KLAMATH RIVER RENEWAL CORPORATION

Klamath River Renewal Project

Daggett Bridge Design Project Design Documentation Report

IFC Design Submittal

Revision No. 03

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McMILLEN JACOBS ASSOCIATES

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The technical material and data contained in this document were prepared under the supervision and direction of the undersigned, whose seals, as professional engineers licensed to practice as such, are affixed.

CIVIL DESIGN
Jeffrey Lowy, P.E.

STRUCTURAL DESIGN
Zachary Autin, P.E.

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Appendix D – Structural Design Calculations
Appendix E – Geotechnical Boring Logs

Distribution

To:	Mark Bransom Klamath River Renewal Corporation	
	Laura Hazlett Klamath River Renewal Corporation	
From:	Jodi Burns McMillen Jacobs Associates	
Prepared By:	Jodi Burns Civil Engineer McMillen Jacobs Associates	Nathan Cox, P.E. Hydraulic Engineer McMillen Jacobs Associates
	Greg Clark Structural Engineer McMillen Jacobs Associates	Shawn Spreng, P.E. Geotechnical Engineer McMillen Jacobs Associates
Reviewed By:	Jeff Lowy, P.E. Civil Engineer McMillen Jacobs Associates	Zachary Autin, P.E. Structural Engineer McMillen Jacobs Associates

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1.0 Introduction and Background

1.1 Purpose

The purpose of this report is to present the design documentation associated with development of the Daggett Bridge Design Project (Project).

1.2 Background

1.2.1 Location

Daggett Road Bridge is located near the confluence of Fall Creek and Iron Gate Reservoir in Siskiyou County northwest of Iron Gate Dam near Yreka, California. The existing City of Yreka (City) water line currently crosses the Klamath River downstream from the existing Daggett Bridge adjacent to the mouth of Fall Creek.

1.2.2 Project Description

1.2.2.1 Background

The Daggett Bridge Project design prepared by Kiewit and Knight Piésold (Kiewit) proposed to construct a bridge located on the upstream side of the existing Daggett Bridge to support the removal of Iron Gate Dam. This bridge was proposed to be used during construction to support Kiewit's construction loads which exceed the existing Daggett Bridge rating. The bridge would be constructed during the pre-drawdown year to support Kiewit's construction activities during the Iron Gate Dam removal process.

The existing City waterline crosses the Klamath River just downstream from the mouth of Fall Creek which enters the Klamath River just below the existing Daggett Bridge. The existing pipeline was placed in shallow trench in the bottom of the river which is currently backwatered in the reservoir created by Iron Gate Dam. When Iron Gate Dam is removed, the Klamath River will return to a free-flowing condition which is expected to erode the existing pipeline crossing with potential failure of the pipeline.

When considering the various project components including the City waterline, Daggett Bridge construction, access for fire protection and recreation boating, and the establishment of anadromous fish runs back to the Fall Creek Hatchery, the Klamath River Renewal Corporation (KRRC) determined that it would be beneficial to consider a permanent bridge replacement at the Daggett Road crossing to replace the existing limited rating bridge and eliminate the need for a temporary construction bridge. The new bridge crossing will be designed to meet current load conditions as well as truck loads required to provide fire protection, support construction equipment, and a permanent elevated support of the City pipeline crossing the new Daggett Bridge eliminating the existing City buried river crossing.

1.2.2.2 Existing Daggett Road Bridge Overview

The Daggett Road Bridge is a single lane, four span bridge that spans the Iron Gate Reservoir and is approximately 233 feet in length. The superstructure of the bridge includes steel girders of varying section types, is 14 feet wide with no shoulders. The substructure of the bridge includes seat type concrete abutments on pile caps with H-piles. The Daggett Road Bridge was reconstructed in 1983 to provide a HS-20 load rating; however, the structure has been de-rated with a 17-ton load limit for double axel vehicles, 27-ton load limit for triple axel vehicles, and 29-ton load limit for 4-axel vehicles, as shown in Figure 1-1.

Figure 1-1. Daggett Road Bridge – Elevation South (Source: Knight Piésold Consulting, 2019)

Knight Piésold Consulting inspected Daggett Road Bridge on July 27, 2019, and they made the following visual observations:

- Overall, the bridge is in generally fair condition.
- There is a posted load limit and speed limit.
- There is a cattle guard (grid) at the north approach.
- Abutments appear to be in good/fair condition. It was not clear from visual inspection how the mud sill abutment was performing due to access restrictions, but general profile and alignment appeared good. No movement noted.
- Railings and deck surfaces in good condition.
- Due to the large loads required during the dam removal construction, the existing bridge was determined to be inadequate for construction loads. As a result, a temporary bridge is planned on the upstream side of the existing bridge.

Figure 1-2. Daggett Road Bridge – West End (Source: Knight Piésold Consulting, 2019)

Figure 1-3. Daggett Road Bridge – Elevation North (Source: Knight Piésold Consulting, 2019)

1.3 Report Organization

This Design Documentation Report (DDR) is a record of the design effort for the Project and specifically describes the details of the design process and work effort. The DDR consists of a summary of the design elements, design criteria, methods and approach, engineering calculations, and pertinent references. The major report sections and intended purpose are presented in Table 1-1.

Table 1-1. Major Report Sections and Purpose

Section	Description	Purpose
1	Introduction and Background	Presents the background, a description of the overall Project, and the report organization.
2	Design Criteria	Summarizes the basic design criteria that are used as the basis for the design of the new Daggett Bridge.
3	Project Description	Describes the Daggett Bridge Project.
4	Civil Design	Includes information related to the civil design associated with the Daggett Bridge.
5	Hydrologic & Hydraulic Design	Presents the hydrologic and hydraulic scour analysis of the Daggett Bridge abutments and revetment stone design of the new riprap within the Klamath River.
6	Geotechnical Design	Includes geotechnical information and design associated with the Daggett Bridge.
7	Structural Design	Includes information related to the structural design of the new Daggett Bridge and the pipe supports for the City of Yreka pipeline crossing across the new Daggett Bridge.
8	References	Documents the references used in developing the design.
Appendices		
A	Civil Design Calculations	Presents the detailed calculations related to civil design.
B	Hydrologic & Hydraulic Design Calculations	Presents the detailed calculations related to hydrologic and hydraulic design.
C	Geotechnical Design Calculations	Presents the detailed calculations related to geotechnical design.
D	Structural Design Calculations	Presents the detailed calculations related to structural design.
E	Geotechnical Boring Logs	Presents the boring logs from the Geotechnical Data Report prepared by CDM Smith and AECOM Technical Services, Inc.

2.0 Design Criteria

2.1 Pertinent Data

Pertinent data for the Project include the assumed survey datum, topographic mapping, and references as described below.

2.1.1 Survey Datum

The Project data provided by the Klamath River Renewal Corporation (KRRC) were supplied in reference to the North American Vertical Datum of 1988 (NAVD88, Geoid 12B). This is the vertical datum that will be used on all drawings and in all calculations submitted as deliverable for the Project. The horizontal coordinate system is the California Coordinate System of 1983, Zone 1 North American Datum of 1983 (NAD83) in feet.

2.1.2 Topographic Mapping

Topographic data was supplied by KRRC and included the Light Detection and Ranging (LiDAR) and sonar survey performed in 2018 by GMA Hydrology, Inc. for the entire site.

2.2 References and Data Sources

A wide range of data sources and references were used in developing this DDR. Specific data related to the conceptual design of the Project were obtained from the various technical analyses and memoranda, which include the following:

- CDM Smith and AECOM. 2019. Klamath River Renewal Project Geotechnical Data Report.
- The California Oregon Power Company. 1981. Daggett Road Bridge Drawings.

Additional data sources, including publicly available aerial imagery, U.S. Geological Survey (USGS) maps, USGS streamflow gaging station data, soils maps, as-constructed drawings, and standard engineering reference documents, were used.

2.3 General Design Criteria and Standards

2.3.1 Standard List of Terms and Abbreviations

ACI	American Concrete Institute
ADM	Aluminum Design Manual
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society of Testing and Materials
AWS	American Welding Society
CCOR	California Code of Regulations
cfs	cubic feet per second

CGP	Construction General Permit
DPS	Distinct Population Segment
ECP	Erosion Control Plan
ft^3	cubic feet
GBR	Geotechnical Baseline Report
gpm	gallons per minute
HEC-RAS	Hydrologic Engineering Center River Analysis System
ksf	kips per square foot
KRRC	Klamath River Renewal Corporation
LiDAR	Light Detection and Ranging Survey
mm	millimeter
NAD	North American Datum
NAVD	North American Vertical Datum
Project	Daggett Bridge Design Project
pcf	pounds per cubic foot
psf	pounds per square foot
RWQCB	Regional Water Quality Control Board
USACE	United States Army Corps of Engineers
USACE EMs	United States Army Corps of Engineers Engineer Manuals
USBR	United States Bureau of Reclamation
USGS	United States Geological Survey

2.4 Civil

2.4.1 Erosion Control Plan

The contractor will be required to obtain a Construction Storm Water General Permit from the California State Water Resources Control Board prior to construction. Construction General Permits (CGPs) are required for construction projects that result in greater than 1 acre of soil disturbance. The CGP requires temporary and post-construction Best Management Practices to prevent erosion and reduce sediment discharges from construction sites. Prior to permit issuance by Siskiyou County, submittal of an Erosion Control Plan (ECP) to the appropriate Director at Siskiyou County is required. The ECP shall include methods for controlling runoff, erosion, and sediment movement. The Contractor's ECP shall meet or exceed the requirements outlined in Specification Section 31 25 00 Erosion Sedimentation Controls prepared by Knight Piésold Consulting.

2.4.2 Roadway

Siskiyou County requested that any new roadways be designed that roadway geometry should be improved upon or maintained to the extent practical (Knight Piésold Consulting and Kiewit. 2020a).

Table 2-1. Civil Roadway Design Criteria

Feature/Consideration	Criteria	Remarks	Reference
Design Vehicle	45 ton off-highway articulated haul truck	CAT 745	Project Company
Minimum Lane Width	11 ft		Project Company
Minimum Curve Radius	35 ft		Project Company
Road Grade	Normal road grade ≤7%. Maximum road grade = 15%.	The maximum roadway slope in the design is 7.1%.	Project Company
Cut/Fill Slopes	1V:3H or flatter	Embankment slopes no steeper than 1V:3H wherever practical and, ideally 1V:6H or flatter	Project Agreement

Knight Piésold Consulting and Kiewit. 2020a

2.5 Hydrology and Hydraulic

2.5.1 Applicable Codes and Standards

The following codes, standards, and specifications will serve as the general design criteria for the hydraulic analysis of the Daggett Bridge abutment and required scour protection. The proposed hydrologic and hydraulic engineering criteria are presented in Tables 2-2 and 2-3 below. The criteria presented within these tables represent the anticipated operation and design elements used in the Project development. A permanent bridge at this location is required to provide adequate hydraulic capacity to pass the 1% Annual Probable Flood (ADF) event (Knight Piésold Consulting and Kiewit. 2020a).

Table 2-2. Hydraulic Standards, References, and Standards of Practice

Standard	Reference
Julien, Pierre Y. 2002	River Mechanics. Cambridge University Press, Cambridge, United Kingdom

Table 2-3. Bridge Hydrologic and Hydraulic Design Criteria

Feature/Consideration	Criteria	Remarks	Reference
Bridge Soffit Minimum Freeboard Requirements	Minimum freeboard for permanent bridges will be 1 ft during the 1% annual probable flood.	Distance is measured from water surface elevation to the lowest point on the bridge deck.	
Design Storm/Discharge Data	1% Annual Probable Flood Post Drawdown		AASHTO
Scour	See Section 5.0	See Section 5.0	Julien (2002)
Erosion Protection	Per California Bank and Shore Rock Slope	See Section 5.0	California Bank and Shore Rock Slope Protection Design (2000)

Knight Piésold Consulting and Kiewit. 2020a

2.5.1.1 Scour Analysis

The HEC-RAS model developed by Knight Piésold Consulting, that was used to analyze the scour potential on the Klamath River, was used to look at the river hydraulics at the proposed new Daggett Bridge location. The HEC-RAS model was originally developed to evaluate the reservoir drawdown period for the Klamath River Renewal Project. The HEC-RAS model was run for the new Daggett Bridge crossing which incorporates river abutments on both banks of the river. The scour depth estimate was based on an equation provided by Julien (2002), where the scour depth below of a grade-control structure is evaluated with the drop height being set to zero.

2.5.1.2 Revetment Stone Sizing

The HEC-RAS model was used to estimate the velocities for the 1% AEP flood to evaluate the required rock size to protect the bridge abutment slopes. The rock size will be based on the California Bank and Shore (CABS) method presented in the Caltrans publication on bank and shore rock slope protection design (2000).

2.6 Geotechnical

2.6.1 Applicable Codes and Standards

The following codes, standards, and specifications will serve as the general design criteria for the geotechnical design of the Daggett Bridge. The applicable version of each document is the latest edition in force unless noted otherwise. References to the specific codes and standards will be included in the applicable technical specifications as the final design documents are prepared. The geotechnical design, engineering, materials, equipment, and construction will conform to the codes and standards listed in Table 2-4.

Table 2-4. Geotechnical Codes and Standards

Code	Standard
AASHTO	AASHTO LRFD Bridge Design Specification, 8th Ed. 2017

2.7 Structural

2.7.1 General Design Description

The new Daggett Road Bridge will consist of a 260-foot, two-lane, 24-foot-wide pre-manufactured bridge superstructure, that will be supported by new seat type concrete abutments. The bridge will span the full distance across the Klamath River and is located approximately 75 feet upstream from the existing Daggett Road Bridge. The superstructure basis of design is the Acrow 700XS Panel Bridge.

The new proposed 24-inch diameter waterline is to be constructed as a steel pipeline and will be located along the underside of the new bridge, centered, with supports every 10'-0." The supports will be attached to the bottom of the transom beams and will be incrementally installed as the bridge is launched.

2.7.2 Applicable Codes and Standards

The following codes, standards, and specifications govern the structural design of the new Daggett Road bridge and abutments, as well as the 24-inch diameter waterline supports attaching to the underside of the new bridge. The latest edition of each code is utilized for the design, except as noted otherwise. The structural design, engineering, materials, equipment, and construction conform to the codes and standards listed in Table 2-5.

Table 2-5. Structural Codes and Standards

Element	Code	Standard
Waterline Support	AISC	AISC360-16 – Steel Construction Manual, 14th Edition
Waterline Support Abutments	ASCE	ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures
Abutments	ACI	ACI 318-14 Building Code Requirements for Structural Concrete
Bridge and Waterline	CBC	2022 California Building Code
Bridge Superstructure	AASHTO	AASHTO LRFD Bridge Design Specifications – 8th Edition, 2017 with CA Amendments
Pipe Support Bridge Superstructure	AWS D1.1	Structural Welding Code – Steel, 2020 Edition

The bridge superstructure design incorporates load combination limit states required by AASHTO.

The bridge substructure (abutment) design is based on the worst-case loads produced by the limit states of Strength (I-V), Service (I-II) and Extreme I, as required by AASHTO. Additional load combination limit states required by AASHTO were not evaluated as the load cases involved do not occur, which would not govern over evaluated limit states.

2.7.3 Materials

The material properties used for the design of the bridge superstructure, substructure, and waterline support components are listed in Table 2-6.

Table 2-6. Structural Material Properties

Structural Stainless Steel	
Wide-Flange Shapes (W)	A572, Grade 50
Other Shapes (M, S, C, MC, L)	ASTM 572 Gr. 50 or A36
Plate & Bars (Low Strength)	A36
Structural Bolts	ASTM F3125 Gr. A325 Type 3
Pipe	A53, Grade B
Nuts and Washers	ASTM A563 and ASTM F436 Type 3
Anchor Bolts	ASTM F1554 Gr. 36
Concrete	
Concrete	4,500 psi normal weight
Rebar	ASTM A615, Grade 60

2.7.4 Design Loads

The vertical and lateral loads considered in the design of the bridge superstructure, bridge substructure and waterline supports are summarized in this section. All loads were factored and combined per the requirements of ASCE 7 and AASHTO LRFD Bridge Design Specifications for the various load combinations to design the structural elements for the worst-case loading that may occur during the life of the structure. The more stringent design criteria requirements were applied in the event of conflicting code requirements.

The vertical and lateral loads produced by the new 24-inch waterline and supports were provided to the pre-manufactured bridge deck supplier (Acrow) to be incorporated into the bridge superstructure design.

The unit weights shown in Table 2-7 were utilized to determine the factored loads for the waterline support and abutment designs.

Table 2-7. Unit Weights of Materials

Unit Weights	
Reinforced Concrete	150 pcf
Soil	125 pcf
Steel Pipe Shell (Waterline)	490 pcf
Water	62.4 pcf

2.7.4.1 Dead Load (DL)

Dead loads consist of the weight of all permanent materials of construction incorporated into the Project, including self-weight and superimposed loads. The weight of the full, 24-inch diameter waterline was included as a dead load in the abutment design, adding approximately 38 kips to each abutment.

The abutment designs include the self-weight of the abutment and soil, in addition to all dead loads from the bridge superstructure. Reactions of 210 kips/corner, provided by Acrow, include the waterline, superstructure components, an epoxy deck coating, and the guardrail.

2.7.4.2 Live Load (LL)

Live loads consist of any loads produced by the use of the structure and do not include environmental loads. The bridge superstructure was designed and to support all live loads, , including the minimum vehicular live load (HL-93) required by AASHTO. Reactions of 165 kips/corner, provided by Acrow, include the maximum of two lanes of HL-93 loading, or one lane of HL-93 in conjunction with one lane of emergency vehicle (EV-2) loading.

In addition to the live load reactions produced by the superstructure, a live load surcharge due to a 32-kip axle load (design truck per AASHTO Section 3.6.1.2.2) as well as a braking force of 25% of the axle weight (AASHTO Section 3.6.4) were also included in the abutment designs.

2.7.4.3 Snow Load (SL)

Siskiyou County requires a minimum uniform roof snow load of 40 psf, with no reductions, based on the region. A uniform snow load of 40 psf was considered in the superstructure design in accordance with AASHTO. The snow load reactions from the superstructure are only considered under the Extreme Event II load combination in the abutment design.

2.7.4.4 Lateral Loads - Wind (WL)

Lateral forces due to wind on the waterline were determined based on ASCE 7 Chapter 26 for the waterline support design. A design wind speed (V) of 115 miles per hour, as required by Siskiyou County, was used to determine the velocity pressure. An Exposure Category of C were determined based on the location of the bridge in conjunction with the appropriate surface roughness category. The velocity pressure of 27.34 psf utilized was calculated from ASCE 7-16 Equation 26.10-1:

$$q_z = 0.00256 * K_z * K_{zt} * K_d * K_e * V^2$$

Table 2-8 provides additional information used to determine the velocity pressure.

Table 2-8. Wind Load Factors

Variable	Description	Value
K_z	Velocity Pressure Exposure Coefficient, Section 26.10.1	0.85
K_{zt}	Topographic Factor, Section 26.8.2	1.0
K_d	Wind Directionality Factor, Section 26.6	0.95
K_e	Ground Elevation Factor, Section 26.9	1.0

A lateral reaction of 86 kips per abutment, provided by Acrow, was incorporated into the design of the abutments, based on AASHTO requirements. The wind loads applied from the bridge superstructure, combined with the reactions from the waterline, were compared to the seismic loads required by AASHTO, with the larger forces governing the design based on load combinations applied.

2.7.4.5 Lateral Loads - Seismic (EQ)

Vertical and lateral seismic forces applied to the waterline were determined based on the requirements of ASCE 7 Chapter 13, utilizing information provided by the geotechnical data. The full waterline weight was used to determine the vertical and lateral seismic loads, producing the maximum seismic forces applied to the pipe support framing. The vertical seismic force was calculated based on Section 13.3.1.2, which indicates a concurrent vertical seismic force of $\pm 0.2S_{DS}W_p$. The horizontal seismic design force was determined from ASCE 7-16 Equation 13.3-1:

$$F_p = \frac{0.4 * a_p * S_{DS} * W_p}{\frac{R_p}{I_p}} * \left(1 + 2 * \frac{z}{h}\right)$$

Table 2-9 provides additional information used to determine the horizontal seismic force.

Table 2-9. Seismic Load Factors for Waterline

Variable	Description	Value
a_p	Component Amplification Factor, Table 13.6-1	2.5
R_p	Component Response Modification Factor, Table 13.6-1	6.0
I_p	Component Importance Factor, Section 13.1.3	1.5
S_{DS}	Spectral Acceleration, Short Period, Section 11.4.5	0.594
W_p	Component Operating Weight (Full Pipe Assumed)	294 plf
z	Height in Structure of Point of Attachment	-
h	Average Roof Height of Structure	-

Per ASCE 7 Section 13.3.1.1, the value of z/h does not need to exceed 1.0, therefore, 1.0 was conservatively used to determine the horizontal seismic force of 190 plf. A vertical seismic force of 50.5 plf was also determined using the above values.

Although the bridge superstructure was determined to be in Seismic Zone 3, a dynamic seismic analysis of the superstructure is not required for single span bridges per AASHTO Section 4.7.4.2. However, the bridge superstructure, abutments, and connections were designed for the minimum seismic forces and displacements as required by AASHTO based on the seismic zone on span. The abutments were designed to accommodate the seismic loads shown in Table 2-10, determined from information obtained from the geotechnical data and the below equations from AASHTO Sections 11.6.5 and 11.6.5.3:

$$Equation\ 11.6.5.1-1{:}\ P_{IR} = k_h * (W_w + W_s)$$

$$Equation\ 11.6.5.3-2{:}\ P_{AE} = 0.5 * \gamma * h^2 * K_{AE}$$

Table 2-10. Seismic Load Factors for Abutments

Variable	Description	Value
P_{AE}	Dynamic Lateral Earth Pressure Force, Section 11.6.5.3	2.52 klf
P_{IR}	Horizontal Force Due to Seismic Loading of Wall Mass, Section 11.6.5	0.573 klf
k_h	Seismic Horizontal Acceleration Coefficient, Section 11.6.5.2	0.04
W_w	Weight of Wall	10.64 klf
W_s	Weight of Soil Immediately Above Wall	3.75 klf
K_{AE}	Seismic Active Earth Pressure Coefficient	0.28
γ	Unit Weight of Soil	125 pcf
h	Total Wall Height	12 ft

A lateral seismic reaction of 54 kips per abutment was provided from Acrow. The lateral seismic load was compared to the wind loads required by code, with the larger forces governing the design based on load combinations applied.

2.7.4.6 Earth Pressures (ES, EH)

The abutments were designed for vertical and lateral loads due to soil backfill acting on the structure. The abutments were analyzed based on full height of soil over the heel and 3 feet over the toe. Lateral soil loads were determined based on information obtained from the geotechnical data, assuming at-rest conditions, in conjunction with AASHTO Equation 3.11.5.1-1:

$$P = k_o * \gamma * h^2$$

The at-rest coefficient of lateral earth pressure, k_o, was calculated as 0.426 using an angle of internal friction of 35 degrees ($\emptyset_f$), which was determined based on the soil profile documented in the Geotechnical Report. AASHTO Equation 3.11.5.2-1 was used to determine k_o:

$$k_o = 1 - sin\emptyset_f$$

Due to the triangular loading of lateral earth pressure acting on the abutment, the resultant lateral earth load due to the weight of the backfill is applied at a height of H/3, where H is the wall height from bottom of footing to top of wall. The vertical earth pressures act uniformly over the length of the heel and the toe.

3.0 Project Description

3.1 General Site Layout

The general site layout is depicted in Figure 3-1 and shows the major components of the proposed Daggett Bridge improvements. The new improvements include construction of a new roadway off of the existing Daggett Road and a new single span bridge deck that will be located just upstream of the existing Daggett Bridge. The City of Yreka's new 24-inch diameter steel pipe will be supported along the new Daggett Bridge across the Klamath River. The design specifics are presented in the following sections within this report.

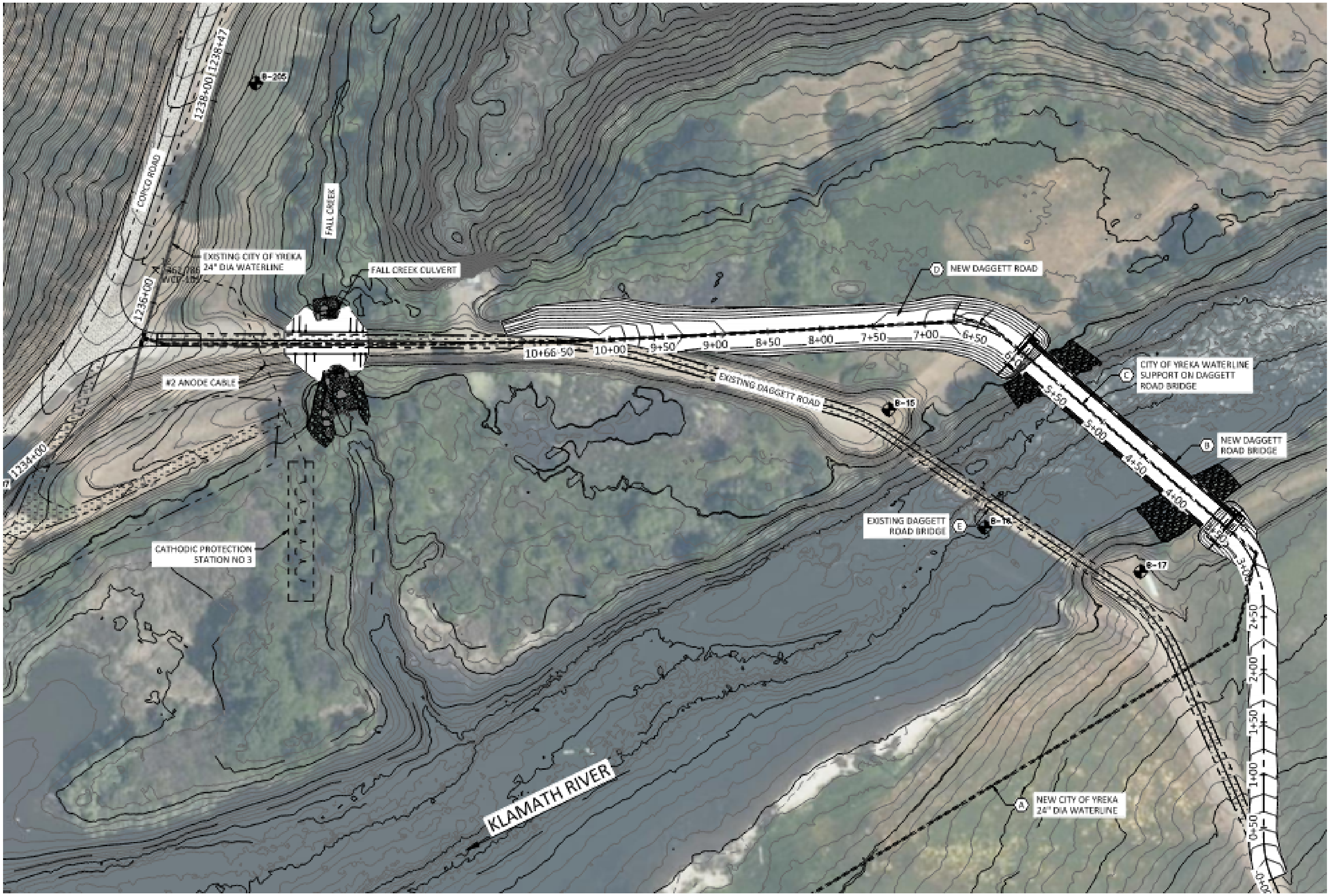

Figure 3-1. General Site Layout

4.0 Civil Design

4.1 General Description

This section presents the civil design elements for the Project.

