



Bridging Historic Downtown Troutdale and The Confluence Site

Spring 2021
Troutdale

Evan Kristof

PSU Civil & Environmental Engineering Capstone



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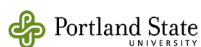
Troutdale

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Evan Kristof

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Acknowledgments

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We would also like to thank Hardman Geotechnical Services Inc (HGSi) for allowing us to use their tools and laboratory equipment to gather data on-site. This hands-on experience was invaluable to our learning process and would not have been possible without their help.

We are honored to contribute in some small way to the betterment of the community and acknowledge that we truly do stand on the shoulders of the giants who have come before.

This report was prepared as part of a class project for the Civil and Environmental Engineering Project Management and Design course at Portland State University. The contents of this report were developed by the student authors and do not necessarily reflect the views of Portland State University. The analyses, conclusions, and recommendations contained in the report should not be construed as an engineering report or used as a substitute for professional engineering services.

This report represents original student work and recommendations prepared by students in the Sustainable City Year Program for the City of Troutdale. Text and images contained in this report may not be used without permission from the University of Oregon.

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About SCI

The Sustainable Cities Institute (SCI) is an applied think tank focusing on sustainability and cities through applied research, teaching, and community partnerships. We work across disciplines that match the complexity of cities to address sustainability challenges, from regional planning to building design and from enhancing engagement of diverse communities to understanding the impacts on municipal budgets from disruptive technologies and many issues in between.

SCI focuses on sustainability-based research and teaching opportunities through two primary efforts:

1. Our Sustainable City Year Program (SCYP), a massively scaled university-community partnership program that matches the resources of the University with one Oregon community each year to help advance that community's sustainability goals; and

2. Our Urbanism Next Center, which focuses on how autonomous vehicles, e-commerce, and the sharing economy will impact the form and function of cities.

In all cases, we share our expertise and experiences with scholars, policymakers, community leaders, and project partners. We further extend our impact via an annual Expert-in-Residence Program, SCI China visiting scholars program, study abroad course on redesigning cities for people on bicycle, and through our co-leadership of the Educational Partnerships for Innovation in Communities Network (EPIC-N), which is transferring SCYP to universities and communities across the globe. Our work connects student passion, faculty experience, and community needs to produce innovative, tangible solutions for the creation of a sustainable society.

About SCYP

The Sustainable City Year Program (SCYP) is a year-long partnership between SCI and a partner in Oregon, in which students and faculty in courses from across the university collaborate with a public entity on sustainability and livability projects. SCYP faculty and students work in collaboration with staff from the partner agency through a variety of studio projects and service-

learning courses to provide students with real-world projects to investigate. Students bring energy, enthusiasm, and innovative approaches to difficult, persistent problems. SCYP's primary value derives from collaborations that result in on-the-ground impact and expanded conversations for a community ready to transition to a more sustainable and livable future.

About City of Troutdale

Troutdale is a dynamic suburban community in Multnomah County, situated on the eastern edge of the Portland metropolitan region and the western edge of the Columbia River Gorge. Settled in the late 1800s and incorporated in 1907, this “Gateway to the Gorge” is approximately six square miles in size with a population of nearly 17,000 residents. Almost 75% of that population is aged 18-64.

Troutdale’s median household income of \$72,188 exceeds the State of Oregon’s \$59,393. Troutdale’s neighbors include Wood Village and Fairview to the west, Gresham to the south, and unincorporated areas of Multnomah County to the east.

For the first part of the 20th century, the city remained a small village serving area farmers and company workers at nearby industrial facilities. Starting around 1970, Troutdale became a bedroom community in the region, with subdivisions and spurts of multi-family residential housing occurring. In the 1990s, efforts were made to improve the aesthetics of the community’s original core, contributing to an award-winning “Main Street” infill project that helped with placemaking. In the 2010s, the City positioned itself as a jobs center as it worked with stakeholders to transform a large superfund area to one of the region’s most attractive industrial centers – the Troutdale-Reynolds Industrial Park.

The principal transportation link between Troutdale and Portland is Interstate 84. The Union Pacific Railroad main line runs just north of Troutdale’s city center. The Troutdale area is the gateway to the famous Columbia River Gorge Scenic Area and Sandy River recreational areas, and its outdoor pursuits. Troutdale’s appealing and

beautiful natural setting, miles of trails, and parkland and conservation areas draw residents and visitors alike. The City’s pride in place is manifested through its monthly gatherings and annual events, ranging from “First Friday” art walks to the city’s long-standing Summerfest celebration each July. A dedicated art scene and an exciting culinary mix have made Troutdale an enviable destination and underscore the community’s quality of life. Troutdale is home to McMenamins Edgefield, one of Portland’s beloved venues for entertainment and hospitality.

In recent years, Troutdale has developed a robust economic development program. The City’s largest employers are Amazon and FedEx Ground, although the City also has numerous local and regional businesses that highlight unique assets within the area. Troutdale’s recent business-related efforts have focused on the City’s Town Center, where 12 “opportunity sites” have been identified for infill development that respects the small-town feel while offering support to the existing retail environment. The next 20 years promise to be an exciting time for a mature community to protect what’s loved and expand opportunities that contribute to Troutdale’s pride in place.

Course Participants

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Design Report (Draft III)

EXECUTIVE SUMMARY

Troutdale's Town Center District was established in the 19th century by pioneering families and is considered the cultural heart of their community. The Town Center District is 270 acres located south of Interstate I-84, and west of the Sandy River. The City of Troutdale has a robust Capital Improvement Plan with goals for the city's growth, supportable employment, civic-use spaces, and future economic development of the empty Confluence site behind the Columbia Gorge Outlets Shopping Center.

Currently, there is no safe or direct route for pedestrian traffic from Historic Downtown Troutdale to the Confluence site as an active main-line Union Pacific Railway creates a barrier between these two areas. The purpose of this project is to connect the Downtown area to the newly developing Confluence site using a pedestrian bridge. This proposed design would be inclusive to pedestrians, cyclists, and neighborhood electric vehicles (golf carts). The scope of this project is a 30% initial design along with costing estimates for design, permitting, and construction that could take place in the next five years.

The foundation design was performed in accordance with geotechnical standards of practice using resources available from previous nearby geotechnical investigations, United States Geological Survey (USGS) data, as well data gathered on-site. In all cases, conservative design values are used in calculations and design as described herein. At this time, any design recommendations should be considered preliminary, and further geotechnical exploration and data collection should be done to verify the design recommendations of this report.

A shallow foundation analysis was performed. A range of allowable vertical loads was determined for a variety of footing geometries, and the most reasonable geometries were selected and recommended herein as potential footing geometries for the loads calculated in the preliminary analysis. Preliminary CAD drawings and typical detail are provided for these shallow foundation geometries in appendix C. In general, the loads determined require relatively large shallow footing supports. Due to the space limitations of the project, it is likely that deep foundations will be preferred as they will take less space and perhaps be less costly. cursory deep foundation analysis has been performed using SHAFT software, and those calculations and preliminary design recommendations are provided in section 3.4.1 and appendix C and D.

Design Report (Draft III)

For the structural design, the truss of the bridge is made out of W18x86 I-beams, except diagonal members that are made of W18x158. The horizontal perpendicular and diagonal members are made of W12x53 and W12x87 respectively since they are only for the lateral stiffness of the bridge. This means that they have a small axial loading where similar ones are used in the Lafayette street pedestrian bridge. The loading conditions and calculations followed the AASHTO LRFD 2012 Bridge Design Specifications 6th Ed (US) code, as shown in section 3.4.2 and the appendix.

The columns of the bridge are constructed of a cast-in-place reinforced concrete with rectangular cross-section 3 ft x 6 ft, concrete compressive strength of 4000 psi, and grade 60 reinforcing steel. The longitudinal reinforcement is provided by 26-#9 bar and the transverse reinforcement is provided by overlapping closed-loop #3 ties, following the American Association of State Highway and Transportation Officials (AASHTO) Bridge specifications.

For the slab design, corrugated steel was used as a reinforcement with a two-inch concrete cover. U.S. BRIDGE provided the tests for the designed loading using different spans. Through two load combinations, the max moment of the floor beams was found and used to determine adequate members. A W10x17 section was selected and placed at a 4.33 feet spacing and at a 3-floor beam per panel.

Design Report (Draft III)

The following report and subsequent sections will cover the background of the area and the need for a pedestrian bridge as well as alternative designs, proposed design along, cost analysis, construction schedule, and permitting prepared by Portland State University CEE Capstone Class.

1.0 PROJECT BACKGROUND

The City of Troutdale is located south of Interstate I-84, approximately 12 miles east of Portland, OR. It is a growing city with a population of about 17,000 people. The Downtown district and Columbia Gorge Outlets Shopping Center are disconnected due to elevation and a Union Pacific Railroad (UPRR) railway (Figure 1.1). This project aims to provide connectivity and access between the two districts by beginning the design of a pedestrian and light vehicle bridge.



Figure 1.1: Troutdale Town Center District (City of Troutdale, 2020)

Due to natural growth, downtown Troutdale has a low population density. The City of Troutdale is expanding but remains disconnected from surrounding areas. To stabilize the population and bring more people into this area, there is a need for more development. The largest developable area is the Confluence site and is located inside the City’s urban renewal area (URA). The URA includes Columbia Gorge Outlets, The Confluence site, and Depot Park. It is located north of an active, main-line Union Pacific railroad, south of I-84, east of Graham Rd. and west of the Sandy River. Figure 1.2 shows the URA adjacent to downtown, also called the central business district (CBD) of the City of Troutdale. The development of the URA will play a critical role in the long-term success of the City of Troutdale. Currently, it is separated from downtown Troutdale by an active, main-line Union Pacific railroad right-of-way. The railroad makes the connections between the two areas involve indirect routes, which are unsafe and inconvenient for pedestrian and bike traffic. A multipurpose bridge would directly connect the CBD of downtown Troutdale to the

Confluence site of the URA (Figure 1.3). Therefore, the proposed bridge would directly link the existing infrastructure of the downtown CBD to the future development in the URA.

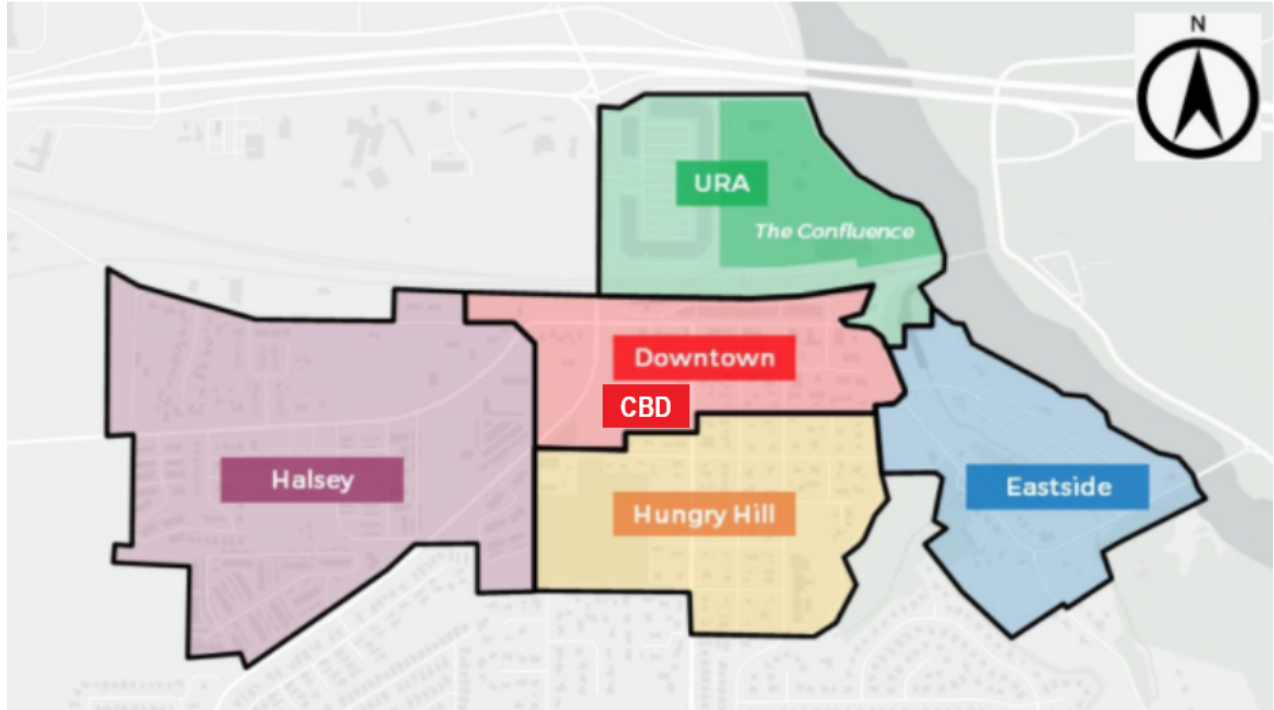


Figure 1.2: Neighborhoods within the Town Center District (City of Troutdale, 2020)



Figure 1.3: Proposed Location of the Pedestrian Bridge (City of Troutdale, 2020)

The Pedestrian bridge will be located north of the East Historic Columbia River Highway, Troutdale, OR, in between the 200 and 300 blocks. The bridge crosses the active, main-line Union Pacific railroad. The yellow dashed lines outline the URA and the location of the bridge, the yellow shading covers the area of the confluence site, and black hash marks were used to outline the location of the Union Pacific Railroad tracks traveling east and west (see Figure 1.3).

1.1 EXISTING SITE CONDITION

In this section, the existing site conditions and structures for the CBD-URA pedestrian bridge location are outlined.

The City of Troutdale owns The Confluence site within the URA. The water tank located in the middle of The Confluence site belongs to the City of Troutdale. The proposed bridge location will pass over an active, main-line Union Pacific railroad South of the URA. The northern border of the URA is shared with the State of Oregon which owns the I-84 property. The southern and western borders are shared with CBD (privately owned properties), and the eastern border is defined by the Sandy River.

1.1.1 Geology

Per the Department of Geology and Mineral Industries (DOGAMI) and the United States Geological Survey (USGS) mapping, the geology of the site is identified as Quaternary (about 10,000 years) alluvial and colluvial deposits caused by the Missoula and Bonneville flood events. This deposit is known as the “Missoula Flood Deposit” (Figure 1.1.1.1).

According to USGS, Missoula Flood Deposits are described as “Deposits of unconsolidated sediments. Includes alluvium, colluvium, river and coastal terrace, landslide, glacial, eolian, beach, lacustrine, playa, and pluvial lake deposits, and outburst flood deposits left by the Missoula and Bonneville floods.”

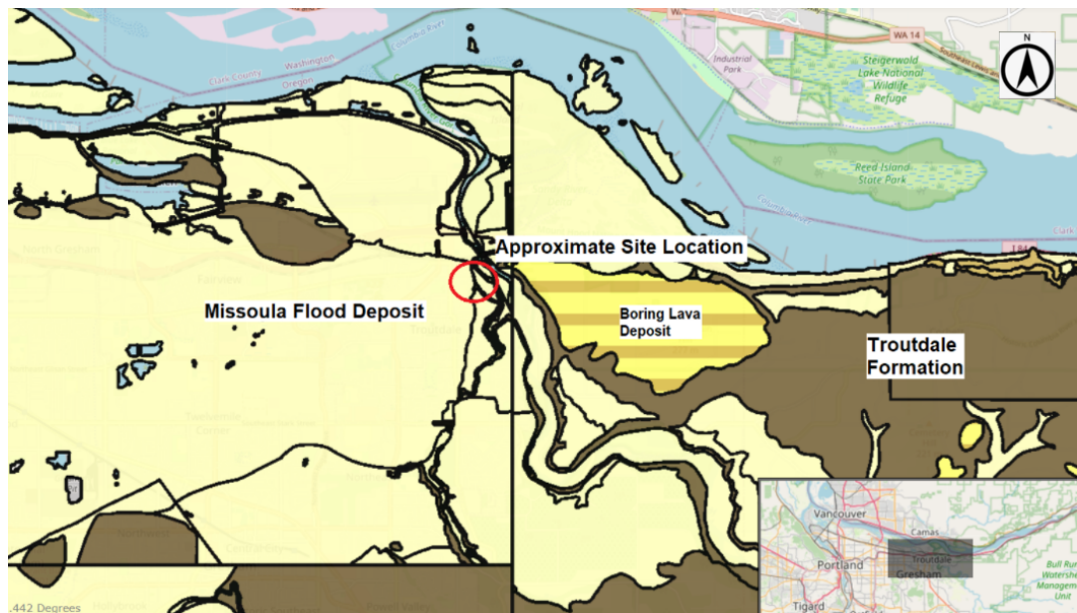


Figure 1.1.1.1: Geologic Map for the City of Troutdale (DOGAMI, 2021)

The thickness of this strata usually extends more than 100 feet. These deposits are expected to be underlain by Boring Lava Flows, which were the dominant geologic formation in the area before the flooding events. The Missoula Flood strata is likely overlain by topsoil in many areas, consisting of a nonplastic, noncohesive, organic silt.

Some surficial geotechnical evaluation, including shallow dynamic cone penetrometer (DCP) tests and hand auger borings, would give valuable data about the project’s specific shallow depth geologic conditions. Deeper Standard Penetrometer Testing (SPT) would give insight on geologic conditions deeper into the strata.

1.1.2 Topography

The topography of the project varies widely from south to north. The southern portion is mainly flat and covered in hardscape and landscape that supports the existing business structures and pedestrian traffic associated with the commercial district. Moving to the North, a concrete stairway and landscaping connect the frontages on Historic Columbia River Highway to the lower back elevation and parking. In this area, the topology is relatively flat. North of the railroad tracks, the grade drops steeply again (Figure 1.1.1.2).

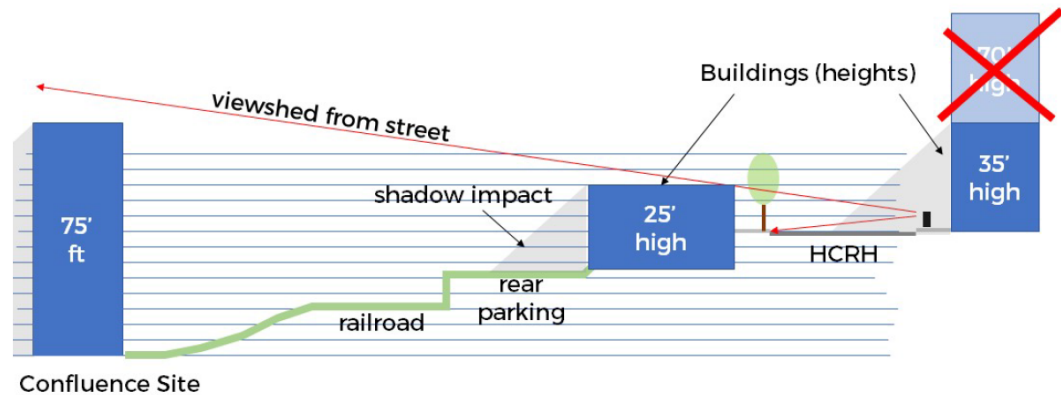


Figure 1.1.1.2: Preliminary Cross-Sectional Sketch of Project Area (City of Troutdale, 2020)



Figure 1.1.1.3: Actual View Point of Proposed Bridge Site. Taken from the Confluence Site Looking South Towards Downtown (Oleson, Patricia. 2021)

1.1.3 Climate

Troutdale has dry and warm summers with powerful winds in the Pacific North-West. Strong eastern winds from the Columbia River Gorge affect the city's temperature as well as its climate. Annually there are about 58 inches of rain, 4 inches of snow, and 145 sunny days (Best Places 2021).

1.1.4 Current Usage

Currently, the Confluence site is empty, except for a temporary disc golf course and a water tower that the City of Troutdale owns. According to Troutdale's 2020-2040 Town Center Plan, the water tower is for emergency fire usage. It should be retained as an iconic feature of the site and future development.

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1.2 STAKEHOLDERS

In this section, the stakeholders for the CBD-URA pedestrian bridge project are outlined.

- Town Center Committee
- Other government agencies in the region, particularly Multnomah County.
- Community and interest-based groups, ranging from visual artists to avid cyclists
- Adjacent property owners that will be affected by the construction of the bridge, and will have their property imposed on by the bridge's presence.
- Pedestrians because they can use this bridge as a link between the downtown CBD city of Troutdale and the URA
- City workers operating electric vehicles will be able to use this bridge as a link between the downtown CBD and future developments in the URA

Chris Damgen, Community Development Director, and Amber Shackelford, Assistant Planner, will use our calculations and ideas when working on the pedestrian bridge design, which will make this process more efficient. This project will serve pedestrians, bikes, small electric vehicles such as golf carts, and emergency vehicles of the designated future build locations.

2.0 ALTERNATIVES ANALYSIS

In this section, the design considerations of the project will be analyzed. Four different designs will be reviewed and scored based on the discussed criteria.

2.1 ALTERNATIVES CONSIDERED

Four alternative bridge types are considered for this project: a concrete box-girder, a steel truss, a timber truss, and a no-build option. This section will explain how each material can be used and the advantages and disadvantages in using the material based on the specified criteria discussed in section 2.2, The Alternatives Selection Criteria.

Option 1: Concrete Box-Girder (Poured-in-Place) Design



Figure 2.1.1: Concrete Box-Girder System (Concrete Construction, 2019)

A concrete box girder bridge is made up of hollow rectangular or trapezoidal girders that span the length of the bridge. Concrete is poured into formwork on-site. Any shape can be formed that is desired for the concrete structure. The costs are estimated to be between \$800 and \$1300 per linear foot. Since excavation is done before concrete pouring, this eliminates potential vibrational damage when a pile is driven into the ground. While any large project may have construction delays, one specific time estimation to poured-in-place concrete is cure time related to weather conditions. Generally, pour-in-place concrete bridges are more economical as long as the formwork does not interfere with traffic (Barker & Puckett, 2013).

Option 2: Steel Truss Design

Figure 2.1.2: Steel Truss Pedestrian Bridge (Architecture & Design, 2008)

In a steel truss bridge (Figure 2.1.2), the superstructure is composed of interconnecting steel triangles. Truss designs include through trusses that contain lateral support or pony trusses that do not contain lateral support. Only upper truss designs will be considered to limit possible interference with the railroad.

Truss designs have effective weight distribution and load-bearing capacity across a short span. This will allow for an economically equivalent design to have a significantly higher capacity, allowing the bridge to be effective for current and future demands (Barker & Puckett, 2013). The use of welded and bolted connections will be considered for ease of construction and effectiveness of connection. Truss design is expected to have lower material costs, and it will create no additional environmental impacts compared to other steel designs. The major advantage to a steel truss is the aesthetically pleasing look that can be obtained.

Option 3: Timber Truss

Figure 2.1.3: Timber Truss Design Pedestrian Bridge (Brampton Woodworks, 2017)

For this option, timber will be the principal material used for the bridge. Comparing Figures 2.1.2 to 2.1.3 shows that a wood truss configuration is similar to a steel truss design. As a structural material, timber has some benefits and drawbacks. Wood is considered one of the most efficient materials as it is low cost, sourced locally, and a faster build time, and can be used to build year-round in almost any climate. (Think Wood, 2021). Additional benefits include the following: easily manufactured and produced, easily modified as the need grows or changes, environmentally friendly (Bergman, 2011), can reduce carbon footprint, and timber bears greater loads per unit weight than concrete or steel (Carmichael, 2018)

Some of its drawbacks are low durability if it is not adequately protected against weather, insects, and significant damage in a fire. Additionally, timber has low shock resistance (for example, due to a possible vehicle collision) and requires yearly maintenance that can be costly (Mt Copeland Technologies, 2021).

Option 4: No Build Option

Figure 2.1.4: Existing Site Conditions, Taken from the Historic Columbia River Highway Looking North Towards the Confluence Site (Kozyaev, Evgeny. 2021)

No bridge is constructed in the proposed area. This option is the most economical where the cost to design and build the bridge is saved toward other essential projects and improvements within the city of Troutdale. One good reason for the bridge not to be constructed is the lack of pedestrian crossing demand. However, some drawbacks of this option are the following: there will be no connection between the central business district (CBD) of downtown Troutdale and the Confluence site. It would reduce a potentially positive impact on the economy since people do not have safe and quick access to downtown.

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2.2 ALTERNATIVE SELECTION CRITERIA

The following is a ranking of importance for all criteria relative to each other. Each criterion is defined and evaluated as an independent value and assigned a score in line with the outlined rubric. The weights assigned to each criterion were evaluated in relation to all other criteria in total effect on the project.

Cost - Cost is defined as the cumulative cost per linear foot for constructing the bridge alternative considered. The estimates within Table 2.2.1 have been calculated using loose figures readily available online. These values may not reflect actual costs and are being used to compare the alternatives, not for cost estimation or proposing purposes. Cost-efficient alternatives will score higher than more expensive or less efficient options. This criterion received a weighting of 8, the greatest of all categories, due to limitations on the available budget and an emphasis on efficient resource allocation.

Table 2.2.1: Cost Rubric

Cost:	Score:
The cumulative construction cost is below \$800 per linear foot of bridge.	3
The cumulative construction cost of the project remains between \$800 and \$1,300 per linear foot of material.	2
Cumulative construction costs exceed \$1,300 per linear foot of material.	1

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Maintenance - Maintenance is defined as the maintenance cost per square foot of bridge decking. Other factors have also been considered, such as the interval at which maintenance could be expected. Also included in the maintenance criterion is the requirement that the City of Troutdale can subcontract local agencies to service the bridge as they do with other bridges in the city. As shown in Table 2.2.2, alternatives that require less frequent or less expensive upkeep will score higher than those that are more expensive. Bridges that will be difficult to subcontract out have been omitted from the list of alternatives. Criteria directly correlated with project cost are prioritized in matrix analysis; this criterion received a weighting of 6 as a result.

Table 2.2.2: Maintenance Rubric

Maintenance:	Score:
Maintenance costs are estimated between \$80/sq.ft and \$95/sq.ft; based on material composition, ease of access, and occurrence interval.	3
Maintenance costs are estimated between \$95/sq.ft and \$110/sq.ft; based on material composition, ease of access, and occurrence interval.	2
Maintenance costs exceed \$110/sq.ft; based on material composition, ease of access, and occurrence interval.	1

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Construction Schedule - The construction schedule criterion is defined as the period of active bridge construction. Construction time estimates readily available via online research have been used to evaluate each of the alternatives. These values are estimates and are used for the comparison of the alternatives only. Bridges with low active construction time limit the disruption to the community and therefore score higher than alternatives that take longer to construct, as shown in Table 2.2.3. Due to low variability between the considered alternatives and an emphasis on cost-efficient design, this criterion received a weighting of 2.

Table 2.2.3: Construction Schedule Rubric

Construction Schedule:	Score:
The bridge is functional after less than 2 years of active construction.	3
The bridge is functional within 2-4 years of construction onset.	2
Project completion requires 4 or more years of active construction.	1

Lifespan - The lifespan criterion is defined as the time before structural decay renders the structure unsafe to use. These values are not intended as design recommendations and are only being used to evaluate the alternatives comparatively. Furthermore, this criterion does not consider increasing demand on bridge design, only the structure’s longevity. This criterion received a weighting of 3.

Table 2.2.4: Lifespan Rubric

Lifespan:	Score:
The structure is expected to remain functional for at least 80 years.	3
The structure is expected to remain functional for 50 to 80 years.	2
The structure is expected to remain functional for 20 to 50 years.	1
The structure is expected to remain functional for less than 20 years, despite routine maintenance.	0

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Aesthetics - Aesthetics ranking is a value assigned to the appearance of the design alternatives. This value is based on the surrounding design of the community, and the preferred stylistic branding of the City of Troutdale. Some consideration has been given to the flexibility of each alternative’s appearance and the visual appeal of the materials. Aesthetic appeal is an important concern for structures located in densely populated environments. This criterion received a weighting of 4 as a result.

Table 2.2.5: Aesthetics Rubric

Aesthetics:	Score:
The bridge has a positive impact on the aesthetic of Troutdale’s Town Center District. No complications due to color, texture, ornamentation, barrier detail, or superstructure shape are observed.	3
Minor complications due to color, texture, ornamentation, barrier detail, or superstructure shape affect the aesthetic rating of the bridge.	2
Complications due to color, texture, ornamentation, barrier detail, or superstructure shape have a significant impact on the aesthetic rating of the bridge.	1
The bridge does not enrich the local aesthetic due to severe color, texture, ornamentation, barrier detail, or superstructure shape complications.	0

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Environmental Impact - The environmental impact criterion accounts for the total impact of design, construction, and the bridge’s existence on the environment. The relevance of this criterion can vary based on the practices of the construction teams involved. The City of Troutdale emphasizes sustainability through design. Alternatives with the potential to damage the local environment are not considered. This criterion received light-weighting, as environmentally problematic alternatives are already eliminated.

Table 2.2.6: Environmental Rubric

Environmental Impact:	Score:
Constructing the bridge poses no risk to the local environment.	3
Mitigating environmental hazards leads to a minor increase in cost or assembly time, but construction causes no significant impact on the environment.	2
Mitigating environmental hazards greatly increases either cost or assembly time, but construction causes no significant impact on the environment.	1
Constructing the bridge may have a negative impact on the local environment, including the Columbia River.	0

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2.3 ALTERNATIVE SCORING

The following Pugh Matrix contains the scoring for all proposed alternatives. Each criterion was assigned a score with a maximum of 3. The total score is a product of the row scores multiplied by the weight of their respective criterion category and divided by 3.

