times \gamma_c \times \frac{(Width)^2}{2} = 4.036 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{DC.barrier} := 32\,in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times \left[4ft + 2in - \left[\frac{8in}{2} + \left(32in \times \frac{4}{21}\right)\right]\right] = 1.265 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{service} := Res_{DC.slab} + Res_{DC.barrier} = 5.302 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{factored} := 0.9 \times \left(Res_{DC.slab} + Res_{DC.barrier}\right) = 4.772 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{Extreme} := Res_{service} \times L_{AS} = 1.06 \times 10^5 \times lb \times ft$$

Strength Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf\right] \ldots + D_{slab} \times \gamma_c \times (Width) + 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c = 2.626 \times 10^3 \times plf$$

$$V_{LL} := \left(w_{LL} \times L_{Overhang} + 50plf\right) = 275 \times plf$$

$$M_{abt.toe} := OM_{factored} - Res_{factored} = -2.984 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL} + 1.75 \times V_{LL}\right)}\right] = 1.034\,ft$$

$$ecc_{limit} := \frac{Width}{4} = 1.042\,ft$$ (10.6.3.3)

Bearing Pressure Check

$$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$$

$$\frac{V_{DL} + V_{LL}}{2 \times \left(\frac{Width}{2} - ecc\right)} = 1.382 \times 10^3 \times psf$$

Bearing Pressure is reasonable

Extreme Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf\right] \ldots + D_{slab} \times \gamma_c \times (Width) + in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c = 2.257 \times 10^3 \times plf$$

$$M_{abt.toe} := OM_{Extreme} - Res_{Extreme} = -5.843 \times 10^4 \times lb \times ft$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL}\right) \times L_{AS}}\right] = 0.645\,ft$$

$$ecc_{limit} := \frac{Width}{3} = 1.389\,ft$$ (10.6.4.2)

$$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$$

Sliding Resistance

$\varphi_{\tau} := 0.80$ (Table 10.5.5.2.2-1)

$\varphi_f := 28deg$ Assuming a reasonably shallow angle of internal friction

$\mu_{R.t} := \tan\left(\varphi_f\right) = 0.532$ (EQ 10.6.3.4-2)

$$Sliding_{res.DC.slab.at.wall} := ft \times \gamma_c \times (1ft + 1.5in + 2in - 1in) \times L_{AS} = 3.746 \times 10^3\,ft \times plf$$

$$Sliding_{res.DC.slab.past} := D_{slab} \times \gamma_c \times [5ft + 2.5in - (1ft + 1.5in) - 2in] \times L_{AS} = 3.643 \times 10^4\,ft \times plf$$

$$Sliding_{res.DC.barrier} := 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times L_{AS} = 7.611 \times 10^3\,ft \times plf$$

$$Sliding_{resistance} := \varphi_{\tau} \times \mu_{R.t} \times \left(Sliding_{res.DC.slab.at.wall} + Sliding_{res.DC.slab.past} + Sliding_{res.DC.barrier}\right) = 2.032 \times 10^4\,ft \times plf$$

$$Check_{C.D}\left(Sliding_{resistance}, 10kips\right) = \text{"SATISFACTORY"}$$

Cantievered Sidewalk Design

$E_s := 29000ksi$ $F_y := 60ksi$ $f'_c := 4ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2.5in = 0.208\,ft$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.438 \times in$

Design Moment:

$$M_{des} := \left[1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + 1.25 \times \left(OM_{DC} + OM_{DC.Railing}\right)\right] \times w_{mem} = 1.993 \times 10^3 \times lb \times ft$$

Service Moment: $M_{service} := OM_{service} \times w_{mem} = 1.307 \times 10^3 \times lb \times ft$

depth to rebar

$$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 3.25 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.207 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.141 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.588 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 1.993 \times kip \times ft$

$\phi M_n = 5.321 \times kip\,ft$

$\frac{\phi M_n}{M_{des}} = 266.9 \times \%$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.24 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 480 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 72 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 2.88 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.5\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 3.456 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf} \right) \right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2} \right) = \left(A_s \times w_{mem} \right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2} \right) \times x^2 + \left(A_s \times w_{mem} \times n \right) \times x - \left(A_s \times w_{mem} \times n \times d \right) = 0$$

$$x := \max \left[Quadratic_{a.b.c} \left[\frac{\left(w_{mem} \times \frac{1}{2} \right)}{ft}, \frac{\left(A_s \times w_{mem} \times n \right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d \right)}{ft^3} \right] \times 12in \right] = 1.053 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 2.899 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 5.411 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 13.528 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 2.5 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c \right)} = 2.02$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi) \ (in) - 2 \times d_c = 14.208 \times in}$$

$spacing \times (1 + bundles) = 6 \times in$

$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles) \right] =$ "SATISFACTORY"

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$V_u := \left[\left(1.75 \times w_{LL} + 1.25 \times \gamma_c \times 6in \right) \times L_{Overhang} + 1.75 \times 50plf + 1.25 \times 36.6plf \right] \times w_{mem} = 0.818 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

Simplified := "yes"

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c \ (ksi)} \times w_{mem} \times (0.9 \times d)$

$\phi_v \times V_c = 3.993 \times kips$

$Check_{C.D}\left(\phi_v \times V_c, V_u \right) =$ "SATISFACTORY"

Slab edge deflection

Moment of Inertia: $I := \frac{(6in)^3}{12} = 216 \times \frac{in^4}{ft}$

Modulus of Elasticity: $E_c = 5.521 \times 10^8 \frac{lb}{ft^2}$

Edge Deflection due to ped load: (Cantilever) $\delta := \frac{w_{LL} \times L_{Overhang}^{\ 4}}{8 \times E_c \times I} = 1.584 \times 10^{-3} \times in$

$\frac{L_{Overhang}}{\delta} = 22722$ Exceeds 300 per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges. section 5

SATISFACTORY


ALLENGES.
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Report a problem


Plants growing through joint indicates slab is not monolithic to abutment wall

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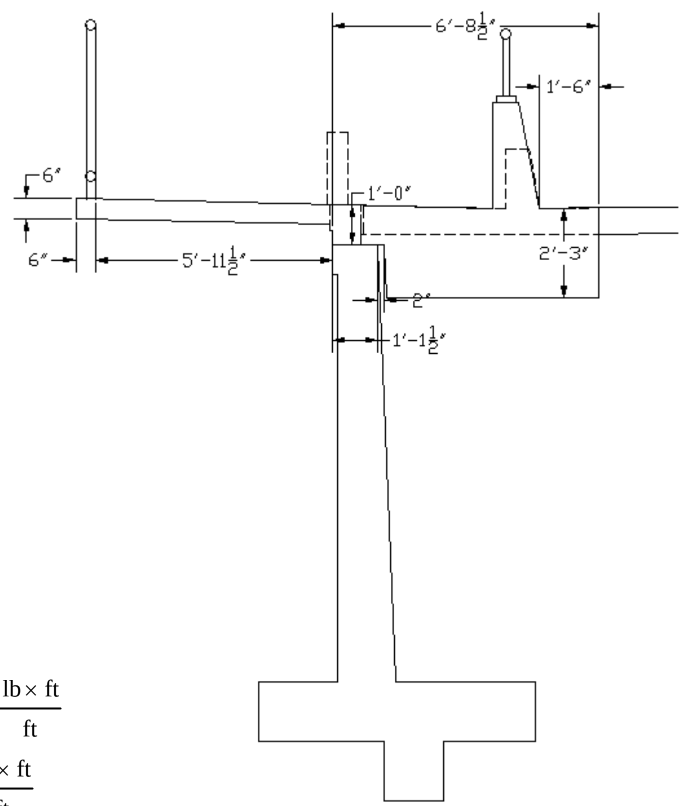
BALLARD Widening - Segment 1 - 10ft Sidewalk Extension

The sidewalk extension in Segment 1 shall be made in the form of an anchor slab with cantilevered sidewalk extension. An anchor slab is preferred in order to isolate the existing retaining wall structure from new loads introduced by the sidewalk extension and TL-4 traffic barrier.

In Segment 1, the anchor slab shall be cast against the top of the existing retaining wall. This will allow for transfer of some vertical loads into the existing stem wall (additional axial loading is minor) while not transferring bending moments (more significant loading condition) or potential lateral loads from the new TL-4 traffic barrier.

The slab has been designed for stability under the following conditions:

1) Unbalanced pedestrian live load overturning using strength factors to check LRFD eccentricity limitations
2.) Extreme limit state 10 kip vehicle impact stability load applied to the top of the traffic barrier



$L_{Overhang} := 5ft + 11.5in$ $\quad w_{LL} := 75psf$ $\quad \gamma_c := 155pcf$

Overturning Moment

$$OM_{LL} := w_{LL} \times \frac{L_{Overhang}^2}{2} = 1.331 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$OM_{DC} := 6in \times \gamma_c \times \frac{(L_{Overhang} + 6in)^2}{2} = 1.616 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Lat.Railing} := 50plf \times 54in = 225 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Vert.Railing} := 50plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 303.906 \times \frac{lb \times ft}{ft}$$

$$OM_{DC.Railing} := 36.6plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 222.459 \times \frac{lb \times ft}{ft}$$

$$OM_{service} := \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + \left(OM_{DC} + OM_{DC.Railing}\right) = 3.699 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$OM_{factored} := 1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + 0.9 \times \left(OM_{DC} + OM_{DC.Railing}\right) = 4.91 \times 10^3 \times \frac{lb \times ft}{ft}$$

Continuous Anchor Slab Length (minimum): $\quad L_{AS} := 20ft$

$$OM_{veh} := 10kips \times (32in + 11in) = 3.583 \times 10^4 ft \times \frac{lb \times ft}{ft}$$

$$OM_{Extreme} := OM_{veh} + L_{AS} \times \left(OM_{DC} + OM_{DC.Railing}\right) = 7.261 \times 10^4 \times lb \times ft$$

Resisting Moment

$D_{slab} := 2ft + 3in$ $\quad$ $Width := 6ft + 8.5in$

$$Res_{DC.slab} := 12in \times \gamma_c \times \frac{(Width)^2}{2} \ldots$$
$$+ \left(D_{slab} - 12in\right) \times \gamma_c \times [Width - (1ft + 1.5in) - 2in] \times \left[\frac{[Width - (1ft + 3.5in)]}{2} + (1ft + 1.5in) + 2in\right]$$

$$Res_{DC.slab} = 7.686 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{DC.barrier} := 32\,in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times \left[4ft + 2in - \left[\frac{8in}{2} + \left(32in \times \frac{4}{21}\right)\right]\right] = 1.265 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{service} := Res_{DC.slab} + Res_{DC.barrier} = 8.951 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{factored} := 0.9 \times \left(Res_{DC.slab} + Res_{DC.barrier}\right) = 8.056 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$Res_{Extreme} := Res_{service} \times L_{AS} = 1.79 \times 10^5 \times lb \times ft$$

Strength Eccentricity

$$V_{DL} := 6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf \ldots$$
$$+ 12in \times \gamma_c \times Width + \left(D_{slab} - 12in\right) \times \gamma_c \times [Width - (1ft + 1.5in) - 2in] + 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c$$

$$V_{DL} = 3.007 \times 10^3 \times plf$$

$$V_{LL} := \left(w_{LL} \times L_{Overhang} + 50plf\right) = 496.875 \times plf$$

$$M_{abt.toe} := OM_{factored} - Res_{factored} = -3.146 \times 10^3 \times \frac{lb \times ft}{ft}$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL} + 1.75 \times V_{LL}\right)}\right] = 2.474\,ft$$

$$ecc_{limit} := \frac{3}{8} \times Width = 2.516\,ft$$ (10.6.3.3) (Foundation on Rock)

Bearing Pressure Check

$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$

$$V_{DL} + V_{LL} = 3.504 \times 10^3 \times plf$$

Bearing Pressure is reasonable as axial load on retaining wall stem

Extreme Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times \left(L_{Overhang} + 6in\right) + 36.6plf\right] \ldots = 2.889 \times 10^3 \times plf$$
$$+ D_{slab} \times \gamma_c \times (Width) + in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c$$

$$M_{abt.toe} := OM_{Extreme} - Res_{Extreme} = -1.064 \times 10^5 \times lb \times ft$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{\left(0.9 \times V_{DL}\right) \times L_{AS}}\right] = 1.308\,ft$$

$$ecc_{limit} := \frac{Width}{3} = 2.236\,ft$$ (10.6.4.2)

$Check_{C.D}\left(ecc_{limit}, ecc\right) = \text{"SATISFACTORY"}$

Sliding Resistance

$\varphi_\tau := 0.80$ (Table 10.5.5.2.2-1)

$\varphi_f := 28\text{deg}$ Assuming a reasonably shallow angle of internal friction

$\mu_{R.t} := \tan(\varphi_f) = 0.532$ (EQ 10.6.3.4-2)

$$Sliding_{res.DC.slab.at.wall} := ft \times \gamma_c \times (1ft + 1.5in + 2in - 1in) \times L_{AS} = 3.746 \times 10^3\, ft \times plf$$

$$Sliding_{res.DC.slab.past} := D_{slab} \times \gamma_c \times [5ft + 2.5in - (1ft + 1.5in) - 2in] \times L_{AS} = 2.732 \times 10^4\, ft \times plf$$

$$Sliding_{res.DC.barrier} := 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times L_{AS} = 7.611 \times 10^3\, ft \times plf$$

$$Sliding_{resistance} := \varphi_\tau \times \mu_{R.t} \times \left(Sliding_{res.DC.slab.at.wall} + Sliding_{res.DC.slab.past} + Sliding_{res.DC.barrier}\right) = 1.645 \times 10^4\, ft \times plf$$

$Check_{C.D}(Sliding_{resistance}, 10kips) = $ "SATISFACTORY"

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Cantievered Sidewalk Design

$E_s := 29000\text{ksi}$ $F_y := 60\text{ksi}$ $f'_c := 4\text{ksi}$

Bar size (US number): $bar_{no} := 6$ $d_{no_{bar_{no}}} = 0.75 \times in$ $diam_{deform_{bar_{no}}} = 0.875 \times in$ $A_{no_{bar_{no}}} = 0.44 \times in^2$

$clear := 2.5in = 0.208\,ft$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6in$ $A_s := \dfrac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.88 \times in^2$ $clear_{spacing} = 5.125 \times in$

Design Moment: $M_{des} := \left[1.75 \times (OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}) \ldots + 1.25 \times (OM_{DC} + OM_{DC.Railing})\right] \times w_{mem} = 5.554 \times 10^3 \times lb \times ft$

Service Moment: $M_{service} := OM_{service} \times w_{mem} = 3.699 \times 10^3 \times lb \times ft$

depth to rebar $d := h_{mem} - clear - \left[\dfrac{d_{no_{bar_{no}}}}{2} + \dfrac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 3.125 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \dfrac{M_{des}}{f'_c \times w_{mem}} \times \dfrac{2}{0.9 \times 0.85}}$ $a_{req} = 0.648 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.441 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 5.554 \times kip \times ft$

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 1.294 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$ $\phi M_n = 9.813 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$ $\dfrac{\phi M_n}{M_{des}} = 176.7 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.24 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 480 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $S = 72 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 2.88 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 6$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.5\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 3.456 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$\boxed{w_c := 0.150kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 1.388 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 2.662 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 16.673 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 18.946 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\boxed{\gamma_e := 0.75}$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2.5 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.02$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(ksi)\ (in) - 2 \times d_c = 8.715 \times in}$$

$spacing \times (1 + bundles) = 6 \times in$

$\boxed{\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$\boxed{V_u := \left[\left(1.75 \times w_{LL} + 1.25 \times \gamma_c \times 6in\right) \times L_{Overhang} + 1.75 \times 50plf + 1.25 \times 36.6plf\right] \times w_{mem} = 1.492 \times kips}$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$\boxed{Simplified := \text{"yes"}}$

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_c = 3.839 \times kips$

$\boxed{\text{Check}_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}}$

Slab edge deflection

Moment of Inertia: $I := \frac{(6in)^3}{12} = 216 \times \frac{in^4}{ft}$

Modulus of Elasticity: $E_c = 5.521 \times 10^8 \frac{lb}{ft^2}$

Edge Deflection due to ped load: (Cantilever) $\delta := \frac{w_{LL} \times L_{Overhang}{}^4}{8 \times E_c \times I} = 0.025 \times in$

$\frac{L_{Overhang}}{\delta} = 2900$ Exceeds 300 per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges. section 5

SATISFACTORY

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BALLARD - 10FT - SEGMENT 2

$\gamma_c := 155pcf$

New construction

Existing overhang = 4ft 1in max to outside of existing pedestal (REF SHT 36)
2ft 0in min to outside of existing pedestal (REF SHT 35)

Distance from outside of existing pedestal to inside of existing curb = 5ft 7in (REF SHT 42)

Additional walkway length (10ft): $Addl_L := \left[10ft - \left[(5ft + 7in) - \left(8in + 32in \times \frac{4}{21}\right)\right]\right] = 5.591\,ft$

Total unsupported walkway length: $Unsupp_L := Addl_L + (4ft + 1in) = 9.675\,ft$

Walkway will be supported along outside edge and connect to existing cantilevered slab (PIN-FIX)

Thickness of Slab (New): $t := 4in$

$w_{DC} := t \times \gamma_c = 51.7 \times psf$

$w_{LL} := 75psf$

Positive Moment in new slab assuming FIX connection at face of existing beam

$M2_{DC.pos} := \frac{9}{128} \times w_{DC} \times Unsupp_L^2 = 340 \times \frac{ft \times lb}{ft}$

$\frac{w_{DC} \times Addl_L^2}{8} = 201.902 \times \frac{lb \times ft}{ft}$

$M2_{LL.pos} := \frac{9}{128} \times w_{LL} \times Unsupp_L^2 = 493.6 \times \frac{ft \times lb}{ft}$

Location of Zero moment for PIN-FIX connections occurs at L/4 from fixed end: $\frac{Unsupp_L}{4} = 2.419\,ft$

Equation assumes uniform moment of inertia along the length of the beam. The true resulting design load is bounded by the positive moment of the full unsupported length of the slab and the negative moment of the additional length (assuming existing slab is rigid). Positive moment governs - Centered rebar used for positive / negative moment capacity.

Negative Moment in Existing Cantilever

Thickness of Slab (Ex): $t_{ex} := 6.5in$

Point location of resultant additional slab thickness: $a := Unsupp_L - \left(\frac{4ft + 1in}{2}\right) = 7.633\,ft$ $b := Unsupp_L - a = 2.042\,ft$

$$M2_{DC.neg} := \frac{w_{DC} \times Unsupp_L^2}{8} + \frac{\left[\left(t_{ex} - t\right) \times \gamma_c \times (4ft + 1in)\right] \times a \times b}{2 \times Unsupp_L^2} \times \left(a + Unsupp_L\right) = 794.5 \times \frac{lb \times ft}{ft}$$

$$M2_{LL.neg} := \frac{w_{LL} \times Unsupp_L^2}{8} = 877.5 \times \frac{lb \times ft}{ft}$$

Shear transfered from new slab to existing cantilever

$Reaction_{DC} := \frac{5}{8} \times w_{DC} \times Addl_L = 180.551 \times plf$

$Reaction_{LL} := \frac{5}{8} \times w_{LL} \times Addl_L = 262.091 \times plf$

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Design New Bonded Reinforcing for Maximum Positive/Negative Moment (Core drilled hole with roughened surface BDM 5.5.4)

$E_s := 29000ksi$ $f'_c := 5ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.8$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.4 \times in^2$ $clear_{spacing} = 5.438 \times in$

Design Moment: $M_{des} := (1.25 \times M2_{DC.pos} + 1.75 \times M2_{LL.pos}) \times w_{mem} = 1.289 \, ft \times kips$

Service Moment: $M_{service} := (M2_{DC.pos} + M2_{LL.pos}) \times w_{mem} = 0.834 \, ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 1.75 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.204 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.174 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.471 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 1.289 \times kip \times ft$

$\phi M_n = 2.726 \times kip \, ft$

$\frac{\phi M_n}{M_{des}} = 211.6 \times \%$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 827.3 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 2.206 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1 \, ft$ x $h_{mem} = 0.333 \, ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.648 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$\boxed{w_c := 0.150kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 4287 \times ksi$$

$$n := \frac{E_s}{E_c} = 6.765$$

<u>Determine strain compatible neutral axis</u>

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[Quadratic_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 0.691 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.52 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 6.583 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 16.457 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\boxed{\gamma_e := 0.75}$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 2 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.429$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(ksi)(in) - 2 \times d_c = 9.136 \times in}$$

$spacing \times (1 + bundles) = 6 \times in$

$\boxed{Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$\boxed{V_u := \left[\frac{5}{8} \times \left(1.25 \times w_{DC} + 1.75 \times w_{LL}\right) \times Unsupp_L\right] \times w_{mem} = 1.184 \times kips}$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$\boxed{Simplified := \text{"yes"}}$

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\,(ksi)} \times w_{mem} \times (0.9 \times d)$

$\phi_v \times V_c = 2.404 \times kips$

$\boxed{Check_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}}$

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Interface Shear Transfer (5.8.4)

$f'_c := 4ksi$ $F_y := 60ksi$ For shear $\phi := 0.9$ (5.5.4.2.1)

Interface between new slab and existing cantilever

Slab on Girder

For cast-in-place concrete slab on clean concrete girder surfaces, intentionally roughened to 0.25in
c = 0.28 ksi   μ = 1.0   K1 = 0.3   K2 = 1.8 ksi (NW) or 1.3 ksi (LW)

For normal weight concrete placed monolithically
c = 0.40 ksi   μ = 1.4   K1 = 0.25   K2 = 1.5 ksi

For normal weight concrete placed monolithically, or against surface intentionally roughened to 0.25in
c = 0.24 ksi   μ = 1.0   K1 = 0.25   K2 = 1.0 ksi

For normal weight concrete against surface intentionally roughened to 0.25in
c = 0.24 ksi   μ = 1.0   K1 = 0.25   K2 = 1.5 ksi

For normal weight concrete against surface not intentionally roughened
c = 0.075 ksi   μ = 0.6   K1 = 0.2   K2 = 0.8 ksi

$c := 0.075ksi$

$\mu := 0.6$

$K_1 := 0.2$

$K_2 := 0.8ksi$

$A_{cv} := 2in = 24 \times \frac{in^2}{ft}$ Area of Concrete Engaged in Shear Transfer (Assuming crack forms to steel centerline)

$A_{vf} := \frac{A_{no.4}}{6in} = 0.4 \times \frac{in^2}{ft}$ Area of Shear Reinforcement Crossing the Shear Plane

$P_c := 0kips$ Permanent net compressive force normal to the shear plane; if force is tensile $P_c = 0.0kips$ (conservative)

$$V_{n'_1} := c \times A_{cv} + \mu \left(A_{vf} \times F_y + P_c\right) = 16.2 \times klf$$

Must be less than:

$$V_{n'_2} := K_1 \times f'_c \times A_{cv} = 19.2 \times klf \qquad V_{n'_3} := K_2 \times A_{cv} = 19.2 \times klf$$

$$V_n := \min\left(V_{n'}\right) = 16.2 \times klf$$

$$\phi \times V_n = 14.58 \times klf$$

Shear Demand:

$$Demand := 1.25 \times Reaction_{DC} + 1.75 \times Reaction_{LL} = 0.684 \times klf$$

$$Check_{C.D}\left(\phi \times V_n, Demand\right) = \text{"SATISFACTORY"}$$

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Check Existing Reinforcing for Maximum Moment $E_s := 29000ksi$ $F_y := 40ksi$

Concrete Strength of Widening: $f'_c := 5ksi$

Bar size (US number): $bar_{no} := 5$ $d_{no_{bar_{no}}} = 0.625 \times in$ $diam_{deform_{bar_{no}}} = 0.688 \times in$ $A_{no_{bar_{no}}} = 0.31 \times in^2$

$clear := 1.5in$ (REF Sheet 42)

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.8$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \frac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6.5in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.62 \times in^2$ $clear_{spacing} = 5.313 \times in$

Design Moment: $M_{des} := (1.25 \times M2_{DC.neg} + 1.75 \times M2_{LL.neg}) \times w_{mem} = 2.529\,ft \times kips$

Service Moment: $M_{service} := (M2_{DC.neg} + M2_{LL.neg}) \times w_{mem} = 1.672\,ft \times kips$

depth to rebar $$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 4.688 \times in$

Rectangular Compression Depth: $$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$ $a_{req} = 0.143 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.183 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.486 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 2.529 \times kip \times ft$

$\phi M_n = 8.267 \times kip\,ft$

$Check_{C.D}(\phi M_n, M_{des}) = "SATISFACTORY"$

$\frac{\phi M_n}{M_{des}} = 326.9 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 827.3 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 84.5 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 5.826 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 5$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.542\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 6.991 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150\text{kcf}$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{\text{kcf}}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (\text{ksi})} = 4287 \times \text{ksi}$$

$$n := \frac{E_s}{E_c} = 6.765$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{\text{ft}}, \frac{\left(A_s \times w_{mem} \times n\right)}{\text{ft}^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{\text{ft}^3}\right] \times 12\text{in}\right] = 1.494 \times \text{in}$$

Moment Arm: $\text{arm} := d - \frac{x}{3} = 4.189 \times \text{in}$

Service Steel Tension: $T := \frac{M_{service}}{\text{arm}} = 4.789 \times \text{kips}$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 7.724 \times \text{ksi}}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := \text{clear} = 1.5 \times \text{in}$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.429$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(\text{ksi})\ (\text{in}) - 2 \times d_c = 44.578 \times \text{in}}$$

$\text{spacing} \times (1 + \text{bundles}) = 6 \times \text{in}$

$\text{Check}_{C.D}\left[s_{max}, \text{spacing} \times (1 + \text{bundles})\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$V_u := \left[\frac{5}{8} \times \left(1.25 \times w_{DC} + 1.75 \times w_{LL}\right) \times \text{Unsupp}_L\right] \times w_{mem} = 1.184 \times \text{kips}$$

Use Simplified Calc Values

Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$\text{Simplified} := \text{"yes"}$

$\beta_w := 2$ $\theta := 45\text{deg}$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (\text{ksi})} \times w_{mem} \times (0.9 \times d)$

$\phi_v \times V_c = 6.439 \times \text{kips}$

$\text{Check}_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

Determine Minimum Embedment to develop new and existing reinforcing

Development Lengths - Existing Bars (AASHTO LRFD)
The basic development length for a #11 bar and smaller is determined by 5.11.2.1.1

$bar_{no} := 5$ $\quad A_b := A_{no_{bar_{no}}} = 0.31 \times in^2$ $\quad d_b := d_{no_{bar_{no}}} = 0.625 \times in$ $\quad F_y := 40ksi$ $\quad f'_c := 5ksi$

$f_{ct} := \text{"NA"}$

$$l_{d'} := \frac{1.25 \times A_b \left(\frac{1}{in^2}\right) \times F_y \left(\frac{1}{ksi}\right)}{\sqrt{f'_c \left(\frac{1}{ksi}\right)}} \bullet (in) \qquad l_{d'} = 6.932 \times in$$

But the development length can not be less than

$$l_{d.min} := 0.4 \times d_b \left(\frac{1}{in}\right) \times F_y \left(\frac{1}{ksi}\right) \bullet (in) \qquad l_{d.min} = 10 \times in$$

or 12 inches

Adjustment factors (LRFD 5.11.2.1.2 & 3):

Bars coated with epoxy with cover less than 3db or clear spacing between bars less than 6 db:

Top bars placed over 12 inches of concrete coated with epoxy with cover less than 3db or clear spacing between bars less than 6 db:

All other epoxy bar cases: $\boxed{1.2 \times l_{d'} = 8.318 \times in}$

Reinforcement being developed in the length under consideration is spaced laterally at least 6 inches on center with at least 3 inches clear cover measured in the direction of spacing: $\boxed{0.8 \times l_{d'} = 5.545 \times in}$

Lap Splice (LRFD 5.11.5.3.1)

$\boxed{\max\left(1.3 \times 0.8 \times 1.2 \times l_{d'}, 12in\right) = 12 \times in}$

Lap new bar with existing steel a minimum of 12" - Assuming 2" cover to bar - use 14"

Existing steel is spaced at 6" on center, new steel therefore can not be located more than 6" from existing steel

Min embedment for new #4 bar is 8" per WSDOT BDM 5.5-6
Increasing by 1.3 for lap splice results in 10.4". Provide 12" embedment.

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Load to Edge Beam

Dead Load of Slab

$$R2_{DC.Slab} := \frac{w_{DC} \times Unsupp_L}{2} = 249.9 \times plf$$

Conservative to assume simple tributary distribution of load to design edge beam

Dead Load of 54 inch Bicycle Railing:

$$R2_{DC.Railing} := 36.6plf$$

Live Load Reaction

$$R2_{LL} := \frac{w_{LL} \times Unsupp_L}{2} = 362.8 \times plf$$

Edge Beam Length = 18ft 0in max (REF SHT 35)

$$L_{edge} := 18ft$$

Edge beam is designed as PIN-FIX
Requiring a full penetration butt weld
(due to deflection criteria issues):

$$V2_{edge} := 1.25 \times \left(R2_{DC.Slab} + R2_{DC.Railing}\right) + 1.75 \times R2_{LL} = 993.055 \times plf$$

$$M2_{edge} := \frac{\left[1.25 \times \left(R2_{DC.Slab} + R2_{DC.Railing}\right) + 1.75 \times R2_{LL}\right] \times L_{edge}^{\ 2}}{8} = 40.2\ ft \times kips$$

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit (AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges (AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges (AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange (AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-12) may be used for the development of AASHTO capacities.

HSS 8x6x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 41.4kip \times ft$$

$$\varphi M_{n.AASHTO.HSS8x6x3} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 46.0 \times kip \times ft$$

$$\frac{M2_{edge}}{\varphi M_{n.AASHTO.HSS8x6x3}} = 0.874$$

$$Check_{C.D}\left(\varphi M_{n.AASHTO.HSS8x6x3}, M2_{edge}\right) = \text{"SATISFACTORY"}$$

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Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

"For square and rectangular HSS, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side of the area of both webs shall be considered effective in resisting the shear" (6.12.1.2.3b)

$V_n = V_{cr} = C \times V_p$

C = if $\frac{D}{t_w} \le 1.12 \times \sqrt{\frac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := 7\frac{3}{16} in$ $\quad E := 29000ksi$ $\quad F_{yw} := 46ksi$

$t_w := \frac{3}{16} in$

$\frac{D}{t_w} = 38.3$ $\quad 1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}} = 62.9$

$C := if\left[\frac{D}{t_w} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$

$V_p := 0.58 \times F_{yw} \times D \times t_w = 36 \times kips$

$\varphi Vn := \varphi_v \times V_p = 36 \times kips$

PIN - FIX

$V_u := \frac{5}{8} \times V2_{edge} \times L_{edge} = 11.172\, kips$

$Check_{C.D}(\varphi Vn, V_u) = \text{"SATISFACTORY"}$

Edge Beam Load transfers to New Channel truss

Distance from Edge beam to face of existing box girder: $L_{Horiz.truss} := Unsupp_L - \frac{6in}{2} = 9.425\,ft$

Depth of Truss:
(2ft Min) $L_{Vert.truss} := 2ft$

Length of Diagonal Compression Strut: $L_{Diag.truss} := \sqrt{L_{Horiz.truss}^2 + L_{Vert.truss}^2} = 9.634\,ft$

Vertical Reaction at Existing cantilevered beam: $R_{edge} := 2 \times V2_{edge} \times \frac{5}{8} \times L_{edge} = 22.344 \times kips$

Compression in Compression Strut: $Compression_{factored} := R_{edge} \times \frac{L_{Diag.truss}}{L_{Vert.truss}} = 107.635 \times kips$

Tension in Tension Strut: $Tension_{factored} := Compression_{factored} \times \frac{L_{Horiz.truss}}{L_{Diag.truss}} = 105.29 \times kips$

Non-Composite Member - Nominal Compressive Resistance (AASHTO 6.9.4.1)
The nominal compressive resistance shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling and flecural-torsional buckling as follows:

Doubly Symmetric members:
Flexural buckling shall be applicable.
Torsional buckling shall also be applicable for open-section members in which the effective torsional unbraced length is larger than the effective leateral unbraced length.

Singly Symmetric members
Flexural buckling shall be applicable.
Flexural-torsional buckling shall also be applicable for open-section members

Unsymmetric members
Only Flexural-torsional buckling shall be applicable for open-section members, except that for single-angle members designed according to the provisions of Article 6.9.4.4, only flexural buckling shall be applicable.
Only flexural buckling shall be applicable for closed-section members.

Eqs. 6.9.4.1.1-1 and 6.9.4.1.1-2 are equivalent to the equations given in AISC (2005) for computing the nominal compressive resistance. (AASHTO C6.9.4.1.1)

if $\frac{P_e}{P_o} \geq 0.44$, then: $P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \times P_o$ Eqn. 6.9.4.1.1-1

if $\frac{P_e}{P_o} < 0.44$, then: $P_n = 0.877 \times P_e$ Eqn. 6.9.4.1.1-2

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Tables in chapter 4) may be used for the development of AASHTO capacities.

Effective length: $K := 1.0$ $L_{Diag.truss} = 9.634\,ft$

$K \times L_{Diag.truss} = 9.634\,ft$

HSS 8x4x3/16 ~ Fy=46ksi ~ φ = 0.90:

$Length_{tabulated} := \begin{pmatrix} 9ft \\ 10ft \end{pmatrix}$ $Capacity_{tabulated} := \begin{pmatrix} 122kips \\ 116kips \end{pmatrix}$

$\varphi P_n := linterp\left(Length_{tabulated}, Capacity_{tabulated}, K \times L_{Diag.truss}\right) = 118.193 \times kips$

$\frac{Compression_{factored}}{\varphi P_n} = 0.911$

$Check_{C.D}\left(\varphi P_n, Compression_{factored}\right) = \text{"SATISFACTORY"}$

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Tensile Resistance - AASHTO LRFD (6.8.2.1)

The factored tensile resistance shall be taken as the lesser of gross yielding and net section fracture

Design Tension Load: $Tension_{factored} = 1.053 \times 10^5\,lb$ $\varphi_y := 0.95$ $\varphi_u := 0.80$

Steel Properties $F_y := 46ksi$ $F_u := 58ksi$

Gross cross section area:
2 C6x8.2 $A_g := 2 \times 2.39in^2$

Size of holes in member: $d_{holes} := 2 \times \left(\frac{3}{4}in + \frac{1}{16}in\right) = 1.625 \times in$

Thickness of penetrated part: $t := 0.20in$

Nominal section area: $A_n := A_g - d_{holes} \times t = 4.455 \times in^2$

Fracture resistance reduction factor: $R_p := 0.9$
0.9 for holes punched full size
1.0 for holes drilled full size or sub-punched and reamed full size

Shear Lag Reduction factor: $U = 1 - \frac{x_{bar}}{L}$ All tension members, except plates, and HSS, where the tension load is transmitted to some but not all of the cross sectional elements.

$x_{bar} := 0.512in$ Connection Eccentricity

$L := 12in$ Length of Connection

$U := 1 - \frac{x_{bar}}{L} = 0.957$

$P_{r_1} := \varphi_y \times F_y \times A_g = 208.886 \times kips$

$P_{r_2} := \varphi_u \times F_u \times A_n \times R_p \times U = 178.103 \times kips$

$\varphi P_n := \min(P_r) = 178.103 \times kips$

$\frac{Tension_{factored}}{\varphi P_n} = 0.591$

$Check_{C.D}(\varphi P_n, Tension_{factored}) = $ "SATISFACTORY"

Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): A325 := "yes" A490 := "No"

ASTM = "A325"

Nominal Bolt Diameter: $d := \frac{3}{4} in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.44 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$$F_{ub} := \begin{array}{|l} FUB \leftarrow 0ksi \\ FUB \leftarrow 150ksi \ \text{ if } \ ASTM = \text{"A490"} \\ \text{if } \ ASTM = \text{"A325"} \\ \quad \begin{array}{|l} FUB \leftarrow 120ksi \ \text{ if } \ d \le 1.0in \\ FUB \leftarrow 105ksi \ \text{ if } \ d > 1.0in \end{array} \end{array}$$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 2$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 40.3 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9)

For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter

Thickness of the connected material: $t := 0.20in$

Tensile strength of the connected material specified in Table 6.4.1-1:

Grade 36: $F_u := 58ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 20.9 \times kips$

Strength Resistance minimum of bolt shear and bearing

$R_n := \min\left(R_{n.shear}, R_{n.holes}\right) = 20.88 \times kips$

$\varphi := 0.80$ For A325 and A490 Bolts in Shear and Tension

$$\frac{Tension_{factored}}{\varphi \times R_n} = 6.303$$

Provide 7 - 3/4" anchors through the existing concrete cantilever

Check Structure for Deflection

Concrete Slab deflection (PIN-FIX - Conservatively use uniform 4" slab thickness)

$$\delta_c := \frac{1}{185} \times \frac{w_{LL} \times (10ft)^4}{\left[E_c \times \frac{(4in)^3}{12}\right]} = 0.026 \times in$$

$$\frac{10ft}{\delta_c} = 4.7 \times 10^3$$

Greater than 500 okay
REF AASHTO LRFD Guide Specifications for Pedestrian Bridges
Section 5

Edge Beam deflection (PIN-FIX)

$$I_{HSS8x6x3} := 43.7in^4$$

$$\delta_s := \frac{1}{185} \times \frac{R2_{LL} \times \left(L_{edge}\right)^4}{\left(E_s \times I_{HSS8x6x3}\right)} = 0.281 \times in$$

Concrete slab max deflection occurs at slab midspan. Edge beam deflection occurs at the edge beam. Because edge beam deflection exceeds the slab deflection, the edge beam governs the maximum deflection.

$$\frac{L_{edge}}{\left(\delta_s\right)} = 769.497$$

Greater than 500 okay
REF AASHTO LRFD Guide Specifications for Pedestrian Bridges
Section 5

Additional dead load to the structure from walkway widening.

Dead Load of Additional Slab: $plf_{slab} := w_{DC} \times Addl_L = 288.882 \times plf$

Dead Load of Railing: $plf_{railing} := R2_{DC.Railing} = 36.6 \times plf$

Dead Load of Edge Beam: $plf_{edge.beam} := 17.10plf$

Dead Load of truss:

$$plf_{truss} := \frac{L_{Diag.truss} \times 14.54plf + 2 \times \left(L_{Horiz.truss} - 1ft\right) \times 8.2plf}{L_{edge}} = 15.458 \frac{lb}{ft}$$

Dead Load of New Traffic Barrier:

$$plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \frac{lb}{ft}$$

Total weight of new work:

$$plf_{new} := plf_{slab} + plf_{railing} + plf_{edge.beam} + plf_{truss} + plf_{barrier} = 745.3 \times plf$$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedistal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 58): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedistal weight:

$$plf_{pedistal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{L_{edge}} = 39.9 \times plf$$

Dead Load of Existing intermediate Conc Rail:

$$plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{L_{edge}} = 127 \times plf$$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 59)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{L_{edge}} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{L_{edge}}\right]$$

$$plf_{metal.rail} = 16.6 \times plf$$

Dead Load of Existing Edge Beam:

$$plf_{ex.edge.beam} := \frac{(1ft + 1.5in) \times 6in \times \left[L_{edge} - (1ft + 4in)\right] \times \gamma_{c.ex}}{L_{edge}} = 78.1 \times plf$$

Dead Load of Existing Curb:
(REF SHT 56)

$$plf_{curb} := 9.5in \times \frac{(9.5in + 8in)}{2} \times \gamma_{c.ex} = 86.589 \times plf$$

Total weight of removals:

$$plf_{rem} := plf_{pedistal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{curb} = 348.3 \times plf$$

Total increased weight: $\boxed{plf_{increase} := plf_{new} - plf_{rem} = 397.0 \times plf}$

Weight of Segment 2 calculated for seismic analysis: $\boxed{Segment2_{weight} := 9000kips}$

Length of Segment 2: $Segment2_{length} := 336.17ft$

Average Per foot weight of Segment 2: $\dfrac{Segment2_{weight}}{Segment2_{length}} = 2.677 \times 10^{4} \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment2_{weight}}{Segment2_{length}}} = 1.483 \times \%$ Less than 10% OKAY

Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: (REF SHT E-6/91/4 : 782-95) $H_{mast} := 30ft + (3ft + 7in) = 33.583\,ft$

Length of Luminaire Mast arm: (12' max REF WSDOT STD J-28.10-01) $L_{arm} := 12ft$

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d \qquad \text{(psf)}$$

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head (REF WSDOT BDM 10.1(B)) $A := 3.3ft^2$

$$V_{wind.lum} := P_z \times A = 44.876\,lb$$

Height to point of connection: $H := H_{mast} = 33.583\,ft$

$$M_{wind.lum} := V_{wind.lum} \times H = 1.507\,ft \times kips$$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

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Wind on Mast Arm

$$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_z := K_{z.eq}\left(H_{arm}, 900ft, 9.5\right) = 1.151$$

$$C_{d.cylinder}(V, d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \dfrac{V}{mph} \times \dfrac{d}{ft} \\ CD \leftarrow \dfrac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \;\; \text{if} \;\; \omega \le 39 \\ CD \leftarrow 0.45 \;\; \text{if} \;\; \omega \ge 78 \\ CD \end{array}$$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Cylinders (3in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 3in) = 1.1$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$$V_{wind.arm} := P_z \times A = 89.752\,lb$$

Height to point of connection: $H := H_{mast} + 4ft = 37.583\,ft$

$$M_{wind.arm} := V_{wind.arm} \times H = 3.373\,ft \times kips$$

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member		Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		
Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37+1.08r-\frac{C_vVd}{19.8}-\frac{C_vVdr}{4.94}$ (SI) $1.37+1.08r-\frac{C_vVd}{145}-\frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55+\frac{(10.66-C_vVd)}{9.67}$ (SI) $0.55+\frac{(78.2-C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$$K_z := K_{z.eq}\left(H_{pole}, 900ft, 9.5\right) = 1.079$$

Cylinders (8in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 8in) = 0.553$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$$V_{wind.arm} := P_z \times A = 315.609\,lb$$

Height to point of connection: $H := \dfrac{H_{mast}}{2} + 4ft = 20.792\,ft$

$$M_{wind.pole} := V_{wind.arm} \times H = 6.562\,ft \times kips$$

Total Moment from wind (omnidirectional):

$$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 11.442\,ft \times kips$$

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Dead Loads

Distance from Pole location to Point of connection: $Offset := \frac{6in}{2} + 2.5in + 3in + 6in = 1.208\,ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B)) $W_{lum} := 60lb$

$$M_{DC.lum} := W_{lum} \times (L_{arm} - Offset) = 0.647\,ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$$

$$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.221\,ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\,lb$$

$$M_{DC.pole} := W_{pole} \times (-Offset) = -0.415\,ft \times kips$$

Total Moment from Dead Load:

$$M_{DC} := M_{DC.lum} + M_{DC.arm} + M_{DC.pole} = 0.453\,ft \times kips$$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$$

$$M_{Ice.lum} := Ice_{lum} \times (L_{arm} - Offset) = 0.641\,ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$$

$$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.135\,ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$Ice_{pole} := \pi \times 8in \times 3psf \times H_{pole} = 294.001\,lb$$

$$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -0.355\,ft \times kips$$

Total Moment from Dead Load:

$$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = 0.421\,ft \times kips$$

Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left|M_{DC}\right| + M_{wind} \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.453 \\ 9.056 \\ 4.959 \end{pmatrix} ft \times kips$$

Governing Load $\quad M_{design} := \max\left(M_{design'}\right) = 9.056\, ft \times kips$

Light pole attaches to new concrete block. Block is attached to new structure through slab reinforcing and headed anchor studs.

Tension / Compression moment couple:

$$TC := \frac{M_{design}}{10in} = 10.867 \times kips$$

$$\frac{\left(3 \times A_{no.4}\right) \times 60ksi}{1.67} = 21.557 \times kips$$

Tension couple would acceptably transfer load to slab steel.