4.2 Design Criteria

For the civil design criteria see Section 2.4.

4.3 Erosion and Sediment Control

The Contractor is required to install, monitor, and maintain erosion and sediment control measures as identified within the Project Drawings, and prepare the required documents discussed in Section 2.4 as determined by the various regulatory agencies. The erosion control measures shall be maintained for the duration of the construction project. The Contractor will be required to install specified permanent post-construction measures as required for the Project. The permanent measures are designed to protect the exposed slopes until the vegetation is fully established. Following construction, the disturbed areas of the Project site will be revegetated with native plant mixes, or riprap to protect the Klamath River banks and the new bridge abutments. The Contractor will be required to submit a Notice of Termination (NOT) to the State Water Resources Control Board (SWRCB) after completing the Project. This is required to be relieved from the Construction General Permit requirements. Final soil stabilization throughout the proposed Project area must be achieved prior to the SWRCB approval of the NOT.

4.4 Roadway Design

The new Daggett Bridge will be accessed by a new gravel road from the existing Daggett Road on the north side of the Klamath River with multiple slopes and vertical curves to the bridge deck and will slope back up to connect back into the existing Daggett Road on the south side of the Klamath River with multiple slopes and vertical curves. The deck elevation 2342.60 ft is 8.8 feet above the post drawdown 100-year elevation of 2333.8 ft. giving 8.8 ft of freeboard during a 100-year storm. The road will be crowned to drain freely to a borrow ditch on both sides of the road. The vegetation within the borrow ditch will serve as a water quality filter, removing particulates prior to infiltrating or flowing back toward the Klamath River.

The new Daggett Road will be gravel road to match the existing road. The roadway section will be constructed of native material topped with a 6" Class II Aggregate Base on top of a 12" layer of Structural fill per Spec 31 05 00 prepared by Knight Piésold Consulting. During construction, an area located to the north of the Klamath River and new Daggett Bridge location will provide a staging area for equipment and materials required to complete construction of the new Daggett Road and Bridge. There is also a staging area located to the south of the Klamath River where the proposed bridge will be built and launched from.

5.0 Hydrologic and Hydraulic Design

5.1 Annual Peak Floods at Iron Gate

Knight Piésold Consulting and Kiewit (2020a) analyzed the annual peak floods for the Klamath River Renewal Project, 100% Design Report, Appendix A-6. They analyzed the historic USGS data and the 2019 BiOp data. The annual peak flood values selected are shown in Table 5-1.

Table 5-1. Annual Peak Floods at Iron Gate.

Probability (%)	Return Period	Flow (cfs)
50%	2-yr	7,500
20%	5-yr	10,900
10%	10-yr	14,900
5%	20-yr	19,300
2%	50-yr	25,700
1%	100-yr	31,200
0.5%	200-yr	37,100
0.2%	500-yr	45,800

Source: Table 3.2, Appendix A-6 (Knight Piésold Consulting and Kiewit, 2020)

5.2 Water Surface Elevation Analysis

The HEC-RAS model, developed by Knight Piésold Consulting, was used to look at the river hydraulics at the proposed bridge location. River Station 491023 was used, as it was closest to the proposed location of the new Daggett Bridge. The HEC-RAS model was run under existing conditions in steady state with the same flows identified in Table 5-1. The pre-drawdown model results are summarized in Table 5-2.

Table 5-2. Pre-Drawdown Water Surface Elevations (HEC-RAS Sta 491023).

Annual Exceedance Probability (%)	Return Period	Flow (cfs)	W.S. Elevation (ft)	Channel Velocity (ft/s)
10%	10-yr	14,900	2334.2	8.2
5%	20-yr	19,300	2336.8	8.7
2%	50-yr	25,700	2339.2	10.0
1%	100-yr	31,200	2341.1	10.9
0.5%	200-yr	37,100	2342.4	11.8
0.2%	500-yr	45,800	2343.8	13.3

The HEC-RAS model was slightly modified by removing the Iron Gate Dam and running the model in steady state with the same flows. The post-drawdown model results are summarized in Table 5-3.

The 100-year water surface elevation (post-drawdown) was used as the basis to set the new Daggett Bridge deck elevation to provide at least 1 ft of freeboard below the waterline supports during a 100-year flood.

Table 5-3. Post-Drawdown Water Surface Elevations (HEC-RAS Sta 491023).

Annual Exceedance Probability (%)	Return Period	Flow (cfs)	W.S. Elevation (ft)	Channel Velocity (ft/s)
10%	10-yr	14,900	2330.1	12.4
5%	20-yr	19,300	2331.7	13.4
2%	50-yr	25,700	2333.7	14.8
1%	100-yr	31,200	2335.1	16.0
0.5%	200-yr	37,100	2336.5	17.1
0.2%	500-yr	45,800	2338.1	18.9

5.3 Proposed Daggett Bridge

The HEC-RAS model, developed by Knight Piésold Consulting, was used to look at the river hydraulics at the proposed Daggett bridge location and analyze the impacts to the river flow to evaluate the potential scour and required revetment size to ensure the stability of the bridge abutments. The HEC-RAS model was slightly modified by removing the Iron Gate Dam and adding the proposed bridge location. The proposed bridge spans the river without any piers or other obstructions. The water surface elevations and the channel velocities upstream of the proposed Daggett bridge location are shown in Table 5-4.

Table 5-4. Water Surface Elevations Upstream of Proposed Daggett Bridge.

Annual Exceedance Probability (%)	Return Period	Flow (cfs)	W.S. Elevation Upstream of Bridge (ft)	Channel Velocity Upstream of Bridge (ft/s)
10%	10-yr	14,900	2328.6	15.1
5%	20-yr	19,300	2330.1	16.0
2%	50-yr	25,700	2332.4	16.7
1%	100-yr	31,200	2333.8	17.7
0.5%	200-yr	37,100	2336.7	17.5
0.2%	500-yr	45,800	2344.7	16.2

5.4 Bridge Scour Analysis

The scour depth estimate is based on an equation provided by Julien (2002), where the scour depth downstream of a grade-control structure is evaluated with the drop height being set to zero. The scour depth estimation is based on the riverbed particle size, or grain size representing fine sand. The riverbed grain size was obtained from the description of the river alluvium as recorded in the log of soil and core borings reported by CDM Smith (2020a). The calculated potential scour depth for the 1% AEP flood is 2.8 feet. The scour depth calculations can be found in Appendix B.

5.5 Revetment Stone Sizing

The revetment size was calculated using the California Bank and Shore (CABS) method. The method consists of one or more layers of Rock Slope Protection (RSP), placed along the streambank at the abutments of the bridge to prevent erosion. The revetment sizing has been chosen to be the same for each abutment, where the revetment size is based on the largest calculated rock rise to maintain bank stability. The typical revetment slope protection would consist of a large-sized outside rock, a smaller-sized inner rock, and then a geotextile fabric. The slope revetment is intended to be flexible, where the rock may move without necessarily compromising the stability of the entire bank. The calculated revetment size for the left and right abutment are shown in Table 5-5. The right bank, or North abutment velocity multiplier was increased by 20 percent to account for the thalweg being located directly next to the bank. The left bank, or South abutment revetment size was increased to match the North abutment for installation simplification. Figure 5-1 shows a typical layout for revetment placement for the right bank or North abutment. The revetment stone sizing calculations can be found in Appendix B.

Table 5-5. Revetment Sizing for Bridge Abutments.

Description	South Abutment	North Abutment
Slope Angle	15 deg	28 deg
Outside Layer, RSP-Class	E7B	E7B
Minimum Layer Thickness (d)	3.3 ft	3.3 ft
Backing Layer, RSP-Class	E6	E6
Minimum Layer Thickness	1.25 ft	1.25 ft
RSP-Fabric Type	12 oz Nonwoven	12 oz Nonwoven
Total Rock Thickness (Perpendicular)	4.6 ft	4.6 ft

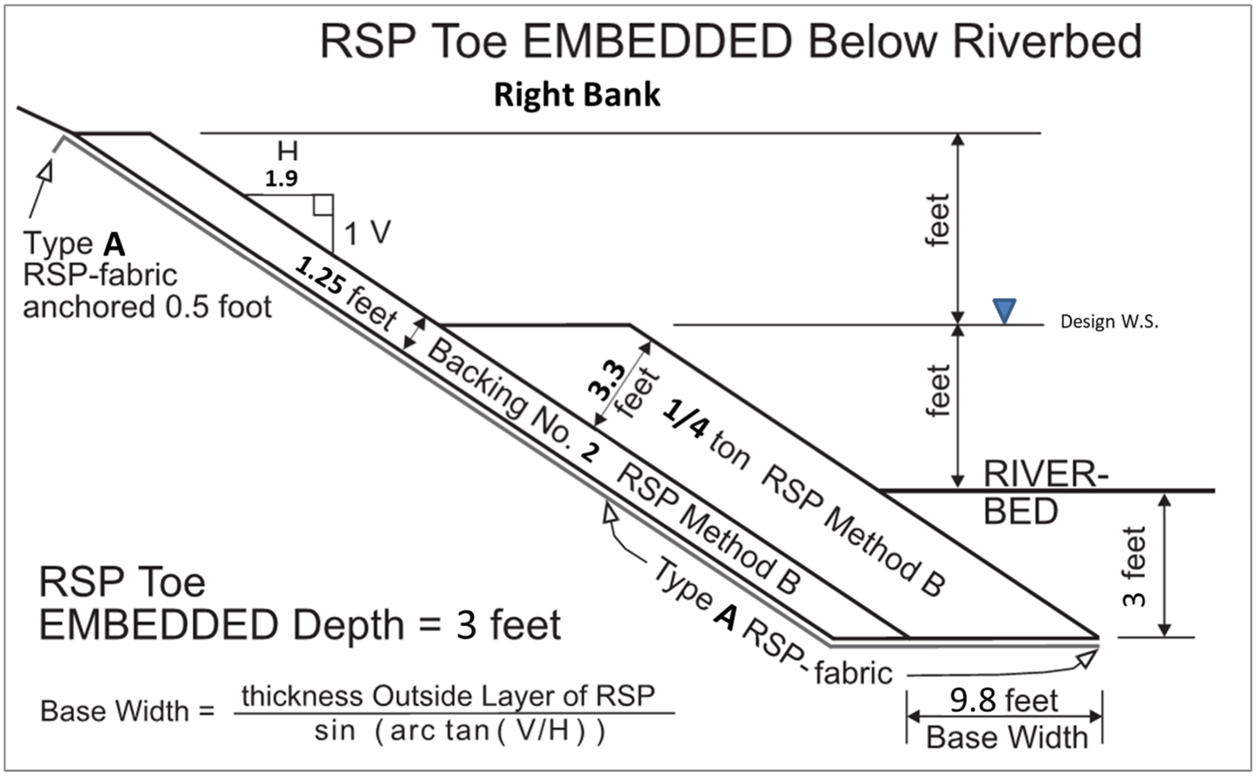

Figure 5-1. Typical Revetment Placement for Right Abutment.

6.0 Geotechnical Design

6.1 Geotechnical Investigations

CDM Smith and AECOM Technical Services, Inc. prepared a Geotechnical Data Report for KRRC in June 2019. Three (3) borings, B-15, B-16, and B-17 were drilled in the vicinity of the Daggett Bridge location. All three boring were drilled by Taber Drilling. Boring B-15 and B17 were drilled with a truck mounted CME-75 drill rig and boring B-16 was drilled over water with a barge mounted CME-45 drill rig. All borings were advanced with rotary wash, HQ-3 rock core methods. The borings reached depths of 51.5 feet (B-15), 24.5 feet (B-16), and 41.5 feet (B-17) below ground surface. Borings are shown relative to the existing Daggett Bridge location in Figure 3-1.

Borings B-15 and B-17, drilled adjacent to the existing bridge abutments, encountered localized fill near the ground surface. The fill is sandy lean clay with gravel to gravelly clay with sand (CL), stiff to very stiff, approximately 8 feet in depth. Underlying the fill in Boring B-15 is very dense clayey gravel with sand (GC) with a thickness of 9 feet, basalt boulders and cobbles in a sand and gravel matrix with a thickness of 11 feet, followed by a volcaniclastic breccia to the total depth of the boring. Underlying the fill in Boring B-17 is a very dense sandy gravel (GP) with a thickness of 4 feet, followed by the volcaniclastic breccia to the total depth of the boring.

Boring B-16, drilled over water near approximately the mid-point of the bridge, encountered the same volcaniclastic breccia from the ground surface to the entire depth of the boring.

6.2 Geotechnical Conditions

Borings B-15 and B-17 are used to develop the geotechnical conditions for the new bridge abutments and foundations. For design purposes the very dense Clayey Gravel with Sand encountered in boring B-15 was conservatively used for the full design section of both bridge abutments. This material has an $N1_{60}$ value greater than 50. Based on one sieve analysis it consists of 42% gravel seized particles, 27% sand, and 31% fines (passing the #200 sieve). The moist unit weight is 125 pounds per cubic foot (pcf). The design friction angle is conservatively taken as 35 degrees and includes a 3-degree reduction to account for the significant fines content of the material. For bearing capacity calculations the cohesion is conservatively taken as 0. For other calculations it is estimated to be 250 psf. Modulus is estimated to be 10 ksi. Poisson's ratio is estimated to be 0.4.

6.3 Bearing Capacity

The bearing capacity was calculated for shallow footings following the methodology in AASHTO for a footing that is 12-ft wide, by 37-ft long. It is assumed that the footing depth is between 7 to 10 feet below the existing ground surface. Ground water condition is assumed to be at the ground surface (i.e. flood conditions). Based on the geometry provided in Drawing C202 a slope reduction factor of 0.69 was calculated. A resistance factor of 0.45 was used, consistent with a theoretical method using STP values. A factored resistance q_R=9.2 ksf was calculated for vertical concentric loads.

6.4 Settlement

Settlement of the bridge foundations was checked following AASHTO methodology. The granular material at the bridge site is not anticipated to experience consolidation settlement. Per AASHTO Section 10.6.2.3 Service Load I is used for settlement calculations. Based on a distributed load of 4.05 ksf over a 12-ft x 37-ft footing, elastic settlements up to 0.52-inches could be seen during construction at each foundation.

6.5 Slope Stability

Global stability of the bridge footing was checked using the limit equilibrium slope stability program Slide2 by Rocscience. The higher and steeper NW bridge abutment was analyzed with an infinite, 2D slope using the Bishop method. The entire subsurface profile was conservatively modeled with the very dense clayey gravel with sand, with a friction angle of 35 degrees, a cohesion of 250 psf, and a unit weight of 125 pcf. The approximately 5 foot high approach fill was conservatively modeled as a distributed load of 450 psf in order to neglect the strength of the fill. The factor of safety with a 4.05 ksf distributed load over a 12-ft wide footing is 1.7. A pseudo-static analysis was performed to model slope stability in a seismic situation. A horizontal acceleration of 0.13, corresponding to half the site PGA was used in the analysis. A pseudo-static factor of safety FS=1.3 was calculated.

7.0 Structural Design

7.1 Bridge Superstructure and Abutments

The single span, pre-engineered bridge superstructure design, provided by Acrow, accommodates vertical and lateral loads required by AASHTO, in addition to loads from equipment required for the dam removal project, and emergency vehicles. The superstructure is supported by non-integral, reinforced concrete seat-type abutments at each end of the bridge, on the North and South sides of the Klamath River. The superstructure is attached to each abutment via bearing pads and post-installed anchors.

The vertical and lateral reactions at each pipe support location were provided to Acrow and incorporated into the superstructure design.

7.1.1 Design Loads

An analysis of the superstructure by Acrow produced the unfactored reactions shown in Table 7-1 and were incorporated into the abutment (substructure) design.

Table 7-1. Superstructure Loads Provided by Acrow

Load Case	Load	Description
Dead Load	210 kips	Total self-weight of superstructure and components, including wearing surface
Live Load	165 kips	Maximum of two lanes of concurrent HL-93 loading, or one lane loaded by an emergency vehicle concurrent with one lane of HL-93 loading
Wind Load	86 kips	Wind load determined from wind pressure and exposed area acting on superstructure
Seismic Load	54 kips	Seismic load determined from product of acceleration coefficient and the tributary permanent load (dead load)

Additional load cases and associated components considered in the substructure design include:

Dead Loads (DL):

- Self-weight of bridge superstructure and components.
- Self-weight of reinforced concrete abutment.
- Self-weight of full, 24-inch diameter steel pipe (waterline).

Live Loads (LL and LS):

- Both lanes loaded with HL-93, or a single lane loaded with HL-93 and a single lane loaded with EV-2.
- Braking force taken as the maximum of 25% of the axle weight of the design truck or tandem, or 5% of the design truck plus lane load, or 5% of the design tandem plus lane load.
- Vertical surcharge from vehicle loading acting on surface of backfill (LS).
- Lateral force due to vehicle surcharge

Snow Load (SL)

- 40 psf uniform load over bridge superstructure.

Wind Loads (WL)

- ASCE 7 specified wind load applied to waterline.

Seismic Loads (EQ)

- Vertical seismic force applied from the bridge superstructure taken as 25% of the tributary permanent load (dead load).
- Lateral seismic force applied from the bridge superstructure taken as the product of the acceleration coefficient, A_s, and the tributary permanent load (dead load).
- Lateral seismic force due to the dynamic lateral earth pressure force.
- Lateral seismic force induced by the self-weight of the substructure.

Earth Pressures (ES, EH)

- Vertical earth pressures due to soil over toe and heel.
- Lateral earth pressure due to backfill.

7.1.2 Substructure Design

The abutments were designed to resist the vertical and lateral loads from the bridge, in addition to vertical and lateral surcharge loads due to vehicles. The abutments also account for the self-weight of concrete and vertical and lateral loads due to soil, and seismic or wind forces. Each load was factored and combined as required per AASHTO Table 3.4.1-1, with the worst-case combined loading governing the abutment design. The load factors for each combination utilized in the design are indicated in Table 7-2.

Table 7-2. Load Combinations and Load Factors

Load Combination Limit State	D	ES	EH	LL	WL	EQ	IC
Strength I	1.25	1.50	1.35	1.75	-	-	-
Strength II	1.25	1.50	1.35	1.35	-	-	-
Strength III	1.25	1.50	1.35	-	1.0	-	-
Strength IV	1.50	1.50	1.35	-	-	-	-
Strength V	1.25	1.50	1.35	1.35	1.0	-	-
Extreme I	1.0	1.0	1.0	0.5	-	1.0	-
Extreme II	1.0	1.0	1.0	0.5	-	-	1.0
Service I	1.0	1.0	1.0	1.0	1.0	-	-
Service II	1.0	1.0	1.0	1.3	-	-	-

The abutments are comprised of a 6’-0” wide body, including a 2’-3” thick, approximately 3’-3” tall back wall and are supported by a 2’-0” thick spread footing, approximately 3’-0” below existing grade. The back wall transitions to 1’-0” thick at the returns to minimize materials and provide some cost savings. As required by AASHTO Sections 11.5.3 and 11.5.4, the abutment designs included evaluations at the Strength Limit and Extreme Limit states for the following:

- bearing resistance failure,
- lateral sliding,
- loss of base contact due to eccentric loading and
- structural failure

The Extreme and Service Limit states were also evaluated for overall stability failure. Each limit state evaluated satisfied the AASHTO requirements of Equation 1.3.2.1-1:

$$\Sigma\eta_i\gamma_i Q_i \leq \emptyset R_n$$

where:

η_i = load modifier relating to ductility, redundancy, and operational classification
γ_i = load factor: a statistically based multiplier applied to force effects
Q_i = force effects
R_n = nominal resistance
$\emptyset$ = resistance factor: a statistically based multiplier applied to nominal resistance

The load factors applied in the above equation for each limit state are shown in Table 7-2. The Strength I load combination controlled the abutment designs over the Service and Extreme Limit states. The abutments were sized to limit eccentricity and to keep the resultant within the middle third, providing the most cost-effective design.

7.2 New Waterline Support

The new 24-inch proposed waterline is to be constructed as a minimum 0.750-inch-thick steel pipeline and will be located along the downstream of the new bridge. The pipeline will transition from a buried pipeline, through the concrete abutments and will span across the Klamath River. Expansion joints in the pipeline exist adjacent to each abutment. The bottom elevation of the pipe supports are 2342.91 ft is 9.1 feet above the post drawdown 100-year elevation of 2333.8 ft. giving 9.1 ft of freeboard during a 100-year storm.

Structural support locations for the waterline occur every 10’-0” and were determined based on the spacing of the pre-manufactured bridge deck transom beams. The transom beams occur at 10’-0” on center and are shallow enough to locate the pipe below the members while still providing adequate attachment surfaces and required freeboard. The main bridge girders are relatively deep members and can provide some shelter for the new waterline from environmental impacts.

7.2.1 Design Loads

The pipe was analyzed for gravity and lateral loads, including wind and seismic. The supports were designed for the following loads cases:

Dead Loads (DL):

- Self-weight of full, 24-inch diameter steel pipe (waterline).

Wind Loads (WL)

- ASCE 7 specified wind load applied to waterline.

Seismic Loads (EQ)

- ASCE 7 specified seismic loads applied to waterline.

Table 7-3. Unfactored Loads

Load Case	Load	Description
Dead Load	294 plf	Total self-weight of 24-inch diameter waterline and water
Wind Load	282 plf	Uniform lateral wind load applied to pipe
Seismic Load	131 plf	Uniform lateral seismic load applied to pipe
Seismic Load	35 plf	Uniform ± vertical seismic load applied to pipe

The loads shown in Table 7-3 were determined using the ASCE 7 equations and coefficients found in Section 2.7.4.4 for wind and 2.7.4.5 for seismic forces.

7.2.2 Waterline Support Design

The steel support components were designed per the steel design code, AISC, which references the load combinations from ASCE 7 to determine the required strength. The design forces applied to the waterline supports were determined using ASCE 7, with applicable load factors conforming to the strength design (LRFD) methodology. Table 7-4 provides the load combinations and associated factors considered. The variable "E" represents calculated seismic forces with the subscripts "v" and "h" denoting vertical and horizontal forces, respectively.

Table 7-4. Load Combinations per ASCE 7 Section 2.3

Load Combination Number	Load Combination
1	1.4*DL
2	1.2*DL + 1.6*LL
3	1.2*DL + LL or 0.5*WL
4	1.2*DL + 1.0*WL + LL
5	0.9*DL + 1.0*WL
6	1.2*DL + 1.0*E_v + 1.0*E_h + LL
6	0.9*DL - 1.0*E_v + 1.0*E_h + LL

The Strength and Serviceability Limit states for each load combination was evaluated, and the supports were sized to satisfy the AISC requirements of Equation B3-1:

$$R_u \leq \emptyset R_n$$

where:

R_u = required strength using LRFD load combinations
R_n = nominal strength

Ø = resistance factor

The waterline support consists of two hanging wide-flange columns, welded to a plate, which is clamped to the bottom of the transom beam. The hanging columns are supported by the same transom beam and are spaced at 5’-0” transversely, allowing space to construct the connections and adequate clearance for the pipe.

A stiffened plate with a curved seat directly supports the waterline and is bolted via shear tab to each hanging column, serving as the pipe support, and transferring vertical and lateral forces to the hanging columns. The hanging columns will act as tension/compression members to resist the lateral (wind and seismic) forces perpendicular to the pipe span, which will avoid inducing torsion on the transom beams.

Angle braces attached to the bottom of the hanging members and extending up to each adjacent transom beam will resist the lateral (seismic and friction) forces parallel with the pipe span, eliminating torsion on the transom beams. The braces will attach to a plate that is welded on the outside face of the hanging columns at the bottom. The brace to plate connection can be bolted or welded, allowing flexibility in the field to install as the bridge is launched. At the transom beam, the flange of a WT member is bolted to the underside of the transom beam, extending the web down. The braces will attach to the WT web via bolt or weld, allowing for flexibility in the field to install as the bridge is launched.

8.0 References

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Appendix A
Civil Design Calculations

Calculation Cover Sheet

McMILLEN JACOBS ASSOCIATES

LICENSED PROFESSIONAL ENGINEER
JEFFREY ALAN LOWY
C 79836
6/24/2022
CIVIL
STATE OF CALIFORNIA

Project: Daggett Bridge Design Project

Client: Kalamath River Renewal Corporation **Proj. No** 21-067

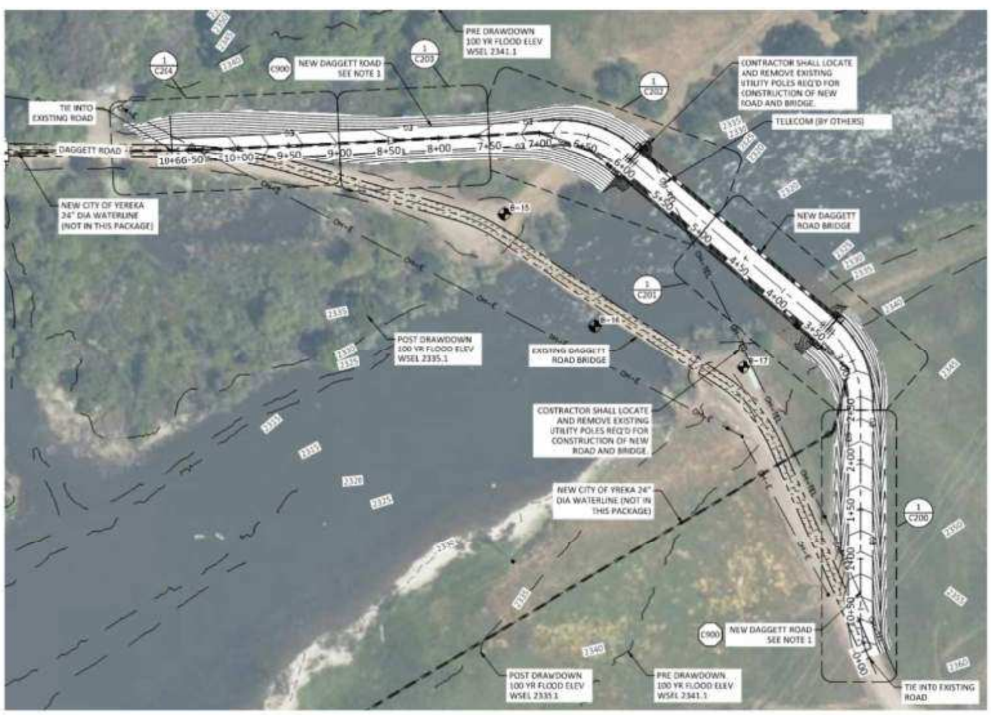
Title: Civil Design Calculations

Prepared By, Name: Ryan Hudson

Prepared By, Signature: Ryan Hudson **Date:** 5/27/2022

Peer Reviewed By, Name: Jeff Lowy

Peer Reviewed, Signature: Jeff Lowy **Date:** 5/27/2022



SUBJECT:	Kalamath River Renewel Cooperation	**BY:** R. Hudson	**CHK'D BY:** J. Lowy
	Daggett Bridge Design	**DATE:** 5/27/2022	
	Civil Design Calculations	**PROJECT NO.:** 21-067	

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EXAMPLE

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- *Vertical Curve Calculations*

SUBJECT:	Kalamath River Renewel Cooperation	**BY:** R. Hudson	**CHK'D BY:** J. Lowy
	Daggett Bridge Design	**DATE:** 5/27/2022	
	Civil Design Calculations	**PROJECT NO.:** 21-067	

Purpose

The purpose of this cacualtion sheet is to show the vertical curves of the proposed Daggett Road meet the AASHTO vertical curve calculations.