Table 2.3.1: Pugh Matrix

ALTERNATIVE	CRITERIA						
	COST	MAINTENANCE	SCHEDULE	LIFESPAN	AESTHETICS	ENVIRONMENT	TOTAL
1) Concrete Box-Girder	2	3	2	3	2	2	19.7
2) Steel Truss Design	3	2	3	3	3	2	22.3
3) Timber Design	3	1	3	1	3	3	19.0
4) No-Build	3	3	3	0	0	3	18.0
WEIGHT	8	6	2	3	4	2	25

The decision process was carefully studied for each of the alternatives. Starting with the No-Build option, which earned the lowest score of 18.0 out of 25.0, this alternative will require no cost, no maintenance cost, no time spent on a project, and it would not cause changes to the environment. However, this option will add zero aesthetics to the community, and there will be no lifespan of a structure connecting the central business district to the Confluence site.

The Timber Truss alternative has two significant drawbacks: maintenance and lifespan. It scored 19.0 out of 25.0. Timber has a low lifespan expectancy compared to the other design options. It also requires consistent care and maintenance that will add to the cost of the project. Other than that, it stands out as the most environmentally friendly bridge design option. The bridge can be fully functional in less construction time than concrete box-girder options.

The Concrete Box-Girder score was the second-best option, earning a score of 19.7. This option is the most cost-intensive out of the three bridge design alternatives, and it requires the most construction time to obtain a finished functional bridge. It stands out as the least maintenance requiring bridge design option, and that will reduce the cost contributed to the maintenance of the bridge.

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The best scoring alternative is the Steel Truss Design, earning 22.3 out of 25. It stands out as the most cost-efficient design option, and it is also the option with the most extended lifespan. The bridge can be fully functional in less construction time than concrete box-girder options. In terms of environmental impact, it scored lower than concrete and timber design.

The leading pedestrian bridge alternative is the steel truss design, with a final score of 22.3. In addition to receiving high scores, this alternative is efficient due to effective weight distribution and load-bearing capacity across short spans. Compared to relevant timber or box-girder designs, steel truss design options also provided more flexibility to fit future needs.

3.0 FACILITY DESIGN

The CBD-URA Bridge is intended to be used as a connection from Troutdale's Central Business District to the Urban Renewal Area. The proposed bridge design will allow pedestrians, bicycles, and neighborhood electric vehicles to safely cross over the railroad that currently divides the two areas.

3.1 DESIGN CRITERIA

The steel truss was the preferred alternative based on the Pugh matrix in Table 2.3.1. The overall structure selected was the Warren truss because the repetitive geometry was aesthetically pleasing, it matched the existing bridge near the site being used by the Union Pacific Railway, and this style of the truss has the ability to spread loads more evenly across a number of different members. Locally, the Lafayette Pedestrian Bridge has the same style truss system that was researched and visited by team members for visualization and comparison purposes. The AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges was used to have a more conservative design that exceeds the required strength needed for the structure. Detailed drawings that support the facility design can be found in Appendix C.



Figure 3.1.1: The Lafayette Pedestrian Bridge, Portland OR (KPF Consulting Engineers, 2021)

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3.2 GEOMETRY

The main geometry of our existing site was provided by our client in the form of LIDAR data, GIS layers, a County design manual, City CIP, and SketchUp modeling. Additionally, both the Historic Downtown area and the Confluence site were visited by members of the team to do geological testing, take pictures, and gain a physical visual perspective of the site. The LIDAR data was used together with AutoCAD Civil3D to determine bridge length, pile locations, bridge height, and clearances.

The current length of the bridge is 239 feet and consists of three spans, the main steel truss and two voided slabs. Further details on dimensions and spans of the bridge can be found in section 3.4.2.1 through 3.4.2.4 of this report. The width of the bridge is 14 feet to accommodate clearances for two way traffic of bicycles and golf carts. Elevation diagrams provided by the client allowed the geotech team to calculate the appropriate graduation for the first four bridge piles. The exact location of the bridge as well as the section that will extend after the fourth pile north of the railway are still being studied by our client, therefore the proposed length and width of the bridge are estimates and will need to be reevaluated or updated as new information is gathered.

3.3 CODES AND STANDARDS

This section describes the codes and standards that are relevant to and control this project. The governing code of this project is the AASHTO Pedestrian Bridge Design Guide. This guide references other manuals such as the AASHTO Highway Bridge Design Manual. The ODOT Bridge Design and Drafting Manual also provides design guidance that overrides AASHTO at times.

3.3.1 Geotech Codes/Standards

The standards from The Engineering of Foundations textbook (Salgado 2008) was used to calculate deep and shallow foundation, and to approximate the values of the SPT Blow Count Correction. The correlations between SPT blow counts and peak friction angle are analysed using the De Mello (1971) method, the correlations for relative density came from Idriss and Boulanger (2003), and the SPT correlations for Clays was from Stroud (1975), all modified by the Salgado textbook.

3.3.2 Transportation Codes/Standards

This section will contain AASHTO, ADA, ODOT, and other codes that relate directly to the way pedestrians, bicycles, and NEVs interact on the bridge. These codes describe sidewalk requirements, lane widths, camber, etc.

3.3.2.1 American Association of State Highway and Transportation Officials (AASHTO)

The American Association of State Highway and Transportation Officials has set forth AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges which references the AASHTO LRFD Bridge Design Specifications. These standards are to guide geometry of facility design.

3.3.2.2 Americans with Disabilities Act (ADA)

The Americans with Disabilities Act sets forth design guides relevant to geometry of the proposed facility design. All design guides are detailed in the *ADA Standards for Accessible Design* and referenced in all AASHTO design guides.

3.3.2.3 Oregon Department of Transportation (ODOT)

The Oregon Department of Transportation has set forth the *ODOT Bridge Design and Drafting Manual* to provide guidance in bridge design and drafting. These standards are used for aspects of the design and drafting process that the City of Troutdale does not cover. The relevant regulatory provision addressed by ODOT is the *Oregon Bridge Design Manual*.

3.3.3 Structural Codes/Standards

This section will contain specifications by AASHTO, ASCE, and other relevant structural design groups that guide this project's design. These codes will describe the design structural elements such as rebar reinforcement, steel member capacity, etc.

AASHTO 6.8 Tension Members, 6.9 Compression Members, 6.10 I-Section Flexural Members

The American Association of State Highway and Transportation Officials has set forth AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges which references the AASHTO LRFD Bridge Design Specifications. This design code provided all the requirements for the structural design process for the bridge for the member selection for the truss.

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ASCE 7-10.2.3 Combining Factored Load Using Strength Design

The American Society of Civil Engineers (ASCE) design code provided the equations for the seismic as well as gravity load combinations for loading conditions that were used for analysis of bridge members.

3.6.1.6 - Pedestrian Loads

Specifications for pedestrian loading on bridges. Bridges intended for only pedestrian, equestrian, light maintenance vehicle, and/or bicycle traffic should be designed in accordance with AASHTO's *LFRD Guide Specifications for the Design of Pedestrian Bridges*.

6.12.2.2.1 - Noncomposite Members

Nominal flexural resistance for I- and H-shaped members. The provisions of this Article apply to I- and H-shaped members consisting of two channel flanges connected by a web plate.

3.4 CALCULATIONS

This section will include calculations for all aspects of the project, including design, material costs, construction costs, and other costs associated with building a pedestrian bridge.

3.4.1 Geotechnical Design

This section details the geotechnical design considerations of the proposed design of the structure.

3.4.1.1 Existing Geotechnical Conditions

To perform initial analysis of the foundation dimensions required to support the expected loads from the bridge, several soil properties were assumed based on a variety of available information. GRI, a local geotechnical engineering firm, performed a geotechnical investigation for a site approximately 1500 feet northwest of the CBD-URA bridge location. This report is titled "Geotechnical Investigation of Sandy Riverfront Park" with a draft date of March 22, 2021. This report forms the basis for many of the estimated soil properties used for these geotechnical calculations and is provided in the appendix.

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In addition to the GRI report, CBD-URA team members visited the site to perform a hand auger boring log and sample collection near the location of the northernmost bridge footing. The hand auger location and boring log are attached in the appendix, and soil descriptions and USCS classifications are summarized as follows:

Fractured Aggregated (GM) - At the location of HA-1, from ground surface to about 1.5 feet below ground surface (bgs) material consisted of a fractured aggregate. The aggregate was a mix of recycled fractured concrete and fractured basalt rock. This section is non-native, and has been installed and used for vehicle access.

Sandy Silt (ML) - From 1.5 to about 7.5 feet bgs the material consisted of a light brown, slightly micaceous sandy silt material. This material is a quaternary deposit of alluvial sediments likely deposited during flooding of the Columbia and the Sandy Rivers. This material was moist at the time of boring, and was medium stiff in situ.

Fine-Grained Sand (SM) - From 7.5 to 10 feet bgs, the strata becomes more sandy and less cohesive. At the time of boring, there was also substantially more moisture in this material than in shallower strata. This material is likely a transitional material from the alluvial deposits to the Troutdale formation which underlays this layer.

Well-Graded Sand (SW) - From 10 feet bgs to the termination of the boring at about 12 feet bgs, the material changed to a brownish-gray well graded sand with rounded cobbles. This material was entirely saturated at the time of boring and standing water was present. This material is known as the “Troutdale Formation” as discussed above in section 1.1.1.

Assumed standard penetration test (SPT) blow count values are assumed using a combination of the blow counts in similar materials from the GRI report. The assumed blow counts are then corrected to N60 values and averaged (Figure 3.4.1.1.1). For complete estimated values and calculations see Appendix D.2.

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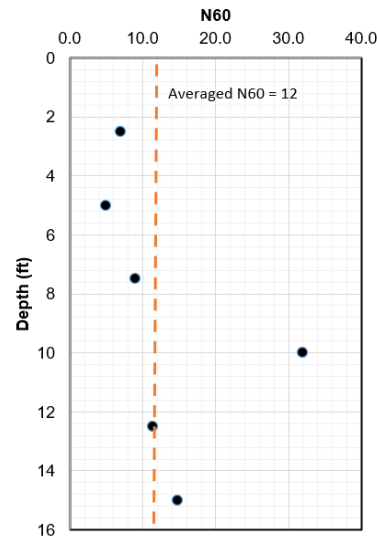


Figure 3.2.1.1.1 - N60 Values With Depth

Other values that have been assumed for the purpose of design have been estimated and are “typical values” assumed based on the soil types identified on site. The average unit weight of the material was assumed to be 120 pounds per cubic foot (pcf). Peak effective friction angle based on the N60 values was assumed to be 28 degrees. Also, for the purposes of the design, the primary soil behavior is assumed to be ideal sand. This means the material is free draining, and entirely non-cohesive. These estimates are intentionally conservative as soil conditions can be highly variable and no data is available for soils at the foot locations. Specific geotechnical analysis can be done to provide more confident design parameters.

3.4.1.2 Shallow Foundation Analysis

Shallow foundations are a method of transferring a load from a structure to the ground by way of spreading that load over an area and distributing it somewhat evenly into the earth. Shallow foundations are typically constructed out of reinforced concrete. The sizing of shallow foundations has been performed in accordance with typical industry standards. At this stage of design, only vertical loads have been analyzed with a factor of safety of 3. In the future, horizontal loading and moment loads should be considered and analyzed.

For the purpose of this analysis, it was assumed that the footings will behave as spread footings as they will likely be rectangular in geometry. Analysis was performed in accordance with Terzaghi’s method as published in “The Engineering of Foundations” by Rodrigo Salgado. A factor of safety of 3 has been used in accordance with industry standards of practice. Other soil properties have been

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assumed as stated above. Figure 3.4.1.2.1 shows allowable loads per footing geometries.

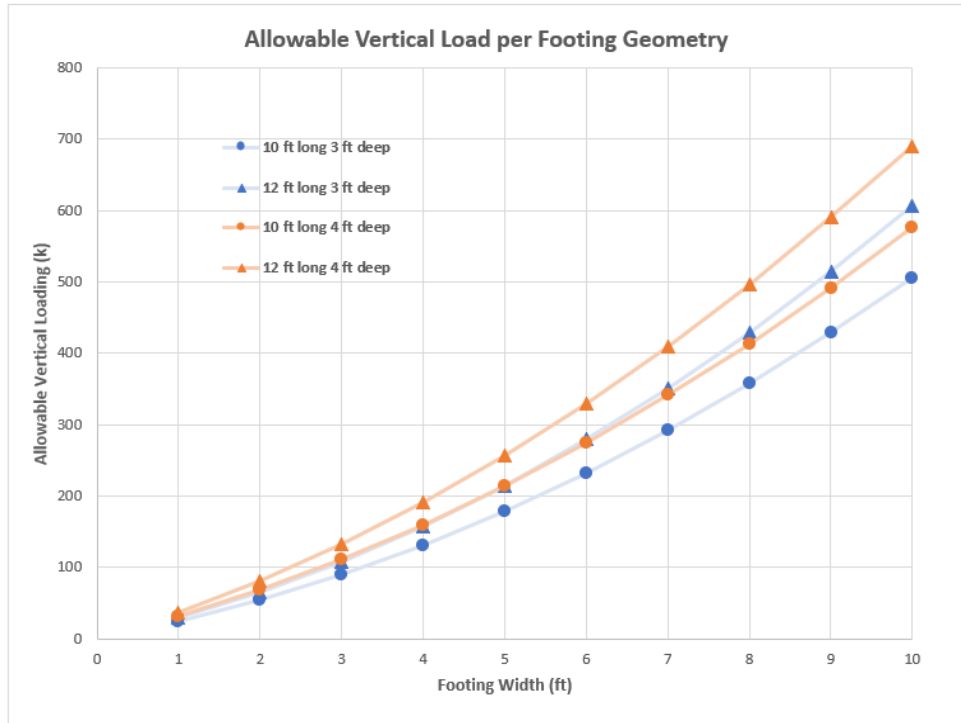


Figure 3.4.1.2.1 - Graph of Allowable Loads per Terzaghi’s Method.

A plot of this nature can be used to appropriately size footing geometry as needed. Should loading conditions change somewhat during the course of design, the footing dimensions can also change to accommodate. At this point in the design, there are four footings with varying expected vertical loads. Four recommended footing geometries have been provided based on these calculations (Table 3.4.1.2.2):

Table 3.4.1.2 - Preliminary footing geometry design recommendations.

Support Designs	Footing Geometry (Length by Depth by Width)	Pallow > Load
DF1	12x3x6	256 > 221
DF2	12x3x10	556 > 506
DF3	12x4x10	633 > 611
DF4	12x4x9	471 > 427

As stated, these geometries are subject to change and are likely to be reduced in size as more precise geotechnical data becomes available through site specific investigations and testing. However, these geometries are quite large considering the space limitations on site. Deep foundations could provide a cost effective and space saving solution for bridge support.

3.4.1.3 Earthquake Design - Liquefaction Analysis

A liquefaction analysis was conducted using standard penetration test parameters obtained from boring B-3 of the attached GRI report. Boring B-3 was selected to represent onsite conditions, as B-3 layer classifications resembled those observed through hand auger investigation, and B-3 was located nearest to the site. Of the parameters required to perform a liquefaction analysis, layer classification, sample depth, water table depth, sample blow count, measured fines content, borehole diameter, and rod length are directly available within the GRI report.

To complete a preliminary liquefaction analysis for 30% design, common soil density values of 18 kN/m³ below the water table and 20 kN/m³ above the water table are assumed. Peak ground acceleration was estimated at 0.24g based on a seismic design contour map for the state of Oregon (ODOT, 2004). The design earthquake magnitude was tabulated as 7.5 for the Cascadia Subduction Zone.

Two examinations were conducted - one assuming the water table was located 1.5m below ground surface (as observed for boring B-3), and another analysis for a water table at ground level. The flood conditions in Analysis 2 are assumed for conservative design as well as potential seasonal variability near the Columbia River.

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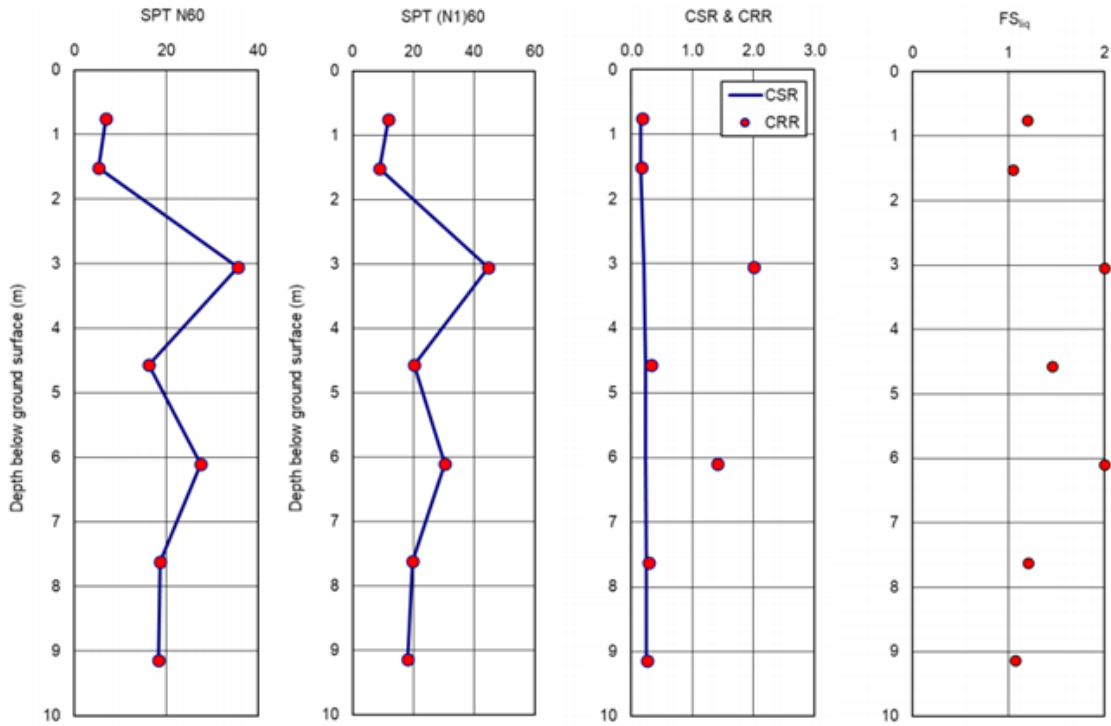


Figure 3.4.1.4.1: Analysis 1 - Water Table 1.5m Below Surface

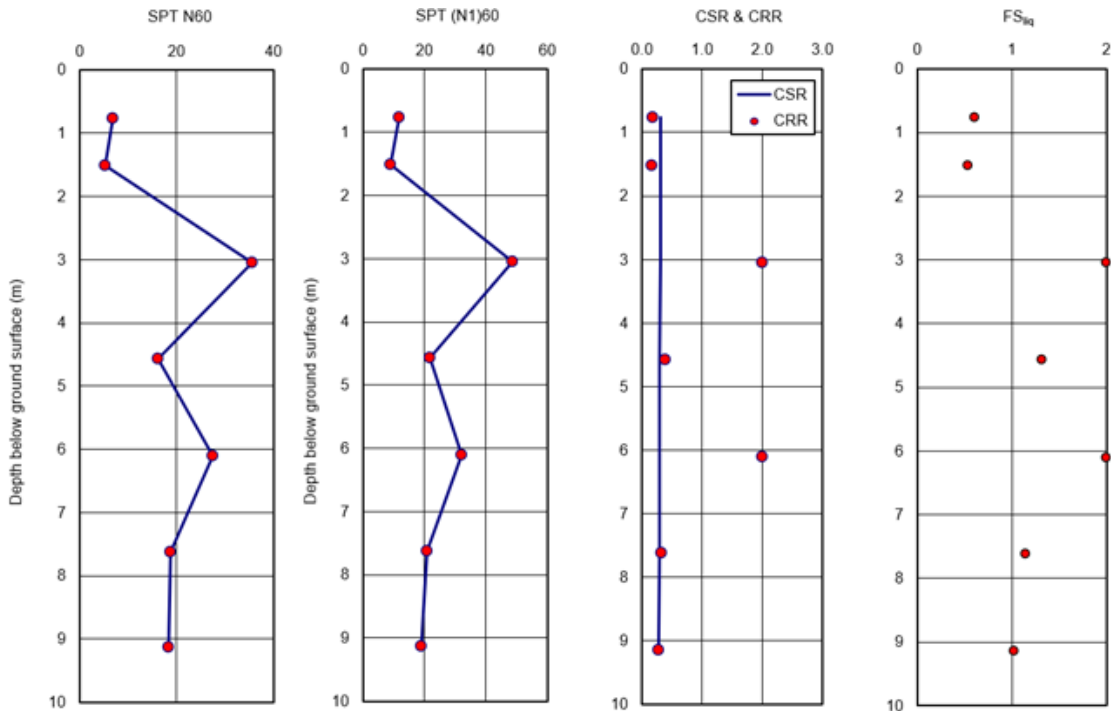


Figure 3.4.1.4.2: Analysis 2 - Water Table at Ground Surface

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Analyzing for expected site conditions yields a factor of safety approximately equal to 1.0. However, the flood conditions analysis predicted topsoil liquefaction. Large spikes in cyclic resistance ratio from data points at 3.1 m. and 6.1 m. are likely due to boulders affecting SPT testing; a complication that was also observed through field visits. As this preliminary examination has produced factors of safety below 1.2, a more comprehensive liquefaction analysis is recommended, and some form of remediation may be necessary.

3.4.2 Structural Design

The structural analysis for the bridge was completed for the slab, floor beam, truss, and column components of the proposed bridge solution. The depth dimensions of the prestressed concrete voided slabs is based on our designed loading conditions and are provided by the Knife River company.

3.4.2.1 Truss Analysis

The MathCad, SAP2000, and Excel computer programs are used for the truss analysis. The dead loads were calculated in MathCAD and confirmed in Excel. The truss structural analysis was performed in SAP2000 and Excel. All our calculations are attached in Appendix D. The applied loads were defined based on slab and floor beam calculations. Due to symmetry, one truss was analyzed using a tributary area of half of the slab and half the weight of floor beams. To account for future loads, the dead load of the slab and floor beams were multiplied by a factor of 15% in addition to the factors determined by the LRFD. The expected future loads that were unaccounted for are a fence, small steel railing, and any necessary utilities. The loading for the slab and floor beams were calculated and applied to the truss at the floor beam points located at one third points on the bottom chords. A load of 0.5 kips was added to the top chord at the panel points to allow for future top chord bracing.

The analysis was prepared using the SAP2000 computer program. The self-weight of the truss members were determined using a self-weight multiplier to the beams. This allowed for a more efficient iteration process. The dead loads, H10 truck live load, and pedestrian load were added to the truss. Arbitrary members were originally chosen to allow for a hand check of the reasonability of the model. Our SAP2000 model was checked with an excel calculation, and got approximately a 5% difference between our results and the SAP2000 results. This difference in the results is quite typical since the moving live load from the H10 truck was not calculated. Our SAP2000 model was determined to be correct by using our Excel

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spreadsheet. For the remainder of calculations, the SAP2000 Model calculated stresses and forces are used.

Once our structural check for the truss was finished, then the most economic beam that will provide optimum demand over capacity ratio (D/C), serviceability check, and frequency of vibration was calculated. For this calculation, an Excel spreadsheet was created to check the sufficiency of beams. Using the first arbitrary beam selection, smaller members were selected. The SAP2000 Model calculated the new results for the truss members. These stresses and members were inputted into the excel spreadsheet to check the structural efficiency. Using the results of the excel, new members were chosen, and the iterative process was executed again. The process was repeated over 20 times to produce the lightest weight beams that were sufficient for the structure according to LRFD Pedestrian Bridge Standards.

3.4.2.2 Floor Beams

All floor beam calculations and analysis are attached in Appendix D.5. The tributary width was found by assuming one-way slab behavior and three floor beams per panel. A 6.5-inch slab was designed (Appendix D.6) and used for the dead load. A dead load factor of 1.25 was used per AASHTO LRFD Table 3.4.1-2. A railing deadweight was assumed to be 5 plf from the truss analysis. Using the tributary width, point loads were found and were placed 6-inches from each side.

A 90 psf live load was placed onto the floor beams per AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges. A live load factor of 1.75 was used per AASHTO LRFD Table 3.4.1-1. A H10 design truck live load was used from Table 3.2-1 of the Pedestrian Bridge AASHTO LRFD Guide. Two loading combinations were determined. Combination 1 included the dead load and pedestrian live load. Combination 2 included the dead load and H10 truck live load. By cutting into the section, moment equations were solved for and calculated. Combination 2 was determined to be the controlling load combination.

By assuming continuous lateral support and a yield strength of 50 ksi, the Z_x required could be calculated per AASHTO LRFD 6.12.2.2.1. From the AISC Steel Manual, a member was picked from the required Z_x . Max moment and shear values were then checked for adequacy.

3.4.2.3 Slab Design

The slab design analysis was performed following The AASHTO LRFD Specifications, and MathCad was used to perform calculations (see Appendix D.6). For the corrugated metal deck, there was no design specification in AASHTO

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LRFD. However, the only restrictions were to show that the concept works. Two, show that the deck can resist the compressive forces associated with the composite action (see section, 9.8.5- AASHTO LRFD-2012, for more information). The proof of concept using corrugated steel for different loads and spans was demonstrated and examined by U.S. BRIDGE (see tables in Appendix D.6).

The slab was designed as a one-way slab with a width of one foot. The slab was checked for flexural resistance and live load deflection limit. A 90 psf live load and a dead load based on a 6.5 slab thickness are used (see Appendix D.6). The live and dead loads factors of 1.75 and 1.25 are used respectively for load combination as per AASHTO LRFD Tables 3.4.1-1 and 3.4.1-2.

For flexural design, the slab was treated as a simply supported beam to get the moment (M_u), which is the moment generated by the load. Then, equation 5.7.3.2.1-1 from the AASHTO LRFD code is used to check for moment strength (ϕM_n), which is the resistance moment provided by the reinforcement. Since the Generated moment (M_u) is less than the resistance moment (ϕM_n), the design is ok.

The deflection check was performed using section 9.5.2 from the AASHTO LRFD code (see Appendix D.6). The maximum allowable deflection was taken for a deck with a significant pedestrian load (Δ allowable). The generated deflection was based on the live load design (Δ). Since the maximum allowable deflection (Δ allowable) is bigger than the live load generated deflection (Δ), the design is ok.

As a result, the total thickness of the slab was determined to be 6.5 inches. A 4.25X12 gage 9 corrugated steel was used for reinforcement, and wire size 8 at 6 inches spacing was used for shrinkage and temperature reinforcement (see Appendix C.6 for visual representation).

3.4.2.4 Columns

For the design of the bridge columns, all calculations were performed using Mathcad Prime (see appendix D.7). The column loading conditions were determined by the bridge weight estimate calculations performed in Excel (see appendix D.8 and D.4). To determine preliminary values for the demand on the columns, the bridge spans are assumed to be simply supported, thus one half of the weight of each span, gravity factored dead and live load, are supported by the columns. The maximum load is supported by column 2 which supports the truss and prestressed spans of the bridge. Therefore, column 2 was chosen as the control for the design of all column cross sections and reinforcing.

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For the calculations all equations and constants are taken from AASHTO LRFD 2012 Bridge Design Specifications 6th Edition (US). The load cases of all axial and no moment, and all moment and no axial are used in the design to produce the maximum load effects for a conservative preliminary design to provide cost estimation of the bridge materials and construction. A large cross section of 3 ft by 6 ft was chosen as an initial value for the cross section of the bridge columns to ensure that no slenderness effects would need to be considered which simplifies calculations, and increases the factor of safety of the columns.

The longitudinal reinforcement was determined using values from AASHTO article 5.10.11.4.1a, the minimum area of steel was chosen as one percent of the gross area of the column cross section. A total of 26 #9 rebars are distributed throughout the cross section to provide well distributed reinforcement which prevents brittle failure modes of the column under loading conditions. The transverse reinforcement in the column was chosen as sets of overlapping #3 rebar closed ties spaced at a minimum 12 inches per AASHTO article 5.8.2.7. Overlapping closed ties were specified due to the wide cross section used, these will help to ensure the multiple rows of longitudinal rebar are well confined. The concrete compressive strength and steel reinforcement grade are specified as 4000 psi, and grade 60 respectively.

The nominal axial capacity of the column is computed using AASHTO eq. 5.7.4.4-3 with a resistance factor of 0.75 per AASHTO article 5.5.4.2.1, the magnitude of nominal axial capacity was calculated to be 8228 kips. The nominal flexural capacity of the column is computed using AASHTO eq. 5.7.3.2.2-1 with a resistance factor of 0.9 per AASHTO article 5.5.4.2.1. The flexural capacity is considered in the weak axis direction as this controls the design for flexure, the magnitude of nominal flexural capacity in the weak axis was calculated to be 3430 kip*ft. The flexural demand is controlled by seismic loading conditions, the factored moment M_u is taken as 5% of superstructure dead weight acting at a lever arm of the column unbraced length with load factor 1.0 per ASCE 7-16 factored load combinations. The distance to the neutral axis from the extreme compressive fiber “c” is approximated using AASHTO eq. 5.7.3.1.2-4 with factor β_1 equal to 0.85 per AASHTO article 5.7.2.2. For both load conditions, axial and flexure, it was found that the capacity is far greater than the demand (see Appendix D.7). Thus, the column design is considered to be highly conservative for the preliminary design and should provide a reasonably accurate cost estimation for a 30 percent design of the pedestrian bridge.