Required diameter of anchor

$$A_{req} := \frac{TC \times 1.67}{3 \times 36ksi} = 0.168 \times in^2$$

$$diameter := \sqrt{4 \times \frac{A_{req}}{\pi}} = 0.463 \times in$$

Tension couple would acceptably transfer load to headed anchor studs

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BALLARD - 10FT - SEGMENTS 3 & 5 - TYPICAL

$\gamma_c := 155\text{pcf}$

New construction

Existing Edge of curb to edge of deck distance = 13in (REF SHT 31)

Distance from back of curb to face of existing concrete barrier / 6" edge beam = 4ft 0in

Width of new Traffic Barrier: $width_{barrier} := (36in + 6in) \times \frac{4}{21} + 8in = 16 \times in$

Walkway will be supported by a new edge beam supported by a new extension of the existing transverse girders (Solution is bounded as PIN-PIN for maximum positive moment and PIN-FIX for maximum negative moment)

Thickness of Slab: $t := 4in$ $\quad w_{DC.slab} := t \times \gamma_c = 51.7 \times psf$

$w_{LL} := 75psf$

Dead Load of 54 inch Bicycle Railing:

$w_{DC.Railing} := 36.6plf$

Slab positive design Moment (PIN-PIN)

$$M35_{pos.slab} := 1.25 \times \left[\frac{w_{DC.slab} \times (10ft)^2}{8}\right] + 1.75 \times \left[\frac{w_{LL} \times (10ft)^2}{8}\right] = 2.4 \times 10^3 \times \frac{lb \times ft}{ft}$$

Slab service Moment

$$M35_{pos.service.slab} := \frac{w_{DC.slab} \times (10ft)^2}{8} + \frac{w_{LL} \times (10ft)^2}{8} = 1.6 \times 10^3 \times \frac{lb \times ft}{ft}$$

Slab negative design Moment (PIN-FIX)

$$M35_{neg.slab} := 1.25 \times \left[\frac{w_{DC.slab} \times (10ft)^2}{8}\right] + 1.75 \times \left[\frac{w_{LL} \times (10ft)^2}{8}\right] = 2.4 \times 10^3 \times \frac{lb \times ft}{ft}$$

Slab service Moment

$$M35_{neg.service.slab} := \frac{w_{DC.slab} \times (10ft)^2}{8} + \frac{w_{LL} \times (10ft)^2}{8} = 1.6 \times 10^3 \times \frac{lb \times ft}{ft}$$

Positive and Negative moments are equal. Slab shall be designed as symmetric

Design Slab Reinforcing for Maximum Positive/Negative Moment $E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in - d_{no_{bar_{no}}} \times 0.5 = 1.75 \times in$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 3$ $spacing := \frac{w_{mem}}{n_{bars}} = 4 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.6 \times in^2$ $clear_{spacing} = 3.437 \times in$

Design Moment: $M_{des} := M35_{pos.slab} \times w_{mem} = 2.448\ ft \times kips$

Service Moment: $M_{service} := M35_{pos.service.slab} \times w_{mem} = 1.583\ ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 2 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.451 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.307 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.882 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$Check_{C.D}(\phi M_n, M_{des}) = "SATISFACTORY"$

$M_{des} = 2.448 \times kip \times ft$

$\phi M_n = 4.209 \times kip\ ft$

$\frac{\phi M_n}{M_{des}} = 171.9 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 3$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 3$)

spaced at $spacing = 4 \times in$

$w_{mem} = 1\ ft$ x $h_{mem} = 0.333\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 0.909 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.697 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 11.195 \times kips$

Stress in the steel at service limit state $f_s := \frac{T}{A_s \times w_{mem}} = 18.659 \times ksi$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.75 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.111$$

$$s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(ksi)\ (in) - 2 \times d_c = 9.828 \times in$$

$spacing \times (1 + bundles) = 4 \times in$

$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$$V_u := \left[1.25 \times \left(w_{DC.slab} \times \frac{10ft}{2}\right) + 1.75 \times \left(w_{LL} \times \frac{10ft}{2}\right)\right] \times w_{mem} = 0.979 \times kips$$

$\phi_v := 0.9$

Use Simplified Calc Values

Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

Simplified := "yes"

$\beta_w := 2$ $\theta := 45deg$

$$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$$

$\phi_v \times V_c = 2.457 \times kips$

$Check_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

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Load to Edge Beam

Dead Load Reaction (PIN-PIN)

$$R_{edge.beam.DC} := w_{DC.Railing} + w_{DC.slab} \times \left(\frac{10ft}{2} + 6in\right) = 320.767 \times plf$$

Live Load Reaction

$$R_{edge.beam.LL} := w_{LL} \times \frac{10ft}{2} = 375 \times plf$$

Edge Beam Length:
(REF SHT 31)

$$L_{edge} := 10ft$$

New edge beam is PIN-FIX:
(Assuming pin at expansion joint)

$$V35_{edge} := 1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL} = 1.057 \times 10^{3} \times plf$$

$$M35_{edge} := \frac{\left(1.25 \times R_{edge.beam.DC} + 1.75 \times R_{edge.beam.LL}\right) \times L_{edge}^{\ 2}}{8} = 13.2\,ft \times kips$$

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit
(AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges
(AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges
(AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange
(AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 6x6x2/16 ~ Fy=46ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 16.2kip \times ft$$

$$\varphi M_{n.AASHTO.HSS6x6x2} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 18.0 \times kip \times ft$$

$$\frac{M35_{edge}}{\varphi M_{n.AASHTO.HSS6x6x2}} = 0.734$$

$$Check_{C.D}\left(\varphi M_{n.AASHTO.HSS6x6x2}, M35_{edge}\right) = \text{"SATISFACTORY"}$$

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Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

"For square and rectangular HSS, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side of the area of both webs shall be considered effective in resisting the shear" (6.12.1.2.3b)

$V_n = V_{cr} = C \times V_p$

$C = \text{if } \frac{D}{t_w} \le 1.12 \times \sqrt{\frac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := 5\frac{7}{16}\text{in}$ $\quad E := 29000\text{ksi}$ $\quad F_{yw} := 46\text{ksi}$

$t_w := \frac{1}{8}\text{in}$

$\frac{D}{t_w} = 43.5$ $\qquad 1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}} = 62.9$

$C := \text{if}\left[\frac{D}{t_w} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$

$V_p := 0.58 \times F_{yw} \times D \times t_w = 18.1 \times \text{kips}$

PIN - FIX

$\varphi Vn := \varphi_v \times V_p = 18.1 \times \text{kips}$ $\qquad V_u := \frac{5}{8} \times V35_{edge} \times L_{edge} = 6.608 \times \text{kips}$

$\text{Check}_{C.D}(\varphi Vn, V_u) = \text{"SATISFACTORY"}$

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New Extension of existing transverse beam

Length of extension: $Length_{ext} := 10ft + 3in + width_{barrier} = 11.583\,ft$

Dead Load reaction to transverse beam extension:

$$R_{beam.ext.DC} := R_{edge.beam.DC} \times L_{edge} = 3.208 \times kips$$

Edge Beam Weight

$$R_{edge.beam} := 9.85plf \times L_{edge} = 0.098 \times kips$$

Live Load reaction to transverse beam extension:

$$R_{bmext.ped} := R_{edge.beam.LL} \times L_{edge} = 3.75 \times kips$$

Beam Extension Self Weight:
(W27x84 to match depth with exsiting structure)

$$w_{beam.ext} := 84plf$$

Service Moment:

$$M_{ext.service} := \left(R_{beam.ext.DC} + R_{edge.beam} + R_{bmext.ped}\right) \times Length_{ext} \ldots = 87.369\,ft \times kips$$
$$+ w_{beam.ext} \times \frac{Length_{ext}^{2}}{2}$$

Factored Moment:

$$M_{ext.factored} := \left(1.25 \times R_{beam.ext.DC} + 1.25 \times R_{edge.beam} + 1.75 \times R_{bmext.ped}\right) \times Length_{ext} \ldots$$
$$+ 1.25 \times w_{beam.ext} \times \frac{Length_{ext}^{2}}{2}$$

$$M_{ext.factored} = 130.93\,ft \times kips$$

Factored Shear

$$V_{ext.factored} := \left(1.25 \times R_{beam.ext.DC} + 1.25 \times R_{edge.beam} + 1.75 \times R_{bmext.ped}\right) \ldots$$
$$+ 1.25 \times w_{beam.ext} \times Length_{ext}$$

$$V_{ext.factored} = 11.911 \times kips$$

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Moment Capacity at strength limit state in Major Axis for Discretely braced flanges in Compression - AASHTO LRFD 6.10.8.1.1

$\varphi_f := 1.00$ for flexure $\qquad E := 29000ksi$

Member information

Top Flange: $b_{tf} := 10in$ $\quad t_{tf} := 0.64in$ $\quad A_{tf} := b_{tf} \times t_{tf} = 6.4 \times in^2$ $\quad I_{tf} := \frac{b_{tf} \times t_{tf}^3}{12} = 0.218 \times in^4$

Web: $h_{web} := 27in$ $\quad t_{web} := 0.46in$ $\quad A_{web} := h_{web} \times t_{web} = 12.42 \times in^2$ $\quad I_{web} := \frac{t_{web} \times h_{web}^3}{12} = 754.515 \times in^4$

Bottom Flange: $b_{bf} := 10in$ $\quad t_{bf} := 0.64in$ $\quad A_{bf} := b_{bf} \times t_{bf} = 6.4 \times in^2$ $\quad I_{bf} := \frac{b_{bf} \times t_{bf}^3}{12} = 0.218 \times in^4$

$$CG := \frac{A_{tf} \times \frac{t_{tf}}{2} + A_{web} \times \left(t_{tf} + \frac{h_{web}}{2}\right) + A_{bf} \times \left(t_{tf} + h_{web} + \frac{t_{bf}}{2}\right)}{A_{tf} + A_{web} + A_{bf}} = 14.14 \times in$$

$$I_{total} := I_{tf} + A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right)^2 + I_{web} + A_{web} \times \left(CG - t_{tf} - \frac{h_{web}}{2}\right)^2 + I_{bf} + A_{bf} \times \left(CG - t_{tf} - h_{web} - \frac{t_{bf}}{2}\right)^2 = 3.2 \times 10^3 \times in^4$$

$$S_x := \frac{I_{total}}{CG} = 226.284 \times in^3 \qquad Z_x := 2 \times \left[A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right) + t_{web} \times \left(CG - t_{tf}\right) \times \frac{\left(CG - t_{tf}\right)}{2}\right] = 260.731 \times in^3$$

(REF SHTS 37 & 41) $\quad F_y := 36ksi$ $\quad L_b := Length_{ext} = 11.583\,ft$

At the strength limit state, the following requirement shall be satisfied: $\quad f_{bu} + \frac{1}{3} \times f_l \le \varphi_f \times F_{nc}$

The local buckling resistance (6.10.8.2.2) of the compression flange shall be taken as

if $\lambda_f \le \lambda_{pf}$ then $F_{nc} = R_b \times R_h \times F_{yc}$ $\quad \lambda_f := \frac{b_{bf}}{2 \times t_{bf}} = 7.8$ $\quad \lambda_{pf} := 0.38 \times \sqrt{\frac{E}{F_y}} = 10.785$

$\text{less}\left(\lambda_f, \lambda_{pf}\right) =$ "Less than or Equal to"

Web load-shedding factor determined as specified in Article 6.10.1.10.2 $\quad R_b = 1.0$ if $\frac{2 \times D_c}{t_w} \le \lambda_{rw}$

Depth of web in compression in elastic range $\quad D_c := h_{web} - \left(CG - t_{tf}\right) + t_{bf} = 14.14 \times in$

Limiting slenderness ratio for a noncompact web:

$\frac{2 \times D_c}{t_{web}} = 61.478$ $\quad \lambda_{rw} := 5.7 \times \sqrt{\frac{E}{F_y}} = 161.779$ $\quad \text{less}\left(\frac{2 \times D_c}{t_{web}}, \lambda_{rw}\right) =$ "Less than or Equal to" $\quad R_b := 1.0$

Hybrid Factor: $\quad R_h = \frac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta}$ $\quad D_n := D_c = 14.14 \times in$ $\quad$ For doubly symmetric sections

$A_{fn} := b_{bf} \times t_{bf} = 6.4 \times in^2$ $\quad \beta := \frac{2 \times D_n \times t_{web}}{A_{fn}} = 2.033$

$\rho := 1.0$ $\quad$ Non Composite with no cover plates on tensile side

$R_h := \frac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta} = 1$ $\quad F_{nc.lb} := R_b \times R_h \times F_y = 36 \times ksi$

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The Lateral Torsional Buckling resistance (6.10.8.2.3) of the compression flange shall be taken as

$L_p = 1.0 \times r_t \times \sqrt{\frac{E}{F_{yc}}}$ Limit of Plastic deformation

$b_{fc} := b_{bf} = 10 \times in$ $t_{fc} := t_{bf} = 0.64 \times in$

$$r_t := \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3} \times \frac{D_c \times t_{web}}{b_{fc} \times t_{fc}}\right)}}$$

$r_t = 0.208\ ft$

$L_p := 1.0 \times r_t \times \sqrt{\frac{E}{F_y}} = 5.901\ ft$

$F_{yr} := 0.7 \times F_y = 25.2 \times ksi$

$L_r := \pi \times r_t \times \sqrt{\frac{E}{F_{yr}}} = 22.158\ ft$

$C_b := 1$ Unbraced Cantilevers or simple spans

$$F_{nc.ltb} := C_b \times \left[1 - \left(1 - \frac{F_{yr}}{R_h \times F_y}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \times F_y = 32.2 \times ksi$$

$F_{nc} := \min\left(F_{nc.lb}, F_{nc.ltb}\right) = 32.2 \times ksi$

The Maximum Allowable Unbraced Length (6.10.1.6-3) is:

$$L_b \le 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{M_{yc}}{M_u}}$$

$$L_{max} := 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{F_y \times S_x}{M_{ext.factored}}} = 16.124\ ft$$

$Check_{C.D}\left(L_{max}, L_b\right) = $ "SATISFACTORY"

No Lateral bending is anticipated at this location $f_l := 0$

$f_{bu} \le \varphi_f \times F_{nc}$ $S_x \times \left(\varphi_f \times F_{nc}\right) = 608 \times kip \times ft$

$M_{ext.factored} = 130.93\ ft \times kips$

$Check_{C.D}\left[S_x \times \left(\varphi_f \times F_{nc}\right), M_{ext.factored}\right] = $ "SATISFACTORY"

$$\frac{M_{ext.factored}}{\left[S_x \times \left(\varphi_f \times F_{nc}\right)\right]} = 0.215$$

Simplified Splice Plate Design

Thickness of Top Flange of W27x84 = 0.64 in
Width of Top Flange = 10 in
Thickness of Web = 0.46 in

Provide 1/2" Flange Splice Plates

Slip Critical Resistence of a single bolt (6.13.2.8)

Number of Slip Planes per bolt $N_s := 2$

Hole Size Factor: $K_h := 1.00$

Surface Condition Factor: $K_s := 0.50$

Minimum Required Bolt Tension: 7/8" diameter A325 bolt $P_t := 39\text{kips}$

$R_{n.slip} := K_h \times K_s \times N_s \times P_t$ $\quad R_{n.slip} = 39 \times \text{kips}$

Table 6.13.2.8-1 Minimum Required Bolt Tension.

Bolt Diameter, in.	Required Tension-P_t (kip)	
	M 164 (A 325)	M 253 (A 490)
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1-1/8	56	80
1-1/4	71	102
1-3/8	85	121
1-1/2	103	148

Table 6.13.2.8-2 Values of K_h.

for standard holes	1.00
for oversize and short-slotted holes	0.85
for long-slotted holes with the slot perpendicular to the direction of the force	0.70
for long-slotted holes with the slot parallel to the direction of the force	0.60

Table 6.13.2.8-3 Values of K_s.

for Class A surface conditions	0.33
for Class B surface conditions	0.50
for Class C surface conditions	0.33

Flange

Bolts required for full yielding of beam flange:

$$\frac{0.64\text{in} \times 10\text{in} \times 36\text{ksi}}{R_{n.slip}} = 5.908$$

Provide 2 lines of 3 - 7/8" diameter bolts

Web

Bolts required for Factored shear at the web splice:

$$\frac{V_{ext.factored}}{R_{n.slip}} = 0.305$$

Provide single column of Bolts @ 3" $\quad \dfrac{27\text{in} - (0.64\text{in} + 3.36\text{in} + 2\text{in}) \times 2}{3\text{in}} = 5$

5 bolts per line

AASHTO LRFD 5th Edition - Block Shear Rupture Resistance (6.13.4)

$\varphi_{bs} := 0.80$

$diam_{bolt} := \frac{7}{8} in$

Existing beam
Governs properties

$F_y := 36ksi$

$F_u := 58ksi$

$t := 0.64in$ **Thickness of Beam Flange**

The resistance to Block Shear Rupture is defined by:

$$R_r = \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) \le \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn})$$

$R_p := 0.90$ Reduction factor for holes punched full size (1.0 for holes drilled full size or subpunched and reamed to size)

$A_{vg} := 2 \times (2in + 2 \times 3in) = 16 \times in$ Gross Shear Area

$A_{vn} := A_{vg} - 2 \times \left[2.5 \times \left(diam_{bolt} + \frac{1}{16} in\right)\right] = 11.313 \times in$ Net shear area (bolts holes taken at nominal diameter+1/16in)

$A_{tg} := 2 \times 2in$ Gross Tension Area

$A_{tn} := A_{tg} - 2 \times 0.5 \times \left(diam_{bolt} + \frac{1}{16} in\right) = 3.063 \times in$ Net tension area (bolts holes taken at nominal diameter+1/16in)

$U_{bs} := 1.0$ 1.0 for uniformly loaded bolts (0.5 for non-uniform load)

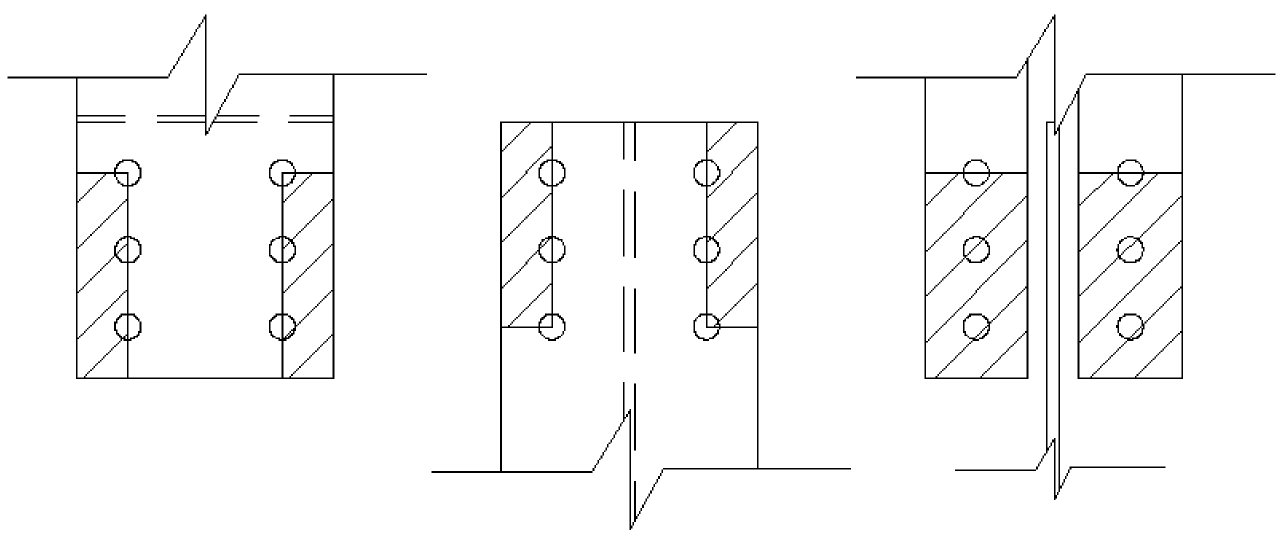
$$R_{r_1} := \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) = 401.888 \times \frac{kips}{in}$$

$$R_{r_2} := \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn}) = 368.428 \times \frac{kips}{in}$$

$$R_r := \min\left(R_{r_1}, R_{r_2}\right) = 368.428 \times \frac{kips}{in}$$

$R_r \times t = 235.8 \times kips$

Full yielding of the flange: $0.64in \times 10in \times 36ksi = 230.4 \times kips$ Bolt pattern is satisfactory



$t := 0.375in$ **Thickness of Outside Splice Plate**

The resistance to Block Shear Rupture is defined by:

$R_r = \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) \le \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn})$

$R_p := 0.90$ Reduction factor for holes punched full size (1.0 for holes drilled full size or subpunched and reamed to size)

$A_{vg} := 2 \times (2in + 2 \times 3in) = 16 \times in$ Gross Shear Area

$A_{vn} := A_{vg} - 2 \times \left[2.5 \times \left(diam_{bolt} + \frac{1}{16}in\right)\right] = 11.313 \times in$ Net shear area (bolts holes taken at nominal diameter+1/16in)

$A_{tg} := 2 \times 2in$ Gross Tension Area

$A_{tn} := A_{tg} - 2 \times 0.5 \times \left(diam_{bolt} + \frac{1}{16}in\right) = 3.063 \times in$ Net tension area (bolts holes taken at nominal diameter+1/16in)

$U_{bs} := 1.0$ 1.0 for uniformly loaded bolts (0.5 for non-uniform load)

$$R_{r_1} := \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) = 401.888 \times \frac{kips}{in}$$

$$R_{r_2} := \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn}) = 368.428 \times \frac{kips}{in}$$

$$R_{r.out} := \min(R_{r_1}, R_{r_2}) = 368.428 \times \frac{kips}{in} \qquad R_r \times t = \begin{pmatrix} 150.7 \\ 138.2 \end{pmatrix} \times kips$$

$t := 0.375in$ **Thickness of Inside Splice Plate**

The resistance to Block Shear Rupture is defined by:

$R_r = \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) \le \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn})$

$R_p := 0.90$ Reduction factor for holes punched full size (1.0 for holes drilled full size or subpunched and reamed to size)

$A_{vg} := 0 \times 2 \times (2in + 2 \times 3in) = 0 \times in$ Gross Shear Area

$A_{vn} := 0 \times A_{vg} - 0 \times 2 \times \left[2.5 \times \left(diam_{bolt} + \frac{1}{16}in\right)\right] = 0 \times in$ Net shear area (bolts holes taken at nominal diameter+1/16in)

$A_{tg} := 2 \times 4in$ Gross Tension Area

$A_{tn} := A_{tg} - 2 \times \left(diam_{bolt} + \frac{1}{16}in\right) = 6.125 \times in$ Net tension area (bolts holes taken at nominal diameter+1/16in)

$U_{bs} := 1.0$ 1.0 for uniformly loaded bolts (0.5 for non-uniform load)

$$R_{r_1} := \varphi_{bs} \times R_p \times (0.58 \times F_u \times A_{vn} + U_{bs} \times F_u \times A_{tn}) = 255.78 \times \frac{kips}{in}$$

$$R_{r_2} := \varphi_{bs} \times R_p \times (0.58 \times F_y \times A_{vg} + U_{bs} \times F_u \times A_{tn}) = 255.78 \times \frac{kips}{in}$$

$$R_{r.in} := \min(R_{r_1}, R_{r_2}) = 255.78 \times \frac{kips}{in} \qquad R_r \times t = \begin{pmatrix} 95.9 \\ 95.9 \end{pmatrix} \times kips$$

$(R_{r.out} + R_{r.in}) \times t = 234.078\,kips$

Full yielding of the flange: $0.64in \times 10in \times 36ksi = 230.4 \times kips$ Bolt Pattern is Satisfactory

Net Section Fracture (AASHTO LRFD 6.10.1.8)

Where bolt holes create a reduced net section in a girder flange, the flange stress (over gross area) is limited to the the value determined in the following calculations.

The failue line connects adjacent bolt holes though straight lines, or zig zags between adjacent holes. Multiple orientations should be checked, with the minimum limiting stress governing the flange.

Width of Flange: $b_{fg} := 10in$

Diameter of Bolt hole: $d_h := \frac{15}{16} in$

Utimate Strength of Steel: $f_u := 58ksi$

Yield Strenght of Steel: $f_y := 36ksi$

Check - Not Staggered

Number of Bolts $num := 2$

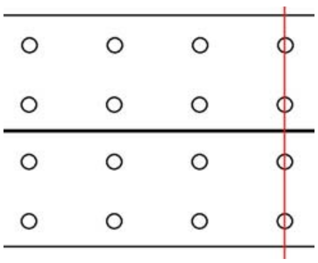
Simple Net Width: $b_{fn} := b_{fg} - num \times d_h = 8.125 \times in$

Pitch of adjacent Bolts in Failure Line: $s := 0in$

Gage of Bolts in Failure Line: $g := 6in$

Addition to Simple Net Width (6.8.3): $w_{add.1} := \frac{s^2}{4 \times g} = 0 \times in$

Limiting Stress: $f_{t.A} := \min\left[0.84 \times \left(\frac{b_{fn}}{b_{fg}}\right) \times f_u, f_y\right] = 36 \times ksi$



Failure Line Example

Greater than stress limit used for moment capacity:

$F_{nc} = 32.225\,ksi$ Satisfactory

Additional dead load to the structure from walkway widening.

Additional Dead Load, Slab and Railing: $plf_{slab.railing} := w_{DC.Railing} + w_{DC.slab} \times 10ft = 553.3 \times plf$

Dead Load of New Traffic Barrier + BP rail: $plf_{barrier} := \dfrac{\left(8in + 32in \times \dfrac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$

Dead Load of New Edge Beam: $\boxed{plf_{edge.beam} := 9.85plf}$

Dead Load of New Girder Extension: (Extension length varies at 10ft increments REF SHT 31)

$$plf_{extension} := w_{beam.ext} \times \frac{\left[Length_{ext} + \left[Length_{ext} - \dfrac{[(52ft + 3in) - (43ft + 8in)]}{2}\right]\right]}{20ft} = 79.3 \times plf$$

Total weight of new work: $plf_{new} := plf_{slab.railing} + plf_{barrier} + plf_{edge.beam} + plf_{extension} = 1.0 \times 10^3 \times plf$

Weight of Removals $\boxed{\gamma_{c.ex} := 150pcf}$

Dead Load of Existing Pedistal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 58 & 37): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedistal weight: $plf_{pedistal} := \dfrac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{20ft} = 35.9 \times plf$

Dead Load of Existing intermediate Conc Rail: $plf_{conc.rail} := \dfrac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{20ft} = 59.3 \times plf$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{20ft} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{20ft}\right]$$

$plf_{metal.rail} = 16.4 \times plf$

Dead Load of Existing Edge Beam:
(REF SHT 38)
$plf_{ex.edge.beam} := 6in \times (1ft + 9in) \times \gamma_{c.ex} = 131.2 \times plf$

Dead Load of Existing sidewalk:
(REF SHT 37)
$plf_{ex.sidewalk} := 3.5in \times 4ft \times \gamma_{c.ex} = 175 \times plf$

Dead Load of Existing Curb:
(REF SHT 38)
$plf_{curb} := \left[18in \times \left(7in + \dfrac{3in}{2}\right) - 3in \times 2in\right] \times \gamma_{c.ex} = 153.125 \times plf$

Total weight of removals:

$$plf_{rem} := plf_{pedistal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{ex.sidewalk} + plf_{curb} = 571 \times plf$$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 458.6 \times plf$

Weight of Segment 3 calculated for seismic analysis: $Segment3_{weight} := 3863kips$

Length of Segment 3: $Segment3_{length} := 419.92ft$

Average Per foot weight of Segment 3: $\dfrac{Segment3_{weight}}{Segment3_{length}} = 9.199 \times 10^3 \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment3_{weight}}{Segment3_{length}}} = 4.985 \times \%$ Less than 10% OKAY

Weight of Segment 5 calculated for seismic analysis: $Segment5_{weight} := 13395kips$

Length of Segment 5: $Segment5_{length} := 1979.51ft - 519.48ft = 1.46 \times 10^3\ ft$

Average Per foot weight of Segment 5: $\dfrac{Segment5_{weight}}{Segment5_{length}} = 9.174 \times 10^3 \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment5_{weight}}{Segment5_{length}}} = 4.999 \times \%$ Less than 10% OKAY

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Check Existing Steel Transverse Beam

Point Load to Edge of Deck

Edge of deck is supported by transverse beams at 10ft on center:
(REF SHT 31)

$L_{deck} := 10ft$

Dead Load Reaction:

$P_{deck.slab.rail} := \left(w_{DC.Railing} + w_{DC.slab} \times \frac{10ft}{2}\right) \times L_{deck} = 2.949 \times kips$

Dead Load of New Traffic Barrier:

$P_{barrier} := plf_{barrier} \times L_{deck} = 3.872 \times kips$

$P_{deck.DC} := P_{deck.slab.rail} + P_{barrier} = 6.822 \times kips$

Live Load Reaction:

$P_{deck.pedLL} := \left(w_{LL} \times \frac{10ft}{2}\right) \times L_{deck} = 3.75 \times kips$

Distributed Loads

Dead Load of existing Slab:
(REF SHT 31)

$w_{ex.slab} := \gamma_{c.ex} \times \left[8\frac{3}{4}in \times L_{deck} + \left(5\frac{7}{8}in - 2\frac{7}{8}in\right) \times 10in\right] = 1.125 \times klf$

Dead Load of existing transverse girder:

$w_{ex.girder} := 91plf$

Longitudinal Support Girder 14ft from centerline of bridge (REF SHT 32)
Inside of Curb is 21ft from centerline of bridge (REF SHT 31)

$Curb_{to.girder} := 21ft - 14ft = 7\,ft$

$Curb_{to.girder} - 2ft = 5\,ft$

Live Load (HS20):
(Girders spaced at 10ft, axles spaced at 14ft by lever rule only consider one axle ~ AASHTO LRFD 4.6.2.2.2f)

$HS20_{wheel} := \frac{32kips}{2} = 16 \times kips$

Only one wheel will load the girder

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Bending at girder support

Service Moment:

$$M_{at.edge} := \left(R_{beam.ext.DC} + R_{edge.beam} + R_{bmext.ped}\right) \times \left(Length_{ext} + width_{barrier} + Curb_{to.girder}\right) \dots = 154.279\,ft \times kips$$
$$+ w_{beam.ext} \times Length_{ext} \times \left(\frac{Length_{ext}}{2} + width_{barrier} + Curb_{to.girder}\right)$$

$$M_{at.EOD} := \left(P_{deck.DC} + P_{deck.pedLL}\right) \times \left(width_{barrier} + Curb_{to.girder}\right) = 88.097\,ft \times kips$$

$$M_{distr} := \frac{w_{ex.slab} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2} + \frac{w_{ex.girder} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2} = 42.222\,ft \times kips$$

$$M_{veh} := HS20_{wheel} \times \left(Curb_{to.girder} - 2ft\right) = 80\,ft \times kips$$

No dynamic allowance is required on vehicles when combined with pedestrian Load (AASHTO 3.6.1.6)

$$M_{Girder.serv} := M_{at.edge} + M_{at.EOD} + M_{distr} + M_{veh} = 364.598\,ft \times kips$$

Factored Moment:

$$M_{at.edge} := \left[1.25 \times \left(R_{beam.ext.DC} + R_{edge.beam}\right) + 1.75 \times R_{bmext.ped}\right] \times \left(Length_{ext} + width_{barrier} + Curb_{to.girder}\right) \dots$$
$$+ 1.25 \times w_{beam.ext} \times Length_{ext} \times \left(\frac{Length_{ext}}{2} + width_{barrier} + Curb_{to.girder}\right)$$

$$M_{at.edge} = 230.192\,ft \times kips$$

$$M_{at.EOD} := \left(1.25 \times P_{deck.DC} + 1.75 \times P_{deck.pedLL}\right) \times \left(width_{barrier} + Curb_{to.girder}\right) = 125.746\,ft \times kips$$

$$M_{distr} := 1.25 \times \left[\frac{w_{ex.slab} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2} + \frac{w_{ex.girder} \times \left(width_{barrier} + Curb_{to.girder}\right)^2}{2}\right] = 52.778\,ft \times kips$$

$$M_{veh} := 1.75 \times HS20_{wheel} \times \left(Curb_{to.girder} - 2ft\right) = 140\,ft \times kips$$

$$M_{Girder.factored} := M_{at.edge} + M_{at.EOD} + M_{distr} + M_{veh} = 548.716\,ft \times kips$$

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Moment Capacity at strength limit state in Major Axis for Discretely braced flanges in Compression - AASHTO LRFD 6.10.8.1.1

$\varphi_f := 1.00$ for flexure $\quad E := 29000ksi$

Member information

Top Flange: $b_{tf} := 10in$ $\quad t_{tf} := 0.745in$ $\quad A_{tf} := b_{tf} \times t_{tf} = 7.45 \times in^2$ $\quad I_{tf} := \dfrac{b_{tf} \times t_{tf}^3}{12} = 0.345 \times in^4$

Web: $h_{web} := 27in$ $\quad t_{web} := 0.46in$ $\quad A_{web} := h_{web} \times t_{web} = 12.42 \times in^2$ $\quad I_{web} := \dfrac{t_{web} \times h_{web}^3}{12} = 754.515 \times in^4$

Bottom Flange: $b_{bf} := 10in$ $\quad t_{bf} := 0.745in$ $\quad A_{bf} := b_{bf} \times t_{bf} = 7.45 \times in^2$ $\quad I_{bf} := \dfrac{b_{bf} \times t_{bf}^3}{12} = 0.345 \times in^4$

Dimensions shown are from the W27x94 shape. The 91# beam weight shown is silicone steel, which is slightly less dense than carbon steel (~97.5%)

$$CG := \frac{A_{tf} \times \frac{t_{tf}}{2} + A_{web} \times \left(t_{tf} + \frac{h_{web}}{2}\right) + A_{bf} \times \left(t_{tf} + h_{web} + \frac{t_{bf}}{2}\right)}{A_{tf} + A_{web} + A_{bf}} = 14.245 \times in$$

$$I_{total} := I_{tf} + A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right)^2 + I_{web} + A_{web} \times \left(CG - t_{tf} - \frac{h_{web}}{2}\right)^2 + I_{bf} + A_{bf} \times \left(CG - t_{tf} - h_{web} - \frac{t_{bf}}{2}\right)^2 = 3.623 \times 10^3 \times in^4$$

$$S_x := \frac{I_{total}}{CG} = 254.311 \times in^3 \qquad Z_x := 2 \times \left[A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right) + t_{web} \times \left(CG - t_{tf}\right) \times \frac{\left(CG - t_{tf}\right)}{2}\right] = 290.535 \times in^3$$

$F_y := 45ksi$ $\quad L_b := width_{barrier} + Curb_{to.girder} + 3ft = 11.333\ ft$

At the strength limit state, the following requirement shall be satisfied: $\quad f_{bu} + \frac{1}{3} \times f_l \le \varphi_f \times F_{nc}$

The local buckling resistance (6.10.8.2.2) of the compression flange shall be taken as

if $\lambda_f \le \lambda_{pf}$ then $F_{nc} = R_b \times R_h \times F_{yc}$ $\quad \lambda_f := \dfrac{b_{bf}}{2 \times t_{bf}} = 6.7$ $\quad \lambda_{pf} := 0.38 \times \sqrt{\dfrac{E}{F_y}} = 9.647$

$less(\lambda_f, \lambda_{pf}) =$ "Less than or Equal to"

Web load-shedding factor determined as specified in Article 6.10.1.10.2 $\quad R_b = 1.0$ if $\dfrac{2 \times D_c}{t_w} \le \lambda_{rw}$

Depth of web in compression in elastic range $\quad D_c := h_{web} - \left(CG - t_{tf}\right) + t_{bf} = 14.245 \times in$

Limiting slenderness ratio for a noncompact web:

$\dfrac{2 \times D_c}{t_{web}} = 61.935$ $\quad \lambda_{rw} := 5.7 \times \sqrt{\dfrac{E}{F_y}} = 144.7$ $\quad less\left(\dfrac{2 \times D_c}{t_{web}}, \lambda_{rw}\right) =$ "Less than or Equal to" $\quad R_b := 1.0$

Hybrid Factor: $\quad R_h = \dfrac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta}$ $\quad D_n := D_c = 14.245 \times in$ $\quad$ For doubly symmetric sections

$A_{fn} := b_{bf} \times t_{bf} = 7.45 \times in^2$ $\quad \beta := \dfrac{2 \times D_n \times t_{web}}{A_{fn}} = 1.759$

$\rho := 1.0$

Non Composite with no cover plates on tensile side

$R_h := \dfrac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta} = 1$ $\quad F_{nc.lb} := R_b \times R_h \times F_y = 45 \times ksi$

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The Lateral Torsional Buckling resistance (6.10.8.2.3) of the compression flange shall be taken as

$L_p = 1.0 \times r_t \times \sqrt{\frac{E}{F_{yc}}}$ Limit of Plastic deformation

$b_{fc} := b_{bf} = 10 \times in$ $t_{fc} := t_{bf} = 0.745 \times in$

$$r_t := \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3} \times \frac{D_c \times t_{web}}{b_{fc} \times t_{fc}}\right)}}$$

$r_t = 0.212\, ft$

$L_p := 1.0 \times r_t \times \sqrt{\frac{E}{F_y}} = 5.37\, ft$

$F_{yr} := 0.7 \times F_y = 31.5 \times ksi$

$L_r := \pi \times r_t \times \sqrt{\frac{E}{F_{yr}}} = 20.165\, ft$

$C_b := 1$ Unbraced Cantilevers or simple spans

$$F_{nc.ltb} := C_b \times \left[1 - \left(1 - \frac{F_{yr}}{R_h \times F_y}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \times F_y = 39.6 \times ksi$$

$F_{nc} := \min(F_{nc.lb}, F_{nc.ltb}) = 39.6 \times ksi$

The Maximum Allowable Unbraced Length (6.10.1.6-3) is:

$$L_b \le 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{M_{yc}}{M_u}}$$

$$L_{max} := 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{F_y \times S_x}{M_{Girder.factored}}} = 8.496\, ft$$

$Check_{C.D}(L_{max}, L_b) =$ "Not Satisfactory"

No Lateral bending is anticipated at this location $f_l := 0$

$f_{bu} \le \varphi_f \times F_{nc}$ $S_x \times (\varphi_f \times F_{nc}) = 838 \times kip \times ft$

$M_{Girder.factored} = 548.716\, ft \times kips$

$Check_{C.D}\left[S_x \times (\varphi_f \times F_{nc}), M_{Girder.factored}\right] =$ "SATISFACTORY"

$$\frac{M_{Girder.factored}}{\left[S_x \times (\varphi_f \times F_{nc})\right]} = 0.655$$

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: $H_{mast} := 30ft + (3ft + 7in) = 33.583\,ft$
(REF SHT E-6/91/4 : 782-95)

Length of Luminaire Mast arm: $L_{arm} := 12ft$
(12' max REF WSDOT STD J-28.10-01)

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d \qquad \text{(psf)}$$

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head $A := 3.3ft^2$
(REF WSDOT BDM 10.1(B))

$$V_{wind.lum} := P_z \times A = 44.876\,lb$$

Height to point of connection: $H := H_{mast} + 4ft = 37.583\,ft$

$$M_{wind.lum} := V_{wind.lum} \times H = 1.687\,ft \times kips$$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

Wind on Mast Arm

$$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$$

$$K_z := K_{z.eq}\left(H_{arm}, 900ft, 9.5\right) = 1.151$$

$$C_{d.cylinder}(V, d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \frac{V}{mph} \times \frac{d}{ft} \\ CD \leftarrow \frac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{ if } \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{ if } \ \omega \ge 78 \\ CD \end{array}$$

Cylinders (3in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 3in) = 1.1$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$$V_{wind.arm} := P_z \times A = 89.752\,lb$$

Height to point of connection: $H := H_{mast} + 4ft = 37.583\,ft$

$$M_{wind.arm} := V_{wind.arm} \times H = 3.373\,ft \times kips$$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$		Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		
Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r - \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$$K_z := K_{z.eq}\left(H_{pole}, 900ft, 9.5\right) = 1.079$$

Cylinders (8in diameter ~ assumed)

$$C_d := C_{d.cylinder}(V, 8in) = 0.553$$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$$V_{wind.pole} := P_z \times A = 315.609\,lb$$

Height to point of connection: $H := \frac{H_{mast}}{2} + 4ft = 20.792\,ft$

$$M_{wind.pole} := V_{wind.pole} \times H = 6.562\,ft \times kips$$

$$V_{wind} := V_{wind.lum} + V_{wind.arm} + V_{wind.pole} = 0.45 \times kips$$

Total Moment from wind (omnidirectional):

$$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 11.622\,ft \times kips$$

Dead Loads

Distance from Pole location to Point of connection: $Offset := 4ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$$W_{lum} := 60lb$$

$$M_{DC.lum} := W_{lum} \times (L_{arm} - Offset) = 0.48\, ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\, lb$$

$$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.092\, ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\, lb$$

$$M_{DC.pole} := W_{pole} \times (-Offset) = -1.374\, ft \times kips$$

Total Moment from Dead Load:

$$M_{DC} := M_{DC.lum} + M_{DC.arm} + M_{DC.pole} = -0.802\, ft \times kips$$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\, lb$$

$$M_{Ice.lum} := Ice_{lum} \times (L_{arm} - Offset) = 0.475\, ft \times kips$$

Weight of Arm:
(Assume 11 gage)

$$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\, lb$$

$$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.057\, ft \times kips$$

Weight of Pole:
(Assume 11 gage)

$$Ice_{pole} := \pi \times 8in \times 3psf \times H_{mast} = 211.01\, lb$$

$$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -0.844\, ft \times kips$$

Total Moment from Dead Load:

$$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -0.312\, ft \times kips$$

Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \frac{1}{100\%} \\ \left(\left|M_{DC}\right| + M_{wind}\right) \times \frac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \frac{M_{wind}}{2}\right) \times \frac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.802 \\ 9.341 \\ 5.207 \end{pmatrix} ft \times kips$$

Governing Load $\quad M_{design} := \max\left(M_{design'}\right) = 9.341\ ft \times kips$

Connect pole to square tube:
Square or Rectangular HSS Bending
For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1) $\quad M_n = M_p = F_y \times Z$

Flange Compact Criteria Buckling Limit (AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1) $\quad \lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$

Flange Local Buckling Limit For Compact Flanges (AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges (AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3) $\quad M_n = F_y \times S_{eff}$

Effective width of compression flange (AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 10x10x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$M_{n.\Omega.AISC} := 42.8kip \times ft$$

$$M_{n.\Omega.AASHTO.HSS10x10x3} := \frac{1.0}{0.9} \times M_{n.\Omega.AISC} = 47.6 \times kip \times ft$$

$$\frac{M_{design}}{M_{n.\Omega.AASHTO.HSS10x10x3}} = 0.196$$

$$Check_{C.D}\left(M_{n.\Omega.AASHTO.HSS10x10x3}, M_{design}\right) = \text{"SATISFACTORY"}$$

BALLARD - 10FT - SEGMENT 6

$\gamma_c := 155pcf$

New construction

Existing overhang = 5ft 4in from inside curb to outside of existing concrete barrier (REF SHT 36)

Distance from outside of existing concrete barrier to centerline of exterior girder = 11.25" (REF SHT 42)

Width of new Traffic Barrier: $width_{barrier} := (32in + 6in) \times \frac{4}{21} + 8in = 15.238 \times in$

Existing Exterior Girder is shown as B36 x 16 1/2 x 230# (REF SHT 52)

Offset from centerline of existing girder to new girder - 3ft

Walkway Length spanning from new traffic barrier to new Girder Centerline: $L_{new.span} := (5ft + 4in) - width_{barrier} - 11.25in + 3ft = 6.126\,ft$

Walkway Length cantilevering from new girder to end of 10 ft walking way $L_{new.canti} := 10ft - L_{new.span} = 3.874\,ft$

Walkway will be supported by new transverse steel tubes, supported on a new exterior girder. The slab will span longitudinally between tubes, while the transverse support will cantilever beyond the new girder and anchor to the new traffic Barrier (FREE - PIN - PIN)

Thickness of Slab: $t := 4in$ $w_{DC.slab} := t \times \gamma_c = 51.7 \times psf$

$w_{LL} := 75psf$

Slab design

slab span: $slab_{span} := 10ft$

Uniform simple span slab:

$$r_{DC.slab} := \frac{w_{DC.slab} \times slab_{span}^2}{8} = 645.833 \times \frac{lb \times ft}{ft}$$

$$r_{LL.slab} := \frac{w_{LL} \times slab_{span}^2}{8} = 937.5 \times \frac{lb \times ft}{ft}$$

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Design Slab Reinforcing for Maximum Positive Moment

$E_s := 29000ksi$ $\quad f'_c := 4ksi$ $\quad F_y := 60ksi$

Bar size (US number): $bar_{no} := 4$ $\quad d_{no_{bar_{no}}} = 0.5 \times in$ $\quad diam_{deform_{bar_{no}}} = 0.563 \times in$ $\quad A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in - d_{no_{bar_{no}}} \times 0.5 = 1.75 \times in$

Number of layers: $full_{layers} := 1$ $\quad bundled := "no"$ $\quad bundles := yes_{no}(bundled) = 0$ $\quad \beta_1 = 0.85$

Width of member $w_{mem} := 12in$ **Bars per Layer:** $n_{bars} := 3$ $\quad spacing := \frac{w_{mem}}{n_{bars}} = 4 \times in$

Height of member $h_{mem} := 4in$ $\quad A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $\quad A_s \times w_{mem} = 0.6 \times in^2$ $\quad clear_{spacing} = 3.437 \times in$

Design Moment: $M_{des} := (1.25 \times r_{DC.slab} + 1.75 \times r_{LL.slab}) \times w_{mem} = 2.448\,ft \times kips$

Service Moment: $M_{service} := (r_{DC.slab} + r_{LL.slab}) \times w_{mem} = 1.583\,ft \times kips$

depth to rebar $$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 2 \times in$

Rectangular Compression Depth: $$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$$ $a_{req} = 0.451 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $\quad A_{s.req} \times w_{mem} = 0.307 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $\quad a = 0.882 \times in$ $\quad \phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 2.448 \times kip \times ft$

$\phi M_n = 4.209 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = "SATISFACTORY"$

$\frac{\phi M_n}{M_{des}} = 171.9 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $\quad F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $\quad S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $\quad M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 3$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 3$)

spaced at $spacing = 4 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.333\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

$M_{cr.Check} = "SATISFACTORY"$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 0.909 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.697 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 11.195 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 18.659 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.75 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.111$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 9.828 \times in}$$

$spacing \times (1 + bundles) = 4 \times in$

$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$$V_u := \left[1.25 \times \left(w_{DC.slab} \times \frac{10ft}{2}\right) + 1.75 \times \left(w_{LL} \times \frac{10ft}{2}\right)\right] \times w_{mem} = 0.979 \times kips$$

$\boxed{\phi_v := 0.9}$

Use Simplified Calc Values

Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

Simplified := "yes"

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$

$\phi_v \times V_c = 2.457 \times kips$

$\text{Check}_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

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Determine Force on Transverse Supporting Beam

Dead Load of 54 inch Bicycle Railing:
(2.5" STD pipe vert @ 6 ft oc 55" high + 1.5" STD horiz pipes ~ 6 total + 15% for misc components)

$$w_{DC.Railing} := 36.6plf \times slab_{span} = 366.0\,lb$$

FREE - PIN - PIN Reactions (PIN-PIN used to determine edge beam reactions - edge beam bias)

$$R_{barrier.railing} := \frac{w_{DC.Railing} \times \left(L_{new.canti} + 1.5in\right)}{-L_{new.span}} = -238.9\,lb$$

(assume 1.5in to centerline of railing)

$$R_{barrier.slab} := \frac{\left(w_{DC.slab} \times slab_{span}\right) \times \left[\frac{\left(L_{new.canti}\right)^2}{2} - \frac{L_{new.span}^{\;2}}{2}\right]}{-L_{new.span}} = 949.7\,lb$$

$$R_{barrier.DC} := R_{barrier.railing} + R_{barrier.slab} = 710.7\,lb$$

$$R_{edge.beam.DC} := w_{DC.Railing} + w_{DC.slab} \times \left[\left(L_{new.canti}\right) + L_{new.span}\right] \times slab_{span} - R_{barrier.DC} = 4.822 \times 10^3\,lb$$

New HSS Cantilever Negative Moment at Edge Beam (FREE-FIX)

$$M6_{canti.neg} := 1.25 \times \left[w_{DC.Railing} \times \left(L_{new.canti} + 1.5in\right) + w_{DC.slab} \times slab_{span} \times \frac{\left(L_{new.canti}\right)^2}{2}\right] \ldots$$
$$+ 1.75 \times \left(w_{LL} \times slab_{span} \times \frac{L_{new.canti}^{\;2}}{2}\right)$$

$$M6_{canti.neg} = 16.525\,ft \times kips$$

FIX - PIN Negative Bending (Design based on higher of cantilever moment and FIX-PIN moment)

$$M6_{span.neg} := \left(1.25 \times w_{DC.slab} \times slab_{span} + 1.75 \times w_{LL} \times slab_{span}\right) \times \frac{\left(L_{new.span}\right)^2}{8} = 9.2\,ft \times kips$$

$$M6_{neg.design} := \max\left(M6_{canti.neg}, M6_{span.neg}\right) = 16.525\,ft \times kips$$

FREE-PIN-PIN Positive Bending (Unbalanced live load used to maximize live load moment)

$$M6_{pos.design} := \left(1.25 \times w_{DC.slab} \times slab_{span} + 1.75 \times w_{LL} \times slab_{span}\right) \times \frac{\left(L_{new.span}\right)^2}{8} \ldots = 2.5\,ft \times kips$$
$$+ \left[-1.25 \times \left[w_{DC.slab} \times slab_{span} \times \frac{L_{new.canti}^{\;2}}{2} + w_{DC.Railing} \times \left(L_{new.canti} + 1.5in\right)\right]\right]$$

Governing Design: $M6_{design} := \max\left(M6_{neg.design}, M6_{pos.design}\right) = 16.525\,ft \times kips$

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Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit
(AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges
(AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges
(AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange
(AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 6x6x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 27.8 kip \times ft$$

$$\varphi M_{n.AASHTO.HSS6x6x3} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 30.9 \times kip \times ft$$

$$\frac{M6_{design}}{\varphi M_{n.AASHTO.HSS6x6x3}} = 0.535$$

$$Check_{C.D}\left(\varphi M_{n.AASHTO.HSS6x6x3}, M6_{design}\right) = \text{"SATISFACTORY"}$$

Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

"For square and rectangular HSS, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side of the area of both webs shall be considered effective in resisting the shear" (6.12.1.2.3b)

$V_n = V_{cr} = C \times V_p$

$C = \text{if } \dfrac{D}{t_w} \le 1.12 \times \sqrt{\dfrac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := 5\frac{7}{16} in$ $\quad E := 29000ksi$ $\quad F_{yw} := 46ksi$

$t_w := \frac{1}{8} in$

$\dfrac{D}{t_w} = 43.5$ $\qquad 1.12 \times \sqrt{\dfrac{E \times 5.0}{F_{yw}}} = 62.9$

$$C := \text{if}\left[\frac{D}{t_w} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$$

$V_p := 0.58 \times F_{yw} \times D \times t_w = 18.1 \times kips$

$\varphi Vn := \varphi_v \times V_p = 18.1 \times kips$

$V_u := 1.25 \times \left(w_{DC.Railing} + w_{DC.slab} \times slab_{span} \times L_{new.canti}\right) + 1.75 \times \left(w_{LL} \times slab_{span} \times L_{new.canti}\right) = 8.044 \times kips$

$Check_{C.D}\left(\varphi Vn, V_u\right) =$ "SATISFACTORY"

Deflection from live load on Cantilever (FREE-FIX governing moment)

$I_{HSS6x6x3} := 22.3in^4$ $\qquad E := 29000ksi$

$$\delta_{cantilever} := \frac{w_{LL} \times slab_{span} \times L_{new.canti}^{\ 4}}{8 \times E \times I_{HSS6x6x3}} = 0.056 \times in$$

PINNED reaction on cast in anchors (Unbalanced live load condition)
Space anchors at 10" on center

$V_{DC} := R_{barrier.DC} = 0.711 \times kips$

$V_{LL} := w_{LL} \times slab_{span} \times \frac{L_{new.span}}{2} = 2.297 \times kips$

Distance from Anchor centerline to Center of compression block (assume 1" block) $dist_{anchor2compr} := 3in$

Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): A325 := "yes" A490 := "No" ASTM = "A325"