References

• *AASHTO. 2018. A Policy on Geometric Design of Highways and Streets 7th Edition. Washington, DC..*

Equations

The following equations were used to calculate vertical curve information for the proposed Daggett Road.

Table 1 - Minimum K value for Crest Curves

U.S. Customary			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, Ka	
		Calculated	Design
15	80	3.0	3
20	115	6.1	7
25	155	11.1	12
30	200	18.5	19
35	250	29.0	29
40	305	43.1	44
45	360	60.1	61
50	425	83.7	84
55	495	113.5	114
60	570	150.6	151
65	645	192.8	193
70	730	246.9	247
75	820	311.6	312
80	910	383.7	384

Table 2 - Minimum K Value for Sag Curves

U.S. Customary			
Design Speed (mph)	Stopping Sight Distance (ft)	Rate of Vertical Curvature, K^a	
		Calculated	Design
15	80	9.4	10
20	115	16.5	17
25	155	25.5	26
30	200	36.4	37
35	250	49.0	49
40	305	63.4	64
45	360	78.1	79
50	425	95.7	96
55	495	114.9	115
60	570	135.7	136
65	645	156.5	157
70	730	180.3	181
75	820	205.6	206
80	910	231.0	231

The proposed speed limit for Dagget Road around the proposed bridge lcoation is 15mph. From Tables 1 and 2, this corresponds to a minimum K value of 3 for crest curves and 10 for sag curves. The following equations will be used to calculate the K value of each proposed curve and checked againt the minimum K value for the 15 mph speed limit.

Rate of Curvature

$$K = \frac{L}{A}$$

Where:
L = Length of vertical curve, ft
A = Algebraic difference between grades, %

Parabolic Constant

$$e = \frac{A}{2L}$$

Where:
L = Length of vertical curve, ft
A = Algebraic difference between grades, %

Elevation of Curve at PVI

$$PVI_{\ Elevation} = PVC_{Elevation} + (g_1 * \frac{L}{2})$$

$$PVT_{\ Elevation} = PVI_{Elevation} + (g_2 * \frac{L}{2})$$

$$Curve\ Elevation_{PVC-PVT} = \frac{PVC_{Elevation} + PVT_{Elevation}}{2}$$

$$Curve\ Elevation_{PVI} = \frac{Curve\ Elevation_{PVC-PVT} + PVT_{Elevation}}{2}$$

Where:

$PVC_{Elevation}$ = Elevation of the point of vertical curvature (PVC), ft
$PVI_{Elevation}$ = Elevation of the point of vertical inflection (PVI), ft
$PVT_{Elevation}$ = Elevation of the point of vertical tangency (PVI), ft
Cuve $Elevation_{PVC\text{-}PVT}$ = Elevation of the vertical curve at the midpoint of the chords PVC-PVT, ft
Curve $Elevation_{PVI}$ = Elevation of the vertical curve at the PVI station, ft
g_1 = Grade into curve, %
g2 = Grade out of curve, %

Curve Station 0+00 to 0+20

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	0+10.00
Elevation of PVI:	2355.35
Elevation of PVC:	2355.35
Length of Vertical Curve:	20.00
Grade into Curve (g1)	0.00%
Grade out of Curve (g2)	-4.47%

Calculated Variables	
Algebraic Difference Between Slopes (G):	-4.47%
Design K Value for the Crest Vertical Curve:	3
K Value of Prop. Curve:	4.5
e Value:	-0.11
Elevation of PVI:	2355.35
Elevation of PVT:	2354.90
Curve $Elevation_{PVC\text{-}PVT}$:	2355.13
Elevation of Curve at PVI:	2355.01
Sta of Point of Vertical Curvature (PVC):	0+00.00
Elevation of PVC:	2355.35
Sta of Point of Vertical Tangency (PVT):	0+20.00

Curve Station 2+70 to 3+20

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	2+95.00
Elevation of PVI:	2342.60
Elevation of PVC:	2343.72
Length of Vertical Curve:	50.00
Grade into Curve (g1)	-4.47%
Grade out of Curve (g2)	0.00%

Calculated Variables	
Algebraic Difference Between Slopes (G):	4.47%
Design K Value for the Crest Vertical Curve:	10
K Value of Prop. Curve:	11.2
e Value:	0.04
Elevation of PVI:	2342.60
Elevation of PVT:	2342.60
Curve $Elevation_{PVC\text{-}PVT}$:	2342.60
Elevation of Curve at PVI:	2342.60
Sta of Point of Vertical Curvature (PVC):	2+70.00
Elevation of PVC:	2343.72
Sta of Point of Vertical Tangency (PVT):	3+20.00

Curve Station 6+46 to 6+86

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	6+66.00
Elevation of PVI:	2342.60
Elevation of PVC:	2342.60
Length of Vertical Curve:	40.00
Grade into Curve (g1)	0.00%
Grade out of Curve (g2)	-3.10%

Calculated Variables	
Algebraic Difference Between Slopes (G):	-3.10%
Design K Value for the Crest Vertical Curve:	3
K Value of Prop. Curve:	12.9
e Value:	-0.04
Elevation of PVI:	2342.60
Elevation of PVT:	2341.98
Curve $Elevation_{PVC\text{-}PVT}$:	2342.29
Elevation of Curve at PVI:	2342.14
Sta of Point of Vertical Curvature (PVC):	6+46.00
Elevation of PVC:	2342.60
Sta of Point of Vertical	6+86.00

Tangency (PVT): 0+80.00

Curve Station 7+30 to 7+70

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	7+50.00
Elevation of PVI:	2340.00
Elevation of PVC:	2340.62
Length of Vertical Curve:	40.00
Grade into Curve (g1)	-3.10%
Grade out of Curve (g2)	0.00%

Calculated Variables	
Algebraic Difference Between Slopes (G):	3.10%
Design K Value for the Crest Vertical Curve:	10
K Value of Prop. Curve:	12.9
e Value:	0.04
Elevation of PVI:	2340.00
Elevation of PVT:	2340.00
Curve Elevation$_{PVC-PVT}$:	2340.00
Elevation of Curve at PVI:	2340.00
Sta of Point of Vertical Curvature (PVC):	7+30.00
Elevation of PVC:	2340.62
Sta of Point of Vertical Tangency (PVT):	7+70.00

Curve Station 8+30 to 9+10

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	8+70.00
Elevation of PVI:	2340.00
Elevation of PVC:	2340.00
Length of Vertical Curve:	80.00
Grade into Curve (g1)	0.00%
Grade out of Curve (g2)	3.03%

Calculated Variables	
Algebraic Difference Between Slopes (G):	3.03%
Design K Value for the Crest Vertical Curve:	10
K Value of Prop. Curve:	26.4
e Value:	0.02
Elevation of PVI:	2340.00
Elevation of PVT:	2341.21
Curve Elevation$_{PVC-PVT}$:	2340.61
Elevation of Curve at PVI:	2340.91
Sta of Point of Vertical Curvature (PVC):	8+30.00
Elevation of PVC:	2340.00
Sta of Point of Vertical Tangency (PVT):	9+10.00

Curve Station 10+10 to 10+50

Input Vertical Curve Information	
Design Speed:	15 mph
Sta of Point of Vertical Inflection (PVI):	10+30.00
Elevation of PVI:	2344.85
Elevation of PVC:	2344.25
Length of Vertical Curve:	40.00
Grade into Curve (g1)	3.03%
Grade out of Curve (g2)	0.00%

Calculated Variables	
Algebraic Difference Between Slopes (G):	-3.03%
Design K Value for the Crest Vertical Curve:	3
K Value of Prop. Curve:	13.2
e Value:	-0.04
Elevation of PVI:	2344.86
Elevation of PVT:	2344.86
Curve Elevation$_{PVC-PVT}$:	2344.85
Elevation of Curve at PVI:	2344.85
Sta of Point of Vertical Curvature (PVC):	10+10.00
Elevation of PVC:	2344.25
Sta of Point of Vertical Tangency (PVT):	10+50.00

Conclusion

Using vertical curve equations provided by AASHTO, the K values for the 6 vertical curves of the proposed Daggett Road were calculated and compared to design K values of 3 and 10 for crest and sag curves, respectivly. The proposed curve K values are all greater than the design K values meeting the limits constraining the stopping sight distance.

Appendix B
Hydrologic & Hydraulic Design Calculations

Calculation Cover Sheet

Project: Daggett Bridge Design Project

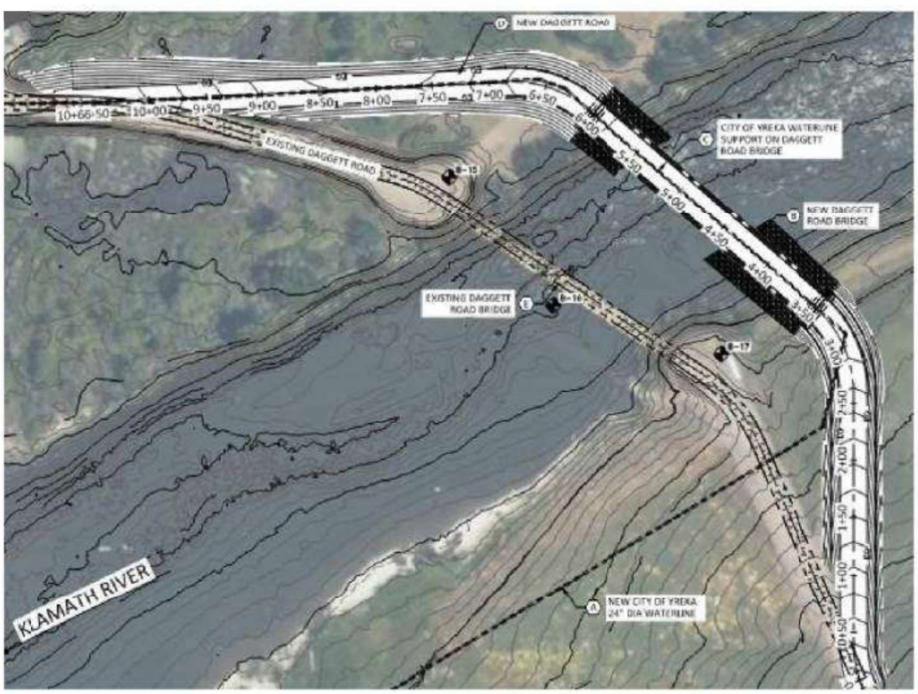
Client: Kalamath River Renewal Corporation **Proj. No**. 21-067

Title: Hydrologic and Hydraulic Design Calcs

Prepared By, Name: Nathan Cox

REGISTERED PROFESSIONAL ENGINEER
NATHAN C. COX
No. C68030
CIVIL
STATE OF CALIFORNIA

P.E.
Experation 6/30/2023
2022.04.28
13:52:21-06'00'



SUBJECT:	Kalamath River Renewal Corporation	BY: N. Cox	CHK'D BY:
	Daggett Bridge Design Project	DATE: 1/0/1900	
	Table of Contents	PROJECT NO.: 21-067	

Table of Content

Description **Page**

Scour Depth B3

- *Determine the Expected Scour at the Bridge*

CABS RSP Design B4

- *Determine the Rock Stone Size Slope Protection*

HEC-RAS Results B6

- *Results of HEC-RAS Simulations*

SUBJECT:	Klamath River Renewal Corp.	**BY:** N.Cox	**CHK'D BY:**	M.Cerucci
	Daggett Bridge Modification	**DATE:** 9/20/2021		
	Scour Depth Estimate	**PROJECT NO.:** 21-067		

Purpose

The purpose of this calculation sheet is to determine the expected scour depth at Bridge.

References

• *Julien, Pierre Y. (2002). River Mechanics. Cambridge University Press, Cambridge, United Kingdom.*

Equations

$$\Delta z = \left[1.8\left(\frac{\sin\phi}{\sin(\theta_j+\phi)}\right)^{0.8}\frac{q^{0.6}V_1\sin\theta_j}{[(G-1)g]^{0.8}d_s^{0.4}}\right]-D_p$$

Eqn. 9.6 (Julien, 2002)

Where:

- Δz = *Scour depth below the grade-control structure (m)*
- D_p = *Drop height of the grade control structure (m)*
- q = *Unit Discharge (m³/s /m)*
- V_1 = *Approach Velocity (m/s)*
- d_s = *Particle Size (m)*
- g = *gravitational constant (m/s²)*
- G = *Specific Gravity of bed Material*
- ϕ = *Angle of Repose of the Bed Material (degrees)*
- θ_j = *Jet Angle measured from the horizontal (degrees)*

Scour Depth Calculations

Unit Weight of Water, γ_w =	62.4	lbs/ft³	
Unit Weight of Stone, γ_s =	156	lbs/ft³	Assumed
Acceleration of Gravity, g =	32.2	ft/s²	
Flow , Q =	31,200	ft³/s	
Average Channel Width, W =	167	ft	
Depth of Flow, d =	13.8	ft	HEC-RAS Model
Depth-Averaged Velocity, V =	17.7	ft/s	HEC-RAS Model
Acceleration of Gravity, g =	9.81	m/s²	
Specif Gravity, G =	2.50		
Angle of Repose, ϕ =	40	degrees	
Flow , Q =	883.5	m³/s	
Channel Width, W =	50.9	m	
Unit Discharge, q =	17.4	m²/s	
Approach Velocity, V_1 =	5.4	m/s	
Jet Angle, θ_j =	1:204	V:H	E.G. Slope of 0.0049
Jet Angle, θ_j =	0.28	degrees	
Drop Height, D_p =	0.0	ft	
Drop Height, D_p =	0.00	m	
Particle Size, d_s =	0.25	mm	Fine Sand (0.125 - 0.250 mm), Medium Sand (0.25 - 0.50 mm)
Particle Size, d_s =	0.00025	m	
Scour Depth, Δ_z =	0.8	m	
Scour Depth, Δ_z =	2.8	ft	

SUBJECT:	Klamath River Renewal Corp.	**BY:** N.Cox	**CHK'D BY:** M.Cerucci
	Daggett Bridge Modification	**DATE:** 9/20/2021	
	Rock Slope Protection Stone Size/Weight	**PROJECT NO.:** 21-067	

Purpose

The purpose of this calculation sheet is to determine the rock slope protection stone size and required revetment layering, using the California Bank and Shore Rock Slope Protection Design Method.

References

- *Caltrans, (2000). California Bank and Shore Rock Slope Protection Design, Practitioner's Guide and Field Evaluations of Riprap Methods, Third Edition. FHWA-CA-TL-95-10. State of California, Department of Transportation, Engineering Service Center, Office of Structural Foundations, Transportation Laboratory. October 2000.*

Equations

$$W = \frac{0.00002V^6 SG}{(SG-1)^3(\sin(r-a))^3}$$

Equation 1 (Caltrans, 2000)

Where:
- *W = Theoretical Minimal Rock Mass (Size or Weight) (lbs)*
- *V = Velocity (ft/s)*
  - *for PARALLEL flow multiply average channel velocity by 0.67 (2/3)*
  - *for IMPINGING flow multiply average channel velocity by 1.33 (4/3)*
- *SG = Specific Gravity of the Rock*
- *r = 70 degrees (for ramdonly placed rubble, a constant)*
- *a = Outside slope face angle with horizontal (degrees)*

Calculations

Unit Weight of Water, γ_w =	62.4	lbs/ft³	
Unit Weight of Stone, γ_s =	156	lbs/ft³	Assumed
Specific Gravity, *SG* =	2.50		
Depth-Averaged Velocity, *V* =	17.7	ft/s	HEC-RAS Model
Left Bank Slope, *a* =	15.0	degrees	
Right Bank Slope, *a* =	27.7	degrees	Thalweg locaged on Right Bank

Left Bank

a =	15.0	degrees	
Velocity Multiplier =	0.67		Parallel with Flow
Rock Mass, *W* =	73	lbs	
Rock Mass, *W* =	33	Kg	
Rock Mass, *W* =	0.03	Tonnes	
Equivalent Rock Size, Diameter =	11.5	inches	
Outside Layer, RSP-Class =	1/4 ton		Table 5-1 (Caltrans, 2000)
Minimum Layer Thickness =	3.3	ft	Table 5-3 (Caltrans, 2000)
Backing, RSP-Class =	2		Table 5-2 (Caltrans, 2000)
Minimum Layer Thickness =	1	ft	Table 5-3 (Caltrans, 2000)
RSP-Fabric Type =	A		Table 5-2 (Caltrans, 2000)
Total Thickness =	4.6	ft	

Right Bank

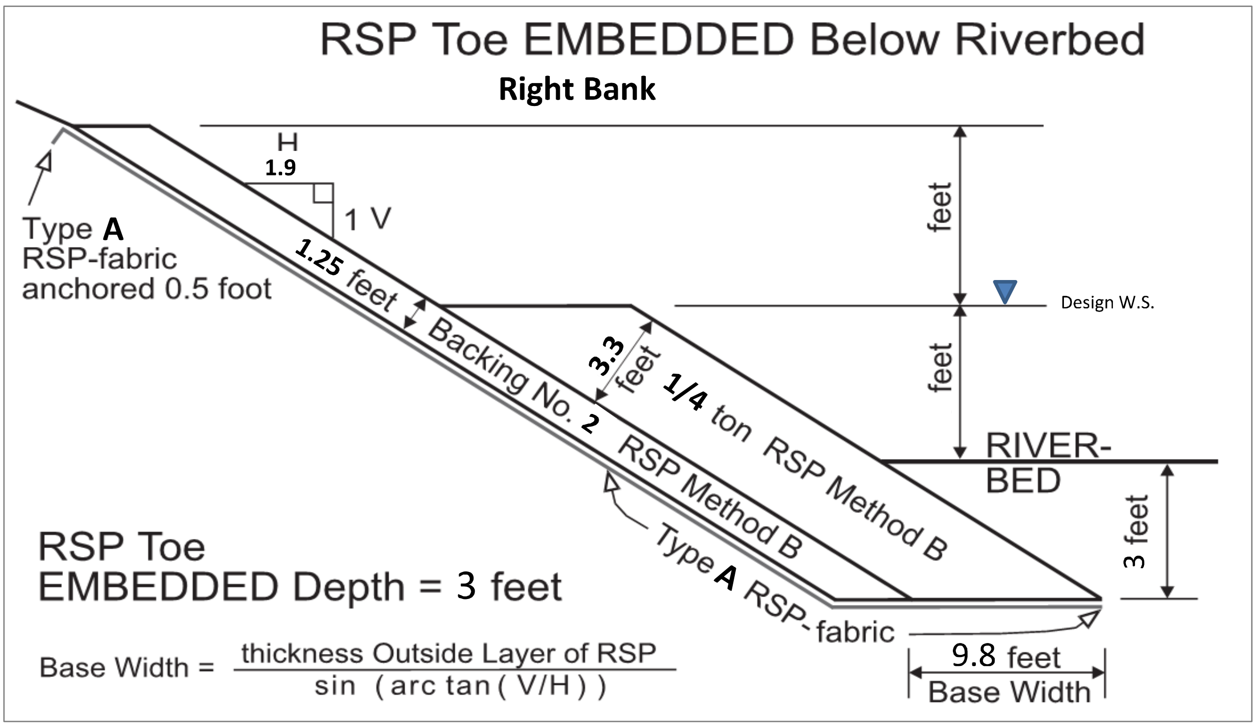
a =	27.7	degrees	
Velocity Multiplier =	0.80		Assumed, Parallel with Flow plus Thalweg located at Bank
Rock Mass, *W* =	392	lbs	
Rock Mass, *W* =	178	Kg	
Rock Mass, *W* =	0.18	Tonnes	
Equivalent Rock Size, Diameter =	20.2	inches	
RSP-Class of Outside Layer =	1/4 ton		Table 5-1 (Caltrans, 2000)
Minimum Layer Thickness =	3.3	ft	Table 5-3 (Caltrans, 2000)
Backing, RSP-Class =	2		Table 5-2 (Caltrans, 2000)
Minimum Layer Thickness =	1.25	ft	Table 5-3 (Caltrans, 2000)
RSP-Fabric Type =	A		Table 5-2 (Caltrans, 2000)
Total Thickness =	4.6	ft	

STANDARD Rock SIZE or Rock MASS or Rock WEIGHT		GRADING OF ROCK SLOPE PROTECTION **PERCENTAGE LARGER THAN**											
		RSP-Classes [A]											
		Method A Placement					Method B Placement						
		RSP-Classes other than Backing									Backing No.		
US unit		8 ton	4 ton	2 ton	1 ton	1/2 ton	1 ton	1/2 ton	1/4 ton	Light	1 [B]	2	3
	SI unit	8 T	4 T	2 T	1 T	1/2 T	1 T	1/2 T	1/4 T	Light	1 [B]	2	3
16 ton	14.5 tonne	0-5											
8 ton	7.25 tonne	50-100	0-5										
4 ton	3.6 tonne	95-100	50-100	0-5									
2 ton	1.8 tonne		95-100	50-100	0-5		0-5						
1 ton	900 kg			95-100	50-100	0-5	50-100	0-5					
1/2 ton	450 kg				95-100	50-100	-----	50-100	0-5				
1/4 ton	220 kg					95-100	95-100	-----	50-100	0-5			
200 lb	90 kg							95-100	-----	50-100	0-5		
75 lb	34 kg								95-100	-----	50-100	0-5	
25 lb	11 kg									95-100	90-100	25-75	0-5
5 lb	2.2 kg											90-100	25-75
1 lb	0.4 kg												90-100

[A] US customary names (units) of RSP-Classes listed above SI names, example US is "2 ton" metric is "2 T".
[B] "Facing" has same gradation as "Backing No. 1". To conserve space "Facing" is not shown .

Example for determining RSP-Class of outside layer. By using Equation 1, if the calculated W=135 kg (minimum stable rock size):
1. Enter table at left and select closest value of STANDARD Rock SIZE which is greater than calculated W, in this case 220 kg
2. Trace to right and locate "50-100" entry 3. Trace upward and read column heading "1/4 T", then 1/4 T is first trial RSP-Class.

Table 5-1. Guide for Determining RSP-Class of Outside Layer



SUBJECT:	Klamath River Renewal Corp.	BY:	N.Cox	CHK'D BY:	
	Daggett Bridge Modification	DATE:	8/11/2021		
	HEC-RAS Model Results for Daggett Bridge	PROJECT NO.:	21-067		

Purpose

The purpose of this calculation sheet is to determine the water surface elevation at the proposed Daggett Road Bridge.

Annual Percent Probable Flood

Appendix A6, "Hydrology", Knight Piésold Consulting. Part of Kiewit Infrastructure West Co., Klamath River Renewal Project, 90% Design Report.

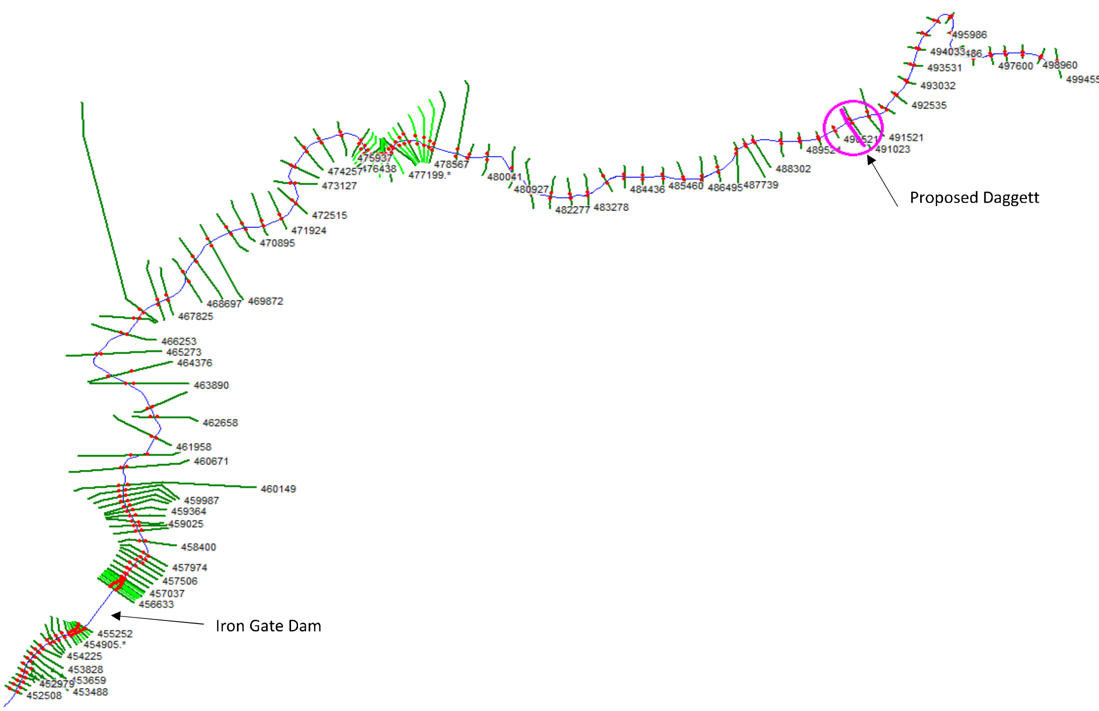
Table 3.2. Annual Peak Floods at Iron Gate.