3.5 CONSTRUCTION COST ESTIMATE

This section will include the cost estimates for the primary aspects of construction. Line-item rates were based on average historical bid prices supplied by the Oregon Department of Transportation (ODOT, 2019) and the RSMMeans pricing database (RSMMeans, 2021). Quantities were estimated using units recommended by the 2018 Oregon Standard Specifications for Construction. Labor expenses associated with bridge construction were included in the various estimates depicted within Table A.1 in the Appendix. Transportation expenses associated with delivering heavy machinery and materials were approximated to be 8.0% of total unfactored material costs.

3.5.1 Steel Beams

A992 steel beams composing the bridge's superstructure were categorized, and quantities of each beam were calculated. The length of each beam was then multiplied by their respective quantities, resulting in the pricings of Table A.1 by linear foot. Beams with non-standard dimensions were upsized to the nearest available unit. The length of required fencing and railing along both sides of the bridge was taken to be twice the structure's length, totaling 540 ft of material.

3.5.2 Aggregate Base

The quantity of crushed aggregate base beneath the bridge's piers was approximated by adding a 1ft perimeter around each column, then multiplying the resulting surface area by a unit depth of 12in and an average density of 168 lb/ft³. Any aggregate used to rebuild the approaches connecting to the bridge was not included in this preliminary estimate.

3.5.3 Concrete Slabs

The cost of six 54.5ft long by 26in deep prestressed concrete voided slabs that constitute the bridge decking were calculated, on a linear foot basis, as shown in Table A.1. Following conservative design for non-standard units, the depth of each 26in slab was upsized so that historic bid prices could be applied.

3.5.4 Wearing Surface

The total cost of the wearing surface was estimated by adding the weight of two 6ft asphalt lifts together (HMAC Level 2). The weight of each asphalt lift was calculated by multiplying an approximate material density of 145 lb/ft³ by a 6ft depth, 13ft width, and 260 in paved surface length.

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3.5.5 Structural Concrete

The amount of structural concrete needed for the bridge was estimated by determining the volume of concrete required for each part of the structure. The concrete elements of the bridge structure consist of the slab, the columns, and the footings. The slab volume was calculated as the bridge length multiplied by the bridge depth multiplied by the bridge width, $240\text{ft} \times 6.5/12\text{ft} \times 15\text{ft} = 1950\text{ft}^3$. Each column's concrete volume was calculated as the column width multiplied by the column length multiplied by the column height, $3\text{ft} \times 6\text{ft} \times 18.8\text{ft} = 338.4\text{ft}^3$. Each footing had different dimensions, and the volume for the footing was calculated as the footing length multiplied by the footing depth multiplied by the footing width, $12\text{ft} \times 3\text{ft} \times 6\text{ft} = 216\text{ft}^3$. The total volume needed for the structural concrete was calculated to be approximately 180 cubic yards.

3.5.6 Reinforcement

The amount of reinforcement used in the structure was determined from the three parts of the bridge that require reinforcement, the slab, the columns, and the footings. The slab was assumed to have a minimum reinforcement, which had a ρ value of 0.0018. This would provide a steel area of approximately $0.0018 \times 14.5\text{ft} \times 6.5/12\text{ft} = 0.014\text{ft}^2$. That totaled to a volume of 3.38ft^3 , after multiplying by the bridge length, for the slab's horizontal reinforcement. Similarly, the transverse reinforcement was also accounted for. Two columns had similar volumes of reinforcing steel, while the third longest one had more volume of reinforcing steel. The columns had a rebar number 9, and there were 26 vertical rebars in each column. The reinforcement volume was calculated as the cross-sectional area of the steel in the columns multiplied by the length of the column, $1\text{in}^2 \times (1\text{ft}/12\text{in})^2 \times 26\text{bar} \times (18.8\text{ft} + 1\text{ft}) = 3.575\text{ft}^3$. Note that an additional foot was added to the length of the rebars in the columns to account for steel development length. A similar approach was taken to estimate the reinforcement volume for the footings. Then, from the total volume of required steel reinforcement, the approximate weight of steel needed was determined to be 17,500 lbs.

3.5.7 Excavation

The amount of general site excavation required was determined by calculating the site surface area and estimating the excavating depth. The site surface area was estimated to be rectangular. The site width was assumed to be 10ft to the left of the bridge width and 10ft to the right of the bridge width, resulting in approximately 35ft total width for the site. The site length was taken as the length of the bridge itself and was approximated to be 240ft. The excavation is assumed to take place in the site surface area. However, the surface area where the rails are located will not

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be included in the excavation. The excavation was assumed to be 1.5ft deep, and therefore, the estimated volume of general site excavation required would approximately be 400 cubic yards.

3.6 CONSTRUCTION SCHEDULE

This section outlines the construction schedule in Appendix B. This preliminary schedule is to provide an estimated timeline of critical tasks for the construction of the proposed design.

4.0 REGULATORY COMPLIANCE AND PERMITTING

This section outlines the regulatory agencies and relevant codes or standards. Any new construction must follow applicable regulations and obtain proper permits from relevant authorities prior to starting construction.

4.1 CITY OF TROUTDALE

The Community Development Department's Building Division is responsible for reviewing construction drawings and documents to insure compliance, issuance of building permits, inspection of construction sites and reporting construction compliance to relevant agencies. Permit requests, plan reviews, and inspections can be scheduled online through the Citizen Self-Service portal on the Community Development website (Community Development).

4.2 AMERICANS WITH DISABILITIES ACT (ADA)

All projects are required to follow 2010 ADA Standards for Accessible Design requirements, including the Title II and III regulations to ensure people with disabilities can use public spaces safely. The 2010 ADA regulations relevant to bridge projects include railing heights, edge protections, accessible routes, path of travel, etc. These regulations should be addressed with the building permit application by providing the following information: applicant information, property owner information, contractor information, project site information, and project description.

4.3 ENVIRONMENTAL PROTECTION AGENCY (EPA)

The Environmental Protection Agency (EPA) issues permits in accordance with the Federal Clean Water Act through the National Pollutant Discharge Elimination System (NPDES). In Oregon, the Department of Environmental Quality (DEQ) issues NPDES permits, and a 1200-C Construction Stormwater permit would be required for stormwater discharges to surface waters from the construction if stormwater leaves the site through a "point source" and reaches surface waters either directly or through storm drainage.

4.4 UNION PACIFIC RAILROAD

A Preliminary Engineering Agreement, location map, and concept plan needs to be provided to Union Pacific for their review of the proposed project. The Preliminary Engineering Agreement is used to address preliminary engineering issues related to operations, property issues, or effect on Union Pacific's facilities. When the final plans and approval of Union Pacific's cost estimate is complete, Union Pacific will prepare the appropriate license, right of entry, and construction and maintenance agreement(s). Construction may begin after this step is completed.

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All aforementioned permits will require their own individual submittal with the necessary project attachments. The project attachments that are needed to complete all permits are as follows: applicant information, consultant/agent information, proposed bridge design, legal authority for proposed action, dimensions of bridge clearance, other agencies with jurisdiction over the project. Plan sheets that are required to be attached and include the following: general dimensions and distances, title blocks, location and vicinity map, plan view, elevation view, typical section view, temporary structure/falsework.

5.0 CONCLUSION

Based on the building site's existing conditions, and the goals set forth by the City of Troutdale, the recommended facility design is three spans that total 239 feet. The main span is a 130-foot steel truss span that connects the urban renewal area up to the central business district downtown area. The other two spans are voided slabs with a length of 54.5 feet each. To preserve the architecture, views of the current tenants, and to minimize the intrusiveness of machinery during construction these slabs will be made from prestressed concrete and will run from the East Historic Columbia Highway to the back edge of the Union Pacific Railroad Property Line. The width of the bridge is 14 feet to accommodate clearances for two-way traffic of bicycles and golf carts.

This design option will be the most economical and practical for this pedestrian bridge's use, as summarized in this report's alternative scoring section. The scope of this project was a 30% initial design that stopped just north of the railway along with cost estimates and a goal of the construction taking place in the next five years. Future steps should include additional analysis and design for the northern side connection of the bridge after the railway, design work of the bent caps, a deep foundation analysis, and finalization of the construction schedule. We recommend further exploration of earthquake loadings, soil liquefaction, runoff effects, and pedestrian-vehicle interaction.

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APPENDICES

The following appendices are attached.

A. Construction Cost Estimate

This section details the cost estimation calculation process described in Section 3.5 Construction Cost Estimate.

B. Construction Schedule

This section details the construction schedule described in section 3.6 Construction Schedule. The construction schedule is based on assumptions provided by section 3.6 Construction Schedule.

C. Drawings

This section contains the computer aided design models to demonstrate facility design.

D. Calculations

This section contains the calculations detailed in section 3.4 Calculations.

E. Project Posters**F. Final Presentation**

APPENDIX A
CONSTRUCTION COST ESTIMATE

Table A.1 Construction Cost Estimate

REF.	ITEM	QUANT.	UNIT	UNIT COST	TOTAL	SECTION TOTAL
MOBILIZATION & TRAFFIC CONTROL						\$232,814
00210-90	Mobilization	1	LS	\$105,664	\$105,664	
00225-90	Temporary Signs	150	Ea.	\$24	\$3,600	
00225-98	Flaggers	2000	Hr.	\$40	\$80,000	
00280-90	Construction Entrances	2	Ea.	\$1,486	\$2,972	
00280-90	Plastic Sheeting	270	S.Y.	\$3	\$788	
00280-90	Matting	270	S.Y.	\$3	\$845	
00290-90	Pollution Control Plan	1	LS	\$9,195	\$9,195	
00290-90	Work Containment Plan	1	LS	\$20,000	\$20,000	
00270-90	Temporary Type 1 Fence	650	L.F.	\$15	\$9,750	
ROADWAY						\$4,089
00150-15	Construction Survey Work	3	Day	\$1,363	\$4,089	
EARTHWORK						\$31,008
00510-90	General Excavation	400	C.Y.	\$11	\$4,236	
00510-90	Structure Excavation	100	C.Y.	\$36	\$3,642	
00320-90	Clearing and Grubbing	0.2	Acre	\$6,127	\$1,225	
01040-90	Soil Testing	6	Ea.	\$415	\$2,492	
01040-90	Topsoil	400	C.Y.	\$49	\$19,412	
BRIDGE						\$811,595
00530-90	Reinforcement	17500	Lb.	\$3	\$59,850	
00540-90	Structural Concrete, 3600	180	C.Y.	\$1,328	\$239,067	
00587-90	Pedestrian Rail	520	L.F.	\$104	\$54,168	
00550-90	Prestressed Concrete Voided Slabs 26"	327	L.F.	\$263	\$85,945	
R051223-10	W18x86, A992	1024	L.F.	\$145	\$148,562	
R051223-10	W12x87, A992	195	L.F.	\$147	\$28,741	
R051223-10	W18x158, A992	368	L.F.	\$263	\$96,902	
R051223-10	W12x53, A992	159.5	L.F.	\$100	\$15,896	
R051223-10	W10x17, A992	449.5	L.F.	\$33	\$14,807	
01050-90	Chain Link Fence	520	L.F.	\$30	\$15,668	
00970-90	Street Lights	1	LS	\$4,050	\$4,050	
00545-90	Reinforced Concrete Bridge End Panels	109	S.Y.	\$404	\$44,036	
R050516-30	Galvanized Coating	5070	S.F.	\$1	\$3,904	
BASE						\$12,710
00640-90	Aggregate Base (12")	310	Ton	\$41	\$12,710	
WEARING SURFACE						\$29,444
00744-90	Level 2, 1/2-in Dense HMA Mixture	122.5	Ton	\$120	\$14,722	
00744-90	Level 2, 1/2-in Dense HMA Mixture	122.5	Ton	\$120	\$14,722	

ENGINEER'S ESTIMATE OF CONSTRUCTION COST \$1,121,660

30% CONTINGENCY \$336,498

TOTAL ESTIMATED CONSTRUCTION COST \$1,458,158

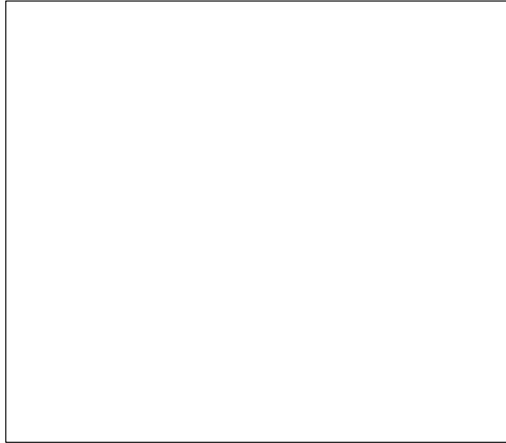
APPENDIX B
CONSTRUCTION SCHEDULE

ID	Task Mode	Task Name	Duration	Start	Finish	May 2021	June 2021	July 2021
1		[Capstone] CBD-URA Pedestrian Bridge Project	57 days?	Tue 5/4/21	Wed 7/21/21	[Gantt Chart: Project Summary]		
2		Begin Construction	57 days?	Tue 5/4/21	Wed 7/21/21	[Gantt Chart: Begin Construction]		
3		1.0 Foundations	51 days	Tue 5/4/21	Tue 7/13/21	[Gantt Chart: Foundations Summary]		
4		1.1 Mobilization & Staging	2 days	Tue 5/4/21	Wed 5/5/21	[Gantt Chart: Mobilization & Staging]		
5		1.2 Demo/Site Prep	5 days	Wed 5/5/21	Wed 5/12/21	[Gantt Chart: Demo/Site Prep]		
11		1.3 Footing Excavation	5 days	Thu 5/13/21	Wed 5/19/21	[Gantt Chart: Footing Excavation]		
12		1.4 Footing Basing & Prep	2 days	Thu 5/20/21	Fri 5/21/21	[Gantt Chart: Footing Basing & Prep]		
13		1.5 Concrete Forms & Reinforcing	4 days	Mon 5/24/21	Thu 5/27/21	[Gantt Chart: Concrete Forms & Reinforcing]		
14		1.6 Concrete Pour	2 days	Fri 5/28/21	Mon 5/31/21	[Gantt Chart: Concrete Pour]		
15		1.7 Concrete Curing	28 days	Tue 6/1/21	Thu 7/8/21	[Gantt Chart: Concrete Curing]		
16		1.8 Backfill & Grading	2 days	Fri 7/9/21	Mon 7/12/21	[Gantt Chart: Backfill & Grading]		
17		1.9 Demobilization	1 day	Tue 7/13/21	Tue 7/13/21	[Gantt Chart: Demobilization]		
18		2.0 Substructure	1 day?	Wed 7/14/21	Wed 7/14/21	[Gantt Chart: Substructure Summary]		
19		2.1 Columns	1 day?	Wed 7/14/21	Wed 7/14/21	[Gantt Chart: Columns]		
20		2.2 Stringers	1 day?	Wed 7/14/21	Wed 7/14/21	[Gantt Chart: Stringers]		
21		3.0 Superstructure	57 days	Tue 5/4/21	Wed 7/21/21	[Gantt Chart: Superstructure Summary]		
22		3.1 Truss Section	40 days	Tue 5/4/21	Mon 6/28/21	[Gantt Chart: Truss Section Summary]		
23		3.1.1 Truss Fabrication	40 days			[Gantt Chart: Truss Fabrication]		
24		3.2 Slab Section	5 days	Thu 7/15/21	Wed 7/21/21	[Gantt Chart: Slab Section]		

Project: [Capstone]CBD-URA Co Date: Fri 6/11/21	Task	Inactive Summary	External Tasks
	Split	Manual Task	External Milestone
	Milestone	Duration-only	Deadline
	Summary	Manual Summary Rollup	Progress
	Project Summary	Manual Summary	Manual Progress
	Inactive Task	Start-only	Progress
Inactive Milestone	Finish-only	Progress	

APPENDIX C
DRAWINGS

SHEET INDEX	
100	TITLE SHEET
101	GENERAL NOTES
200	SITE PLAN
300	RD PLAN & PROFILE
400	BR PLAN & ELEVATION
401	FOUNDATIONS
402	SPAN 1 & 2
403	SPAN 3
404A	TYPICAL SECTIONS
404B	TYPICAL SECTIONS
405A	DETAIL
405B	DETAIL

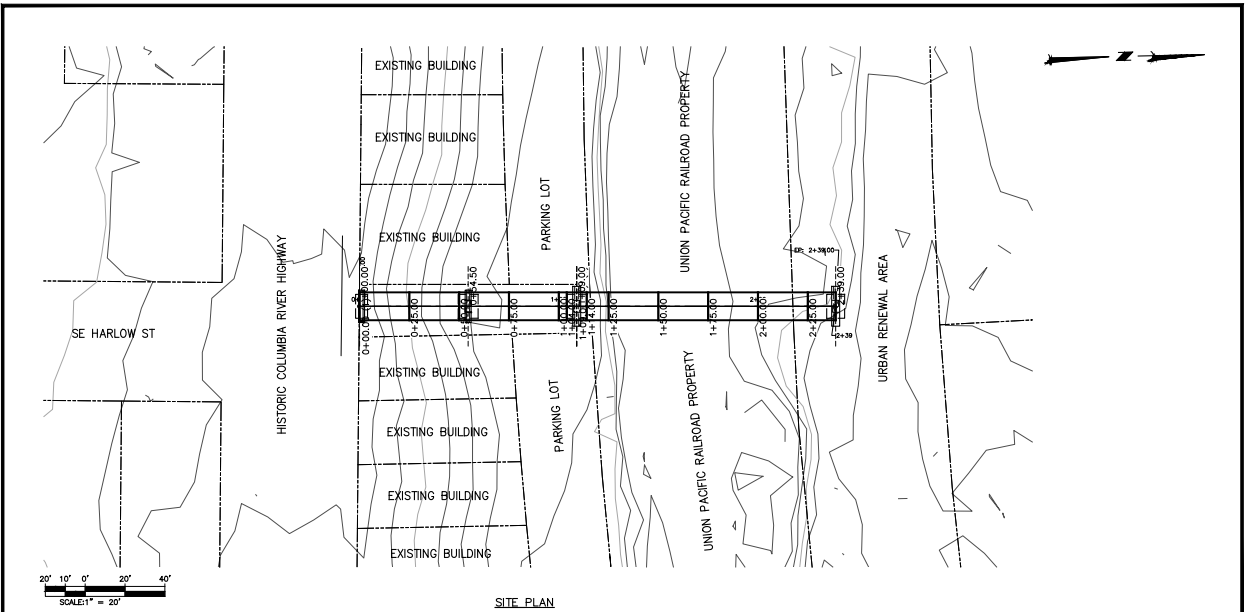


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LAYOUT SHEETING

DATE	REVISION	BY	DRAWN	CHECKED	REVIEWED	CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	
			ETHAN JUDD			TITLE SHEET	
							SHEET NO. 100

PLOT DATE: 6/9/2021 PLOT TIME: 11:33 AM USER: vjdd



SITE PLAN

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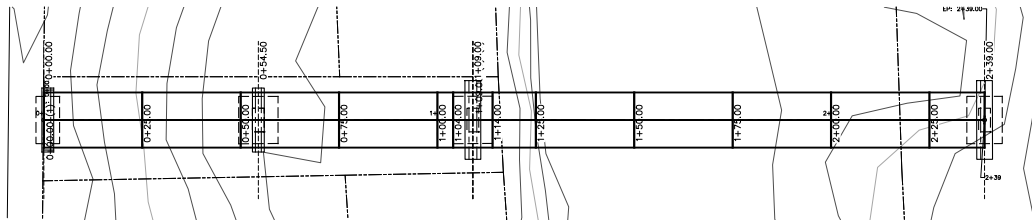
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 DESIGNER: _____
 CHECKER: _____
 REVIEWER: _____

CBD-URA PEDESTRIAN BRIDGE
 PSU CAPSTONE PROJECT
 SITE PLAN
 CBD-URA PED. BRIDGE PLAN VIEW

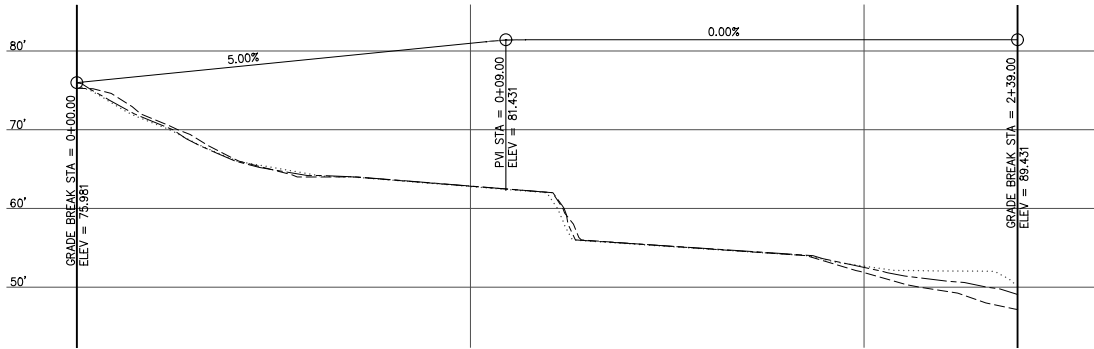
SHEET NO.
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LAYOUT SHEETS

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MULTI-USE PATH PLAN

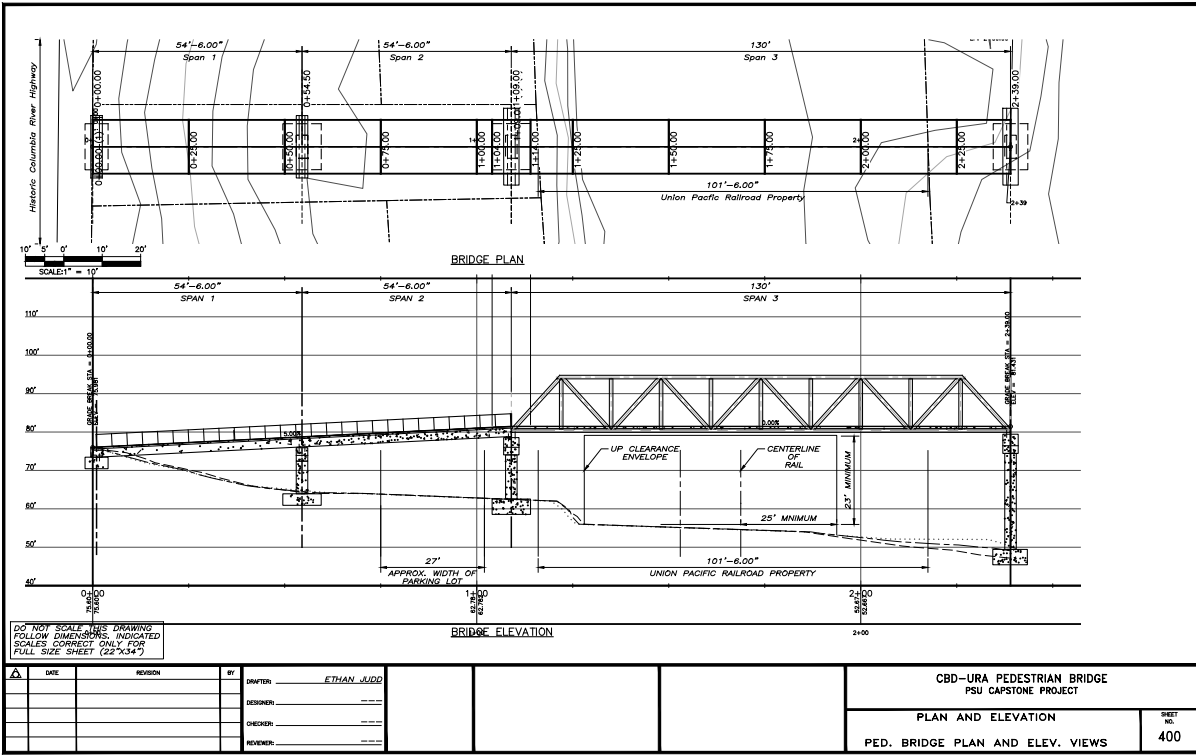


MULTI-USE PATH PROFILE

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				CHECKED: ---	SHEET NO. 300
				REVIEWED: ---	

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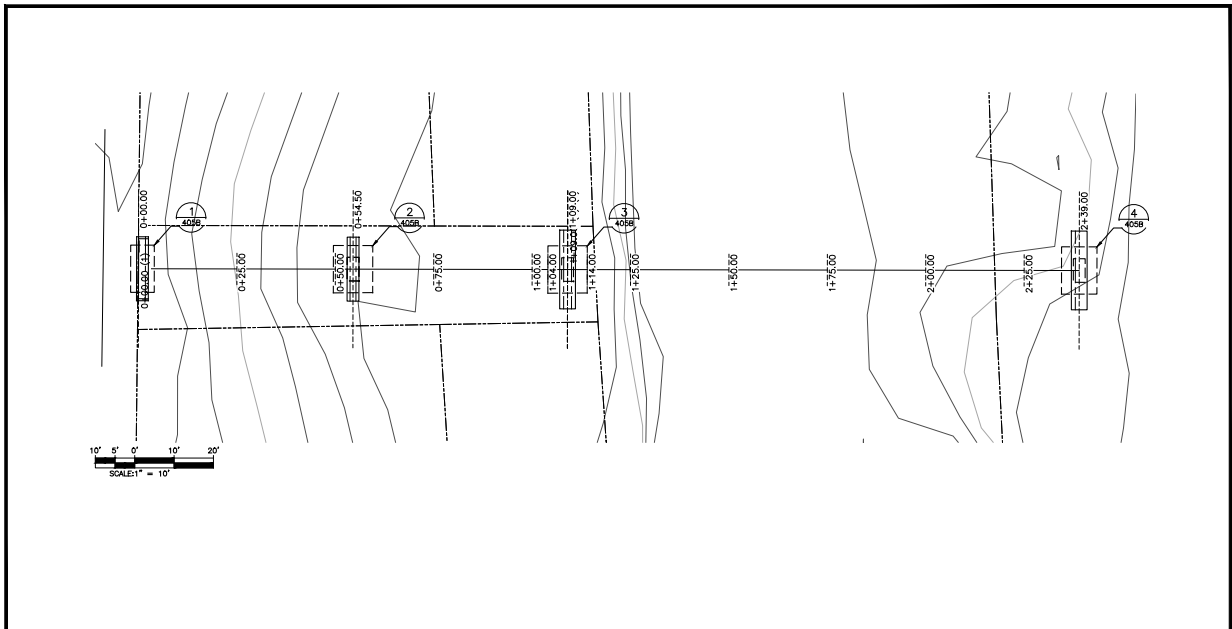
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DATE	REVISION	BY
		DRAWN: <u>ETHAN JUDD</u>
		DESIGNER: ---
		CHECKER: ---
		REVISOR: ---

CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	
PLAN AND ELEVATION	
PED. BRIDGE PLAN AND ELEV. VIEWS	
SHEET NO. 400	

LAYOUT SHEETING

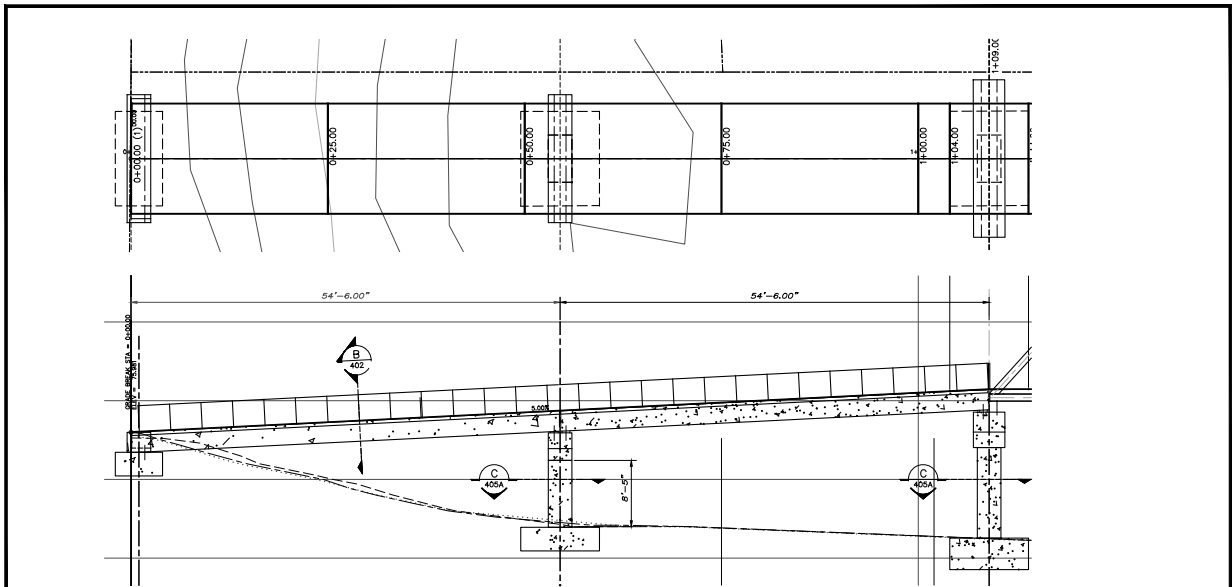
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LAYOUT SHEETS: 306	DATE	REVISION	BY	DRAWN: <u>ETHAN JUDD</u>	CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	FOUNDATION LOCATIONS	SHEET NO. 401
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				CHECKED: ---			
				REVIEWED: ---			