Number of Bolts: $num := 2$

Nominal Bolt Diameter: $d := \frac{5}{8} in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.31 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$F_{ub} :=$ | FUB ← 0ksi
FUB ← 150ksi if ASTM = "A490"
if ASTM = "A325"
  | FUB ← 120ksi if $d \le 1.0in$
  | FUB ← 105ksi if $d > 1.0in$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 1$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 14 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9) *For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter*

Thickness of the connected material: $t := 0.375in$

Tensile strength of the connected material specified in Table 6.4.1-1: Grade 36: $F_u := 58ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 32.6 \times kips$

Strength Resistance minimum of bolt shear and bearing $R_n := \min(R_{n.shear}, R_{n.holes}) = 13.99 \times kips$

$\varphi := 0.80$ For A325 and A490 Bolts in Shear and Tension $\varphi \times R_n = 11.192 \times kips$

$\frac{1.25 \times V_{DC} + 1.75 \times V_{LL}}{num \times (\varphi \times R_n)} = 0.219$

$Check_{C.D}(\varphi \times R_n, 1.25 \times V_{DC} + 1.75 \times V_{LL}) = $ "SATISFACTORY"

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Anchor Bolt Design - ACI 318-08 D.4

*If **post installed anchors** are designed for seismic loads then the following criteria must be satisfied (D3.3)*
D.3.3.2 - Post installed anchors shall be qualified for use in cracked concrete
D.3.3.3 - Concrete failure modes shall have an additional 0.75 reduction factor
D.3.3.4 - Anchors shall be designed to be governed by ductile steel
D.3.3.6 - Alternative to using a ductile steel element, it is permitted to use a concrete failure mode using an additional 0.4 reduction beyond what is specified in D.3.3.3

General:

Number of anchors in tension: $n_t := 2$ Number of anchors in shear: $n_v := 2$

Headed Anchor or hooked anchor $\text{Headed}_{yes.no} := \text{"yes"}$ $\text{Hooked}_{yes.no} := \text{"no"}$

Anchor pattern:

Center to Center distance for outermost Anchors-

$s_1 := 0in$ Longitudinal Direction Number of bolts in line: $N_L := 1$

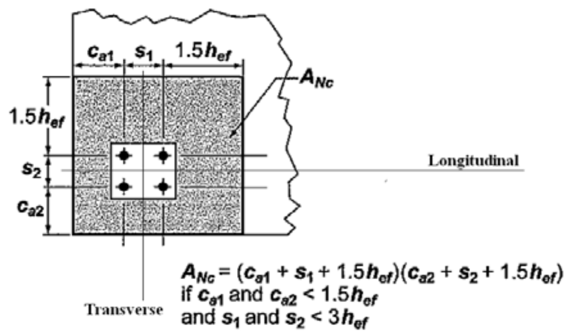
Limiting concrete dimensions (a1 = Positive Load Breakout Side)

$c_{a1} := 4in$ $c_{b1} := 4in$

$s_2 := 10in$ Transverse Direction Number of bolts in line: $N_T := 4$

Limiting concrete dimensions (a2 = Positive Load Breakout Side)

$c_{a2} := 999in$ $c_{b2} := 999in$



Depth of concrete member: $h_a := 6in$

Diameter of anchor bolt: $d_a := 0.625in$

Specify = "Area of Head"

set other length to 0

Area of Head: $A_{brg} := 1in^2$

Hook Length: $e_h := 0$
3da<eh<4.5da

Section area of anchor bolt:

$A_{se.N} := 0.226in^2$ $A_{se.V} := \pi \times \frac{d_a^{\,2}}{4} = 0.307 \times in^2$

Nominal Tensile Stress of Fasteners and Threaded parts (AISC Table 2-5) & F1554 values:

$A_{307} := \text{"no"}$ $A_{325} := \text{"no"}$ $A_{490} := \text{"no"}$ $\text{Specified} := 0$

$\text{F1554}_{G36} := \text{"yes"}$ $\text{F1554}_{G55} := \text{"no"}$ $\text{F1554}_{G105} := \text{"no"}$

$f_{uta} = 58 \times ksi$

Embedment depth of anchor: $h_{ef} := 4in$

Concrete strength: $f'_c := 4ksi$

"shall not exceed 10,000 psi for cast-in anchors, and 8,000 psi for post-installed anchors"

F1554 GR 55 Data

Anchor Diameter in. (mm)	Drill Diameter in. (mm)	Tensile Stress Area
1/2 (12.7)	5/8 (15.9)	0.142
5/8 (15.9)	3/4 (19.1)	0.226
3/4 (19.1)	7/8 (22.2)	0.334
7/8 (22.2)	1 (25.4)	0.462
1 (25.4)	1 1/8 (28.6)	0.606
1 1/8 (28.6)	1 1/4 (31.8)	0.763
1 1/4 (31.8)	1 3/8 (34.9)	0.969
1 3/8 (34.9)	1 1/2 (38.1)	1.155
1 1/2 (38.1)	1 3/4 (44.4)	1.405
1 3/4 (44.4)	2 (50.8)	1.900
2 (50.8)	2 1/4 (57.2)	2.500
2 1/4 (57.2)	2 1/2 (63.5)	3.250
2 1/2 (63.5)	2 3/4 (69.9)	4.000
2 3/4 (69.9)	3 (76.2)	4.930
3 (76.2)	3 1/4 (82.5)	5.970

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Loads

"The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or C9.2" (D.3.2)

Tension

Use Negative Values for Compression Forces

Dead Loads: $D_T := 0\text{kips}$ Wind Loads: $W_T := 0\text{kips}$ Live Loads: $L_T := 0\text{kips}$

$LC_{1T} := \left|1.4 \times (D_T)\right|$ — $LC_{1T} = 0 \times \text{kips}$

$LC_{2T} := 1.25 \times (D_T) + 1.75 \times (L_T)$ <-- INCREASED TO AASHTO STR I — $LC_{2T} = 0 \times \text{kips}$

$LC_{3T} := 1.2 \times D_T + \max(L_T, 0.8 \times W_T)$ — $LC_{3T} = 0 \times \text{kips}$

$LC_{4T} := 1.2 \times D_T + 1.6 \times W_T + L_T$ <-- EXCEEDS TO AASHTO STR III — $LC_{4T} = 0 \times \text{kips}$

$LC_{5T} := 1.25 \times D_T + 1.35 \times L_T + 0.4 \times W_T$ <-- ALTERED TO AASHTO STR V — $LC_{5T} = 0 \times \text{kips}$

$LC_{6T} := 0.9 \times D_T + 1.6 \times W_T$ — $LC_{6T} = 0 \times \text{kips}$

$LC_{7T} := 0.9 \times D_T$ — $LC_{7T} = 0 \times \text{kips}$

Shear - Transverse

Dead Loads: $D_{V.tr} := 0\text{kips}$ Wind Loads: $W_{V.tr} := 0\text{kips}$ Live Loads: $L_{V.tr} := 0\text{kips}$

$LC_{1V.tr} := 1.4 \times (D_{V.tr})$ — $LC_{1V.tr} = 0 \times \text{kips}$

$LC_{2V.tr} := 1.25 \times (D_{V.tr}) + 1.75 \times (L_{V.tr})$ <-- INCREASED TO AASHTO STR I — $LC_{2V.tr} = 0 \times \text{kips}$

$LC_{3V.tr} := 1.2 \times D_{V.tr} + \max(L_{V.tr}, 0.8 \times W_{V.tr})$ — $LC_{3V.tr} = 0 \times \text{kips}$

$LC_{4V.tr} := 1.2 \times D_{V.tr} + 1.6 \times W_{V.tr} + L_{V.tr}$ <-- EXCEEDS TO AASHTO STR III — $LC_{4V.tr} = 0 \times \text{kips}$

$LC_{5V.tr} := 1.25 \times D_{V.tr} + 1.35 \times L_{V.tr} + 0.4 \times W_{V.tr}$ <-- ALTERED TO AASHTO STR V — $LC_{5V.tr} = 0 \times \text{kips}$

$LC_{6V.tr} := 0.9 \times D_{V.tr} + 1.6 \times W_{V.tr}$ — $LC_{6V.tr} = 0 \times \text{kips}$

$LC_{7V.tr} := 0.9 \times D_{V.tr}$ — $LC_{7V.tr} = 0 \times \text{kips}$

Shear - Longitudinal

Dead Loads: $D_{V.lo} := V_{DC}$ Wind Loads: $W_{V.lo} := 0\text{kips}$ Live Loads: $L_{V.lo} := V_{LL}$

$LC_{1V.lo} := 1.4 \times (D_{V.lo})$ — $LC_{1V.lo} = 0.995 \times \text{kips}$

$LC_{2V.lo} := 1.25 \times (D_{V.lo}) + 1.75 \times (L_{V.lo})$ <-- INCREASED TO AASHTO STR I — $LC_{2V.lo} = 4.909 \times \text{kips}$

$LC_{3V.lo} := 1.2 \times D_{V.lo} + \max(L_{V.lo}, 0.8 \times W_{V.lo})$ — $LC_{3V.lo} = 3.15 \times \text{kips}$

$LC_{4V.lo} := 1.2 \times D_{V.lo} + 1.6 \times W_{V.lo} + L_{V.lo}$ <-- EXCEEDS TO AASHTO STR III — $LC_{4V.lo} = 3.15 \times \text{kips}$

$LC_{5V.lo} := 1.25 \times D_{V.lo} + 1.35 \times L_{V.lo} + 0.4 \times W_{V.lo}$ <-- ALTERED TO AASHTO STR V — $LC_{5V.lo} = 3.99 \times \text{kips}$

$LC_{6V.lo} := 0.9 \times D_{V.lo} + 1.6 \times W_{V.lo}$ — $LC_{6V.lo} = 0.64 \times \text{kips}$

$LC_{7V.lo} := 0.9 \times D_{V.lo}$ — $LC_{7V.lo} = 0.64 \times \text{kips}$

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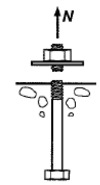
(a) steel strength of anchor in tension (D.5.1)

Nominal steel strength of Anchor in tension:

$N_{sa} := n_t \times A_{se.N} \times f_{uta}$ $\boxed{N_{sa} = 26.216 \times kips}$

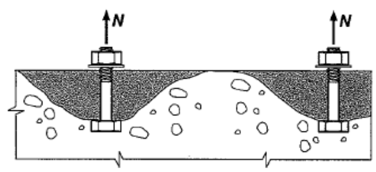


(i) Steel failure

(b) steel strength of anchor in shear (D.6.1)

For cast-in headed bolt and hooked bolt anchors:

$V_s := n_v \times 0.6 \times A_{se.V} \times f_{uta}$ $\boxed{V_s = 21.353 \times kips}$



(iii) Concrete breakout

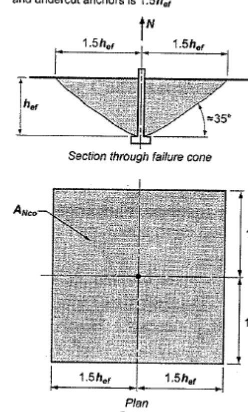
(c) concrete breakout strength of anchor in tension (D.5.2)

$$N_{cbg} = \frac{A_N}{A_{No}} \times \Psi_1 \times \Psi_2 \times \Psi_3 \times N_b$$

The projected area of the failure surface of a single anchor remote from edges:

$A_{Nco} := 9 \times h_{ef}^{\ 2}$ $\boxed{A_{Nco} = 144 \times in^2}$

The critical edge distance for headed studs, headed bolts, expansion anchors, and undercut anchors is $1.5h_{ef}$



$A_{Nco} = (2 \times 1.5h_{ef}) \times (2 \times 1.5h_{ef}) = 9h_{ef}^2$

$1.5 \times h_{ef} = 6 \times in$

The basic concrete breakout strength of a single anchor in tension in cracked concrete

for cast-in anchors k = 24, for post installed anchors k = 17

$Cast_{yes.no} := \text{"yes"}$ $Post_{yes.no} := \text{"no"}$

$k_c = 24$

$N_b := k_c \times \sqrt{\frac{f'_c}{psi}} \times \left(\frac{h_{ef}}{in}\right)^{1.5} \bullet (lb)$ $\boxed{N_b = 12.143 \times kips}$

Projected area of the failure surface for a group of anchors:

A_{Nc} Projected area of the failure surface for the group of anchors

$$inside_1 := \text{if}\left[\frac{s_1}{N_L - 1} > 2 \times 1.5 \times h_{ef}, 1.5 \times h_{ef} \times 2 \times (N_L - 1), s_1\right] = 0 \times in$$

$$inside_2 := \text{if}\left[\frac{s_2}{N_T - 1} > 2 \times 1.5 \times h_{ef}, 1.5 \times h_{ef} \times 2 \times (N_T - 1), s_2\right] = 10 \times in$$

$proj_{1A} := \min(1.5 \times h_{ef}, c_{a1})$ $proj_{1A} = 4 \times in$

$proj_{1B} := \min(1.5 \times h_{ef}, c_{b1})$ $proj_{1B} = 4 \times in$ $proj_{1A} + inside_1 + proj_{1B} = 8 \times in$

$proj_{2A} := \min(1.5 \times h_{ef}, c_{a2})$ $proj_{2A} = 6 \times in$

$proj_{2B} := \min(1.5 \times h_{ef}, c_{b2})$ $proj_{2B} = 6 \times in$ $proj_{2A} + inside_2 + proj_{2B} = 22 \times in$

$A_{Nc} := (proj_{1A} + s_1 + proj_{1B}) \times (proj_{2A} + s_2 + proj_{2B}) \times \frac{n_t}{n_v}$ $A_{Nc} = 176 \times in^2$

The modification factor for eccentrically loaded anchor groups (D.5.2.4):

$$\Psi_{ec.N} = \frac{1}{1 + \frac{2 \times e'_N}{3 \times h_{ef}}} \le 1$$

D.5.2.4 Eccentricity of normal force on a group of anchors:
"If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered."
"Where eccentric loading exists about two axes, the modification factor shall be computed for each axis individually and the product of the factors used"

$e'_N := 0$

$$\Psi_{ec.N} := \min\left(\frac{1}{1 + \frac{2 \times e'_N}{3 \times h_{ef}}}, 1\right) \qquad \boxed{\Psi_{ec.N} = 1}$$

D.5.2.5 Modification factor for edge effects:

$\Psi_{ed.N} = 1$ if $c_{min} \ge 1.5h_{ef}$ $\qquad$ $\Psi_{ed.N} = \left(0.7 + \frac{0.3 \times c_{min}}{1.5 \times h_{ef}}\right)$ if $c_{min} < 1.5h_{ef}$

$c_{min} := \min(c_{a1}, c_{a2}, c_{b1}, c_{b2})$ $\qquad$ $c_{min} = 4 \times in$

$$\Psi_{ed.N} := \text{if}\left(c_{min} \ge 1.5h_{ef}, 1, 0.7 + \frac{0.3 \times c_{min}}{1.5 \times h_{ef}}\right) \qquad \boxed{\Psi_{ed.N} = 0.9}$$

D.5.2.6 When an anchor is located in a region of a concrete member where analysis indicates no cracking at service load levels the following modification factor shall be permitted.

$\Psi_{c.N} = 1$ for cracking in service

Otherwise $\Psi_{c.N} = 1.25$ if anchors are cast-in and $\Psi_{c.N} = 1.4$ if anchors are post installed

$\boxed{Cast_{yes.no} = \text{"yes"}}$ $\qquad$ $\boxed{Post_{yes.no} = \text{"no"}}$

Cracking in service? $\qquad$ $Cracking_{yes.no} := \text{"yes"}$

$\boxed{\Psi_{c.N} = 1}$

The modification factor for post-installed anchors designed for uncracked concrete(D.5.2.7):

$c_{ac} := 2.5 \times h_{ef}$ for undercut anchors (D.8.6)

$c_{ac} = 10 \times in$

If c.ac is less than or equal to the smallest edge distance, the factor = 1
otherwise take maximum of the following equations

$$\frac{\min(c_{a1}, c_{a2})}{c_{ac}} = 0.4 \qquad \frac{1.5 \times h_{ef}}{c_{ac}} = 0.6$$

$$\boxed{\Psi_{cp.N} := \text{if}\left[yes_{no}(Post_{yes.no}) = 0 \vee yes_{no}(Cracking_{yes.no}) = 1, 1, \left(\text{if}\left(c_{ac} \le \min(c_{a1}, c_{a2}), 1.0, \max\left(\frac{\min(c_{a1}, c_{a2})}{c_{ac}}, \frac{1.5 \times h_{ef}}{c_{ac}}\right)\right)\right)\right] = 1}$$

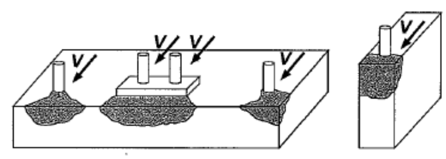
$A_{Nc} = 176 \times in^2$ $\Psi_{ec.N} = 1$ $N_b = 12.143 \times kips$

$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \times \Psi_{ec.N} \times \Psi_{ed.N} \times \Psi_{c.N} \times \Psi_{cp.N} \times N_b$ $A_{Nco} = 144 \times in^2$ $\Psi_{ed.N} = 0.9$ $\Psi_{cp.N} = 1$

$\boxed{N_{cbg} = 13.357 \times kips}$ $\Psi_{c.N} = 1$

(d) concrete breakout strength of anchor in shear (D.6.2)

$V_{cb} = \frac{A_{V.c}}{A_{Vco}} \times \Psi_{ed.V} \times \Psi_{c.V} \times \Psi_{h.V} \times V_b$



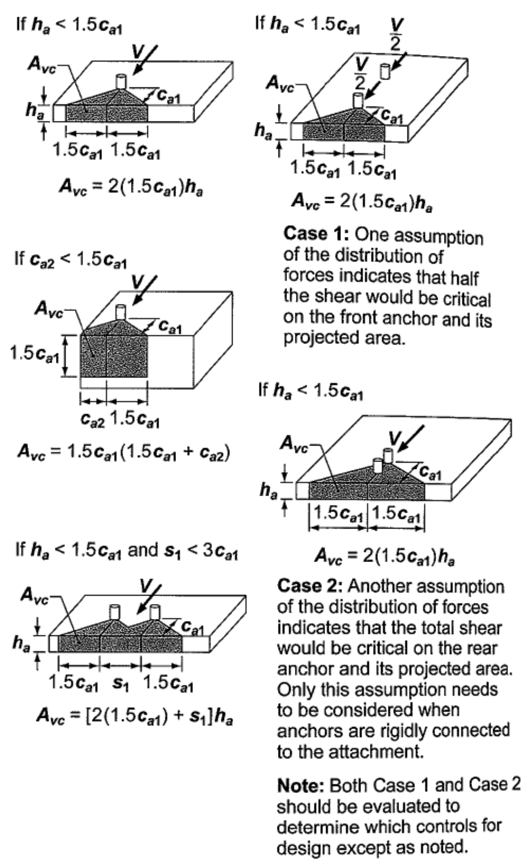
(iii) Concrete breakout

Is the concrete Normal weight or Light weight?

$Normal_{Weight}$:= "yes" Lightweight := "no"

$\boxed{\lambda = 1}$

Longitudinal Shear Capacity:



Projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. The depth of this failure surface is permitted to be the smaller of 1.5 x c1 or the full depth of the member

Where anchors are influenced by three or more edges, the value of c.a1 used shall not exceed the greatest of c.a2/1.5 in either direction, h.a/1.5; and one third of the maximum spacing between anchors within the group. (D.6.2.4)

If hairpins are provided to prevent concrete breakout, Seperate analysis must be conducted to determine the adequacy of the reinforcing.

Depth Check Case 1

$h_a = 6 \times in$ $1.5 \times c_{a1} = 6 \times in$

$\min(h_a, 1.5 \times c_{a1}) = 6 \times in$

Breakout Width Case 1

$c_{a2} = 999 \times in$ $\quad s_2 = 10 \times in$ $\quad c_{b2} = 999 \times in$

$\min(c_{a2}, 1.5 \times c_{a1}) = 6 \times in$ $\quad \min[s_2, 2 \times (1.5 \times c_{a1}) \times (N_T - 1)] = 10 \times in$ $\quad \min(c_{b2}, 1.5 \times c_{a1}) = 6 \times in$

$Edge_2 := if(c_{a2} < 1.5 \times c_{a1}, "yes", "no") = "no"$ $\quad Edge_3 := if(c_{b2} < 1.5 \times c_{a1}, "yes", "no") = "no"$

$\boxed{Edges = "Anchors influenced by less than 3 edges"}$

$A_{vc.case1} = 132 \times in^2$

Depth Check Case 2

$h_a = 6 \times in$ $\quad 1.5 \times (c_{a1} + s_1) = 6 \times in$

$\min[h_a, 1.5 \times (c_{a1} + s_1)] = 6 \times in$

Breakout Width Case 2

$c_{a2} = 999 \times in$ $\quad s_2 = 10 \times in$ $\quad c_{b2} = 999 \times in$

$\min[c_{a2}, 1.5 \times (c_{a1} + s_1)] = 6 \times in$ $\quad \min[s_2, 2 \times [1.5 \times (c_{a1} + s_1)] \times (N_T - 1)] = 10 \times in$ $\quad \min[c_{b2}, 1.5 \times (c_{a1} + s_1)] = 6 \times in$

$Edge_2 := if[c_{a2} < 1.5 \times (c_{a1} + s_1), "yes", "no"] = "no"$ $\quad Edge_3 := if[c_{b2} < 1.5 \times (c_{a1} + s_1), "yes", "no"] = "no"$

$\boxed{Edges = "Anchors influenced by less than 3 edges"}$

$A_{vc.case2} = 132 \times in^2$

c.a1 with possible change for 3 edge effects

$c_{a1} = 4 \times in$

$$c_{a1'.case1} := if\left[(Edge_2 = "yes" \wedge Edge_3 = "yes"), \min\left[c_{a1}, \max\left[\frac{c_{a2}}{1.5}, \frac{c_{b2}}{1.5}, \frac{h_a}{1.5}, \frac{s_2}{(N_T - 1) \times 3}\right]\right], c_{a1}\right]$$

$$c_{a1'.case2} := if\left[(Edge_2 = "yes" \wedge Edge_3 = "yes"), \min\left[c_{a1}, \max\left[\frac{c_{a2}}{1.5}, \frac{c_{b2}}{1.5}, \frac{h_a}{1.5}, \frac{s_2}{(N_T - 1) \times 3}\right]\right], c_{a1} + s_1\right]$$

$c_{a1'.case1} = 4 \times in$

$c_{a1'.case2} = 4 \times in$

D.6.2.5 The modification factor for eccentrically loaded anchor groups

$$\Psi_{ec.V} = \frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1}}} \le 1$$

Eccentricity $\quad e'_v := 0$

$$\Psi_{ec.V.case1} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1'.case1}}}, 1\right) \qquad \boxed{\Psi_{ec.V.case1} = 1}$$

$$\Psi_{ec.V.case2} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1'.case2}}}, 1\right) \qquad \boxed{\Psi_{ec.V.case2} = 1}$$

D.6.2.6 The modification factor for edge effect

$\Psi_{ed.V} = 1$ if $c_{a2} \geq 1.5 \times c_{a1'}$

$\Psi_{ed.V} = \left(0.7 + \dfrac{0.3 \times c_{a2}}{1.5 \times c_{a1'}}\right)$ if $c_{a2} < 1.5h_e$

$\Psi_{ed.V.case1} := \text{if}\left(c_{a2} \geq 1.5 \times c_{a1'.case1}, 1, 0.7 + \dfrac{0.3 \times c_{a2}}{1.5 \times c_{a1'.case1}}\right)$ $\boxed{\Psi_{ed.V.case1} = 1}$

$\Psi_{ed.V.case2} := \text{if}\left(c_{a2} \geq 1.5 \times c_{a1'.case2}, 1, 0.7 + \dfrac{0.3 \times c_{a2}}{1.5 \times c_{a1'.case2}}\right)$ $\boxed{\Psi_{ed.V.case2} = 1}$

D.6.2.7 Anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the following modification factor shall be used.

$\Psi_{c.V} = 1.4$ where no cracking occurs in service $\boxed{Cracking_{yes.no} = \text{"yes"}}$

Otherwise $\Psi_{c.V} = 1.0$ if anchors are cast in concrete without supplementary reinforcement or edge reinforcement smaller than a No 4 bar.

$\Psi_{c.V} = 1.2$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar.

$\Psi_{c.V} = 1.4$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar and with stirrups spaced at not more than 4"

Is Anchor reinforced by at least #4 bars? $Reinf_{with.no4} := \text{"yes"}$

Is anchor enclosed by stirrups at 4" maximum spacing? $Stirrups_{4inch.spa.max} := \text{"no"}$

Are Haripins provided to prevent Concrete Breakout? $Hairpins_{Long.yesno} := \text{"no"}$ $\boxed{\Psi_{c.V} = 1.2}$

$A_{Vco.loLoad.case1} := 4.5 \times c_{a1'.case1}^{\ 2}$ $A_{Vco.loLoad.case1} = 72 \times in^2$

$A_{Vco.loLoad.case2} := 4.5 \times c_{a1'.case2}^{\ 2}$ $A_{Vco.loLoad.case2} = 72 \times in^2$

D.6.2.8 The modification factor for anchors located in a concrete member where h.a<1.5*c.a1

$h_a = 6 \times in$ $c_{a1'.case1} = 4 \times in$ $c_{a1'.case2} = 4 \times in$

$$\Psi_{h.V.case1} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a1'.case1}, \sqrt{\frac{1.5 \times c_{a1'.case1}}{h_a}}, 1\right), 1\right) = 1$$

$$\Psi_{h.V.case2} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a1'.case2}, \sqrt{\frac{1.5 \times c_{a1'.case2}}{h_a}}, 1\right), 1\right) = 1$$

The basic concrete breakout strength V.b in shear of a single anchor in cracked concrete shall not exceed (D6.2.2):

$$V_b = \left[7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a}\right] \times \lambda \times \sqrt{f'_c} \times \left(c_{a1}\right)^{1.5}$$

Load bearing length of anchor for shear (for anchors with constant stiffness)

$l_e := h_{ef}$ $l_e = 4 \times in$ $d_a = 0.625 \times in$

$$V_{b.case1} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a1'.case1} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb)$$ $\boxed{V_{b.case1} = 4.059 \times kips}$

$$V_{b.case2} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a1'.case2} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb)$$ $\boxed{V_{b.case2} = 4.059 \times kips}$

$A_{vc.case1} = 132 \times in^2$ $\quad \Psi_{ec.V.case1} = 1$ $\quad \Psi_{c.V} = 1.2$

$A_{Vco.loLoad.case1} = 72 \times in^2$ $\quad \Psi_{ed.V.case1} = 1$ $\quad \Psi_{h.V.case1} = 1$

$$V_{cbg.Long.case1} := \frac{A_{vc.case1}}{A_{Vco.loLoad.case1}} \times \Psi_{ec.V.case1} \times \Psi_{ed.V.case1} \times \Psi_{c.V} \times \Psi_{h.V.case1} \times V_{b.case1}$$

$\boxed{V_{cbg.Long.case1} = 8.929 \times kips}$

$A_{vc.case2} = 132 \times in^2$ $\quad \Psi_{ec.V.case2} = 1$ $\quad \Psi_{c.V} = 1.2$

$A_{Vco.loLoad.case2} = 72 \times in^2$ $\quad \Psi_{ed.V.case2} = 1$ $\quad \Psi_{h.V.case2} = 1$

$$V_{cbg.Long.case2} := \frac{A_{vc.case2}}{A_{Vco.loLoad.case2}} \times \Psi_{ec.V.case2} \times \Psi_{ed.V.case2} \times \Psi_{c.V} \times \Psi_{h.V.case2} \times V_{b.case2}$$

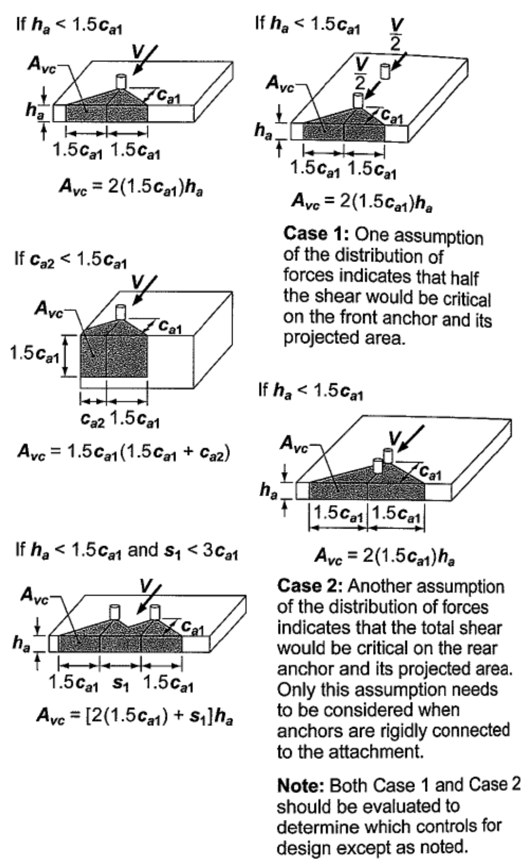
$\boxed{V_{cbg.Long.case2} = 8.929 \times kips}$

Transverse Shear Capacity:

Projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. The depth of this failure surface is permitted to be the smaller of 1.5 x c1 or the full depth of the member

Where anchors are influenced by three or more edges, the value of c.a1 used shall not exceed the greatest of c.a2/1.5 in either direction, h.a/1.5; and one third of the maximum spacing between anchors within the group. (D.6.2.4)

If hairpins are provided to prevent concrete breakout, Seperate analysis must be conducted to determine the adequacy of the reinforcing.



Depth Check Case 1

$h_a = 6 \times in \qquad 1.5 \times c_{a2} = 1.498 \times 10^3 \times in$

$\min(h_a, 1.5 \times c_{a2}) = 6 \times in$

Breakout Width Case 1

$c_{a1} = 4 \times in$ $\quad s_1 = 0 \times in$ $\quad c_{b1} = 4 \times in$

$\min(c_{a1}, 1.5 \times c_{a2}) = 4 \times in$ $\quad \min[s_1, 2 \times (1.5 \times c_{a2}) \times (N_L - 1)] = 0 \times in$ $\quad \min(c_{b1}, 1.5 \times c_{a2}) = 4 \times in$

$Edge_2 := \text{if}(c_{a1} < 1.5 \times c_{a2}, \text{"yes"}, \text{"no"}) = \text{"yes"}$ $\quad Edge_3 := \text{if}(c_{b1} < 1.5 \times c_{a2}, \text{"yes"}, \text{"no"}) = \text{"yes"}$

$\boxed{Edges = \text{"Anchors influenced by 3 edges"}}$

$A_{vc.case1} = 48 \times in^2$

Depth Check Case 2

$h_a = 6 \times in$ $\quad 1.5 \times (c_{a2} + s_2) = 1.513 \times 10^3 \times in$

$\min[h_a, 1.5 \times (c_{a2} + s_2)] = 6 \times in$

Breakout Width Case 2

$c_{a1} = 4 \times in$ $\quad s_1 = 0 \times in$ $\quad c_{b1} = 4 \times in$

$\min[c_{a1}, 1.5 \times (c_{a2} + s_2)] = 4 \times in$ $\quad \min[s_1, 2 \times [1.5 \times (c_{a2} + s_2)] \times (N_T - 1)] = 0 \times in$ $\quad \min[c_{b1}, 1.5 \times (c_{a2} + s_2)] = 4 \times in$

$Edge_2 := \text{if}[c_{a1} < 1.5 \times (c_{a2} + s_2), \text{"yes"}, \text{"no"}] = \text{"yes"}$ $\quad Edge_3 := \text{if}[c_{b1} < 1.5 \times (c_{a2} + s_2), \text{"yes"}, \text{"no"}] = \text{"yes"}$

$\boxed{Edges = \text{"Anchors influenced by 3 edges"}}$

$A_{vc.case2} = 48 \times in^2$

c.a2 with possible change for 3 edge effects

$c_{a2} = 999 \times in$

$$c_{a2'.case1} := \text{if}\left[(Edge_2 = \text{"yes"} \wedge Edge_3 = \text{"yes"}), \min\left[c_{a2}, \max\left[\frac{c_{a1}}{1.5}, \frac{c_{b1}}{1.5}, \frac{h_a}{1.5}, \frac{s_1}{(N_L - 1) \times 3}\right]\right], c_{a2}\right]$$

$$c_{a2'.case2} := \text{if}\left[(Edge_2 = \text{"yes"} \wedge Edge_3 = \text{"yes"}), \min\left[c_{a2}, \max\left[\frac{c_{a1}}{1.5}, \frac{c_{b1}}{1.5}, \frac{h_a}{1.5}, \frac{s_1}{(N_L - 1) \times 3}\right]\right], c_{a2} + s_2\right]$$

$c_{a2'.case1} = 4 \times in$

$c_{a2'.case2} = 4 \times in$

D.6.2.5 The modification factor for eccentrically loaded anchor groups

$$\Psi_{ec.V} = \frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2}}} \leq 1$$

Eccentricity $\quad e'_v := 0$

$$\Psi_{ec.V.case1} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2'.case1}}}, 1\right)$$ $\boxed{\Psi_{ec.V.case1} = 1}$

$$\Psi_{ec.V.case2} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2'.case2}}}, 1\right)$$ $\boxed{\Psi_{ec.V.case2} = 1}$

D.6.2.6 The modification factor for edge effect

$\Psi_{ed.V} = 1 \quad \text{if} \quad c_{a1} \geq 1.5 \times c_{a2'}$ $\qquad \Psi_{ed.V} = \left(0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'}}\right) \quad \text{if} \quad c_{a1} < 1.5h_e$

$$\Psi_{ed.V.case1} := \text{if}\left(c_{a1} \geq 1.5 \times c_{a2'.case1}, 1, 0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'.case1}}\right) \qquad \boxed{\Psi_{ed.V.case1} = 0.9}$$

$$\Psi_{ed.V.case2} := \text{if}\left(c_{a1} \geq 1.5 \times c_{a2'.case2}, 1, 0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'.case2}}\right) \qquad \boxed{\Psi_{ed.V.case2} = 0.9}$$

D.6.2.7 Anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the following modification factor shall be used.

$\boxed{\text{Cracking}_{yes.no} = \text{"yes"}}$

$\Psi_{c.V} = 1.4$ where no cracking occurs in service

Otherwise $\Psi_{c.V} = 1.0$ if anchors are cast in concrete without supplementary reinforcement or edge reinforcement smaller than a No 4 bar.

$\Psi_{c.V} = 1.2$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar.

$\Psi_{c.V} = 1.4$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar and with stirrups spaced at not more than 4"

Is Anchor reinforced by at least #4 bars? $\text{Reinf}_{with.no4} := \text{"yes"}$

Is anchor enclosed by stirrups at 4" maximum spacing? $\text{Stirrups}_{4inch.spa.max} := \text{"no"}$

Are Haripins provided to prevent Concrete Breakout? $\text{Hairpins}_{Trans.yesno} := \text{"no"}$

$\boxed{\Psi_{c.V} = 1.2}$

$A_{Vco.trLoad.case1} := 4.5 \times c_{a2'.case1}^{\ 2}$ $\qquad A_{Vco.trLoad.case1} = 72 \times in^2$

$A_{Vco.trLoad.case2} := 4.5 \times c_{a2'.case2}^{\ 2}$ $\qquad A_{Vco.trLoad.case2} = 72 \times in^2$

D.6.2.8 The modification factor for anchors located in a concrete member where h.a<1.5*c.a1

$h_a = 6 \times in \qquad c_{a2'.case1} = 4 \times in \qquad c_{a2'.case2} = 4 \times in$

$$\Psi_{h.V.case1} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a2'.case1}, \sqrt{\frac{1.5 \times c_{a2'.case1}}{h_a}}, 1\right), 1\right) = 1$$

$$\Psi_{h.V.case2} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a2'.case2}, \sqrt{\frac{1.5 \times c_{a2'.case2}}{h_a}}, 1\right), 1\right) = 1$$

The basic concrete breakout strength V.b in shear of a single anchor in cracked concrete shall not exceed (D6.2.2):

$$V_b = \left[7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a}\right] \times \lambda \times \sqrt{f'_c} \times \left(c_{a2}\right)^{1.5}$$

Load bearing length of anchor for shear (for anchors with constant stiffness)

$l_e := h_{ef} \qquad l_e = 4 \times in \qquad d_a = 0.625 \times in$

$$V_{b.case1} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a2'.case1} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb) \qquad \boxed{V_{b.case1} = 4.059 \times kips}$$

$$V_{b.case2} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a2'.case2} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb) \qquad \boxed{V_{b.case2} = 4.059 \times kips}$$

$A_{vc.case1} = 48 \times in^2$ $\Psi_{ec.V.case1} = 1$ $\Psi_{c.V} = 1.2$

$A_{Vco.trLoad.case1} = 72 \times in^2$ $\Psi_{ed.V.case1} = 0.9$ $\Psi_{h.V.case1} = 1$

$$V_{cbg.Trans.case1} := \frac{A_{vc.case1}}{A_{Vco.trLoad.case1}} \times \Psi_{ec.V.case1} \times \Psi_{ed.V.case1} \times \Psi_{c.V} \times \Psi_{h.V.case1} \times V_{b.case1}$$

$\boxed{V_{cbg.Trans.case1} = 2.922 \times kips}$

$A_{vc.case2} = 48 \times in^2$ $\Psi_{ec.V.case2} = 1$ $\Psi_{c.V} = 1.2$

$A_{Vco.trLoad.case2} = 72 \times in^2$ $\Psi_{ed.V.case2} = 0.9$ $\Psi_{h.V.case2} = 1$

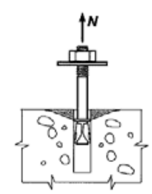
$$V_{cbg.Trans.case2} := \frac{A_{vc.case2}}{A_{Vco.trLoad.case2}} \times \Psi_{ec.V.case2} \times \Psi_{ed.V.case2} \times \Psi_{c.V} \times \Psi_{h.V.case2} \times V_{b.case2}$$

$\boxed{V_{cbg.Trans.case2} = 2.922 \times kips}$

(e) pullout strength of anchor in tension (D.5.3)

$N_{pn} = \Psi_{c.P} \times N_p$

$Headed_{yes.no} = \text{"yes"}$ $Hooked_{yes.no} = \text{"no"}$



D.5.3.4 The pullout strength in tension of a single headed stud or headed bolt shall not exceed:

$N_p = A_{brg} \times 8 \times f'_c$ $A_{brg} \times 8 \times f'_c = 32 \times kips$

D.5.3.5 The pullout strength in tension of a single hooked bolt shall not exceed:

$N_p = 0.9 \times f'_c \times e_h \times d_a$ $0.9 \times f'_c \times e_h \times d_a = 0 \times kips$

$N_p := yes_{no}(Headed_{yes.no}) \times (A_{brg} \times 8 \times f'_c) + yes_{no}(Hooked_{yes.no}) \times (0.9 \times f'_c \times e_h \times d_a)$ $\boxed{N_p = 32 \times kips}$

D.5.3.6 For an anchor located in a region of a concrete member where analysis indicates no cracking at service load levels

$\Psi_{c.P} = 1.4$ where no cracking is expected under service load

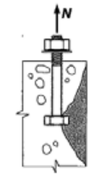
$\Psi_{c.P} = 1.0$ when cracking is expected

$Cracking_{yes.no} = \text{"yes"}$ $\boxed{\Psi_{c.P} = 1}$

$N_{pn} := \Psi_{c.P} \times N_p$ $\boxed{N_{pn} = 32 \times kips}$

$N_{png} := n_t \times N_{pn}$ $\boxed{N_{png} = 64 \times kips}$

(f) concrete side-face blowout strength of anchor in tension (D.5.4)



(iv) Side-face blowout

For a single headed anchor with deep embedment close to an edge.

Applicable for : $h_{ef} > 2.5 \times \min(c_{a1}, c_{a2})$

$h_{ef} = 4 \times in \qquad 2.5 \times \min(c_{a1}, c_{a2}) = 10 \times in \qquad \lambda = 1$

D.5.4.1 For a Single headed anchor with deep embedment close to an edge:

if c.a2 for the single headed anchor is less than 3.ca1, the value N.sb shall be factored by:

$$N_{sb} := \text{if}\left[h_{ef} > 2.5 \times \min(c_{a1}, c_{a2}), \left[160 \times \frac{\min(c_{a1}, c_{a2})}{in} \times \sqrt{\frac{A_{brg}}{in^2}} \times \lambda \times \sqrt{\frac{f'_c}{psi}} \bullet (lb)\right], 1 \times 10^{10} kips\right] \qquad \boxed{N_{sb} = 1 \times 10^{10} \times kips}$$

$$\psi_{multiside.blowout} := \text{if}\left[\max(c_{a1}, c_{a2}) < 3 \times \min(c_{a1}, c_{a2}), \left(1 + \max\left(\min\left(\frac{\max(c_{a1}, c_{a2})}{\min(c_{a1}, c_{a2})}, 3\right), 1\right)\right) \times \frac{1}{4}, 1\right] = 1$$

$$N_{sb'} := \psi_{multiside.blowout} \times N_{sb} + 1 \times 10^{10} kips \times yes_{no}(Hooked_{yes.no}) \qquad \boxed{N_{sb'} = 1 \times 10^{10} \times kips}$$

D.5.4.2 For multiple headed anchors with deep embedment close to an edge, and spacing less than 6c

$$N_{sbg} = \left(1 + \frac{s_{edge}}{6 \times \min(c_{a1}, c_{a2})}\right) \times N_{sb}$$

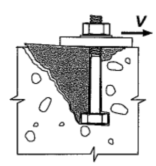
Spacing of outer anchors along the edge in the group $\qquad s_{edge} := s_1 \qquad s_{edge} = 0 \times in$

$$N_{sbg} := \left(1 + \frac{s_{edge}}{6 \times \min(c_{a1}, c_{a2})}\right) \times N_{sb} \qquad \boxed{N_{sbg} = 1 \times 10^{10} \times kips}$$

$$N_{sb'.g'} := \left(N_{sbg} + kips \times yes_{no}(Hooked_{yes.no})\right) \times (n_t > 1) + N_{sb'} \times (n_t = 1)$$

$$\boxed{N_{sb'.g'} = 1 \times 10^{10} \times kips}$$

(g) concrete pryout strength of anchor in shear (D.6.3)



(ii) Concrete pryout for anchors far from a free edge

The nominal pryout strength shall not exceed:

$$V_{cp} = k_{cp} \times N_{cbg}$$

$k_{cp} = 1.0$ for $h_{ef} < 2.5in$ $\qquad h_{ef} = 4 \times in$

$k_{cp} = 2.0$ for $h_{ef} \geq 2.5in$

$$k_{cp} := \text{if}(h_{ef} < 2.5in, 1.0, 2.0) \qquad \boxed{k_{cp} = 2}$$

D.5.2.1 The Nominal Concrete breakout strength of an anchor in tension

$N_{cbg} = 13.357 \times kips$

$N_{cbg} = 13.357 \times kips$

$$V_{cpg} := k_{cp} \times N_{cbg} \qquad \boxed{V_{cpg} = 26.715 \times kips}$$

Summary

$Reinf_{with.no4} = \text{"yes"}$

$Stirrups_{4inch.spa.max} = \text{"no"}$

Seismic Restrictions Apply?