Probability (%)	Return Period	Flow (cfs)
50%	2-yr	7,500
20%	5-yr	10,900
10%	10-yr	14,900
5%	20-yr	19,300
2%	50-yr	25,700
1%	100-yr	31,200
0.5%	200-yr	37,100
0.2%	500-yr	45,800

HEC-RAS Results

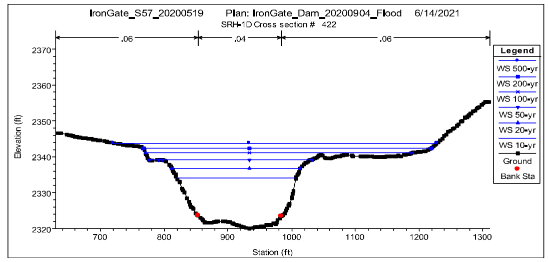
The HEC-RAS model was developed by Knight Piésold Consulting. The model was originally used to evaluate how the system would respond to historic flooding during drawdown conditions, in preparation for dam removel, at Iron Gate Dam.

HEC-RAS model was modified by removing Iron Gate Dam and running in steady-state conditions. Model was minimially changed. The results are below are based on the closest upstream cross-section to the proposed feature, or a bridge section was added to model the influcence of the proposed bridge. For documentation on the model please see documentation by Knight Piesold.



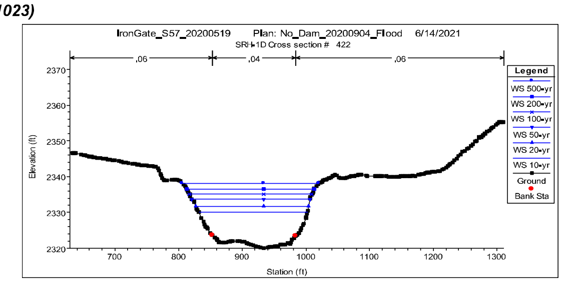
Pre-Drawdown Conditions (Existing Conditions) (HEC-RAS STA 491023)

Probability (%)	Return Period	Flow (cfs)	W.S. Elevation (ft)	Channel Velocity (ft/s)
10%	10-yr	14,900	2334.2	8.2
5%	20-yr	19,300	2336.8	8.7
2%	50-yr	25,700	2339.2	10.0
1%	100-yr	31,200	2341.1	10.9
0.5%	200-yr	37,100	2342.4	11.8
0.2%	500-yr	45,800	2343.8	13.3



Post-Drawdown Conditions (Iron Gate Dam is Removed) (HEC-RAS STA 491023)

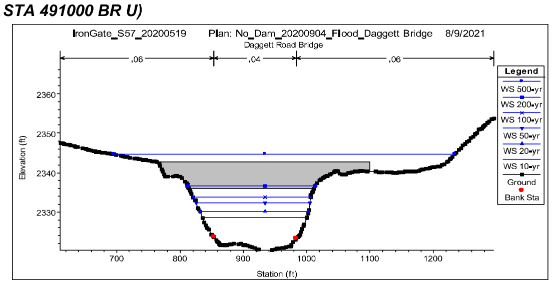
Probability (%)	Return Period	Flow (cfs)	W.S. Elevation (ft)	Channel Velocity (ft/s)
10%	10-yr	14,900	2330.1	12.4
5%	20-yr	19,300	2331.7	13.4
2%	50-yr	25,700	2333.7	14.8
1%	100-yr	31,200	2335.1	16.0
0.5%	200-yr	37,100	2336.5	17.1
0.2%	500-yr	45,800	2338.1	18.9



HEC-RAS Results - Proposed Daggett Bridge

Proposed Daggett Bridge Conditions (Iron Gate Dam is Removed) (HEC-RAS STA 491000 BR U)

Probability (%)	Return Period	Flow (cfs)	W.S. Elevation (ft)	Channel Velocity (ft/s)
10%	10-yr	14,900	2328.6	15.1
5%	20-yr	19,300	2330.1	16.0
2%	50-yr	25,700	2332.4	16.7
1%	100-yr	31,200	2333.8	17.7
0.5%	200-yr	37,100	2336.7	17.5
0.2%	500-yr	45,800	2344.7	16.2



Appendix C
Geotechnical Design Calculations

Calculation Cover Sheet

McMILLEN JACOBS ASSOCIATES

Project: Daggett Bridge Design Project

Client: Klamath River Renewal Corporation **Proj. No.** 21-067

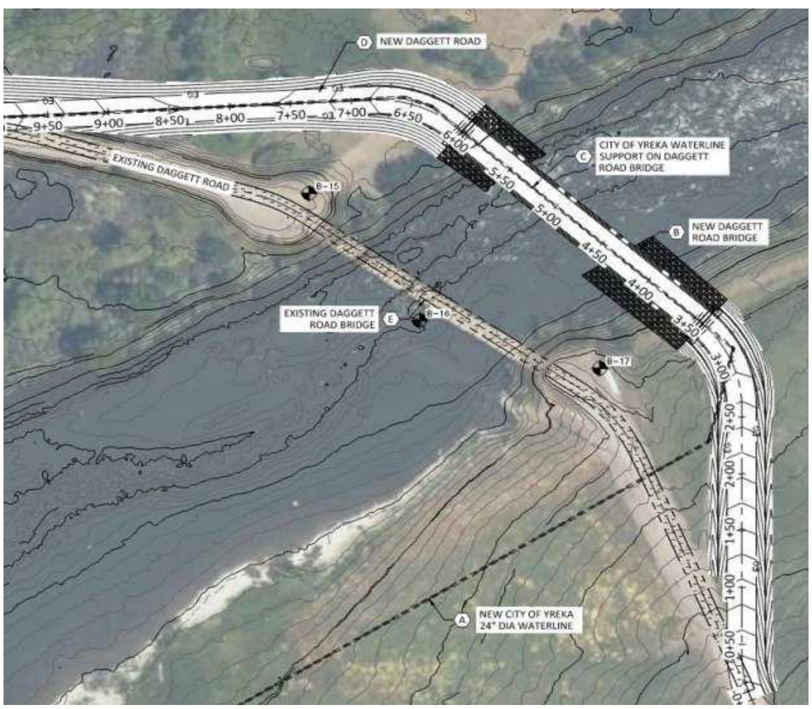
Title: Geotechnical Design Calculations

Prepared By, Name: Shawn Spreng

Prepared By, Signature: **Date:** 5/19/2022

Peer Reviewed By, Name: Thomas Borden

Peer Reviewed, Signature: Thomas A Borden **Date:** 5/19/2022



SUBJECT:	Klamath River Renewal Corperation	**BY:** S. Spreng	**CHK'D BY:** T. Borden
	Dagget Bridge Geotechnical Design	**DATE:** 5/19/2022	
	Table of Contents	**PROJECT NO.:** 21-067	

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Design Section	5
Settlement Calculation	6

- *Analyze settlement at abutments.*

Bearing Capacity	7

- *Presumptive Bearing Capacity*
- *Limit Strength Bearing Capacity*

Slope Stability Output	17

McMILLEN JACOBS ASSOCIATES	PROJECT Daggett Bridge		SHEET 3
	SUBJECT Geotechnical Design		DATE 10/11/21
	BY SPS	CHECKED	PROJECT NO.

BORINGS B-15, B-16, B-17

USE B-15 AS REPRESENTATIVE OF CONDITIONS AT BOTH ABUTMENTS

B-15 IS CLAYEY GRAVEL w/ SAND @ EL = 2336' to EL = 2327'

$N = 54$

$\sigma'_v = 7.5 \times 125 + 4.5 \times 130 + 4\,(130 - 62.4) = 1790\ \text{psf} = 1.8\ \text{ksf}$

$C_N = 0.77 \log_{10} (40/\sigma'_v) = 1.037$

$ER = 80\%$ AUTO TRIP HAMMER

$N1_{60} = C_N\, N\, (ER/60\%)$ AASHTO CH 10*

$= 1.037 \times 54\, (.8/.6) = 75$

$\phi_f \approx 38°$ AASHTO Table 10.4.6.2.4.1

BUT DEDUCT 3° FOR FINES CONTENT

$\phi_f = \underline{\underline{35°}}$

$E_s = 13$ ksi for Med. Dense Gravel or V. Stiff Clay AASHTO Table C10.4.6.3.1

AND

$E_s =$ BETWEEN 4.2 TO 12.5 ksi using $N1_{60} = 50$ Table C10.4.6.3.1

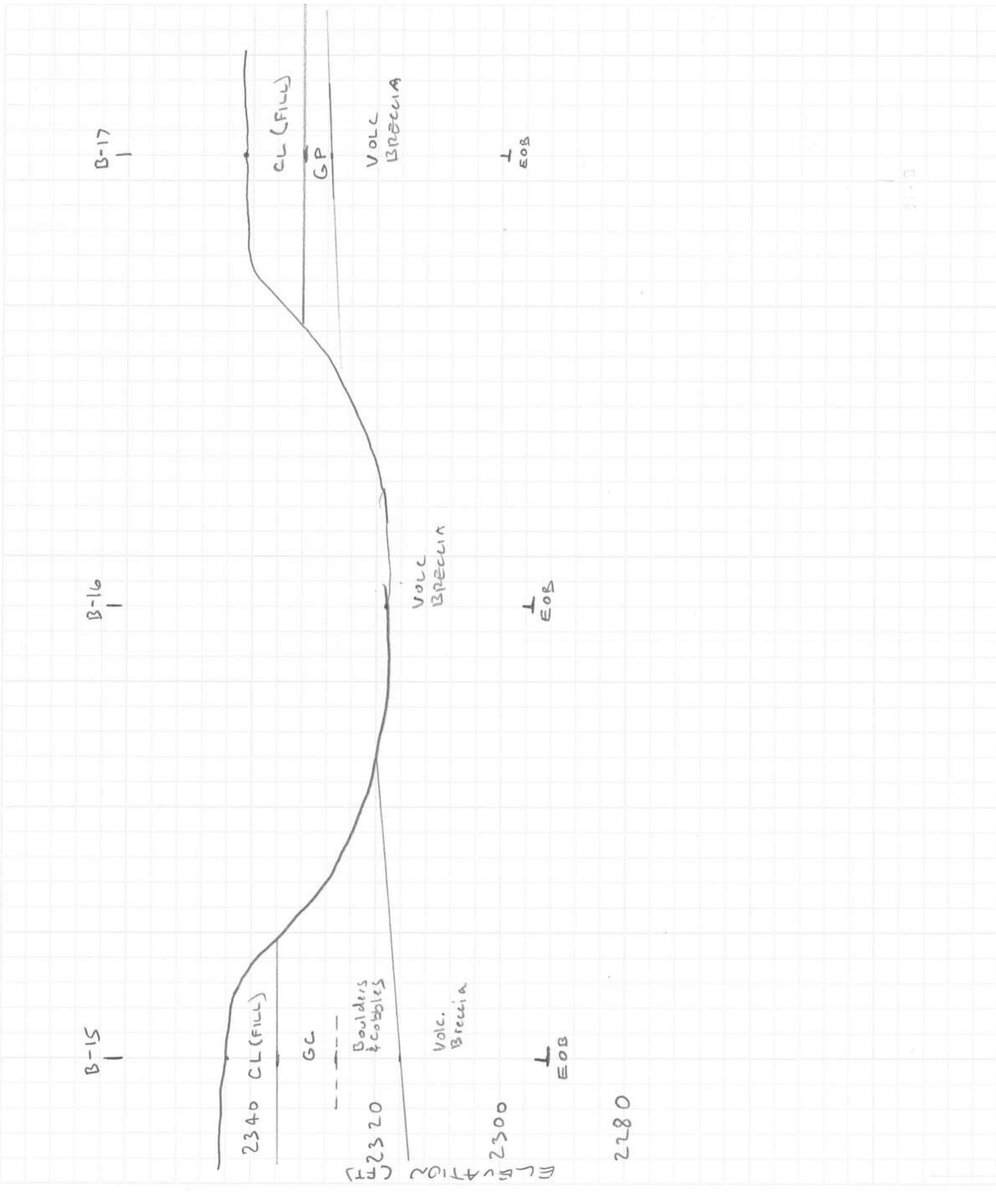
USE $E_s = 10$ ksi

$\nu = 0.4$ AASHTO Table C10.4.6.3.1

* ALL REFERENCES ARE TO AASHTO LRFD BRIDGE DESIGN SPECIFICATION, 8th ED. 2017

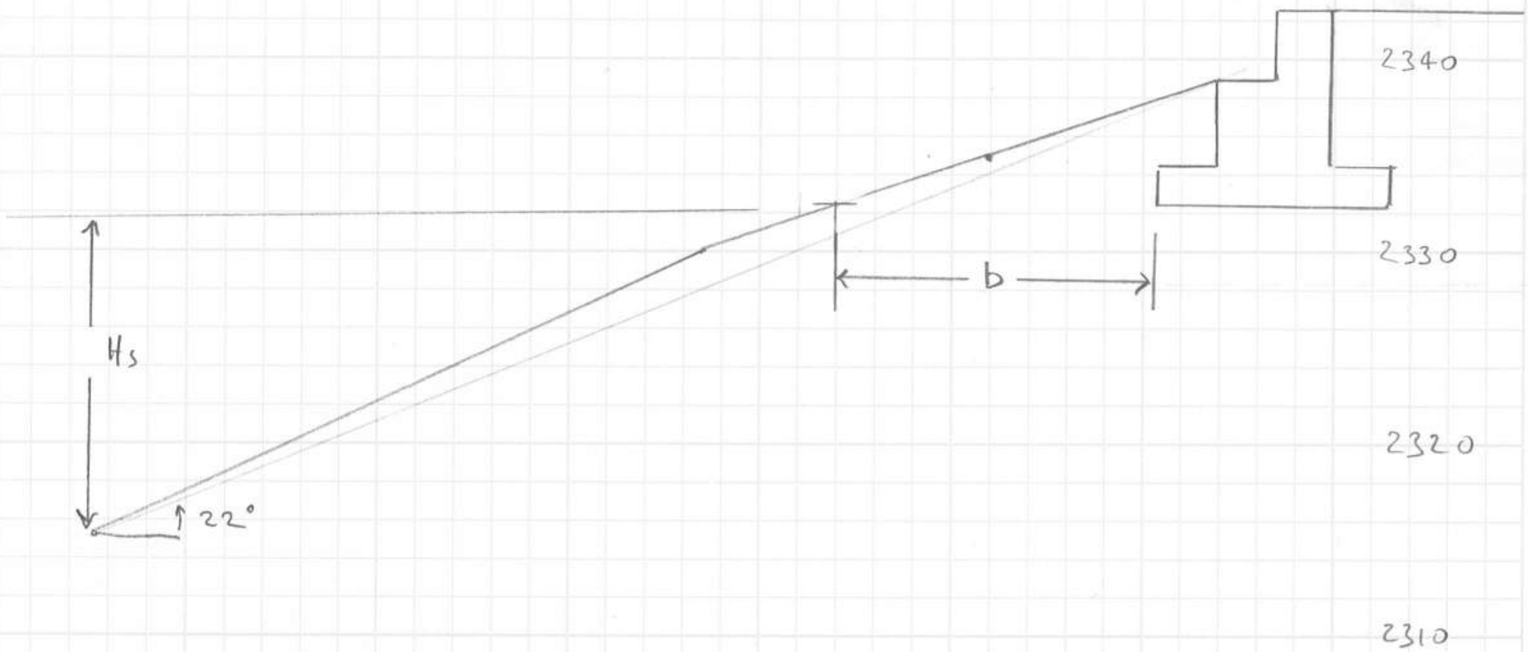
McMILLEN JACOBS ASSOCIATES

PROJECT Daggett Bridge		SHEET 4
SUBJECT		DATE 10/11/21
BY	CHECKED	PROJECT NO.



McMILLEN JACOBS ASSOCIATES

PROJECT Dagget Bridge		SHEET 5
SUBJECT		DATE 5/13/22
BY	CHECKED	PROJECT NO.



GC
$\gamma = 125$ pcf
$\phi = 35°$
$C = 0$ (250 psf)
$E_s = 10$ ksi
$\nu = 0.4$

SETTLEMENT

Soil is clayey gravel (GC) ⇒ Granular cohesionless soil will not experience long term consolidation settlement.

Check elastic settlement S_e using AASHTO Section 10.6.2.4.2

$$S_e = \frac{q_o (1-\nu^2)\sqrt{A}}{144 E_s \beta_z}$$

$F_v = 51.40$ kips/ft From Structural Engineer, using Service Load I (10.6.2.3)

$q_o = (51.4 \text{ kips/FT} \times 35 \text{ FT}) / (12 \text{ FT} \times 37 \text{ FT}) = 4.05 \text{ ksf}$

$A = 12' \times 37' = 444 \text{ sf}$

$E_s = 10 \text{ ksi}$

$\beta_z = L/B = 37/12 = 3.1 \Rightarrow \beta_z = 1.15$ Table 10.6.2.4.2-1

$\nu = 0.4$

$$S_e = \frac{4.05 (1-.4^2)\sqrt{444}}{144 \cdot 10 \cdot 1.15} = 0.043 \text{ ft} = \underline{0.52 \text{ inches}}$$

BEARING CAPACITY

Determine bearing capacity of footings per AASHTO Section 10.6.2.6

① Check Presomptive bearing resistance

GC & SC very Dense ⟹ 16 - 24 KSF

② Calculate Strength Limit State

$q_R = \Phi_b q_n$

Φ_b = Resistance Factor = 0.45 AASHTO Table 10.5.5.2.2-1

$q_n = c N_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5 \gamma B N_{\gamma m} C_{w\gamma}$

$C = 0$

$\gamma = 125 \text{ pcf} = 0.125 \text{ kcf}$

$D_f = 6.25'$

$N_{qm} = N_q s_q d_q i_q$

$N_q = 33.3$

$s_q = 1 + (12/37 \tan 35) = 1.23$

$d_q = i_q = 1$

$N_{qm} = 33.3 \times 1.23 \times 1 = 41.0$

$C_{wq} = 0.5$ Assumes GW at ground surface (very conservative)

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$

$N_\gamma = 48$

$s_\gamma = 1 - .4\left(\frac{12}{37}\right) = 0.87$

$i_\gamma = 1.0$

$N_{\gamma m} = 48 \cdot .87 \cdot 1 = 41.8$

$C_{w\gamma} = 0.5$ Assumes GW at ground Surface (conservative)

BEARING CAPACITY (cont.)

$q_n = 0 + 0.125(6.25)(41.0)(0.5) + 0.5(0.125)(12)(41.8)(0.5)$

$= 0 + 16.01 + 15.68$

$= 31.69$ KSF

Slope Reduction Factor

$q_{n\text{-sloping ground}} = RC_{BC}\, q_n$

$H_s = 17.75'$

$b = 16'$

$B = 12'$

$B/H = 0.68$

$b/B = 16/12 = 1.33$

$\beta = 22°$

$RC_{BC} = 0.64$ Table 10.6.3.1.2c-1

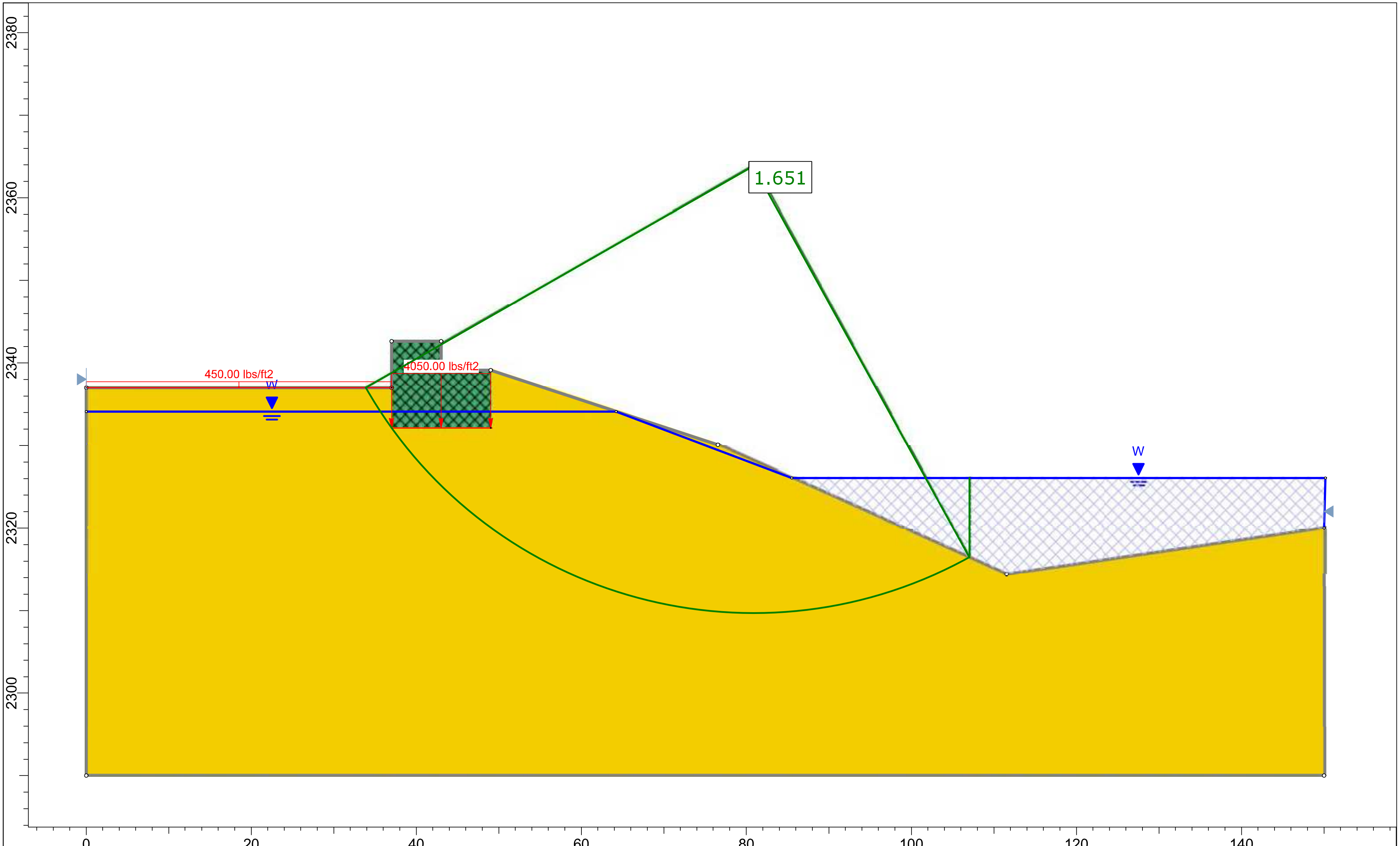
$q_n = 0.64(31.69) = 20.4$ ksf

$q_R = \phi_b q_n$

$= 0.45(20.4) = 9.2$ ksf

considering presumptive bearing resistance from ① above = 16-24 ksf

use 9.2 ksf



rocscience	Project	Daggett Rd Bridge	
	Group	Group 1	Scenario: Master Scenario
	Drawn By	Shawn Spreng	Company: McMillen Jacobs Associates
SLIDEINTERPRET 9.008	Date	5/17/2022	File Name: Daggett Bridge with load_Final.slmd

Daggett Bridge with load_Final
Daggett Rd Bridge
McMillen Jacobs Associates
Date Created: 5/17/2022
Software Version: 9.008

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Slide Analysis Information

Daggett Bridge with load_Final

Project Summary

File Name:	Daggett Bridge with load_Final.slmd
Slide Modeler Version:	9.008
Compute Time:	00h:00m:09.16s
Project Title:	Daggett Rd Bridge
Author:	Shawn Spreng
Company:	McMillen Jacobs Associates
Date Created:	6/11/2021

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Random Numbers

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Surface Type:	Circular
Search Method:	Auto Refine Search
Divisions along slope:	20
Circles per division:	10
Number of iterations:	10
Divisions to use in next iteration:	50%
Composite Surfaces:	Disabled
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic Loading

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Loading

2 Distributed Loads present

Distributed Load 1	
Distribution:	Constant
Magnitude [psf]:	4050
Orientation:	Vertical

Distributed Load 2	
Distribution:	Constant
Magnitude [psf]:	450
Orientation:	Normal to boundary

Materials

Material 1	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	125
Cohesion [psf]	250
Friction Angle [deg]	35
Water Surface	Water Table
Hu Value	1
Material 2	
Color	
Strength Type	Infinite strength
Unit Weight [lbs/ft3]	120
Allow Sliding Along Boundary	Yes
Water Surface	Water Table
Hu Value	1

Global Minimums

Method: bishop simplified

FS	1.650540
Center:	80.868, 2363.834
Radius:	54.123
Left Slip Surface Endpoint:	33.865, 2337.000
Right Slip Surface Endpoint:	107.014, 2316.445
Left Slope Intercept:	33.865 2337.000
Right Slope Intercept:	107.014 2326.068
Resisting Moment:	5.86172e+06 lb-ft
Driving Moment:	3.5514e+06 lb-ft
Total Slice Area:	1230.66 ft2
Surface Horizontal Width:	73.1489 ft
Surface Average Height:	16.8241 ft

Global Minimum Support Data

No Supports Present

Valid and Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces:	8772
Number of Invalid Surfaces:	0