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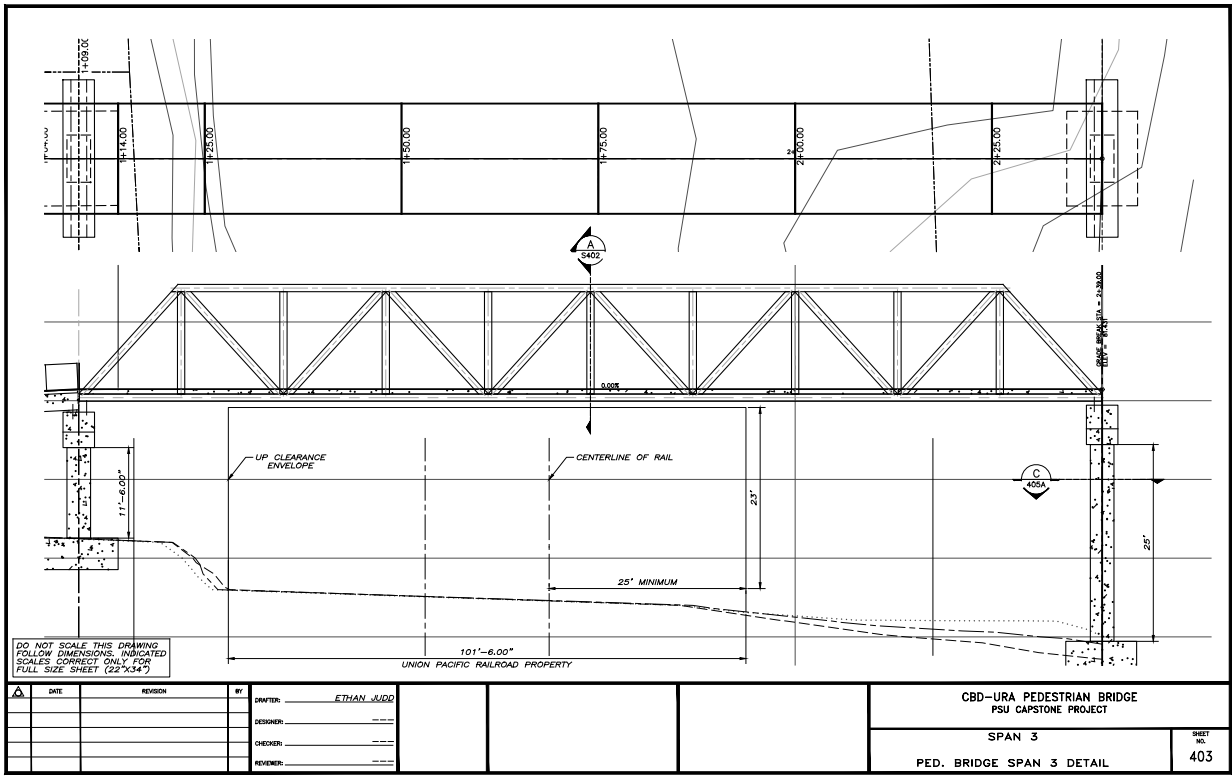
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LAYOUT SHEETS: BNC

DATE	REVISION	BY	DRWING
			ETHAN JUDD
			DESIGNER
			CHECKER
			REVIEWER

PLOT DATE: 6/9/2021 PLOT TIME: 11:33 AM USER: ethj

CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	
SPAN 1 AND 2	
PED. BRIDGE SPAN 1 AND 2 DETAIL	SHEET NO. 402



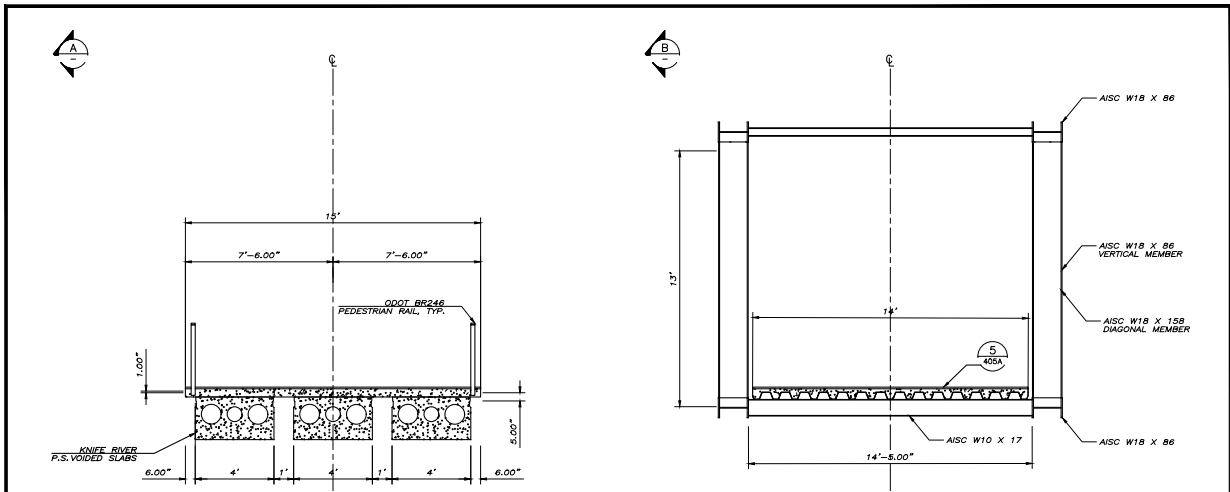
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CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	
SPAN 3	
PED. BRIDGE SPAN 3 DETAIL	SHEET NO. 403

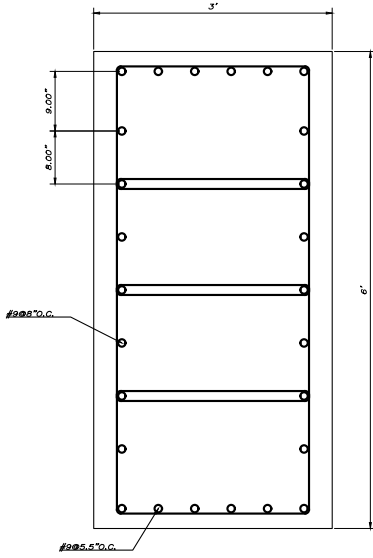
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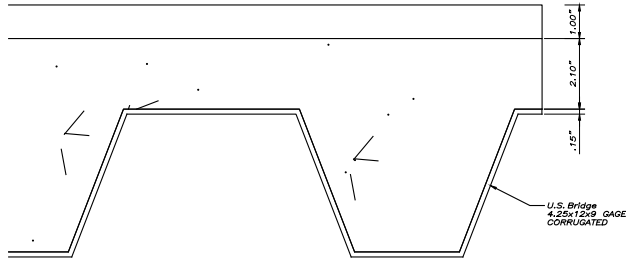
LAYOUT SHEETS: 08	DATE	REGION	BY	DRAWN: <u>ETHAN JUDD</u>				CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT
				DESIGNED: ---				
				CHECKED: ---				
			REVIEWED: ---				TYPICAL SECTIONS PED. BRIDGE SPAN SECTIONS	SHEET NO. 404A

PLOT DATE: 6/9/2021 PLOT TIME: 11:33 AM USER: ethj



DO NOT SCALE THIS DRAWING.
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 SCALES CORRECT ONLY FOR
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LAYOUT SHEETS/306	DATE	REVISION	BY	DRAWN: <u>ETHAN JUDD</u>	CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	TYPICAL SECTIONS PED. BRIDGE SECTIONS	SHEET NO. 404B
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				CHECKER: ---			
				REVISOR: ---			
PLOT DATE: 4/9/2021 PLOT TIME: 11:33 AM USER: ethj							

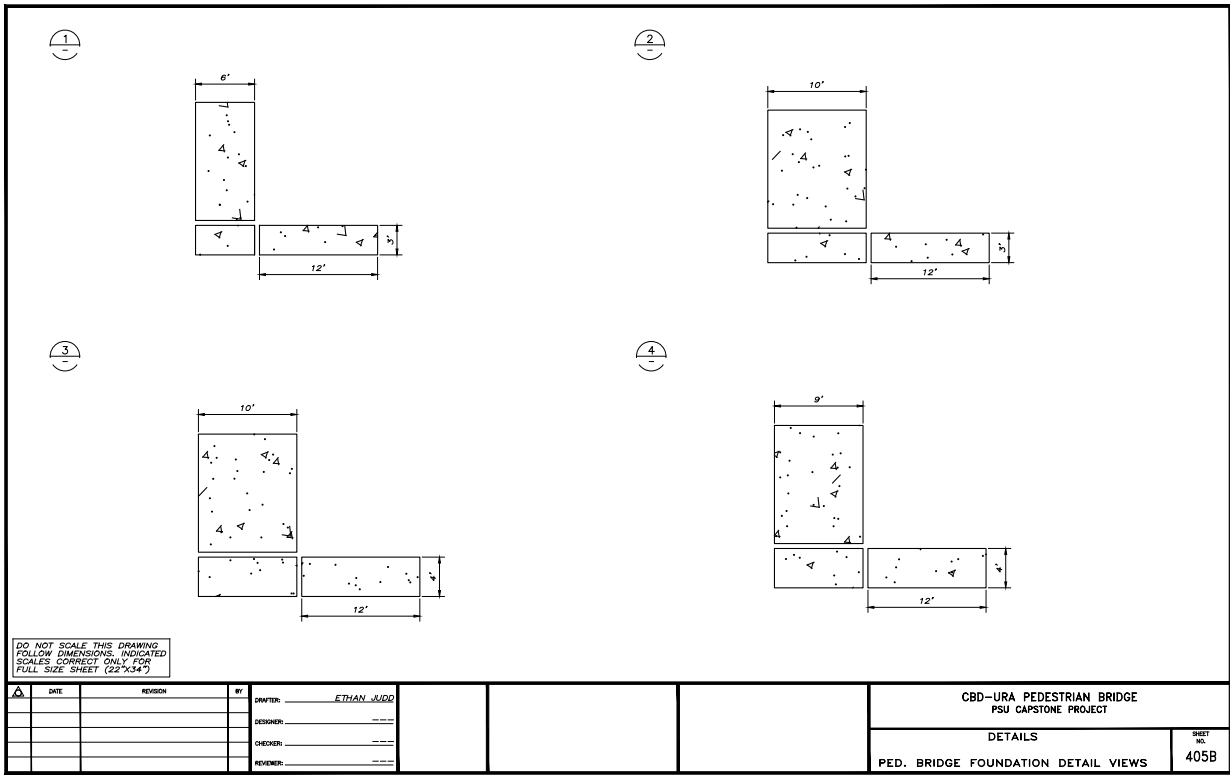


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LAYOUT SHEETING

DATE	REVISION	BY	DRAWN	DESIGNED	CHECKED	REVIEWED	CBD-URA PEDESTRIAN BRIDGE PSU CAPSTONE PROJECT	
			ETHAN JUDD				DETAILS	
							PED. BRIDGE DETAIL VIEWS	SHEET NO. 405A

PLOT DATE: 6/9/2021 PLOT TIME: 11:33 AM USER: vjudd



NO.	DATE	REVISION	BY

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 CHECKER: ---
 REVIEWER: ---

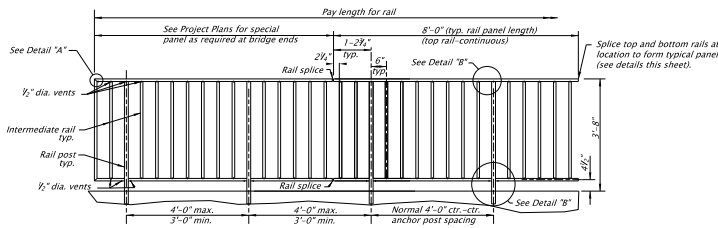
CBD-URA PEDESTRIAN BRIDGE
PSU CAPSTONE PROJECT

DETAILS

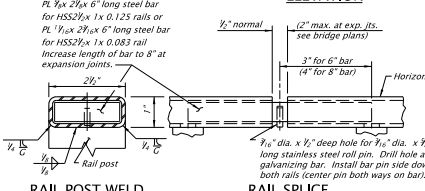
PED. BRIDGE FOUNDATION DETAIL VIEWS

SHEET NO.
405B

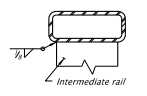
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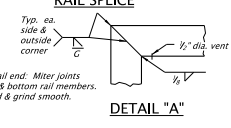
ELEVATION



RAIL POST WELD

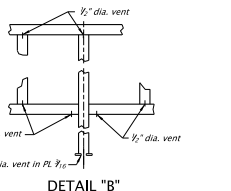


INTERMEDIATE VERTICAL RAIL WELD



RAIL SPLICE

DETAIL "A"



DETAIL "B"

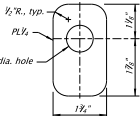
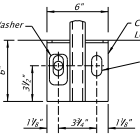
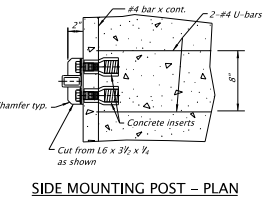
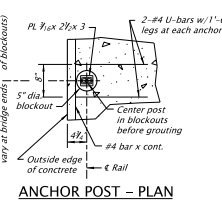
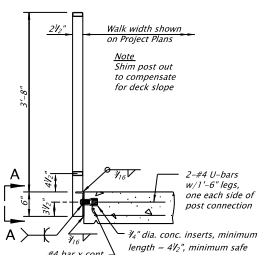
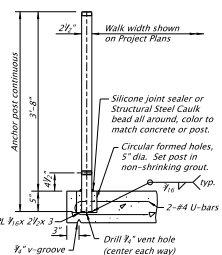


PLATE WASHER
(2 required per post)



VIEW A-A
SIDE MOUNTING PLATE

GENERAL NOTES
 Provide rectangular structural steel tubing according to Oregon Standard Specification 02810.20.
 Provide HSS 2 1/2" x 1 1/2" x 1/4" for anchor post.
 Provide HSS 2 1/2" x 1 x 0.083 (1.4 gauge) or HSS 2 1/2" x 1 x 0.125 (1.1 gauge) for horizontal rails and terminal end verticals.
 Provide HSS 2 x 1 x 0.083 (1.4 gauge), for intermediate verticals.
 (TS 2 1/2" x 1 x 0.125 (1.1 gauge) may be used for intermediate verticals if the anchor post weld details are used).
 Provide all other structural steel conforming to AASHTO M183 (ASTM A36).
 Hot-dip galvanize all structural steel including fasteners and inserts after fabrication.
 Provide Galvalume Control Silicon according to Oregon Standard Specification 02530.70.
 Construct railing conforming to the horizontal and vertical alignment of the structure.
 Install posts normal to grade in the longitudinal direction and vertical in the transverse direction.
 Provide continuous top horizontal rail elements over a minimum of two posts unless shown otherwise on detail plans. Provide a minimum of two anchor posts per rail panel.



CALC. BOOK NO. _____	SDR DATE: 20-Apr-2018
NOTE: All material and workmanship shall be in accordance with the current Oregon Standard Specifications.	
OREGON STANDARD DRAWINGS	
PEDESTRIAN RAIL	
2021	
DATE	REVISION DESCRIPTION

The selection and use of this Standard Drawing, while designed in accordance with generally accepted engineering principles and practices, is the sole responsibility of the user and should not be used without consulting a Registered Professional Engineer.

GENERAL:
 FENCE SHALL BE PROVIDED AS INDICATED ON THE CROSS SECTION ON BOTH SIDES OF THE VIADUCT. THE FENCE SHALL EXTEND COMPLETELY ACROSS THE STRUCTURE OR UPRR RIGHT-OF-WAY, WHICHEVER IS SHORTER.

SPLASH BOARDS SHALL BE PROVIDED ON BOTH SIDES OF THE VIADUCT IN LOCATIONS WHERE SWITCHING OR OTHER FREQUENT RAILROAD ACTIVITIES ARE PERFORMED, THE SPLASHBOARD SHALL EXTEND COMPLETELY ACROSS THE STRUCTURE OR UPRR RIGHT-OF-WAY, WHICHEVER IS SHORTER.

LIGHTS ARE TO BE INSTALLED ON THE UNDERSIDE OF THE VIADUCT WHERE SHADOWS CAST BY THE STRUCTURE WOULD INTERFERE WITH RAILROAD OPERATIONS.

SLOPE PAVING SHALL BE PROVIDED WHERE END SLOPES EXCEED 2 HORIZONTAL TO 1 VERTICAL.

FALSEWORK, NETTING OR OTHER SUITABLE PROTECTION SHALL BE PROVIDED TO PREVENT DEBRIS FROM FALLING ON THE TRACK DURING DEMOLITION AND CONSTRUCTION OPERATIONS.

APPLICANT SHALL BE RESPONSIBLE FOR IDENTIFICATION, LOCATION AND PROTECTION OF EXISTING UTILITIES.

CONTACT UPRR'S "CALL BEFORE YOU DIG" AT LEAST 48 HOURS PRIOR TO COMMENCING WORK AT 1-800-336-9193 TO DETERMINE LOCATION OF FIBER OPTICS.

CERTAIN LOCATIONS MAY REQUIRE ADDITIONAL CLEARANCES OR FEATURES BEYOND THOSE SHOWN IN THIS DRAWING BASED ON LOCAL CONDITIONS

EXCEPTIONS TO THESE STANDARDS MUST BE APPROVED BY UPRR'S GENERAL DIRECTOR OF DESIGN.

NOTES:

CLEARANCES:
 MINIMUM VERTICAL CLEARANCE SHALL BE 23' ABOVE THE PLANE OF TOP-OF-RAIL. ADDITIONAL CLEARANCE MAY BE REQUIRED 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.

MINIMUM CONSTRUCTION CLEARANCES SHALL BE 21' VERTICAL ABOVE THE PLANE OF TOP-OF-RAIL AND 12' HORIZONTAL AT RIGHT ANGLE FROM CENTERLINE OF TRACK.

HORIZONTAL CLEARANCES ARE TO BE INCREASED 1/2" PER DEGREE OF CURVE WHERE THE STRUCTURE IS LOCATED ADJACENT TO OR WITHIN 80' OF THE CURVE LIMITS.

FUTURE TRACKS:
 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 ACCESS ROADS ON BOTH SIDES OF TRACK.

PIERS:
 PIER PROTECTION (CRASH WALLS) SHALL BE PROVIDED IN ACCORDANCE WITH AREA CHAPTER 8, PART 2.1.5 FOR PIERS WITHIN 25 FEET OF THE CENTERLINE OF TRACK.

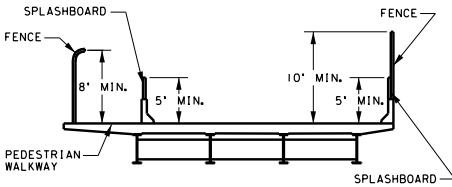
TOP OF FOOTING SHALL BE A MINIMUM OF 6' BELOW BASE OF RAIL AND A MINIMUM OF 1 FOOT BELOW FLOW LINE OF DITCH.

TEMPORARY OR PERMANENT SHORING SHALL BE DESIGNED IN ACCORDANCE WITH UPRR SHORING REQUIREMENTS (DRAWING NO. 1066131).

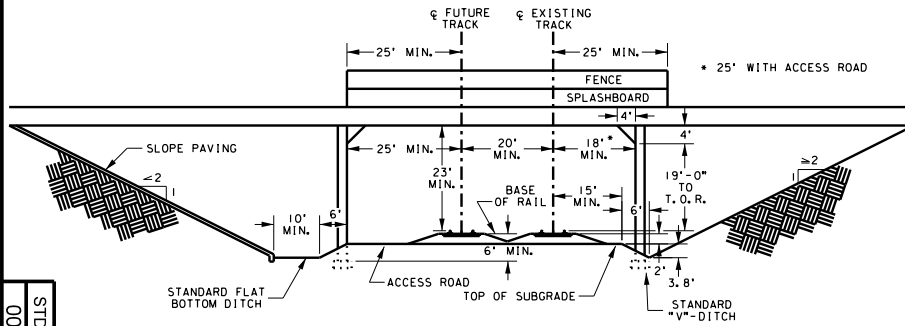
DRAINAGE:
 DRAINAGE FROM THE OVERPASS SHALL BE DIVERTED AWAY FROM UPRR TRACKS AND NOT DISCHARGED ONTO THE TRACKS OR ROADBED.

A STANDARD "V"-SHAPE OR FLAT-BOTTOM DITCH SHALL BE PROVIDED ON EACH SIDE OF TRACKS AS NECESSARY

CULVERTS MAY BE INSTALLED ON OPPOSITE SIDES OF COLUMN FROM TRACK IN LIEU OF STANDARD RAILROAD DITCHES WHEN APPROVED BY GENERAL DIRECTOR OF DESIGN. MAINTENANCE OF CULVERTS IS TO BE AT APPLICANT'S EXPENSE.



VIADUCT CROSS SECTION



ELEVATION
 PERPENDICULAR TO TRACKS

UNION PACIFIC RAILROAD
 ENGINEERING STANDARDS

DESIGN CLEARANCES
 FOR HIGHWAY AND
 PEDESTRIAN OVERPASS



APPROVED: *Casey Robinson*
 VP ENGINEERING
 ADOPTED: DEC. 30, 1996
 REVISED: NOV. 11, 2019
 FILE NO: 0035A

STD DWG

0035A

STD DWG
 0035A

APPENDIX D
CALCULATIONS

D.1 Earthquake Design - Liquefaction Analysis

D.2 Shallow Foundation Analysis and Footings

D.3 Truss Analysis (Excel)

D.4 Truss Analysis I-Beam Selection

D.5 Floor Beams

D.6 Slab Design

D.7 Columns

D.8 Prestressed Concrete Voided Slabs

D.1 Earthquake Design - Liquefaction Analysis

Liquefaction Analysis 1 - Water Table 1.5m Below Surface

Boring B-3

Input Parameters

Peak Ground Acceleration (g) =

0.24

Earthquake Magnitude, M =

7.5

Water Table Depth (m) =

1.52

Average γ above water table (kN/m³) =

18

Average γ below water table (kN/m³) =

20

Borehole Diameter (mm) =

127

Requires correction for sampler liners (Yes/No)

No

Rod lengths assumed equal to the depth plus 1.5m (for the above ground extension)

SPT sample number	Depth (m)	Measured N	Soil Type	Flag "Clay" "Unsaturated" "Unreliable"	Fines Content (%)	Energy ratio, ER (%)	C_E	C_B	C_R	C_S	N_{60}
1	0.76	7	FILL		22	75	1.25	1.05	0.75	1	6.9
2	1.52	5	FILL		24	75	1.25	1.05	0.8	1	5.3
4	3.05	32	FILL		36	75	1.25	1.05	0.85	1	35.7
7	4.57	13	Sandy SILT		31	75	1.25	1.05	0.95	1	16.2
8	6.1	22	SILT		31	75	1.25	1.05	0.95	1	27.4
9	7.62	15	SILT		26	75	1.25	1.05	0.95	1	18.7
10	9.14	14	SILT		38	75	1.25	1.05	1	1	18.4

σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N1)_{60}$	ΔN for fines content	$(N1)_{60-CS}$	Stress reduct. Coeff. r_d	CSR	MSF for Sand	K_σ for sand	CRR for $M=7.5$ & $svc' = 1atm$	CRR	Factor of Safety
14	14	1.70	11.7	4.8	16.5	1.00	0.156	1.00	1.10	0.169	0.186	1.19
27	27	1.70	8.9	5.0	13.9	1.00	0.155	1.00	1.10	0.147	0.162	1.04
58	43	1.25	44.7	5.5	50.2	0.98	0.207	1.00	1.10	2.000	2.000	2.00
88	58	1.24	20.1	5.4	25.5	0.97	0.228	1.00	1.09	0.303	0.331	1.45
119	74	1.11	30.3	5.4	35.7	0.95	0.238	1.00	1.09	1.301	1.412	2.00
149	90	1.05	19.6	5.1	24.8	0.93	0.242	1.00	1.02	0.285	0.290	1.20
180	105	0.98	18.1	5.6	23.6	0.91	0.243	1.00	0.99	0.261	0.260	1.07

Liquefaction Analysis 2 - Water Table at Ground Surface

Boring B-3

Input Parameters

Peak Ground Acceleration (g) =	0.24
Earthquake Magnitude, M =	7.5
Water Table Depth (m) =	0
Average γ above water table (kN/m ³) =	18
Average γ below water table (kN/m ³) =	20
Borehole Diameter (mm) =	127
Requires correction for sampler liners (Yes/No)	No
Rod lengths assumed equal to the depth plus 1.5m (for the above ground extension)	

SPT sample number	Depth (m)	Measured N	Soil Type	Flag "Clay" "Unreliable"	Fines Content (%)	Energy ratio, ER (%)	C _E	C _B	C _R	C _S	N ₆₀
1	0.76	7	FILL		22	75	1.25	1.05	0.75	1	6.9
2	1.52	5	FILL		24	75	1.25	1.05	0.8	1	5.3
4	3.05	32	FILL		36	75	1.25	1.05	0.85	1	35.7
7	4.57	13	Sandy SILT		31	75	1.25	1.05	0.95	1	16.2
8	6.1	22	SILT		31	75	1.25	1.05	0.95	1	27.4
9	7.62	15	SILT		26	75	1.25	1.05	0.95	1	18.7
10	9.14	14	SILT		38	75	1.25	1.05	1	1	18.4

σ_{vc} (kPa)	σ_{vc}' (kPa)	C _N	(N1) ₆₀	ΔN for fines content	(N1) _{60-CS}	Stress reduct. Coeff. r_d	CSR	MSF for Sand	K σ for sand	CRR for M=7.5 & $sv_c' = 1atm$	CRR	Factor of Safety
15	8	1.70	11.7	4.8	16.5	1.00	0.306	1.00	1.10	0.169	0.186	0.61
30	15	1.70	8.9	5.0	13.9	1.00	0.305	1.00	1.10	0.147	0.162	0.53
61	31	1.36	48.7	5.5	54.2	0.98	0.300	1.00	1.10	2.000	2.000	2.00
91	47	1.35	21.8	5.4	27.2	0.97	0.296	1.00	1.10	0.354	0.389	1.32
122	62	1.16	32.0	5.4	37.4	0.95	0.290	1.00	1.10	1.916	2.000	2.00
152	78	1.11	20.7	5.1	25.9	0.93	0.284	1.00	1.04	0.313	0.327	1.15
183	93	1.03	19.0	5.6	24.5	0.91	0.278	1.00	1.01	0.280	0.283	1.02

D.2 Shallow Foundation Analysis and Footings

Blow Count Normalization Calculations

Sublayer #	Sublayer Name	Depth (ft)	Depth (m)	Assumed N	Energy ratio (%)	Energy factor, Ch	Diameter factor, Cd	Sampler factor, Cs	Rod length factor, Cr	N ₆₀
1	SILT	2.5	0.76	7	80	1.33	1	1	0.75	7.0
1	SILT	5	1.52	5	80	1.33	1	1	0.75	5.0
1	SILT	7.5	2.29	9	80	1.33	1	1	0.75	9.0
2	SAND	10	3.05	32	80	1.33	1	1	0.75	32.0
2	SAND	12.5	3.81	10	80	1.33	1	1	0.85	11.3
2	SAND	15	4.57	13	80	1.33	1	1	0.85	14.7

Design Normalized Blow Count: 12.0

Figure D.2.1 Normalization and determination of design blow count calculation spreadsheet.

Terzaghi bearing capacity equation:

$$q_{ult} = q_b (N_q + 0.5) * \gamma_s * b * N_\gamma$$

Where:

q_{ult} = Ultimate bearing capacity of soil

q_b = Weight of soil removed = γ_sd_f

N_q = Coefficient of lateral earth pressure

γ_s = Unit weight of soil

b = Footing width

N_γ = Coefficient of soil unit weight

Allowable bearing capacity:

$$q_{allow} = FS * q_{ult}$$

Allowable axial load:

$$P_{allow} = q_{allow} / A_f$$

Where:

A_f = Total area of the footing

Allowable axial load example calculation spreadsheet.

Terzaghi Bearing Capacity and Allowable Axial Load Example Calculations

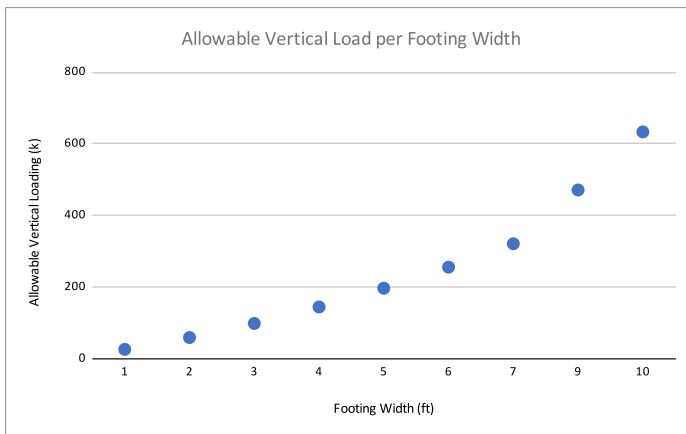
Footing Length l (ft)	Footing Depth df (ft)	Footing Width, b (ft)	Footing Area, Af (sq ft)	qb	Nq	unit weight of sand, ws (pcf)	Nsig	qult	FS	qallow	Pallow (kips)
10	3	1	10	330	17.6	110	14.7	6616.5	3	2205.5	22.055
10	3	2	20	330	17.6	110	14.7	7425	3	2475	49.5
10	3	3	30	330	17.6	110	14.7	8233.5	3	2744.5	82.335
10	3	4	40	330	17.6	110	14.7	9042	3	3014	120.56
10	3	5	50	330	17.6	110	14.7	9850.5	3	3283.5	164.18
10	3	6	60	330	17.6	110	14.7	10659	3	3553	213.18
10	3	7	70	330	17.6	110	14.7	11468	3	3822.5	267.58
10	3	8	80	330	17.6	110	14.7	12276	3	4092	327.36
10	3	9	90	330	17.6	110	14.7	13085	3	4361.5	392.54

Figure D.2.2 Example of allowable axial loads based on formulas above.