$Seismic_{Restrictions} := \text{"no"}$

Force Seismic Concrete? (severe reduction)

$Seismic_{Concrete} := \text{"no"}$

$Seismic_{input} = \text{"ok"}$

$LC_{1T} = 0 \times kips$

$LC_{2T} = 0 \times kips$

$LC_{3T} = 0 \times kips$

$LC_{4T} = 0 \times kips$

$LC_{5T} = 0 \times kips$

$LC_{6T} = 0 \times kips$

$LC_{7T} = 0 \times kips$

Tension

(a) steel strength of anchor in tension (D.5.1)

$N_{sa} = 26.216 \times kips$

(c) concrete breakout strength of anchor in tension (D.5.2)

$N_{cbg} = 13.357 \times kips$

(e) pullout strength of anchor in tension (D.5.3)

$N_{png} = 64 \times kips$

(f) concrete side-face blowout strength of anchor in tension (D.5.4)

$N_{sb'.g'} = 1 \times 10^{10} \times kips$

$Govern_{Tension} = \text{"Governed by Concrete"}$

Strength Reduction factor assuming anchor has low sensitivity to installation and has high reliability

$\phi_T = 0.7$

$$\phi Pn := \phi_T \times \min\left[N_{sa} \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), N_{cbg}, N_{png}, N_{sb'.g'}\right]$$

$\phi Pn = 9.35 \times kips$

$\frac{LC_{1T}}{\phi Pn} = 0$ $\frac{LC_{2T}}{\phi Pn} = 0$ $\frac{LC_{3T}}{\phi Pn} = 0$

$\frac{LC_{4T}}{\phi Pn} = 0$ $\frac{LC_{5T}}{\phi Pn} = 0$ $\frac{LC_{6T}}{\phi Pn} = 0$ $\frac{LC_{7T}}{\phi Pn} = 0$

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Shear

(b) steel strength of anchor in shear (D.6.1)

$V_s = 21.353 \times kips$

Transverse	Longitudinal
$LC_{1V.tr} = 0 \times kips$	$LC_{1V.lo} = 0.995 \times kips$
$LC_{2V.tr} = 0 \times kips$	$LC_{2V.lo} = 4.909 \times kips$
$LC_{3V.tr} = 0 \times kips$	$LC_{3V.lo} = 3.15 \times kips$
$LC_{4V.tr} = 0 \times kips$	$LC_{4V.lo} = 3.15 \times kips$
$LC_{5V.tr} = 0 \times kips$	$LC_{5V.lo} = 3.99 \times kips$
$LC_{6V.tr} = 0 \times kips$	$LC_{6V.lo} = 0.64 \times kips$
$LC_{7V.tr} = 0 \times kips$	$LC_{7V.lo} = 0.64 \times kips$

(d) concrete breakout strength of anchor in shear (D.6.2)

$V_{cbg.Long.case1} = 8.929 \times kips$ $\quad V_{cbg.Trans.case1} = 2.922 \times kips$

$V_{cbg.Long.case2} = 8.929 \times kips$ $\quad V_{cbg.Trans.case2} = 2.922 \times kips$

(g) concrete pryout strength of anchor in shear (D.6.3)

$V_{cpg} = 26.715 \times kips$

$Govern_{Long.case} = \text{"Cases are identical"}$ $\quad Govern_{Trans.case} = \text{"Cases are identical"}$

$Govern_{Long.Shear} = \text{"Governed by Concrete"}$ $\quad Govern_{Trans.Shear} = \text{"Governed by Concrete"}$

$\phi_{V.Long} = 0.75$ $\quad \phi_{V.Trans} = 0.75$

Transverse

$$\phi Vn_{tr} := \phi_{V.Trans} \times \left| \begin{array}{l} V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Trans.case1}, V_{cpg}\right] \\ V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Trans.case2}, V_{cpg}\right] \text{ if } Govern_{Trans.case} = \text{"Governed by} \end{array} \right.$$

$\phi Vn_{tr} = 2.192 \times kips$

$\frac{LC_{1V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{2V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{3V.tr}}{\phi Vn_{tr}} = 0$

$\frac{LC_{4V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{5V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{6V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{7V.tr}}{\phi Vn_{tr}} = 0$

Longitudinal

$$\phi Vn_{lo} := \phi_{V.Long} \times \left| \begin{array}{l} V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Long.case1}, V_{cpg}\right] \\ V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Long.case2}, V_{cpg}\right] \text{ if } Govern_{Long.case} = \text{"Governed by} \end{array} \right.$$

$\phi Vn_{lo} = 6.697 \times kips$

$\frac{LC_{1V.lo}}{\phi Vn_{lo}} = 0.149$ $\quad \frac{LC_{2V.lo}}{\phi Vn_{lo}} = 0.733$ $\quad \frac{LC_{3V.lo}}{\phi Vn_{lo}} = 0.47$

$\frac{LC_{4V.lo}}{\phi Vn_{lo}} = 0.47$ $\quad \frac{LC_{5V.lo}}{\phi Vn_{lo}} = 0.596$ $\quad \frac{LC_{6V.lo}}{\phi Vn_{lo}} = 0.096$ $\quad \frac{LC_{7V.lo}}{\phi Vn_{lo}} = 0.096$

Combine Tension and Shear: (D.7)

D.7.1 - if $V_{ua} \le 0.2 \times \varphi \times V_n$ then full strength in tension shall be permitted

D.7.2 - if $N_{ua} \le 0.2 \times \varphi \times N_n$ then full strength in shear shall be permitted

D.7.3 - otherwise $\dfrac{N_{ua}}{\varphi N_n} + \dfrac{V_{ua}}{\varphi V_n} \le 1.2$

Combined Stress Ratio less than 1.0 is Satisfactory

Transverse Combination:

$$CSR_{T1} := \frac{\left(\frac{LC_{1T}}{\phi Pn}\right) \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) + \frac{LC_{1V.tr}}{\phi Vn_{tr}} \times \left(\frac{LC_{1V.tr}}{\phi Vn_{tr}} > 0.2\right)}{1 + 0.2 \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) \times \left(\frac{LC_{1V.tr}}{\phi Vn_{tr}} > 0.2\right)} = 0$$

Longitudinal Combination:

$$CSR_{L1} := \frac{\left(\frac{LC_{1T}}{\phi Pn}\right) \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) + \frac{LC_{1V.lo}}{\phi Vn_{lo}} \times \left(\frac{LC_{1V.lo}}{\phi Vn_{lo}} > 0.2\right)}{1 + 0.2 \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) \times \left(\frac{LC_{1V.lo}}{\phi Vn_{lo}} > 0.2\right)} = 0$$

$CSR_{T2} = 0$ $CSR_{L2} = 0.733$

$CSR_{T3} = 0$ $CSR_{L3} = 0.47$

$CSR_{T4} = 0$ $CSR_{L4} = 0.47$

$CSR_{T5} = 0$ $CSR_{L5} = 0.596$

$CSR_{T6} = 0$ $CSR_{L6} = 0$

$CSR_{T7} = 0$ $CSR_{L7} = 0$

$Seismic_{restrictions}$ = "D.3.3 seismic restrictions do not apply"

Load to Edge Beam

Dead Load Reaction

$$plf_{edge.beam.DC} := \frac{R_{edge.beam.DC}}{slab_{span}} = 482.193 \times plf$$

Live Load Reaction

$$plf_{edge.beam.LL} := \left(\frac{L_{new.span}}{2} + L_{new.canti}\right) \times w_{LL} = 520.275 \times plf$$

Edge Beam Length:
(REF SHT 50 - Elev. Girder 4)

$$L_{edge} := 490.26ft - 406.96ft = 83.3\,ft$$

Edge beam is simply supported:

$$V6_{edge} := 1.25 \times plf_{edge.beam.DC} + 1.75 \times plf_{edge.beam.LL} = 1.513 \times 10^{3} \times plf$$

$$M6_{edge.pos} := \frac{V6_{edge} \times L_{edge}^{\ 2}}{8} = 1.3 \times 10^{3}\,ft \times kips$$

Moment Capacity at strength limit state in Major Axis for Discretely braced flanges in Compression - AASHTO LRFD 6.10.8.1.1

$\varphi_f := 1.00$ for flexure $\quad E := 29000ksi$

W36 x 150 Member information

Top Flange: $b_{tf} := 12in$ $\quad t_{tf} := 0.940in$ $\quad A_{tf} := b_{tf} \times t_{tf} = 11.28 \times in^2$ $\quad I_{tf} := \dfrac{b_{tf} \times t_{tf}^3}{12} = 0.831 \times in^4$

Web: $h_{web} := 35.9in$ $\quad t_{web} := 0.625in$ $\quad A_{web} := h_{web} \times t_{web} = 22.437 \times in^2$ $\quad I_{web} := \dfrac{t_{web} \times h_{web}^3}{12} = 2.41 \times 10^3 \times in$

Bottom Flange: $b_{bf} := 12in$ $\quad t_{bf} := 0.940in$ $\quad A_{bf} := b_{bf} \times t_{bf} = 11.28 \times in^2$ $\quad I_{bf} := \dfrac{b_{bf} \times t_{bf}^3}{12} = 0.831 \times in^4$

$$CG := \frac{A_{tf} \times \frac{t_{tf}}{2} + A_{web} \times \left(t_{tf} + \frac{h_{web}}{2}\right) + A_{bf} \times \left(t_{tf} + h_{web} + \frac{t_{bf}}{2}\right)}{A_{tf} + A_{web} + A_{bf}} = 18.89 \times in$$

$$I_{total} := I_{tf} + A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right)^2 + I_{web} + A_{web} \times \left(CG - t_{tf} - \frac{h_{web}}{2}\right)^2 + I_{bf} + A_{bf} \times \left(CG - t_{tf} - h_{web} - \frac{t_{bf}}{2}\right)^2 = 1.007 \times 10^4 \times in^4$$

$$S_x := \frac{I_{total}}{CG} = 532.874 \times in^3 \qquad Z_x := 2 \times \left[A_{tf} \times \left(CG - \frac{t_{tf}}{2}\right) + t_{web} \times \left(CG - t_{tf}\right) \times \frac{\left(CG - t_{tf}\right)}{2}\right] = 616.932 \times in^3$$

$M_u := M6_{edge.pos} = 1.313 \times 10^3\,ft \times kips$ $\quad F_y := 50ksi$ $\quad L_b := 10ft$

At the strength limit state, the following requirement shall be satisfied: $f_{bu} + \frac{1}{3} \times f_l \le \varphi_f \times F_{nc}$

The local buckling resistance (6.10.8.2.2) of the compression flange shall be taken as

if $\lambda_f \le \lambda_{pf}$ then $F_{nc} = R_b \times R_h \times F_{yc}$ $\quad \lambda_f := \dfrac{b_{bf}}{2 \times t_{bf}} = 6.4$ $\quad \lambda_{pf} := 0.38 \times \sqrt{\dfrac{E}{F_y}} = 9.152$

$less\left(\lambda_f, \lambda_{pf}\right) = $ "Less than or Equal to"

Web load-shedding factor determined as specified in Article 6.10.1.10.2 $\quad R_b = 1.0$ if $\dfrac{2 \times D_c}{t_w} \le \lambda_{rw}$

Depth of web in compression in elastic range $\quad D_c := h_{web} - \left(CG - t_{tf}\right) + t_{bf} = 18.89 \times in$

Limiting slenderness ratio for a noncompact web:

$\dfrac{2 \times D_c}{t_{web}} = 60.448$ $\quad \lambda_{rw} := 5.7 \times \sqrt{\dfrac{E}{F_y}} = 137.274$ $\quad less\left(\dfrac{2 \times D_c}{t_{web}}, \lambda_{rw}\right) = $ "Less than or Equal to" $\quad R_b := 1.0$

Hybrid Factor: $\quad R_h = \dfrac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta}$ $\quad D_n := D_c = 18.89 \times in$ $\quad$ For doubly symmetric sections

$A_{fn} := b_{bf} \times t_{bf} = 11.28 \times in^2$ $\quad \beta := \dfrac{2 \times D_n \times t_{web}}{A_{fn}} = 2.093$

$\rho := 1.0$ $\quad$ Non Composite with no cover plates on tensile side

$R_h := \dfrac{12 + \beta \times \left(3 \times \rho - \rho^3\right)}{12 + 2 \times \beta} = 1$ $\quad F_{nc.lb} := R_b \times R_h \times F_y = 50 \times ksi$

The Lateral Torsional Buckling resistance (6.10.8.2.3) of the compression flange shall be taken as

$L_p = 1.0 \times r_t \times \sqrt{\frac{E}{F_{yc}}}$ Limit of Plastic deformation

$b_{fc} := b_{bf} = 12 \times in$ $t_{fc} := t_{bf} = 0.94 \times in$

$r_t := \frac{b_{fc}}{\sqrt{12 \times \left(1 + \frac{1}{3} \times \frac{D_c \times t_{web}}{b_{fc} \times t_{fc}}\right)}}$ $r_t = 0.249\ ft$

$\boxed{L_p := 1.0 \times r_t \times \sqrt{\frac{E}{F_y}} = 5.986\ ft}$

$F_{yr} := 0.7 \times F_y = 35 \times ksi$

$\boxed{L_r := \pi \times r_t \times \sqrt{\frac{E}{F_{yr}}} = 22.477\ ft}$

$C_b := 1$ Unbraced Cantilevers or simple spans

$\boxed{F_{nc.ltb} := C_b \times \left[1 - \left(1 - \frac{F_{yr}}{R_h \times F_y}\right) \times \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \times F_y = 46.3 \times ksi}$

$\boxed{F_{nc} := \min(F_{nc.lb}, F_{nc.ltb}) = 46.3 \times ksi}$

The Maximum Allowable Unbraced Length (6.10.1.6-3) is: $L_b \le 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{M_{yc}}{M_u}}$

$L_{max} := 1.2 \times L_p \times \sqrt{C_b \times R_b \times \frac{F_y \times S_x}{M_u}} = 9.343\ ft$

$\boxed{Check_{C.D}(L_{max}, L_b) = \text{"Not Satisfactory"}}$

No Lateral bending is anticipated at this location $f_l := 0$

$f_{bu} \le \varphi_f \times F_{nc}$

$\boxed{S_x \times (\varphi_f \times F_{nc}) = 2058 \times kip \times ft}$

$M_u = 1.313 \times 10^3\ ft \times kips$

$\boxed{Check_{C.D}[S_x \times (\varphi_f \times F_{nc}), M_u] = \text{"SATISFACTORY"}}$

Check Beam for deflection due to live Load

$I_{W36x150} := 9040 in^4$

$\delta_{span} := \frac{5}{384} \times \frac{plf_{edge.beam.LL} \times L_{edge}^{4}}{E \times I_{W36x150}} = 2.15 \times in$

$\frac{L_{edge}}{\delta_{span} + \delta_{cantilever}} = 453.05$

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Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

$V_n = V_{cr} = C \times V_p$

C = if $\frac{D}{t_w} \le 1.12 \times \sqrt{\frac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := h_{web} = 35.9 \times in$ $E := 29000ksi$ $F_{yw} := 50ksi$

$t_{web} = 0.625 \times in$

$\frac{D}{t_{web}} = 57.4$ $1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}} = 60.3$

$C := \text{if}\left[\frac{D}{t_{web}} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$

$V_p := 0.58 \times F_{yw} \times D \times t_{web} = 650.7 \times kips$

$\varphi_v \times V_p = 650.7 \times kips$

$Reaction_{Factored} := \frac{V6_{edge} \times L_{edge}}{2} = 63.026 \times kips$

$Check_{C.D}\left(\varphi_v \times V_p, Reaction_{Factored}\right) = \text{"SATISFACTORY"}$

Service Reactions to Bearings:

$DC_{reaction} := plf_{edge.beam.DC} \times \frac{L_{edge}}{2} = 20.083 \times kips$

$LL_{reaction} := plf_{edge.beam.LL} \times \frac{L_{edge}}{2} = 21.669 \times kips$

Steel Reinforced Elastomeric Bearing Pads
Design Method A

AASHTO LRFD 2009 14.7.6

Preliminary Sizing

(LRFD 14.7.6.3.4) "The maximum horizontal superstructure displacement shall be computed in accordance with Article 14.4. The maximum shear deformation of the pad at the service limit state, Δ_s, shall be taken as the maximum horizontal superstructure displacement, reduced to account for the pier flexibility and modified for construction procedures."

Temperature

$\alpha := 0.0000065$ Coefficient of Expansion for Steel

$Length := L_{edge}$ (REF SHT 50 - Elev. Girder 4)

$Length = 83.3\ ft$ Continuous Deck Length

Temperature Range for Steel Structures (Western Washington): 0°F to 120°F (WSDOT 9.1.2-B)

Effective Deck Length per elastomeric pad: $L_{eff} := \frac{Length}{2}$ $L_{eff} = 41.65\ ft$

$\Delta_{TL} := L_{eff} \times \alpha \times (64 - 0)$ $\Delta_{TL} = 0.208 \times in$

$\Delta_{TH} := L_{eff} \times \alpha \times (120 - 64)$ $\Delta_{TH} = 0.182 \times in$

Sliding Elastomeric Bearing Assembly:
(Allows for unrestricted movement parallel to bridge centerline, used when deflections cause excessive pad height)

$Slide_{Assembly} := "no"$

Total Movement (Displacement from Center Line)

Per WSDOT 2010 Bridge Design Manual p 9.2-4

Total Motion = $\frac{3}{4} \times \left(\Delta Temp_{Rise} + \Delta Temp_{Fall}\right) + \Delta_{shrink} + \Delta_{PT.creep}$

$\delta_{Total} := \frac{3}{4} \times \left(\Delta_{TL} + \Delta_{TH}\right)$ $\delta_{Total} = 0.292 \times in$

Sizing (AASHTO LRFD 2009 14.7.6.3.4)

$h_{rt.min} := 2 \times \delta_{Total}$ $\boxed{h_{rt.min} = 0.585 \times in}$ $Shear_{Deformation} = \text{"Allowed"}$

Try $n_{layers} := 1$ interior elastomeric layers $h_{ri} := \frac{1}{2} in$ thick $SD_{Incr} = 100 \times \%$

Exterior layers are $h_{exl} = 0.25 \times in$ thick

$h_{rt} := n_{layers} \times h_{ri} + 2 \times h_{exl}$ $\boxed{h_{rt} = 1 \times in}$

$\boxed{Pad_{Height.Check} = \text{"SATISFACTORY"}}$

Use $h_s := ga_{pl_{14}}$ gauge steel plates between layers $h_s = 0.075 \times in$

$h_{st} := (n_{layers} + 1) \times h_s$ $h_{st} = 0.149 \times in$

Total Bearing Thickness: $h_{pad} := h_{rt} + h_{st}$ $\boxed{h_{pad} = 1.149 \times in}$

Allowable Compression Stress (AASHTO LRFD 14.7.6.3.2)

"For steel-reinforced" elastomeric bearings... $\sigma_s \le 1.25ksi$ and $\sigma_s \le 1.25 \times G \times S$

"The stress limits may be increased by 10% where shear deformation is prevented"

Select Length for Pad (1" increments are preferred): $L := 9in$

Number of Bearing Pads: $Number_{pads} := 1$

For pad loadings, AASHTO requests design forces be based on the "average compressive stress" in at the service limit state (LRFD 14.7.6.3.2). ***(Only SERVICE 1 typically Applies - Dynamic Increase not required 14.4.1)***

Maximum Dead and Live Loads on single pad:
Live loads derived from Standard truck:

$R_{D.max} := DC_{reaction}$ $R_{D.max} = 20.083 \times kips$ $R_{L.max} := LL_{reaction} = 21.669 \times kips$

Minimum Dead and Live Loads on single pad:
Live loads derived from Standard truck:

$R_{D.min} := DC_{reaction}$ $R_{D.min} = 20.083 \times kips$ $R_{L.min} := 0kips$

For σ_s less than or equal to $1.25ksi \times SD_{Incr} = 1.25 \times ksi$

$P_{max} := R_{D.max} + R_{L.max}$ $\boxed{P_{max} = 41.753 \times kips}$ $P_{min} := R_{D.min} + R_{L.min}$ $\boxed{P_{min} = 20.083 \times kips}$

$\frac{P}{L \times W} \le 1.25ksi \times SD_{Incr}$ $W_{min} := \frac{P_{max}}{L \times 1.25ksi \times SD_{Incr}}$ Minimum W dim. for stress check is $W_{min} = 3.71 \times in$

Choose $W := 9in$

$\frac{P_{max}}{L \times W} = 0.515 \times ksi$ $Check_{C.D}\left(1.25ksi \times SD_{Incr}, \frac{P_{max}}{L \times W}\right) = \text{"SATISFACTORY"}$

Method A only applies where $S_i^2 \times n^{-1} \le 22$ **(AASHTO LRFD 2009**

$S_i := \frac{L \times W}{2 \times h_{ri} \times (L + W)} = 4.5$ $\frac{S_i^2}{n_{layers}} = 20.25$ < 20 $Check_{C.D}\left(20, \frac{S_i^2}{n_{layers}}\right) = \text{"Not Satisfactory"}$ Acceptable

Elastomer Hardness (LRFD Table 14.7.6.2-1)

Select a Durometer Hardness 50, 55, or 60:
(NDOT Prefers 50 or 55 Durometer)

$\text{hardness} := \text{"50"}$

$G_{min} = 0.095 \times ksi$ $\quad G_{max} = 0.130 \times ksi$

$creep = 0.25$

Bearing Stability (LRFD 14.7.6.3.6)

The total thickness of the pad ($h_{pad} = 1.149 \times in$) shall not exceed the least of:

$\frac{W}{3} = 3 \times in$ $\qquad \frac{L}{3} = 3 \times in$

$$\text{Stability} := \text{if}\left(h_{pad} > \frac{W}{3}, \text{"not satisfied, W is too small"}, \text{if}\left(h_{pad} > \frac{L}{3}, \text{"not satisfied, L is too small"}, \text{"SATISFACTORY"}\right)\right)$$

$\text{Stability} = \text{"SATISFACTORY"}$

Steel Reinforcement (LRFD 14.7.6.3.7)
The thickness of the steel reinforcement shall satisfy:

$F_y := 36ksi$

$h_s \geq \frac{3 \times h_{ri} \times \sigma_s}{F_y}$ $\qquad h_{ri} = \frac{1}{2} \times in$ Thickness of the thickest elastomeric layer

$\sigma_{s.max} := \frac{P_{max}}{L \times W}$ $\qquad \sigma_{s.max} = 0.515 \times ksi$ Service average compressive stress due to total load

$\sigma_{s.min} := \frac{P_{min}}{L \times W}$ $\qquad \sigma_{s.min} = 0.248 \times ksi$ Service average compressive stress due to total load

$h_{s.min} := \max\left(\frac{3 \times h_{ri} \times \sigma_{s.max}}{F_y}, 0.0625in\right)$ $\qquad h_{s.min} = 0.0625 \times in$

Must Also Satisfy:

$h_s \geq \frac{2.0 \times h_{max} \times \sigma_L}{\Delta F_{HT}}$

$\sigma_L := \frac{R_{L.max}}{L \times W}$ $\qquad \sigma_L = 0.268 \times ksi$ Service average compressive stress due to live load

$\Delta F_{TH} := 24ksi$ *Allowable Fatigue Stress Range - Category A - 24 ksi over 2,000,000 stress cycles (AASHTO LRFD Table 6.6.1.2.3-1)*

$h_{s.min.2} := \frac{2.0 \times h_{ri} \times \sigma_L}{\Delta F_{TH}}$ $\qquad h_{s.min.2} = 0.011 \times in$ $\qquad h_s = 0.075 \times in$

$$\text{plate} := \text{if}\left(h_s > h_{s.min}, \text{if}\left(h_s > h_{s.min.2}, \text{"SATISFACTORY"}, \text{"Not Satisfactory"}\right), \text{"Not Satisfactory"}\right)$$

$\text{plate} = \text{"SATISFACTORY"}$

Second Compression Check

$G_{min} = 0.095 \times ksi$ $\qquad \sigma_s \leq 1.25 \times G \times S \times SD_{Incr}$

$1.25 \times G_{min} \times S_i \times SD_{Incr} = 0.534 \times ksi$ $\qquad \sigma_{s.max} = 0.515 \times ksi$

Conservatively use flexible limit $\qquad \text{Check}_{C.D}\left(1.25 \times G_{min} \times S_i \times SD_{Incr}, \sigma_{s.max}\right) = \text{"SATISFACTORY"}$

Rotation

"An allowance for uncertainties, which shall be taken as 0.005 rad. unless an approved quality control plan justifies a smaller value" (LRFD 14.7.3.6.2.5 & 14.4.2.1)

$\delta_{max} := 1.5in$ $\qquad$ $\delta_{min} := 0in$

$\Theta_{max} := \frac{3}{2} \times \frac{\delta_{max}}{\frac{L_{edge}}{2}} = 0.005$ $\qquad$ $\Theta_{min} := \frac{3}{2} \times \frac{\delta_{min}}{\frac{L_{edge}}{2}} = 0$

$\theta_{max} := |\Theta_{max}| + 0.005rad$ $\qquad$ $\theta_{min} := |\Theta_{min}| + 0.005rad$

$\theta_{max} = 0.0095 \times rad$ $\qquad$ $\theta_{min} = 0.005 \times rad$

Equation 14.7.6.3.5d-1 & 2:

Caltrans recommends 0.2 ksi minimum

$$\sigma_s \geq 0.5 \times G_{max} \times S_i \times \left(\frac{L}{h_{ri}}\right)^2 \times \frac{\theta_{s.x}}{n}$$

$\sigma_{s.max} = 0.515 \times ksi$ $\qquad$ $\sigma_{s.min} = 0.248 \times ksi$

$$\sigma_s \geq 0.5 \times G_{max} \times S_i \times \left(\frac{W}{h_{ri}}\right)^2 \times \frac{\theta_{s.z}}{n}$$

$L = 9 \times in$ $\qquad$ $W = 9 \times in$ $\qquad$ $h_{ri} = \frac{1}{2} \times in$

Shape Factor $\quad S_i := \frac{L \times W}{2 \times h_{ri} \times (L + W)}$ $\qquad$ $S_i = 4.5$

Shear Modulus of elastomer $\quad hardness = "50"$ $\qquad$ $G_{max} = 0.130 \times ksi$ $\qquad$ *Conservatively use stiff limit*

$$\sigma_{rot.max} := 0.5 \times G_{max} \times S_i \times \left(\frac{L}{h_{ri}}\right)^2 \times \frac{\theta_{max}}{n_{layers} + 1}$$

$\sigma_{rot.max} = 0.45 \times ksi$

$$\sigma_{rot.min} := 0.5 \times G_{max} \times S_i \times \left(\frac{L}{h_{ri}}\right)^2 \times \frac{\theta_{min}}{n_{layers} + 1}$$

$\sigma_{rot.min} = 0.237 \times ksi$

$Check_{C.D}(\sigma_{s.max}, \sigma_{rot.max}) = "SATISFACTORY"$ $\qquad$ $Check_{C.D}(\sigma_{s.min}, \sigma_{rot.min}) = "SATISFACTORY"$

(n layers +1) is valid for layer thicknesses of 0.5in (with 0.25in or greater exterior layers) and 0.625in (with 0.3125in or greater exterior layers)

$\frac{\sigma_{rot.min}}{\sigma_{s.min}} = 0.956$

Acceptable

$$\sigma_{rot.max.ortho} := 0.5 \times G_{max} \times S_i \times \left(\frac{W}{h_{ri}}\right)^2 \times \frac{0.005rad}{n_{layers} + 1}$$

$\sigma_{rot.max.ortho} = 0.237 \times ksi$

$$\sigma_{rot.min.ortho} := 0.5 \times G_{max} \times S_i \times \left(\frac{W}{h_{ri}}\right)^2 \times \frac{0.005rad}{n_{layers} + 1}$$

$\sigma_{rot.min.ortho} = 0.237 \times ksi$

$Check_{C.D}(\sigma_{s.max}, \sigma_{rot.max.ortho}) = "SATISFACTORY"$ $\qquad$ $Check_{C.D}(\sigma_{s.min}, \sigma_{rot.min.ortho}) = "SATISFACTORY"$

If a Sliding Elastomeric Bearing Assembly is used. Shear forces in the pad are limited to the Teflon / Stainless Steel interface friction.

$\boxed{Slide_{Assembly} = \text{"no"}}$

Coefficient of friction: $\mu := 0.06$

Shear Force from Slide Assembly: $Shear_{slide} := \mu \times R_{D.max} \times yes_{no}(Slide_{Assembly})$

$\boxed{Shear_{slide} = 0 \times kips}$

Anchorage

"If the factored shear force sustained by the deformed pad at the strength limit state exceeds one-fifth of the Minimum vertical force due to permanent loads, the pad shall be secured against horizontal movement"

$G_{max} = 0.130 \times ksi$ $\quad G = \dfrac{shear_{stress}}{shear_{strain}}$ *Conservatively use stiff limit*

$shear_{strain} := \dfrac{\delta_{Total}}{h_{rt}}$ $\quad \boxed{shear_{strain} = 0.292}$

$shear_{stress} := G_{max} \times shear_{strain}$ $\quad \boxed{shear_{stress} = 0.038 \times ksi}$

$Shear_{force} := shear_{stress} \times L \times W$ $\quad \boxed{Shear_{force} = 3.079 \times kips}$

$Friction := \dfrac{1}{5} \times R_{D.max}$ $\quad \boxed{Friction = 4.017 \times kips}$

$Anchorage := if(Shear < Friction, \text{"Not Required"}, \text{"Required"})$

$\boxed{Anchorage = \text{"Not Required"}}$

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Vertical Deflection

(Vertical compressive strain is based on AASHTO LRFD charts C14.7.6.3.3-1. Values are linearly interpolated between shown shape factor lines and durometer graphs)

Input Parameters

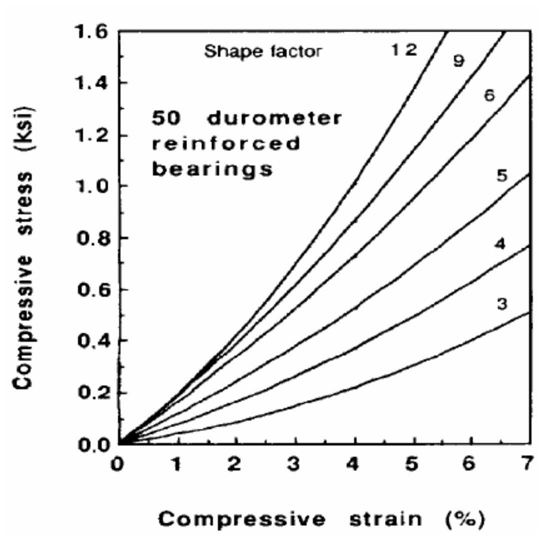
$hardness = "50"$

DL Stress: $\sigma_D := \frac{R_{D.max}}{L \times W}$ $\quad \sigma_D = 0.248 \times ksi$

LL stress: $\sigma_L = 0.268 \times ksi$

Max combined stress: $\sigma_{s.max} = 0.515 \times ksi$

Shape Factor: $S_i = 4.5$



Short Term Deflection:

Dead Load:

The vertical strain and deflection on the pad is:

$\varepsilon_{DL} := \varepsilon(S_i, \sigma_D, hardness)$ $\quad \varepsilon_{DL} = 2.4 \times \%$

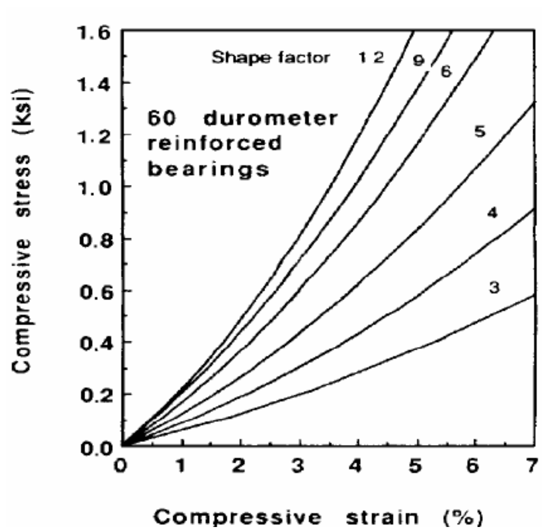
$\delta_{DL} := h_{rt} \times \varepsilon_{DL}$ $\quad \delta_{DL} = 0.024 \times in$

Live Load:

The vertical strain and deflection on the pad is:

$\varepsilon_{LL} := \varepsilon(S_i, \sigma_{s.max}, hardness) - \varepsilon_{DL}$ $\quad \varepsilon_{LL} = 2.1 \times \%$

$\delta_{LL} := h_{rt} \times \varepsilon_{LL}$ $\quad \delta_{LL} = 0.021 \times in$



Long Term Deformation:

The creep factor is: $\quad creep = 0.25$

$\delta_{creep} := creep \times \delta_{DL}$ $\quad \delta_{creep} = 6.095 \times 10^{-3} \times in$

Maximum Long term Deflection:

$\delta_{max} := \delta_{creep} + \delta_{DL} + \delta_{LL}$ $\quad \delta_{max} = 0.051 \times in$

Performance Validation:

Vertical strain should be limited to 7% $\quad \varepsilon_{DL} + \varepsilon_{LL} = 4.5 \times \%$

$Check_{C.D}(7\%, \varepsilon_{DL} + \varepsilon_{LL}) = "SATISFACTORY"$

The maximum relative displacement across a joint is 0.125in (AASHTO LRFD 2002 C14.7.5.3.3).
The relative displacement used in this calculation is a combination of creep deformation and live load deflection as the dead load deformation is negated by the expansion joint construction pour.

$\Delta := \delta_{LL} + \delta_{creep}$ $\quad \Delta = 0.027 \times in$

$Check_{C.D}(0.125in, \Delta) = "SATISFACTORY"$

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Final Design

$n_{layers} = 1$ Layers of Elastomer ($hardness = "50"$) @ $h_{ri} = \frac{1}{2} \times in$ each

with two $h_{exl} = \frac{1}{4} \times in$ exterior Layers

Separated by $n_{layers} + 1 = 2$ - 14 gauge steel plates ($h_s = 0.075 \times in$ each)

Total Depth $h_{pad} = 1.149 \times in$

Final Dimensions: $Number_{pads} = 1$ @

$L = 9 \times in$ X $W = 9 \times in$ X $h_{pad} = 1.149 \times in$

If a Sliding Elastomeric Bearing Assembly is used, a teflon sliding surface must be provided. $Slide_{Assembly} = "no"$

Dead Load deflection = $\delta_{DL} = 0.024 \times in$

Live Load deflection = $\delta_{LL} = 0.021 \times in$

Maximum Long term deflection = $\delta_{max} = 0.051 \times in$

Shear force applied to substructure from single pad under full deformation $Shear = 3.079 \times kips$

Total shear force applied to substructure from all pads under full deformation: $Number_{pads} \times Shear = 3.079 \times kips$

Additional dead load to the structure from walkway widening.

Additional Dead Load, Slab and Railing: $plf_{slab.railing} := \frac{R_{barrier.DC} + R_{edge.beam.DC}}{slab_{span}} = 553.3 \times plf$

Dead Load of New Traffic Barrier + BP rail: $plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$

Dead Load of New Transverse Beam: (10ft support + 10ft perimeter beam) $plf_{trans.beam} := \frac{14.51plf \times \left(10ft + slab_{span}\right)}{slab_{span}} = 29.02 \times plf$

Dead Load of New Support Beam: $plf_{support.beam} := 150plf$

Total weight of new work: $plf_{new} := plf_{slab.railing} + plf_{barrier} + plf_{trans.beam} + plf_{support.beam} = 1.1 \times klf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedistal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 38 & 37): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedistal weight: $plf_{pedistal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{20ft} = 35.9 \times plf$

Dead Load of Existing intermediate Conc Rail: $plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times [20ft - (1ft + 4in) - 0.5in] \times \gamma_{c.ex}}{20ft} = 128 \times plf$

Dead Load of Existing metal rail: (8.15plf top beam REF SHT 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{20ft} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{20ft}\right]$$

$plf_{metal.rail} = 16.4 \times plf$

Dead Load of Existing Edge Beam: $plf_{ex.edge.beam} := (6in \times 10in + 10in \times 10in) \times \gamma_{c.ex} = 166.7 \times plf$

Dead Load of Existing sidewalk: (REF SHT 37) $plf_{ex.sidewalk} := 3.5in \times 4ft \times \gamma_{c.ex} = 175 \times plf$

Dead Load of Existing Curb: (REF SHT 38) $plf_{curb} := \left[18in \times \left(7in + \frac{3in}{2}\right) - 3in \times 2in\right] \times \gamma_{c.ex} = 153.125 \times plf$

Total weight of removals:

$$plf_{rem} := plf_{pedistal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{ex.sidewalk} + plf_{curb} = 675.2 \times plf$$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 444.3 \times plf$

Weight of Segment 6 $Segment6_{weight} := 645kips$

$Segment6_{length} := 84.91ft$

Length of Segment 7:

Average Per foot weight of Segment 7: $\frac{Segment6_{weight}}{Segment6_{length}} = 7.596 \times 10^{3} \times plf$

Percent Increase due to extended sidewalk load: $\frac{plf_{increase}}{\frac{Segment6_{weight}}{Segment6_{length}}} = 5.849 \times \%$ Less than 10% OKAY

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: $H_{mast} := 30ft + (3ft + 7in) = 33.583\ ft$
(REF SHT E-6/91/4 : 782-95)

Length of Luminaire Mast arm: $L_{arm} := 12ft$
(12' max REF WSDOT STD J-28.10-01)

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d$ (psf)

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\ ft$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head $A := 3.3ft^2$
(REF WSDOT BDM 10.1(B))

$V_{wind.lum} := P_z \times A = 44.876\ lb$

Height to point of connection: $H := H_{mast} = 33.583\ ft$

$M_{wind.lum} := V_{wind.lum} \times H = 1.507\ ft \times kips$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

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Wind on Mast Arm

$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$

$K_z := K_{z.eq}\left(H_{arm}, 900ft, 9.5\right) = 1.151$

$$C_{d.cylinder}(V,d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \frac{V}{mph} \times \frac{d}{ft} \\ CD \leftarrow \frac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \text{ if } \omega \le 39 \\ CD \leftarrow 0.45 \text{ if } \omega \ge 78 \\ CD \end{array}$$

Cylinders (3in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 3in) = 1.1$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$V_{wind.arm} := P_z \times A = 89.752\,lb$

Height to point of connection: $H := H_{mast} = 33.583\,ft$

$M_{wind.arm} := V_{wind.arm} \times H = 3.014\,ft \times kips$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$K_z := K_{z.eq}\left(H_{pole}, 900ft, 9.5\right) = 1.079$

Cylinders (8in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 8in) = 0.553$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$V_{wind.pole} := P_z \times A = 315.609\,lb$

Height to point of connection: $H := \frac{H_{mast}}{2} = 16.792\,ft$

$M_{wind.pole} := V_{wind.pole} \times H = 5.3\,ft \times kips$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using V = 45 m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30		
Traffic Signals (see note 2)	1.2		
Luminaires (with generally rounded surfaces)	0.5		
Luminaires (with rectangular flat side shapes)	1.2		
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o}-1\right)+C_{dD}\left(2-\frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1-0.7\left(\frac{D}{d_o}-1\right)^{1/4}\right]$	
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)		
Single Member or Truss Member	**$C_vVd \le 5.33$ (39)**	**5.33(39) < C_vVd < 10.66(78)**	**$C_vVd \ge 10.66(78)$**
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37+1.08r-\frac{C_vVd}{19.8}-\frac{C_vVdr}{4.94}$ (SI) $1.37+1.08r-\frac{C_vVd}{145}-\frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55+\frac{(10.66-C_vVd)}{9.67}$ (SI) $0.55+\frac{(78.2-C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

$V_{wind} := V_{wind.lum} + V_{wind.arm} + V_{wind.pole} = 0.45 \times kips$

Total Moment from wind (omnidirectional):

$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 9.821\,ft \times kips$

Dead Loads

Distance from Pole location to Point of connection: $Offset := L_{new.canti} + 2.5in + 3in + 6in = 4.832\ ft$

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B)) $W_{lum} := 60lb$

$M_{DC.lum} := W_{lum} \times (L_{arm} - Offset) = 0.43\ ft \times kips$

Weight of Arm:
(Assume 11 gage)

$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\ lb$

$M_{DC.arm} := W_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.054\ ft \times kips$

Weight of Pole:
(Assume 11 gage)

$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\ lb$

$M_{DC.pole} := W_{pole} \times (-Offset) = -1.66\ ft \times kips$

Total Moment from Dead Load:
(+15% for electrical etc.)

$M_{DC} := 115\% \times (M_{DC.lum} + M_{DC.arm} + M_{DC.pole}) = -1.353\ ft \times kips$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\ lb$

$M_{Ice.lum} := Ice_{lum} \times (L_{arm} - Offset) = 0.426\ ft \times kips$

Weight of Arm:

$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\ lb$

$M_{Ice.arm} := Ice_{arm} \times \left(\frac{L_{arm}}{2} - Offset\right) = 0.033\ ft \times kips$

Weight of Pole:

$Ice_{pole} := \pi \times 8in \times 3psf \times H_{pole} = 294.001\ lb$

$M_{Ice.pole} := Ice_{pole} \times (-Offset) = -1.421\ ft \times kips$

Total Moment from Dead Load:

$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} + M_{Ice.pole} = -0.962\ ft \times kips$

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Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left(\left|M_{DC}\right| + M_{wind}\right) \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 1.353 \\ 8.401 \\ 5.432 \end{pmatrix} ft \times kips$$

Governing Pole Load $\quad \max\left(M_{design'}\right) = 8.401\ ft \times kips$

Pole location requires special transverse support beam.

Combine governing pole load with governing design forces for transverse beam:
(Wind combinations STR III and V)

$$M_{STR3} := 1.25 \times \left[w_{DC.Railing} \times \left(L_{new.canti} + 1.5in\right) + w_{DC.slab} \times slab_{span} \times \frac{\left(L_{new.canti}\right)^2}{2} \right] \ldots = 23.455\ ft \times kips$$
$$+ \left[\left(1.25 \times w_{DC.slab} \times slab_{span}\right) \times \frac{\left(L_{new.span}\right)^2}{8} \right] + 1.4 \times M_{wind}$$

$$M_{STR5} := 1.25 \times \left[w_{DC.Railing} \times \left(L_{new.canti} + 1.5in\right) + w_{DC.slab} \times slab_{span} \times \frac{\left(L_{new.canti}\right)^2}{2} \right] \ldots$$
$$+ 1.35 \times \left(w_{LL} \times slab_{span} \times \frac{L_{new.canti}{}^2}{2} \right) + \left[\left(1.25 \times w_{DC.slab} \times slab_{span} + 1.75 \times w_{LL} \times slab_{span}\right) \times \frac{\left(L_{new.span}\right)^2}{8} \right] \ldots$$
$$+ 0.4 \times M_{wind}$$

$$M_{STR5} = 27.388\ ft \times kips$$

$$M_{lum.support} := \max\left(M_{STR3}, M_{STR5}\right) = 27.388\ ft \times kips$$

Connect pole to square tube:
Square or Rectangular HSS Bending
For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit
(AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges
(AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges
(AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange
(AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-12) may be used for the development of AASHTO capacities.

HSS 6x10x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$M_{n.\Omega.AISC} := 31.7\text{kip} \times \text{ft}$$

$$M_{n.\Omega.AASHTO.HSS6x10x3} := \frac{1.0}{0.9} \times M_{n.\Omega.AISC} = 35.2 \times \text{kip} \times \text{ft}$$

$$\frac{M_{lum.support}}{M_{n.\Omega.AASHTO.HSS6x10x3}} = 0.778$$

$$\text{Check}_{C.D}\left(M_{n.\Omega.AASHTO.HSS6x10x3}, M_{lum.support}\right) = \text{"SATISFACTORY"}$$

Shear on Anchors

PINNED reaction on cast in anchors (Unbalanced live load condition)
Space anchors at 10" on center

$V_{DC} := R_{barrier.DC} = 0.711 \times kips$

$V_{LL} := w_{LL} \times slab_{span} \times \frac{L_{new.span}}{2} = 2.297 \times kips$

$V_{wind} := \frac{M_{wind}}{L_{new.span}} = 1.603 \times kips$

Dead load reaction from luminaire results in an uplift reaction which reduces the demand on the connection. Reaction is neglected.

$\frac{M_{DC}}{L_{new.span}} = -0.221 \times kips$

Distance from Anchor centerline to Center of compression block (assume 1" block) $dist_{anchor2compr} := 3in$

Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): $A325 := \text{"yes"}$ $A490 := \text{"No"}$ $ASTM = \text{"A325"}$

Nominal Bolt Diameter: $d := \frac{5}{8} in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.31 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$$F_{ub} := \begin{array}{|l} FUB \leftarrow 0ksi \\ FUB \leftarrow 150ksi \ \text{ if } \ ASTM = \text{"A490"} \\ \text{if } \ ASTM = \text{"A325"} \\ \quad \begin{array}{|l} FUB \leftarrow 120ksi \ \text{ if } \ d \le 1.0in \\ FUB \leftarrow 105ksi \ \text{ if } \ d > 1.0in \end{array} \end{array}$$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 1$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 14 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9) *For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter*

Thickness of the connected material: $t := 0.375in$

Tensile strength of the connected material specified in Table 6.4.1-1: Grade 36: $F_u := 58ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 32.6 \times kips$

Strength Resistance minimum of bolt shear and bearing $R_n := \min(R_{n.shear}, R_{n.holes}) = 13.99 \times kips$

$\varphi := 0.80$ For A325 and A490 Bolts in Shear and Tension $\varphi \times R_n = 11.192 \times kips$

$\frac{1.25 \times V_{DC} + 1.75 \times V_{LL}}{4 \times (\varphi \times R_n)} = 0.11$

$Check_{C.D}(\varphi \times R_n, 1.25 \times V_{DC} + 1.75 \times V_{LL}) = \text{"SATISFACTORY"}$

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Anchor Bolt Design - ACI 318-08 D.4

*If **post installed anchors** are designed for seismic loads then the following criteria must be satisfied (D3.3)*
D.3.3.2 - Post installed anchors shall be qualified for use in cracked concrete
D.3.3.3 - Concrete failure modes shall have an additional 0.75 reduction factor
D.3.3.4 - Anchors shall be designed to be governed by ductile steel
D.3.3.6 - Alternative to using a ductile steel element, it is permitted to use a concrete failure mode using an additional 0.4 reduction beyond what is specified in D.3.3.3

General:

Number of anchors in tension: $n_t := 2$

Number of anchors in shear: $n_v := 2$

Headed Anchor or hooked anchor $Headed_{yes.no} := \text{"yes"}$ $Hooked_{yes.no} := \text{"no"}$

Anchor pattern:

Center to Center distance for outermost Anchors-

$s_1 := 0in$ Longitudinal Direction Number of bolts in line: $N_L := 1$

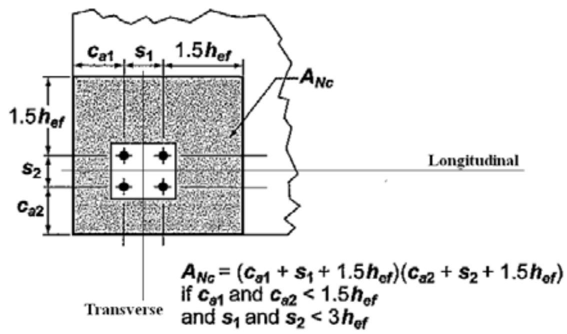
Limiting concrete dimensions (a1 = Positive Load Breakout Side)

$c_{a1} := 4in$ $c_{b1} := 4in$

$s_2 := 10in$ Transverse Direction Number of bolts in line: $N_T := 4$

Limiting concrete dimensions (a2 = Positive Load Breakout Side)

$c_{a2} := 999in$ $c_{b2} := 999in$



Depth of concrete member: $h_a := 6in$

Diameter of anchor bolt: $d_a := 0.625in$

Specify = "Area of Head"

set other length to 0

Area of Head: $A_{brg} := 1in^2$

Hook Length: $e_h := 0$
3da<eh<4.5da

Section area of anchor bolt:

$A_{se.N} := 0.226in^2$ $$A_{se.V} := \pi \times \frac{d_a^{\,2}}{4} = 0.307 \times in^2$$

Nominal Tensile Stress of Fasteners and Threaded parts (AISC Table 2-5) & F1554 values:

$A_{307} := \text{"no"}$ $A_{325} := \text{"no"}$ $A_{490} := \text{"no"}$ $Specified := 0$

$F1554_{G36} := \text{"yes"}$ $F1554_{G55} := \text{"no"}$ $F1554_{G105} := \text{"no"}$

$f_{uta} = 58 \times ksi$

Embedment depth of anchor: $h_{ef} := 4in$

Concrete strength: $f'_c := 4ksi$
"shall not exceed 10,000 psi for cast-in anchors, and 8,000 psi for post-installed anchors"

F1554 GR 55 Data

Anchor Diameter in. (mm)	Drill Diameter in. (mm)	Tensile Stress Area
1/2 (12.7)	5/8 (15.9)	0.142
5/8 (15.9)	3/4 (19.1)	0.226
3/4 (19.1)	7/8 (22.2)	0.334
7/8 (22.2)	1 (25.4)	0.462
1 (25.4)	1 1/8 (28.6)	0.606
1 1/8 (28.6)	1 1/4 (31.8)	0.763
1 1/4 (31.8)	1 3/8 (34.9)	0.969
1 3/8 (34.9)	1 1/2 (38.1)	1.155
1 1/2 (38.1)	1 3/4 (44.4)	1.405
1 3/4 (44.4)	2 (50.8)	1.900
2 (50.8)	2 1/4 (57.2)	2.500
2 1/4 (57.2)	2 1/2 (63.5)	3.250
2 1/2 (63.5)	2 3/4 (69.9)	4.000
2 3/4 (69.9)	3 (76.2)	4.930
3 (76.2)	3 1/4 (82.5)	5.970

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Loads

"The design strength of anchors shall equal or exceed the largest required strength calculated from the applicable load combinations in 9.2 or C9.2" (D.3.2)

Tension

Use Negative Values for Compression Forces

Dead Loads: $D_T := 0\text{kips}$ Wind Loads: $W_T := 0\text{kips}$ Live Loads: $L_T := 0\text{kips}$

$LC_{1T} := \left|1.4 \times (D_T)\right|$ — $LC_{1T} = 0 \times \text{kips}$

$LC_{2T} := 1.25 \times (D_T) + 1.75 \times (L_T)$ <-- INCREASED TO AASHTO STR I — $LC_{2T} = 0 \times \text{kips}$

$LC_{3T} := 1.2 \times D_T + \max(L_T, 0.8 \times W_T)$ — $LC_{3T} = 0 \times \text{kips}$

$LC_{4T} := 1.2 \times D_T + 1.6 \times W_T + L_T$ <-- EXCEEDS TO AASHTO STR III — $LC_{4T} = 0 \times \text{kips}$

$LC_{5T} := 1.25 \times D_T + 1.35 \times L_T + 0.4 \times W_T$ <-- ALTERED TO AASHTO STR V — $LC_{5T} = 0 \times \text{kips}$

$LC_{6T} := 0.9 \times D_T + 1.6 \times W_T$ — $LC_{6T} = 0 \times \text{kips}$

$LC_{7T} := 0.9 \times D_T$ — $LC_{7T} = 0 \times \text{kips}$

Shear - Transverse

Dead Loads: $D_{V.tr} := 0\text{kips}$ Wind Loads: $W_{V.tr} := 0\text{kips}$ Live Loads: $L_{V.tr} := 0\text{kips}$

$LC_{1V.tr} := 1.4 \times (D_{V.tr})$ — $LC_{1V.tr} = 0 \times \text{kips}$

$LC_{2V.tr} := 1.25 \times (D_{V.tr}) + 1.75 \times (L_{V.tr})$ <-- INCREASED TO AASHTO STR I — $LC_{2V.tr} = 0 \times \text{kips}$

$LC_{3V.tr} := 1.2 \times D_{V.tr} + \max(L_{V.tr}, 0.8 \times W_{V.tr})$ — $LC_{3V.tr} = 0 \times \text{kips}$

$LC_{4V.tr} := 1.2 \times D_{V.tr} + 1.6 \times W_{V.tr} + L_{V.tr}$ <-- EXCEEDS TO AASHTO STR III — $LC_{4V.tr} = 0 \times \text{kips}$

$LC_{5V.tr} := 1.25 \times D_{V.tr} + 1.35 \times L_{V.tr} + 0.4 \times W_{V.tr}$ <-- ALTERED TO AASHTO STR V — $LC_{5V.tr} = 0 \times \text{kips}$

$LC_{6V.tr} := 0.9 \times D_{V.tr} + 1.6 \times W_{V.tr}$ — $LC_{6V.tr} = 0 \times \text{kips}$

$LC_{7V.tr} := 0.9 \times D_{V.tr}$ — $LC_{7V.tr} = 0 \times \text{kips}$

Shear - Longitudinal

Dead Loads: $D_{V.lo} := V_{DC}$ Wind Loads: $W_{V.lo} := V_{wind}$ Live Loads: $L_{V.lo} := V_{LL}$

$LC_{1V.lo} := 1.4 \times (D_{V.lo})$ — $LC_{1V.lo} = 0.995 \times \text{kips}$

$LC_{2V.lo} := 1.25 \times (D_{V.lo}) + 1.75 \times (L_{V.lo})$ <-- INCREASED TO AASHTO STR I — $LC_{2V.lo} = 4.909 \times \text{kips}$

$LC_{3V.lo} := 1.2 \times D_{V.lo} + \max(L_{V.lo}, 0.8 \times W_{V.lo})$ — $LC_{3V.lo} = 3.15 \times \text{kips}$

$LC_{4V.lo} := 1.2 \times D_{V.lo} + 1.6 \times W_{V.lo} + L_{V.lo}$ <-- EXCEEDS TO AASHTO STR III — $LC_{4V.lo} = 5.715 \times \text{kips}$

$LC_{5V.lo} := 1.25 \times D_{V.lo} + 1.35 \times L_{V.lo} + 0.4 \times W_{V.lo}$ <-- ALTERED TO AASHTO STR V — $LC_{5V.lo} = 4.631 \times \text{kips}$

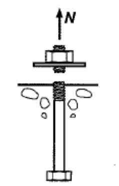
$LC_{6V.lo} := 0.9 \times D_{V.lo} + 1.6 \times W_{V.lo}$ — $LC_{6V.lo} = 3.205 \times \text{kips}$

$LC_{7V.lo} := 0.9 \times D_{V.lo}$ — $LC_{7V.lo} = 0.64 \times \text{kips}$

(a) steel strength of anchor in tension (D.5.1)

Nominal steel strength of Anchor in tension:

$N_{sa} := n_t \times A_{se.N} \times f_{uta}$ $\qquad$ $\boxed{N_{sa} = 26.216 \times kips}$

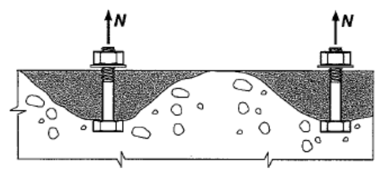


(i) Steel failure

(b) steel strength of anchor in shear (D.6.1)

For cast-in headed bolt and hooked bolt anchors:

$V_s := n_v \times 0.6 \times A_{se.V} \times f_{uta}$ $\qquad$ $\boxed{V_s = 21.353 \times kips}$



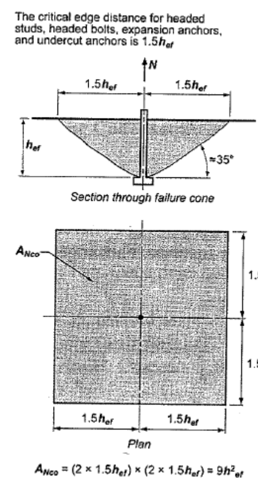
(iii) Concrete breakout

(c) concrete breakout strength of anchor in tension (D.5.2)

$$N_{cbg} = \frac{A_N}{A_{No}} \times \Psi_1 \times \Psi_2 \times \Psi_3 \times N_b$$

The projected area of the failure surface of a single anchor remote from edges:

$A_{Nco} := 9 \times h_{ef}^2$ $\qquad$ $\boxed{A_{Nco} = 144 \times in^2}$



$1.5 \times h_{ef} = 6 \times in$

The basic concrete breakout strength of a single anchor in tension in cracked concrete

for cast-in anchors k = 24, for post installed anchors k = 17

$Cast_{yes.no} := \text{"yes"}$ $\qquad$ $Post_{yes.no} := \text{"no"}$

$k_c = 24$

$N_b := k_c \times \sqrt{\frac{f'_c}{psi}} \times \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot (lb)$ $\qquad$ $\boxed{N_b = 12.143 \times kips}$

Projected area of the failure surface for a group of anchors:

A_{Nc} Projected area of the failure surface for the group of anchors

$inside_1 := \text{if}\left[\frac{s_1}{N_L - 1} > 2 \times 1.5 \times h_{ef}, 1.5 \times h_{ef} \times 2 \times (N_L - 1), s_1\right] = 0 \times in$

$inside_2 := \text{if}\left[\frac{s_2}{N_T - 1} > 2 \times 1.5 \times h_{ef}, 1.5 \times h_{ef} \times 2 \times (N_T - 1), s_2\right] = 10 \times in$

$proj_{1A} := \min(1.5 \times h_{ef}, c_{a1})$ $\qquad$ $proj_{1A} = 4 \times in$

$proj_{1B} := \min(1.5 \times h_{ef}, c_{b1})$ $\qquad$ $proj_{1B} = 4 \times in$ $\qquad$ $proj_{1A} + inside_1 + proj_{1B} = 8 \times in$

$proj_{2A} := \min(1.5 \times h_{ef}, c_{a2})$ $\qquad$ $proj_{2A} = 6 \times in$

$proj_{2B} := \min(1.5 \times h_{ef}, c_{b2})$ $\qquad$ $proj_{2B} = 6 \times in$ $\qquad$ $proj_{2A} + inside_2 + proj_{2B} = 22 \times in$

$A_{Nc} := (proj_{1A} + s_1 + proj_{1B}) \times (proj_{2A} + s_2 + proj_{2B}) \times \frac{n_t}{n_v}$ $\qquad$ $A_{Nc} = 176 \times in^2$

The modification factor for eccentrically loaded anchor groups (D.5.2.4):

$$\Psi_{ec.N} = \frac{1}{1+\frac{2\times e'_N}{3\times h_{ef}}} \leq 1$$

D.5.2.4 Eccentricity of normal force on a group of anchors:
"If the loading on an anchor group is such that only some anchors are in tension, only those anchors that are in tension shall be considered."
"Where eccentric loading exists about two axes, the modification factor shall be computed for each axis individually and the product of the factors used"

$e'_N := 0$

$$\Psi_{ec.N} := \min\left(\frac{1}{1+\frac{2\times e'_N}{3\times h_{ef}}}, 1\right) \qquad \boxed{\Psi_{ec.N} = 1}$$

D.5.2.5 Modification factor for edge effects:

$$\Psi_{ed.N} = 1 \text{ if } c_{min} \geq 1.5h_{ef} \qquad \Psi_{ed.N} = \left(0.7 + \frac{0.3\times c_{min}}{1.5\times h_{ef}}\right) \text{ if } c_{min} < 1.5h_{ef}$$

$c_{min} := \min(c_{a1}, c_{a2}, c_{b1}, c_{b2})$ $\qquad$ $c_{min} = 4\times in$

$$\Psi_{ed.N} := \text{if}\left(c_{min} \geq 1.5h_{ef}, 1, 0.7 + \frac{0.3\times c_{min}}{1.5\times h_{ef}}\right) \qquad \boxed{\Psi_{ed.N} = 0.9}$$

D.5.2.6 When an anchor is located in a region of a concrete member where analysis indicates no cracking at service load levels the following modification factor shall be permitted.