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.65054

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	1.76955	319.19	-58.486	Material 1	250	35	247.673	408.794	226.781	0	226.781	630.726	630.726
2	1.45672	776.724	-55.338	Material 1	250	35	384.296	634.296	614.542	65.7088	548.833	1170.32	1104.61
3	1.45672	2027.19	-52.7093	Material 1	250	35	1528.26	2522.46	3436.52	191.121	3245.4	5443.33	5252.21
4	1.45672	2360.71	-50.2311	Material 1	250	35	1608.37	2654.68	3739.66	305.413	3434.24	5672.22	5366.8
5	1.45672	2666.73	-47.876	Material 1	250	35	1683.22	2778.22	4020.95	410.282	3610.67	5882.24	5471.96
6	1.45672	2948.93	-45.6238	Material 1	250	35	1753.64	2894.45	4283.66	506.99	3776.67	6075.9	5568.91
7	1.45672	2632.22	-43.4589	Material 1	250	35	1700.2	2806.25	4247.21	596.509	3650.7	5858.33	5261.82
8	1.45672	2840.46	-41.3691	Material 1	250	35	1753.82	2894.75	4456.7	679.603	3777.1	6001.22	5321.62
9	1.45672	3065.99	-39.3445	Material 1	250	35	1811.81	2990.47	4670.69	756.888	3913.8	6155.99	5399.1
10	1.45672	3276.03	-37.3771	Material 1	250	35	1867.36	3082.16	4873.61	828.867	4044.74	6300.13	5471.27
11	1.45672	3484.16	-35.4601	Material 1	250	35	834.388	1377.19	2505.75	895.958	1609.79	3100.04	2204.08
12	1.45672	3589.91	-33.5877	Material 1	250	35	616.705	1017.9	2055.18	958.512	1096.67	2464.73	1506.22
13	1.45672	3673.08	-31.7553	Material 1	250	35	625.643	1032.65	2134.56	1016.83	1117.73	2521.8	1504.97
14	1.45672	3744.61	-29.9584	Material 1	250	35	632.933	1044.68	2206.08	1071.15	1134.93	2570.89	1499.74
15	1.45672	3805.15	-28.1935	Material 1	250	35	638.639	1054.1	2270.09	1121.71	1148.38	2612.43	1490.72
16	1.45672	3855.25	-26.4573	Material 1	250	35	642.812	1060.99	2326.9	1168.69	1158.21	2646.8	1478.11
17	1.45672	3895.39	-24.747	Material 1	250	35	645.496	1065.42	2376.8	1212.26	1164.54	2674.33	1462.07
18	1.45672	3925.99	-23.0599	Material 1	250	35	646.728	1067.45	2420	1252.56	1167.44	2695.32	1442.76
19	1.45672	3947.41	-21.3937	Material 1	250	35	646.54	1067.14	2456.71	1289.71	1167	2710	1420.29
20	1.45672	3959.98	-19.7463	Material 1	250	35	644.956	1064.53	2487.1	1323.84	1163.26	2718.61	1394.77
21	1.45672	3963.98	-18.1157	Material 1	250	35	641.998	1059.64	2511.31	1355.02	1156.29	2721.34	1366.32
22	1.45672	3959.65	-16.5002	Material 1	250	35	649.172	1071.49	2526.06	1352.86	1173.2	2718.36	1365.5
23	1.45672	3947.23	-14.8981	Material 1	250	35	656.741	1083.98	2535.09	1344.05	1191.04	2709.81	1365.76
24	1.45672	3926.88	-13.3079	Material 1	250	35	663.266	1094.75	2538.95	1332.52	1206.43	2695.83	1363.31
25	1.45672	3898.79	-11.728	Material 1	250	35	668.753	1103.8	2537.7	1318.34	1219.36	2676.53	1358.19
26	1.45672	3863.09	-10.1571	Material 1	250	35	673.202	1111.15	2531.4	1301.55	1229.85	2652.01	1350.46
27	1.45672	3819.89	-8.59392	Material 1	250	35	676.614	1116.78	2520.09	1282.2	1237.89	2622.34	1340.14
28	1.45672	3769.31	-7.03715	Material 1	250	35	678.984	1120.69	2503.78	1260.31	1243.47	2587.6	1327.29
29	1.45672	3711.42	-5.48558	Material 1	250	35	680.306	1122.87	2482.51	1235.92	1246.59	2547.85	1311.93
30	1.45672	3632.71	-3.93805	Material 1	250	35	676.732	1116.97	2447.21	1209.04	1238.17	2493.79	1284.75
31	1.45672	3528.9	-2.39339	Material 1	250	35	666.878	1100.71	2394.64	1179.71	1214.93	2422.51	1242.8
32	1.45672	3417.84	-0.850472	Material 1	250	35	655.711	1082.28	2336.53	1147.91	1188.62	2346.26	1198.35
33	1.45672	3299.64	0.69183	Material 1	250	35	643.232	1061.68	2272.87	1113.67	1159.2	2265.1	1151.43
34	1.45672	3174.29	2.23463	Material 1	250	35	629.41	1038.87	2203.6	1076.98	1126.62	2179.04	1102.06
35	1.45672	3041.78	3.77906	Material 1	250	35	614.214	1013.78	2128.64	1037.84	1090.8	2088.07	1050.23
36	1.45672	2920.04	5.32625	Material 1	250	35	598.811	988.361	2060.31	1005.82	1054.49	2004.48	998.66
37	1.45672	2830.86	6.87734	Material 1	250	35	583.108	962.443	2013.58	996.102	1017.47	1943.25	947.145
38	1.45672	2735.82	8.43352	Material 1	250	35	566.399	934.864	1961.97	983.881	978.09	1877.99	894.113
39	1.45672	2633.4	9.996	Material 1	250	35	548.188	904.806	1904.29	969.132	935.158	1807.67	838.537
40	1.45672	2523.5	11.566	Material 1	250	35	528.41	872.162	1840.36	951.82	888.541	1732.22	780.4
41	1.45672	2406	13.1449	Material 1	250	35	506.99	836.807	1769.95	931.904	838.046	1651.55	719.646
42	1.45672	2280.77	14.734	Material 1	250	35	483.84	798.598	1692.82	909.338	783.479	1565.58	656.238
43	1.45672	2147.64	16.3349	Material 1	250	35	458.862	757.37	1608.67	884.065	724.601	1474.18	590.117
44	1.45672	2006.43	17.9489	Material 1	250	35	431.939	712.932	1517.16	856.022	661.135	1377.24	521.216
45	1.45672	1856.92	19.5778	Material 1	250	35	402.935	665.06	1417.9	825.135	592.768	1274.6	449.465
46	1.45672	1698.86	21.2234	Material 1	250	35	371.692	613.493	1310.44	791.321	519.119	1166.1	374.774
47	1.45672	1531.99	22.8876	Material 1	250	35	338.026	557.925	1194.24	754.484	439.758	1051.54	297.057
48	1.45672	1355.97	24.5724	Material 1	250	35	301.712	497.988	1068.68	714.514	354.162	930.717	216.203
49	1.45672	1170.45	26.2803	Material 1	250	35	262.487	433.246	932.99	671.289	261.701	803.373	132.084
50	1.45672	975.023	28.0137	Material 1	250	35	220.029	363.167	786.285	624.666	161.619	669.226	44.5605

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.65054

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [deg]
1	33.8654	2337	0	0	0
2	35.6349	2334.11	216.612	0	0
3	37.0917	2332.01	951.966	0	0
4	38.5484	2330.09	5301.24	0	0
5	40.0051	2328.34	9505.98	0	0
6	41.4618	2326.73	13533.2	0	0
7	42.9185	2325.24	17358.3	0	0
8	44.3753	2323.86	20746.5	0	0
9	45.832	2322.58	23911.3	0	0
10	47.2887	2321.39	26852	0	0
11	48.7454	2320.27	29557.6	0	0
12	50.2022	2319.24	30942.9	0	0
13	51.6589	2318.27	32033.5	0	0
14	53.1156	2317.37	33047.5	0	0
15	54.5723	2316.53	33978.6	0	0
16	56.029	2315.75	34821.7	0	0
17	57.4858	2315.02	35573	0	0
18	58.9425	2314.35	36229.4	0	0
19	60.3992	2313.73	36788.9	0	0
20	61.8559	2313.16	37249.9	0	0
21	63.3127	2312.64	37611.7	0	0
22	64.7694	2312.16	37874.1	0	0
23	66.2261	2311.73	38019.3	0	0
24	67.6828	2311.34	38045.9	0	0
25	69.1395	2311	37955.4	0	0
26	70.5963	2310.69	37749.4	0	0
27	72.053	2310.43	37430.3	0	0
28	73.5097	2310.21	37000.3	0	0
29	74.9664	2310.03	36462.3	0	0
30	76.4232	2309.89	35819.4	0	0
31	77.8799	2309.79	35079.8	0	0
32	79.3366	2309.73	34255	0	0
33	80.7933	2309.71	33351.2	0	0
34	82.25	2309.73	32375	0	0
35	83.7068	2309.78	31333.6	0	0
36	85.1635	2309.88	30234.8	0	0
37	86.6202	2310.02	29075.4	0	0
38	88.0769	2310.19	27839.1	0	0
39	89.5337	2310.41	26530.7	0	0
40	90.9904	2310.67	25157.1	0	0
41	92.4471	2310.96	23726.2	0	0
42	93.9038	2311.3	22246.5	0	0
43	95.3605	2311.69	20727.6	0	0
44	96.8173	2312.11	19180.4	0	0
45	98.274	2312.59	17616.7	0	0
46	99.7307	2313.1	16050.2	0	0
47	101.187	2313.67	14495.9	0	0
48	102.644	2314.28	12971.1	0	0
49	104.101	2314.95	11495.2	0	0
50	105.558	2315.67	10090.7	0	0
51	107.014	2316.44	2889.5	0	0

Entity Information

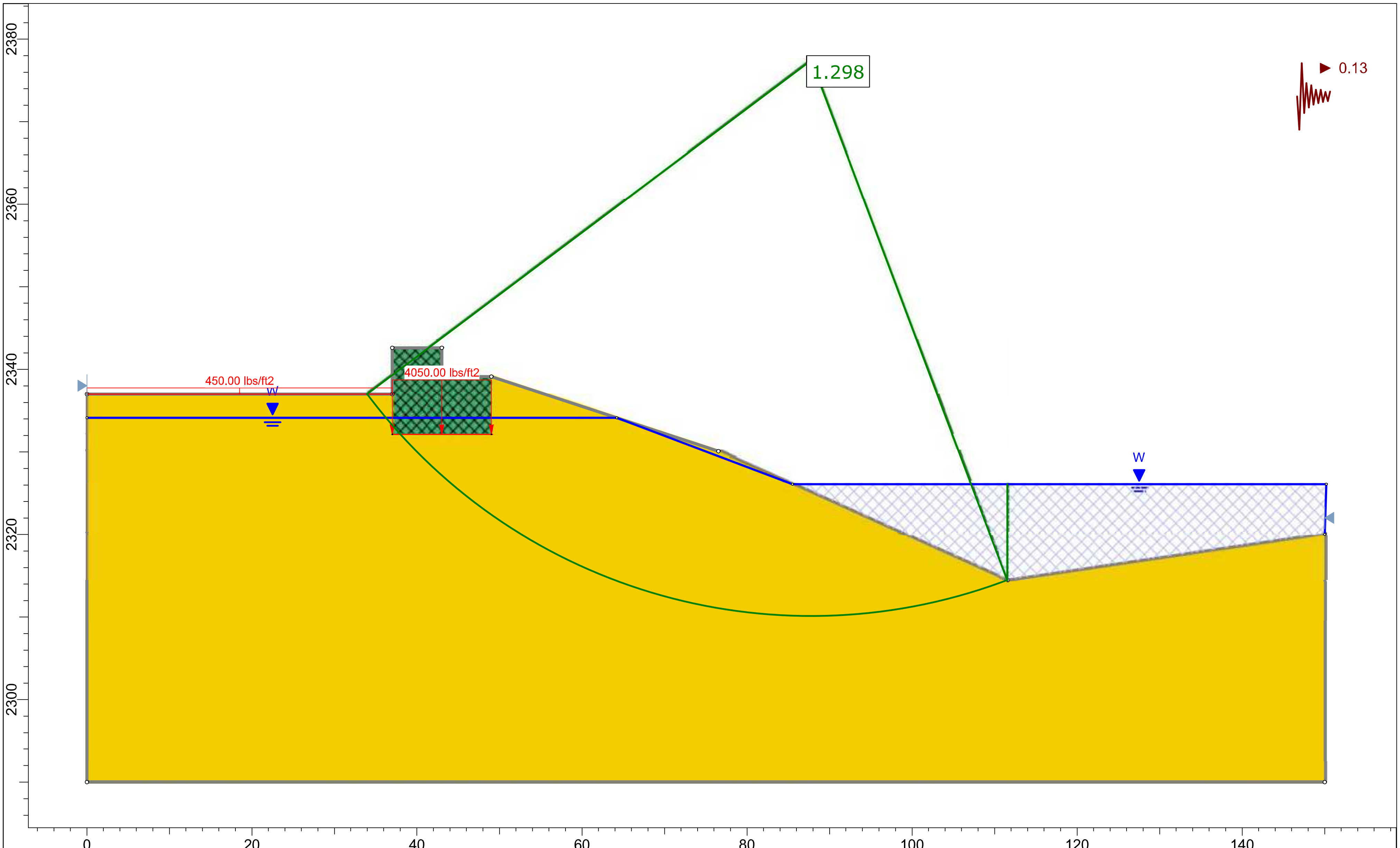
Group 1

Shared Entities

Type	Coordinates (x,y)
External Boundary	0, 2337 0, 2290 150, 2290 150, 2320 111.533, 2314.43 76.529, 2330.07 49, 2339.1 43, 2339.1 43, 2342.6 37, 2342.6 37, 2337
Material Boundary	37, 2337 37, 2332.12 49, 2332.12 49, 2339.1

Scenario-based Entities

Type	Coordinates (x,y)	Master Scenario
Water Table	-2.77556e-17, 2334.11 64.2053, 2334.11 85.4855, 2326.07 150.17, 2326.07 150, 2320	Assigned to: Material 1 Material 2
Distributed Load	49, 2332.12 37, 2332.12	Constant DistributionOrientation: VerticalMagnitude: 4050 lbs/ft2Creates Excess Pore Pressure: No
Distributed Load	37, 2337 0, 2337	Constant DistributionOrientation: Normal to boundaryMagnitude: 450 lbs/ft2Creates Excess Pore Pressure: No



Project	Daggett Rd Bridge		
Group	Group 1	Scenario	Master Scenario
Drawn By	Shawn Spreng	Company	McMillen Jacobs Associates
Date	5/17/2022	File Name	Daggett Bridge with load and Seismic_Final.slmd

rocscience

SLIDEINTERPRET 9.008

Daggett Bridge with load and Seismic_Final
Daggett Rd Bridge
McMillen Jacobs Associates
Date Created: 5/17/2022
Software Version: 9.008

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Slide Analysis Information

Daggett Bridge with load and Seismic_Final

Project Summary

File Name:	Daggett Bridge with load and Seismic_Final.slmd
Slide Modeler Version:	9.008
Compute Time:	00h:00m:08.313s
Project Title:	Daggett Rd Bridge
Author:	Shawn Spreng
Company:	McMillen Jacobs Associates
Date Created:	6/11/2021

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Left to Right

Analysis Options

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Groundwater Analysis

Groundwater Method:	Water Surfaces
Pore Fluid Unit Weight [lbs/ft3]:	62.4
Use negative pore pressure cutoff:	Yes
Maximum negative pore pressure [psf]:	0
Advanced Groundwater Method:	None

Random Numbers

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Surface Type:	Circular
Search Method:	Auto Refine Search
Divisions along slope:	20
Circles per division:	10
Number of iterations:	10
Divisions to use in next iteration:	50%
Composite Surfaces:	Disabled
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Seismic Loading

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.13

Loading

2 Distributed Loads present

Distributed Load 1	
Distribution:	Constant
Magnitude [psf]:	4050
Orientation:	Vertical

Distributed Load 2	
Distribution:	Constant
Magnitude [psf]:	450
Orientation:	Normal to boundary

Materials

Material 1	
Color	
Strength Type	Mohr-Coulomb
Unit Weight [lbs/ft3]	125
Cohesion [psf]	250
Friction Angle [deg]	35
Water Surface	Water Table
Hu Value	1
Material 2	
Color	
Strength Type	Infinite strength
Unit Weight [lbs/ft3]	120
Allow Sliding Along Boundary	Yes
Water Surface	Water Table
Hu Value	1

Global Minimums

Method: bishop simplified

FS	1.298050
Center:	87.771, 2377.410
Radius:	67.319
Left Slip Surface Endpoint:	33.930, 2337.000
Right Slip Surface Endpoint:	111.533, 2314.425
Left Slope Intercept:	33.930 2337.000
Right Slope Intercept:	111.533 2326.068
Resisting Moment:	6.58681e+06 lb-ft
Driving Moment:	5.07438e+06 lb-ft
Total Slice Area:	1232.73 ft2
Surface Horizontal Width:	77.6031 ft
Surface Average Height:	15.8851 ft

Global Minimum Support Data

No Supports Present

Valid and Invalid Surfaces

Method: bishop simplified

Number of Valid Surfaces:	10076
Number of Invalid Surfaces:	0

Slice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.29805

Slice Number	Width [ft]	Weight [lbs]	Angle of Slice Base [deg]	Base Material	Base Cohesion [psf]	Base Friction Angle [deg]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]	Base Vertical Stress [psf]	Effective Vertical Stress [psf]
1	2.29229	413.481	-51.5412	Material 1	250	35	317.437	412.049	231.429	0	231.429	631.092	631.092
2	1.53695	1218.61	-48.9761	Material 1	250	35	440.675	572.018	514.985	55.0951	459.89	1021.5	966.401
3	1.53695	2048.3	-47.0201	Material 1	250	35	1715.31	2226.56	2984.49	161.672	2822.82	4825.23	4663.56
4	1.53695	2354.94	-45.1335	Material 1	250	35	1987.6	2580	3588.89	261.308	3327.59	5585.77	5324.46
5	1.53695	2642.43	-43.3074	Material 1	250	35	2065.18	2680.71	3826.1	354.685	3471.42	5772.74	5418.05
6	1.53695	2531.21	-41.5348	Material 1	250	35	2049.49	2660.34	3884.69	442.363	3442.33	5700.14	5257.78
7	1.53695	2520.32	-39.8095	Material 1	250	35	2056.04	2668.84	3979.27	524.806	3454.46	5692.87	5168.06
8	1.53695	2759.24	-38.1265	Material 1	250	35	2123.38	2756.25	4181.71	602.408	3579.3	5848.24	5245.83
9	1.53695	2984.29	-36.4816	Material 1	250	35	2188.78	2841.15	4376.05	675.504	3700.55	5994.58	5319.07
10	1.53695	3210.36	-34.8708	Material 1	250	35	1165.71	1513.15	2548.34	744.379	1803.96	3360.67	2616.29
11	1.53695	3334.94	-33.2911	Material 1	250	35	684.492	888.505	1721.16	809.284	911.879	2170.64	1361.35
12	1.53695	3426.37	-31.7395	Material 1	250	35	694.35	901.301	1800.59	870.435	930.151	2230.09	1359.65
13	1.53695	3506.82	-30.2135	Material 1	250	35	702.511	911.895	1873.31	928.021	945.286	2282.4	1354.38
14	1.53695	3576.82	-28.7108	Material 1	250	35	709.017	920.339	1939.55	982.211	957.343	2327.9	1345.69
15	1.53695	3636.81	-27.2295	Material 1	250	35	713.897	926.674	1999.54	1033.15	966.392	2366.9	1333.75
16	1.53695	3687.2	-25.7676	Material 1	250	35	717.18	930.936	2053.45	1080.98	972.474	2399.65	1318.67
17	1.53695	3728.36	-24.3235	Material 1	250	35	718.89	933.155	2101.44	1125.8	975.645	2426.39	1300.59
18	1.53695	3760.6	-22.8957	Material 1	250	35	719.045	933.356	2143.66	1167.73	975.929	2447.33	1279.6
19	1.53695	3784.21	-21.4828	Material 1	250	35	717.658	931.556	2180.21	1206.85	973.362	2462.66	1255.81
20	1.53695	3799.45	-20.0834	Material 1	250	35	719.532	933.988	2209.46	1232.63	976.832	2472.54	1239.91
21	1.53695	3806.55	-18.6965	Material 1	250	35	731.695	949.777	2229.51	1230.13	999.384	2477.13	1247
22	1.53695	3805.71	-17.3208	Material 1	250	35	742.744	964.119	2244.92	1225.05	1019.87	2476.55	1251.5
23	1.53695	3797.12	-15.9554	Material 1	250	35	752.679	977.015	2255.74	1217.45	1038.29	2470.93	1253.48
24	1.53695	3780.94	-14.5992	Material 1	250	35	761.499	988.464	2262.03	1207.39	1054.64	2460.37	1252.98
25	1.53695	3757.32	-13.2513	Material 1	250	35	769.201	998.462	2263.83	1194.92	1068.91	2444.97	1250.05
26	1.53695	3726.38	-11.9109	Material 1	250	35	775.782	1007	2261.18	1180.06	1081.12	2424.82	1244.76
27	1.53695	3688.25	-10.5771	Material 1	250	35	781.232	1014.08	2254.09	1162.87	1091.22	2399.97	1237.1
28	1.53695	3632.46	-9.24898	Material 1	250	35	782.139	1015.25	2236.28	1143.38	1092.9	2363.64	1220.26
29	1.53695	3545.95	-7.92591	Material 1	250	35	774.076	1004.79	2199.55	1121.6	1077.95	2307.32	1185.72
30	1.53695	3451.62	-6.60708	Material 1	250	35	764.29	992.087	2157.38	1097.57	1059.81	2245.91	1148.34
31	1.53695	3350.4	-5.29175	Material 1	250	35	753.045	977.49	2110.28	1071.31	1038.97	2180.02	1108.71
32	1.53695	3242.36	-3.97922	Material 1	250	35	740.316	960.967	2058.19	1042.83	1015.36	2109.69	1066.86
33	1.53695	3127.52	-2.66878	Material 1	250	35	726.071	942.476	2001.09	1012.14	988.954	2034.94	1022.8
34	1.53695	3035.49	-1.35973	Material 1	250	35	711.883	924.06	1958.13	995.477	962.657	1975.03	979.554
35	1.53695	2972.93	-0.0513982	Material 1	250	35	698.052	906.107	1933.67	996.658	937.016	1934.3	937.642
36	1.53695	2903.72	1.25691	Material 1	250	35	682.709	886.191	1904.22	995.649	908.575	1889.25	893.596
37	1.53695	2827.77	2.56587	Material 1	250	35	665.777	864.212	1869.63	992.448	877.182	1839.79	847.347
38	1.53695	2745.05	3.87618	Material 1	250	35	647.203	840.102	1829.8	987.05	842.752	1785.95	798.9
39	1.53695	2655.54	5.18852	Material 1	250	35	626.93	813.787	1784.62	979.446	805.171	1727.69	748.243
40	1.53695	2559.2	6.5036	Material 1	250	35	604.893	785.181	1733.94	969.625	764.317	1664.98	695.359
41	1.53695	2455.98	7.82213	Material 1	250	35	581.018	754.191	1677.63	957.571	720.061	1597.81	640.242
42	1.53695	2345.83	9.14485	Material 1	250	35	555.224	720.708	1615.5	943.264	672.239	1526.12	582.861
43	1.53695	2228.67	10.4725	Material 1	250	35	527.417	684.614	1547.37	926.681	620.693	1449.88	523.204
44	1.53695	2104.42	11.8059	Material 1	250	35	497.495	645.773	1473.02	907.794	565.226	1369.03	461.241
45	1.53695	1972.98	13.1458	Material 1	250	35	465.338	604.032	1392.18	886.572	505.609	1283.5	396.93
46	1.53695	1834.24	14.493	Material 1	250	35	430.813	559.217	1304.58	862.977	441.608	1193.23	330.248
47	1.53695	1688.06	15.8485	Material 1	250	35	393.768	511.13	1209.9	836.969	372.93	1098.11	261.145
48	1.53695	1534.31	17.2132	Material 1	250	35	354.027	459.545	1107.76	808.5	299.26	998.081	189.581
49	1.53695	1372.82	18.588	Material 1	250	35	311.392	404.202	997.742	777.517	220.225	893.02	115.503
50	1.53695	1203.39	19.9774	Material 1	250	35	265.64	344.813	879.367	743.958	135.409	782.801	38.8432

Interslice Data

Global Minimum Query (bishop simplified) - Safety Factor: 1.29805

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [deg]
1	33.9299	2337	0	0	0
2	36.2222	2334.11	-4.68492	0	0
3	37.7592	2332.35	387.404	0	0
4	39.2961	2330.7	2944.48	0	0
5	40.8331	2329.15	5742.95	0	0
6	42.3701	2327.71	8461.02	0	0
7	43.907	2326.34	10934.5	0	0
8	45.444	2325.06	13205.1	0	0
9	46.9809	2323.86	15350.4	0	0
10	48.5179	2322.72	17353.8	0	0
11	50.0548	2321.65	18712	0	0
12	51.5918	2320.64	19832.5	0	0
13	53.1287	2319.69	20924.5	0	0
14	54.6657	2318.79	21979.2	0	0
15	56.2026	2317.95	22989.2	0	0
16	57.7396	2317.16	23948.1	0	0
17	59.2766	2316.42	24850.6	0	0
18	60.8135	2315.72	25692.3	0	0
19	62.3505	2315.08	26469.5	0	0
20	63.8874	2314.47	27179.2	0	0
21	65.4244	2313.91	27810.8	0	0
22	66.9613	2313.39	28342.7	0	0
23	68.4983	2312.91	28773.9	0	0
24	70.0352	2312.47	29104	0	0
25	71.5722	2312.07	29332.8	0	0
26	73.1091	2311.71	29460.5	0	0
27	74.6461	2311.38	29487.7	0	0
28	76.1831	2311.1	29415.6	0	0
29	77.72	2310.85	29247.5	0	0
30	79.257	2310.63	28991.5	0	0
31	80.7939	2310.45	28651.7	0	0
32	82.3309	2310.31	28232.4	0	0
33	83.8678	2310.2	27738.1	0	0
34	85.4048	2310.13	27174.1	0	0
35	86.9417	2310.1	26530.9	0	0
36	88.4787	2310.1	25794.1	0	0
37	90.0156	2310.13	24966.9	0	0
38	91.5526	2310.2	24053.2	0	0
39	93.0895	2310.3	23057.5	0	0
40	94.6265	2310.44	21984.7	0	0
41	96.1635	2310.62	20840.4	0	0
42	97.7004	2310.83	19630.9	0	0
43	99.2374	2311.08	18363.2	0	0
44	100.774	2311.36	17045	0	0
45	102.311	2311.68	15684.9	0	0
46	103.848	2312.04	14292.5	0	0
47	105.385	2312.44	12878.4	0	0
48	106.922	2312.87	11454.6	0	0
49	108.459	2313.35	10034.2	0	0
50	109.996	2313.87	8631.86	0	0
51	111.533	2314.43	4229.74	0	0

Entity Information

Group 1

Shared Entities

Type	Coordinates (x,y)
External Boundary	0, 2337 0, 2290 150, 2290 150, 2320 111.533, 2314.43 76.529, 2330.07 49, 2339.1 43, 2339.1 43, 2342.6 37, 2342.6 37, 2337
Material Boundary	37, 2337 37, 2332.12 49, 2332.12 49, 2339.1

Scenario-based Entities

Type	Coordinates (x,y)	Master Scenario
Water Table	-2.77556e-17, 2334.11 64.2053, 2334.11 85.4855, 2326.07 150.17, 2326.07 150, 2320	Assigned to: Material 1 Material 2
Distributed Load	49, 2332.12 37, 2332.12	Constant DistributionOrientation: VerticalMagnitude: 4050 lbs/ft2Creates Excess Pore Pressure: No
Distributed Load	37, 2337 0, 2337	Constant DistributionOrientation: Normal to boundaryMagnitude: 450 lbs/ft2Creates Excess Pore Pressure: No

Hazards by Location

Search Information

Address: 18800 Daggett Rd, Hornbrook, CA 96044,

Coordinates: 41.9732391, -122.3674723

Elevation: 2362 ft

Timestamp: 2021-06-11T22:36:43.799Z

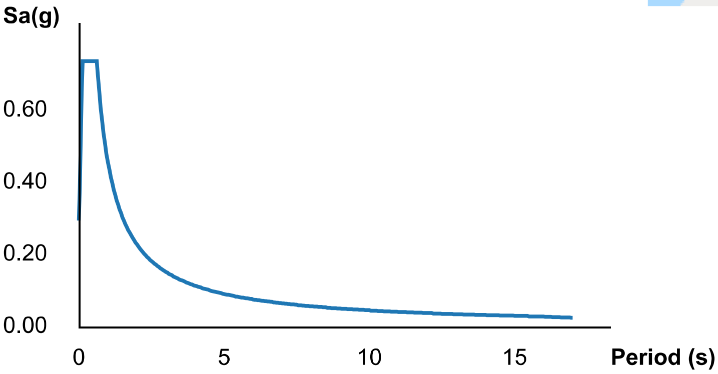
Hazard Type: Seismic

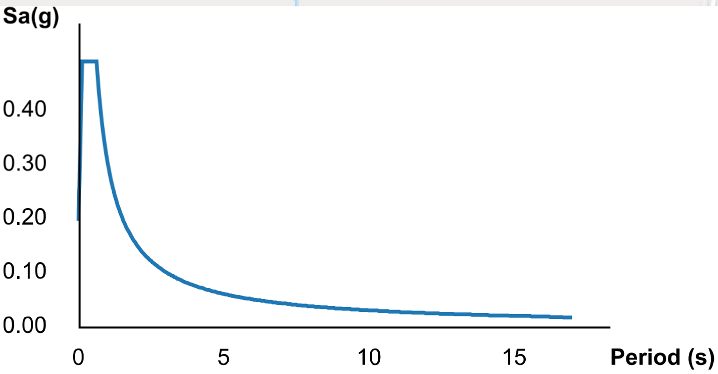
Reference Document: ASCE7-16

Risk Category: II

Site Class: C

MCER Horizontal Response Spectrum





Basic Parameters

Name	Value	Description
S_S	0.581	MCE_R ground motion (period=0.2s)
S_1	0.304	MCE_R ground motion (period=1.0s)
S_{MS}	0.737	Site-modified spectral acceleration value
S_{M1}	0.456	Site-modified spectral acceleration value
S_{DS}	0.491	Numeric seismic design value at 0.2s SA
S_{D1}	0.304	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1.267	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s

CR_S	0.893	Coefficient of risk (0.2s)
CR_1	0.877	Coefficient of risk (1.0s)
PGA	0.264	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.316	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.581	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.651	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.304	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.347	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Appendix D
Structural Design Calculations

Calculation Cover Sheet

Project: Daggett Road Bridge at Iron Gate Reservoir

Client: Klamath River Renewal Corporation **Proj. No.:** 21-067

Title: Daggett Road Bridge Abutment and Yreka Waterline Support Calculations

Prepared By, Name: KNH/GAC

Prepared By, Signature: **Date:** 6/24/2022

Peer Reviewed By, Name: ZDA

Peer Reviewed, Signature: **Date:** 6/24/2022

SUBJECT:	Klamath River Renewal Corporation	**BY:** KNH/GAC	**CHK'D BY:** 0
	Daggett Road Bridge at Iron Gate Reservoir	**DATE:** 10/15/2021	
	Daggett Road Bridge Abutment and Yreka Waterline Support Calculations	**PROJECT NO.:** 21-067	

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Purpose

Summarize general structural design information applicable to all calculations.