Footing Design

Assumptions are many, behavior is sand like, no cohesion, peak effective friction angle is 28 degrees, unit weight of sand is assumed.

Footing Length, l (ft)	Footing Depth, df (ft)	Footing Width, b (ft)	Footing Area, Af (sq ft)	qb	Nq	Unit weight of sand, ws (pcf)	Nsig	qult	FS	qallow	Pallow (kips)
12	3	1	12	330	17.6	110	14.7	6616.5	3	2205.5	26.466
12	3	2	24	330	17.6	110	14.7	7425	3	2475	59.4
12	3	3	36	330	17.6	110	14.7	8233.5	3	2744.5	98.802
12	3	4	48	330	17.6	110	14.7	9042	3	3014	144.672
12	3	5	60	330	17.6	110	14.7	9850.5	3	3283.5	197.01
12	3	6	72	330	17.6	110	14.7	10659	3	3553	255.816
12	3	7	84	330	17.6	110	14.7	11467.5	3	3822.5	321.09
12	3	9	108	330	17.6	110	14.7	13084.5	3	4361.5	471.042
12	4	10	120	440	17.6	110	14.7	15829	3	5276.333333	633.16



Support	Load
F1	221 South
F2	506
F3	611
F4	427 North

Footing Designs	L x d x b	Pallow > Load
F1	12x3x6	256>221
F2	12x3x10	556>506
F3	12x4x10	633>611
F4	12x4x9	471>427

*Where DF = Design for Footing

D.3 Truss Analysis (Excel)

Bridge Truss Dimensions and Weight		
Height of the truss	13	ft
Legth of each horizontal chord of the truss	13	ft
Number of each horizontal chord of the truss	10	pc
Length of the truss	130	ft
Width of the truss (inside)	14.5	ft
Live Load (LL)		
Loading factor	1.75	-
Pedestrian	90	lb/ft ²
Truck H10	20	kips
Total pedestrian load	169650	lb
Total pedestrian load	169.7	kips
Factored Weight of Truck H10	35	kips
Factored Total pedestrian load	296888	lb
Factored Total pedestrian load	296.9	kips
Maximum Live Load (LL)	296.9	kips
Unit conversion factor 1ft = 12in		
1 foot	12	in
Dead Load (DcL)		
Unit weight of concrete	150	lb/ft ³
Slab (Using corrugated steel sheets gauge #9)		
Thickness	2	in
Ribs thickness (effective)	2.1	in
Effective Slab Thickness	4.1	in
Linear weight	747.7	lb/ft
Linear weight for a single truss	373.8	lb/ft
Total Weight	97195	lb
Total Weight	97.2	kips
Corrugated Steel (Gauge #9)		
Area	1885	ft ²
Weight per ft ²	12.5	lb/ft ²
Total weight	23562.5	lb
Total weight	23.6	kips
Tributary load	54.2	lb/ft
Tributary load for a single truss	27.1	lb/ft
Decking		
Total weight	120758	lb
Total weight	120.8	kips
Linear weight	928.9	lb/ft
Linear weight for a single truss	464.5	lb/ft
Tributary point load at the middle section of the truss	4025.3	lb
Tributary point load at the middle section for a single truss	2012.6	lb
Tributary point load at the end of truss	2012.6	lb
Tributary point load at the end for a single truss	1006.3	lb
Floor Beams		
Length	14.5	ft
Linear weight	17	lb
Weight of each	246.5	lb
Point load from each to a single truss	123.3	lb
Number of floor beams per span	3	pc
Distance between floor beams	4.3	ft
Number of floor beams per truss	31	pc
Total weight	7642	lb
Total weight	7.6	kips

Loading at Panel Points for a single truss		
Decking load (at the ends of the truss)	1006	lb
Floor Beams	123.3	lb
Total	1130	lb
Decking load (in the middle of the truss)	2013	lb
Floor Beams	123.3	lb
Total	2136	lb
Truss		
Bottom Chord		
Number of sides of the bridge	2	sides
Length	260	ft
Linear weight	86	lb/ft
Total weight	22360	lb
Total weight	22.4	kips
Top Chord		
Number of sides of the bridge	2	sides
Length	208	ft
Linear weight	86	lb/ft
Total weight	17888	lb
Total weight	17.9	kips
Vertical Diagonal		
Number of sides of the bridge	2	sides
Length of each	18.4	ft
Length	367.7	ft
Linear weight	158	lb/ft
Distance between diagonal beams	13	ft
Number of vertical diagonal beams	20	pc
Total weight	58096	lb
Total weight	58.1	kips
Vertical Straight		
Number of sides of the bridge	2	sides
Length of each	13	ft
Linear weight	86	lb/ft
Distance between vertical beams	13	ft
Number of vertical beams	22	pc
Total weight	24596	lb
Total weight	24.6	kips
Horizontal Perpendicular beams above the bridge		
Length of each	14.5	ft
Linear weight	53	lb/ft
Distance between perpendicular beams	13	ft
Number of perpendicular beams	11	pc
Total weight	8454	lb
Total weight	8.5	kips
Horizontal Diagonal beams above the bridge		
Length of each	19.5	ft
Linear weight	87	lb/ft
Distance between perpendicular beams	13	ft
Number of perpendicular beams	10	pc
Total weight	16943	lb
Total weight	16.9	kips
Connections (10% of structural steel weight)		
Total weight	17954	lb
Total weight	18.0	kips

Total Dead Load (DcL)		
Loading factor	1.25	-
Summation of all Dead Loads	294689	lb
Summation of all Dead Loads	294.7	kips
Factored Total Deadload	368362	lb
Factored Total Deadload	368.4	kips
Maximum Dead Load DcL	368.4	kips
Dead Load (DwL)		
Pavement (DwL)		
Loading factor	1.5	-
Thickness	1	in
Linear weight	181.3	lb/ft
Total Weight	23563	lb
Total Weight	23.6	kips
Factored weight	35344	lb
Factored weight	35.3	kips
Utilities DwL		
Loading factor	0.15	-
Linear weight	148.2	lb/ft
Length of the bidge	130	ft
Number of sides of the bridge	2	sides
Total length of utilities	260	ft
Total weight	38520	lb
Total weight	38.5	kips
Total factored weight	57780	lb
Total factored weight	57.8	kips
Utilities DwL as a point load		
Loading factor	1.15	-
At the end of the truss	1299	lb
In the middle of the truss	2456	lb
Total Dead Load (DwL)		
Loading factor	1.5	-
Summation of all Dead Loads	62082	lb
Summation of all Dead Loads	62.1	kips
Factored Total Deadload	93123	lb
Factored Total Deadload	93.1	kips
Maximum Dead Load DwL	93.1	kips
Total Load From the Truss		
Dead Load (DC) + Dead Load (DW) + Live Load	526422	lb
Dead Load (DC) + Dead Load (DW) + Live Load	526.4	kips
Factored Dead Load + Live Load	758373	lb
Factored Dead Load + Live Load	758.4	kips
Total Unfactored Load From the Truss	526.4	kips
Total Factored Load From the Truss	758.4	kips
AASHTO 2009 page 5 Section C3.3; and page 21 Load Factors (AASHTO LRFD, Table 3.4.1-1)		

Total Unfactored Weight of the Steel Truss Section of the Bridge	526.4	kips
Total Unfactored Load From the Single Truss	-263.2	kips
0.5*Total Unfactored Load From the Truss (Applied point load to each of 2 supports)	-131.6	kips
Support Reaction at Point A	131.6	kips
Support Reaction at Point B	131.6	kips
Number of Sections	10	pc
Number of Point Loads	11	pc
Legnth of Truss	130	ft
0.5 Legnth of Truss	65	ft

Top Chord Horizontals	101-108	Element Number
Bottom Chord Horizontals	201-210	Element Number
Diagonals	301-310	Element Number
Verticals	401-409	Element Number
Point Loads	P ₁₋₁₁	Element Number

Height (vertical) =	13.0	ft
Length (horizontal) =	13.0	ft
Length (diagonal) =	18.3848	ft
Unfactored Loadings		
P _{1,11} =	-13.16	kips
P _{2,10} =	-26.32	kips

Zero-force members		
401	0	kips
403	0	kips
405	0	kips
407	0	kips
409	0	kips

Vertical members			
402	-26.32	kips	Compression
404	-26.32	kips	Compression
406	-26.32	kips	Compression
408	-26.32	kips	Compression

Diagonal members			
301	-167.50	kips	Compression
302	130.28	kips	Tension
303	-93.06	kips	Compression
304	55.83	kips	Tension
305	-18.61	kips	Compression
306	-18.61	kips	Compression
307	55.83	kips	Tension
308	-93.06	kips	Compression
309	130.28	kips	Tension
310	-167.50	kips	Compression

Horizontal Members Top Chord			
101	-118.44	kips	Compression
102	-210.56	kips	Compression
103	-276.36	kips	Compression
104	-315.84	kips	Compression
105	-315.84	kips	Compression
106	-276.36	kips	Compression
107	-210.56	kips	Compression
108	-118.44	kips	Compression

Horizontal Members Bottom Chord			
201	118.44	kips	Tension
202	210.56	kips	Tension
203	276.36	kips	Tension
204	315.84	kips	Tension
205	329.00	kips	Tension
206	329.00	kips	Tension
207	315.84	kips	Tension
208	276.36	kips	Tension
209	210.56	kips	Tension
210	118.44	kips	Tension

Total Factored Weight of the Steel Truss Section of the Bridge	758.4	kips
Total Factored Load From the Single Truss	-379.2	kips
0.5*Total Factored Load From the Truss (Applied point load to each of 2 supports)	-189.6	kips
Support Reaction at Point A	189.6	kips
Support Reaction at Point B	189.6	kips
Number of Sections	10	pc
Number of Point Loads	11	pc
Legnth of Truss	130	ft
0.5 Legnth of Truss	65	ft

Top Chord Horizontals	101-108	Element Number
Bottom Chord Horizontals	201-210	Element Number
Diagonals	301-310	Element Number
Verticals	401-409	Element Number
Point Loads	P ₁₋₁₁	Element Number

Height (vertical) =	13.0	ft
Length (horizontal) =	13.0	ft
Length (diagonal) =	18.3848	ft
Factored Loadings		
P _{1,11} =	-18.96	kips
P _{2,10} =	-37.92	kips

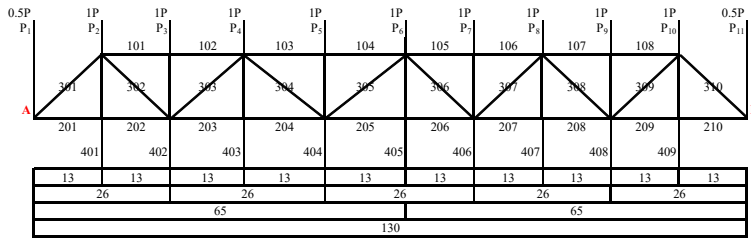
Zero-force members		
401	0	kips
403	0	kips
405	0	kips
407	0	kips
409	0	kips

Vertical members			
402	-37.92	kips	Compression
404	-37.92	kips	Compression
406	-37.92	kips	Compression
408	-37.92	kips	Compression

Diagonal members			
301	-241.32	kips	Compression
302	187.69	kips	Tension
303	-134.07	kips	Compression
304	80.44	kips	Tension
305	-26.81	kips	Compression
306	-26.81	kips	Compression
307	80.44	kips	Tension
308	-134.07	kips	Compression
309	187.69	kips	Tension
310	-241.32	kips	Compression

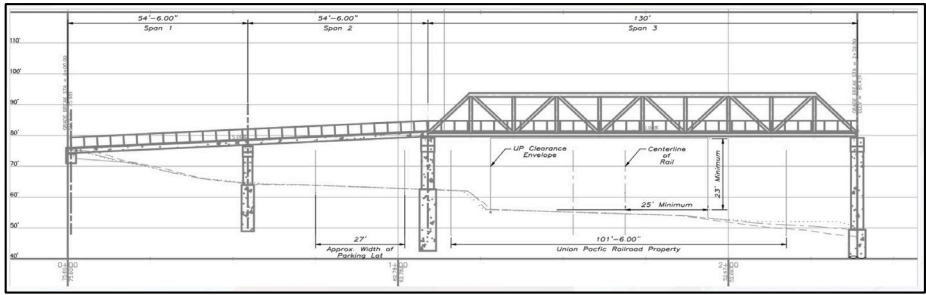
Horizontal Members Top Chord			
101	-170.64	kips	Compression
102	-303.36	kips	Compression
103	-398.16	kips	Compression
104	-455.04	kips	Compression
105	-455.04	kips	Compression
106	-398.16	kips	Compression
107	-303.36	kips	Compression
108	-170.64	kips	Compression

Horizontal Members Bottom Chord			
201	170.64	kips	Tension
202	303.36	kips	Tension
203	398.16	kips	Tension
204	455.04	kips	Tension
205	474.00	kips	Tension
206	474.00	kips	Tension
207	455.04	kips	Tension
208	398.16	kips	Tension
209	303.36	kips	Tension
210	170.64	kips	Tension



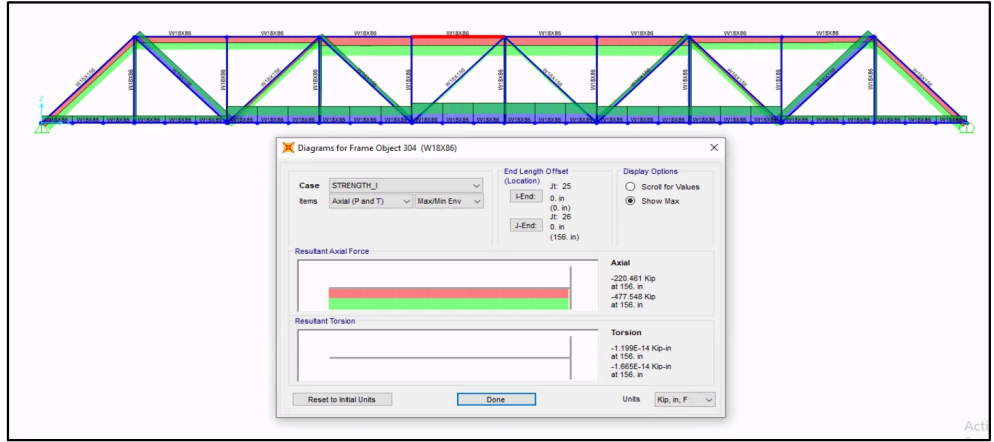
Point Load Distribution
Point Loads

Top Chord member number
Diagonal member number
Bottom Chord member number
vertical member number
ft
ft
ft
ft



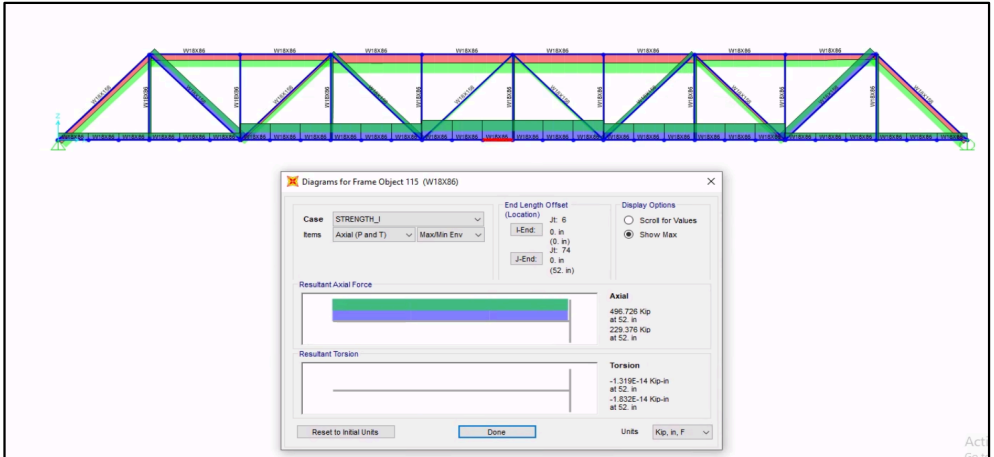
477.5	Load taken from SAP2000 in kips
-455.0	Maximum absolute value for compression
4.9%	Percent difference compared to SAP2000

Most likely that this percent difference is because of the H10 truck moving live load that was not calculated in Excel and all the load was applied to the top chord, where in SAP2000 the weight of the top chord and top half of the diagonal members will be attached to the top chord and everything else will be attached to the bottom chord.



496.7	Load taken from SAP2000 in kips
474.0	Maximum absolute value for tension
4.8%	Percent difference compared to SAP2000

Most likely that this percent difference is because of the H10 truck moving live load that was not calculated in Excel and all the load was applied to the top chord, where in SAP2000 the weight of the top chord and top half of the diagonal members will be attached to the top chord and everything else will be attached to the bottom chord.



D.4 Truss Analysis I-Beam Selection

Member	Top Chord	Bottom Chord	Diagonal Vertical	Diagonal Vertical	Vertical Straight	Vertical Straight	Vertical Straight	Horizontal Perpendicular	Horizontal Diagonal
Section Shape:	W18X36	W18X36	W18X158	W18X158	W18X36	W18X36	W18X36	W12X53	W12X87
Slenderness Check for Flanges	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender
Slenderness Check for Web	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender	Non-Slender
Compaction Check for Web min	Non-Compact	Non-Compact	Compact	Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact
Compaction Check for Web max	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact
Compaction Check for Flanges min	Compact	Compact	Compact	Compact	Compact	Compact	Compact	Compact	Compact
Compaction Check for Flanges max	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact	Non-Compact
Load Type	C	T	C	T	C	T	T	T	T
Length (ft)	13	13	18.385	18.385	13	13	14.5	20.506	
Point Load (kips)	547.960	571.374	292.392	227.285	7.809	40.695	5.042	7.642	
Moment 3-3 (kip-ft)	36.193	58.8239	5.8835	5.8835	0	19.7744	23.6666		
Shear 2-2 (kip)	2.96	14.54	1.28	1.28	0	2.548	1.916		
Spacing between girders "s" (ft)	13	13	13	13	13	13	13	13	
K_x	1.49	1.49	1.49	1.49	1.49	1.49	1.49	1.49	
K_y	1.49	1.49	1.49	1.49	1.49	1.49	1.49	1.49	
L_x	9.29	9.29	9.68	9.68	9.29	9.29	8.76	10.84	
L_y	28.57	28.57	42.83	42.83	28.57	28.57	28.21	43.05	
$KLx \leq 120$	29.92	29.92	40.48	40.48	29.92	29.92	49.57	68.15	
$KLx \leq 120$	88.38	88.38	119.97	119.97	88.38	88.38	104.54	119.43	
Design Check	Demand over Capacity Ratio								
Available Compressive Strength $\phi_c P_n$	0.85	0.88	0.53	0.42	0.01	0.06	0.02	0.02	0.02
Verify limit $0.017147 > 0.003$	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007
Available Flexural Strength $\phi_b M_{nc}$	0.06	0.09	0.00	0.00	0.00	0.00	0.08	0.05	0.05
Available Strength in Tensile Yielding $\phi_t P_n$	0.48	0.50	0.14	0.11	0.01	0.04	0.01	0.01	0.01
Available Strength in Tensile Rupture $\phi_t P_n$	0.58	0.60	0.17	0.13	0.01	0.04	0.01	0.01	0.01
Available Strength in Shear $\phi_v V_n$	0.01	0.05	0.00	0.00	0.00	0.00	0.02	0.01	0.01
Available Strength in Flexure about Y-Y axis $\phi_b M_{nx}$	0.20	0.32	0.02	0.02	0.00	0.00	0.18	0.10	0.10

From SAP2000 Analysis
Based on our design
From SAP2000 Analysis
From SAP2000 Analysis
From SAP2000 Analysis
Based on our design
AAASHTO LRFD, Article 6.9.3
AAASHTO LRFD, Article 6.9.3
AAASHTO LRFD, Article (6.10.1.6) (6.10.10.4.2) (6.12.2.2.5)
AAASHTO LRFD, Article (6.7.4.2) (6.12.2.2.5)
AAASHTO LRFD, Eq 6.9.3
AAASHTO LRFD, Eq 6.9.3

AAASHTO LRFD, Eq 6.9.2.1-1
AAASHTO LRFD, APPENDIX B3, page 3-169
AAASHTO LRFD, Article 6.10.6.1.1 or 6.12
AAASHTO LRFD, Eq 6.8.2.1-1
AAASHTO LRFD, Eq 6.8.2.1-1
AAASHTO LRFD, Eq 6.10.3.3-1
AAASHTO LRFD, Eq 6.10.3.2, Article 6.12

DEFECTION SPECIFICATION, AAASHTO, ARTICLE 5b	
Maximum pedestrian LL Deflection = 1/360 of the span length	
Length of truss	130 ft
Maximum pedestrian LL Deflection	4.33 in
Deflection from Truss Analysis Due to Live Load	0.6132 in
Deflection from Truss Analysis Due to Full factored Load	2.4377 in
VIBRATIONS SPECIFICATION, AAASHTO, ARTICLE 6i	
Vertical Direction	
g	32.2 ft/s ²
maximum vertical deflection of the truss due to the dead load Δ_{Dv} (ft)	1.0917 in
maximum vertical deflection of the truss due to the dead load Δ_{Dv} (ft)	0.0910 in
f	3.3864 Hz

Based on our design
From SAP2000 Analysis
From SAP2000 Analysis
acceleration due to gravity (ft/s²)
From SAP2000 Analysis
From SAP2000 Analysis
The fundamental frequency in a vertical mode without consideration of live load should be greater than 3.0 Hz to avoid the first harmonic.

TOP CHORD COMPRESSIVE RESISTANCE (AASHTO LRFD, ARTICLE 6.9.2):			
Section Shape:	W18X86	I-beam	
Z_x	186	in ³	
Z_y	48.4	in ³	
E	29000	ksi	
F_y	50	ksi	
F_u	65	ksi	
Slenderness Check			
b_f/t_f	11.1	in	
t_f	0.77	in	
$(b_f)/(2t_f)$	7.21	-	AASHTO LRFD, Eq 6.10.8.2.2-3
$\lambda_c = 0.56\sqrt{N/(E F_y)}$	13.49	-	AASHTO LRFD, Eq 6.10.8.2.2-5
b_f/t_f	33.40	-	
$\lambda_c = 1.49\sqrt{N/(E F_y)}$	35.88	-	
Slenderness Check for Flanges	Non-Slender	Good	
Slenderness Check for Web	Non-Slender	Good	
Compaction Check			
b_f/t_f	14.42	-	
b_f	11.1	in	
t_w	0.48	in	
b_w/t_w	23.13	-	
$\lambda_p = 0.38\sqrt{N/(E F_y)}$	9.15	-	AASHTO LRFD, Eq AISC Design Manual 16.1.17-19
$\lambda_p = 1.0\sqrt{N/(E F_y)}$	24.08	-	AASHTO LRFD, Eq AISC Design Manual 16.1.17-19
$\lambda_p = 3.76\sqrt{N/(E F_y)}$	90.55	-	AASHTO LRFD, Eq AISC Design Manual 16.1.17-19
$\lambda_p = 5.70\sqrt{N/(E F_y)}$	137.27	-	AASHTO LRFD, Eq AISC Design Manual 16.1.17-19
Compaction Check for web min λ_p	Non-Compact	Good	
Compaction Check for web max λ_c	Non-Compact	Good	
Compaction Check for flanges min λ_p	Compact	OK	
Compaction Check for flanges max λ_c	Non-Compact	Good	
A_g	25.3	in ²	
d	18.4	in	
t_w	0.48	in	
r_x	7.77	in	
r_y	2.63	in	
r_u	3.05	in	
J_x	4.1	in ⁴	
S_x	166	in ³	
h_o	17.6	in	
K_1	1.49	-	Plate buckling coefficient as specified in Table 6.9.4.2.1-1
K_2	1.49	-	Plate buckling coefficient as specified in Table 6.9.4.2.1-1
L_p	9.29	ft	Limiting Unbraced Lengths, ft
L_r	28.6	ft	Limiting Unbraced Lengths, ft
L	156	in	
$KL/r_x < 120$	29.92	Good	Slenderness ratio AASHTO LRFD, Eq 6.9.3 6.8.4—Limiting Slenderness Ratio
$KL/r_y < 120$	88.38	Good	Slenderness ratio AASHTO LRFD, Eq 6.9.3 6.8.4—Limiting Slenderness Ratio
KL/r_x	88.38	-	
r_x	2.63	in	
$\lambda = (KL/r_x)\sqrt{N/(E)}$	1.36	-	AASHTO LRFD, Eq 6.9.5.1-3
If $\lambda < 2.25$, then			
$P_n = 0.66 F_y A_g$	717.55	kips	AASHTO LRFD, Eq 6.9.5.1-1
If $\lambda > 2.25$, then			
$P_n = (0.66 F_y A_g) / \lambda^2$	611.86	kips	AASHTO LRFD, Eq 6.9.5.1-2
ϕ_c	0.9	-	AASHTO LRFD, Eq 6.5.4.2
P_n	717.55	kips	Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)
$P_r = \phi_c P_n$	645.80	kips	AASHTO LRFD, Eq 6.9.2.1-1
D/C	0.85	Good	Demand / Capacity for compression
M_n	8370	kip-in	nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
M_{cr}	136710029.4	kip-in	critical elastic lateral-torsional buckling moment of one girder (kip-in.)
M_{cr}	11392502.45	kip-in	critical elastic lateral-torsional buckling moment of one girder (kip-ft)
s	13	ft	spacing between girders (ft)
s	156	in	spacing between girders (in.)
L	13	ft	effective buckling length for lateral-torsional buckling (ft)
I_{po}	175	in ⁴	out-of-plane moment of inertia of one girder (in ⁴)
I_{po}	1530	in ⁴	in-of-plane moment of inertia of one girder (in ⁴)
$\phi_b M_{pn}$	697.5	kip-ft	in-plane plastic moment of one girder (kip-ft)
$\phi_b M_{pn}$	8370	kip-in	in-plane plastic moment of one girder (kip-in.)
K	1.49	-	
P_{cr}	548	kips	
Verify limit $0.01/1.49 > 0.003$	0.007	OK	AASHTO LRFD, APPENDIX B3, page 3-169
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1.1):			
H_f	3.68	kip	
Length of vertical	1.56	in	
Lateral Moment in Vertical	573.70	kip-in	
Lateral Moment in Vertical	47.81	kip-ft	
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code			
Section Shape:	W18X86	I-beam	
Gross Area, A_g	25.3	in ²	
$A_n = 0.75 A_g$	18.98	in ²	
ϕ_t	0.90	-	AASHTO LRFD, Eq 6.5.4.2
ϕ_t	0.75	-	AASHTO LRFD, Eq 6.5.4.2
Yielding $\phi_t P_n$	1138.5	kips	AASHTO LRFD, Eq 6.8.2.1-1
Rupture $\phi_t P_n$	948.75	kips	AASHTO LRFD, Eq 6.8.2.1-1
Minimum	948.75	kips	
Design Check D/C Yielding	0.48	Good	
Design Check D/C Rupture	0.58	Good	
Available Flexural Strength $\phi_b M_{nx}$			
Section Shape:	W18X86	I-beam	
M_n	484.17	kip-ft	factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12
$\phi_b M_{nx}$	435.75	kip-ft	factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.13
M_p	775.00	kip-ft	AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2
$\phi_b M_{nx}$	697.50	kip-ft	AASHTO LRFD, Eq 6.10.7.1.1
M_{px}	719.04	kip-ft	Interpolation between M_n and M_{px}
$\phi_b M_{nx}$	647.13	kip-ft	Interpolation between M_n and M_{pn}
Design Check D/C	0.06	Good	
Available Flexural Strength $\phi_b M_{ny}$			
Section Shape:	W18X86	I-beam	
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft	AASHTO LRFD, Eq 6.10.3.2, Article 6.12
Design Check D/C	0.20	Good	
Available Strength in Shear $\phi_v V_n$			
Section Shape:	W18X86	I-beam	
Available Strength in Shear $V_r = \phi_v V_n$	264.96	kips	AASHTO LRFD, Eq 6.10.3.3-1
Design Check D/C	0.01	Good	
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces AISC (page 6-)			
Section Shape:	W18X86	I-beam	Design Check
Available Compressive Strength $\phi_c P_n$	645.80	kips	Good
Available Flexural Strength $\phi_b M_{nx}$	647.13	kip-ft	Good
Available Strength in Tensile Yielding $\phi_t P_n$	1138.5	kips	N/A
Available Strength in Tensile Rupture $\phi_t P_n$	948.75	kips	N/A
Available Strength in Shear $\phi_v V_n$	264.96	kips	Good
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft	Good

Bottom Chord Member, Tension Resistance		
Section Shape:	W18X86	I-beam
Z_x	186	in ³
Z_y	48.4	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	11.1	in
t_f	0.77	in
$(b_f)(2t_f)$	7.21	-
$0.56*(F_y)$	13.49	-
h/t_w	33.40	-
$\lambda_c = 1.49*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	14.42	-
b_f	11.1	in
t_w	0.48	in
b_w/λ_w	23.13	-
$\lambda_c = 0.38*(E/F_y)$	9.15	-
$\lambda_c = 1.0*(E/F_y)$	24.08	-
$\lambda_c = 3.76*(E/F_y)$	90.55	-
$\lambda_c = 5.70*(E/F_y)$	137.27	-
Compaction Check for web min	Non-Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_x	25.3	in ²
d	18.4	in
t_w	0.48	in
r_x	7.77	in
r_y	2.63	in
r_u	3.05	in
J_x	4.1	in ⁴
S_x	166	in ³
h_u	17.6	in
K_x	1.49	-
K_y	1.49	-
L_p	9.29	ft
L_r	28.6	ft
L	156	in
$KL/r_x < 120$	29.92	Good
$KL/r_y < 120$	88.38	Good
KL/r_x	88.38	-
r_x	2.63	in
$\lambda = (KL/r_x)^2*(F_y/E)$	1.36	-
If $\lambda < 2.25$, then		
$P_n = 0.66*F_u*A_x$	717.55	kips
If $\lambda > 2.25$, then		
$P_n = (0.66F_u A_x)(\lambda)$	611.86	kips
ϕ_c	0.9	-
P_n	717.55	kips
$P_n = \phi_c P_n$	645.80	kips
D/C	0.88	Good
D/C	0.88	Good
M_n	8370	kip-in
M_{cr}	136710029.4	kip-in
M_{cr}	11392502.45	kip-in
s	13	ft
L	156	in
L	13	ft
I_{op}	175	in ⁴
I_{op}	1530	in ⁴
$\phi_b M_{pn}$	697.5	kip-ft
$\phi_b M_{pn}$	8370	kip-in
K	1.49	-
P_{cr}	571.37	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1.1.1):		
H _v	3.83	kip
Length of vertical	156	in
Lateral Moment in Vertical	598.22	kip-in
Lateral Moment in Vertical	49.85	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W18X86	I-beam
Gross Area, A_g	25.3	in ²
$A_n = 0.75A_g$	18.98	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	1138.5	kips
Rupture $\phi_t P_n$	948.75	kips
Minimum	948.75	kips
Design Check D/C Yielding	0.50	Good
Design Check D/C Rupture	0.60	Good
Available Flexural Strength $\phi_b M_{nx}$		
Section Shape:	W18X86	I-beam
M_{nx}	484.17	kip-ft
$\phi_b M_{nx}$	435.75	kip-ft
M_p	775.00	kip-ft
$\phi_b M_p$	697.50	kip-ft
M_{nx}	719.04	kip-ft
$\phi_b M_{nx}$	647.13	kip-ft
Design Check D/C	0.09	Good
Available Flexural Strength $\phi_b M_{ny}$		
Section Shape:	W18X86	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft
Design Check D/C	0.32	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W18X86	I-beam
Available Strength in Shear $\phi_v V_n$	264.96	kips
Design Check D/C	0.05	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces AISC (page 6-89)		
Section Shape:	W18X86	I-beam
Available Compressive Strength $\phi_c P_n$	645.80	kips
Available Flexural Strength $\phi_b M_{nx}$	647.13	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	1138.5	kips
Available Strength in Tensile Rupture $\phi_t P_n$	948.75	kips
Available Strength in Shear $\phi_v V_n$	264.96	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-
Plate buckling coefficient as specified in Table 6.9.4.2.1-

Limiting Unbraced Lengths, ft
Limiting Unbraced Lengths, ft

AASHTO LRFD, Eq 6.9.4.1-3

AASHTO LRFD, Eq 6.9.4.1-2

Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression
nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
critical elastic lateral-torsional buckling moment of one girder (kip-in.)
critical elastic lateral-torsional buckling moment of one girder (kip-ft)
spacing between girders (ft)
spacing between girders (in.)
effective buckling length for lateral-torsional buckling (ft)
out-of-plane moment of inertia of one girder (in.⁴)
in-of-plane moment of inertia of one girder (in.⁴)
in-plane plastic moment of one girder (kip-ft)
in-plane plastic moment of one girder (kip-in.)