$\Psi_{c.N} = 1$ for cracking in service

Otherwise $\Psi_{c.N} = 1.25$ if anchors are cast-in and $\Psi_{c.N} = 1.4$ if anchors are post installed

$\boxed{Cast_{yes.no} = \text{"yes"}}$ $\qquad$ $\boxed{Post_{yes.no} = \text{"no"}}$

Cracking in service? $\qquad$ $Cracking_{yes.no} := \text{"yes"}$

$\boxed{\Psi_{c.N} = 1}$

The modification factor for post-installed anchors designed for uncracked concrete(D.5.2.7):

$c_{ac} := 2.5\times h_{ef}$ for undercut anchors (D.8.6)

$c_{ac} = 10\times in$

If c.ac is less than or equal to the smallest edge distance, the factor = 1
otherwise take maximum of the following equations

$$\frac{\min(c_{a1}, c_{a2})}{c_{ac}} = 0.4 \qquad \frac{1.5\times h_{ef}}{c_{ac}} = 0.6$$

$$\boxed{\Psi_{cp.N} := \text{if}\left[yes_{no}(Post_{yes.no}) = 0 \vee yes_{no}(Cracking_{yes.no}) = 1, 1, \left(\text{if}\left(c_{ac} \leq \min(c_{a1}, c_{a2}), 1.0, \max\left(\frac{\min(c_{a1}, c_{a2})}{c_{ac}}, \frac{1.5\times h_{ef}}{c_{ac}}\right)\right)\right)\right] = 1}$$

$A_{Nc} = 176 \times in^2$ $\Psi_{ec.N} = 1$ $N_b = 12.143 \times kips$

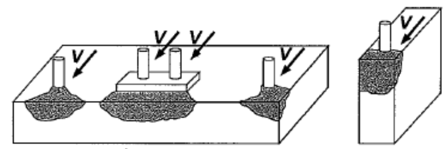
$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \times \Psi_{ec.N} \times \Psi_{ed.N} \times \Psi_{c.N} \times \Psi_{cp.N} \times N_b$$

$A_{Nco} = 144 \times in^2$ $\Psi_{ed.N} = 0.9$ $\Psi_{cp.N} = 1$

$\boxed{N_{cbg} = 13.357 \times kips}$ $\Psi_{c.N} = 1$

(d) concrete breakout strength of anchor in shear (D.6.2)

$$V_{cb} = \frac{A_{V.c}}{A_{Vco}} \times \Psi_{ed.V} \times \Psi_{c.V} \times \Psi_{h.V} \times V_b$$



(iii) Concrete breakout

Is the concrete Normal weight or Light weight?

$Normal_{Weight} := "yes"$ $Lightweight := "no"$

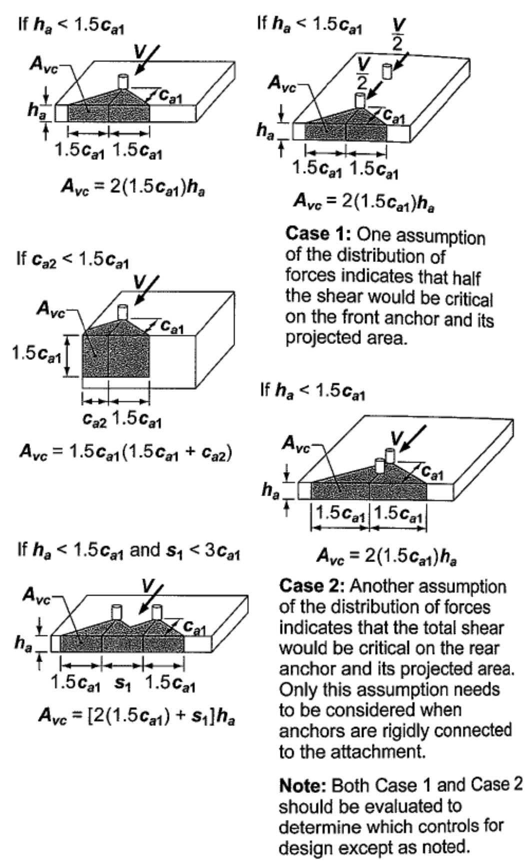
$\boxed{\lambda = 1}$

Longitudinal Shear Capacity:

Projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. The depth of this failure surface is permitted to be the smaller of 1.5 x c1 or the full depth of the member

Where anchors are influenced by three or more edges, the value of c.a1 used shall not exceed the greatest of c.a2/1.5 in either direction, h.a/1.5; and one third of the maximum spacing between anchors within the group. (D.6.2.4)

If hairpins are provided to prevent concrete breakout, Seperate analysis must be conducted to determine the adequacy of the reinforcing.



Depth Check Case 1

$h_a = 6 \times in$ $1.5 \times c_{a1} = 6 \times in$

$\min(h_a, 1.5 \times c_{a1}) = 6 \times in$

Breakout Width Case 1

$c_{a2} = 999 \times in$ $s_2 = 10 \times in$ $c_{b2} = 999 \times in$

$\min(c_{a2}, 1.5 \times c_{a1}) = 6 \times in$ $\min[s_2, 2 \times (1.5 \times c_{a1}) \times (N_T - 1)] = 10 \times in$ $\min(c_{b2}, 1.5 \times c_{a1}) = 6 \times in$

$Edge_2 := if(c_{a2} < 1.5 \times c_{a1}, "yes", "no") = "no"$ $Edge_3 := if(c_{b2} < 1.5 \times c_{a1}, "yes", "no") = "no"$

$\boxed{Edges = "Anchors influenced by less than 3 edges"}$

$A_{vc.case1} = 132 \times in^2$

Depth Check Case 2

$h_a = 6 \times in$ $1.5 \times (c_{a1} + s_1) = 6 \times in$

$\min[h_a, 1.5 \times (c_{a1} + s_1)] = 6 \times in$

Breakout Width Case 2

$c_{a2} = 999 \times in$ $s_2 = 10 \times in$ $c_{b2} = 999 \times in$

$\min[c_{a2}, 1.5 \times (c_{a1} + s_1)] = 6 \times in$ $\min[s_2, 2 \times [1.5 \times (c_{a1} + s_1)] \times (N_T - 1)] = 10 \times in$ $\min[c_{b2}, 1.5 \times (c_{a1} + s_1)] = 6 \times in$

$Edge_2 := if[c_{a2} < 1.5 \times (c_{a1} + s_1), "yes", "no"] = "no"$ $Edge_3 := if[c_{b2} < 1.5 \times (c_{a1} + s_1), "yes", "no"] = "no"$

$\boxed{Edges = "Anchors influenced by less than 3 edges"}$

$A_{vc.case2} = 132 \times in^2$

c.a1 with possible change for 3 edge effects

$c_{a1} = 4 \times in$

$$c_{a1'.case1} := if\left[(Edge_2 = "yes" \wedge Edge_3 = "yes"), \min\left[c_{a1}, \max\left[\frac{c_{a2}}{1.5}, \frac{c_{b2}}{1.5}, \frac{h_a}{1.5}, \frac{s_2}{(N_T - 1) \times 3}\right]\right], c_{a1}\right]$$

$$c_{a1'.case2} := if\left[(Edge_2 = "yes" \wedge Edge_3 = "yes"), \min\left[c_{a1}, \max\left[\frac{c_{a2}}{1.5}, \frac{c_{b2}}{1.5}, \frac{h_a}{1.5}, \frac{s_2}{(N_T - 1) \times 3}\right]\right], c_{a1} + s_1\right]$$

$c_{a1'.case1} = 4 \times in$

$c_{a1'.case2} = 4 \times in$

D.6.2.5 The modification factor for eccentrically loaded anchor groups

$$\Psi_{ec.V} = \frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1}}} \le 1$$

Eccentricity $\boxed{e'_v := 0}$

$$\Psi_{ec.V.case1} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1'.case1}}}, 1\right)$$ $\boxed{\Psi_{ec.V.case1} = 1}$

$$\Psi_{ec.V.case2} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a1'.case2}}}, 1\right)$$ $\boxed{\Psi_{ec.V.case2} = 1}$

D.6.2.6 The modification factor for edge effect

$\Psi_{ed.V} = 1 \quad \text{if} \quad c_{a2} \geq 1.5 \times c_{a1'}$

$\Psi_{ed.V} = \left(0.7 + \frac{0.3 \times c_{a2}}{1.5 \times c_{a1'}}\right) \quad \text{if} \quad c_{a2} < 1.5h_e$

$\Psi_{ed.V.case1} := \text{if}\left(c_{a2} \geq 1.5 \times c_{a1'.case1}, 1, 0.7 + \frac{0.3 \times c_{a2}}{1.5 \times c_{a1'.case1}}\right)$ $\quad \Psi_{ed.V.case1} = 1$

$\Psi_{ed.V.case2} := \text{if}\left(c_{a2} \geq 1.5 \times c_{a1'.case2}, 1, 0.7 + \frac{0.3 \times c_{a2}}{1.5 \times c_{a1'.case2}}\right)$ $\quad \Psi_{ed.V.case2} = 1$

D.6.2.7 Anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the following modification factor shall be used.

$Cracking_{yes.no} = \text{"yes"}$

$\Psi_{c.V} = 1.4$ where no cracking occurs in service

Otherwise $\Psi_{c.V} = 1.0$ if anchors are cast in concrete without supplementary reinforcement or edge reinforcement smaller than a No 4 bar.

$\Psi_{c.V} = 1.2$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar.

$\Psi_{c.V} = 1.4$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar and with stirrups spaced at not more than 4"

Is Anchor reinforced by at least #4 bars? $\quad Reinf_{with.no4} := \text{"yes"}$

Is anchor enclosed by stirrups at 4" maximum spacing? $\quad Stirrups_{4inch.spa.max} := \text{"no"}$

Are Haripins provided to prevent Concrete Breakout? $\quad Hairpins_{Long.yesno} := \text{"no"}$ $\quad \Psi_{c.V} = 1.2$

$A_{Vco.loLoad.case1} := 4.5 \times c_{a1'.case1}^{\;2}$ $\quad A_{Vco.loLoad.case1} = 72 \times in^2$

$A_{Vco.loLoad.case2} := 4.5 \times c_{a1'.case2}^{\;2}$ $\quad A_{Vco.loLoad.case2} = 72 \times in^2$

D.6.2.8 The modification factor for anchors located in a concrete member where h.a<1.5*c.a1

$h_a = 6 \times in \qquad c_{a1'.case1} = 4 \times in \qquad c_{a1'.case2} = 4 \times in$

$\Psi_{h.V.case1} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a1'.case1}, \sqrt{\frac{1.5 \times c_{a1'.case1}}{h_a}}, 1\right), 1\right) = 1$

$\Psi_{h.V.case2} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a1'.case2}, \sqrt{\frac{1.5 \times c_{a1'.case2}}{h_a}}, 1\right), 1\right) = 1$

The basic concrete breakout strength V.b in shear of a single anchor in cracked concrete shall not exceed (D6.2.2):

$$V_b = \left[7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a}\right] \times \lambda \times \sqrt{f'_c} \times (c_{a1})^{1.5}$$

Load bearing length of anchor for shear (for anchors with constant stiffness)

$l_e := h_{ef} \qquad l_e = 4 \times in \qquad d_a = 0.625 \times in$

$V_{b.case1} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a1'.case1} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb)$ $\quad V_{b.case1} = 4.059 \times kips$

$V_{b.case2} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a1'.case2} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb)$ $\quad V_{b.case2} = 4.059 \times kips$

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$A_{vc.case1} = 132 \times in^2$ $\qquad \Psi_{ec.V.case1} = 1$ $\qquad \Psi_{c.V} = 1.2$

$A_{Vco.loLoad.case1} = 72 \times in^2$ $\qquad \Psi_{ed.V.case1} = 1$ $\qquad \Psi_{h.V.case1} = 1$

$$V_{cbg.Long.case1} := \frac{A_{vc.case1}}{A_{Vco.loLoad.case1}} \times \Psi_{ec.V.case1} \times \Psi_{ed.V.case1} \times \Psi_{c.V} \times \Psi_{h.V.case1} \times V_{b.case1}$$

$\boxed{V_{cbg.Long.case1} = 8.929 \times kips}$

$A_{vc.case2} = 132 \times in^2$ $\qquad \Psi_{ec.V.case2} = 1$ $\qquad \Psi_{c.V} = 1.2$

$A_{Vco.loLoad.case2} = 72 \times in^2$ $\qquad \Psi_{ed.V.case2} = 1$ $\qquad \Psi_{h.V.case2} = 1$

$$V_{cbg.Long.case2} := \frac{A_{vc.case2}}{A_{Vco.loLoad.case2}} \times \Psi_{ec.V.case2} \times \Psi_{ed.V.case2} \times \Psi_{c.V} \times \Psi_{h.V.case2} \times V_{b.case2}$$

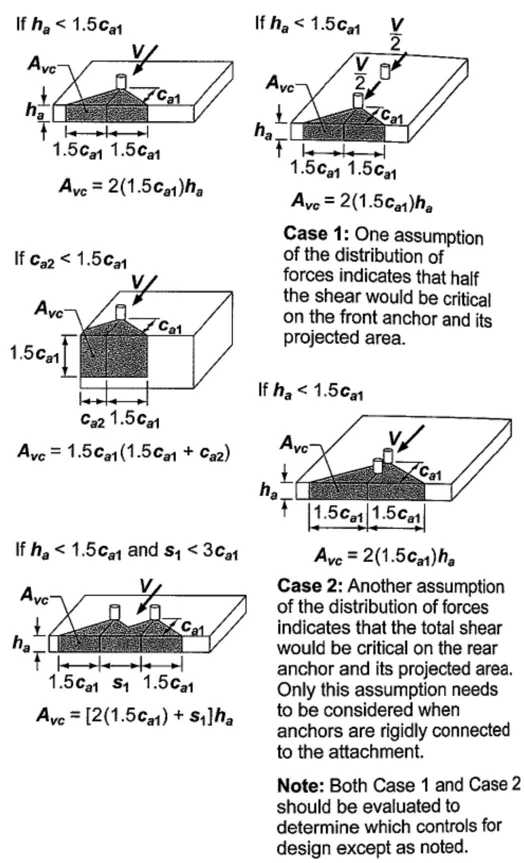
$\boxed{V_{cbg.Long.case2} = 8.929 \times kips}$

Transverse Shear Capacity:

Projected area of the failure surface on the side of the concrete member at its edge for a single anchor or a group of anchors. The depth of this failure surface is permitted to be the smaller of 1.5 x c1 or the full depth of the member

Where anchors are influenced by three or more edges, the value of c.a1 used shall not exceed the greatest of c.a2/1.5 in either direction, h.a/1.5; and one third of the maximum spacing between anchors within the group. (D.6.2.4)

If hairpins are provided to prevent concrete breakout, Seperate analysis must be conducted to determine the adequacy of the reinforcing.



Depth Check Case 1

$h_a = 6 \times in$ $\qquad 1.5 \times c_{a2} = 1.498 \times 10^3 \times in$

$\min(h_a, 1.5 \times c_{a2}) = 6 \times in$

Breakout Width Case 1

$c_{a1} = 4 \times in$ $\quad s_1 = 0 \times in$ $\quad c_{b1} = 4 \times in$

$\min(c_{a1}, 1.5 \times c_{a2}) = 4 \times in$ $\quad \min[s_1, 2 \times (1.5 \times c_{a2}) \times (N_L - 1)] = 0 \times in$ $\quad \min(c_{b1}, 1.5 \times c_{a2}) = 4 \times in$

$Edge_2 := if(c_{a1} < 1.5 \times c_{a2}, \text{"yes"}, \text{"no"}) = \text{"yes"}$ $\quad Edge_3 := if(c_{b1} < 1.5 \times c_{a2}, \text{"yes"}, \text{"no"}) = \text{"yes"}$

$\boxed{Edges = \text{"Anchors influenced by 3 edges"}}$

$A_{vc.case1} = 48 \times in^2$

Depth Check Case 2

$h_a = 6 \times in$ $\quad 1.5 \times (c_{a2} + s_2) = 1.513 \times 10^3 \times in$

$\min[h_a, 1.5 \times (c_{a2} + s_2)] = 6 \times in$

Breakout Width Case 2

$c_{a1} = 4 \times in$ $\quad s_1 = 0 \times in$ $\quad c_{b1} = 4 \times in$

$\min[c_{a1}, 1.5 \times (c_{a2} + s_2)] = 4 \times in$ $\quad \min[s_1, 2 \times [1.5 \times (c_{a2} + s_2)] \times (N_T - 1)] = 0 \times in$ $\quad \min[c_{b1}, 1.5 \times (c_{a2} + s_2)] = 4 \times in$

$Edge_2 := if[c_{a1} < 1.5 \times (c_{a2} + s_2), \text{"yes"}, \text{"no"}] = \text{"yes"}$ $\quad Edge_3 := if[c_{b1} < 1.5 \times (c_{a2} + s_2), \text{"yes"}, \text{"no"}] = \text{"yes"}$

$\boxed{Edges = \text{"Anchors influenced by 3 edges"}}$

$A_{vc.case2} = 48 \times in^2$

c.a2 with possible change for 3 edge effects

$c_{a2} = 999 \times in$

$$c_{a2'.case1} := if\left[(Edge_2 = \text{"yes"} \wedge Edge_3 = \text{"yes"}), \min\left[c_{a2}, \max\left[\frac{c_{a1}}{1.5}, \frac{c_{b1}}{1.5}, \frac{h_a}{1.5}, \frac{s_1}{(N_L - 1) \times 3}\right]\right], c_{a2}\right]$$

$$c_{a2'.case2} := if\left[(Edge_2 = \text{"yes"} \wedge Edge_3 = \text{"yes"}), \min\left[c_{a2}, \max\left[\frac{c_{a1}}{1.5}, \frac{c_{b1}}{1.5}, \frac{h_a}{1.5}, \frac{s_1}{(N_L - 1) \times 3}\right]\right], c_{a2} + s_2\right]$$

$c_{a2'.case1} = 4 \times in$

$c_{a2'.case2} = 4 \times in$

D.6.2.5 The modification factor for eccentrically loaded anchor groups

$$\Psi_{ec.V} = \frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2}}} \le 1$$

Eccentricity $\quad e'_v := 0$

$$\Psi_{ec.V.case1} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2'.case1}}}, 1\right) \qquad \boxed{\Psi_{ec.V.case1} = 1}$$

$$\Psi_{ec.V.case2} := \min\left(\frac{1}{1 + \frac{2 \times e'_v}{3 \times c_{a2'.case2}}}, 1\right) \qquad \boxed{\Psi_{ec.V.case2} = 1}$$

D.6.2.6 The modification factor for edge effect

$\Psi_{ed.V} = 1$ if $c_{a1} \geq 1.5 \times c_{a2'}$ $\qquad \Psi_{ed.V} = \left(0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'}}\right)$ if $c_{a1} < 1.5h_e$

$$\Psi_{ed.V.case1} := \text{if}\left(c_{a1} \geq 1.5 \times c_{a2'.case1}, 1, 0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'.case1}}\right) \qquad \Psi_{ed.V.case1} = 0.9$$

$$\Psi_{ed.V.case2} := \text{if}\left(c_{a1} \geq 1.5 \times c_{a2'.case2}, 1, 0.7 + \frac{0.3 \times c_{a1}}{1.5 \times c_{a2'.case2}}\right) \qquad \Psi_{ed.V.case2} = 0.9$$

D.6.2.7 Anchors located in a region of a concrete member where analysis indicates no cracking at service loads, the following modification factor shall be used.

$\Psi_{c.V} = 1.4$ where no cracking occurs in service $\qquad Cracking_{yes.no} = \text{"yes"}$

Otherwise $\Psi_{c.V} = 1.0$ if anchors are cast in concrete without supplementary reinforcement or edge reinforcement smaller than a No 4 bar.

$\Psi_{c.V} = 1.2$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar.

$\Psi_{c.V} = 1.4$ if anchors are cast in concrete with edge reinforcement of at least a No 4 bar and with stirrups spaced at not more than 4"

Is Anchor reinforced by at least #4 bars? $\qquad Reinf_{with.no4} := \text{"yes"}$

Is anchor enclosed by stirrups at 4" maximum spacing? $\qquad Stirrups_{4inch.spa.max} := \text{"no"}$

Are Haripins provided to prevent Concrete Breakout? $\qquad Hairpins_{Trans.yesno} := \text{"no"}$ $\qquad \Psi_{c.V} = 1.2$

$A_{Vco.trLoad.case1} := 4.5 \times c_{a2'.case1}^{\,2} \qquad A_{Vco.trLoad.case1} = 72 \times in^2$

$A_{Vco.trLoad.case2} := 4.5 \times c_{a2'.case2}^{\,2} \qquad A_{Vco.trLoad.case2} = 72 \times in^2$

D.6.2.8 The modification factor for anchors located in a concrete member where h.a<1.5*c.a1

$h_a = 6 \times in \qquad c_{a2'.case1} = 4 \times in \qquad c_{a2'.case2} = 4 \times in$

$$\Psi_{h.V.case1} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a2'.case1}, \sqrt{\frac{1.5 \times c_{a2'.case1}}{h_a}}, 1\right), 1\right) = 1$$

$$\Psi_{h.V.case2} := \max\left(\text{if}\left(h_a < 1.5 \times c_{a2'.case2}, \sqrt{\frac{1.5 \times c_{a2'.case2}}{h_a}}, 1\right), 1\right) = 1$$

The basic concrete breakout strength V.b in shear of a single anchor in cracked concrete shall not exceed (D6.2.2):

$$V_b = \left[7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a}\right] \times \lambda \times \sqrt{f'_c} \times (c_{a2})^{1.5}$$

Load bearing length of anchor for shear (for anchors with constant stiffness)

$l_e := h_{ef} \qquad l_e = 4 \times in \qquad d_a = 0.625 \times in$

$$V_{b.case1} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a2'.case1} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb) \qquad V_{b.case1} = 4.059 \times kips$$

$$V_{b.case2} := 7 \times \left(\frac{l_e}{d_a}\right)^{0.2} \times \sqrt{d_a \cdot \left(\frac{1}{in}\right)} \times \lambda \times \sqrt{f'_c \cdot \left(\frac{1}{psi}\right)} \times \left[c_{a2'.case2} \cdot \left(\frac{1}{in}\right)\right]^{1.5} \bullet (lb) \qquad V_{b.case2} = 4.059 \times kips$$

$A_{vc.case1} = 48 \times in^2$ $\Psi_{ec.V.case1} = 1$ $\Psi_{c.V} = 1.2$

$A_{Vco.trLoad.case1} = 72 \times in^2$ $\Psi_{ed.V.case1} = 0.9$ $\Psi_{h.V.case1} = 1$

$$V_{cbg.Trans.case1} := \frac{A_{vc.case1}}{A_{Vco.trLoad.case1}} \times \Psi_{ec.V.case1} \times \Psi_{ed.V.case1} \times \Psi_{c.V} \times \Psi_{h.V.case1} \times V_{b.case1}$$

$\boxed{V_{cbg.Trans.case1} = 2.922 \times kips}$

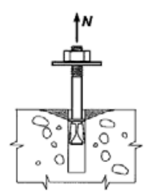
$A_{vc.case2} = 48 \times in^2$ $\Psi_{ec.V.case2} = 1$ $\Psi_{c.V} = 1.2$

$A_{Vco.trLoad.case2} = 72 \times in^2$ $\Psi_{ed.V.case2} = 0.9$ $\Psi_{h.V.case2} = 1$

$$V_{cbg.Trans.case2} := \frac{A_{vc.case2}}{A_{Vco.trLoad.case2}} \times \Psi_{ec.V.case2} \times \Psi_{ed.V.case2} \times \Psi_{c.V} \times \Psi_{h.V.case2} \times V_{b.case2}$$

$\boxed{V_{cbg.Trans.case2} = 2.922 \times kips}$

(e) pullout strength of anchor in tension (D.5.3)



$N_{pn} = \Psi_{c.P} \times N_p$

$Headed_{yes.no} = "yes"$ $Hooked_{yes.no} = "no"$

D.5.3.4 The pullout strength in tension of a single headed stud or headed bolt shall not exceed:

$N_p = A_{brg} \times 8 \times f'_c$ $A_{brg} \times 8 \times f'_c = 32 \times kips$

D.5.3.5 The pullout strength in tension of a single hooked bolt shall not exceed:

$N_p = 0.9 \times f'_c \times e_h \times d_a$ $0.9 \times f'_c \times e_h \times d_a = 0 \times kips$

$N_p := yes_{no}(Headed_{yes.no}) \times (A_{brg} \times 8 \times f'_c) + yes_{no}(Hooked_{yes.no}) \times (0.9 \times f'_c \times e_h \times d_a)$ $\boxed{N_p = 32 \times kips}$

D.5.3.6 For an anchor located in a region of a concrete member where analysis indicates no cracking at service load levels

$\Psi_{c.P} = 1.4$ where no cracking is expected under service load

$\Psi_{c.P} = 1.0$ when cracking is expected

$Cracking_{yes.no} = "yes"$ $\boxed{\Psi_{c.P} = 1}$

$N_{pn} := \Psi_{c.P} \times N_p$ $\boxed{N_{pn} = 32 \times kips}$

$N_{png} := n_t \times N_{pn}$ $\boxed{N_{png} = 64 \times kips}$

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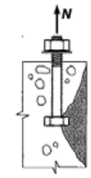
(f) concrete side-face blowout strength of anchor in tension (D.5.4)



(iv) Side-face blowout

For a single headed anchor with deep embedment close to an edge.

Applicable for : $h_{ef} > 2.5 \times \min(c_{a1}, c_{a2})$

$h_{ef} = 4 \times in$ $\qquad 2.5 \times \min(c_{a1}, c_{a2}) = 10 \times in$ $\qquad \lambda = 1$

D.5.4.1 For a Single headed anchor with deep embedment close to an edge:

if c.a2 for the single headed anchor is less than 3.ca1, the value N.sb shall be factored by:

$$N_{sb} := \text{if}\left[h_{ef} > 2.5 \times \min(c_{a1}, c_{a2}), \left[160 \times \frac{\min(c_{a1}, c_{a2})}{in} \times \sqrt{\frac{A_{brg}}{in^2}} \times \lambda \times \sqrt{\frac{f'_c}{psi}} \bullet (lb)\right], 1 \times 10^{10} kips\right]$$

$\boxed{N_{sb} = 1 \times 10^{10} \times kips}$

$$\psi_{multiside.blowout} := \text{if}\left[\max(c_{a1}, c_{a2}) < 3 \times \min(c_{a1}, c_{a2}), \left(1 + \max\left(\min\left(\frac{\max(c_{a1}, c_{a2})}{\min(c_{a1}, c_{a2})}, 3\right), 1\right)\right) \times \frac{1}{4}, 1\right] = 1$$

$$N_{sb'} := \psi_{multiside.blowout} \times N_{sb} + 1 \times 10^{10} kips \times yes_{no}(Hooked_{yes.no})$$

$\boxed{N_{sb'} = 1 \times 10^{10} \times kips}$

D.5.4.2 For multiple headed anchors with deep embedment close to an edge, and spacing less than 6c

$$N_{sbg} = \left(1 + \frac{s_{edge}}{6 \times \min(c_{a1}, c_{a2})}\right) \times N_{sb}$$

Spacing of outer anchors along the edge in the group $\boxed{s_{edge} := s_1}$ $\qquad s_{edge} = 0 \times in$

$$N_{sbg} := \left(1 + \frac{s_{edge}}{6 \times \min(c_{a1}, c_{a2})}\right) \times N_{sb}$$

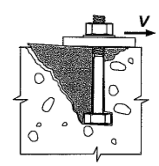
$\boxed{N_{sbg} = 1 \times 10^{10} \times kips}$

$$N_{sb'.g'} := \left(N_{sbg} + kips \times yes_{no}(Hooked_{yes.no})\right) \times (n_t > 1) + N_{sb'} \times (n_t = 1)$$

$\boxed{N_{sb'.g'} = 1 \times 10^{10} \times kips}$

(g) concrete pryout strength of anchor in shear (D.6.3)

The nominal pryout strength shall not exceed:



(ii) Concrete pryout for anchors far from a free edge

$V_{cp} = k_{cp} \times N_{cbg}$

$k_{cp} = 1.0$ for $h_{ef} < 2.5in$ $\qquad h_{ef} = 4 \times in$

$k_{cp} = 2.0$ for $h_{ef} \geq 2.5in$

$k_{cp} := \text{if}(h_{ef} < 2.5in, 1.0, 2.0)$ $\qquad \boxed{k_{cp} = 2}$

D.5.2.1 The Nominal Concrete breakout strength of an anchor in tension

$N_{cbg} = 13.357 \times kips$

$N_{cbg} = 13.357 \times kips$

$V_{cpg} := k_{cp} \times N_{cbg}$ $\qquad \boxed{V_{cpg} = 26.715 \times kips}$

Summary

$Reinf_{with.no4} = \text{"yes"}$

$Stirrups_{4inch.spa.max} = \text{"no"}$

Seismic Restrictions Apply?

$Seismic_{Restrictions} := \text{"no"}$

Force Seismic Concrete? (severe reduction)

$Seismic_{Concrete} := \text{"no"}$

$Seismic_{input} = \text{"ok"}$

$LC_{1T} = 0 \times kips$

$LC_{2T} = 0 \times kips$

$LC_{3T} = 0 \times kips$

$LC_{4T} = 0 \times kips$

$LC_{5T} = 0 \times kips$

$LC_{6T} = 0 \times kips$

$LC_{7T} = 0 \times kips$

Tension

(a) steel strength of anchor in tension (D.5.1)

$N_{sa} = 26.216 \times kips$

(c) concrete breakout strength of anchor in tension (D.5.2)

$N_{cbg} = 13.357 \times kips$

(e) pullout strength of anchor in tension (D.5.3)

$N_{png} = 64 \times kips$

(f) concrete side-face blowout strength of anchor in tension (D.5.4)

$N_{sb'.g'} = 1 \times 10^{10} \times kips$

$Govern_{Tension} = \text{"Governed by Concrete"}$

Strength Reduction factor assuming anchor has low sensitivity to installation and has high reliability

$\phi_T = 0.7$

$$\phi Pn := \phi_T \times \min\left[N_{sa} \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), N_{cbg}, N_{png}, N_{sb'.g'}\right]$$

$\phi Pn = 9.35 \times kips$

$\frac{LC_{1T}}{\phi Pn} = 0$ $\quad \frac{LC_{2T}}{\phi Pn} = 0$ $\quad \frac{LC_{3T}}{\phi Pn} = 0$

$\frac{LC_{4T}}{\phi Pn} = 0$ $\quad \frac{LC_{5T}}{\phi Pn} = 0$ $\quad \frac{LC_{6T}}{\phi Pn} = 0$ $\quad \frac{LC_{7T}}{\phi Pn} = 0$

Shear

(b) steel strength of anchor in shear (D.6.1)

$V_s = 21.353 \times kips$

Transverse	Longitudinal
$LC_{1V.tr} = 0 \times kips$	$LC_{1V.lo} = 0.995 \times kips$
$LC_{2V.tr} = 0 \times kips$	$LC_{2V.lo} = 4.909 \times kips$
$LC_{3V.tr} = 0 \times kips$	$LC_{3V.lo} = 3.15 \times kips$
$LC_{4V.tr} = 0 \times kips$	$LC_{4V.lo} = 5.715 \times kips$
$LC_{5V.tr} = 0 \times kips$	$LC_{5V.lo} = 4.631 \times kips$
$LC_{6V.tr} = 0 \times kips$	$LC_{6V.lo} = 3.205 \times kips$
$LC_{7V.tr} = 0 \times kips$	$LC_{7V.lo} = 0.64 \times kips$

(d) concrete breakout strength of anchor in shear (D.6.2)

$V_{cbg.Long.case1} = 8.929 \times kips$ $\quad V_{cbg.Trans.case1} = 2.922 \times kips$

$V_{cbg.Long.case2} = 8.929 \times kips$ $\quad V_{cbg.Trans.case2} = 2.922 \times kips$

(g) concrete pryout strength of anchor in shear (D.6.3)

$V_{cpg} = 26.715 \times kips$

$Govern_{Long.case}$ = "Cases are identical" $\quad Govern_{Trans.case}$ = "Cases are identical"

$Govern_{Long.Shear}$ = "Governed by Concrete" $\quad Govern_{Trans.Shear}$ = "Governed by Concrete"

$\phi_{V.Long} = 0.75$ $\quad \phi_{V.Trans} = 0.75$

Transverse

$$\phi Vn_{tr} := \phi_{V.Trans} \times \left| \begin{array}{l} V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Trans.case1}, V_{cpg}\right] \\ V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Trans.case2}, V_{cpg}\right] \text{ if } Govern_{Trans.case} = \text{"Governed by} \end{array}\right.$$

$\phi Vn_{tr} = 2.192 \times kips$

$\frac{LC_{1V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{2V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{3V.tr}}{\phi Vn_{tr}} = 0$

$\frac{LC_{4V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{5V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{6V.tr}}{\phi Vn_{tr}} = 0$ $\quad \frac{LC_{7V.tr}}{\phi Vn_{tr}} = 0$

Longitudinal

$$\phi Vn_{lo} := \phi_{V.Long} \times \left| \begin{array}{l} V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Long.case1}, V_{cpg}\right] \\ V \leftarrow \min\left[V_s \times \left(1 + 999 \times yes_{no}\left(Seismic_{Concrete}\right)\right), V_{cbg.Long.case2}, V_{cpg}\right] \text{ if } Govern_{Long.case} = \text{"Governed by} \end{array}\right.$$

$\phi Vn_{lo} = 6.697 \times kips$

$\frac{LC_{1V.lo}}{\phi Vn_{lo}} = 0.149$ $\quad \frac{LC_{2V.lo}}{\phi Vn_{lo}} = 0.733$ $\quad \frac{LC_{3V.lo}}{\phi Vn_{lo}} = 0.47$

$\frac{LC_{4V.lo}}{\phi Vn_{lo}} = 0.853$ $\quad \frac{LC_{5V.lo}}{\phi Vn_{lo}} = 0.692$ $\quad \frac{LC_{6V.lo}}{\phi Vn_{lo}} = 0.479$ $\quad \frac{LC_{7V.lo}}{\phi Vn_{lo}} = 0.096$

Combine Tension and Shear: (D.7)

D.7.1 - if $V_{ua} \le 0.2 \times \varphi \times V_n$ then full strength in tension shall be permitted

D.7.2 - if $N_{ua} \le 0.2 \times \varphi \times N_n$ then full strength in shear shall be permitted

D.7.3 - otherwise $\frac{N_{ua}}{\varphi N_n} + \frac{V_{ua}}{\varphi V_n} \le 1.2$

Combined Stress Ratio less than 1.0 is Satisfactory

Transverse Combination:

$$CSR_{T1} := \frac{\left(\frac{LC_{1T}}{\phi Pn}\right) \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) + \frac{LC_{1V.tr}}{\phi Vn_{tr}} \times \left(\frac{LC_{1V.tr}}{\phi Vn_{tr}} > 0.2\right)}{1 + 0.2 \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) \times \left(\frac{LC_{1V.tr}}{\phi Vn_{tr}} > 0.2\right)} = 0$$

Longitudinal Combination:

$$CSR_{L1} := \frac{\left(\frac{LC_{1T}}{\phi Pn}\right) \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) + \frac{LC_{1V.lo}}{\phi Vn_{lo}} \times \left(\frac{LC_{1V.lo}}{\phi Vn_{lo}} > 0.2\right)}{1 + 0.2 \times \left(\frac{LC_{1T}}{\phi Pn} > 0.2\right) \times \left(\frac{LC_{1V.lo}}{\phi Vn_{lo}} > 0.2\right)} = 0$$

Transverse	Longitudinal
$CSR_{T2} = 0$	$CSR_{L2} = 0.733$
$CSR_{T3} = 0$	$CSR_{L3} = 0.47$
$CSR_{T4} = 0$	$CSR_{L4} = 0.853$
$CSR_{T5} = 0$	$CSR_{L5} = 0.692$
$CSR_{T6} = 0$	$CSR_{L6} = 0.479$
$CSR_{T7} = 0$	$CSR_{L7} = 0$

$Seismic_{restrictions}$ = "D.3.3 seismic restrictions do not apply"

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Design Abutment Extension Reinforcing

$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Bar size (US number): $bar_{no} := 7$ $d_{no_{bar_{no}}} = 0.875 \times in$ $diam_{deform_{bar_{no}}} = 1 \times in$ $A_{no_{bar_{no}}} = 0.6 \times in^2$

$clear := 1.5in - d_{no_{bar_{no}}} \times 0.5 = 1.063 \times in$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft + 2in$ **Bars per Layer:** $n_{bars} := 3$ $spacing := \frac{w_{mem}}{n_{bars}} = 4.667 \times in$

Height of member $h_{mem} := 36in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 1.8 \times in^2$ $clear_{spacing} = 3.667 \times in$

Design Moment: $M_{des} := (1.25 \times DC_{reaction} + 1.75 \times LL_{reaction}) \times 40in = 210.086\ ft \times kips$

Service Moment: $M_{service} := (DC_{reaction} + LL_{reaction}) \times 40in = 139.176\ ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 34.5 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 1.75 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 1.388 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 210.086 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 2.269 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 270.261 \times kip\ ft$

$Check_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

$\frac{\phi M_n}{M_{des}} = 128.6 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 2.592 \times 10^3 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 159.84 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 3$ $bar_{no} = 7$

in $full_{layers} = 1$ (total bars $n = 3$)

spaced at $spacing = 4.667 \times in$

$w_{mem} = 1.167\ ft$ x $h_{mem} = 3\ ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 223.776 \times kip \times ft$

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Distribution of Reinforcement (LRFD 5.7.3.4)
The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[Quadratic_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 7.276 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 32.075 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 52.07 \times kips$

Stress in the steel at service limit state

$$\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 28.928 \times ksi}$$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 1.063 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.043$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 15.268 \times in}$$

$$spacing \times (1 + bundles) = 4.667 \times in$$

$$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

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BALLARD - 10FT - SEGMENT 7

New construction

$\gamma_c := 155\text{pcf}$

Walkway will be supported along outside edge and connect to the new barrier

New slab will be designed as PIN-PIN for positive moments, and PIN-FIX for the negative moment connection to the barrier

Thickness of New Slab: $t := 4\text{in}$

$w_{DC} := t \times \gamma_c = 51.7 \times \text{psf}$

$w_{LL} := 75\text{psf}$

PIN-PIN

$$M7_{DC.pos} := \frac{w_{DC} \times (10\text{ft})^2}{8} = 645.8 \times \frac{\text{ft} \times \text{lb}}{\text{ft}}$$

$$M7_{LL.pos} := \frac{w_{LL} \times (10\text{ft})^2}{8} = 937.5 \times \frac{\text{ft} \times \text{lb}}{\text{ft}}$$

PIN-FIX

$$M7_{DC.neg} := \frac{w_{DC} \times (10\text{ft})^2}{8} = 645.8 \times \frac{\text{ft} \times \text{lb}}{\text{ft}}$$

$$M7_{LL.neg} := \frac{w_{LL} \times (10\text{ft})^2}{8} = 937.5 \times \frac{\text{ft} \times \text{lb}}{\text{ft}}$$

Positive and Negative moments are the same. Slab design shall have a symmetric capacity.

Service Moment:

$$M_{serv.slab} := M7_{DC.pos} + M7_{LL.pos} = 1.583 \times \frac{\text{kip} \times \text{ft}}{\text{ft}}$$

Factored Moment:

$$M_{fact.slab} := 1.25 \times M7_{DC.pos} + 1.75 \times M7_{LL.pos} = 2.448 \times \frac{\text{kip} \times \text{ft}}{\text{ft}}$$

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$E_s := 29000ksi$ $f'_c := 4ksi$ $F_y := 60ksi$

Design New slab for Maximum Positive Moment

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in - \frac{d_{no_{bar_{no}}}}{2} = 1.75 \times in$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 3$ $spacing := \frac{w_{mem}}{n_{bars}} = 4 \times in$

Height of member $h_{mem} := 4in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.6 \times in^2$ $clear_{spacing} = 3.438 \times in$

Design Moment: $M_{des} := M_{fact.slab} \times w_{mem} = 2.448\, ft \times kips$

Service Moment: $M_{service} := M_{serv.slab} \times w_{mem} = 1.583\, ft \times kips$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 2 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.451 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.307 \times in^2$

Check compression for full yielding of actual steel

$M_{des} = 2.448 \times kip \times ft$

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.882 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$ $\phi M_n = 4.209 \times kip\, ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = "SATISFACTORY"$ $\frac{\phi M_n}{M_{des}} = 171.9 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 740 \times psi$

Section Modulus: $S := \frac{h_{mem}^{\ 2}}{6}$ $S = 32 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 1.973 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "SATISFACTORY"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 3$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 3$)

spaced at $spacing = 4 \times in$

$w_{mem} = 1\, ft$ x $h_{mem} = 0.333\, ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 2.368 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf} \right) \right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2} \right) = \left(A_s \times w_{mem} \right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2} \right) \times x^2 + \left(A_s \times w_{mem} \times n \right) \times x - \left(A_s \times w_{mem} \times n \times d \right) = 0$$

$$x := \max\left[Quadratic_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2} \right)}{ft}, \frac{\left(A_s \times w_{mem} \times n \right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d \right)}{ft^3} \right] \times 12in \right] = 0.909 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 1.697 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 11.195 \times kips$

Stress in the steel at service limit state $f_s := \frac{T}{A_s \times w_{mem}} = 18.659 \times ksi$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion <u>not</u> a concern
0.75 Class 2 - cracks and corrosion <u>are</u> a concern

$d_c := clear = 1.75 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c \right)} = 2.111$$

$$s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 9.828 \times in$$

$spacing \times (1 + bundles) = 4 \times in$

$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles) \right] = "SATISFACTORY"$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\phi_v := 0.9$

$$V_u := \left[1.25 \times \left(\frac{w_{DC} \times 10ft}{4} \right) + 1.75 \times \left(\frac{w_{LL} \times 10ft}{4} \right) \right] \times w_{mem} = 0.49 \times kips$$

Use Simplified Calc Values

Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv $Simplified := "yes"$

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_c = 2.457 \times kips$

$Check_{C.D}\left(\phi_v \times V_c, V_u \right) = "SATISFACTORY"$

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Load to Edge Beam (PIN-PIN)

Dead Load of Slab

$$R7_{DC.Slab} := w_{DC} \times \left(\frac{10ft}{2} + 6in\right) = 284.2 \times plf$$

Dead Load of 54 inch Bicycle Railing:

$$\boxed{R7_{DC.Railing} := 36.6plf}$$

Live Load Reaction

$$R7_{LL} := w_{LL} \times \frac{10ft}{2} = 375.0 \times plf$$

Edge Beam Length = 18ft 0in max (REF SHT 59)

$$\boxed{L_{edge} := 18ft}$$

Edge beam is designed as PIN-FIX due to deflection criteria:

$$V7_{edge} := 1.25 \times \left(R7_{DC.Slab} + R7_{DC.Railing}\right) + 1.75 \times R7_{LL} = 1.057 \times 10^{3} \times plf$$

$$M7_{edge} := \frac{\left[1.25 \times \left(R7_{DC.Slab} + R7_{DC.Railing}\right) + 1.75 \times R7_{LL}\right] \times L_{edge}^{\ 2}}{8} = 42.8\,ft \times kips$$

Square or Rectangular HSS Bending

For square and rectangular HSS bent about either axis, the nominal flexural resistance shall be taken as the smallest value based on yielding, flange local buckling or web local buckling, as applicable

AASHTO Equations all match AISC 13th Edition Equations for HSS Flexure.
However the reduction factor in AASHTO is 1.0 vs the 0.9 factor in AISC.