References

- *2017 AASHTO LRFD Bridge Design Specifications - 8th Edition*
- *ASCE 7-16: Minimum Design Loads for Buildings and Other Structures*
- *AISC 360-16: Specification for Structural Steel Buildings*

General Information

Unit Weights

γ_w =	62.4	pcf	*Water*
γ_c =	150	pcf	*Concrete*
γ_s =	125	pcf	*Soil*

Concrete Properties

f'c=	4500	psi	*Compressive Strength of Concrete*
E_c =	120915.3	ksi	*Modulus of Elasticity of Concrete*

Steel Properties

F_y =	60	ksi	*Yield Strength of Steel Reinforcing Bar*
F_u =	75	ksi	*Tensile Strength of Steel Reinforcing Bar*
E_s =	29000	ksi	*Modulus of Elasticity of Reinforcing Bar*

Soil Properties

$Ø_f$ =	35	degrees	*Internal Friction Angle*
	0.6109	radians	
K_a =	0.271		*Lateral Active Pressure Coefficient (N/A)*
K_o =	0.426		*Lateral At Rest Pressure Coefficient*
K_p =	3.690		*Passive Pressure Coefficient (N/A)*
q_{allow} =	3500	psf	*Allowable Bearing Pressure*

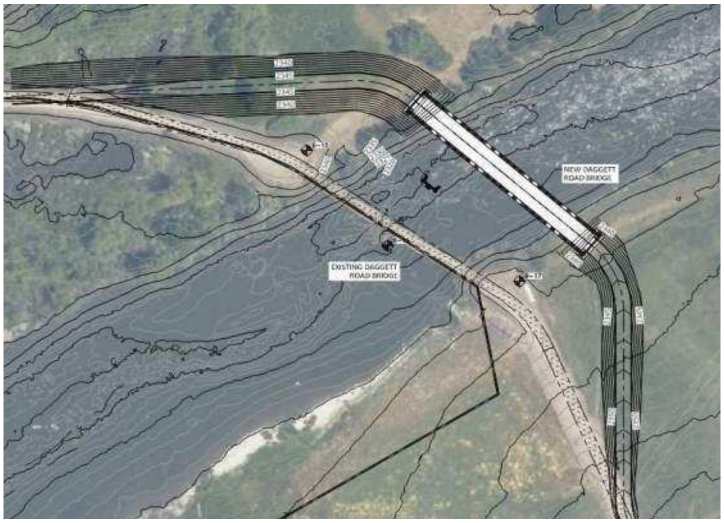
Reduction Factors

$Ø_m$ =	0.9	*AASHTO Section 5.5.4.2 for Tension-Controlled Reinforced Sections*
$Ø_v$ =	0.9	*AASHTO Section 5.5.4.2 for Shear in Reinforced Concrete Sections*
$Ø_b$ =	0.7	*AASHTO Section 5.5.4.2 for Bearing on Concrete*
$Ø_c$ =	0.75	*AASHTO Section 5.5.4.2 for Compression-Controlled Sections with Spirals or Ties*

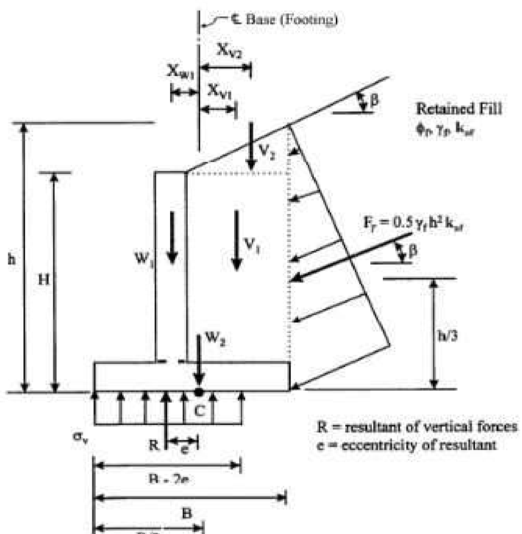
New Bridge Information

Top of Wall Elev.	2342.60 ft	Wall Height =	12.00 ft
Bottom of Wall Elev.	2332.00 ft	Length of Wall =	35.00 ft
Water Elev. (R)=	2335.10 ft	Bridge Length =	260.00 ft

Based on the Caltrans Seismic Design Criteria (SDC) Version 2.0 (April 2019), the new bridge will be classified as Ordinary Standard Bridge to determine the seismic forces for the abutment design. Based on AASHTO LRFD Bridge Design Specifications, the new bridge will be classified as an Other bridge for the general abutment design.



Abutment Bearing and Stability per AASHTO LRFD Bridge Design Specifications Section 11.6.3



Summing moments about Point C:

$$e = \frac{(F_T Cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{V1} - V_2 X_{V2} + W_1 X_{W1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Figure 11.6.3.2-1—Bearing Stress Criteria for Conventional Wall Foundations on Soil

DESIGN CRITERIA PROVIDED TO BRIDGE MANUFACTURER FOR SUPERSTRUCTURE

Dead Loads

- *24" Diameter, 1/4" Thick Pipe (Waterline): 490 pcf*
- *Weight of Water in Pipe: 62.4 pcf*

Live Loads

- *HL-93 Vehicle Load*

Snow Loads

- *Uniform Snow Load: 40 psf*

Seismic Criteria

Site Class determination was made based on the blow counts documented by the Standard Penetration Test (SPT) and boring logs (15 and 17) in the Geotechnical Report and AASHTO Table 3.10.3.1-1. The new bridge is classified as Other

- N_{avg_15} = (1.5ft + 1.5ft)/((1.5ft/54 blows/ft) + (1.5ft/100 blows/ft) = 70.1
- N_{avg_17} = 100 (Refusal Before 100)
- *Site Class C*

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s < 5{,}000$ ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s < 2{,}500$ ft/s, or with either $\bar{N}$ > 50 blows/ft, or $\bar{s}_u$ > 2.0 ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s < 1{,}200$ ft/s, or with either 15 < $\bar{N}$ < 50 blows/ft, or 1.0 < $\bar{s}_u$ < 2.0 ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays (H > 10.0 ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays (H > 25.0 ft with PI > 75) • Very thick soft/medium stiff clays (H >120 ft)

PGA = 0.2137
Ss = 0.4949
S1 = 0.2132

Using Site Class C and AASHTO Tables 3.10.3.2-1 through 3.10.3.2-3

F_{pga} = 1.1863
F_a = 1.200
F_v = 1.587

A_s = 0.2535
S_{DS} = 0.5939
S_{D1} = 0.3383
T_s = 0.5697
T_0 = 0.1139
C_{sm} = 0.5939 *Equal to SDS per single-mode method*

Seismic Zone 3

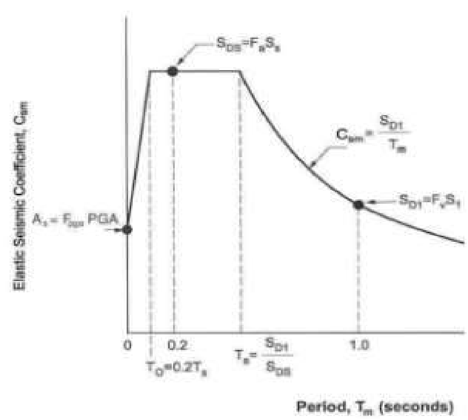


Figure 3.10.4.1-1—Design Response Spectrum

Load Combinations and Load Factors from AASHTO

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.00	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	1.00	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	1.00	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	γ_{LL}	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	1.00	—	1.00	1.00/1.20	—	1.00	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.75	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.80	—	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.40	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (2010) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
○ Metal Box Culverts, Structural Plate Culverts with Deep Corrugations, and Fiberglass Culverts		1.50	0.90
○ Thermoplastic Culverts		1.30	0.90
○ All others		1.95	0.90
ES: Earth Surcharge		1.50	0.75

Design Criteria for Substructure (Abutments)

SUMMARY OF PROVIDED AND CALCULATED VERTICAL LOADS

Dead Loads

- *Self Weight of Abutment: 150 pcf*
- *Weight of Soil: 125 pcf*
- *Weight of Wearing Surface: 35 psf (Included in Reactions Provided by Manufacturer)*
- *24" Diameter Waterline (Assuming Full)*

Calculated Allowable Bearing Pressure at Footing (Geotechnical Calculations)

q_{allow} = 8.90 ksf

Calculated Superimposed Dead Loads (DL)

W_{pipe_water} = 196.04 plf

W_{pipe_self} = 97.715 plf

Provided Superimposed Dead Loads (DL)

P_{DL_Pipe} = 0.0 kips *Dead Load Reaction from 24" Waterline at Each Abutment (Included in Bridge Reaction)*

P_{DL_Veh} = 244.00 kips *Dead Load Reaction from Bridge (Provided by Manufacturer) - 2 Per Abutment*

Provided Superimposed Live Loads (LL)

P_{LL_Veh} = 165.00 kips *Live Load Reaction from Bridge (Provided by Manufacturer) - 2 Per Abutment*

Provided Superimposed Wind Loads (WL)

P_{L_W} = 86.00 kips *Wind Load from Bridge (Provided by Manufacturer)*

Provided Superimposed Earthquake Loads (EQ)

P_{L_EQ} = 62.00 kips *Seismic Load from Bridge (Provided by Manufacturer)*

****No Seismic Analysis of Superstructure Required for Single-Span Bridges per AASHTO Table 4.7.4.3.1-1. Seismic Loads Based on Product of Permanent Dead Load and Acceleration Coefficient***

Calculated Earthquake Loads (EQ) For Connection Design

P_{L_EQ} = 61.86 kips *Seismic Load from Superstructure at Each Abutment for Connection Design*

SUBJECT: Klamath River Renewal Corporation
Daggett Road Bridge at Iron Gate Reservoir
Daggett Road Bridge Abutment and Yreka Water

BY: KNH/GAC **CHK'D BY:** 0
DATE: ########
PROJECT NO.: 21-067

Bridge Name: Daggett Road Bridge
Lat/Long: 41.972865, -122.364372

ARS INFORMATION PROVIDED BY CALTRANS: https://arsonline.dot.ca.gov/output2-5.php

Period, T (s)	Design Spectral Acceleration, Sa (g)
0.00	0.21
0.10	0.4
0.20	0.49
0.30	0.45
0.50	0.34
0.75	0.26
1.00	0.21
2.00	0.11
3.00	0.07
4.00	0.05
5.00	0.04

Caltrans Design Spectrum (5% damping)

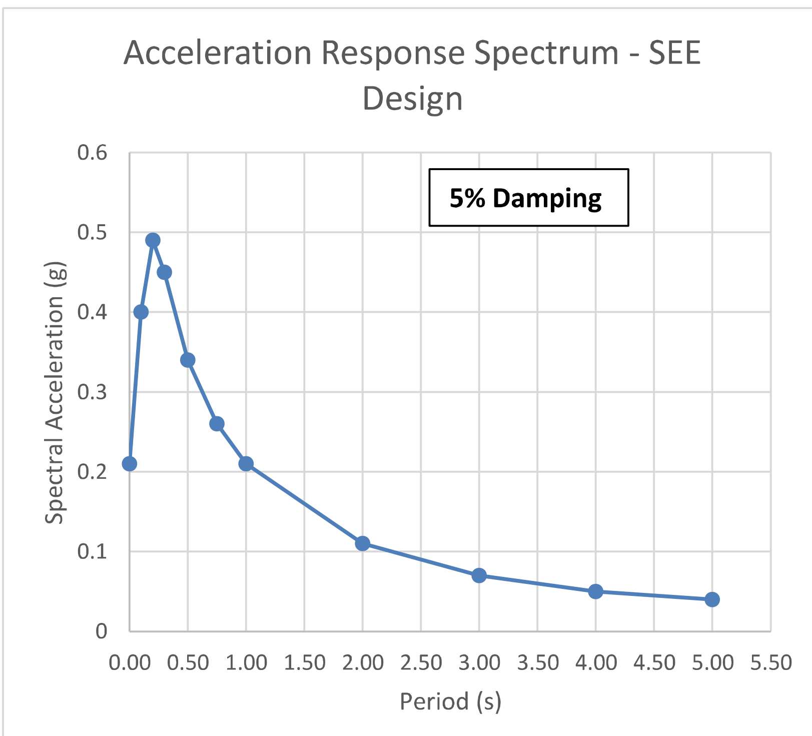
Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.22	0.21	1	1	1	0.22	0.21
0.10	0.43	0.4	1	1	1	0.43	0.4
0.20	0.52	0.49	1	1	1	0.52	0.49
0.30	0.5	0.45	1	1	1	0.5	0.45
0.50	0.39	0.34	1	1	1	0.39	0.34
0.75	0.29	0.26	1	1	1	0.29	0.26
1.0	0.22	0.21	1	1	1	0.22	0.21
2.0	0.12	0.11	1	1	1	0.12	0.11
3.0	0.07	0.07	1	1	1	0.07	0.07
4.0	0.04	0.05	1	1	1	0.04	0.05
5.0	0.03	0.04	1	1	1	0.03	0.04

Copy table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA)	7.72
mean site-source distance (km, for Sa at 1s)	108.7

V_{s30} = 540 m/s *Site Class C*
PGA = 0.2137
Mean Moment Magnitude (For PGA) = 7.72



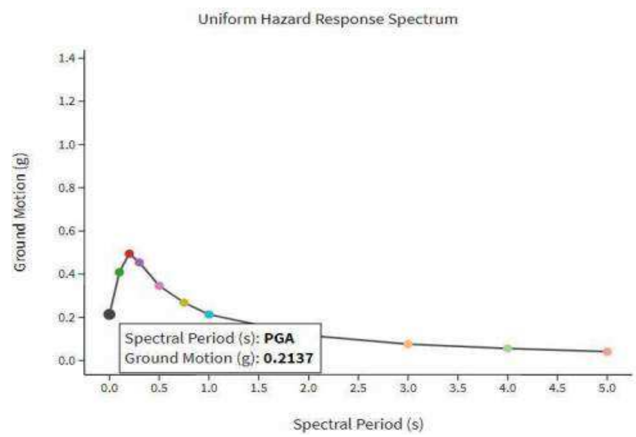
The ARS was based on the USGS 2014 National Seismic Hazard Map for **975-years return period**, *Hazard Model/Edition Dynamic Conterminous U.S. 2014 (Update)(v4.2.0)* hazard data obtained by using ARS Online V3.0

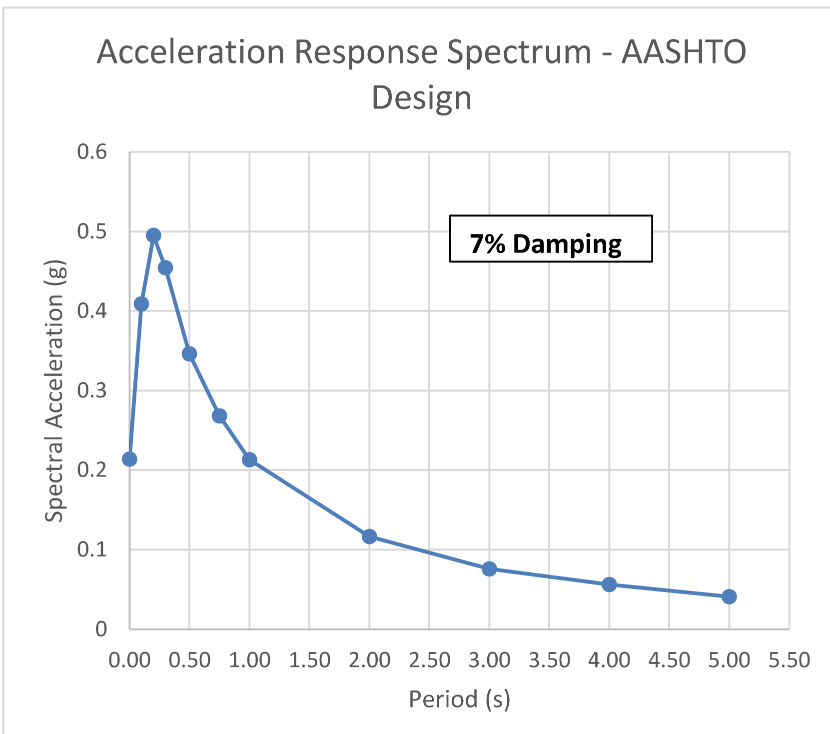
No Near Fault or Basin Amplification Factors Required

Bridge Name: Daggett Road Bridge
Lat/Long: 41.972865, -122.364372

ARS INFORMATION PROVIDED BY USGS: https://earthquake.usgs.gov/hazards/interactive/

Period, T (s)	Design Spectral Acceleration, Sa (g)
0.00	0.2137
0.10	0.4088
0.20	0.4949
0.30	0.4542
0.50	0.3461
0.75	0.2682
1.00	0.2132
2.00	0.1163
3.00	0.0759
4.00	0.056
5.00	0.041





The ARS was based on the USGS 2014 National Seismic Hazard Map for **1000-years return period**, *Dynamic Conterminous U.S. 2014 (Update)(v4.2.0)* hazard data obtained by using online Unified Hazard Tool

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Klamath River Renewal Corporation	**BY:** KNH/GAC	**CHK'D BY:** 0
	Daggett Road Bridge at Iron Gate Reservoir	**DATE:** 10/15/2021	
	Daggett Road Bridge Abutment and Yreka Waterline Support Calculations	**PROJECT NO.:** 21-067	

Purpose

Design of 24" diameter waterline support to resist vertical and lateral loads due to pipe and environmental loads, including wind and seismic lateral loads.

Calculations

24" Waterline Calculations: General Properties and Loads

Pipe Properties:

CEMENT MORTAR LINING

O.D. SIZE	THEORETICAL WEIGHTS PER LINEAL FOOT FOR CEMENT MORTAR LINING						
	THICKNESS 1/4"	THICKNESS 5/16"	THICKNESS 3/8"	THICKNESS 1/2"	THICKNESS 5/8"	THICKNESS 3/4"	THICKNESS 1"
	POUNDS PER LINEAL FOOT						
3.500"	2.66						
4.500"	3.42	4.20					
6.625"	5.18	6.41	7.61				
8.625"	6.83	8.48	10.09				
10.75"	8.59	10.68	12.73				
12.750"	10.25	12.75	15.22	20.08			
15.250"	12.94	16.11	19.25	25.46	31.57	37.57	48.85
17.375"	15.59	18.18	21.74	28.77	35.71	42.54	55.48
19.781"		20.70	24.06	31.88	39.59	47.20	62.10
21.781"		22.77	26.55	35.19	43.73	52.16	68.72
25.750"			31.52	41.81	52.01	52.10	81.97
31.875"			38.64	51.75	64.43	77.00	101.84
37.875"			45.92	61.69	76.85	91.90	121.71
43.875"				71.56	89.18	106.69	141.39
49.875"				81.56	99.18	116.69	151.39

CEMENT MORTAR COATING

O.D. SIZE	THEORETICAL WEIGHTS PER LINEAL FOOT FOR CEMENT MORTAR COATING					
	THICKNESS 1/2"	THICKNESS 5/8"	THICKNESS 3/4"	THICKNESS 1"	THICKNESS 1 1/4"	THICKNESS 1 1/2"
	POUNDS PER LINEAL FOOT					
3.500"	6.44	8.25	10.14	14.17	18.51	23.18
4.500"	8.28	10.61	13.04	18.22	23.81	29.81
6.625"	11.80	15.01	18.32	25.26	32.61	40.36
8.625"	15.11	19.15	23.29	31.88	40.88	50.30
10.75"	18.63	23.55	28.57	38.92	49.68	60.86
12.750"	21.94	27.69	33.54	45.54	57.96	70.78
15.250"		34.42	41.61	56.30	71.42	86.91
17.375"		38.56	46.58	62.93	79.70	96.97
19.781"		42.70	51.55	69.55	87.98	106.83
21.781"			56.51	76.17	96.26	116.70
25.750"			66.45	89.48	112.81	136.62
31.875"			81.35	109.29	137.65	166.42
37.875"			96.26	129.16	162.49	196.21
43.875"			111.25	149.19	187.58	226.42
49.875"			126.03	163.97	202.36	241.20

Nominal Diameter	AWWA Pipe Size (O.D.)
4"	4 1/2" (4.500)
6"	6 5/8" (6.625)
8"	8 5/8" (8.625)
10"	10 3/4" (10.750)
12"	12 3/4" (12.750)
14"	15 1/4" (15.250)
16"	17 3/8" (17.375)
18"	19 25/32" (19.781)
20"	21 25/32" (21.781)
24"	25 3/4" (25.750)
30"	31 7/8" (31.875)
36"	37 7/8" (37/875)
40"	41 7/8" (41.875)
42"	43 7/8" (43.875)
48"	49 7/8" (49.875)

Nominal I.D. sizes refer to Cement Lined Pipe

γ_w =	62.4	pcf	*Unit Weight of Water*
γ_s =	490.0	pcf	*Unit Weight of Steel*
F_{y_pipe} =	50.00	ksi	*Yield Strength of Steel Pipe (City of Yreka Calculations)*
E_{pipe} =	29000.00	ksi	*Modulus of Elasticity of Steel Pipe (City of Yreka Calculations)*
t_{pipe} =	0.38	in	*Pipe Wall Thickness (City of Yreka Calculations - Assuming 1/2" Wall Thickness Max)*
d_{pipe_in} =	24.0	in	*Nominal Pipe Diameter (Inside Diameter of Pipe)*
d_{pipe_out} =	24.8	in	*Outside Diameter of Pipe (Assuming 1/2" Wall Thickness)*
A_{pipe} =	28.72	in^2	*Area of Pipe*
S_{pipe} =	172.38	in^3	*Section Modulus of Pipe*
W_{water} =	196.04	plf	*Weight of Water Assuming Pipe is Full*
W_{pipe_shell} =	97.71	plf	*Weight of Pipe*
W_{lining} =	0.00	plf	*Theoretical Weight of 0.375" Mortar Lining (Doesn't Occur at Bridge)*
$W_{coating}$ =	0.00	plf	*Theoretical Weight of 0.75" Mortar Coating (Doesn't Occur at Bridge)*
W_{pipe_empty} =	97.71	plf	*Uniform Weight of Empty Pipe*
W_{pipe_full} =	**293.75**	**plf**	***Uniform Weight of Full Pipe***
L_{span} =	10.00	ft	*Typical Pipe Span (Attachment at Every Other Transom Beam)*

$$A_{pipe} = \frac{\pi(d_o^2 - d_i^2)}{4}$$

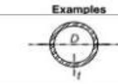
$$S_{pipe} = \frac{\pi(d_o^4 - d_i^4)}{32 * d_o}$$

$$W_{water} = \gamma_w * \pi * \left(\frac{d_{pipe\_in}}{2}\right)^2$$

$$W_{pipe\_shell} = \gamma_s * A_{pipe}$$

24" Waterline Calculations: Moment Capacity Check of Pipe for Required Span

D/t =	66	*Pipe Outside Diameter to Thickness Ratio*
0.45 * E_{pipe} / F_{y_pipe} =	261	
IF 0.45*E_{pipe} /F_{y_pipe} ≥ D/t	Use AISC 360-10 Section F8	
λ_p =	40.6	0.07 * E_{pipe} / F_{y_pipe}
λ_r =	179.8	0.31 * E_{pipe} / F_{y_pipe}
λ_p ≤ D/t ≤ λ_r	Member Noncompact	

Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (compact/noncompact)	Limiting Width-to-Thickness Ratio λ_r (noncompact/slender)	Examples
20	Round HSS	D/t	$0.07\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	

$$M_a = \frac{W_{pipe\_full} * L_{span}^2}{8}$$

Ω =	1.67		*Safety Factor for Flexure*
M_a =	3.67	k-ft	*Moment Applied to Pipe Due to Self-Weight and Water*
M_n / Ω =	430.09	k-ft	*Flexural Strength of Pipe (Yielding Per AISC Section F8.1)*
M_n / Ω =	509.46	k-ft	*Flexural Strength of Pipe (Local Buckling Per AISC Section F8.2)*
IF ØM_n ≥ M_a	**GOOD**		***Supports at 10'-0" oc. are OK***

$$M_n = M_p = F_y Z$$

$$M_n = \left(\frac{0.021E}{\left(\frac{D}{t}\right)} + F_y\right)S$$

Waterline Support Calculations: Load Determination

Self-Weight:

$P_{support}$ =	2937.50	lbs	*Reaction at Each Pipe Support Location (Transom Beam)*
$\mathbf{P_{vert}}$ =	**1468.75**	**lbs**	***Reaction at Each Pipe Support Hanging Beam (2 Per Transom Beam)***
μ =	0.50		*Kinetic Coefficient of Friction (Assumed Steel-Steel with Not Clean Surface)*
$F_{sliding}$ =	1468.75	lbs	*Lateral Reaction at Each Pipe Support Location (Transom Beam) Due to Sliding Friction*
$\mathbf{P_{friction}}$ =	**734.37**	**lbs**	***Reaction at Each Pipe Support Hanging Beam (2 Per Transom Beam)***

Wind:

26.10.2 Velocity Pressure. Velocity pressure, q_z, evaluated at height z above ground shall be calculated by the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d K_e V^2 \text{ (lb/ft}^2\text{)}; \ V \text{ in mi/h} \qquad (26.10\text{-}1)$$

V_{ult} =	115	mph	*Design Wind Speed*
Exposure =	C		*Exposure Category (ASCE 7-16 Section 26.7.3)*
K_z =	0.85		*Velocity Pressure Coefficient (ASCE 7-16 Table 26.10-1)*
K_{zt} =	1.00		*Topographic Factor (ASCE 7-16 Section 26.8-1)*
K_d =	0.95		*Wind Directionality Factor (ASCE 7-16 Table 26.6-1)*
K_e =	1.00		*Ground Elevation Factor (ASCE 7-16 Table 26.9-1)*
q_z =	27.34	psf	*Velocity Pressure*
w_{wind} =	56.39	plf	*Uniform Loading Along Full Length of Pipe*
$\mathbf{P_{wind}}$ =	**281.93**	**lbs**	***Reaction at Each Pipe Support Hanging Beam (2 Per Transom Beam)***

Seismic:

13.3.1.1 Horizontal Force. The horizontal seismic design force (F_p) shall be applied at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined in accordance with Eq. (13.3-1):

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) \qquad (13.3\text{-}1)$$

F_p is not required to be taken as greater than

$$F_p = 1.6 S_{DS} I_p W_p \qquad (13.3\text{-}2)$$

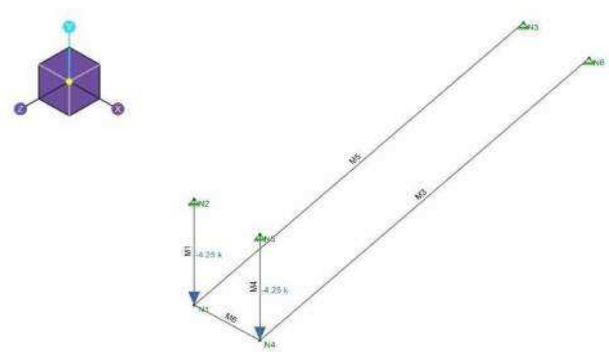
and F_p shall not be taken as less than

$$F_p = 0.3 S_{DS} I_p W_p \qquad (13.3\text{-}3)$$

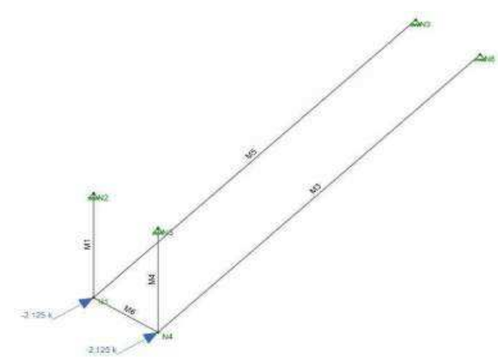
13.3.1.2 Vertical Force. The component shall be designed for a concurrent vertical force $\pm 0.2 S_{DS} W_p$.

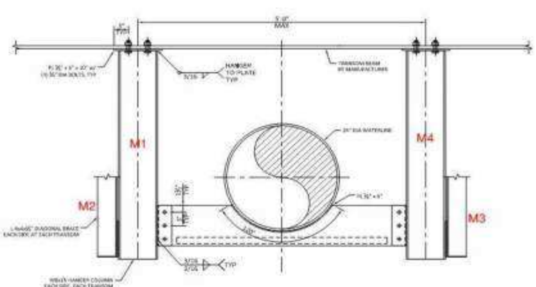
S_{DS} =	0.594		*Design Earthquake Spectral Response Acceleration Parameter at Short Periods*
I_p =	1.50		*Importance Factor per ASCE 7-10 Section 13.1.3*
a_p =	2.50		*ASCE 7-10 Table 13.6-1*
R_p =	6.00		*ASCE 7-10 Table 13.6-1*
z/h =	1.00		*ASCE 7-10 Section 13.3.1*
F_p =	130.84	plf	*Uniform Seismic Loading Along Full Length of Pipe (ASCE 7-10 Eq. 13.3-1)*
F_{p_min} =	78.50	plf	*Minimum Uniform Seismic Loading Along Full Length of Pipe (ASCE 7-10 Eq. 13.3-3)*
F_{p_max} =	418.69	plf	*Maximum Uniform Seismic Loading Along Full Length of Pipe (ASCE 7-10 Eq. 13.3-2)*
	Use Fp		
$\mathbf{F_{p\_lat}}$ =	**654.20**	**lbs**	***Reaction at Each Pipe Support Hanging Beam (2 Per Transom Beam)***
$\mathbf{F_{p\_vert}}$ =	**174.45**	**lbs**	***Reaction at Each Pipe Support Hanging Beam (2 Per Transom Beam)***

Vertical Loading Application: Pipe Load Shown



Lateral Loading Application: Friction Force Shown





Load Combinations

Combinations | Design

LC Generator | RSA Scaling Factor

	Description	Solve	P-Delta	SRSS	BLC	Factor	BLC	Factor	BLC	Factor
1	IBC 16-1	☑	Y		DL	1.4				
2	IBC 16-2 (a)	☑	Y		DL	1.2	LL	1.6		
3	IBC 16-3 (...	☑	Y		DL	1.2	WLX	0.5		
4	IBC 16-3 (...	☑	Y		DL	1.2	WLZ	0.5		
5	IBC 16-3 (...	☑	Y		DL	1.2	WLX	-0.5		
6	IBC 16-3 (...	☑	Y		DL	1.2	WLZ	-0.5		
7	IBC 16-4 (...	☑	Y		DL	1.2	WLX	1	LL	0.5
8	IBC 16-4 (...	☑	Y		DL	1.2	WLZ	1	LL	0.5
9	IBC 16-4 (...	☑	Y		DL	1.2	WLX	-1	LL	0.5
10	IBC 16-4 (...	☑	Y		DL	1.2	WLZ	-1	LL	0.5
11	IBC 16-6 (a)	☑	Y		DL	0.9	WLX	1		
12	IBC 16-6 (b)	☑	Y		DL	0.9	WLZ	1		
13	IBC 16-6 (c)	☑	Y		DL	0.9	WLX	-1		
14	IBC 16-6 (d)	☑	Y		DL	0.9	WLZ	-1		
15	IBC 16-5 (a)	☑	Y		DL	1.2	EL	1	LL	0.5
16	IBC 16-5 (b)	☑	Y		DL	1.2	EL	-1	LL	0.5
17	IBC 16-7 (a)	☑	Y		DL	0.9	EL	1		
18	IBC 16-7 (b)	☑	Y		DL	0.9	EL	-1		

Reactions:

Worst Case Nodes 2 and 5

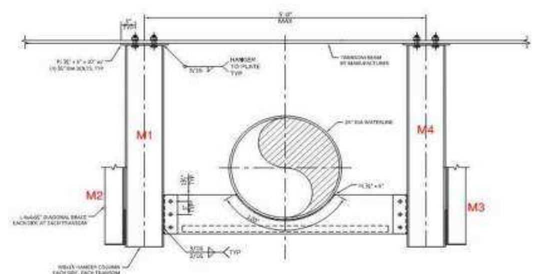
P_{max} =	**3.305**	**kips**	***LRFD Vertical Reaction at Each Pipe Support Location (2 Per Transom Beam)***
V_x =	0.323	kips	*LRFD Lateral Reaction at Each Pipe Support Location (2 Per Transom Beam)*
V_z =	0.045	kips	*LRFD Lateral Reaction at Each Pipe Support Location (2 Per Transom Beam)*
M_x =	0.000	k-ft	*LRFD Moment Reaction at Each Pipe Support Location (2 Per Transom Beam)*
M_z =	0.000	k-ft	*LRFD Moment Reaction at Each Pipe Support Location (2 Per Transom Beam)*

Worst Case Nodes 3 and 6

P_{max} =	**0.251**	**kips**	***LRFD Vertical Reaction at Each Pipe Support Location (2 Per Transom Beam)***
V_x =	0.000	kips	*LRFD Lateral Reaction at Each Pipe Support Location (2 Per Transom Beam)*
V_z =	1.155	kips	*LRFD Lateral Reaction at Each Pipe Support Location (2 Per Transom Beam)*
M_x =	0.000	k-ft	*LRFD Moment Reaction at Each Pipe Support Location (2 Per Transom Beam)*
M_z =	0.000	k-ft	*LRFD Moment Reaction at Each Pipe Support Location (2 Per Transom Beam)*

W8x15 Member Check: Members M1 and M4

A =	4.44	in^2	*Area of Beam*
d =	8.11	in	*Beam depth*
t_{web} =	0.245	in	*Web Thickness*
b_f =	4.020	in	*Flange Width*
t_f =	0.315	in	*Flange Thickness*



AISC 15th (360-16): LRFD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	15	-	-	-	-
Applied Loading - Shear + Torsion	17	-	-	-	-
Axial Tension Analysis	15	3.251 k	199.8 k	-	-
Axial Compression Analysis	15	0.000 k	176.608 k	-	-
Flexural Analysis (Strong Axis)	15	0.957 k-ft	51 k-ft	-	-
Flexural Analysis (Weak Axis)	15	0.133 k-ft	10.012 k-ft	-	-
Shear Analysis (Major Axis y)	17	0.319 k	59.609 k	0.005	Pass
Shear Analysis (Minor Axis z)	17	0.036 k	68.38 k	0.000	Pass
Bending & Axial Interaction Check (UC Bending Max)	15	-	-	0.04	Pass

L4x4x1/4 Brace Member Check: Members M2 and M3

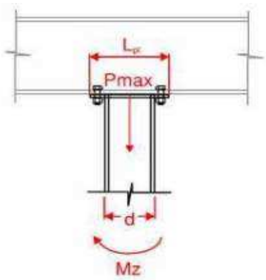
A =	1.94	in^2	*Area of Angle*
d =	4	in	*Angle Leg Depth*
t_{leg} =	0.250	in	*Angle Leg Thickness*

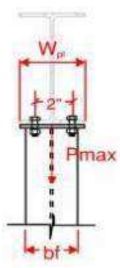
AISC 15th (360-16): LRFD Code Check

Limit State	Gov. LC	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	15	-	-	-	-
Applied Loading - Shear + Torsion	1	-	-	-	-
Axial Tension Analysis	15	0.000 k	62.532 k	-	-
Axial Compression Analysis	15	1.107 k	17.031 k	-	-
Flexural Analysis (Strong Axis)	15	0.143 k-ft	5.361 k-ft	-	-
Flexural Analysis (Weak Axis)		0.046 k-ft	3.138 k-ft	-	-
Shear Analysis (Major Axis y)	1	0.056 k	19.44 k	0.003	Pass
Shear Analysis (Minor Axis z)	1	0.001 k	19.44 k	0.000	Pass
Bending & Axial Interaction Check (UC Bending Max)	15	-	-	0.107	Pass

Connection Calculation: Hanging Column to Transom Beam

Tensile Yielding and Rupture: AISC 360-16 Section J4.1





P_{u_vert} =	3.556	kips	*LRFD Tension Force Due to Vertical Load of Column and Brace (Per Plate - 2 Per Connection)*	
P_u =	**0.889**	**kips**	***Maximum LRFD Tension Force Applied to Each Girder Clamp (4 Total)***	
Ø =	0.9		*Reduction Factor for Tensile Yielding (EQ J4-1)*	
Ø =	0.75		*Reduction Factor for Tensile Rupture (EQ J4-2)*	
F_y =	36.0	ksi	*Yield Strength of Plate*	
F_u =	58.0	ksi	*Tensile Strength of Plate*	
E =	29000.0	ksi	*Modulus of Elasticity*	
d_{bolt} =	0.750	in	*Bolt Diameter*	
t_{plate} =	0.375	in	*Plate Thickness*	
l_{plate} =	10	in	*Length of Plate*	
w_{plate} =	5	in	*Width of Plate*	
A_{hole} =	0.305	in^2	*Area of Hole*	
U =	1.0		*Shear Lag Factor (Table D3.1)*	
A_g =	1.875	in^2	*Gross Area of Plate*	
A_n =	0.656	in^2	*Net Area Subject to Tension*	
A_e =	0.656	in^2	*Effective Net Area in Tension*	$A_e = A_n * U$
$ØR_n$ =	60.75	kips	*Tensile Yielding Capacity (EQ J4-1)*	$ØR_n = Ø * F_y * A_g$
$ØR_n$ =	28.55	kips	*Tensile Rupture Capacity (EQ J4-2)*	$ØR_n = Ø * F_u * A_e$
$ØR_n \geq P_u$	**GOOD**		***Pipe Support Plate Tension Check***	

Shear Yielding, Rupture and Block Shear: AISC 360-16 Section J4.2 and J4.3

V_{u_lat} =	1.200	kips	*LRFD Shear Force Due to Lateral Loads*	
V_u =	**0.300**	**kips**	***Maximum LRFD Shear Force Applied to Each Girder Clamp (4 Total)***	
Ø =	1.0		*Reduction Factor for Shear Yielding (EQ J4-3)*	
Ø =	0.75		*Reduction Factor for Shear Rupture (EQ J4-3)*	
A_{gv} =	1.875	in^2	*Gross Area of Plate*	
A_{nt} =	0.656	in^2	*Net Area Subject to Tension*	
A_{nv} =	0.656	in^2	*Net Area Subject to Shear*	
$ØR_n$ =	40.50	kips	*Shear Yielding Capacity (EQ J4-3)*	$ØR_n = Ø * 0.6 * F_y * A_{gv}$
$ØR_n$ =	17.13	kips	*Shear Rupture Capacity (EQ J4-4)*	$ØR_n = Ø * 0.6 * F_u * A_{nv}$
$ØR_n$ =	45.68	kips	*Block Shear Strength (EQ J4-4)*	$ØR_n = Ø(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq Ø(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$
$ØR_n \geq V_u$	**GOOD**		***Pipe Support Plate Shear Check***	

Compression Yielding and Buckling: AISC 360-16 Section J4.4

S_{plate} =	0.117	in^3	*Section Modulus About Strong Axis*	
A_{plate} =	1.875	in^2	*Area of Plate*	
$ØR_n$ =	60.75	kips	*Compression Yielding/Buckling Capacity (EQ J4-6)*	$ØR_n = Ø * F_y * A_g$
$ØR_n \geq P_u$	**GOOD**		***Pipe Support Plate Compression Check***	

Flexural Yielding: AISC 360-16

M_{u_x} =	8.890	k-in	*LRFD Moment Applied to Plate Due to Vertical Load*	
M_{u_x} =	**0.741**	**k-ft**	***LRFD Moment Applied to Plate Due to Vertical Load***	
$ØM_n$ =	3.80	k-ft	*Flexural Capacity of Plate About Strong Axis*	$ØM_n = Ø * F_y * S_{plate}$
$ØM_n \geq M_{u\_x}$ =	**GOOD**		***Pipe Support Plate Flexure Check***	

Bolt Bearing and Tearout: AISC 360-16 Section J3.10

$\emptyset R_n = \emptyset * 2.4 * d_{bolt} * t_{plate} * F_u$ | $ØR_n$ = 29.36 kips | *Available Strength at Bolt Hole Due to Bearing (EQ J3-6a)*

$\emptyset R_n = \emptyset * 1.2 * l_c * t_{plate} * F_u$ | $ØR_n$ = 97.88 kips | *Available Strength at Bolt Hole Due to Tearout (EQ J3-6c)*

$ØR_n \geq P_u$ GOOD | ***3/4" Diameter Bolts are OK for Use in 3/4" Thick Plate***

Allowable Lindapter Girder Clamp Loads

Technical Specification

IMPERIAL DATA | METRIC DATA

Product Code	Bolt Grd. 5 / A325 Z	Safe Working Loads (FOS 5:1) Tensile Resistance / 1 Bolt lbs	Slip Resistance / 2 Bolts lbs	Tightening Torque* ft lb	Clamping Range[1] V	Dimensions Y	X	T	Width with Saddle
LLR037	3/8"	337	-	15	1/8" - 3/8"	13/16" - 15/16"	15/16" - 1"	13/16" - 15/16"	1 5/16"
LLR050	1/2"	1304	202	50	1/8" - 1/2"	1" - 1 1/8"	1" - 1 1/4"	1" - 1 1/8"	1 9/16"
LLR062	5/8"	1911	382	108	1/8" - 5/8"	1 3/16" - 1 3/8"	1 5/16" - 1 7/16"	1 3/16" - 1 7/16"	1 13/16"
LLR075	3/4"	3305	674	210	1/8" - 3/4"	1 5/8" - 1 15/16"	1 13/16" - 2"	1 5/8" - 1 7/8"	2 1/4"
LLR100	1"	4430	1012	362	1/8" - 1"	1 7/8" - 2 1/4"	2 1/16" - 2 1/4"	1 3/4" - 2 1/8"	3"

Girder Clamp Check: Combined Shear and Tension

P_u =	**0.889**	**kips**	***Maximum LRFD Tension Force Applied to Each Girder Clamp (4 Total)***
V_u =	**0.300**	**kips**	***Maximum LRFD Shear Force Applied to Each Girder Clamp (4 Total)***
$ØR_{n\_tension}$ =	3.31	kips	*Available Tensile Strength of A307 3/4" Diameter Bolt per Table 7-2*
$ØR_{n\_shear}$ =	0.67	kips	*Available Tensile Strength of A307 3/4" Diameter Bolt per Table 7-2*
Utilization Ratio:	**0.71**		**Ratio < 1 for Combined Shear and Tension Loads**
	GOOD		***3/4" Diameter Bolt Girder Clamp (Lindapter LLR075) OK for Hanging Pipe Support Connection***

Plate 3/8" x 5" x 10" with (4) 3/4" Diameter Lindapter LLR075 Girder Clamps to Transom Beam OK

Weld Check: Combined Shear and Tension

P_{u_vert} =	3.305	kips	*Tension Load at Weld due to Vertical Load*
V_{u_total} =	1.200	kips	*Shear Load at Weld due to Lateral Loads*
t_{weld} =	3	in	*Weld Thickness in Sixteenths of an Inch*
L_{weld} =	12	in	*Weld Length Along Hanging Column Perimeter*
$ØR_n$ =	50.11	kips	*AISC 360-16 Chapter 8*
Weld Utilization Ratio:	**0.09**		**Ratio < 1 for Combined Shear and Tension Loads**
	GOOD		***3/16" Fillet Weld OK for Hanging Pipe Support Connection***

3/16" Fillet Weld from Hanging Column to Plate OK

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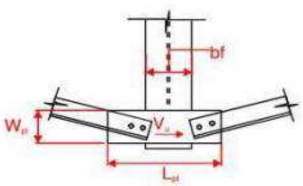
Connection Calculation: Hanging Column to Brace

Tensile Yielding and Rupture: AISC 360-16 Section J4.1



**Bottom connection shown - top connection similar*

V_u =	3.516	kips	*Shear Load at Base of Hanging Column Due to Lateral Loads*
V_{u_total} =	**1.758**	**kips**	***Shear Load at Each Bolt Location (2 Total)***
Ø =	0.9		*Reduction Factor for Tensile Yielding (EQ J4-1)*
Ø =	0.75		*Reduction Factor for Tensile Rupture (EQ J4-2)*
F_y =	36.0	ksi	*Yield Strength of Plate*
F_u =	58.0	ksi	*Tensile Strength of Plate*
E =	29000.0	ksi	*Modulus of Elasticity*
d_{bolt} =	0.750	in	*Bolt Diameter*
t_{plate} =	0.375	in	*Plate Thickness*
l_{plate} =	6	in	*Minimum Length of Plate*
w_{plate} =	6	in	*Minimum Width of Plate*
A_{hole} =	0.305	in^2	*Area of Hole*
U =	1.0		*Shear Lag Factor (Table D3.1)*
A_g =	2.25	in^2	*Gross Area of Plate*
A_n =	1.031	in^2	*Net Area Subject to Tension*
A_e =	1.031	in^2	*Effective Net Area in Tension* $A_e = A_n * U$
$ØR_n$ =	72.90	kips	*Tensile Yielding Capacity (EQ J4-1)* $ØR_n = Ø * F_y * A_g$
$ØR_n$ =	44.86	kips	*Tensile Rupture Capacity (EQ J4-2)* $ØR_n = Ø * F_u * A_e$
$ØR_n \geq P_u$	**GOOD**		***Pipe Support Plate Tension Check***

Shear Yielding, Rupture and Block Shear: AISC 360-16 Section J4.2 and J4.3

Ø =	1.0		*Reduction Factor for Shear Yielding (EQ J4-3)*
Ø =	0.75		*Reduction Factor for Shear Rupture (EQ J4-3)*
A_{gv} =	2.25	in^2	*Gross Area of Plate*
A_{nt} =	1.031	in^2	*Net Area Subject to Tension*
A_{nv} =	1.031	in^2	*Net Area Subject to Shear*
$ØR_n$ =	48.60	kips	*Shear Yielding Capacity (EQ J4-3)* $ØR_n = Ø * 0.6 * F_y * A_{gv}$
$ØR_n$ =	26.92	kips	*Shear Rupture Capacity (EQ J4-4)* $ØR_n = Ø * 0.6 * F_u * A_{nv}$
$ØR_n$ =	71.78	kips	*Block Shear Strength (EQ J4-4)* $ØR_n = Ø(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq Ø(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$
$ØR_n \geq V_u$	**GOOD**		***Pipe Support Plate Shear Check***

Compression Yielding and Buckling: AISC 360-16 Section J4.4

S_{plate} =	0.141	in^3	*Section Modulus About Strong Axis*
A_{plate} =	2.250	in^2	*Area of Plate*
$ØR_n = Ø * F_y * A_g$ $ØR_n$ =	72.90	kips	*Compression Yielding/Buckling Capacity (EQ J4-6)*
$ØR_n \geq P_u$	**GOOD**		***Pipe Support Plate Compression Check***

Bolt Check:

$ØR_{n\_shear}$ =	7.95	kips	*Available Tensile Strength of A307 3/4" Diameter Bolt per Table 7-2*
$ØR_n \geq V_u$	**GOOD**		***3/4" Diameter Bolts OK***

Plate 3/8" x 8" x 16" with (2) 3/4" Diameter Bolts to Angle Brace OK

Weld Check:

t_{weld} =	3	in	*Weld Thickness in Sixteenths of an Inch*
L_{weld} =	4	in	*Weld Length Along One Face of Brace Connection Plate*
$ØR_n$ =	16.70	kips	*AISC 360-16 Chapter 8*
Weld Utilization Ratio:	**0.21**		**Ratio < 1 for Combined Shear and Tension Loads**
	GOOD		***3/16" Fillet Weld OK for Brace Connection Plate to Hanging Column***

3/16" Fillet Weld from Hanging Column to Brace Connection Plate OK

SUBJECT:	Klamath River Renewal Corporation	**BY:** KNH/GAC	**CHK'D BY:** 0
	Daggett Road Bridge at Iron Gate Reservoir	**DATE:** 10/15/2021	
	Daggett Road Bridge Abutment and Yreka Waterline Support Calculations	**PROJECT NO.:** 21-067	

Purpose

Design of abutments with backwall to support new, permanent Daggett Road Bridge with superstructure self-weight, 24" dia. water pipe, loading from HL-93 (Design Truck per AASHTO Section 3.6.1.2) and loading due to soil at active conditions. Drains exist in abutment so undrained conditions are assumed. Vehicle surcharging is included in design.