AASHTO LRFD, APPENDIX B3, page 3-169

factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12
factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.13
AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2
AASHTO LRFD, Eq 6.10.7.1.1
Interpolation between M_{nx} and M_{pn}
Interpolation between M_{nx} and M_{pn}

AASHTO LRFD, Eq 6.10.3.2, Article 6.12

AASHTO LRFD, Eq 6.10.3.3-1

Design Check	
Available Compressive Strength $\phi_c P_n$	N/A
Available Flexural Strength $\phi_b M_{nx}$	Good
Available Strength in Tensile Yielding $\phi_t P_n$	Good
Available Strength in Tensile Rupture $\phi_t P_n$	Good
Available Strength in Shear $\phi_v V_n$	Good
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	Good

Diagonal Member, Compression Resistance		
Section Shape:	W18x158	I-beam
Z_x	356	in ³
Z_y	94.8	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	11.3	in
t_f	1.44	in
$(b_f/2t_f)$	3.92	-
$0.56^*(N/E/F_y)$	13.49	-
h/t_w	19.80	-
$\lambda_c = 1.49^*(N/E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	7.85	-
b_f	11.3	in
t_w	0.81	in
b_w/t_w	13.95	-
$\lambda_{c1} = 0.38^*(N/E/F_y)$	9.15	-
$\lambda_{c2} = 1.0^*(N/E/F_y)$	24.08	-
$\lambda_{c3} = 3.76^*(N/E/F_y)$	90.55	-
$\lambda_{c4} = 5.70^*(N/E/F_y)$	137.27	-
Compaction Check for web min	Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_x	46.3	in ²
d	19.7	in
t_w	0.81	in
r_x	8.12	in
r_y	2.74	in
r_w	3.2	in
J_x	25.2	in ⁴
S_x	310	in ³
h_o	18.3	in
K_x	1.49	-
K_y	1.49	-
L_p	9.68	ft
L_r	42.8	ft
L	220.62	in
$KL/r_x < 120$	40.48	Good
$KL/r_y < 120$	119.97	Good
KL/r_x	119.97	-
r_x	2.74	in
$\lambda = (KL/r_x)^2 * (F_y/E)$	2.51	-
$P_n = 0.66^*F_uA_x$ If $\lambda < 2.25$, then	814.37	kips
$P_n = (0.66F_uA_x)/\lambda$ If $\lambda > 2.25$, then	607.68	kips
ϕ_c	0.9	-
P_n	607.68	kips
$P_n = \phi_c P_n$	546.91	kips
D/C	0.53	Good
M_n	16020	kip-in
M_{cr}	136122868.4	kip-in
M_{cr}	11343572.37	kip-in
s	13	ft
L_b	156	in
L	18.38	ft
I_{op}	347	in ⁴
I_{op}	3060	in ⁴
$\phi_b M_{pn}$	1335	kip-ft
$\phi_b M_{pn}$	16020	kip-in
K	1.49	-
$P_{0.1}$	292	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1.1):		
H_f	1.96	kip
Length of vertical	156	in
Lateral Moment in Vertical	306.13	kip-in
Lateral Moment in Vertical	25.51	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W18x158	I-beam
Gross Area, A_g	46.3	in ²
$A_n = 0.75A_g$	34.73	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	2083.5	kips
Rupture $\phi_t P_n$	1736.25	kips
Minimum	1736.25	kips
Design Check D/C Yielding	0.14	Good
Design Check D/C Rupture	0.17	Good
Available Flexural Strength $\phi_b M_n$		
Section Shape:	W18x158	I-beam
M_n	904.17	kip-ft
$\phi_b M_n$	813.75	kip-ft
M_p	1483.33	kip-ft
$\phi_b M_p$	1335.00	kip-ft
$M_{1.5}$	1425.30	kip-ft
$\phi_b M_{1.5}$	1198.10	kip-ft
Design Check D/C	0.00	Good
Available Flexural Strength $\phi_b M_{1.5}$		
Section Shape:	W18x158	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{1.5}$	355.5	kip-ft
Design Check D/C	0.02	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W18x158	I-beam
Available Strength in Shear $\phi_v V_n$	478.71	kips
Design Check D/C	0.00	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces AISC (page 6-)		
Section Shape:	W18x158	I-beam
Available Compressive Strength $\phi_c P_n$	546.91	kips
Available Flexural Strength $\phi_b M_{1.5}$	1198.10	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	2083.5	kips
Available Strength in Tensile Rupture $\phi_t P_n$	1736.25	kips
Available Strength in Shear $\phi_v V_n$	478.71	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{1.5}$	355.5	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-1

Plate buckling coefficient as specified in Table 6.9.4.2.1-1

Limiting Unbraced Lengths, ft

Limiting Unbraced Lengths, ft

Slenderness ratio AASHTO LRFD, Eq 6.9.3

Slenderness ratio AASHTO LRFD, Eq 6.9.3

AASHTO LRFD, Eq 6.9.4.1-3

AASHTO LRFD, Eq 6.9.4.1-2

Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression

nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)

critical elastic lateral-torsional buckling moment of one girder (kip-in.)

critical elastic lateral-torsional buckling moment of one girder (kip-ft.)

spacing between girders (ft)

spacing between girders (in.)

effective buckling length for lateral-torsional buckling (ft)

out-of-plane moment of inertia of one girder (in.⁴)

in-of-plane moment of inertia of one girder (in.⁴)

in-plane plastic moment of one girder (kip-ft.)

in-plane plastic moment of one girder (kip-in.)

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

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AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

AASHTO LRFD, APPENDIX B3, page 3-169

Diagonal Member, Tension Resistance		
Section Shape:	W18x158	I-beam
Z_x	356	in ³
Z_y	94.8	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	11.3	in
t_f	1.44	in
$(b_f/2t_f)$	3.92	-
$0.56^*(E/F_y)$	13.49	-
h/t_w	19.80	-
$\lambda_c = 1.49^*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	7.85	-
b_f	11.3	in
t_w	0.81	in
b_w/t_w	13.95	-
$\lambda_{c1} = 0.38^*(E/F_c)$	9.15	-
$\lambda_{c2} = 1.0^*(E/F_c)$	24.08	-
$\lambda_{c3} = 3.76^*(E/F_c)$	90.55	-
$\lambda_{c4} = 5.70^*(E/F_c)$	137.27	-
Compaction Check for web min	Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_s	46.3	in ²
d	19.7	in
t_w	0.81	in
r_x	8.12	in
r_y	2.74	in
r_w	3.2	in
J_x	25.2	in ⁴
S_x	310	in ³
h_o	18.3	in
K_x	1.49	-
K_y	1.49	-
L_p	9.68	ft
L_r	42.8	ft
L	220.62	in
$KL/r_x < 120$	40.48	Good
$KL/r_y < 120$	119.97	Good
KL/r_x	119.97	-
r_x	2.74	in
$\lambda = (KL/r_x)^2 * (F_y/E)$	2.51	-
$P_n = 0.66^*F_uA_s$ If $\lambda < 2.25$, then	814.37	kips
$P_n = (0.66F_uA_s)/\lambda$ If $\lambda > 2.25$, then	607.68	kips
ϕ_c	0.9	-
P_n	607.68	kips
$P_n = \phi_c P_n$	546.91	kips
D/C	0.42	Good
M_n	16020	kip-in
M_{cr}	136122868.4	kip-in
M_{cr}	11343572.37	kip-in
s	13	ft
L_s	156	in
L	18.38	ft
I_{op}	347	in ⁴
I_{op}	3060	in ⁴
$\phi_b M_{pn}$	1335	kip-ft
$\phi_b M_{pn}$	16020	kip-in
K	1.49	-
$P_{0.1}$	227	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1.1):		
H_f	1.53	kip
Length of vertical	156	in
Lateral Moment in Vertical	237.96	kip-in
Lateral Moment in Vertical	19.83	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W18x158	I-beam
Gross Area, A_g	46.3	in ²
$A_n = 0.75A_g$	34.73	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	2083.5	kips
Rupture $\phi_t P_n$	1736.25	kips
Minimum	1736.25	kips
Design Check D/C Yielding	0.11	Good
Design Check D/C Rupture	0.13	Good
Available Flexural Strength $\phi_b M_n$		
Section Shape:	W18x158	I-beam
M_n	904.17	kip-ft
$\phi_b M_n$	813.75	kip-ft
M_p	1483.33	kip-ft
$\phi_b M_p$	1335.00	kip-ft
$M_{1.5}$	1425.30	kip-ft
$\phi_b M_{1.5}$	1198.10	kip-ft
Design Check D/C	0.00	Good
Available Flexural Strength $\phi_b M_{1.5}$		
Section Shape:	W18x158	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{1.5}$	355.5	kip-ft
Design Check D/C	0.02	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W18x158	I-beam
Available Strength in Shear $\phi_v V_n$	478.71	kips
Design Check D/C	0.00	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces AISC (page 6-)		
Section Shape:	W18x158	I-beam
Available Compressive Strength $\phi_c P_n$	546.91	kips
Available Flexural Strength $\phi_b M_{1.5}$	1198.10	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	2083.5	kips
Available Strength in Tensile Rupture $\phi_t P_n$	1736.25	kips
Available Strength in Shear $\phi_v V_n$	478.71	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{1.5}$	355.5	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Limiting Unbraced Lengths, ft
Limiting Unbraced Lengths, ft

Slenderness ratio AASHTO LRFD, Eq 6.9.3
Slenderness ratio AASHTO LRFD, Eq 6.9.3

AASHTO LRFD, Eq 6.9.4.1-3

AASHTO LRFD, Eq 6.9.4.1-2

Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression
nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
critical elastic lateral-torsional buckling moment of one girder (kip-in.)
critical elastic lateral-torsional buckling moment of one girder (kip-ft.)
spacing between girders (ft)
spacing between girders (in.)
effective buckling length for lateral-torsional buckling (ft)
out-of-plane moment of inertia of one girder (in.⁴)
in-of-plane moment of inertia of one girder (in.⁴)
in-plane plastic moment of one girder (kip-ft.)
in-plane plastic moment of one girder (kip-in.)

kip

in

kip-in

kip-ft

AASHTO LRFD, Eq 6.5.4.2

AASHTO LRFD, Eq 6.5.4.2

AASHTO LRFD, Eq 6.8.2.1-1

AASHTO LRFD, Eq 6.8.2.1-1

factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12

factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12

AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2

AASHTO LRFD, Eq 6.10.7.1.1

Interpolation between $M_{1.5}$ and $M_{1.5}$

Interpolation between $M_{1.5}$ and $M_{1.5}$

AASHTO LRFD, Eq 6.10.3.2, Article 6.12

AASHTO LRFD, Eq 6.10.3.3-1

Design Check

N/A

Good

Good

Good

Good

Vertical Member, Compression Resistance		
Section Shape:	W18x86	I-beam
Z_x	186	in ³
Z_y	48.4	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	11.1	in
t_f	0.77	in
$(b_f/2t_f)$	7.21	-
$0.56^*(E/F_y)$	13.49	-
h/t_w	33.40	-
$\lambda_c = 1.49^*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	14.42	-
b_f	11.1	in
t_w	0.48	in
h_u/t_w	23.13	-
$\lambda_p = 0.38^*(E/F_c)$	9.15	-
$\lambda_i = 1.0^*(E/F_c)$	24.08	-
$\lambda_o = 3.76^*(E/F_c)$	90.55	-
$\lambda_s = 5.70^*(E/F_c)$	137.27	-
Compaction Check for web min	Non-Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_s	25.3	in ²
d	18.4	in
t_w	0.48	in
r_x	7.77	in
r_y	2.63	in
r_u	3.05	in
J_x	4.1	in ⁴
S_x	166	in ³
h_o	17.6	in
K_x	1.49	-
K_y	1.49	-
L_p	9.29	ft
L_r	28.6	ft
L	156	in
$KL/r_x < 120$	29.92	Good
$KL/r_y < 120$	88.38	Good
KL/r_x	88.38	-
r_x	2.63	in
$\lambda = (KL/r_x)^2 * (F_y/E)$	1.36	-
$P_n = 0.66^*F_uA_s$ If $\lambda < 2.25$, then	717.55	kips
$P_n = (0.66F_uA_s)/(\lambda)$ If $\lambda > 2.25$, then	611.86	kips
ϕ_c	0.9	-
P_n	717.55	kips
$P_c = \phi_c P_n$	645.80	kips
D/C	0.01	Good
M_n	8370	kip-in
M_{cr}	136710029.43	kip-in
M_{cr}	11392502.45	kip-in
s	13	ft
L	156	in
L_p	175	in ²
L_{p0}	1330	in ²
$\phi_b M_{px}$	697.5	kip-ft
$\phi_b M_{py}$	8370	kip-in
K	1.49	-
$P_{0.1}$	7.81	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1.1):		
H_f	0.05	kip
Length of vertical	156	in
Lateral Moment in Vertical	8.18	kip-in
Lateral Moment in Vertical	0.68	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W18x86	I-beam
Gross Area, A_g	25.3	in ²
$A_n = 0.75A_g$	18.98	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	1138.5	kips
Rupture $\phi_t P_n$	948.75	kips
Minimum	948.75	kips
Design Check D/C Yielding	0.01	Good
Design Check D/C Rupture	0.01	Good
Available Flexural Strength $\phi_b M_n$		
Section Shape:	W18x86	I-beam
M_n	484.17	kip-ft
$\phi_b M_n$	435.75	kip-ft
M_p	775.00	kip-ft
$\phi_b M_p$	697.50	kip-ft
M_{px}	719.04	kip-ft
$\phi_b M_{px}$	647.13	kip-ft
Design Check D/C	0.00	Good
Available Flexural Strength $\phi_b M_{ny}$		
Section Shape:	W18x86	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft
Design Check D/C	0.00	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W18x86	I-beam
Available Strength in Shear $\phi_v V_n$	264.96	kips
Design Check D/C	0.00	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces AISC (page 6-		
Section Shape:	W18x86	I-beam
Available Compressive Strength $\phi_c P_n$	645.80	kips
Available Flexural Strength $\phi_b M_{nx}$	647.13	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	1138.5	kips
Available Strength in Tensile Rupture $\phi_t P_n$	948.75	kips
Available Strength in Shear $\phi_v V_n$	264.96	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Limiting Unbraced Lengths, ft
Limiting Unbraced Lengths, ft
Slenderness ratio AASHTO LRFD, Eq 6.9.3
Slenderness ratio AASHTO LRFD, Eq 6.9.3

AASHTO LRFD, Eq 6.9.4.1-3

Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression
nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
critical elastic lateral-torsional buckling moment of one girder (kip-in.)
critical elastic lateral-torsional buckling moment of one girder (kip-ft)
spacing between girders (ft)
spacing between girders (in.)
effective buckling length for lateral-torsional buckling (ft)
out-of-plane moment of inertia of one girder (in.⁴)
in-of-plane moment of inertia of one girder (in.⁴)
in-plane plastic moment of one girder (kip-ft.)
in-plane plastic moment of one girder (kip-in.)

factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12
factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.13
AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2
AASHTO LRFD, Eq 6.10.7.1.1
Interpolation between M_{nx} and M_{px}
Interpolation between M_{ny} and M_{py}

AASHTO LRFD, Eq 6.10.3.2, Article 6.12

AASHTO LRFD, Eq 6.10.3.3-1

Vertical Member, Tension Resistance		
Section Shape:	W18x86	I-beam
Z_x	186	in ³
Z_y	48.4	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	11.1	in
t_f	0.77	in
$(b_f)(2t_f)$	7.21	-
$0.56*(E/F_y)$	13.49	-
h/t_w	33.40	-
$\lambda_c = 1.49*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	14.42	-
b_f	11.1	in
t_w	0.48	in
h_w/t_w	23.13	-
$\lambda_p = 0.38*(E/F_y)$	9.15	-
$\lambda_r = 1.0*(E/F_y)$	24.08	-
$\lambda_o = 3.76*(E/F_y)$	90.55	-
$\lambda_s = 5.70*(E/F_y)$	137.27	-
Compaction Check for web min	Non-Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_g	25.3	in ²
d	18.4	in
t_w	0.48	in
r_x	7.77	in
r_y	2.63	in
r_u	3.05	in
J_x	4.1	in ⁴
S_x	166	in ³
h_s	17.6	in
K_x	1.49	-
K_y	1.49	-
L_p	9.29	ft
L_r	28.6	ft
L	156	in
$KL/r_x < 120$	29.92	Good
$KL/r_y < 120$	88.38	Good
KL/r_x	88.38	-
r_x	2.63	in
$\lambda = (KL/r_x)^2(E/E)$	1.36	-
$P_n = 0.66F_u A_g$ If $\lambda > 2.25$, then	717.55	kips
$P_n = 0.66F_y A_g(\lambda)$ If $\lambda > 2.25$, then	611.86	kips
ϕ_c	0.9	-
P_n	717.55	kips
$P_n = \phi_c P_n$	645.80	kips
D/C	0.06	Good
M_n	8370	kip-in
M_{cr}	136710029.43	kip-in
M_{cr}	11392502.45	kip-in
s	13	ft
s	156	in
L_b	13	ft
I_{yy}	175	in ⁴
I_{zz}	1530	in ⁴
$\phi_b M_{px}$	697.5	kip-ft
$\phi_b M_{py}$	8370	kip-in
K	1.49	-
P_{eff}	40.70	kip
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1)		
H_f	0.27	kip
Length of vertical	156	in
Lateral Moment in Vertical	42.61	kip-in
Lateral Moment in Vertical	3.55	kip-ft
Available Tensile Strength in Axial Tension From AISC		
Section Shape:	W18x86	I-beam
Gross Area, A_g	25.3	in ²
$A_n = 0.75A_g$	18.98	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	1138.5	kips
Rupture $\phi_t P_n$	948.75	kips
Minimum	948.75	kips
Design Check D/C Yielding	0.04	Good
Design Check D/C Rupture	0.04	Good
Available Flexural Strength $\phi_b M_{nx}$		
Section Shape:	W18x86	I-beam
M_{nx}	484.17	kip-ft
$\phi_b M_{nx}$	455.75	kip-ft
M_{py}	775.00	kip-ft
$\phi_b M_{py}$	697.50	kip-ft
M_{nx}	719.04	kip-ft
$\phi_b M_{nx}$	647.13	kip-ft
Design Check D/C	0.00	Good
Available Flexural Strength $\phi_b M_{ny}$		
Section Shape:	W18x86	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft
Design Check D/C	0.00	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W18x86	I-beam
Available Strength in Shear $\phi_v V_n$	264.96	kips
Design Check D/C	0.00	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces		
Section Shape:	W18x86	I-beam
Available Compressive Strength $\phi_c P_n$	645.80	kips
Available Flexural Strength $\phi_b M_{nx}$	647.13	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	1138.5	kips
Available Strength in Tensile Rupture $\phi_t P_n$	948.75	kips
Available Strength in Shear $\phi_v V_n$	264.96	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	181.5	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Limiting Unbraced Lengths, ft
Limiting Unbraced Lengths, ft

Slenderness ratio AASHTO LRFD, Eq 6.9.3
Slenderness ratio AASHTO LRFD, Eq 6.9.3

AASHTO LRFD, Eq 6.9.4.1-3

AASHTO LRFD, Eq 6.9.4.1-2

Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression
nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
critical elastic lateral-torsional buckling moment of one girder (kip-in.)
critical elastic lateral-torsional buckling moment of one girder (kip-ft)
spacing between girders (ft)
spacing between girders (in.)
effective buckling length for lateral-torsional buckling (ft)
out-of-plane moment of inertia of one girder (in.⁴)
in-of-plane moment of inertia of one girder (in.⁴)
in-plane plastic moment of one girder (kip-ft.)
in-plane plastic moment of one girder (kip-in.)

Factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12
Factored flexural resistance equal to ϕ_b times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.13
AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2
AASHTO LRFD, Eq 6.10.7.1.1
Interpolation between M_{nx} and M_{px}
Interpolation between M_{nx} and M_{px}

AASHTO LRFD, Eq 6.10.3.2, Article 6.12

AASHTO LRFD, Eq 6.10.3.3-1

Horizontal Perpendicular Member, Tension Resistance		
Section Shape:	W12x53	I-beam
Z_x	77.9	in ³
Z_y	29.1	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	10	in
t_f	0.575	in
$(b_f/2t_f)$	8.70	-
$0.56*(E/F_u)$	13.49	-
h/t_w	28.10	-
$\lambda = 1.49*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	17.39	-
b_f	10.0	in
t_w	0.35	in
h_w/t_w	28.99	-
$\lambda_x = 0.38*(E/F_u)$	9.15	-
$\lambda_y = 1.0*(E/F_u)$	24.08	-
$\lambda_z = 3.76*(E/F_u)$	90.55	-
$\lambda = 5.70*(E/F_u)$	137.27	-
Compaction Check for web min	Non-Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_x	15.6	in ²
d	12.1	in
t_w	0.345	in
r_x	5.23	in
r_y	2.48	in
r_z	2.79	in
J_x	1.58	in ⁴
S_x	70.6	in ³
h_o	11.5	in
K_x	1.49	-
K_y	1.49	-
L_p	8.76	ft
L_r	28.2	ft
L	174	in
$KL/r_x < 120$	49.57	Good
$KL/r_y < 120$	104.54	Good
KL/r_z	104.54	-
r_x	2.48	in
$\lambda = (KL/r_x)*\sqrt{(F_u/E)}$	1.91	-
If $\lambda < 2.25$, then		
$P_n = 0.66*F_u*A_x$	352.84	kips
If $\lambda > 2.25$, then		
$P_n = (0.88*F_u*A_x)/\lambda$	359.53	kips
ϕ_c	0.9	-
P_n	352.84	kips
$P_n > \phi_c P_n$	317.56	kips
D/C	0.02	Good
M_n	3505.5	kip-in
M_{cr}	42851256.08	kip-in
M_{cr}	3570938.01	kip-in
s	13	ft
L	156	in
L	14.5	ft
I_{yy}	95.8	in ⁴
I_{zz}	425	in ⁴
$\phi_b M_{pn}$	292.125	kip-ft
$\phi_b M_{px}$	3505.5	kip-in
K	1.49	-
P_{eff}	5.04	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
GENERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1)		
H	0.03	kip
Length of vertical	156	in
Lateral Moment in Vertical	5.28	kip-in
Lateral Moment in Vertical	0.44	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W12x53	I-beam
Gross Area A_g	15.6	in ²
$A_n = 0.75A_g$	11.70	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	702.0	kips
Rupture $\phi_t P_n$	585	kips
Minimum	585	kips
Design Check D/C Yielding	0.01	Good
Design Check D/C Rupture	0.01	Good
Available Flexural Strength $\phi_b M_n$		
Section Shape:	W12x53	I-beam
M_n	205.92	kip-ft
$\phi_b M_n$	185.33	kip-ft
M_p	324.58	kip-ft
$\phi_b M_p$	292.13	kip-ft
M_{ix}	298.71	kip-ft
$\phi_b M_{ix}$	260.60	kip-ft
Design Check D/C	0.08	Good
Available Flexural Strength $\phi_b M_{oy}$		
Section Shape:	W12x53	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{oy}$	109.125	kip-ft
Design Check D/C	0.18	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W12x53	I-beam
Available Strength in Shear $\phi_v V_n$	125.235	kips
Design Check D/C	0.02	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined		
Section Shape:	W12x53	I-beam
Available Compressive Strength $\phi_c P_n$	317.56	kips
Available Flexural Strength $\phi_b M_n$	260.60	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	702.0	kips
Available Strength in Tensile Rupture $\phi_t P_n$	585	kips
Available Strength in Shear $\phi_v V_n$	125.235	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{oy}$	109.125	kip-ft

AASHTO LRFD, Eq 6.10.8.2.2-3
AASHTO LRFD, Eq 6.10.8.2.2-5

AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19
AASHTO LRFD, Eq 6.9.4.2.2 AISC Design Manual 16.1.17-19

Plate buckling coefficient as specified in Table 6.9.4.2.1-1
Plate buckling coefficient as specified in Table 6.9.4.2.1-1

Limiting Unbraced Lengths, ft
Limiting Unbraced Lengths, ft

Slenderness ratio AASHTO LRFD, Eq 6.9.3
Slenderness ratio AASHTO LRFD, Eq 6.9.3

AASHTO LRFD, Eq 6.9.4.1-3

AASHTO LRFD, Eq 6.9.4.1-2
Nominal compressive resistance per AASHTO LRFD, Article 6.9.4 (kips)

Demand / Capacity for compression
nominal in-plane flexural resistance of one girder (kip-in.) Equation (7.2.2-1)
critical elastic lateral-torsional buckling moment of one girder (kip-in.)
critical elastic lateral-torsional buckling moment of one girder (kip-ft)
spacing between girders (ft)
spacing between girders (in.)
effective buckling length for lateral-torsional buckling (ft)
out-of-plane moment of inertia of one girder (in⁴)
in-of-plane moment of inertia of one girder (in⁴)
in-plane plastic moment of one girder (kip-ft)
in-plane plastic moment of one girder (kip-in.)

factored flexural resistance equal to ϕ times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.12
factored flexural resistance equal to ϕ times the nominal flexural resistance determined, Article 6.10, 6.11 or 6.13
AASHTO LRFD, Eq 6.10.7.1.2-1, Eq 6.10.7.1.2-2
AASHTO LRFD, Eq 6.10.7.1.1
Interpolation between M_{ix} and M_{px}
Interpolation between M_{ix} and M_{px}