Yielding Limit (AASHTO 6.12.2.2.2-2 matches AISC EQ F7-1)

$$M_n = M_p = F_y \times Z$$

Flange Compact Criteria Buckling Limit (AASHTO 6.12.2.2.2-5&6 matches AISC Table B4.1)

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} \qquad \lambda_{rf} = 1.40 \times \sqrt{\frac{E}{F_y}}$$

Flange Local Buckling Limit For Compact Flanges (AASHTO 6.12.2.2.2-3 matches AISC EQ F7-2)

$$M_n = M_p - \left(M_p - F_y \times S\right) \times \left(3.57 \times \frac{b_f}{t_f} \times \sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p$$

Flange Local Buckling Limit for Non-Compact Flanges (AASHTO 6.12.2.2.2-4 matches AISC EQ F7-3)

$$M_n = F_y \times S_{eff}$$

Effective width of compression flange (AASHTO 6.12.2.2.2-7 matches AISC EQ F7-4)

$$b_e = 1.92 \times t_f \times \sqrt{\frac{E}{F_y}} \times \left[1 - \frac{0.38}{\left(\frac{b_f}{t_f}\right)} \times \sqrt{\frac{E}{F_y}}\right] \le b_f$$

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Table 3-13) may be used for the development of AASHTO capacities.

HSS 8x6x3/16 ~ Fy=46ksi ~ φ = 0.90:

$$\varphi M_{n.AISC} := 41.4kip \times ft$$

$$\varphi M_{n.AASHTO.HSS8x6x3} := \frac{1.0}{0.9} \times \varphi M_{n.AISC} = 46.0 \times kip \times ft$$

$$\frac{M7_{edge}}{\varphi M_{n.AASHTO.HSS8x6x3}} = 0.931$$

$$\boxed{Check_{C.D}\left(\varphi M_{n.AASHTO.HSS8x6x3}, M7_{edge}\right) = \text{"SATISFACTORY"}}$$

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Nominal Resistance of Unstiffened Webs - AASHTO LRFD 6.10.9.2

$\varphi_v := 1.0$

"For square and rectangular HSS, the web depth, D, shall be taken as the clear distance between flanges less the inside corner radius on each side of the area of both webs shall be considered effective in resisting the shear" (6.12.1.2.3b)

$V_n = V_{cr} = C \times V_p$

$C = \text{if } \frac{D}{t_w} \le 1.12 \times \sqrt{\frac{E \times (k = 5.0)}{F_{yw}}}$ then C=1.0

$D := 5\frac{7}{16} in$ $\quad E := 29000ksi$ $\quad F_{yw} := 46ksi$

$t_w := \frac{1}{8} in$

$\frac{D}{t_w} = 43.5$ $\qquad 1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}} = 62.9$

$C := \text{if}\left[\frac{D}{t_w} \le \left(1.12 \times \sqrt{\frac{E \times 5.0}{F_{yw}}}\right), 1.0, 0\right] = 1$

$V_p := 0.58 \times F_{yw} \times D \times t_w = 18.1 \times kips$

PIN - FIX

$\varphi Vn := \varphi_v \times V_p = 18.1 \times kips$ $\qquad V_u := \frac{5}{8} \times V7_{edge} \times L_{edge} = 11.894 \times kips$

$Check_{C.D}(\varphi Vn, V_u) = \text{"SATISFACTORY"}$

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Edge Beam Load transfers to New Channel Outrigger Truss

Length from face of connected existing girder to face of curb: (REF SHT 12)

$$L_{ex.gird.curb} := 1ft + 6in$$

Width of new Traffic Barrier:

$$width_{barrier} := (32in + 6in) \times \frac{4}{21} + 8in = 15.238 \times in$$

Length from face of connected existing girder to new vertical tension strut node:

$$L_{ex.gird.node} := 3ft$$

Depth of Truss (min):

$$Depth := 2ft$$

Reaction from edge beam to new channel truss outrigger:

$$R_{fact.truss} := 2 \times V7_{edge} \times \frac{5}{8} \times L_{edge} = 23.8 \times kips$$

$$R_{serv.truss} := 2 \times \left[\left(R7_{DC.Slab} + R7_{DC.Railing}\right) + R7_{LL}\right] \times \frac{L_{edge}}{2} = 12.5 \times kips$$

$$L_{Diagonal} := \sqrt{\left(L_{ex.gird.curb} + width_{barrier} + 10ft + 4in - \frac{6in}{2} - L_{ex.gird.node}\right)^2 + (Depth)^2} = 10.054\,ft$$

Diagonal compression strut:

$$Diag_{fact.compr} := R_{fact.truss} \times \frac{L_{Diagonal}}{Depth} = 119.579 \times kips$$

Horizontal Tension Strut (carries tension):

$$Horiz_{fact.tens} := \frac{\left(L_{ex.gird.curb} + width_{barrier} + 10ft - \frac{6in}{2} - L_{ex.gird.node}\right)}{L_{Diagonal}} \times Diag_{fact.compr} = 113.225 \times kips$$

Non-Composite Member - Nominal Compressive Resistance (AASHTO 6.9.4.1)

The nominal compressive resistance shall be taken as the smallest value based on the applicable modes of flexural buckling, torsional buckling and flecural-torsional buckling as follows:

Doubly Symmetric members:

Flexural buckling shall be applicable.

Torsional buckling shall also be applicable for open-section members in which the effective torsional unbraced length is larger than the effective leateral unbraced length.

Singly Symmetric members

Flexural buckling shall be applicable.

Flexural-torsional buckling shall also be applicable for open-section members

Unsymmetric members

Only Flexural-torsional buckling shall be applicable for open-section members, except that for single-angle members designed according to the provisions of Article 6.9.4.4, only flexural buckling shall be applicable.

Only flexural buckling shall be applicable for closed-section members.

Eqs. 6.9.4.1.1-1 and 6.9.4.1.1-2 are equivalent to the equations given in AISC (2005) for computing the nominal compressive resistance. (AASHTO C6.9.4.1.1)

if $\frac{P_e}{P_o} \geq 0.44$, then: $P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \times P_o$ Eqn. 6.9.4.1.1-1

if $\frac{P_e}{P_o} < 0.44$, then: $P_n = 0.877 \times P_e$ Eqn. 6.9.4.1.1-2

Because the given equations for member capacity are the same between AASHTO and AISC, AISC tabulated values (Tables in chapter 4) may be used for the development of AASHTO capacities.

Effective length: $K := 1.0$ $L := L_{Diagonal} = 10.054\,ft$

$K \times L = 10.054\,ft$

HSS 8x4x1/4 ~ Fy=46ksi ~ φ = 0.90:

$Length_{tabulated} := \begin{pmatrix} 10ft \\ 11ft \end{pmatrix}$ $Capacity_{tabulated} := \begin{pmatrix} 152kips \\ 142kips \end{pmatrix}$

$\varphi P_n := linterp\left(Length_{tabulated}, Capacity_{tabulated}, K \times L\right) = 151.459 \times kips$

$\frac{Diag_{fact.compr}}{\varphi P_n} = 0.79$ $Check_{C.D}\left(\varphi P_n, Diag_{fact.compr}\right) = \text{"SATISFACTORY"}$

Tensile Resistance - AASHTO LRFD (6.8.2.1)

The factored tensile resistance shall be taken as the lesser of gross yielding and net section fracture

Design Tension Load: $Horiz_{fact.tens} = 1.132 \times 10^5\,lb$ $\varphi_y := 0.95$ $\varphi_u := 0.80$

Steel Properties $F_y := 46ksi$ $F_u := 58ksi$

Gross cross section area:
2 C6x8.2 $A_g := 2 \times 2.39in^2$

Size of holes in member: $d_{holes} := 2 \times \left(\frac{3}{4}in + \frac{1}{16}in\right) = 1.625 \times in$

Thickness of penetrated part: $t := 0.20in$

Nominal section area: $A_n := A_g - d_{holes} \times t = 4.455 \times in^2$

Fracture resistance reduction factor: $R_p := 0.9$

0.9 for holes punched full size
1.0 for holes drilled full size or sub-punched and reamed full size

Shear Lag Reduction factor: $U = 1 - \frac{x_{bar}}{L}$

All tension members, except plates, and HSS, where the tension load is transmitted to some but not all of the cross sectional elements.

$x_{bar} := 0.512in$ Connection Eccentricity

$L := 12in$ Length of Connection

$$U := 1 - \frac{x_{bar}}{L} = 0.957$$

$$P_{r_1} := \varphi_y \times F_y \times A_g = 208.886 \times kips$$

$$P_{r_2} := \varphi_u \times F_u \times A_n \times R_p \times U = 178.103 \times kips$$

$$\varphi P_n := \min(P_r) = 178.103 \times kips$$

$$\frac{Horiz_{fact.tens}}{\varphi P_n} = 0.636$$

$Check_{C.D}(\varphi P_n, Horiz_{fact.tens}) = $ "SATISFACTORY"

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Shear Resistence of a single bolt (6.13.2.7)

Bolt Type (mark "yes" or "no"): $A325 := \text{"yes"}$ $A490 := \text{"No"}$

$ASTM = \text{"A325"}$

Nominal Bolt Diameter: $d := \frac{3}{4} in$

Area of bolt (corresponding to nominal diameter): $A_b := \frac{\pi \times d^2}{4} = 0.44 \times in^2$

Specified minimum tensile strength of the bolt specified in Article 6.4.3:

$$F_{ub} := \begin{array}{|l} FUB \leftarrow 0ksi \\ FUB \leftarrow 150ksi \ \text{ if } \ ASTM = \text{"A490"} \\ \text{if } \ ASTM = \text{"A325"} \\ \quad \begin{array}{|l} FUB \leftarrow 120ksi \ \text{ if } \ d \le 1.0in \\ FUB \leftarrow 105ksi \ \text{ if } \ d > 1.0in \end{array} \end{array}$$

$F_{ub} = 120 \times ksi$

Number of Shear Planes per bolt: $N_s := 2$

Where threads are included from the shear plane: $R_{n.shear} := 0.38 \times A_b \times F_{ub} \times N_s$ $R_{n.shear} = 40.3 \times kips$

Bearing Resistance at Bolt Holes (6.13.2.9)

For Standard, Oversize, or Short Slotted Holes spaced at not less than 2 x Bolt Diameter

Thickness of the connected material: $t := 0.20in$

Tensile strength of the connected material specified in Table 6.4.1-1:

Grade 36: $F_u := 58ksi$

$R_{n.holes} := 2.4 \times d \times t \times F_u$ $R_{n.holes} = 20.9 \times kips$

Strength Resistance minimum of bolt shear and bearing

$R_n := \min(R_{n.shear}, R_{n.holes}) = 20.88 \times kips$

$\varphi := 0.80$ For A325 and A490 Bolts in Shear and Tension

$$\frac{\sqrt{Horiz_{fact.tens}{}^2 + R_{fact.truss}{}^2}}{\varphi \times R_n} = 6.926$$

Provide 7 - 3/4" anchors through the existing concrete cantilever

Design Moment on Existing Cantilever

$M_{fact.ex} := R_{fact.truss} \times L_{ex.gird.node} = 71.362 \, ft \times kips$

$M_{serv.ex} := R_{serv.truss} \times L_{ex.gird.node} = 37.571 \, ft \times kips$

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Check Existing Cantilever Beam for applied moments

$E_s := 29000\text{ksi}$

Concrete Strength of Existing Structure: $f'_c := 8\text{ksi}$

Rebar Strength in Existing Structure: $F_y := 33\text{ksi}$

Bar info: (REF SHT 12) $\text{Area} := 1.125\text{in}^2$ $\quad d_{bar} := \sqrt{\text{Area}} = 1.061 \times \text{in}$

Assumed Clear: (REF SHT 12) $\text{clear} := 3\text{in}$

Number of layers: $\text{full}_{layers} := 1$ $\quad \text{bundled} := \text{"no"}$ $\quad \text{bundles} := \text{yes}_{no}(\text{bundled}) = 0$ $\quad \beta_1 = 0.65$

Width of member $w_{mem} := 8\text{in}$

Bars per Layer: $n_{bars} := 2$ $\quad \text{spacing} := \dfrac{w_{mem}}{n_{bars}} = 4 \times \text{in}$

Height of member: (REF SHTS 12) $h_{mem} := 3\text{ft} + 3\text{in} - (3.5\text{in} + 2.5\text{in})$

$$A_s := \frac{\text{Area}}{\text{spacing}} \times \text{full}_{layers} \qquad A_s \times w_{mem} = 2.25 \times \text{in}^2 \qquad \text{clear}_{spacing} = 2.939 \times \text{in}$$

Design Moment: $M_{des} := M_{fact.ex} = 71.362\ \text{ft} \times \text{kips}$

Service Moment: $M_{service} := M_{serv.ex} = 37.571\ \text{ft} \times \text{kips}$

depth to rebar
$$d := h_{mem} - \text{clear} - \left[\frac{d_{bar}}{2} + \frac{\text{full}_{layers} - 1}{2} \times \left(\max(1 \times \text{in}, d_{bar}) + d_{bar}\right)\right]$$

$d = 29.47 \times \text{in}$

Rectangular Compression Depth:
$$a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}} \qquad a_{req} = 0.6 \times \text{in}$$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $\quad A_{s.req} \times w_{mem} = 0.988 \times \text{in}^2$

Check compression for full yielding of actual steel

$$a := \frac{A_s \times F_y}{0.85 \times f'_c} \qquad a = 1.365 \times \text{in} \qquad \phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$$

$\text{Check}_{C.D}(\phi M_n, M_{des}) = \text{"SATISFACTORY"}$

$M_{des} = 71.362 \times \text{kip} \times \text{ft}$

$\phi M_n = 160.309 \times \text{kip ft}$

$\dfrac{\phi M_n}{M_{des}} = 224.6 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $\text{allow}_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.37 \times \sqrt{f'_c \cdot (\text{ksi})}$ $\quad F_r = 1 \times 10^3 \times \text{psi}$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $\quad S = 2.178 \times 10^3 \times \dfrac{\text{in}^3}{\text{ft}}$

Cracking Moment: $M_{cr} := F_r \times S$ $\quad M_{cr} = 189.943 \times \dfrac{\text{kip} \times \text{ft}}{\text{ft}}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $\quad \text{bar}_{no} = 4$

in $\text{full}_{layers} = 1$ (total bars $n = 2$)

spaced at $\text{spacing} = 4 \times \text{in}$

$w_{mem} = 0.667\ \text{ft}$ x $h_{mem} = 2.75\ \text{ft}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 151.954 \times \text{kip} \times \text{ft}$

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Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 5422 \times ksi$$

$$n := \frac{E_s}{E_c} = 5.348$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[\text{Quadratic}_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 8.031 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 26.793 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 16.828 \times kips$

Stress in the steel at service limit state

$$\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 7.479 \times ksi}$$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 3 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 1.143$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s}(ksi)\ (in) - 2 \times d_c = 55.423 \times in}$$

$$spacing \times (1 + bundles) = 4 \times in$$

$$\text{Check}_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"SATISFACTORY"}$$

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Check Structure for Deflection

Concrete Slab deflection (PIN-PIN - Conservative)

$$\delta_c := \frac{5}{384} \times \frac{w_{LL} \times (10ft)^4}{\left[E_c \times \frac{(4in)^3}{12}\right]} = 0.049 \times in$$

$$\frac{10ft}{\delta_c} = 2.468 \times 10^3$$

Greater than 500 okay
REF AASHTO LRFD Guide Specifications for Pedestrian Bridges
Section 5

Edge Beam deflection (PIN-FIX)

$$I_{HSS8x6x3} := 43.7in^4$$

$$\delta_s := \frac{1}{185} \times \frac{R7_{LL} \times \left(L_{edge}\right)^4}{\left(E_s \times I_{HSS8x6x3}\right)} = 0.29 \times in$$

Concrete slab max deflection occurs at slab midspan. Edge beam deflection occurs at the edge beam. Because edge beam deflection exceeds the slab deflection, the edge beam governs the maximum deflection.

$$\frac{L_{edge}}{\left(\delta_s\right)} = 744.457$$

Greater than 500 okay
REF AASHTO LRFD Guide Specifications for Pedestrian Bridges
Section 5

Additional dead load to the structure from walkway widening.

Dead Load of Additional Slab: $plf_{slab} := w_{DC} \times (10ft + 6in) = 542.5 \times plf$

Dead Load of Railing: $plf_{railing} := R7_{DC.Railing} = 36.6 \times plf$

$L_{Diagonal} = 10.054\,ft$

Dead Load of Edge Beam: $plf_{edge.beam} := 17.10plf$

Dead Load of New Outrigger Extensions:

$$plf_{new.outrigger} := \frac{\left[18.99plf \times \left(L_{Diagonal} + L_{ex.gird.node}\right) + 2 \times 8.2plf \times \left[\left(L_{Diagonal} + 9in\right) + Depth\right]\right]}{L_{edge}} = 25.438 \times plf$$

Dead Load of New Traffic Barrier + BP rail:

$$plf_{barrier} := \frac{\left(8in + 32in \times \frac{4}{21}\right) + 8in}{2} \times 32in \times \gamma_c + 6.7plf = 387.229 \times plf$$

Total weight of new work: $plf_{new} := plf_{slab} + plf_{railing} + plf_{edge.beam} + plf_{new.outrigger} + plf_{barrier} = 1.0 \times klf$

Weight of Removals $\gamma_{c.ex} := 150pcf$

Dead Load of Existing Pedestal at 8" cantilever:
(Neglect 12" cantilever due to irregular placement)

Section area (REF SHT 38): $A_{ped} := (1ft + 4in) \times 12in - 4 \times 1.25in \times 2in = 1.264\,ft^2$

Pedestal weight:

$$plf_{pedestal} := \frac{A_{ped} \times (3ft + 9.5in) \times \gamma_{c.ex}}{L_{edge}} = 39.9 \times plf$$

Dead Load of Existing intermediate Conc Rail:

$$plf_{conc.rail} := \frac{(1ft + 10in) \times 6in \times \left[L_{edge} - (1ft + 4in) - 0.5in\right] \times \gamma_{c.ex}}{L_{edge}} = 127 \times plf$$

Dead Load of Existing metal rail: (8.15plf top beam REF SHTS 37 & 38)

$$plf_{metal.rail} := \left[8.15plf + 2 \times 2.5in \times \frac{3}{8}in \times 490pcf + \frac{2 \times \frac{3}{4}in \times 3in \times (4.5in + 5in + 7.5in) \times 490pcf}{L_{edge}} + \frac{12 \times 2.5in \times \frac{3}{8}in \times 5in \times 490pcf}{L_{edge}}\right]$$

$$plf_{metal.rail} = 16.6 \times plf$$

Dead Load of Existing Edge Beam:

$$plf_{ex.edge.beam} := \frac{(1ft + 9in) \times 6in \times \left[L_{edge} - (8in)\right] \times \gamma_{c.ex}}{L_{edge}} = 126.4 \times plf$$

Dead Load of Existing Curb:
(REF SHT 30)

$$plf_{curb} := \left[18in \times \left(7in + \frac{3in}{2}\right) - 3in \times 2.5in\right] \times \gamma_{c.ex} = 151.563 \times plf$$

Total weight of removals: $plf_{rem} := plf_{pedestal} + plf_{conc.rail} + plf_{metal.rail} + plf_{ex.edge.beam} + plf_{curb} = 461.5 \times plf$

Total increased weight: $plf_{increase} := plf_{new} - plf_{rem} = 547.4 \times plf$

Weight of Segment 7 calculated for seismic analysis: $Segment7_{weight} := 5350kips$

Length of Segment 7: $Segment7_{length} := 287.32ft$

Average Per foot weight of Segment 7: $\dfrac{Segment7_{weight}}{Segment7_{length}} = 1.862 \times 10^{4} \times plf$

Percent Increase due to extended sidewalk load: $\dfrac{plf_{increase}}{\dfrac{Segment7_{weight}}{Segment7_{length}}} = 2.94 \times \%$ Less than 10% OKAY

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Luminaire attachment

Approximate Grade Seperation: $H_{grade.sep} := 30ft$

Height of Luminaire Pole above deck: $H_{mast} := 30ft + (3ft + 7in) = 33.583\ ft$
(REF SHT E-6/91/4 : 782-95)

Length of Luminaire Mast arm: $L_{arm} := 12ft$
(12' max REF WSDOT STD J-28.10-01)

AASHTO Std Specs. for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

Wind Pressure Equation (Eq 3-1)

$$P_z = 0.00256 \times K_z \times G \times V^2 \times I_r \times C_d \qquad \text{(psf)}$$

Selecting a wind speed of: 90mph $V := 90mph$

Wind on Luminaire

$H_{lum} := H_{grade.sep} + H_{mast} = 63.583\ ft$

$$K_{z.eq}(z, z_g, \alpha) := \text{if}\left[z > 16.4ft, 2.01 \times \left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}, 0.865\right]$$

$z_g := 900ft$ $\alpha := 9.5$

$K_z := K_{z.eq}(H_{lum}, 900ft, 9.5) = 1.151$

$I_r := 1.00$ Table 3-2 - 50 year recurrence as recommended by Table 3-3

$G := 1.14$ Gust Effect Factor (3.8.5)

$C_d := 0.5$ Luminaires (with generally rounded surfaces)

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 13.599 \times psf$$

Effective Projected Area of Luminaire Head (REF WSDOT BDM 10.1(B)) $A := 3.3ft^2$

$V_{wind.lum} := P_z \times A = 44.876\ lb$

Height to point of connection: $H := H_{mast} = 33.583\ ft$

$M_{wind.lum} := V_{wind.lum} \times H = 1.507\ ft \times kips$

Table 3–5. Height and Exposure Factors, K_z

Height, m(ft)	K_z
5.0(16.4) or less	0.87
7.5 (24.6)	0.94
10.0 (32.8)	1.00
12.5 (41.0)	1.05
15.0 (49.2)	1.09
17.5 (57.4)	1.13
20.0 (65.6)	1.16
22.5 (73.8)	1.19
25.0 (82.0)	1.21
27.5 (90.2)	1.24
30.0 (98.4)	1.26
35.0 (114.8)	1.30
40.0 (131.2)	1.34
45.0 (147.6)	1.37
50.0 (164.0)	1.40
55.0 (180.5)	1.43
60.0 (196.9)	1.46
70.0 (229.7)	1.51
80.0 (262.5)	1.55
90.0 (295.3)	1.59
100.0 (328.1)	1.63

Note: See Eq. C 3–1 for calculation of K_z.

Once the terrain exposure conditions are established, the height and exposure factor, K_z, is calculated using the following relationship that is presented in ASCE/SEI 7:

$$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}} \qquad \text{(C3-1)}$$

where z is height above the ground at which the pressure is calculated or 5 m (16 ft), whichever is greater, and z_g and α are constants that vary with the exposure condition. Based on information presented in ASCE/SEI 7, α should be taken to be 9.5 and z_g should be taken to be 274.3 m (900 ft) for exposure C. These values are for 3-s gust wind speeds and are different from similar constants that have been used for fastest-mile wind speeds. Table 3-5 presents the variation of the height and exposure factor, K_z, as a function of height based on the above relation.

Wind on Mast Arm

$H_{arm} := H_{grade.sep} + H_{mast} = 63.583\,ft$

$K_z := K_{z.eq}(H_{arm}, 900ft, 9.5) = 1.151$

$$C_{d.cylinder}(V, d) := \begin{array}{|l} \omega \leftarrow 1.105 \times \frac{V}{mph} \times \frac{d}{ft} \\ CD \leftarrow \frac{129}{\omega^{1.3}} \\ CD \leftarrow 1.10 \ \text{ if } \ \omega \le 39 \\ CD \leftarrow 0.45 \ \text{ if } \ \omega \ge 78 \\ CD \end{array}$$

Cylinders (3in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 3in) = 1.1$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 29.917 \times psf$$

Effective Projected Area of mast arm $A := 3in \times L_{arm} = 3\,ft^2$

$V_{wind.arm} := P_z \times A = 89.752\,lb$

Height to point of connection: $H := H_{mast} = 33.583\,ft$

$M_{wind.arm} := V_{wind.arm} \times H = 3.014\,ft \times kips$

Wind on Pole

$$H_{pole} := H_{grade.sep} + \frac{H_{mast}}{2} = 46.792\,ft$$

$K_z := K_{z.eq}(H_{pole}, 900ft, 9.5) = 1.079$

Cylinders (8in diameter ~ assumed)

$C_d := C_{d.cylinder}(V, 8in) = 0.553$

$$P_z := \left[0.00256 \times K_z \times G \times \left(\frac{V}{mph}\right)^2 \times I_r \times C_d\right] psf = 14.097 \times psf$$

Effective Projected Area of mast arm $A := 8in \times H_{mast} = 22.4\,ft^2$

$V_{wind.arm} := P_z \times A = 315.609\,lb$

Height to point of connection: $H := \frac{H_{mast}}{2} = 16.792\,ft$

$M_{wind.pole} := V_{wind.arm} \times H = 5.3\,ft \times kips$

Table 3-2—Wind Importance Factors, I_r

Recurrence Interval Years	Basic Wind Speed in Nonhurricane Regions	Basic Wind Speed in Hurricane Regions with $V > 45$ m/s (100 mph)	Alaska
100	1.15	1.15	1.13
50	1.00	1.00	1.00
25	0.87	0.77[a]	0.89
10	0.71	0.54[a]	0.76

[a] The design wind pressure for hurricane wind velocities greater than 45 m/s (100 mph) should not be less than the design wind pressure using $V = 45$ m/s (100 mph) with the corresponding nonhurricane I_r value.

Table 3-3—Recommended Minimum Design Life

Design Life	Structure Type
50 yr	Overhead sign structures Luminaire support structures[a] Traffic signal structures[a]
10 yr	Roadside sign structures

[a] Luminaire support structures less than 15 m (50 ft) in height and traffic signal structures may be designed for a 25-yr design life, where locations and safety considerations permit and when approved by the Owner.

Table 3–6. Wind Drag Coefficients, C_d (see note 1)

Sign Panel (by ratio of length to width) L/W = 1.0 2.0 5.0 10.0 15.0	 1.12 1.19 1.20 1.23 1.30	
Traffic Signals (see note 2)	1.2	
Luminaires (with generally rounded surfaces)	0.5	
Luminaires (with rectangular flat side shapes)	1.2	
Elliptical Member	Broadside Facing Wind $1.7\left(\frac{D}{d_o} - 1\right) + C_{dD}\left(2 - \frac{D}{d_o}\right)$	Narrow Side Facing Wind $C_{dd}\left[1 - 0.7\left(\frac{D}{d_o} - 1\right)^{1/4}\right]$
Two members or trusses (one in front of other) (all trusses with small solidity ratios) (see note 3)	1.20 (cylindrical) 2.00 (flat)	

Single Member or Truss Member	$C_vVd \le 5.33$ (39)	$5.33(39) < C_vVd < 10.66(78)$	$C_vVd \ge 10.66(78)$
Cylindrical	1.10	$\frac{9.69}{(C_vVd)^{1.3}}$ (SI) $\frac{129}{(C_vVd)^{1.3}}$ (U.S. Customary)	0.45
Flat (See note 4)	1.70	1.70	1.70
Hexdecagonal: $0 \le r < 0.26$	1.10	$1.37 + 1.08r - \frac{C_vVd}{19.8} - \frac{C_vVdr}{4.94}$ (SI) $1.37 + 1.08r - \frac{C_vVd}{145} - \frac{C_vVdr}{36}$ (U.S. Customary)	0.83 - 1.08r
Hexdecagonal: $r \ge 0.26$	1.10	$0.55 + \frac{(10.66 - C_vVd)}{9.67}$ (SI) $0.55 + \frac{(78.2 - C_vVd)}{71}$ (U.S. Customary)	0.55
Dodecagonal (see note 5)	1.20	$\frac{3.28}{(C_vVd)^{0.6}}$ (SI) $\frac{10.8}{(C_vVd)^{0.6}}$ (U.S. Customary)	0.79
Octagonal	1.20	1.20	1.20

Total Moment from wind (omnidirectional):

$M_{wind} := M_{wind.lum} + M_{wind.arm} + M_{wind.pole} = 9.821\,ft \times kips$

Dead Loads

Distance from Pole location to Point of connection:

Weight of Luminaire:
(REF WSDOT BDM 10.1.1(B))

$W_{lum} := 60lb$

$M_{DC.lum} := W_{lum} \times L_{arm} = 0.72\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$W_{arm} := \pi \times 3in \times ga_{pl_{11}} \times 490pcf \times L_{arm} = 46.027\,lb$

$M_{DC.arm} := W_{arm} \times \frac{L_{arm}}{2} = 0.276\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$W_{pole} := \pi \times 8in \times ga_{pl_{11}} \times 490pcf \times H_{mast} = 343.501\,lb$

Total Moment from Dead Load:

$M_{DC} := M_{DC.lum} + M_{DC.arm} = 0.996\,ft \times kips$

Ice Loads

Ice on Luminaire:
(assuming 6 sides
of equal projected area)

$Ice_{lum} := 3.3ft^2 \times 6 \times 3psf = 59.4\,lb$

$M_{Ice.lum} := Ice_{lum} \times L_{arm} = 0.713\,ft \times kips$

Weight of Arm:
(Assume 11 gage)

$Ice_{arm} := \pi \times 3in \times 3psf \times L_{arm} = 28.274\,lb$

$M_{Ice.arm} := Ice_{arm} \times \frac{L_{arm}}{2} = 0.17\,ft \times kips$

Weight of Pole:
(Assume 11 gage)

$Ice_{pole} := \pi \times 8in \times 3psf \times H_{mast} = 211.01\,lb$

Total Moment from Dead Load:

$M_{Ice} := M_{Ice.lum} + M_{Ice.arm} = 0.882\,ft \times kips$

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Anchorage from Combined Loads

$$M_{design'} := \begin{bmatrix} \left|M_{DC}\right| \times \dfrac{1}{100\%} \\ \left|M_{DC}\right| + M_{wind} \times \dfrac{1}{133\%} \\ \left(\left|M_{DC} + M_{Ice}\right| + \dfrac{M_{wind}}{2}\right) \times \dfrac{1}{133\%} \end{bmatrix} = \begin{pmatrix} 0.996 \\ 8.38 \\ 5.105 \end{pmatrix} ft \times kips$$

Governing Load $M_{design} := \max\left(M_{design'}\right) = 8.38\, ft \times kips$

Light pole attaches to new concrete block. Block is attached to new structure through slab reinforcing and headed anchor studs.

Tension / Compression moment couple: $TC := \dfrac{M_{design}}{10in} = 10.056 \times kips$

$$\frac{\left(3 \times A_{no.4}\right) \times 60ksi}{1.67} = 21.557 \times kips$$

Tension couple would acceptably transfer load to slab steel.

Required diameter of anchor $A_{req} := \dfrac{TC \times 1.67}{3 \times 36ksi} = 0.156 \times in^2$

$$diameter := \sqrt{4 \times \frac{A_{req}}{\pi}} = 0.445 \times in$$

Tension couple would acceptably transfer load to headed anchor studs

BALLARD Widening - Segment 8 - 10ft Sidewalk Extension

The sidewalk extension in Segment 8 shall be made in the form of an anchor slab with cantilevered sidewalk extension. An anchor slab is preferred in order to isolate the existing retaining wall structure from new loads introduced by the sidewalk extension and TL-4 traffic barrier.

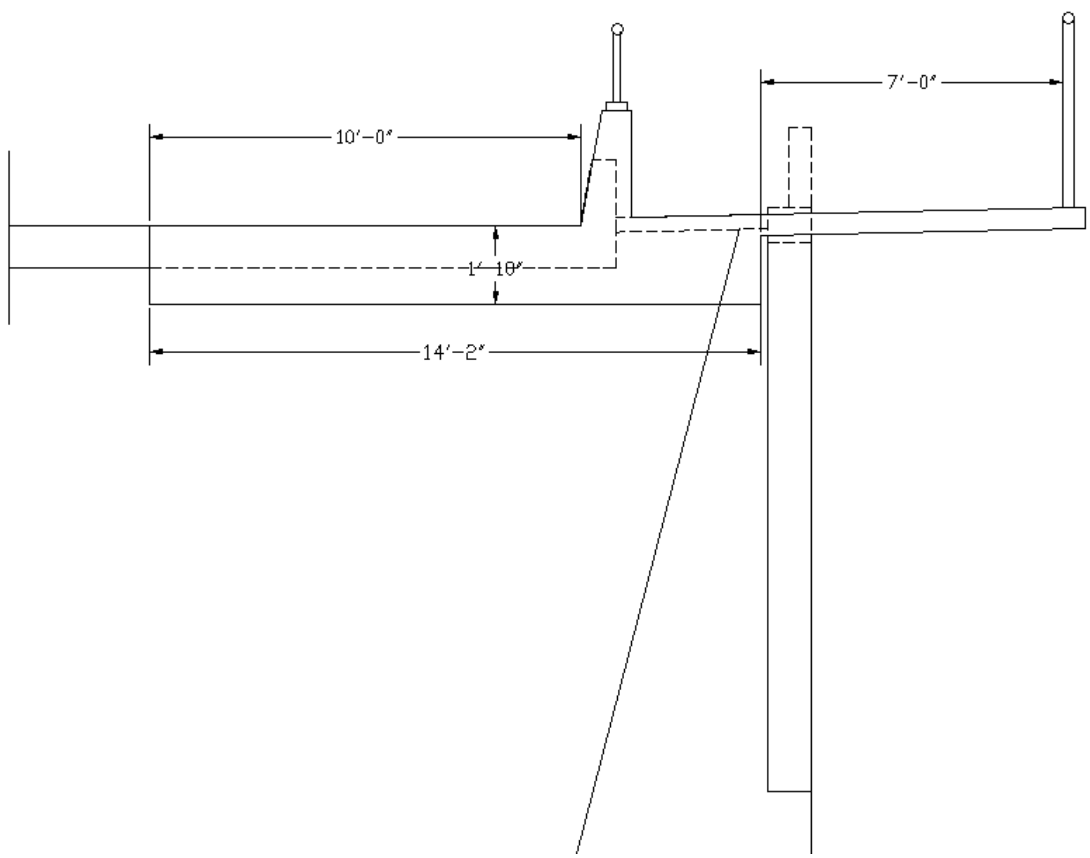
The 10ft extension option at Segment 8 exhibits high overturning forces due to the length of the cantilever. These forces resulted in a requirement for a large back-span anchor slab. The existing wall was checked as an alternative to carry the pedestrian load, but is insufficient.

A 10ft extension into the existing roadway was selected as a minimum to align the edge of concrete with the edge of driving lane. If the in situ distance to driving lane exceeds 10 ft the anchor slab can only benefit from the length increase with potential for reanalysis leading to a thinner slab section.

The existing concrete approach slab extends only slightly beyond the proposed anchor slab, the entirety of the approach slab could be removed and replaced with the anchor slab, leading to potential for a thinner slab section, however the length of the existing approach slab may extend into a second driving lane, which may not be preferred for traffic control at time of construction.

The length of the anchor slab may be reduced from that shown with the additional thickening below the slab. Design iterations suggest that at a minimum, the anchor slab would need to extend 4.5ft beyond the face of barrier with a depth of 4ft.

The alternative chosen was selected due to its depth, and better appearance to drivers due to a lane aligned longitudinal edge.



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The slab has been designed for stability under the following conditions:

1) Unbalanced pedestrian live load overturning using strength factors to check LRFD eccentricity limitations
2.) Extreme limit state 10 kip vehicle impact stability load applied to the top of the traffic barrier

$L_{Overhang} := 7ft + 0in$

$w_{LL} := 75psf$ $\quad \gamma_c := 155pcf$

Overturning Moment

$$OM_{LL} := w_{LL} \times \frac{L_{Overhang}^{2}}{2} = 1.838 \times 10^{3} \times \frac{lb \times ft}{ft}$$

$$OM_{DC} := 6in \times \gamma_c \times \frac{\left(L_{Overhang} + 6in\right)^2}{2} = 2.18 \times 10^{3} \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Lat.Railing} := 50plf \times 54in = 225 \times \frac{lb \times ft}{ft}$$

$$OM_{LL.Vert.Railing} := 50plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 355.99 \times \frac{lb \times ft}{ft}$$

$$OM_{DC.Railing} := 36.6plf \times \left(L_{Overhang} + \frac{2.875in}{2}\right) = 260.584 \times \frac{lb \times ft}{ft}$$

$$OM_{service} := \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + \left(OM_{DC} + OM_{DC.Railing}\right) = 4.859 \times 10^{3} \times \frac{lb \times ft}{ft}$$

$$OM_{factored} := 1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + 0.9 \times \left(OM_{DC} + OM_{DC.Railing}\right) = 6.429 \times 10^{3} \times \frac{lb \times ft}{ft}$$

Continuous Anchor Slab Length (minimum): $\quad L_{AS} := 20ft$

$$OM_{veh} := 10kips \times (32in + 11in) = 3.583 \times 10^{4}\, ft \times \frac{lb \times ft}{ft}$$

$$OM_{Extreme} := OM_{veh} + L_{AS} \times \left(OM_{DC} + OM_{DC.Railing}\right) = 8.464 \times 10^{4} \times lb \times ft$$

Resisting Moment $\quad D_{slab} := 22in \quad Width := 14ft + 2in$

$$Res_{DC.slab} := D_{slab} \times \gamma_c \times \frac{(Width)^2}{2} = 2.852 \times 10^{4} \times \frac{lb \times ft}{ft}$$

$$Res_{DC.barrier} := 32\,in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times \left[4ft + 2in - \left[\frac{8in}{2} + \left(32in \times \frac{4}{21}\right)\right]\right] = 1.265 \times 10^{3} \times \frac{lb \times ft}{ft}$$

$$Res_{service} := Res_{DC.slab} + Res_{DC.barrier} = 2.978 \times 10^{4} \times \frac{lb \times ft}{ft}$$

$$Res_{factored} := 0.9 \times \left(Res_{DC.slab} + Res_{DC.barrier}\right) = 2.68 \times 10^{4} \times \frac{lb \times ft}{ft}$$

$$Res_{Extreme} := Res_{factored} \times L_{AS} = 5.361 \times 10^{5} \times lb \times ft$$

Strength Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times (L_{Overhang} + 6in) + 36.6plf\right] \ldots + D_{slab} \times \gamma_c \times (Width) + 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c = 5.024 \times 10^3 \times plf$$

$$V_{LL} := (w_{LL} \times L_{Overhang} + 50plf) = 575 \times plf$$

$$M_{abt.toe} := OM_{factored} - Res_{factored} = -2.037 \times 10^4 \times \frac{lb \times ft}{ft}$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{(0.9 \times V_{DL} + 1.75 \times V_{LL})}\right] = 3.398\,ft \qquad ecc_{limit} := \frac{Width}{4} = 3.542\,ft \qquad (10.6.3.3)$$

Bearing Pressure Check

$Check_{C.D}(ecc_{limit}, ecc) = \text{"SATISFACTORY"}$

$$\frac{V_{DL} + V_{LL}}{2 \times \left(\frac{Width}{2} - ecc\right)} = 759.574 \times psf$$

Bearing Pressure is reasonable

Extreme Eccentricity

$$V_{DL} := \left[6in \times \gamma_c \times (L_{Overhang} + 6in) + 36.6plf\right] \ldots + D_{slab} \times \gamma_c \times (Width) + 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c = 5.024 \times 10^3 \times plf$$

$$M_{abt.toe} := OM_{Extreme} - Res_{Extreme} = -4.514 \times 10^5 \times lb \times ft$$

$$ecc := \frac{(Width)}{2} + \left[\frac{M_{abt.toe}}{(0.9 \times V_{DL}) \times L_{AS}}\right] = 2.092\,ft$$

$$ecc_{limit} := \frac{Width}{3} = 4.722\,ft \qquad (10.6.4.2)$$

$Check_{C.D}(ecc_{limit}, ecc) = \text{"SATISFACTORY"}$

Sliding Resistance

$\varphi_\tau := 0.80$ (Table 10.5.5.2.2-1)

$\varphi_f := 28deg$ Assuming a reasonably shallow angle of internal friction

$\mu_{R.t} := \tan(\varphi_f) = 0.532$ (EQ 10.6.3.4-2)

$$Sliding_{res.DC.slab.at.wall} := ft \times \gamma_c \times (1ft + 1.5in + 2in - 1in) \times L_{AS} = 3.746 \times 10^3\,ft \times plf$$

$$Sliding_{res.DC.slab.past} := D_{slab} \times \gamma_c \times [5ft + 2.5in - (1ft + 1.5in) - 2in] \times L_{AS} = 2.226 \times 10^4\,ft \times plf$$

$$Sliding_{res.DC.barrier} := 32in \times \left[8in + \left(32in \times \frac{4}{21}\right) \times \frac{1}{2}\right] \times \gamma_c \times L_{AS} = 7.611 \times 10^3\,ft \times plf$$

$$Sliding_{resistance} := \varphi_\tau \times \mu_{R.t} \times (Sliding_{res.DC.slab.at.wall} + Sliding_{res.DC.slab.past} + Sliding_{res.DC.barrier}) = 1.43 \times 10^4\,ft \times plf$$

$Check_{C.D}(Sliding_{resistance}, 10kips) = \text{"SATISFACTORY"}$

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Cantievered Sidewalk Design

$E_s := 29000ksi$ $F_y := 60ksi$ $f'_c := 4ksi$

Bar size (US number): $bar_{no} := 5$ $d_{no_{bar_{no}}} = 0.625 \times in$ $diam_{deform_{bar_{no}}} = 0.688 \times in$ $A_{no_{bar_{no}}} = 0.31 \times in^2$

$clear := 2.5in = 0.208\,ft$

Number of layers: $full_{layers} := 1$ $bundled := \text{"no"}$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.85$

Width of member $w_{mem} := 1ft$ **Bars per Layer:** $n_{bars} := 2$ $spacing := \dfrac{w_{mem}}{n_{bars}} = 6 \times in$

Height of member $h_{mem} := 6in$ $A_s := \dfrac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.62 \times in^2$ $clear_{spacing} = 5.313 \times in$

Design Moment:

$$M_{des} := \left[1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) \ldots + 1.25 \times \left(OM_{DC} + OM_{DC.Railing}\right)\right] \times w_{mem} = 7.283 \times 10^3 \times lb \times ft$$

Service Moment: $M_{service} := OM_{service} \times w_{mem} = 4.859 \times 10^3 \times lb \times ft$

depth to rebar

$$d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$$

$d = 3.187 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \dfrac{M_{des}}{f'_c \times w_{mem}} \times \dfrac{2}{0.9 \times 0.85}}$ $a_{req} = 0.864 \times in$

Required Area of Steel: $A_{s.req} := \dfrac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.587 \times in^2$

Check compression for full yielding of actual steel

$a := \dfrac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.912 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \dfrac{a}{2}\right) \times w_{mem}$

$M_{des} = 7.283 \times kip \times ft$

$\phi M_n = 7.621 \times kip\,ft$

$\dfrac{\phi M_n}{M_{des}} = 104.6 \times \%$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = \text{"SATISFACTORY"}$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := \text{"yes"}$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.24 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 480 \times psi$

Section Modulus: $S := \dfrac{h_{mem}^2}{6}$ $S = 72 \times \dfrac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 2.88 \times \dfrac{kip \times ft}{ft}$

$M_{cr.Check} = \text{"SATISFACTORY"}$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 2$ $bar_{no} = 5$

in $full_{layers} = 1$ (total bars $n = 2$)

spaced at $spacing = 6 \times in$

$w_{mem} = 1\,ft$ x $h_{mem} = 0.5\,ft$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 3.456 \times kip \times ft$

Distribution of Reinforcement (LRFD 5.7.3.4)

The spacing s of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

Modulus of Elasticity - LRFD 5.4.2.4

$K_1 := 1.0$ Correction factor for source of aggregate to be taken as 1.0 unless determined by physical test

$w_c := 0.150kcf$ Unit weight of concrete (kcf)

$$E_c := 33000 \times K_1 \times \left[w_c \cdot \left(\frac{1}{kcf}\right)\right]^{1.5} \times \sqrt{f'_c \cdot (ksi)} = 3834 \times ksi$$

$$n := \frac{E_s}{E_c} = 7.563$$

Determine strain compatible neutral axis

$$w_{mem} \times x \times \left(\frac{x}{2}\right) = \left(A_s \times w_{mem}\right) \times n \times (d - x)$$

$$\left(w_{mem} \times \frac{1}{2}\right) \times x^2 + \left(A_s \times w_{mem} \times n\right) \times x - \left(A_s \times w_{mem} \times n \times d\right) = 0$$

$$x := \max\left[Quadratic_{a.b.c}\left[\frac{\left(w_{mem} \times \frac{1}{2}\right)}{ft}, \frac{\left(A_s \times w_{mem} \times n\right)}{ft^2}, \frac{-\left(A_s \times w_{mem} \times n \times d\right)}{ft^3}\right] \times 12in\right] = 1.235 \times in$$

Moment Arm: $arm := d - \frac{x}{3} = 2.776 \times in$

Service Steel Tension: $T := \frac{M_{service}}{arm} = 21.005 \times kips$

Stress in the steel at service limit state $\boxed{f_s := \frac{T}{A_s \times w_{mem}} = 33.879 \times ksi}$

$$s \le \frac{700 \times \gamma_e}{\beta_s \times f_s} - 2 \times d_c$$

$\gamma_e := 0.75$ exposure factor

1.00 Class 1 - cracks and corrosion not a concern
0.75 Class 2 - cracks and corrosion are a concern

$d_c := clear = 2.5 \times in$

$$\beta_s := 1 + \frac{d_c}{0.7 \times \left(h_{mem} - d_c\right)} = 2.02$$

$$\boxed{s_{max} := \frac{700 \times \gamma_e}{\beta_s \times f_s} (ksi)\ (in) - 2 \times d_c = 2.67 \times in}$$

$spacing \times (1 + bundles) = 6 \times in$

$Check_{C.D}\left[s_{max}, spacing \times (1 + bundles)\right] = \text{"Not Satisfactory"}$

Shear Strength Provided by Concrete (LRFD 5.8.3.3)

$\boxed{\phi_v := 0.9}$

$$V_u := \left[\left(1.75 \times w_{LL} + 1.25 \times \gamma_c \times 6in\right) \times L_{Overhang} + 1.75 \times 50plf + 1.25 \times 36.6plf\right] \times w_{mem} = 1.73 \times kips$$

Use Simplified Calc Values
Min Av provided / Section is less than 16in deep / Foundation cantilever < 3dv