AASHTO LRFD, Eq 6.5.4.2
AASHTO LRFD, Eq 6.5.4.2
AASHTO LRFD, Eq 6.8.2.1-1
AASHTO LRFD, Eq 6.8.2.1-1

AASHTO LRFD, Eq 6.10.3.2, Article 6.12

AASHTO LRFD, Eq 6.10.3.3-1

Design Check

Design Check

Design Check

Design Check

Design Check

Design Check

Horizontal Diagonal Member, Tension Resistance		
Section Shape:	W12X87	I-beam
Z_x	132	in ³
Z_y	60.4	in ³
E	29000	ksi
F_y	50	ksi
F_u	65	ksi
b_f	12.1	in
t_f	0.81	in
$(b_f)(2t_f)$	7.47	-
$0.56*(E/F_y)$	13.49	-
b_f/t_f	18.90	-
$\lambda = 1.49*(E/F_y)$	35.88	-
Slenderness Check	Non-Slender	Good
Slenderness Check for Web	Non-Slender	Good
Compaction Check		
b_f/t_f	14.94	-
b_f	12.1	in
t_f	0.52	in
b_f/t_f	23.50	-
$\lambda_c = 0.38*(E/F_y)$	9.15	-
$\lambda_s = 1.0*(E/F_y)$	24.08	-
$\lambda_p = 3.76*(E/F_y)$	90.55	-
$\lambda_r = 5.70*(E/F_y)$	137.27	-
Compaction Check for web min	Non-Compact	Good
Compaction Check for web max	Non-Compact	Good
Compaction Check for flanges min	Compact	OK
Compaction Check for flanges max	Non-Compact	Good
A_x	25.6	in ²
d	12.5	in
t_w	0.515	in
r_x	5.38	in
r_y	3.07	in
r_u	3.46	in
J_x	5.1	in ⁴
S_x	118	in ³
h_o	11.7	in
K_x	1.49	-
K_y	1.49	-
L_p	10.84	ft
L_r	43.1	ft
L	246.0731599	in
$KL/r_x < 120$	68.15	Good
$KL/r_y < 120$	119.43	Good
KL/r_x	119.43	-
r_x	3.07	in
$\lambda = (KL/r_x)^2*(F_y/E)$	2.49	-
AASHTO LRFD, Eq 6.9.4-1-3		
$P_n = 0.66*F_u*A_x$	454.53	kips
AASHTO LRFD, Eq 6.9.4-1-2		
$P_n = (0.88F_u A_x)(\lambda)$	452.06	kips
ϕ_c	0.9	-
P_n	452.06	kips
$P_n = \phi_c P_n$	406.85	kips
D/C	0.02	Good
Demand / Capacity for compression		
M_n	5940	kip-in
M_{cr}	44841561.01	kip-in
M_{cr}	3736796.75	kip-in
s	13	ft
s	1.56	in
L_e	20.506	ft
I_{yy}	241	in ⁴
I_{zz}	740	in ⁴
$\phi_b M_{pn}$	495	kip-ft
$\phi_b M_{pn}$	5940	kip-in
K	1.49	-
P_{eff}	7.64	-
Verify limit $0.01/1.49 > 0.003$	0.007	OK
LATERAL FORCE TO BE RESISTED BY VERTICALS (SPECIFICATION, ARTICLE 7.1)		
H	0.05	kip
Length of vertical	156	in
Lateral Moment in Vertical	8.00	kip-in
Lateral Moment in Vertical	0.67	kip-ft
Available Tensile Strength from AISC Chapter 6.8 of the AASHTO LRFD code		
Section Shape:	W12X87	I-beam
Gross Area, A_g	25.6	in ²
$A_n = 0.75A_g$	19.20	in ²
ϕ_t	0.90	-
ϕ_t	0.75	-
Yielding $\phi_t P_n$	1152.0	kips
Rupture $\phi_t P_n$	960	kips
Minimum	960	kips
Design Check D/C Yielding	0.01	Good
Design Check D/C Rupture	0.01	Good
Available Flexural Strength $\phi_b M_{nx}$		
Section Shape:	W12X87	I-beam
M_{nx}	344.17	kip-ft
$\phi_b M_{nx}$	309.75	kip-ft
M_{py}	550.00	kip-ft
$\phi_b M_{py}$	495.00	kip-ft
M_{nx}	536.22	kip-ft
$\phi_b M_{nx}$	439.43	kip-ft
Design Check D/C	0.05	Good
Available Flexural Strength $\phi_b M_{ny}$		
Section Shape:	W12X87	I-beam
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	226.5	kip-ft
Design Check D/C	0.10	Good
Available Strength in Shear $\phi_v V_n$		
Section Shape:	W12X87	I-beam
Available Strength in Shear $\phi_v V_n$	193.125	kips
Design Check D/C	0.01	Good
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces		
Section Shape:	W12X87	I-beam
Available Compressive Strength $\phi_c P_n$	406.85	kips
Available Flexural Strength $\phi_b M_{nx}$	439.43	kip-ft
Available Strength in Tensile Yielding $\phi_t P_n$	1152.0	kips
Available Strength in Tensile Rupture $\phi_t P_n$	960	kips
Available Strength in Shear $\phi_v V_n$	193.125	kips
Available Strength in Flexure about Y-Y axis $\phi_b M_{ny}$	226.5	kip-ft

D.5 Floor Beams

Floor Beam Dimensions

$$L_{truss.to.truss} := 14.5 \text{ ft}$$

$$width_{tributary} := \frac{14}{3} \text{ ft}$$

Dead Load

$$\gamma_{concrete} := 150 \text{ pcf}$$

$$t_{deck} := \frac{6.5}{12} \text{ ft}$$

$$w_{slab} := \gamma_{concrete} \cdot t_{deck} \cdot width_{tributary}$$

$$\gamma_p := 1.25 \quad (\text{AASHTO LFRD Table 3.4.1-2})$$

$$w_{dead, fact} := \gamma_p \cdot w_{slab}$$

$$w_{railing} := 5 \text{ plf}$$

$$P_{railing} := width_{tributary} \cdot w_{railing}$$

$$P_{rail, fact} := \gamma_p \cdot P_{railing} \quad (\text{acting 6 inches from each side})$$

Live Load

$$W_{ped.} := 90 \text{ psf} \quad (\text{AASHTO LFRD GSDPB 3.1})$$

$$w_{ped.} := W_{ped.} \cdot width_{tributary}$$

$$\gamma := 1.75 \quad (\text{AASHTO LFRD Table 3.4.1-1})$$

$$w_{ped, fact} := \gamma \cdot w_{ped.}$$

$$P_{H10} := 8 \text{ kip} \quad (\text{AASHTO LFRD GSDPB Table 3.2-1})$$

$$P_{H10, fact} := \gamma \cdot P_{H10}$$

$$L_{truss.to.truss} = 14.5 \text{ ft}$$

$$width_{tributary} = 4.667 \text{ ft}$$

$$\gamma_{concrete} = 150 \text{ pcf}$$

$$t_{deck} = 0.542 \text{ ft}$$

$$w_{slab} = 379.167 \text{ plf}$$

$$\gamma_p = 1.25$$

$$w_{dead, fact} = 0.474 \text{ klf}$$

$$w_{railing} = 5 \text{ plf}$$

$$P_{railing} = 0.023 \text{ kip}$$

$$P_{rail, fact} = 0.029 \text{ kip}$$

$$W_{ped.} = 90 \text{ psf}$$

$$w_{ped.} = 0.42 \text{ klf}$$

$$\gamma = 1.75$$

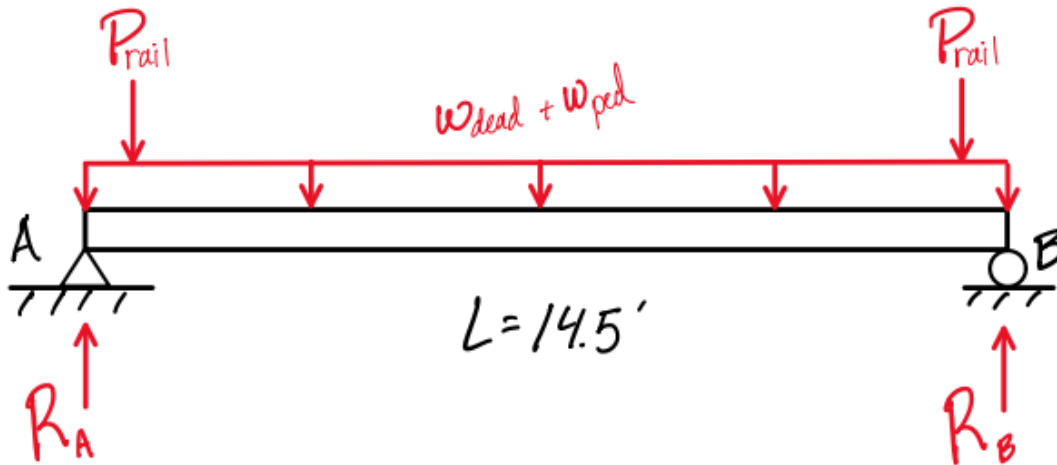
$$w_{ped, fact} = 0.735 \text{ klf}$$

$$P_{H10} = 8 \text{ kip}$$

$$P_{H10, fact} = 14 \text{ kip}$$

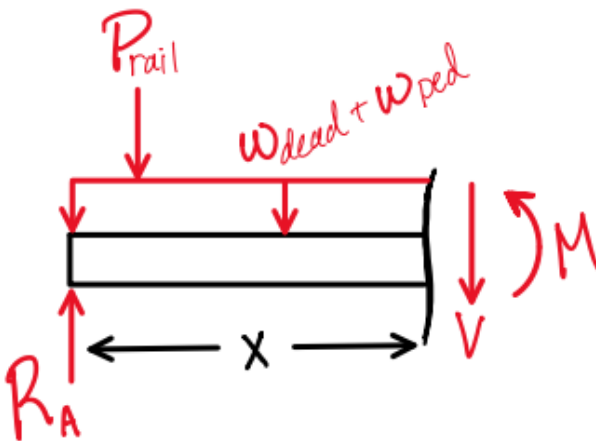
Structural Analysis (2 load combinations)

Combination 1: Dead + Pedestrian



$$R_{A1} := \frac{2 \cdot (P_{rail, fact}) + (w_{dead, fact} + w_{ped, fact}) \cdot (L_{truss, to, truss})}{2}$$

$$R_{A1} = 8.794 \text{ kip}$$



$$x_{max.M} := \frac{L_{truss, to, truss}}{2}$$

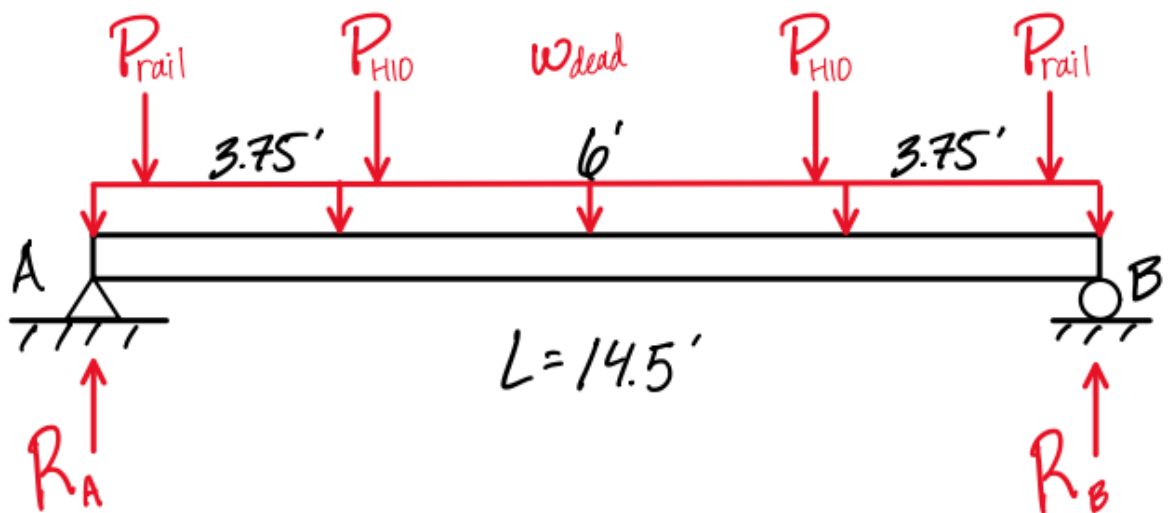
$$x_{max.M} = 7.25 \text{ ft}$$

$$x_1 := x_{max.M}$$

$$M_{max.1} := \frac{-x_1^2}{2} (w_{dead, fact} + w_{ped, fact}) - P_{rail, fact} \cdot (x_1 - 0.5 \text{ ft}) + R_{A1} \cdot (x_1) \quad M_{max.1} = 31.788 \text{ kip} \cdot \text{ft}$$

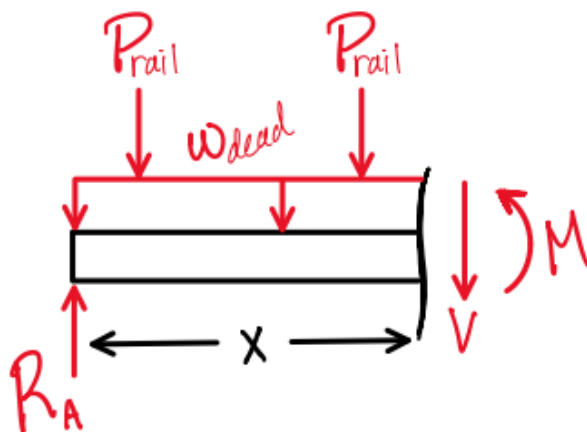
$$V_{max.1} := R_{A1} \quad V_{max.1} = 8.794 \text{ kip}$$

Combination 2: Dead + H10 Truck



$$R_{A2} := \frac{2 P_{rail, fact} + 2 P_{H10, fact} + w_{dead, fact} \cdot (14.5 \text{ ft})}{2}$$

$$R_{A2} = 17.465 \text{ kip}$$



$$M_{max.2} := \frac{-x_1^2}{2} (w_{dead, fact}) - P_{rail, fact} \cdot (x_1 - 0.5 \text{ ft}) - P_{H10, fact} \cdot (x_1 - 4.25 \text{ ft}) + R_{A2} \cdot (x_1)$$

$$M_{max.2} = 71.971 \text{ kip}\cdot\text{ft}$$

$$V_{max.2} := R_{A2}$$

$$V_{max.2} = 17.465 \text{ kip}$$

Combination 2 controls design.

$$M_{max} := M_{max.2}$$

$$V_{max} := V_{max.2}$$

Steel Design

Assuming continuous lateral support.

$$F_y := 50 \text{ ksi}$$

$$F_y = 50 \text{ ksi}$$

$$M_p := M_{max}$$

$$M_p = 71.971 \text{ kip}\cdot\text{ft}$$

$$Z_{x.req} := \frac{M_p}{F_y} \quad (\text{AASHTO LFRD 6.12.2.2.1})$$

$$Z_{x.req} = 17.273 \text{ in}^3$$

from Table 1-1 (AISC Steel Manual): Use **W10x17** section

$$Z_{x.new} := 18.7 \text{ in}^3$$

$$Z_{x.new} = 18.7 \text{ in}^3$$

$$M_{p.new} := F_y \cdot Z_{x.new}$$

$$M_{p.new} = 77.917 \text{ kip}\cdot\text{ft}$$

$$M_{p.new} > M_p$$

$$\phi_V V_n := 72.7 \text{ kip} \quad \text{from Table 6-2}$$

$$\phi_V V_n = 72.7 \text{ kip}$$

$$\phi_V V_n > V_{max}$$

D.6 Slab Design

Slab Design:			
$Span := 4.334 \text{ ft}$	Span Length	$I_x := 9.1137 \text{ in}^4$	Moment of inertia
$P := 200 \text{ kip}$	Point Load	$E := 29000000 \text{ psi}$	Modules of elasticity of steel
$W := 7.5 \frac{\text{lb}}{\text{in}}$	Distributed Load	$f_c := 4000 \text{ psi}$	Compressive strength of concrete
$b := 1 \text{ ft}$	Design Width	$A_{s1} := 3.013 \text{ in}^2$	Area of steel for tension
$db := 4.4 \text{ in}$	Steel Diameter	$h := 6.5 \text{ in}$	Thickness of the slab
$\gamma := 150 \text{ pcf}$	Unit weight of concrete	$F_y := 50000 \text{ psi}$	Yield strength of steel
$\beta_1 := 0.85$	Stress block factor	$\phi := 0.75$	Flexural resistance factor
$d := h - \frac{db}{2} = 4.3 \text{ in}$	Effective depth	$SteelW := 8.63 \text{ psf}$	Weight of corrugated steel
$SlabW := h \cdot \gamma = 81.25 \text{ psf}$	Weight of slab		
Flexural Design:			
$DL := SteelW + SlabW = 89.88 \text{ psf}$	Dead Load		
$LL := 90 \text{ psf}$	Live Load		
$W_u := (1.25 \cdot DL) + (1.75 \cdot LL) = 269.85 \text{ psf}$	Factored Load	(AASHTO LRF 3.4)	
$W_{u1} := W_u \cdot b = 269.85 \text{ plf}$			

$$M_u := \frac{Wu1 \cdot Span^2}{8} = 7.603 \text{ in} \cdot \text{kip} \quad \text{Factored Moment}$$

$$C := \frac{As1 \cdot Fy}{0.85 \cdot fc \cdot b} = 3.692 \text{ in} \quad \text{Distance between the neutral axis and the compressive face} \quad (\text{AASHTO LRF 5.7.3.1.1-4})$$

$$a := C \cdot \beta1 = 3.139 \text{ in} \quad \text{Depth of the equivalent stress block} \quad (\text{AASHTO LRF 5.7.2.2})$$

$$M_n := Fy \cdot As1 \cdot \left(d - \frac{a}{2} \right) = 411.384 \text{ in} \cdot \text{kip} \quad \text{Nominal resistance} \quad (\text{AASHTO LRF 5.7.3.2.2 - 1})$$

$$\phi M_n := M_n \cdot \phi = 308.538 \text{ in} \cdot \text{kip} \quad \text{Factored resistance}$$

Since $M_u < \phi M_n$ design is OK

Use 4.25 X 12 Gage 9 corrugated steel
For reinforcement

$$As2 := 0.002 \cdot h \cdot b = 0.156 \text{ in}^2 \quad \text{Shrinkage and temperature reinforcement}$$

Use wire size 8 @ 6 inches spacing

Deflection Design:

$$\Delta_{allowable} := \frac{Span}{1200} = 0.043 \text{ in} \quad \text{Maximum allowable deflection} \quad (\text{AASHTO LRF 9.5.2})$$

$$\Delta := \frac{5 \cdot W \cdot Span^4}{384 \cdot E \cdot Ix} = 0.003 \text{ in} \quad \text{Deflection due to Live load}$$

Since $\Delta < \Delta_{allowable}$ Design is ok

4 1/4 x 12 Flooring

PROPERTIES ⁹												DEFLECTION LIMIT ⁸					
NOMINAL GAGE	DESIGN THICKNESS	PLANK WIDTH	PLANK AREA	I_{xx}	d_b	d_t	S_{xt} S_{xb}	Z_{xt} Z_{xb}	WEIGHT OF PLANK	MATERIAL SPECIFICATIONS	GRADE (f_y)	ASD HS20		ASD HS25		LRFD HL93	
												SPAN	SPAN/ Δ	SPAN	SPAN/ Δ	SPAN	SPAN/ Δ
												in.	in.	in ²	in ⁴	in.	in.
9.0	0.1495	14.25	3.013	9.1137	2.108	2.292	3.976 4.323	4.661 5.074	8.63	ASTM A 1011-SS	50	51.00	826	44.00	842	43.00	804
7.0	0.1793	14.25	3.6114	10.9236	2.123	2.307	4.735 5.145	5.587 6.082	10.35	ASTM A 1011-SS	50	58.00	836	50.00	810	47.50	801
5.0	0.2092	14.25	4.211	12.738	2.138	2.322	5.486 5.958	6.514 7.091	12.07	ASTM A 1011-SS	50	66.00	817	55.00	839	53.25	806
3.0	0.2391	14.25	4.810	14.552	2.153	2.337	6.227 6.759	7.441 8.100	13.78	ASTM A 1018-HSLAS	50	71.50	809	59.50	824	58.50	813

3 x 9 Flooring

PROPERTIES ⁹												DEFLECTION LIMIT ⁸					
NOMINAL GAGE	DESIGN THICKNESS	PLANK WIDTH	PLANK AREA	I_{xx}	d_b	d_t	S_{xt} S_{xb}	Z_{xt} Z_{xb}	WEIGHT OF PLANK	MATERIAL SPECIFICATIONS	GRADE (f_y)	ASD HS20		ASD HS25		LRFD HL93	
												SPAN	SPAN/ Δ	SPAN	SPAN/ Δ	SPAN	SPAN/ Δ
												in.	in.	in ²	in ⁴	in.	in.
7.0	0.1793	20.125	4.973	7.550	1.584	1.595	4.734 4.767	5.605 5.724	10.09	ASTM A 1011-SS	50	39.00	833	36.00	839	36.00	820
5.0	0.2092	20.125	5.804	8.783	1.596	1.613	5.445 5.503	6.513 6.690	11.78	ASTM A 1011-SS	50	42.00	808	38.50	800	37.50	829
3.0	0.2391	20.125	6.633	9.986	1.609	1.630	6.126 6.206	7.397 7.644	13.46	ASTM A 1018-HSLAS	50	44.00	826	40.00	810	39.00	815

Assumptions for Span Table Calculations:

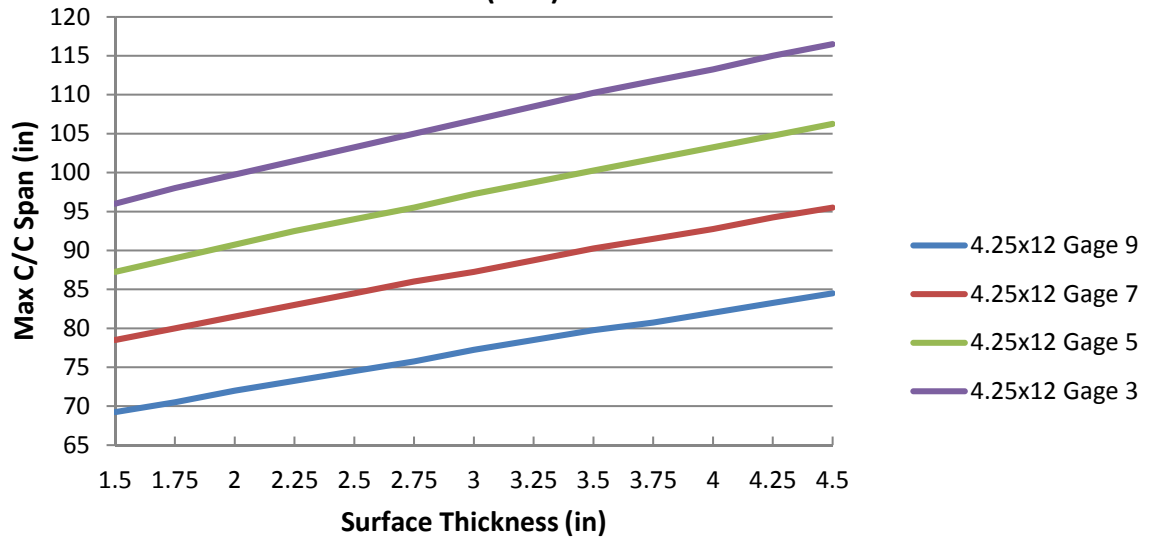
- beam flange width = 9 in.
- roadway width = 24 ft.
- crown = 3/16" per ft.
- asphalt density = 140 pcf

Notes:

- 1 Strength and deflection limits report the maximum center to center dimension based on various wearing surface thickness above the corrugations
- 2 Effective Span for ASD defined as per AASHTO 3.25.1.2.
- 3 ASD/HS20 curves use a wheel load of 12k as per notes from AASHTO Figure 3.7.7.A.
- 4 ASD/HS25 curves use a wheel load of 15k proportioned from AASHTO Figure 3.7.7.A.
- 5 LRFD/HL93 curves use a wheel load of 16k per AASHTO LRFD 3.6.1.2.5 and distribution per 9.8.5.2
- 6 Analysis assumes a pinned end-span support condition at each end
- 7 The LRFD span limits are based on a continuous beam analysis using load positioning to produce a maximum load effect
- 8 The deflection limits are based on meeting AASHTO's optional criteria of L/800. These values are calculated using an asphalt thickness of 1.5 in above the plank corrugations. Like the strength charts show, a higher deflection limit can be expected for thicker asphalt amounts.
- 9 Plank dimensions and property calculations can be downloaded at www.usbridge.com
- 10 Detailed proof-of-method for these span limits can be downloaded at www.usbridge.com

**STRENGTH LIMIT
For 4 1/4 x 12 FLOORING**

ALLOWABLE STRESS DESIGN (ASD) USING HS20 TRUCK



D.7 Columns

Material Properties

$$f_y := 60000 \text{ psi}$$

$$f'_c := 4000 \text{ psi}$$

Design of Column 2 for compression and flexure

Column Info

$$b := 36 \text{ in} \quad \text{Column width}$$

$$h := 72 \text{ in} \quad \text{Column depth}$$

$$A_g := h \cdot b = (2.592 \cdot 10^3) \text{ in}^2 \quad \text{Gross area}$$

$$P_u := 548.86 \text{ kip} \quad \text{Factored axial load}$$

$$\text{cover} := 3.0 \text{ in} \quad \text{AASHTO Table 5.12.3-1}$$

$$k := 0.65 \quad \text{Design effective length factor (fixed-pinned)}$$

$$L_u := 18.8 \text{ ft} \quad \text{Unbraced column length}$$

$$r := 0.3 \cdot h = 21.6 \text{ in} \quad \text{Radius of gyration}$$

Select Reinforcing

Longitudinal Reinforcement

$$A_{s_{required}} := \frac{(0.135 \cdot A_g \cdot f'_c)}{f_y} = 23.328 \text{ in}^2 \quad \text{AASHTO eqn. 5.7.4.2-3}$$

Check $A_{s_{min}}$ and $A_{s_{max}}$

$$\text{Check: } A_{s_{min}} := 0.01 \cdot A_g = 25.92 \text{ in}^2$$

$$A_{s_{max}} := 0.04 \cdot A_g = 103.68 \text{ in}^2 \quad \text{AASHTO 5.10.11.4.1a}$$

$$\text{Choose: } A_s := A_{s_{min}} = 25.92 \text{ in}^2$$

Select 26 #9 bars for longitudinal reinforcement

$$A_{s_{provided}} := 26.00 \text{ in}^2$$

Transverse Reinforcing

$$s_{min} := 12.0 \text{ in} \quad \text{AASHTO 5.8.2.7}$$

Choose #3 bar at 12 inches on center

Axial Capacity

$$P_n := 0.80 \cdot [0.85 \cdot f'_c \cdot (A_g - A_{s_{provided}}) + f_y \cdot A_{s_{provided}}] = [8.228 \cdot 10^3] \text{ kip}$$

AASHTO eq. 5.7.4.4-3

$$\phi_{axial} := 0.75 \quad \text{AASHTO 5.5.4.2}$$

$$\phi_{axial} \cdot P_n = [6.171 \cdot 10^3] \text{ kip} > P_u = 548.86 \text{ kip} \quad \text{OK}$$

Flexural Capacity

Weak Axis bending

$$\beta_1 := 0.85 \quad \text{AASHTO 5.7.2.2}$$

$$c := \frac{(A_{s_{provided}} \cdot f_y)}{0.85 f'_c \cdot \beta_1 \cdot h} = 7.497 \text{ in} \quad \text{AASHTO eqn. 5.7.3.1.2-4}$$

$$a := c \cdot \beta_1 = 6.373 \text{ in} \quad \text{AASHTO 5.7.3.2.3}$$

$$d_b := 1.0 \text{ in}$$

$$d_{weak} := b - cover - \frac{d_b}{2} = 32.5 \text{ in}$$

$$Mn_{weak} := A_{s_{provided}} \cdot f_y \cdot \left(d_{weak} - \frac{a}{2} \right) = (3.811 \cdot 10^3) \text{ kip} \cdot \text{ft} \quad \text{AASHTO eq. 5.7.3.2.2-1}$$

$$\phi_{flexure} := 0.9 \quad \text{AASHTO 5.5.4.2}$$

$$\phi_{flexure} \cdot Mn_{weak} = (3.43 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Moment demand is controlled by seismic loading conditions, factored moment M_u is taken as 5% of superstructure dead weight acting at a lever arm of column unbraced length with load factor 1.0 per ASCE 7-16 factored load combinations.

$$M_u := 246 \text{ kip} \cdot \text{ft} < \phi Mn = 3430 \text{ kip} \cdot \text{ft} \quad \text{OK}$$

Check Slenderness

$$\frac{(k \cdot L_u)}{r} = 6.789 < 22 \quad \text{OK}$$


D.8 Prestressed Concrete Voided Slabs

Bridge Beam Dimensions and Weight (DcL)		
Loading Factor	1.25	-
Height of the beam	26	in
Height of the beam	2.167	ft
Cross-sectional area of concrete only	846	in ²
Cross-sectional area of concrete only	5.875	ft ²
Legth of each beam	54.5	ft
Width of the beam	4	ft
Width of the bridge	15	ft
Number of beams	3.75	pc
Assigned number of beams	3	pc
Unit weight of concrete	150	lb/ft ³
1 foot	12	in
Concrete/Total volume ratio	0.68	-
Weight of each beam	48.03	kips
Factored weight of each beam	60.04	kips
Total weight from all beams	144.08	kips
Factored weight of all beams	180.11	kips
Live Load (LL)		
Loading factor	1.75	-
Pedestrian	90	lb/ft ²
Truck H10	20	kips
Total pedestrian load	73575	lb
Total pedestrian load	73.575	kips
Factored Weight of Truck H10	35	kips
Factored Total pedestrian load	128756.25	lb
Factored Total pedestrian load	128.76	kips
Maximum Live Load (LL)	128.76	kips
Unit conversion factor 1ft = 12in		
1 foot	12	in
Dead Load (DcL)		
Unit weight of concrete	150	lb/ft ³
Slab (DcL)		
Thickness	5	in
Linear weight	937.5	lb/ft
Total Weight	51093.75	lb
Total Weight	51.09	kips
Fence (DcL)		
Number of sides of the bridge	2	sides
Length	54.5	ft
Linear weight	20	lb/ft
Total weight	2180	lb
Total weight	2.18	kips
Street Lights (DcL)		
Number of sides of the bridge	2	sides
Length	54.5	ft
Linear weight	5	lb/ft
Total weight	545	lb
Total weight	0.545	kips

Hand Rails (DcL)		
Number of sides of the bridge	2	sides
Length	109	ft
Linear weight	349	lb/ft
Total weight	38041	lb
Total weight	38.04	kips
Dead Load (DwL)		
Pavement (DwL)		
Loading factor	1.5	-
Thickness	1	in
Linear weight	187.5	lb/ft
Total Weight	10218.75	lb
Total Weight	10.22	kips
Factored weight	15328.13	lb
Factored weight	15.33	kips
Utilities (DwL)		
Loading factor	1.5	-
Linear weight	20	lb/ft
Length of the bidge	54.5	ft
Number of sides of the bridge	2	sides
Total length of utilities	109	ft
Total weight	2180	lb
Total weight	2.18	kips
Total factored weight	3270	lb
Total factored weight	3.27	kips
Total Live Load (LL)		
Loading factor	1.75	-
Summation of all Dead Loads	73.58	kips
Factored Total Deadload	128.76	kips
Maximum Dead Load DwL	128.76	kips
Total Dead Load (DcL)		
Loading factor	1.25	-
Summation of all Dead Loads	235.94	kips
Factored Total Deadload	294.93	kips
Maximum Dead Load DwL	294.93	kips
Total Dead Load (DwL)		
Loading factor	1.5	-
Summation of all Dead Loads	12.40	kips
Factored Total Deadload	18.60	kips
Maximum Dead Load DwL	18.60	kips
Total Load From the Prestressed Concrete Single Beam		
Dead Load (DcL) + Dead Load (DwL) + Live Load (LL)	321.92	kips
Factored Dead Load + Live Load	442.28	kips
Total Unfactored Load From the Prestressed Beam	321.92	kips
Total Factored Load From the Prestressed Beam	442.28	kips


APPENDIX E

PROJECT POSTERS



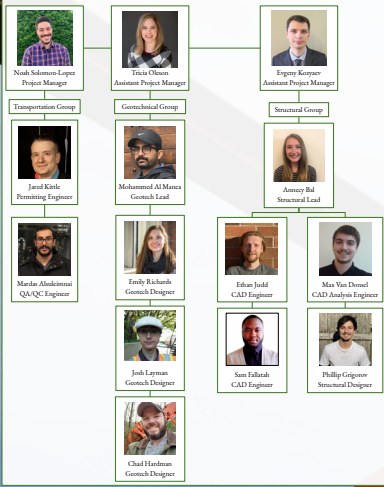
Portland State UNIVERSITY
Department of Civil and Environmental Engineering, Capstone 2021

CBD-URA PEDESTRIAN BRIDGE PROJECT



THE CITY OF TROUTDALE OREGON
EST. 1917

Capstone Organization Chart



Need/Purpose

The purpose of this project is to provide connectivity and access between the two districts by designing a mixed-use (pedestrian, bicycle, and light vehicle) bridge over a railroad that separates the districts.

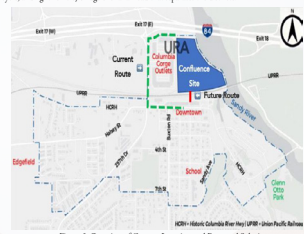


Figure 2: Overview of Current Location and Proposed Solution

Troutdale currently has a low population density with 15,965 people as of 2010. The City of Troutdale has been expanding by 1.4% for the last 10 years due to natural growth but remains disconnected from surrounding areas. To stabilize the population and bring more people into the City of Troutdale, there is a need for more development in the Troutdale's Town Center District. The Confluence Site is considered the largest developable area in the city's urban renewal area (URA) shown in Figure 2, above. It is separated from downtown Troutdale by an active, multi-line Union Pacific Railroad right-of-way, making the connections between the two areas indirect. The desired outcomes of the multipurpose bridge are to directly link existing and future development, which will cause improvements to infrastructure and civic pride.

Proposed Design Solution

Based on our alternatives analysis, the Steel Truss Design ranks the highest in meeting the needs of the City of Troutdale. The steel truss system has a high strength-to-weight ratio making it cost-efficient. The other positive aspect of this design is its effective weight distribution and load-bearing capacity across short spans. The beginning portion of the bridge will be made out of precast, prestressed concrete for ease of construction and to prevent obstruction of the historic architecture and views of tenants.

Steel is incredibly durable, fire and pest resistant, and has more flexibility to be retrofitted for future needs. Currently, Troutdale is revitalizing the area, and more than one bridge is in the design process. A truss system allows for large ordering from one supplier lowering the overall cost of materials and working with the same contractor.

Project Background

Location: Between the 200 and 300 blocks of the Historic Columbia River Highway, Troutdale, Oregon




Figure 1: Overview of the URA with proposed pedestrian crossing location

Overview:

- Urban Renewal Area (URA) to the north
- Central Business District (CBD) to the south
- URA and CBD are separated by active railroad tracks (UPRR)

Alternatives Analysis

Material and design alternatives were evaluated through the decision-matrix method, as shown below. Selection criteria included cost, upkeep time, lifespan, aesthetics, and environmental impact. Each alternative was assigned rankings with a maximum value of 3 and a minimum value of 0. Categories were weighted with an emphasis on efficient resource allocation, due to limitations on the available budget. Bridge designs that posed subcontracting difficulties or had the potential to damage the local environment were omitted from the list of alternatives.

ALTERNATIVE	CRITERIA						TOTAL
	COST	UPKEEP	TIME	URBAN	AESTHETICS	ENVIRONMENT	
1) Concrete Box-Girder	2	3	2	3	2	2	19.7
2) Steel Truss Design	3	2	3	3	3	2	22.3
3) Timber Design	3	1	3	1	3	3	19.0
4) No-Build	3	3	3	0	0	3	18.0
WEIGHT	6	6	2	3	4	2	25

Figure 3: Alternatives Matrix

- Cost (8) - The cumulative material, labor, and transportation expenses required to construct a given alternative.
- Upkeep (6) - The sum of all expenses required to maintain the structure after construction.
- Time (2) - The period of active bridge construction, where the structure is not yet serviceable to the community.
- Lifespan (3) - The period before structural decay renders the bridge unsafe to use.
- Aesthetics (4) - The value assigned to the appearance of each design alternative.
- Environmental Impact (2) - The impact that design, construction, and the bridge's existence have on the environment.

Total Possible: 25 Maximum Score: 22.3 Minimum Score: 18

Multidisciplinary Aspect of the Project

This project required more than one engineering discipline to complete. Collaboration efforts were made with professionals to gather information and resources. The project team communicated over Zoom, Slack, and group text messages.

Three main disciplines involved:

- Structural
- Geotech
- Transportation

Professional Interactions:

- Evan Kinard - CAD and general project assistance.
- Mary Ann Finke - Structural codes and calculations.
- Kuif River Corporation - Information and calculations on prestressed concrete beams.

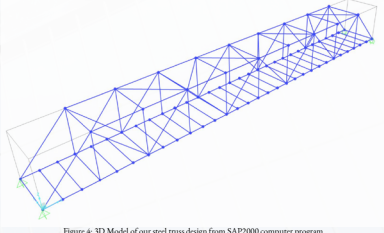


Figure 4: 3D Model of our steel truss design from SAP2000 computer program

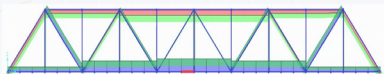


Figure 5: Axial load diagram for our steel truss model from SAP2000 computer program

Geotechnical Considerations and Designs

- Site photos from the visit conducted by Chad Hardman, Evgeny Kozayev, Tricia Olson, and Emily Richards. Thank you to Chad and Evgeny for providing the necessary tools to make the sampling as efficient as possible.
- Hand tools were used to obtain soil samples and observe the geology near the location of the northern footing.
- The geologic information collected will be extrapolated to all footing locations for preliminary design.
- On site, soil consist of a quaternary surficial deposit of alluvial sandy silt. This material transitions to well-graded sand and large rounded rock (Troutdale Formation) at a depth of 10 to 12 feet.
- SPT data collected from nearby locations were aggregated to provide some preliminary soil bearing capacity values that allow for initial footing design.
- The preliminary average allowable bearing capacity for shallow foundation design is 1950 pounds per square foot. This value will govern the design dimensions of the footings.
- Should shallow foundations be prohibitively costly or spatially inefficient, deep foundation design could be pursued as the design develops further.

LOG OF HAND AUGER BORING		
Project: CBD-URA Pedestrian Bridge PSU Capstone 2021	Field Team: Tricia Olson, Evgeny Kozayev, Emily Richards, Chad Hardman	Date: April 19, 2021 Boring No.: HA-1
Depth (ft)	Soil Type	Material Description
1-11	GM	Traffic compacted driveway material, recycled concrete and mixed rounded rock fragments.
11-12	7	Silt, to brown, moist, sandy SILT, slightly cohesive, non-plastic, orange and dark brown to black mottling, slightly micaceous. (Quaternary Alluvial Deposits)
12-13	5	
13-14	ML	
14-15	5	
15-16	9	Medium stiff, to brown, very moist, silty fine grained SAND, non-cohesive, non-plastic, slightly micaceous. (Quaternary Alluvial Deposits)
16-17	SM	
17-18	32	Dense, grey-brown, saturated, well graded SAND. (Troutdale Formation)
18-19	SP	
19-20		Boring terminated at 12 feet (Refused on large rock (assumed))
20-21	13	

Figure 6: Hand Auger Boring Log from site visit on 19 April 2021



Figure 7: Soil sample taken with a hand auger at roughly 5 feet deep



Figure 8: Tricia Olson taking a sample with the hand auger



Figure 9: Evgeny Kozayev starting the sampling with a drilling tool

Impact on Society

The impact of this multi-purpose crossing pedestrian bridge on society is significant, especially in the near future. Some of the project's potential effects include the following:

- Improvement of public safety
- Potential increase to land price mark
- Easier access from the Confluence site to downtown
- Easy and quick route and crossing for cyclists
- Positive impact on businesses in the area
- A step forward toward tourism promotion

Railway Clearance

The main span of this design will extend over Union Pacific Railroad property into the urban renewal area, per their standard regulations all proposed structures are required to maintain a minimum clearance of 23' vertically from the top surface of any existing rail, as well as 25' horizontally from its centerline. Figures 10 and 11 show our CAD drafts of what the proposed will look like.

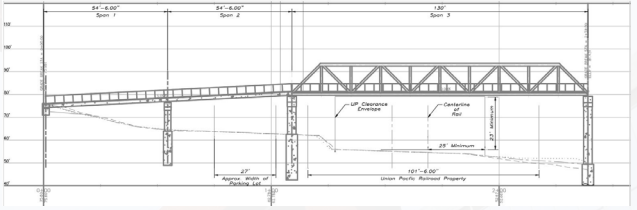


Figure 10: Proposed Bridge Elevation View

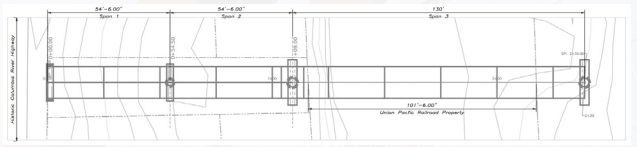


Figure 11: Proposed Bridge Plan View

Consideration of Professional Practice

This project has provided opportunities for the team to enhance their technical skills and knowledge which will contribute to their development and success. The subsequent skills have been utilized to complete project analysis.

- | | |
|---|--|
| Professional Discipline: <ul style="list-style-type: none"> • Project management • QA/QC • Engineering design codes & standards • Construction scheduling & cost estimation • Effective communication • Problem solving • Technical writing | Utilized Software: <ul style="list-style-type: none"> • AutoCad • Sap2000 • MicroCAD • Microsoft office suite • Civil 3D • Clockify • Projectliber • MS Project |
|---|--|

Next Steps

At the current stage of the project, we have completed extensive research of the site conditions as well as an overall design of the proposed bridge and its loading conditions.

Looking forward, further steps the project will need to take to finalize the design are:

- Earthquake loading analysis
- Soil liquefaction analysis
- Foundational cost analysis (load capacity)
- Runoff analysis
- Transportation impacts (pedestrian vs vehicle interaction)

Acknowledgements

The Portland State University Capstone Team working on the CBD-URA Pedestrian Bridge Project would like to acknowledge all those who have instructed us in all aspects of life throughout our education at Portland State University. Special acknowledgments and thanks to those who have directly guided our work on this project.

- | | |
|---|--|
| <ul style="list-style-type: none"> • Amber Shackelford, Assistant Planner, City of Troutdale • Chris Dauges, Community Development Director, City of Troutdale • Evan Kristof, Portland State University • Patrick McLaughlin, Instructor, Portland State University • Dr. Thomas Schumacher, Associate Professor, Portland State University | <ul style="list-style-type: none"> • Dr. Amirah Usufshahin, Associate Professor, Portland State University • Mary Ann Triksa, Instructor, Portland State University • Dr. Richard Corst, Dean, Portland State University • Eric Hodge, Estimator, Knife River Presses • Sam Jewell, Outside Sales - Bridge, Knife River Presses |
|---|--|

APPENDIX F

FINAL PRESENTATION

CBD-URA Pedestrian Bridge

(2021.TROUT.01)

2021 CEE Capstone: Final Design Presentation



Noah Solomon-Lopez
Tricia Oleson
Evgeny Kozyaev
Mohammed Al Manea

Anney Bal
Phillip Grigorov
Chad Hardman
Hussam Fallatah

Ethan Judd
Josh Layman
Jared Kittle

Max Van Donsel
Mardas Al Suleimani
Emily Richards



Introduction

■ Org. Chart

▫ **Client:** City of Troutdale: Amber Shackelford & Chris Damgen

▫ **PSU Capstone Team:**

- **PM Team:** Noah Solomon-Lopez, Tricia Oleson, Evgeny Kozyaev

- **Transportation Team:** Mardas AlSuleimani, Jared Kittle

- **Geotech Team:** Mohammed Al Manea, Emily Richards, Joshua Layman, Chad Hardman

- **Structural Team:** Anney Bal, Ethan Judd, Sam Fallatah, Max Van Donsel, Phillip Grigorov

Roadmap

- **Project Background**
- **Alternative Analysis**
- **Facility Design**
 - General Layout
 - Design Choices
 - Truss Design
 - Slab Design
 - Floor Beam Design
- Column Design
- Shallow Foundation Design Parameters
- Footing Design
- Cost Analysis
- Construction Schedule
- **Conclusion**

3

Project Background

- **City of Troutdale**
- **Multipurpose Bridge to connect the Downtown Central Business District (CBD) and Urban Renewal Area (URA)**
 - Areas are disconnected by elevation and Union Pacific railway
- **Current Population of 17,000 people**
 - Downtown has low population density
 - Confluence site is the largest developable area
 - Located in the City's Urban Renewable Area



4

Alternative Analysis

Hybrid Truss Design

- Cost-Effective
 - less than \$800/ft³ to install
 - less than \$95/ft³ to maintain
- Aesthetically Pleasing

ALTERNATIVE	CRITERIA						TOTAL
	COST	MAINTENANCE	SCHEDULE	LIFESPAN	AESTHETICS	ENVIRONMENT	
1) Concrete Box-Girder	2	3	2	3	2	2	19.7
2) Steel Truss Design	3	2	3	3	3	2	22.3
3) Timber Design	3	1	3	1	3	3	19.0
4) No-Build	3	3	3	0	0	3	18.0
WEIGHT	8	6	2	3	4	2	25

1. Concrete Box-Girder



Concrete Construction, 2008

2. Steel Truss Design



Architecture & Design, 2008

3. Timber Design



Brampton Woodworks, 2017

5

Facility Design - General layout

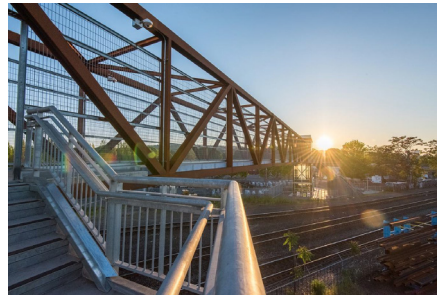
Bridge Spans

- Voided Slabs
 - Knife River
- Warren Truss

Design inspiration

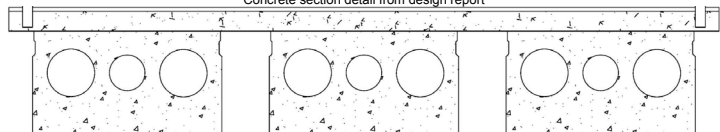
- Lafayette Street Pedestrian Bridge

Lafayette Street Pedestrian Bridge



Lafayette Pedestrian Bridge (Source: www.kpff.com)

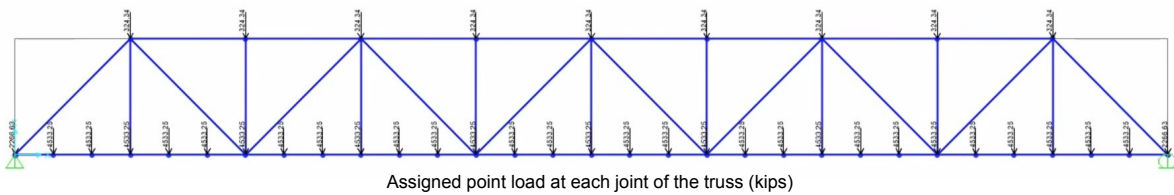
Concrete section detail from design report



6

Facility Design - Truss Design

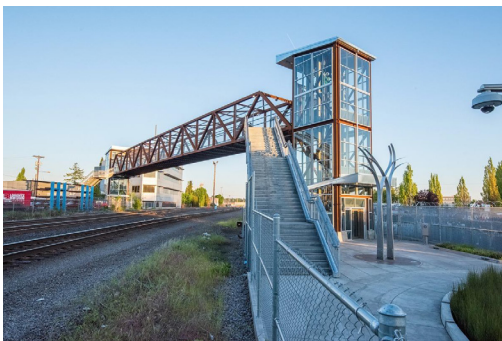
- **Conservative weight choices**
 - 10% connections
 - 15% utilities (future / building)
 - Simplified calculations



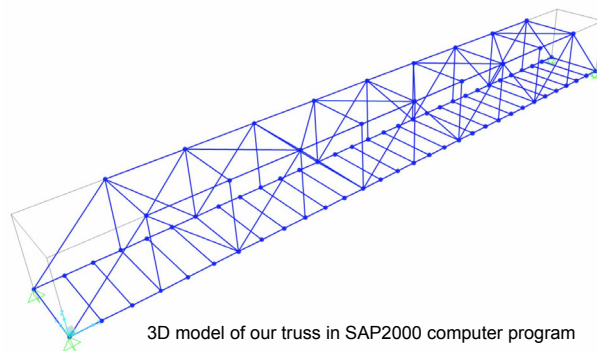
9

Facility Design - Truss Design

- **Initial member selection**
 - Example from Lafayette bridge
- **Loading conditions 2D vs. 3D (SAP2000 model)**



Lafayette Pedestrian Bridge (Source: www.kpff.com)



3D model of our truss in SAP2000 computer program

10

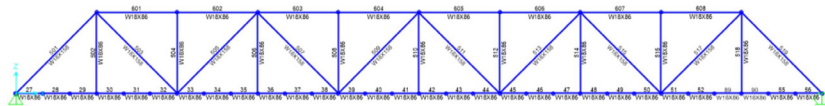
Facility Design - Truss Design

- **Difference in results between SAP2000 analysis and hand check in Excel**

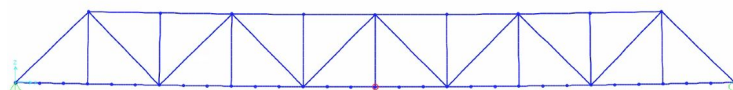
- SAP2000 vs Excel

- **I-beam selection**

- Serviceability check
 - Vibration analysis
 - Design check
 - Depth of each I-beam
 - *Economy*
 - Final selection



Selected members for the truss



Joint Displacements			
Joint Class #	Joint Element #	Joint Element #	
1	2	3	
Trans	0.3001	-0.29425	-0.29092
Rot	0.00027	4.2102E-04	-3.7605E-04

Exaggerated deflection of the truss due to full factored loading condition (in)

11

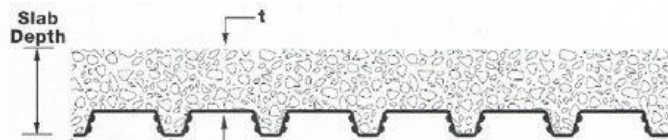
Facility Design - Slab Design

- **Design**

- Load design
 - *Heavy pedestrian use*
 - *Small trucks*
 - Design check
 - *Maximum allowable deflection*
 - *Moment resistance*

- **Design Decision**

- Traditional rebar
 - Corrugated steel



- **Corrugated Steel**

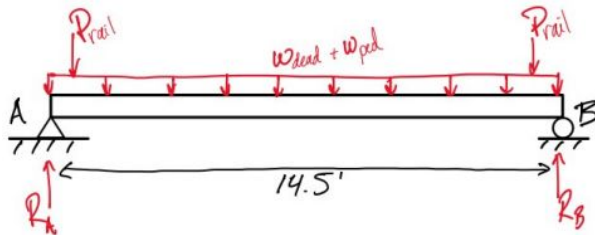
- Cost reduction
 - *Reduce thickness by 2" (save on material)*
 - *No need for wooden cast (save on labor and material)*

12

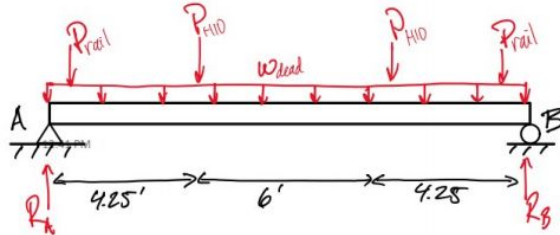
Facility Design - Floor Beam Design

Load Combinations

Combination 1: Dead + Pedestrian



Combination 2: Dead + H10 Truck



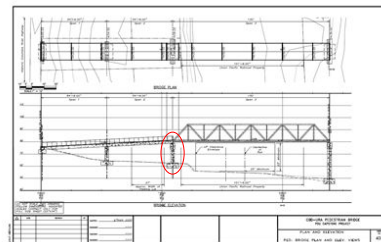
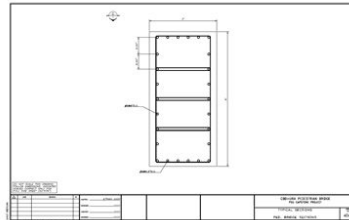
- Designed section: W10x17 (AISC Steel Manual)

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Facility Design - Column Design

Cast in place column

- Rectangular cross section: 3ftx6ft
- 4000 psi concrete
- Grade 60 steel
- Vertical reinforcement: #9 bar
- Transverse reinforcement: #3 bar
 - Overlapping closed ties
- AASHTO Bridge Design Specifications
 - Gravity and seismic loading



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Facility Design - Shallow Foundation Design Parameters

Hand Auger Boring 80 feet NE of northern bridge footing

- Existing Soil Conditions
 - Surface to 10 feet bgs
 - Alluvial silt
 - 10 to 12 feet
 - Gradual (transition) contact
 - 12 to deep
 - Troutdale Formation
 - Well-graded sand, gravel, and boulders
- Shallow groundwater
 - About 12 feet at time of investigation

Photos from geotechnical investigation by CBD/URA Bridge Design Team.

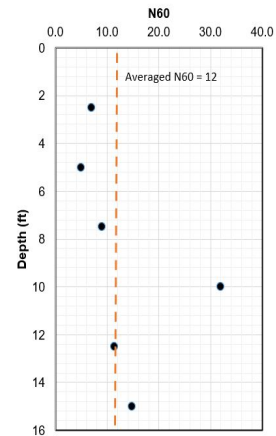


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Facility Design - Shallow Foundation Design Parameters

GRI Report Boring and Standard Penetration Test (SPT) Logs

- SPT blow counts (N) give information about the strength of the soil.
 - The N value is adjusted for energy, depth, and sampling method
 - Correlations between a soil's "bearing capacity" and its N value are well established experimentally
- Throughout the analysis conservative values were used
 - For less conservative values perform foundation specific geotechnical investigations
 - Liquefaction and seismic factors should be considered in future designs



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Facility Design - Footing Design

- **We decided to use shallow foundations.**
 - Shallow foundation analysis was the initial option based on the loadings given.
 - Footings were designed based off Terzaghi's bearing capacity method.

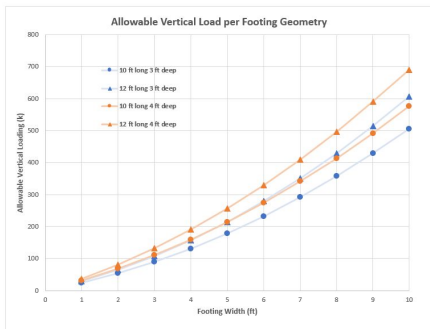


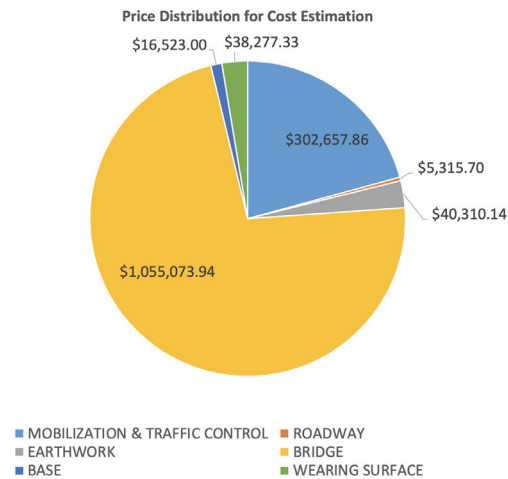
Figure 1

Footing Designs	L x d x b	Pallow > Load
F1	12x3x6	256>221
F2	12x3x10	556>506
F3	12x4x10	633>611
F4	12x4x9	471>427

Figure 2

Facility Design - Cost Analysis

- **Sources of Pricing**
 - ODOT, 2019 data
 - RSMears
- **The units of quantities**
 - Oregon Standard Specifications for Construction of 2018
- **Cost of Labor**
 - Accounted for in each construction item
- **See Appendix A for more details**

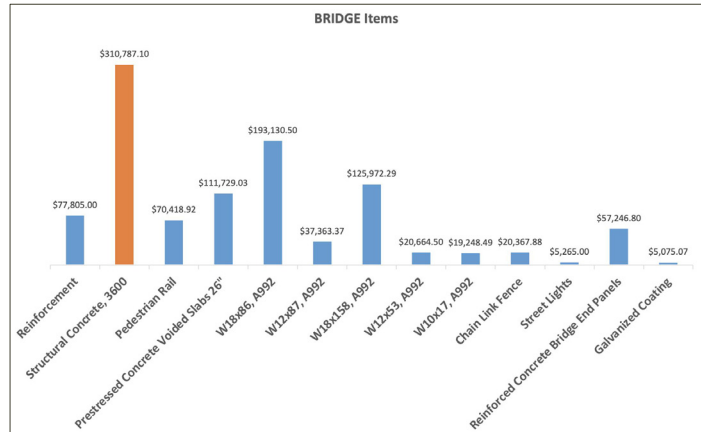
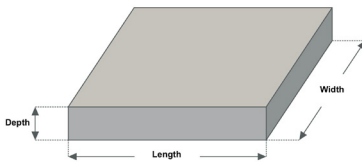


Facility Design - Cost Analysis

Construction concrete quantity estimation

- Three major bridge parts
 - The slab
 - The columns
 - The Footings

Having precautions assumptions and measurements



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Facility Design - Construction Schedule

Tentative Schedule

- Mobilization
 - Permits and Site Preparation
- Substructure Installation
 - Foundations, Abutment, and Columns
- Superstructure Fabrication
 - Delivery, Inspection, and Assembly
- Superstructure Placement
 - Truss, Precast, and Details
- Demobilization
 - Inspection, Consolidation, and Opening



Tandem crane emplacing pedestrian bridge in Fort Collins, CO over BNSF Railway tracks. (Shelby, 2014)

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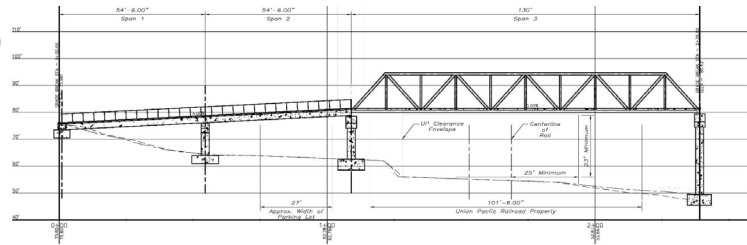
Conclusion

Final Design

- Total Length 239 feet
 - 2 prestressed concrete slabs (54.5')
 - Warren Steel Truss System (130')

Current Estimated Cost

- \$1,458,158



Next Steps and Further Exploration

- | | |
|---------------------------|----------------------------------|
| ▫ Northern Side of Bridge | ▫ Earthquake Loadings |
| ▫ Bent Caps | ▫ Runoff Effects |
| ▫ Deep Foundations | ▫ Pedestrian Vehicle Interaction |

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