$Simplified := \text{"yes"}$

$\beta_w := 2$ $\theta := 45deg$

$V_c := 0.0316 \times \beta_w \times \sqrt{f'_c\ (ksi)} \times w_{mem} \times (0.9 \times d)$ $\phi_v \times V_c = 3.916 \times kips$

$Check_{C.D}\left(\phi_v \times V_c, V_u\right) = \text{"SATISFACTORY"}$

Slab edge deflection

Moment of Inertia: $I := \frac{(6in)^3}{12} = 216 \times \frac{in^4}{ft}$

Modulus of Elasticity: $E_c = 5.521 \times 10^8 \frac{lb}{ft^2}$

Edge Deflection
due to ped load:
(Cantilever)

$$\delta := \frac{w_{LL} \times L_{Overhang}^{\;4}}{8 \times E_c \times I} = 0.047 \times in$$

$\frac{L_{Overhang}}{\delta} = 1789$ Exceeds 300 per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges. section 5

SATISFACTORY

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Existing Wingwall to Carry Load Check - Not Satisfactory

$E_s := 29000ksi$ $F_y := 60ksi$ $f'_c := 6ksi$

Bar size (US number): $bar_{no} := 4$ $d_{no_{bar_{no}}} = 0.5 \times in$ $diam_{deform_{bar_{no}}} = 0.563 \times in$ $A_{no_{bar_{no}}} = 0.2 \times in^2$

$clear := 2in = 0.167\ ft$

Number of layers: $full_{layers} := 1$ $bundled := "no"$ $bundles := yes_{no}(bundled) = 0$ $\beta_1 = 0.75$

Width of member $w_{mem} := 1ft + 4in$ **Bars per Layer:** $n_{bars} := 1$ $spacing := \frac{w_{mem}}{n_{bars}} = 16 \times in$

Height of member $h_{mem} := 12in$ $A_s := \frac{A_{no_{bar_{no}}}}{spacing} \times full_{layers}$ $A_s \times w_{mem} = 0.2 \times in^2$ $clear_{spacing} = 15.438 \times in$

Design Moment: $M_{des} := \left[1.75 \times \left(OM_{LL} + OM_{LL.Lat.Railing} + OM_{LL.Vert.Railing}\right) ... + 1.25 \times \left(OM_{DC} + OM_{DC.Railing}\right)\right] \times w_{mem} = 9.71 \times 10^3 \times lb \times ft$

Service Moment: $M_{service} := OM_{service} \times w_{mem} = 6.478 \times 10^3 \times lb \times ft$

depth to rebar $d := h_{mem} - clear - \left[\frac{d_{no_{bar_{no}}}}{2} + \frac{full_{layers} - 1}{2} \times \left(\max\left(1 \times in, d_{no_{bar_{no}}}\right) + d_{no_{bar_{no}}}\right)\right]$

$d = 9.75 \times in$

Rectangular Compression Depth: $a_{req} := d - \sqrt{d^2 - \frac{M_{des}}{f'_c \times w_{mem}} \times \frac{2}{0.9 \times 0.85}}$ $a_{req} = 0.164 \times in$

Required Area of Steel: $A_{s.req} := \frac{0.85 \times f'_c \times a_{req}}{F_y}$ $A_{s.req} \times w_{mem} = 0.223 \times in^2$

Check compression for full yielding of actual steel

$a := \frac{A_s \times F_y}{0.85 \times f'_c}$ $a = 0.147 \times in$ $\phi M_n := 0.9 \times A_s \times F_y \times \left(d - \frac{a}{2}\right) \times w_{mem}$

$M_{des} = 9.71 \times kip \times ft$

$\phi M_n = 8.709 \times kip\ ft$

$Check_{C.D}\left(\phi M_n, M_{des}\right) = "Not Satisfactory"$

$\frac{\phi M_n}{M_{des}} = 89.7 \times \%$

Minimum Reinforcement for Flexural Member (AASHTO LRFD 5.4.2.6, 5.7.3.3.2)

allow 4/3 exception? $allow_{exception} := "yes"$

Modulus of Rupture for Normal Weight Concrete: $F_r := 0.24 \times \sqrt{f'_c \cdot (ksi)}$ $F_r = 587.9 \times psi$

Section Modulus: $S := \frac{h_{mem}^2}{6}$ $S = 288 \times \frac{in^3}{ft}$

Cracking Moment: $M_{cr} := F_r \times S$ $M_{cr} = 14.109 \times \frac{kip \times ft}{ft}$

$M_{cr.Check} = "Not Satisfactory"$

xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

Reinforcing Summary

use $n_{bars} = 1$ $bar_{no} = 4$

in $full_{layers} = 1$ (total bars $n = 1$)

spaced at $spacing = 16 \times in$

$w_{mem} = 1.333\ ft$ x $h_{mem} = 1\ ft$

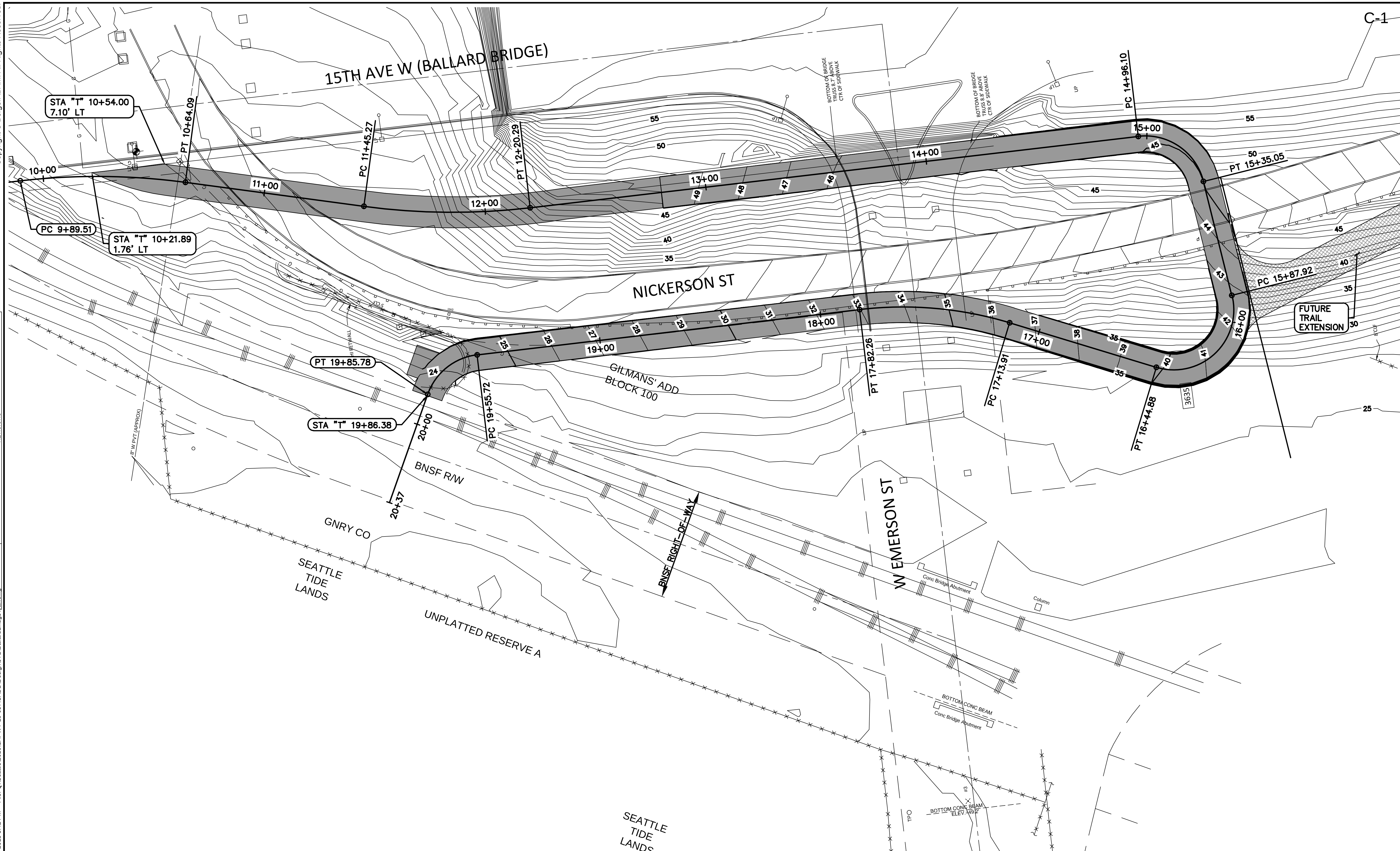
xXxXxXxXxXxXxXxXxXxXxXxXxXxXx

$1.2M_{cr} \times w_{mem} = 22.574 \times kip \times ft$

Ballard Bridge Sidewalk Widening Alternative Study
Seattle Department of Transportation, Seattle, Washington

Appendix I

Emerson Underpass Trail Exhibits



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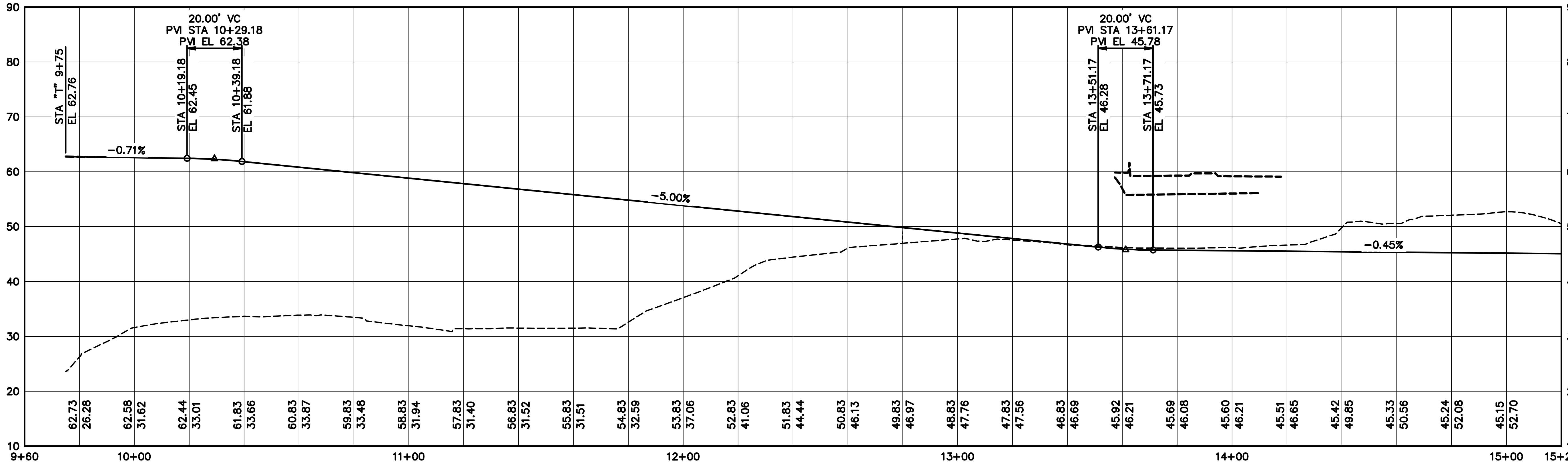
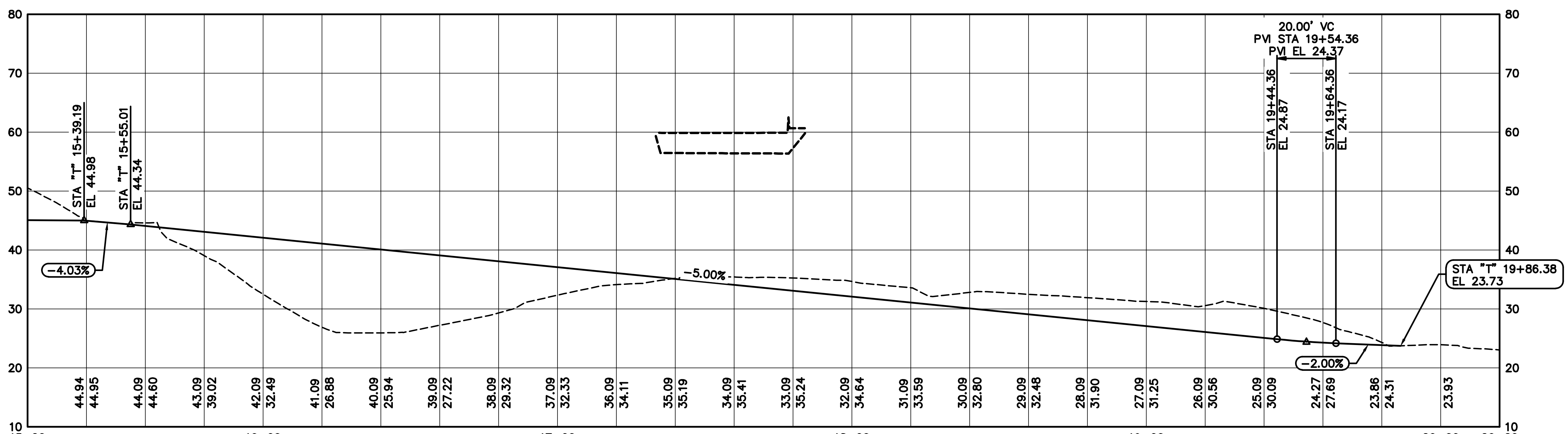
SHEET 1 OF 5



T PROFILE BEGIN TO 15+20



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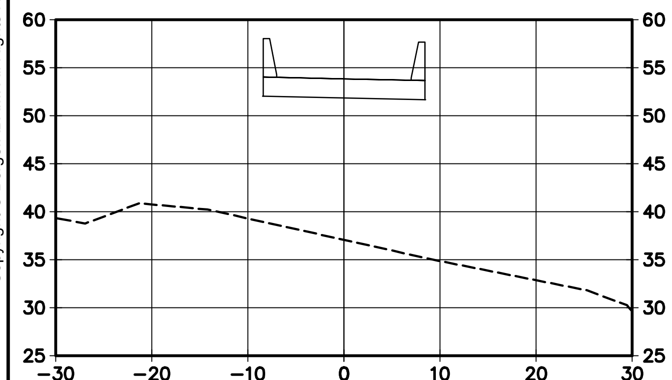

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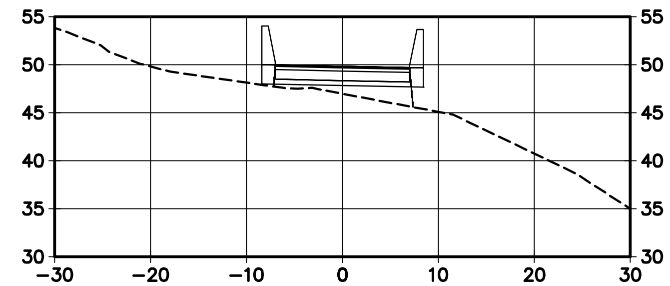

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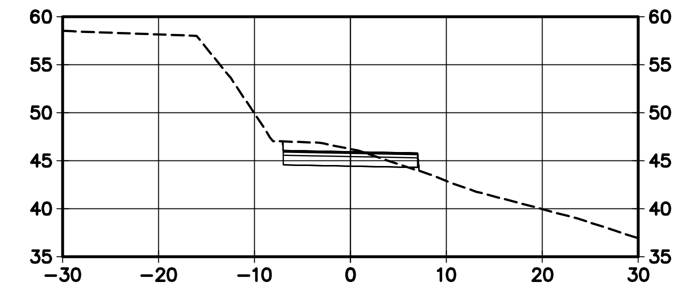

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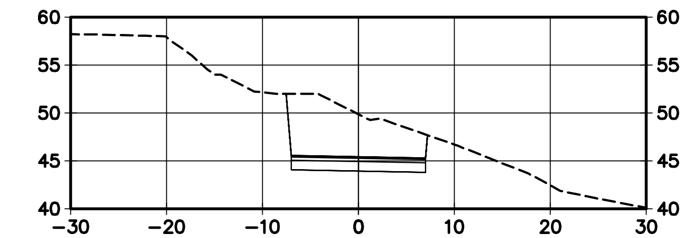

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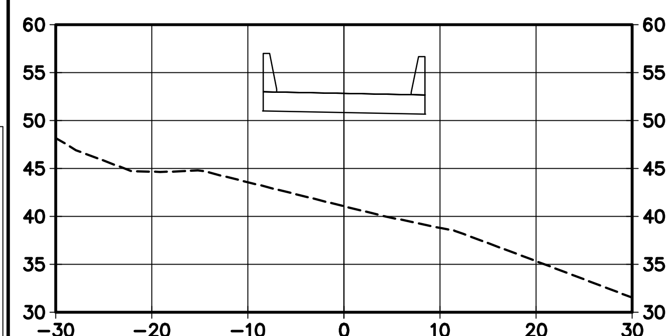

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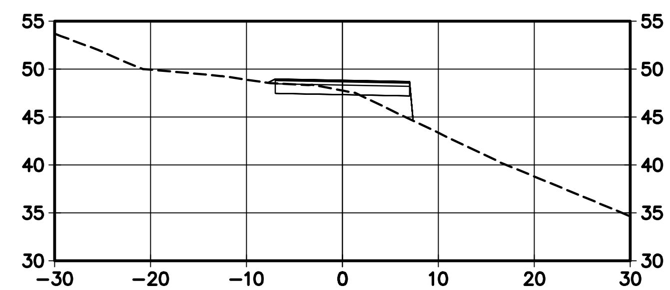

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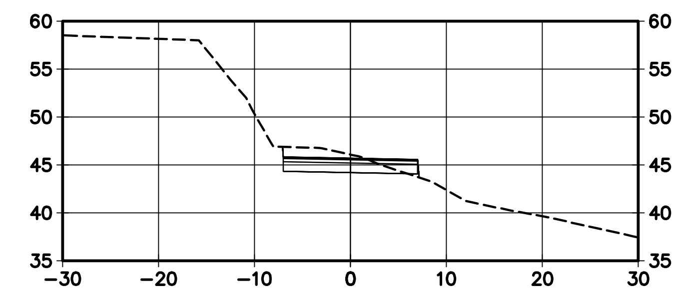

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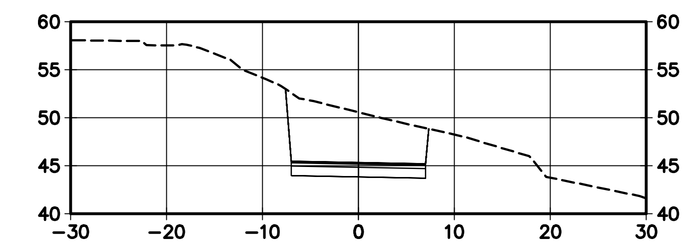

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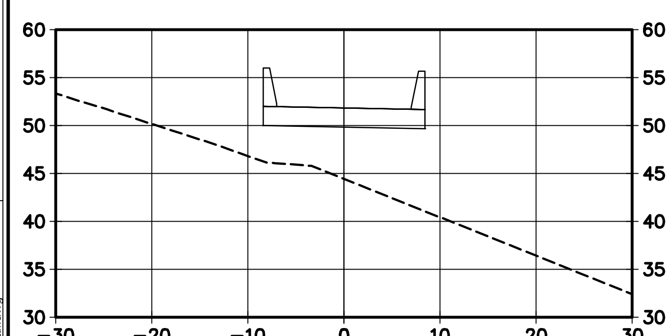

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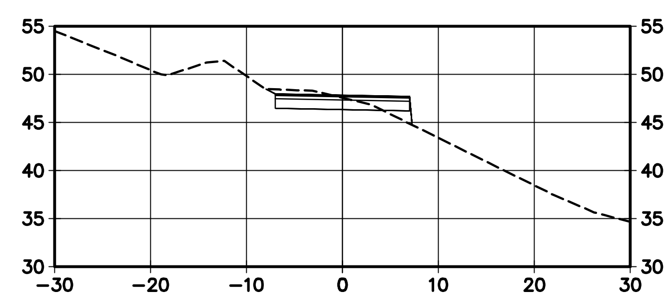

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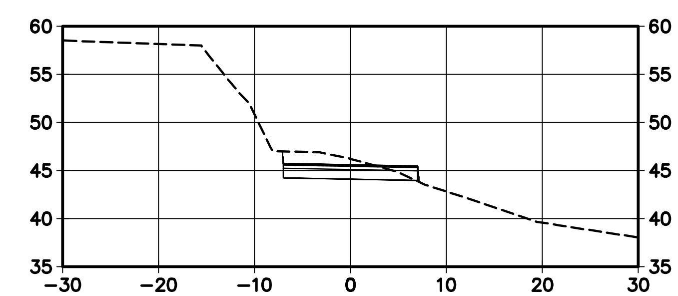

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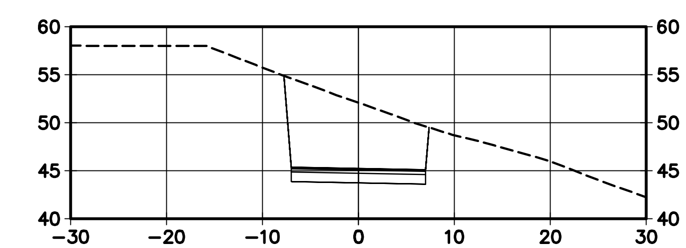

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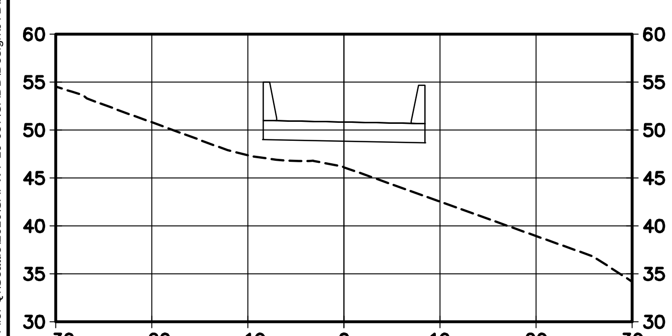

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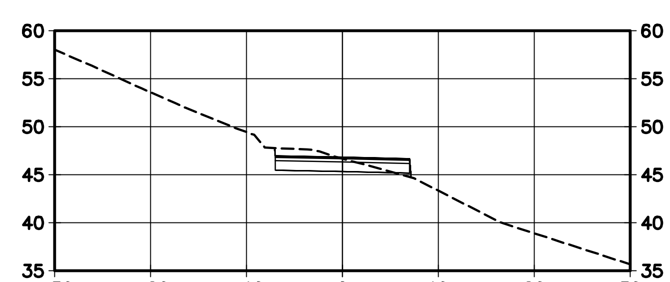

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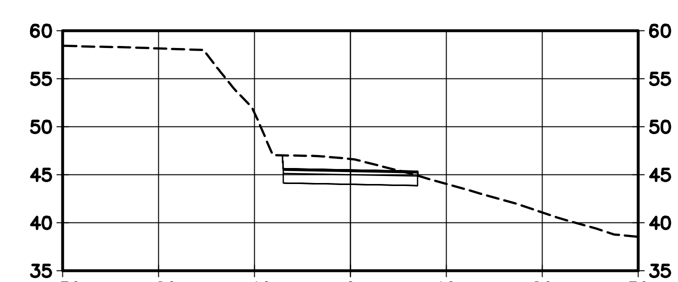

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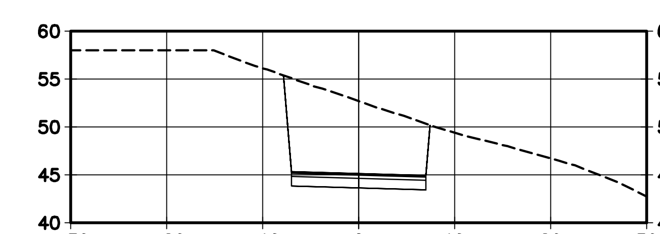

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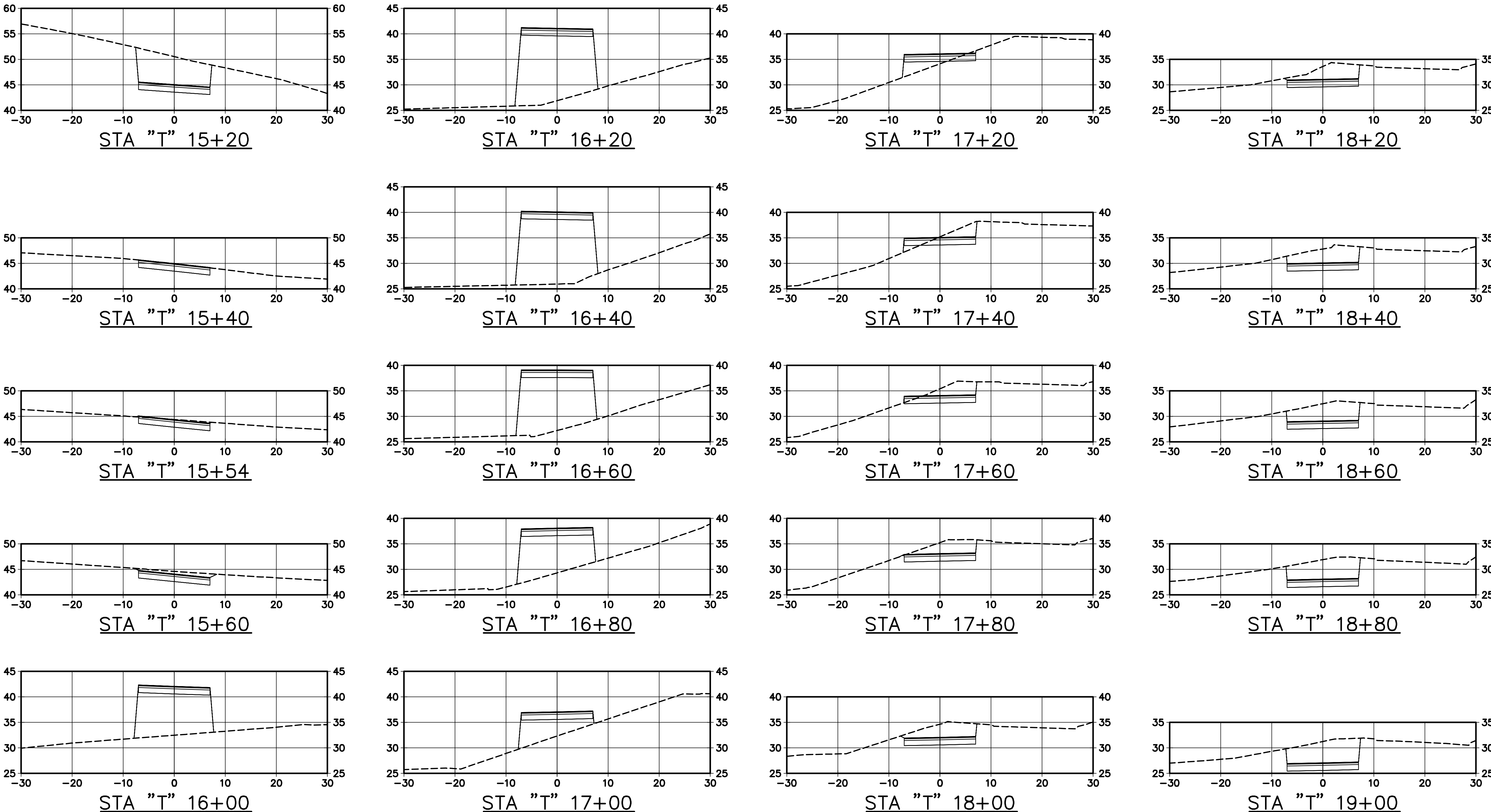

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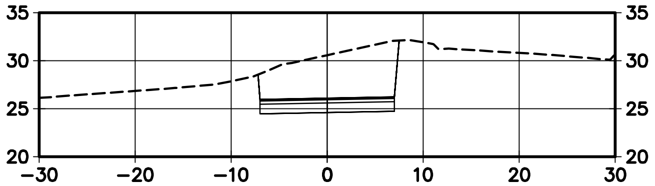

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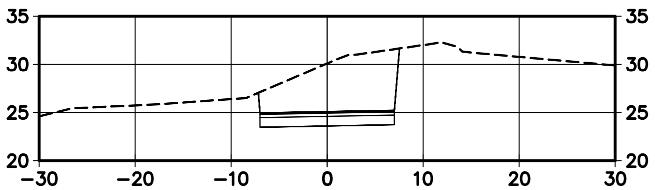

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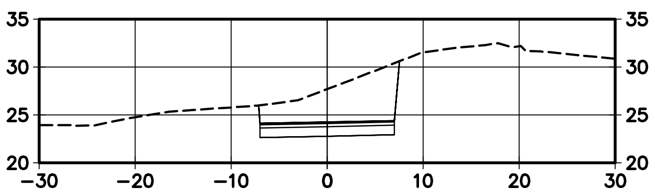

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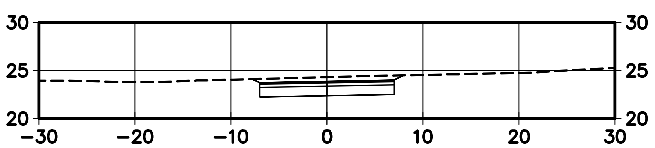

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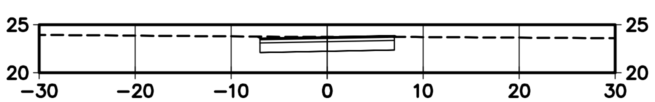

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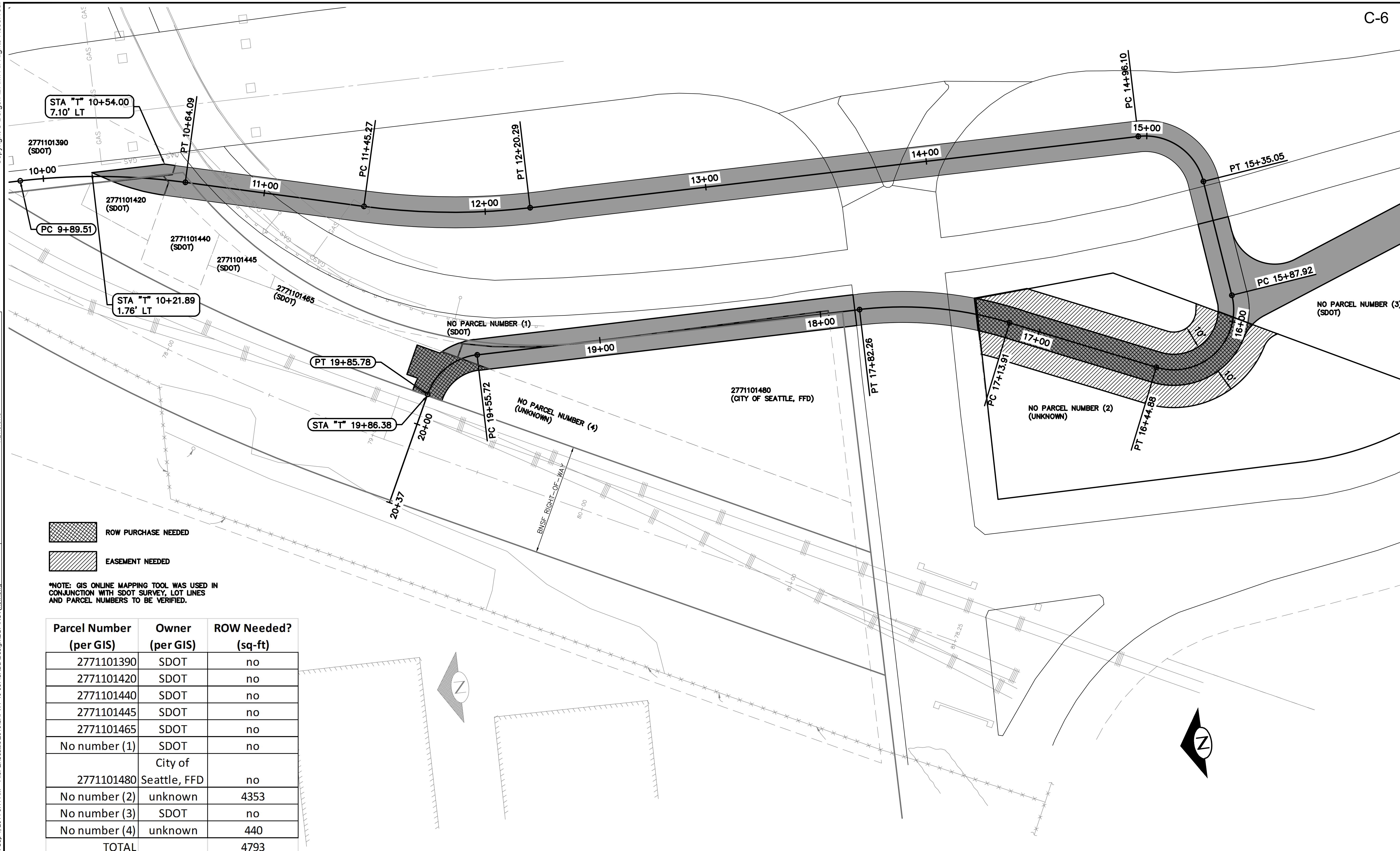

ROW PURCHASE NEEDED

EASEMENT NEEDED

*NOTE: GIS ONLINE MAPPING TOOL WAS USED IN CONJUNCTION WITH SDOT SURVEY, LOT LINES AND PARCEL NUMBERS TO BE VERIFIED.

Parcel Number (per GIS)	Owner (per GIS)	ROW Needed? (sq-ft)
2771101390	SDOT	no
2771101420	SDOT	no
2771101440	SDOT	no
2771101445	SDOT	no
2771101465	SDOT	no
No number (1)	SDOT	no
2771101480	City of Seattle, FFD	no
No number (2)	unknown	4353
No number (3)	SDOT	no
No number (4)	unknown	440
TOTAL		4793

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NANCY LOCKE
DEPARTMENT OF FINANCE & ADMINISTRATIVE SERVICES
SEATTLE, WASHINGTON .... 20 ..
BY: ......
PURCHASING AND CONTRACTING SERVICES DIRECTOR

NAME OR INITIALS AND DATE
DESIGNED ... JTB 3/18/13.
CHECKED ... GAB 3/18/13.
DRAWN ... JTB 3/18/13.
CHECKED ... GAB 3/18/13.

INITIALS AND DATE
REVIEWED:
PE .... CONST. ....
PROJ. MGR. ....
RECEIVED ....
REVISED AS BUILT ....

ALL WORK DONE IN ACCORDANCE WITH THE CITY OF SEATTLE STANDARD PLANS AND SPECIFICATIONS AND OTHER DOCUMENTS CALLED FOR IN SECTION 0-02.3 OF THE PROJECT MANUAL.

City of Seattle
Seattle Department of Transportation
ORDINANCE NO. .... APPROVED ....
FUND: ....
INSPECTOR'S BOOK ......

BALLARD BRIDGE
BRIDGE TO SHIP CANAL TRAIL CONNECTION

JOB NO. PC R/W CO
VAULT PLAN NO.
SHEET 1 OF 5

DATE | MARK | NATURE | MADE | CHK'D | REV'D
REVISIONS
VAULT SERIAL NO.

Last Saved by: Erin.pothier on: Sep 4, 2014 9:41 AM File: Q:\Seattle\2010\SAPWT-10-057\CADD\Design\EGP\ROW\new.dwg

Appendix J
Emerson Underpass Trail Cost Estimate (Planning Level)

Ballard Bridge Sidewalk Widening Concept Study
Proposed Emerson Underpass Trail Costs

Std Spec	WSDOT STD #	Item Description	Unit Price	Unit of Measure	Schedule A Quantity	Total Quantity	Schedule A Cost	Total Cost	Special Provision
		PREPARATION							
GSP	0050	REMOVAL OF STRUCTURE AND OBSTRUCTION	$ 25,000	L.S.	1	1	25,000	$ 25,000	2-02
REQ SPEC PROV	0100	REMOVING CEMENT CONC. SIDEWALK	$ 15	S.Y.	85	85	1,275	$ 1,275	2-02
REQ SPEC PROV	0108	REMOVING CEMENT CONC. CURB AND GUTTER	$ 8	L.F.	85	85	680	$ 680	2-02
REQ SPEC PROV	P-05	REMOVAL OF CONTAMINATED SOIL	$ 30	C.Y.	900	900	27,000	$ 27,000	2-09
GSP ITEM	0215	REMOVING MISCELLANEOUS TRAFFIC ITEM	$ 10,000	L.S.	1	1	10,000	$ 10,000	2-02
								$ 63,955	
		GRADING							
STD	0310	ROADWAY EXCAVATION INCL. HAUL	$ 18	C.Y.	1,800	1,800	32,400	$ 32,400	2-03
								$ 32,400	
		DRAINAGE							
STD	1160	UNDERDRAIN PIPE 6 IN. DIAM.	$ 8	L.F.	840	840	6,720	$ 6,720	7-01
								$ 6,720	
		STORM SEWER *							
		STORM SEWER SYSTEM - LUMP SUM	$ 25,000	L.S.	1	1	25,000	$ 25,000	7-05
								$ 25,000	
		HOT MIX ASPHALT AND SURFACING							
GSP	5767	HMA CL. 1/2 IN. PG 64-22	$ 70	TON	270	270	18,900	$ 18,900	5-04
STD	5100	CRUSHED SURFACING BASE COURSE	$ 25	TON	30	30	750	$ 750	4-04
								$ 18,900	
		EROSION CONTROL AND ROADSIDE RESTORATION							
		EROSION CONTROL - LUMP SUM	$ 40,000	L.S.	1	1	40,000	$ 40,000	8-01
		LANDSCAPE RESTORATION	$ 15,000	L.S.	1	1	15,000	$ 15,000	
								$ 40,000	
		TRAFFIC							
STD	4117	PEDESTRIAN BARRIER	$ 120	L.F.	2,050	2,050	246,000	$ 246,000	
STD	4415	TRAFFIC BARRIER	$ 100	L.F.	2,050	2,050	205,000	$ 205,000	
STD	6857	PLASTIC CROSSWALK LINE	$ 6	S.F.	400	400	2,400	$ 2,400	8-22
STD	6890	PERMANENT SIGNING	$ 1,000	L.S.	1	1	1,000	$ 1,000	8-21
GSP ITEM	6982	CONSTRUCTION SIGNS CLASS A	$ 20	S.F.	160	160	3,200	$ 3,200	1-10
STD	6973	OTHER TEMPORARY TRAFFIC CONTROL - PEDESTRIAN	$ 5,000	L.S.	1	1	5,000	$ 5,000	1-10
STD	6904	ILLUMINATION SYSTEM, COMPLETE	$ 100,000	L.S.	1	1	100,000	$ 100,000	8-20
GSP ITEM	6971	PROJECT TEMPORARY TRAFFIC CONTROL	$ 5,000	L.S.	1	1	5,000	$ 5,000	1-10
								$ 567,600	
		STRUCTURE							
STD	4006	STRUCTURE EXCAVATION CLASS A INCL. HAUL	$ 35	C.Y.	700	700	24,500	$ 24,500	2-09
		ELEVATED STRUCTURE TOTAL	$ 2,462,300	L.S.	1	1	2,462,300	$ 2,462,300	
		SOLDIER PILE WALL	$ 130	S.F.	1,624	1,624	211,120	$ 211,120	
STD	7170	BACKFILL FOR STRUCTURAL EARTH WALL INCL. HAUL	$ 25	C.Y.	320	320	8,000	$ 8,000	
STD	7169	STRUCTURAL EARTH WALL	$ 40	S.F.	432	432	17,280	$ 17,280	
GSP ITEM	4329	BRIDGE DECK	$ 140	S.F.	5,810	5,810	813,400	$ 813,400	
GSP ITEM	4339	EXPANSION JOINT SYSTEM STRIP SEAL - SUPERSTR.	$ 250	L.F.	85	85	21,250	$ 21,250	
STD	7008	SHORING OR EXTRA EXCAVATION CLASS B INCL. HAUL	$ 2	S.F.	3,000	3,000	6,000	$ 6,000	2-09
GSP ITEM	7037	STRUCTURE SURVEYING	$ 5,000	L.S.	1	1	5,000	$ 5,000	1-05
GSP ITEM	7038	ROADWAY SURVEYING	$ 2,000	L.S.	1	1	2,000	$ 2,000	1-05
GSP ITEM	7164	GRAVITY BLOCK WALL	$ 50	S.F.	520	520	26,000	$ 26,000	8-24
STD	7480	ROADSIDE CLEANUP	$ 5,000	EST.	-	-	5,000	$ 5,000	2-01
GSP ITEM	7571	FA-SITE CLEANUP OF BIO. AND PHYSICAL HAZARDS	$ 10,000	EST.	-	-	10,000	$ 10,000	1-07
REQ SPEC PROV	OI-08	UTILITY POTHOLING	$ 5,000	EST.	-	-	5,000	$ 5,000	8-31
REQ SPEC PROV	OI-01	AS-BUILT SURVEY AND RECORD DRAWINGS	$ 2,000	L.S.	1	1	2,000	$ 2,000	1-04
*Estimate provided by SDOT, lump sum for trail and bridge.								$ 3,618,850	
SUB TOTAL							4,389,175	$ 4,373,425	
CONTINGENCY							30%	$ 1,312,000	
TOTAL ESTIMATED CONSTRUCTION COST (2013)								$ 5,685,425	

City of Seattle
SDOT

Ballard Bridge Sidewalk Widening Concept Study
Proposed Emerson Underpass Trail Costs

Planning Level
Estimate of Probable Cost

Std Spec	WSDOT STD #	Item Description	Unit Price	Unit of Measure	Schedule A Quantity	Total Quantity	Schedule A Cost	Total Cost	Special Provision
		PREPARATION							
REQ SPEC PROV	0100	REMOVING CEMENT CONC. SIDEWALK	$ 20	S.Y.	1,100	1,100	22,000	$ 22,000	2-02
REQ SPEC PROV	0145	REMOVING CONC. BARRIER	$ 15	L.F.	5,600	5,600	84,000	$ 84,000	
N/A	N/A	LUMINAIRE REMOVAL	$ 150	EACH	28	28	4,200	$ 4,200	
								$ 110,200	
		HOT MIX ASPHALT AND SURFACING							
N/A	N/A	SURFACE PREPARATION	$ 5	S.F.	4,800	4,800	24,000	$ 24,000	
								$ 24,000	
		TRAFFIC							
STD	4117	PEDESTRIAN BARRIER (CONCRETE MIX - CL 4000)	$ 900	C.Y.	570	570	513,000	$ 513,000	
STD	6904	ILLUMINATION SYSTEM, COMPLETE	$ 950,000	L.S.	LUMP SUM	-	950,000	$ 950,000	8-20
								$ 1,463,000	
		#REF!							
N/A	N/A	DRILL HOLE 5/8" DIAMETER	$ 30	L.F.	680	680	20,400	$ 20,400	
N/A	N/A	DRILL HOLE 3/4" DIAMETER	$ 30	L.F.	890	890	26,700	$ 26,700	
N/A	N/A	EPOXY-COATED STEEL REINFORCING BAR	$ 2	LB	67,000	67,000	134,000	$ 134,000	
N/A	N/A	EPOXY BONDING COMPOUND	$ 20	EACH	2,300	2,300	46,000	$ 46,000	
N/A	N/A	REINFORCING BAR (DOWELED)	$ 1	LB	10,000	10,000	10,000	$ 10,000	
STD	4240	STRUCTURAL HIGH STRENGTH STEEL	$ 4	LB	157,000	157,000	628,000	$ 628,000	
								$ 865,100	
SUB TOTAL							2,462,300	$ 2,462,300	
CONTINGENCY							40%	$ 984,900	
TOTAL ESTIMATED CONSTRUCTION COST (2013)								$ 3,447,200	

BergerABAM

Project: BALLARD BRIDGE
TASK 16 - TRAIL
Subject: COST ESTIMATE
(CUT / FILL)

Sheet: 1 of
Job Number:
Designer: EGP
Date: 5/8/13
REVISIONS 6/20/13

- BRIDGE DECK : $140/SF
- STRUCTURAL EARTH WALL (MSE) : $40/SF
- SOLDIER PILE WALL : $130/SF

} NUMBERS FROM GREG BANKS

BRIDGE DECK STA 9+75 TO STA 12+60 = 285 LF + ~~70~~ 130 LF
STA 15+80 TO STA ~~16+50~~ 17+10 × ~~16~~ 14 FT (WIDTH)

~~5,680 SF~~ 5,810 SF

STRUCTURAL EARTH WALLS :

STA 12+60 TO STA 13+20 → CADD AREA : 133 SF
~~STA 15+45 TO STA 17+00 → CADD AREA : 564~~ 40
STA 15+45 TO STA 15+80 = 80 ~~127~~ 43
STA ~~16+50 TO STA 17+00~~ 17+10 TO STA 17+40

~~260~~ SF 216
× 2 WALLS

ASSUMES ℄ OF TRAIL, DELTA BETWEEN WALL HEIGHT AVERAGES OUT

~~520 SF~~ 432 SF

SOLDIER PILE WALLS :

STA 13+60 TO STA 15+40 → CADD AREA : 642 SF
STA 17+40 TO STA ~~17+85~~ 19+80 → CADD AREA : ~~45~~ SF 705 SF
~~STA 18+00 TO STA 19+25 → CADD AREA : 125 SF~~

~~812~~ SF 1,347 SF
× 2 WALLS

~~1624 SF~~ 2,694 SF

TOTAL COSTS :

- BRIDGE DECK = ~~5,680 SF~~ 5,810 × $140 = ~~$795,200~~ $813,400
- MSE WALLS = ~~520 SF~~ 432 × $40 = $ ~~20,800~~ $17,280
- SOLDIER PILE WALLS = ~~1,624~~ 2,694 × $130 = $ ~~211,120~~ $350,220

* ASSUMED BRIDGE DECK FOR FILL ZONES > 5 FEET
ASSUMED STRUCTURAL EARTH WALL FOR FILL ZONES 0 TO 5 FEET

Eng-002-062209

Project	BALLARD BRIDGE TASK 16 - TRAIL
Subject	COST ESTIMATE
Sheet	2 of
Job Number	
Designer	EGP
Date	5/23/13 6/20/13

BergerABAM

- PEDESTRIAN BARRIER (RAILING): 9+75 TO 20+00 = 1,~~025~~ FT × 2 SIDES = 2,050 LF

  $ ______

- EXPANSION JOINT: 9+75 TO 10+60 = 85 LF

  $ ______

- CURB REMOVAL: " " = 85 LF

  $ ______

- ~~H~~OT MIX ASPHALT: 9+75 TO 20+00 = 1,025 LF

  × ~~16~~ 14 FT WIDTH

  ~~16,400~~ SF 14,350

  3" THICKNESS

  0.25' × 16,400 = ~~4,100~~ 3588 C.F.

  ASSUME 150 lb/CF (UTAH DOT)

  $ ______ ⟶ 150 lb/CF (~~4,100~~ 3588 CF) = ~~615,000~~ 538,125 lb

  ÷ 2,000

  ~~308~~ 269 TONS

- WALL DRAIN (SOLDIER PILE WALLS): 13+60 TO 15+40 = 180 LF

  17+40 TO ~~17+85~~ 19+80 = ~~45~~ 240 LF

  ~~18+00 TO 19+25 = 125 LF~~

  ~~350~~ 420 LF

  x2 walls

  ~~700~~ LF 840 LF

  $ 10.00/LF

- CSBC: AT GRADE TRAIL = ~~17+00~~ 13+20 17+16 TO ~~18+00~~ 14+00 17+40 = ~~180~~ 110 LF

  4" THICKNESS?

  × ~~16~~ 14 FT WIDTH

  ~~2880~~ SF 1540 × 0.33' = ~~950~~ 508 CF

  ÷ 27 C.Y.

  ~~105~~ CY 20 CY

  ASSUME 2950 lb/CY (WSDOT)

  2950 lb/CY (~~105~~ 20 CY) = ~~309,750~~ 59000 lb ÷ 2000 ton = ~~155~~ 30 TONS

Eng-002-062209

Project BALLARD BRIDGE | Sheet 3 of

BergerABAM | Job Number

Subject COST ESTIMATE TASK 16 - TRAIL | Designer EGP

Date ~~5/29/13~~ 6/20/13

- BACK FILL ~~FOR~~ MSE WALL : ~~520~~ 432 SF WALL SURFACE

ASSUME : 2:1 SLOPES

AVE HEIGHT = ~~8~~ 10 FT → WIDTH = $2 \cdot \cancel{8}\,10' = \cancel{16}\,20$ FT

$\cancel{520}\,432\ SF \times \cancel{16}\,20\ FT = \dfrac{\cancel{8{,}320}\ 8{,}640\ CF}{27\ CY} = \cancel{924}\ 320\ CY$

- REMOVAL OF CONTAMINATED SOILS : SAME AS CUT ZONES

SOLDIER PILE WALL : ~~1,624~~ 2694 SF / 2

WIDTH : ~~16~~ 14 FT + (2 FT PER SIDE OF TRAIL)

$= \cancel{20}\ 18\ FT \times \cancel{812}\ 1347\ SF$

$= \dfrac{\cancel{16{,}240}\ 24{,}246\ CF}{27\ CY} = \cancel{1804}\ E.Y.$ 900 C.Y.

Appendix K
Historic Register Information/E-mail Correspondence

STATE OF WASHINGTON

DEPARTMENT OF ARCHAEOLOGY & HISTORIC PRESERVATION

1063 S. Capitol Way, Suite 106 • Olympia, Washington 98501
Mailing address: PO Box 48343 • Olympia, Washington 98504-8343
(360) 586-3065 • Fax Number (360) 586-3067 • Website: www.dahp.wa.gov

February 7, 2011

NOTE: THIS LETTER DOES NOT APPLY TO THIS PROJECT, BUT REFERENCES EARLIER BRIDGE SEISMIC RETROFITTING. A NEW DETERMINATION OF THE IMPACTS TO THE HISTORIC CHARACTER OF THE BRIDGE DUE TO THIS PROJECT WILL NEED TO BE MADE.

D. R. Peloquin
Chief, Waterways Management Branch
U. S. Coast Guard
Thirteenth Coast Guard District
915 Second Avenue
Seattle, WA 98174-1067

In future correspondence please refer to:
Log: 122310-06-USCG
Property: Ballard Bridge (15th Ave) Pier Strengthening
Re: NO Adverse Effect

Dear Mr. Peloquin:

Thank you for contacting the Washington State Department of Archaeology and Historic Preservation (DAHP). The proposed seismic stabilization to the Ballard Bridge has been reviewed on behalf of the State Historic Preservation Officer under provisions of Section 106 of the National Historic Preservation Act of 1966 (as amended) and 36 CFR Part 800. My review is based upon documentation contained in correspondences between our agencies and the Seattle Department of Transportation (SDOT).

Based on information provided to us by SDOT regarding the seismic retrofits that will be applied to the Ballard Bridge, a National Register of Historic Properties listed resource, I concur that the current project as proposed will have **no adverse effect** on this property. If additional information on the project becomes available, or if any archaeological resources are uncovered during construction, please halt work in the area of discovery and contact the appropriate Native American Tribes and DAHP for further consultation.

Thank you for the opportunity to review and comment. If you have any questions, please contact me.

Sincerely,

Matthew Sterner, M.A.
Transportation Archaeologist
(360) 586-3082
matthew.sterner@dahp.wa.gov

Cc: Mark Mazzola, SDOT

FULL RECORD DISPLAY

Current Record: 1 of 1 in NPS Digital Library

Go back to: Title List | Revise Search

For advanced viewing install **DjVu browser plugin**.

1. Choose the option for Autoinstallation
2. takes about 20 seconds
3. About DjVu and plugin help

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For advanced viewing install **DjVu browser plugin**.

1. Choose the option for Autoinstallation
2. takes about 20 seconds
3. About DjVu and plugin help

Choose format:
JPG | DjVu

Begin DjVu install

Ballard Bridge [Image]

URL:	http://pdfhost.focus.nps.gov/docs/NRHP/Text/82004231.pdf Link will open in a new browser window
URL:	http://pdfhost.focus.nps.gov/docs/NRHP/Photos/82004231.pdf Link will open in a new browser window
Publisher:	National Park Service
Published:	07/16/1982
Access:	Public access
Restrictions:	All Rights Reserved
Is Part Of:	Historic Bridges/Tunnels in Washington State TR
Format/Size:	Physical document with text, photos and map
Language:	eng: English
Note:	Spans Lake Washington Ship Canal
Item No.:	82004231 *NRIS (National Register Information System)*
Subject:	EVENT
Subject:	ARCHITECTURE/ENGINEERING
Subject:	ENGINEERING
Subject:	TRANSPORTATION
Subject:	STRUCTURE
Subject:	1900-1924
Keywords:	Dimock,A.H.;1917
Place:	WASHINGTON -- King County -- Seattle

Record Number: 389540

Record Owner: National Register of Historic Places

Historic Register Report

Historic Name: Ballard Bridge

Address: Spans Lake Washington Ship Canal
City: Seattle
County: King

Download nomination form

Historic Use: Transportation
Style: None
Built: 1919
Architect:
Builder: US Steel Corp.

Smithsonian Number: 45KI00261
Date Listed: 7/16/1982
Listing Status: WHR/NR
Classification: STR
Resource Count: 1
Area of Significance: Engineering
Level of Significance: Local
Listing Criteria: C

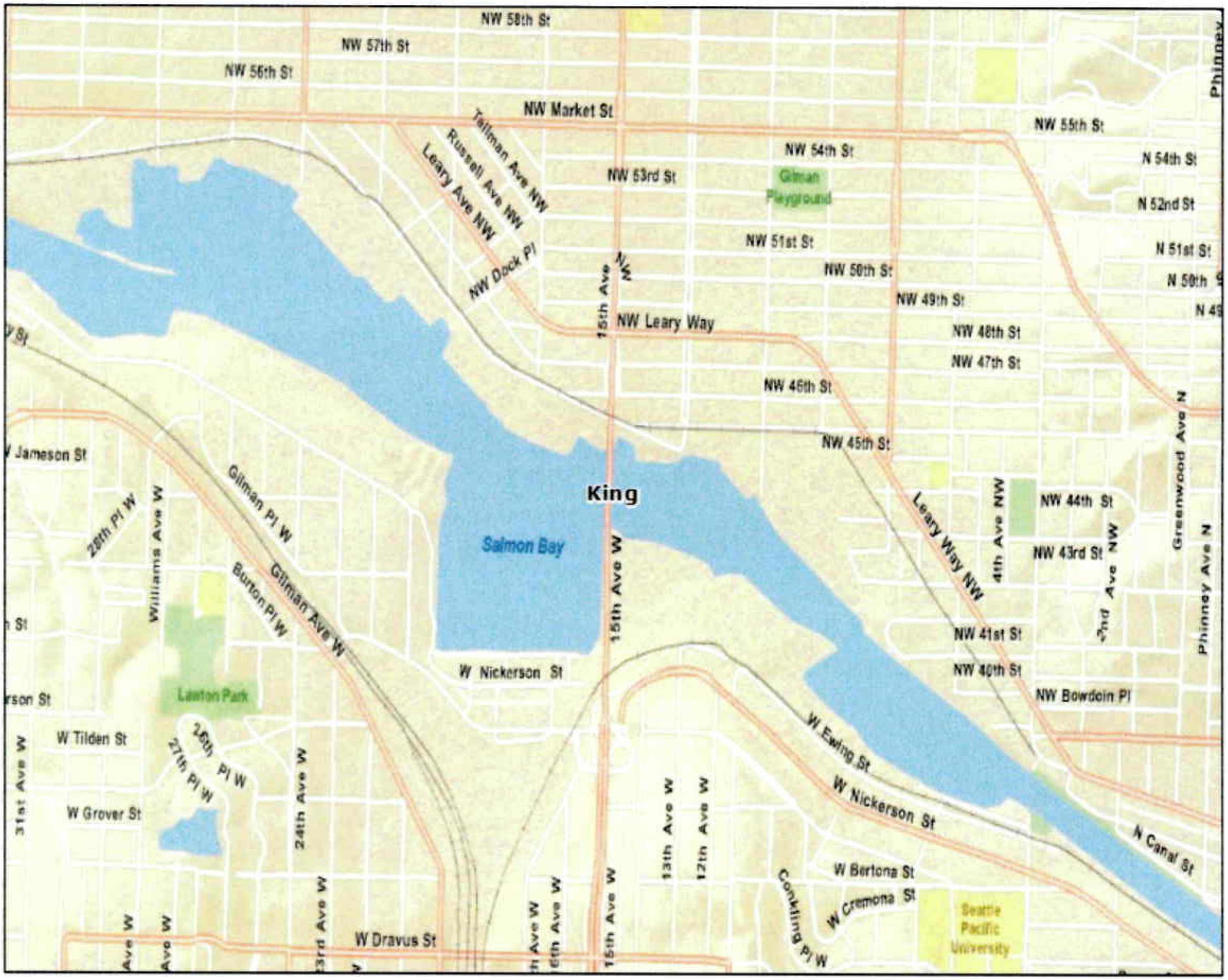
Statement of Significance

Photos

LEGEND

County Boundaries

World Street Map



Title: BB Map

Description:

665 1995 3990 6650
0 1330 2660 5320 Feet

NAD83 State Plane Washington South (HARN)

DISCLAIMER: This map is for reference purposes only. All distances and locations are approximate.

DEPARTMENT OF ARCHAEOLOGY & HISTORIC PRESERVATION

25N 3E 11, 12, 13, 14

KI 261

NAER INVENTORY

U.S. Department of the Interior
Heritage Conservation and Recreation Service

1. SITE I.D. NO

2 INDUSTRIAL CLASSIFICATION: Bridges, Trestles, and Aqueducts

MOVE: bascule — 7 6 5 0

3 PRIORITY: 1

5 DATE: 1915-1919

4. DANGER OF DEMOLITION? (SPECIFY THREAT): ☐ YES ☐ NO ☒ UNKNOWN

6. GOVT SOURCE OF THREAT — OWNER — ADMIN

7 OWNER/ADMIN: City of Seattle

8 NAME(S) OF STRUCTURE:
h. University Bridge
c. Fremont Bridge
a, b. Ballard Bridge

9 OWNER'S ADDRESS:
Engineering Department
Seattle Municipal Building, Room 704
Seattle, WA 98104

10 STATE / COUNTY: W A / 0 3 3 — COUNTY NAME: King — CITY VICINITY: Seattle — CONG DIST: 0 3

STATE / COUNTY — COUNTY NAME — CITY/VICINITY — CONG DIST

11. SITE ADDRESS (STREET & NO.):
Crossing: Lake Washington Ship Canal
h. 10 551150 5277780
c. 10 548920 5277150

12 EXISTING SURVEYS: ☐ NR ☐ NHL ☐ HABS ☐ HAER—I ☐ HAER ☐ NPS ☐ CL6 ☐ CONF ☐ STATE ☐ COUNTY ☐ LOCAL ☐ OTHER

13 SPECIAL FEATURES (DESCRIBE BELOW): ☐ INTERIOR INTACT ☐ EXTERIOR INTACT ☐ ENVIRONS INTACT

14

	UTM ZONE	EASTING	NORTHING	SIGN	SCALE	QUAD NAME
a	1 0	5 4 6 9 3 0	5 2 7 8 8 2 0		☒ 1:24 ☐ 1:625 ☐ OTHER	Seattle North, Washington
b	1 0	5 4 6 9 3 0	5 2 7 8 8 2 0		☒ 1:24 ☐ 1:625 ☐ OTHER	Seattle North, Washington

15 CONDITION: 70 ☐ EXCELLENT 71 ☐ GOOD 72 ☐ FAIR 73 ☐ DETERIORATED 74 ☐ RUINS 75 ☐ UNEXPOSED 76 ☐ ALTERED L. ☐ DESTROYED 85 ☐ DEMOLISHED

16 INVENTORIED BY: Lisa Soderberg — AFFILIATION: HAER/Washington State Bridge Inventory — DATE: September 1980

17. DESCRIPTION AND BACKGROUND HISTORY, INCLUDING CONSTRUCTION DATE(S), HISTORICAL DATES, PHYSICAL DIMENSIONS MATERIALS, EXTANT EQUIPMENT AND IMPORTANT BUILDERS ENGINEERS ETC

The construction of several moveable spans was incorporated into the design of Seattle's Lake Washington Ship Canal. Between 1915 and 1919, three double-leaf trunnion bascule bridges of the transverse cross-girder type were constructed to span the waterway at Fremont Avenue, at 15th Avenue Northwest, and at Eastlake Avenue. The bridges, which are the earliest examples within the State of a double-leaf bascule bridge, were designed by the City of Seattle under the direction of A.H. Dimock, City Engineer. They were erected under the supervision of F.A. Rapp.

The bascule bridge design was selected over a fixed span and vertical lift design. The fixed span design was eliminated immediately because it necessitated the construction of extremely long approaches. In a letter to the city council, the city engineer wrote that a vertical lift bridge would require 200 foot towers in order to provide the necessary vertical clearance of 150 feet. "Such towers ... of steel are far from pleasing ornaments to any waterfront."

(CONT OVER)

18 ORIGINAL USE: vehicular — PRESENT USE: vehicular — ADAPTIVE USE:

19 REFERENCES—HISTORICAL REFERENCES PERSONAL CONTACTS AND/OR OTHER

City Engineering Department files.
"Three Double-Leaf Bascule Bridges at Seattle, Washington," Engineering News-Record, Vol. 84, pp. 718-722.
"Service Performance of Grid Deck on University Bridge, Seattle," Engineering News-Record, 20 September 1934, p. 376.

(CONT OVER)

20 URBAN AREA 50,000 POP OR MORE? ☒ YES ☐ NO

21: N W

22 PUBLIC ACCESSIBILITY: ☐ YES LIMITED ☒ YES UNLIMITED ☐ NO ☐ UNKNOWN

23 EDITOR — INDEXER

50
51
52

24 LOCATED IN AN HISTORIC DISTRICT? ☐ YES ☒ NO — NAME — DISTRICT I.D. NO

Description (continued)

He emphasized the merits of the bascule bridge design and claimed that "the advantage of this type from the navigator's point of view is that it provides a perfectly clear and unobstructed channel permitting the passage of a vessel of any height. This feature of the bascule bridge was in direct contrast to the design of the lift bridge in which the height of the vessel passing beneath the bridge was limited by the height of the lift span.

The double-leaf trunnion bascule design adopted by the city of Seattle has its origins in a general design developed by the Chicago Department of Public Works in 1898. The three bridges consist of half-through type trusses with a horizontal top chord and a curved bottom chord. The trusses are raised and lowered by means of two counterweights that are built into the rear of the trusses, below the deck. These counterweights are composed of steel boxes that are filled with concrete. Two pockets were formed in the concrete to provide for a means of adjusting the weight according to wet and dry seasons.

The leaves are each operated by two direct current motors of 100 horsepower capacity at 550 volts. Each leaf was designed to be operated independently, and by one motor. The internal gears in the operating mechanism are composed of cast steel concave racks that were designed and patented by Alexander Van Babo, engineer of bridge design at the Chicago Department of Public Works. The gear trains drive operating pinions of forged steel that engage the innerfaces of the racks which are built into the counterweight arms of the trusses. There is also an emergency hand operating connection which can open the bridge in six hours. In 1928, auxiliary power equipment was placed in the three bridges.

All connections were assembled and reamed before the trusses were erected. The leaves were erected in the horizontal position. However, when one leaf was completed it was raised to the vertical position so that half of the channel remained unobstructed throughout construction.

Because the Federal government assumed a share of the cost of the canal, it placed conditions upon the general proportions of the bridges. The government maintained that "the structures should be of a permanent character and should give a clear channel width of 200 feet with a clearance height of 30 feet above the lake level for a width of 150 feet." All three bascule spans are greater than 200 feet in length. The curb of the Fremont Avenue Bridge is 37 feet above the waterline. The clearance height of the other two bridges is 52 feet, substantially above the height set by the Federal government. The additional height enabled small craft to pass beneath the bridges and minimized the number of openings. Because of the greater height of the Eastlake Avenue and 15th Avenue Bridges, there was no need to construct counterweight pits. The three bridges were each 40 feet wide and were designed to carry a double-track railway.

Construction began first on the University Bridge at Eastlake Avenue which was to replace two temporary timber draw spans. However, the 291 foot structure which consists of a 218 foot bascule span, was not completed until 1919 because of delays in carrying out specifications for the substructure. The massive, concrete substructure is 20 feet thick, 65 feet high, and 40 feet wide. The foundation rested directly on firm material on one side of the channel. However, on the other side of the channel, it was necessary to drive deep pile foundations in order to support the bridge. Booker, Kiehl, and Whipple were the contractors for the substructure. The United States Steel Products Company was the contractor for the superstructure. Construction was supervised by E.K. Triol.

The total cost of constructing the University Bridge which included a permanent steel span and two temporary untreated timber trestle approaches was $825,275, almost twice the cost of each of the other two bascule bridges. This was due to the cost of the massive concrete foundations and to the reletting of portions of the work at wartime prices.

Description (continued)

In 1933, an open mesh deck was installed to reduce the floor weight which permitted the widening of the roadway. The decking was designed and built by the Irving Iron Works of Long Island City, New York. Shop-welded cantilever girders were extended from the steel span to support the two additional traffic lanes.

The 502 foot bridge at Fremont Avenue was completed in 1917, and provided the primary entranceway to the community of Fremont. The steel for the 242 foot bascule span was fabricated by the Pacific Coast Steel Company. The United States Steel Products Company was the contractor for the superstructure. The substructure was built by the Pacific States Construction Company. In contrast to the University Bridge, permanent concrete approaches were built initially at Fremont Avenue by the West Coast Construction Company. The Fremont Avenue Bridge was equipped with four 100 horse-power motors. The total cost of the bridge was $410,000. In 1928, the original wood block paving was removed and replaced with open, steel pavement. At this time, new operating motors with hydraulic variable speed transmission were also added. These motors were considered to be a "new venture in moveable bridge machinery."

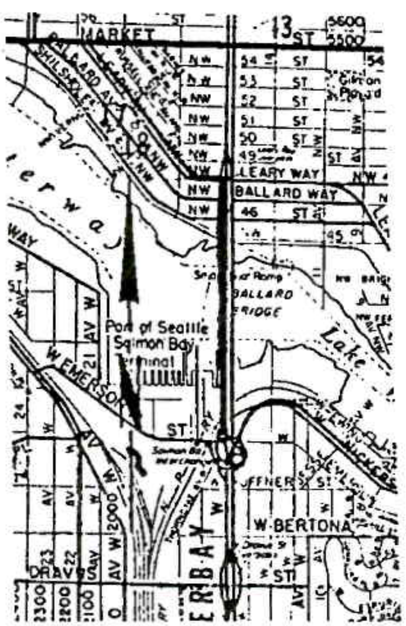
In 1917, the 15th Avenue N.W. Bridge was also completed, firmly linking Seattle and Ballard. The 295 foot structure which consisted of a 218 foot bascule span cost $479,000. The steel was fabricated by the Dyer Brothers of San Francisco. Hans Pederson was the contractor for both the substructure and superstructure, and J. Charles Rathburn was the city's superintendent for the construction of the bridge. In 1941, the temporary approaches were replaced by permanent approach spans. The four towers were replaced by a single tower in 1969.

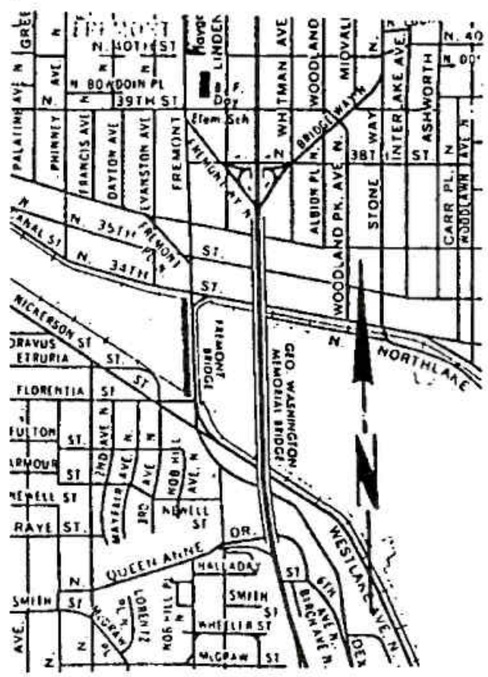
The design engineers in Seattle articulated the importance of aesthetics in city bridge design. On April 20, 1914 the city engineer wrote a letter to the city council: "of late years, it is recognized that it may be possible to secure graceful and pleasing lines, even in steel structures, without spending any large additional amount of money. It is fortunately possible owing to the height at which our bridges will be built above the water level to secure equal mechanical efficiency with a well balanced and pleasing effect." D.R. Huntington, City Architect, was responsible for the architectural treatment of the piers of the three bascule bridges. The massive, concrete piers of the University Bridge and the handsome towers on the Fremont Bridge provide an appropriate architectural frame for the passageway between Puget Sound and Lake Washington. However, the architectural treatment of these three bascule bridges do not equal the monumental stature of the cross-girder bascule bridge built across the canal at Montlake Avenue in 1924.

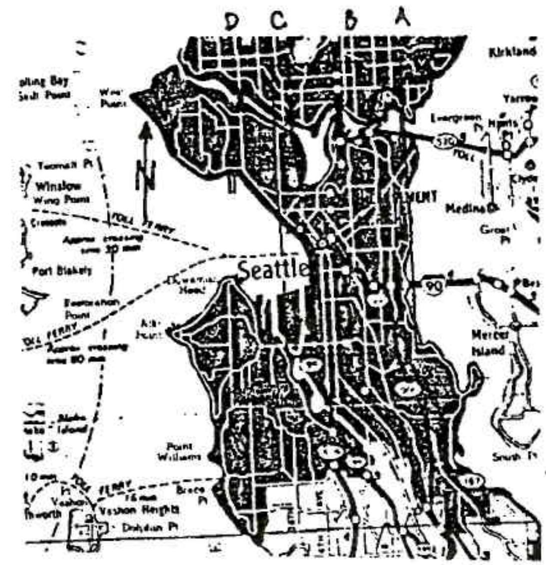
References (continued)

F.A. Rapp, "Heavy Foundation Work for Bascule Bridge at Seattle," Engineering News-Record, 15 April 1920, pp. 774-776.
Letter from City Engineer to City Council, April 20, 1914.

25. Sketch Map of Location







A Montlake Avenue Bridge
B University Bridge
C Fremont Bridge
D Ballard Bridge

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(1)

BALLARD BRIDGE SEATTLE

KI 0261

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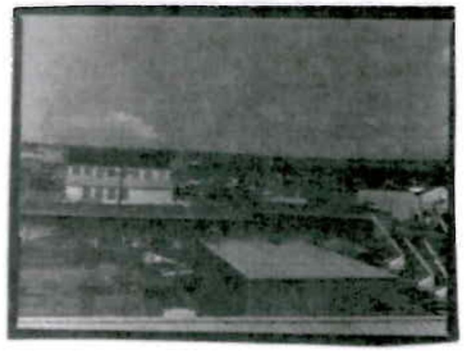

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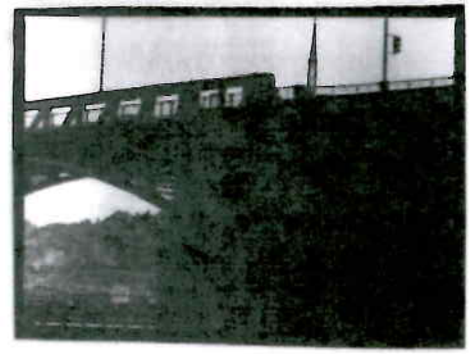

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② BALLARD BRIDGE SEATTLE KI 0261

17 2 10

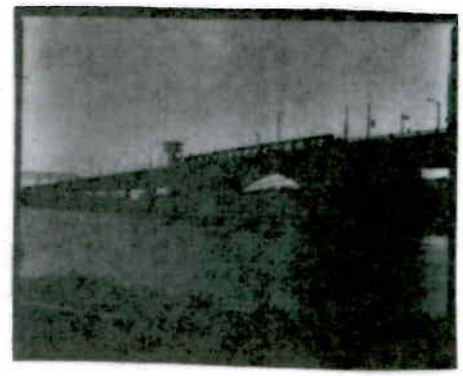
17 2 13

17 2 16

17 2 11

17 2 14

17 2 17

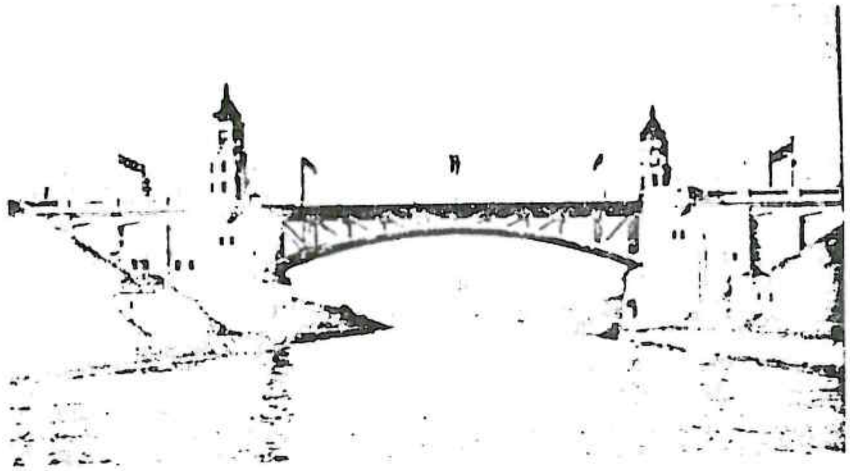


17 2 12

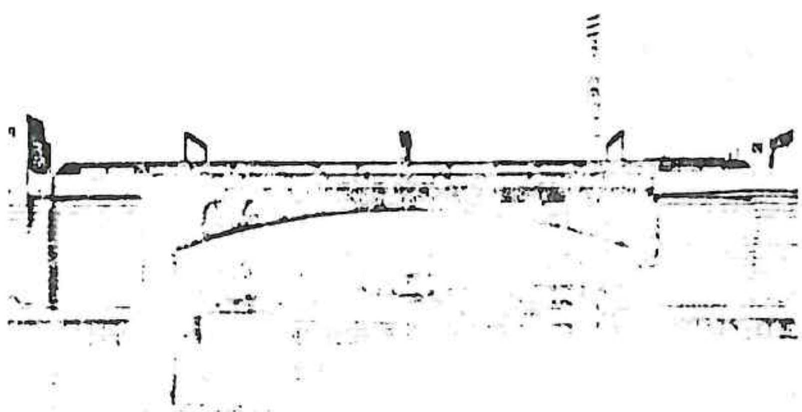
17 2 15

7 2 18

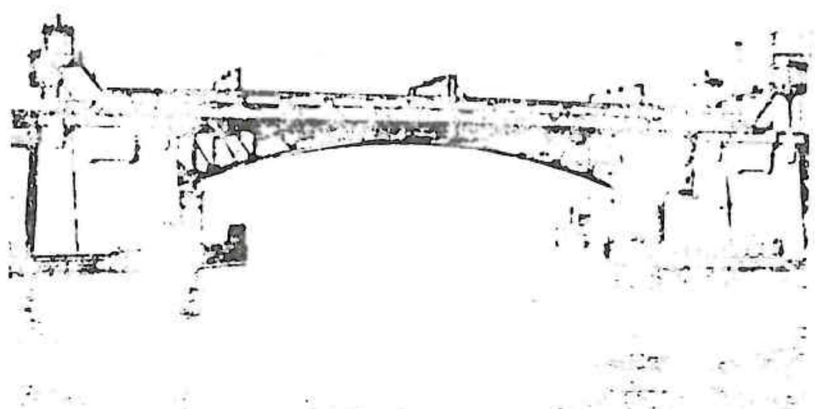
CANAL BRIDGES



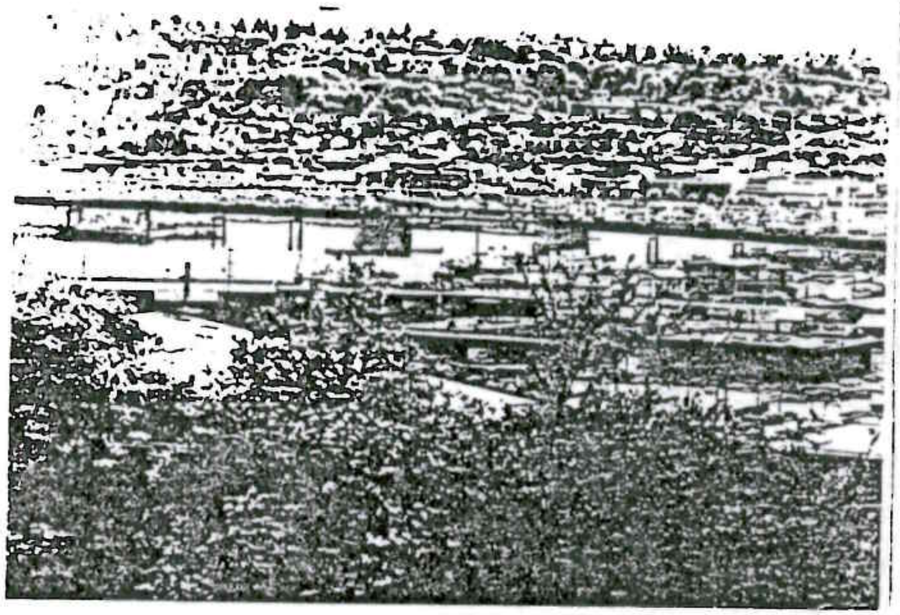
Montlake Bridge



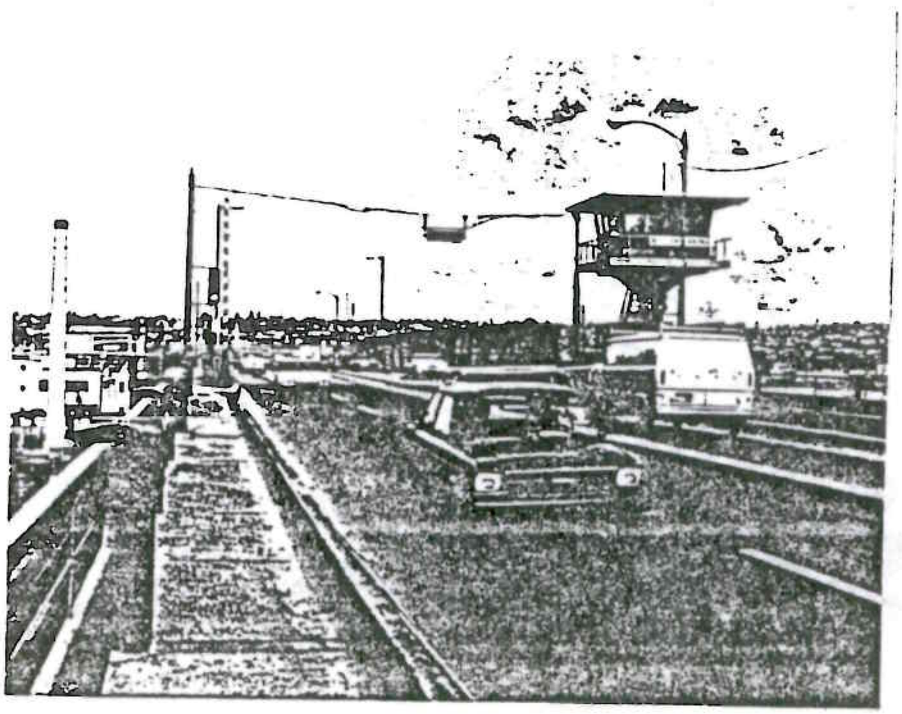
University Bridge



Ballard Bridge



Ballard Bridge,
side elevation, looking west



Ballard Bridge,
looking south

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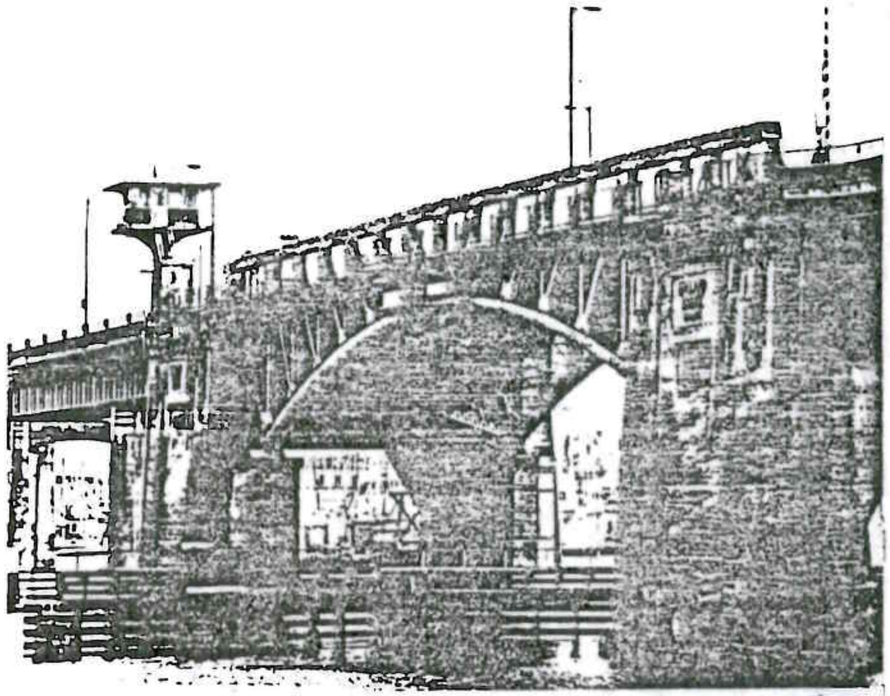
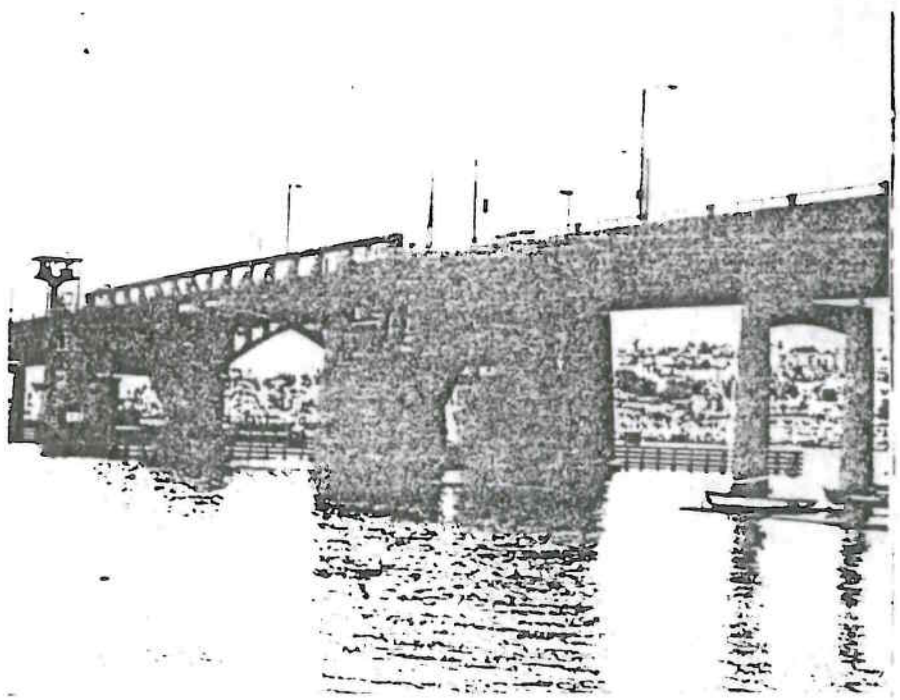


Ballard Bridge,
side elevation, looking east



Ballard Bridge,
side elevation, looking east

From: Banks, Greg <Greg.Banks@abam.com>
Sent: Monday, April 29, 2013 4:03 PM
To: Teo, Yuling
Cc: Zimmerman, Connie; Hoglund-Gray, Ginny; Fernandes, Bob
Subject: RE: Ballard - Sidewalk Widening

Yuling,

I think we are saying the same thing and will work on rewording things a bit to make it clearer.

We think the "Do No Harm" philosophy probably does not strictly apply here without some mathematical proof. We are increasing the mass on the retrofitted structure without adding any additional substructure, and thus, the demands will increase. Therefore, it seems we are "harming" the retrofitted structure to some small degree. We are recommending this be assessed by analyzing the retrofitted structure using the additional mass from the widening. It is our opinion that everything will pencil out and the harm will not rise to the level of requiring additional retrofit measures. In other words, if we had done the widening at the same time as the retrofits we would likely end up with the same retrofits. We have been avoiding using the words "do no harm" because it leads to arguments about the definition of the word "harm." It would be better to simply say it needs to be checked and we can then show the retrofitted structure is OK.

We referenced WSDOT policy on waiving the need to reanalyze the retrofitted structure as an example of a policy that SDOT could adopt for this project. We have looked at the increases in added mass and came to the conclusion that engineering judgment is insufficient to waive the analysis, at least for the 10-foot widening's. We can work on wording this better or remove it altogether.

Bob Fernandes and I are available to discuss if necessary. Also, are you available this next Wednesday (08 May 2013) to talk about the Post Earthquake Inspection Manual?

Regards-
Greg

From: Teo, Yuling [mailto:Yuling.Teo@seattle.gov]
Sent: Friday, April 26, 2013 11:55 AM
To: Banks, Greg
Cc: Zimmerman, Connie; Hoglund-Gray, Ginny; Fernandes, Bob
Subject: RE: Ballard - Sidewalk Widening

Greg,
I am able to follow your intent to describe the implication of the added mass to the performance of the existing structure, but I think the write-up needs revisions. Two issues I see with the write-up:

- It is misleading to the readers by referring to a WSDOT's policy in the BDM for a SDOT's project study.
- The 10% or less additional mass threshold for waiving retrofit requirement is a not a hard number; rather, the key is to determine if the seismic performance impact to the existing structure elements is **insignificant**. This involves engineering judgment. The fundamental requirement is to demonstrate the "Do No Harm" effect.

Similar to WSDOT's policy, we would want to separate the seismic risk of the existing bridge from the widening. WSDOT calls for the evaluation of C/D ratios for pre and post widening to demonstrate the Do-No-Harm effect. If the existing structure is already at seismic risk and the post-widening C/D is less than pre-widening C/D, seismic retrofit is required. If the post C/D is more than the pre C/D yet the element is at seismic risk, we would retain the decision to seismic retrofit the element. Is our structure which has just been retrofitted at seismic risk after the widening? It is your judgment to provide for this Study. You may use the 10 percent threshold based on your experience as well as referring to WSDOT's practice. SDOT reserves the right to confirm your judgment by appropriate analysis in the future, outside of this Study. Cost contingency for this future effort should be captured in this Study.

The data you presented in the table is good info, but for non-technical readers, it is perhaps more meaningful to describe the Do-No-Harm policy that you're trying to meet. As far as for the engineering judgment to the Do-No-Harm, one thing to be cautioned of is the assessment of capacity protected elements. If following the BDM 4.3.1 roadmap in comparing the before and after widening C/D ratios, the actual impact to the existing capacity protected elements may not be obvious. The demand is the forces associated with the column overstrength moment capacity and thus the before and after C/D ratios will not change. This does not tell you if the capacity protected elements are being made worse or not.
SDOT would like to know the actual impact. You may address this as a design contingency for future analysis in this Study.

Hope this helps. Feel free to give me a call if you have any questions.

YULING TEO
206-733-9244 (Tel)

From: Banks, Greg [mailto:Greg.Banks@abam.com]
Sent: Wednesday, April 24, 2013 5:18 PM
To: Teo, Yuling
Cc: Zimmerman, Connie; Hoglund-Gray, Ginny; Fernandes, Bob
Subject: Ballard - Sidewalk Widening

Yuling:

See italicized text and table below. When we update the subject report, we were thinking about adding this. Any comments? Please call to discuss. Thanks.

Per Section 4.3 of the WSDOT Bridge Design Manual a seismic analysis of a bridge widening without new substructure may be waived with the Owner's approval if the added mass from the widening is 10% or less of the original structure weight. Per the table below, all combinations of widening considered result in an increase in added mass of less than 10% of the structure weight. When considering solely the weight of the superstructure, the only combination that exceeds 10% of the superstructure mass is the 10-foot widening (both sides) for segments 3, 5 and 6. Based on our experience with the seismic retrofit design and the preparation of load ratings for these structures, we are of the opinion that the retrofitted capacity/demand ratios are high enough to accommodate this added mass. However, this should be confirmed by appropriate analysis if the City determines it would like to construct the 10 foot widening on both sides of the bridge.

Superstructure Mass Increase Percentages for Sidewalk Widening Alternatives						
		Widening Alternatives				
		6ft	10ft	6ft & 6ft	6ft & 10ft	10ft & 10ft
Segment	2	1.0%	1.5%	2.0%	2.5%	3.0%
	3 & 5	2.5%	5.0%	5.0%	7.5%	10.0%
	6	3.2%	5.8%	6.4%	9.0%	11.6%
	7	1.3%	2.9%	2.6%	4.2%	5.8%

Regards-
Greg

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BergerABAM

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Fax 206/431-2250
Email greg.banks@abam.com
33301 Ninth Avenue South, Suite 300
Federal Way, Washington 98003-2600
http://www.abam.com

Pothier, Erin

From:	Banks, Greg
Sent:	Tuesday, June 25, 2013 1:54 PM
To:	Pothier, Erin
Subject:	FW: Ballard Bridge Bike Widening Study

From: Jason Holdridge [mailto:jmh@cetransportation.com]
Sent: Thursday, February 07, 2013 11:54 AM
To: Anderson, Susan
Cc: Mark Yand (mcy@dksassociates.com); Banks, Greg
Subject: Re: Ballard Bridge Bike Widening Study

Susan,
Our calculations show about 40 poles. So the ballpark for the illumination system including poles, bracket arms, fixtures, and wiring would be about $450,000. The conduit work assuming SCL will want at least 2-2" conduits and also assuming they will be exposed under the bridge with NEMA boxes, this would be about $500,000. So total would be about $950,000.

Let me know if you have any questions.

Thanks,
Jason

Jason Holdridge, PE, PTOE
CONCORD ENGINEERING
TRANSPORTATION CONSULTING

710 2nd Avenue, Suite 830
Seattle, WA 98104
Ph: 206.682.0567
Cell: 206.604.5358
jmh@cetransportation.com

On Thu, Feb 7, 2013 at 11:28 AM, Anderson, Susan <Susan.Anderson@abam.com> wrote:

And do you have a ballpark cost for illumination?

From: Jason Holdridge [mailto:jmh@cetransportation.com]
Sent: Thursday, February 07, 2013 11:27 AM
To: Anderson, Susan

Cc: Mark Yand (mcy@dksassociates.com); Banks, Greg
Subject: Re: Ballard Bridge Bike Widening Study

Susan,

Sorry, I labeled the wrong file. Attached are the lighting calcs. I should have the sketches and will take a look.

Thanks,

Jason

Jason Holdridge, PE, PTOE
CONCORD ENGINEERING
TRANSPORTATION CONSULTING

710 2nd Avenue, Suite 830
Seattle, WA 98104
Ph: 206.682.0567
Cell: 206.604.5358
jmh@cetransportation.com

On Thu, Feb 7, 2013 at 11:11 AM, Anderson, Susan <Susan.Anderson@abam.com> wrote:

Jason,

Your attachment was not lighting calcs. Our sketches haven't changed since we sent them to you. Do you need a cadd file?

Susan

From: Jason Holdridge [mailto:jmh@cetransportation.com]
Sent: Thursday, February 07, 2013 11:01 AM
To: Anderson, Susan
Cc: Mark Yand (mcy@dksassociates.com); Banks, Greg
Subject: Re: Ballard Bridge Bike Widening Study

Susan,

Attached are preliminary results of the lighting analysis using 120 LED fixtures with 12 foot luminaire arms. This analysis assumed the poles will be mounted on the outside of the structure. We are getting about 160' spacing with a staggered layout. The light levels we assumed were based on SDOT's Right-of-Way Lighting Level Design Guidelines, which requires a light level of 2.0 Average Illuminance (fc).

As for the conduit, can you send me your current sketches? That was the thing I was most concerned with when I looked at the original sketches you sent me.

Thanks,

Jason

Jason Holdridge, PE, PTOE
CONCORD ENGINEERING
TRANSPORTATION CONSULTING

710 2nd Avenue, Suite 830
Seattle, WA 98104
Ph: 206.682.0567
Cell: 206.604.5358
jmh@cetransportation.com

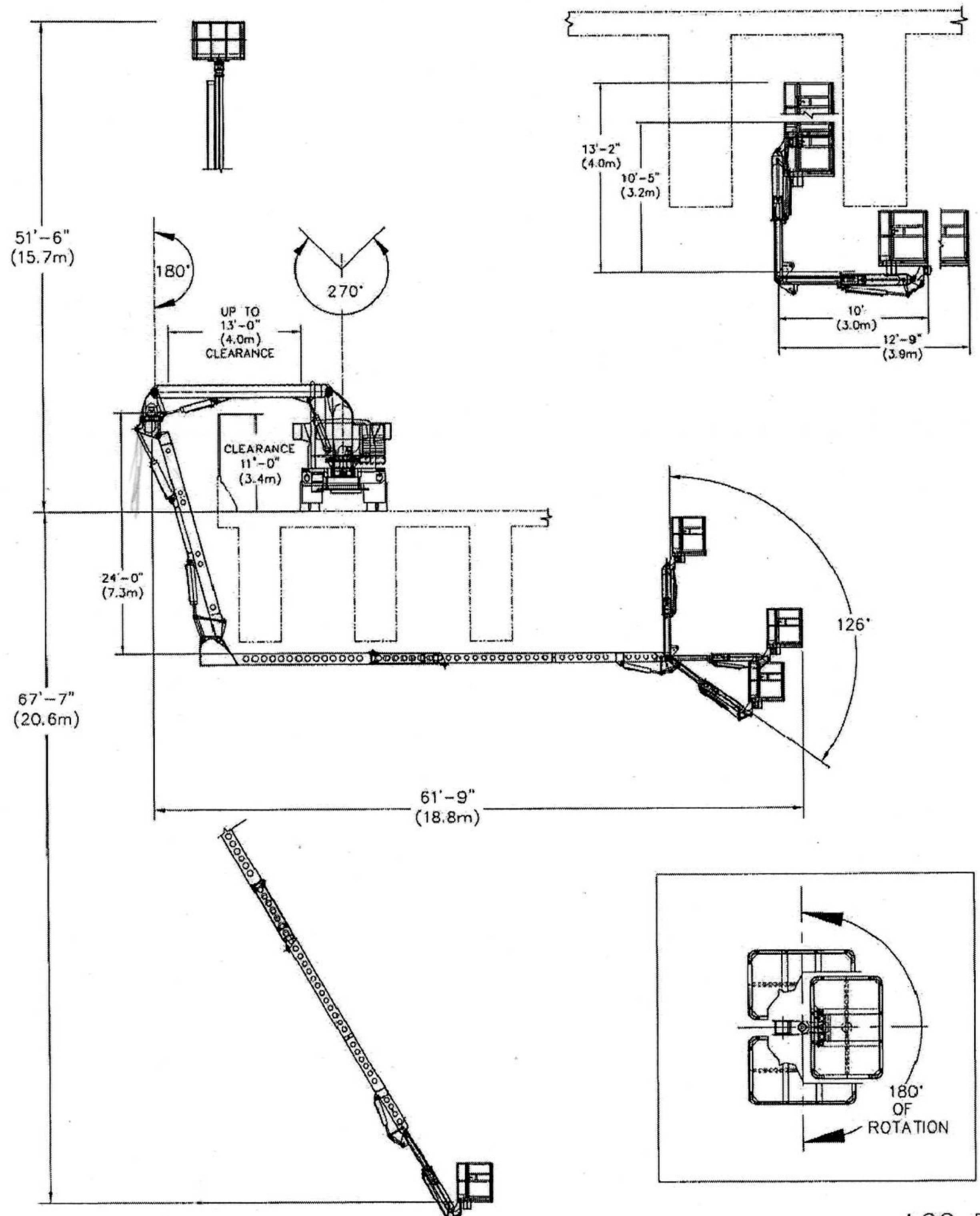
On Thu, Feb 7, 2013 at 10:30 AM, Anderson, Susan <Susan.Anderson@abam.com> wrote:

Hi Jason,

Do you have any preliminary results to report? We need to show conduit feed on our sketches. Can you comment?

Thanks,

Appendix L
Under Bridge Inspection Truck (UBIT)



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PATTERN