Information

Loading Factors and Combinations Applicable to Abutment Design: AASHTO Section 3.3.2

Description	Symbol	
Force Effects Due to Creep	CR	*Not Applicable for abutment design*
Downdrag Force	DD	*Not Applicable for abutment design*
Dead Load of Structural Components and Nonstructural Attachments	DC	*Provided by Manufacturer (Acrow)*
Dead Load of Wearing Surfaces and Utilities	DW	*Provided by Manufacturer (Acrow)*
Horizontal Earth Pressures	EH	*Calculated*
Misc. Locked-in Force Effects from Construction Process (e.g. jacking)	EL	*Not Applicable for abutment design*
Earth Surcharge Load	ES	*Calculated*
Vertical Pressure from Self-weight of Earth Fill	EV	*Calculated*
Secondary Forces from Post-Tensioning for Strength	PS	*Not Applicable for abutment design*
Force Effects Due to Shrinkage	SH	*Not Applicable for abutment design*
Blast Loading	BL	*Not Applicable for abutment design*
Vehicular Braking Force	BR	*Provided by Manufacturer (Acrow)*
Vehicular Centrifugal Force	CE	*Not Applicable for abutment design*
Vehicular Collision Force	CT	*Not Applicable for abutment design*
Vessel Collision Force	CV	*Not Applicable for abutment design*
Earthquake Load	EQ	*Calculated*
Friction Load	FR	*Not Applicable for abutment design*
Ice Load	IC	*Not Applicable for abutment design*
Vehicular Dynamic Load Allowance	IM	*Not Applicable for abutment design*
Vehicular Live Load	LL	*Provided by Manufacturer (Acrow)*
Live Load Surcharge	LS	*Calculated*
Pedestrian Live Load	PL	*Not Applicable for abutment design*
Force Effect Due to Settlement	SE	*Not Applicable for abutment design*
Force Effect Due to Temperature Gradient	TG	*Not Applicable for abutment design*
Force Effect Due to Uniform Temperature	TU	*Not Applicable for abutment design*
Water Load and Stream Pressure	WA	*Not Applicable for abutment design*
Wind on Live Load	WL	*Provided by Manufacturer (Acrow)*
Wind Load on Structure	WS	*Provided by Manufacturer (Acrow)*

Strength I:	Load combo with normal vehicle use w/o wind	LC1	*1.25DC + 1.35EH + 1.5ES + 1.35EV + 1.75LL+ 1.75BR + 1.75LS*
Strength II:	Load combo with special design vehicle use w/o wind	LC2	*1.25DC + 1.35EH + 1.5ES + 1.35EV + 1.35LL+ 1.35BR + 1.35LS*
Strength III:	Load combo with design wind speed at location	LC3	*1.25DC + 1.35EH + 1.5ES + 1.35EV + 1.0WS*
Strength IV:	Load combo emphasizing dead load force effects	LC4	*1.5DC + 1.35EH + 1.5ES + 1.35EV*
Strength V:	Load combo with normal vehicle use w/ 80mph wind	LC5	*1.25DC + 1.35EH + 1.5ES + 1.35EV + 1.35LL+ 1.35BR + 1.35LS + 1.0WS*
Extreme I	Load combo including earthquake forces	LC6	*1.0DC + 1.0EH + 1.0ES + 1.0EV + 0.5LL+ 0.5BR + 0.5LS + 1.0EQ*
Extreme II	Load combo with ice, collisions, and hydraulic effects	LC7	*Reviewed to verify Snow Load doesn't govern*
Service I	Load combo with normal vehicle use w/ 70mph wind	LC8	*1.0DC + 1.0DD + 1.0DW + 1.0EH + 1.0ES + 1.0EV + 1.0LL+ 1.0BR + 1.0LS*
Service II	Load combo intended to control yielding of steel	LC9	*1.0DC + 1.0EH + 1.0ES + 1.0EV + 1.3LL+ 1.3BR + 1.3LS*
Service III	Load combo for long. analysis (prestressed concrete)	LC10	*Not Applicable*
Service IV	Load combo for tension (prestressed concrete col.)	LC11	*Not Applicable*

All limit states (Strength, Extreme and Service) are equal to 1.0 for ductility, redundancy and operational classification based on conventional designs and details and conventional levels of redundancies for a typical bridge per AASHTO Section 1.3.2

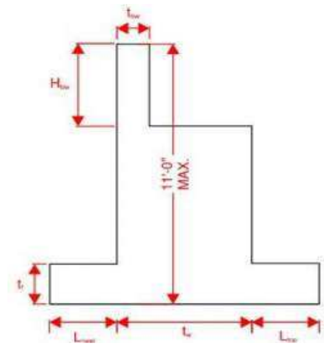
Abutment Analysis Calculations: General Properties and Vehicle Surcharge Loads

Soil Properties: Assumed 2(V):1(H) Pressure Distribution

γ_w =	62.4 pcf	*Unit weight of water*	
$Ø_f$ =	35 degrees	*Angle of Internal Friction (Geotech Report)*	
γ_s =	125 pcf	*Unit weight of soil*	
K_o =	0.426	*Lateral At-Rest Pressure Coefficient*	$K_o = 1 - sinØ_f$ *per AASHTO Sect.* 3.11.5.2

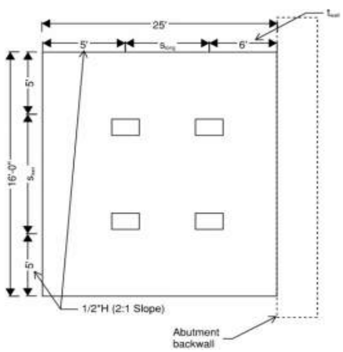
Properties of Abutment:

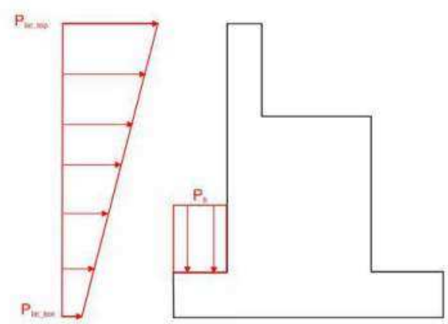
H =	12.00 ft	*Height of Abutment Wall*
L_{eff} =	35.0 ft	*Effective Abutment Length*
L_{abut} =	37.0 ft	*Total Abutment Length*
t_w =	6.00 ft	*Thickness of Abutment Wall*
t_{bw} =	2.25 ft	*Thickness of Back Wall*
H_{bw} =	3.50 ft	*Height of Back Wall*
t_f =	2.00 ft	*Thickness of Footing*
L_{heel} =	3.00 ft	*Length of Heel*
L_{toe} =	3.00 ft	*Length of Toe*
γ_c =	150.0 pcf	*Unit weight of Concrete*
A_{abut} =	70.9 ft^2/ ft	*Cross-Sectional Area of Abutment*



Vertical Surcharge Loading Applied from Design Truck:

W_{axle} =	32 kips	*Worst Case Axle Load (AASHTO Section 3.6.1.2.2 for Design Truck)*	
s_{long} =	14 ft	*Spacing of Wheels Along Length of Vehicle (CL-CL)*	
s_{tran} =	6 ft	*Spacing of Wheels Along Width of Vehicle (Out-Out)*	
w_{tire} =	20 in	*Width of Tire (AASHTO Section 3.6.1.2.5)*	
L_{tire_c} =	10.00 in	*Minimum Length of Wheel Contact (AASHTO Section 3.6.1.2.5)*	
L_{surch} =	6.0 ft	*Length of Surcharge Based on 2:1 Slope*	
P_{max} =	16 kips	*Maximum Wheel Load (1/2 Axle Load)*	$P_{max} = \frac{W_{axle}}{2}$
W_{sur_t}	18.00 ft	*Width of Surcharge (Transverse)*	
W_{sur_l}	26.00 ft	*Width of Surcharge (Longitudinal)*	
P_t =	11520 psf	*Vehicle Surcharge at Height 0' from Top*	
P_b =	68 psf	*Vehicle Surcharge at bottom of Abutment*	$p_b = \frac{W_{axle}}{(2 * L_{surch} + s_{tran}) * (L_{surch} + s_{long} + t_{wall})}$





Horizontal Loading Applied Due to Seismic Forces:

F_{pga} =	0.1863		
PGA =	0.2137		
A_s =	0.2535		
k_h =	0.040	*Seismic Horizontal Acceleration Coefficient*	
K_{ae} =	0.280	*Seismic Active Pressure Coefficient (AASHTO Figure A.11.3.2-3)*	
P_{AE} =	2520.0 plf	*Dynamic Lateral Earth Pressure Force*	$P_{AE} = 0.5 * \gamma_s * H^2 * K_{AE}$
P_{IR} =	572.55 plf	*Horizontal Inertial Force Due to Seismic Loading of Wall Mass*	$P_{IR} = k_h(W_{wall} + W_{soil})$
P_{eq} =	3092.55 plf	*Total Horizontal Seismic Load*	$P_{eq} = P_{AE} + P_{IR}$
h_{eq} =	4.00	*Height of Lateral Force*	

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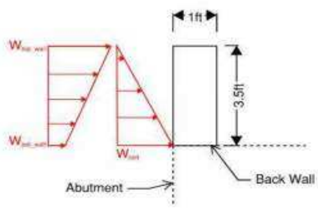
Backwall Analysis Calculations:

Properties of Back Wall :

t_{bw} =	2.25	ft	*Thickness of Back Wall*
H_{bw} =	3.50	ft	*Height of Back Wall*
d =	24.00	in	*Distance from Extreme Compression Fiber to CL of Tension Reinforcement (3" Cover)*
f'c =	4.50	ksi	*Compressive Strength of Concrete*
b_w =	80.00	in	*Width of Wall based on Equivalent Strip (see w_{eq_soil} equation below)*
β =	0.83		*AASHTO Section 5.6.2.2*
λ =	1.00		*Normalweight Concrete per ACI 318-14 Table 19.2.4.2*
A_s =	0.31	in^2	*Area of #5 bar*
f_y =	60	ksi	
s =	12	in	*Spacing of Vertical Bars*
N =	7		*Number of Bars within Equivalent Strip*

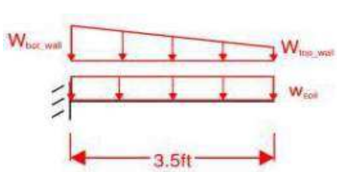
Vehicle and Soil Loading :



A_{top} =	1.39	ft^2	*Area of Loading at Top of Back Wall (Based on Wheel Dimensions)*	$A_{top} = w_{tire} * L_{tire_c}$
P_{top_wall} =	4912.4	psf	*Vehicle Surcharge Pressure at Top of Back Wall*	$P_{top\_wall} = \frac{K_o * P_{max}}{A_{top}}$
$L_{pressure}$ =	1.67	ft	*Vehicle Surcharge Pressure Distribution Length at Top of Back Wall*	
W_{top_wall} =	8.19	klf	*Vehicle Surcharge Pressure at Top of Back Wall*	
A_{bot} =	13.35	ft^2	*Area of Loading at Bottom of Back Wall*	$A_{bot} = (w_{tire} + H_{bw}) * (L_{tire_c} + H_{bw})$
P_{bot_wall} =	511.2	psf	*Vehicle Surcharge Pressure at Bottom of Back Wall*	$P_{bot\_wall} = \frac{K_o * P_{max}}{A_{top}}$
$L_{pressure}$ =	5.17	ft	*Vehicle Surcharge Pressure Distribution Length at Bottom of Back Wall (2:1 Slope)*	
W_{bot_wall} =	2.64	klf	*Vehicle Surcharge Load at bottom of Back Wall*	
w_{eq_soil} =	6.67	ft	*Equivalent Strip for Soil Loading (AASHTO Table 4.6.2.1.3-1)*	$w_{eq} = 45 + 10X$
$w_{soil} = K_o * \gamma_s * H_{bw} * w_{eq}$ =	1.24	klf	*Soil Loading at Bottom of Wall*	$where\ X = wall\ height\ in\ ft$

Shear and Moments:

DL Factor =	1.25	*Multiplication Factor for Dead Loads (AASHTO Table 3.4.1-1 for Strength I)*
LL Factor =	1.75	*Multiplication Factor for Live Loads (AASHTO Table 3.4.1-1 for Strength I)*



M_{soil} =	9.52	k-ft	*Applied Moment Due to Soil*	$M = \frac{w * H_{bw}^2}{2}$ *for distributed load*
M_{v_dist} =	28.31	k-ft	*Applied Moment Due to Vehicle Loading at Bottom of Wall*	
M_{v_pt} =	39.63	k-ft	*Applied Moment Due to Vehicle Loading at Top of Wall*	$M = P * H_{bw}$ *for triangular load*
M_{self} =	2.58	k-ft	*Applied Moment Due to Self-Weight of Wall*	
M_u =	**80.05**	**k-ft**	***Total Applied LRFD Factored Moment***	
V_{soil} =	7.62	k	*Applied Shear Due to Soil*	$V = w * H_{bw}$ *for distributed load*
V_{v_dist} =	16.18	k	*Applied Shear Due to Vehicle Loading at Bottom of Wall*	
V_{v_pt} =	16.99	k	*Applied Shear Due to Vehicle Loading at Top of Wall*	$V = P$ *for triangular load*
V_{self} =	1.48	k	*Applied Shear Due to Self-Weight of Wall*	
V_u =	**42.26**	**k**	***Total Applied LRFD Factored Shear***	
$Ø_m$ =	0.9		*AASHTO Table 11.5.7-1*	
$Ø_v$ =	0.9		*AASHTO Table 11.5.7-1*	

Shear and Moment Checks: Using #5 Vert. Bars at 12" oc. Each Face and (3) #5 horiz. Bars

μ =	0.60		*Coefficient of Friction (ACI 318-14 Table 22.9.4.2)*	
a =	0.43	in	*Depth of Equivalent Rectangular Stress Block*	$a = \frac{A_s * f_y}{0.85 * f_c' * b_w}$
$ØM_n$ =	232.28	k-ft	*Allowable Moment Capacity of Back Wall*	$\phi M_n = \phi * A_s * f_y * \left(d - \frac{a}{2}\right)$
IF $ØM_n \geq M_u$	GOOD			
$ØV_c$ =	231.8	k	*Allowable Shear Capacity of Back Wall*	$ØV_c = \phi * 2 * \lambda * \sqrt{f_c'} * b * d$
IF $ØV_c \geq V_u$	GOOD			
$ØV_n$ =	70.308	k	*Allowable Shear Friction (ACI 318-14 EQ. 22.5.5.1)*	$ØV_n = \phi * \mu * A_{vf} * f_y$
IF $ØV_n \geq V_u$	GOOD		*Reinforcing Capable of Resisting Shear at Abutment Interface*	
A_{s_min} =	0.178	in^2/ft	*Minimum Area of Steel Required (AASHTO Eq. 5.10.6-1)*	$A_{s\_min} = \frac{1.3 * b * h}{2 * (b + h) * F_y}$
$A_{s\_min} < A_s$ =	GOOD			

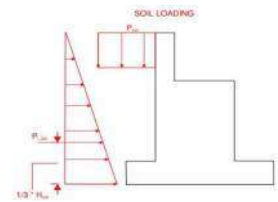
Abutment Calculations: Loads Applied to Abutment Summing Moments About Footing CL

Lateral Surcharge Loading Applied from Design Truck:

$$p_{lat\_top} = K_o * \frac{2 * p_t}{L_{eff}}$$

$$p_{lat\_bot} = K_o * \frac{2 * P_{max}}{W_{sur\_t} * W_{sur\_l}} * \frac{w_{sur_t}}{L_{eff}}$$

P_{lat_top} =	281	plf/f	*Lateral Pressure from Vehicle at Height 0' from Top*
P_{lat_bot} =	15	plf/f	*Lateral Pressure from Vehicle at bottom*
F_1 =	1.59	kips/ft	*Lateral Force from Vehicle (Trianglular Loading)*
y_1 =	8.00	ft	*Distance from Bottom of Abutment to Lateral Force*
F_2=	0.18	kips/ft	*Lateral Force from Vehicle (Uniform Loading)*
y_2 =	6.00	ft	*Distance from Bottom of Abutment to Lateral Force*



Lateral Surcharge Loading Applied from Soil:

$$p_{h\_soil} = k_o * \gamma_s * H$$

$$p_{l\_soil} = 0.5 * p_{h\_soil} * H$$

H_{soil} =	10.00	ft	*Height of Soil Above Heel*
P_{h_soil} =	533	psf	*Lateral Soil Pressure at Base of Abutment*
P_{l_soil} =	2665	plf	*Lateral Force Due to Soil*
y_{l_soil} =	3.333	ft	*Height of Resultant Lateral Force*

Lateral Loading Applied from Bridge/Vehicles:

F_{LL} =	20	kips	*Brake Load at Each Abutment: 25% of Axle Loading*
P_w =	86	kips	*Wind Load on Each Abutment: Reaction Provided by Bridge Manuf.*

Lateral Loading Due to Seismic:

P_{eq} =	3.09	kip/ft	*Total Horizontal Seismic Load (Abutment Self-Weight and Soil Above Heel)*
h_{eq} =	4.00		*Height of Lateral Force*
P_{eq_l} =	3.53	kip/ft	*Total Horizontal Seismic Load Over Abutment Length from Bridge Superstructure*
h_{eq_l} =	9.750	ft	*Centroid of concrete section from CL*

Vertical Surcharge Loading Applied from Soil Over Heel:

$$p_{soil} = H_{soil} * \gamma_s$$

H_{soil} =	10.00	ft	*Height of Soil Above Heel*
P_{v_soil} =	1250	psf	*Vertical Soil Pressure (Surcharge)*
x_{soil} =	-4.50	ft	*Centroid of Soil from CL*

Vertical Surcharge Loading Applied from Soil Over Toe:

$$p_{soil} = H_{soil} * \gamma_s$$

H_{soil} =	3.00	ft	*Height of Soil Above Heel*
P_{v_soil} =	375	psf	*Height of Soil Above Toe (Assumed for Slope Stability)*
x_{soil} =	4.50	ft	*Centroid of Soil from CL*

Vertical Loading Applied from Snow Load on Bridge:

P_{snow} =	40.00	psf	*Design Snow Load Acting on Bridge*
w_{snow} =	5.20	kip/ft	*Vertical Snow Load from Bridge Along Abutment Length*
x_{bridge} =	0.125	ft	*Centroid of concrete section from CL*

Vertical Loading Applied from Bridge:

P_{DL_Pipe} =	0.00	kips	*Point Load from 24" Waterline at Each Abutment (Included in Bridge Reactions Provided by Manuf.)*
P_{DL} =	244	kips	*Point Load at Each Corner (2 per Abutment): Reactions Provided by Bridge Manuf.*
P_{LL} =	165	kips	*Point Load at Each Corner (2 per Abutment): Reactions Provided by Bridge Manuf.*
d =	10.5	in	*Distance from Face of Backwall to Load Location*
x_{bridge} =	0.125	ft	*Centroid of concrete section from CL*

Vertical Seismic Loading Applied from Bridge Superstructure:

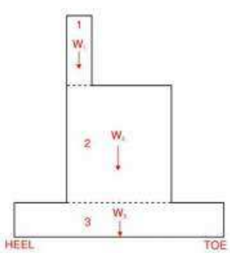
$$p_{eq\_v} = \frac{0.25 * 2 * P_{DL}}{L_{abut}}$$

P_{eq_v} =	3.53	kip/ft	*Total Vertical Seismic Load Over Abutment Length from Bridge Superstructure*
x_{bridge} =	0.125	ft	*Centroid of concrete section from CL*

Concrete Wall Sections:

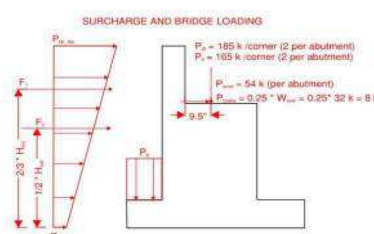
Section 1

t_1 =	2.25	ft	*Thickness of Section 1*
H_1 =	3.50	ft	*Height of Section 1*
$W_1 = \gamma_c * t_1 * H_1$ =	1.18	kip/ft	*Weight of Section 1*
x_1 =	-1.88	ft	*Centroid of concrete section from CL*



Section 2

t_2 =	6.00	ft	*Thickness of Section 2*
H_2 =	6.50	ft	*Height of Section 2*
$W_2 = \gamma_c * t_2 * H_2$ =	5.85	kip/ft	*Weight of Section 2*
x_2 =	0.00	ft	*Centroid of concrete section from CL*



Section 3

t_3 =	12.00	ft	*Thickness of Section 3*
H_3 =	2.00	ft	*Height of Section 3*
$W_3 = \gamma_c * t_3 * H_3$ =	1.80	kip/ft	*Weight of Section 3*
x_3 =	0.00	ft	*Centroid of concrete section from CL*

Abutment Calculations: Load Combinations and Factored Loads

Summation of Forces Applied to Abutment:

Vertical Loads:

DL	$F_{v\_abutment}$ =	8.83	kips/ft	*Positive Vertical Loads (Down) From Abutment Self-Weight Over Abutment Length*
DL	F_{vD_Bridge} =	13.94	kips/ft	*Positive Vertical Dead Loads (Down) From Bridge and Vehicle Loading Over Abutment Length*
LL	F_{vL_Bridge} =	9.43	kips/ft	*Positive Vertical Live Loads (Down) From Bridge and Vehicle Loading Over Abutment Length*
ES	$F_{v\_soil\_h}$ =	3.75	kips/ft	*Positive Vertical Loads (Down) From Soil Loading Over Heel Over Abutment Length*
ES	$F_{v\_soil\_t}$ =	1.13	kips/ft	*Positive Vertical Loads (Down) From Soil Loading Over Toe Over Abutment Length*
IC (Snow)	F_{v_snow} =	5.20	kips/ft	*Positive Vertical Loads (Down) From Snow Loading Over Abutment Length*
EQ	F_{v_eq} =	3.53	kips/ft	*Positive Vertical Loads (Down) From Superstructure Seismic Loads Over Abutment Length*
	F_v =	**45.81**	**kips/ft**	***Total Unfactored Vertical Load Over Abutment Length***

Vehicle Surcharge Loads:

LS	P_{top_v} =	0.068	ksf	*Vertical Vehicle Pressure (Surcharge) at Top of Wall*
LS	P_{base_v} =	0.035	ksf	*Vertical Vehicle Pressure (Surcharge) at Top of Footing/Base of Wall*
	P_b =	**0.104**	**ksf**	***Total Unfactored Vertical Pressure (Surcharge) Over Heel***

Lateral Loads:

LL	F_{h_Bridge} =	0.571	kips/ft	*Lateral Loads From Bridge and Vehicles Over Abutment Length (Braking Force)*
DL	$F_{h\_bw\_soil}$=	1.244	kips/ft	*Lateral Loads From Soil at Base of Back Wall*
EH	F_{h_soil}=	2.665	kips/ft	*Lateral Loads From Soil at Base of Footing (Includes Lateral Load from Soil at Base of Back Wall)*
LS	F_{h_v} =	1.774	kips/ft	*Lateral Loads From Vehicle at Base of Footing*
WL	F_{h_w} =	2.457	kips/ft	*Lateral Wind Load Over Abutment Length*
EQ	$F_{h\_eq\_self}$ =	3.093	kips/ft	*Lateral Seismic Load Over Abutment Length Due to Soil and Abutment Self-Weight*
EQ	$F_{h\_eq\_bridge}$ =	3.535	kips/ft	*Lateral Seismic Load Over Abutment Length Due to Bridge Superstructure*
	F_h =	**12.882**	**kips/ft**	***Unfactored Lateral Load Over Abutment Length***

								Factored Loads (kips/ft)	
Load Combination	DL Factor	γ_p Factor (ES)	γ_p Factor (EH)	LL Factor	WL Factor	EQ Factor	IC Factor	**F_v**	**F_h**
Strength I	1.25	1.50	1.35	1.75	-	-	-	**52.46**	**9.26**
Strength II	1.25	1.50	1.35	1.35	-	-	-	**48.65**	**8.32**
Strength III	1.25	1.50	1.35	-	1.0	-	-	**35.78**	**7.61**
Strength IV	1.50	1.50	1.35	-	-	-	-	**41.47**	**5.46**
Strength V	1.25	1.50	1.35	1.35	1.0	-	-	**48.65**	**10.78**
Extreme Event I	1.0	1.0	1.0	0.5	-	1.0	-	**35.95**	**11.71**
Extreme Event II	1.0	1.0	1.0	0.5	-	-	1.0	**37.62**	**5.08**
Service I	1.0	1.0	1.0	1.0	1.0	-	-	**37.18**	**8.71**
Service II	1.0	1.0	1.0	1.3	-	-	-	**40.04**	**5.71**

Sum of Moments (About Centerline of Footing):

Vertical Load Moments	M_{conc} =	-2.21	k-ft/ft	*Moment About CL Due to Self-Weight of Abutment*
	M_{bridge_D} =	1.74	k-ft/ft	*Moment About CL Due to Vertical Dead Load from Bridge*
	M_{bridge_L} =	1.18	k-ft/ft	*Moment About CL Due to Vertical Live Load from Bridge*
	M_{L_surch} =	3.26	k-ft/ft	*Moment About CL Due to Vertical Live Load Surcharge*
	M_{soil_h} =	-16.88	k-ft/ft	*Moment About CL Due to Vertical Loading from Soil*
	M_{soil_t} =	5.06	k-ft/ft	*Moment About CL Due to Vertical Loading from Soil*
	M_{snow} =	0.65	k-ft/ft	*Moment About CL Due to Vertical Loading from Soil*
	M_{eq} =	0.44	k-ft/ft	*Moment About CL Due to Vertical Seismic Loading from Bridge Superstructure*
Lateral Load Moments	M_{veh} =	13.83	k-ft/ft	*Moment About CL Due to Vehicle Surcharge Loading*
	M_{soil} =	8.88	k-ft/ft	*Moment About CL Due to Soil Loading*
	M_{bridge} =	4.86	k-ft/ft	*Moment About CL Due to Bridge (Braking Force)*
	M_{wind} =	20.89	k-ft/ft	*Moment About CL Due to Wind*
	M_{eq_self} =	12.37	k-ft/ft	*Moment About CL Due to Horizontal Seismic Loading from Self-Weight and Soil*
	M_{eq_bridge} =	30.04	k-ft/ft	*Moment About CL Due to Horizontal Seismic Loading from Bridge Superstructure*
	$M_{pos\_service\_I}$ =	59.71	k-ft/ft	*Service I and II Positive Moment About CL (CW)*
	$M_{pos\_service\_II}$ =	45.76	k-ft/ft	*Service II Positive Moment About CL (CW)*
	$M_{pos\_strength\_I}$ =	62.24	k-ft/ft	*Strength I Positive Moment About CL (CW)*
	$M_{pos\_strength\_II}$ =	52.99	k-ft/ft	*Strength II Positive Moment About CL (CW)*
	$M_{pos\_strength\_III}$ =	42.65	k-ft/ft	*Strength III Positive Moment About CL (CW)*
	$M_{pos\_strength\_IV}$ =	22.20	k-ft/ft	*Strength IV Positive Moment About CL (CW)*
	$M_{pos\_strength\_V}$ =	73.88	k-ft/ft	*Strength V Positive Moment About CL (CW)*
	$M_{pos\_Extreme\_I}$ =	70.11	k-ft/ft	*Extreme I Positive Moment About CL (CW)*
	$M_{pos\_Extreme\_II}$ =	27.90	k-ft/ft	*Extreme II Positive Moment About CL (CW)*
	$M_{neg\_service\_I}$ =	-19.09	k-ft/ft	*Service I Negative Moment About CL (CCW)*
	$M_{neg\_service\_II}$ =	-19.09	k-ft/ft	*Service II Negative Moment About CL (CCW)*
	$M_{neg\_strength\_I}$ =	-28.08	k-ft/ft	*Strength I Negative Moment About CL (CCW)*
	$M_{neg\_strength\_II}$ =	-28.08	k-ft/ft	*Strength II Negative Moment About CL (CCW)*
	$M_{neg\_strength\_III}$ =	-28.08	k-ft/ft	*Strength III Negative Moment About CL (CCW)*
	$M_{neg\_strength\_IV}$ =	-28.63	k-ft/ft	*Strength IV Negative Moment About CL (CCW)*
	$M_{neg\_strength\_V}$ =	-28.08	k-ft/ft	*Strength V Negative Moment About CL (CCW)*
	$M_{neg\_Extreme\_I}$ =	-19.09	k-ft/ft	*Extreme I Negative Moment About CL (CW)*
	$M_{neg\_Extreme\_II}$ =	-19.09	k-ft/ft	*Extreme II Negative Moment About CL (CW)*
	$\mathbf{M_{net\_service\_I}}$ =	**40.62**	**k-ft/ft**	***Service I Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_service\_II}}$ =	**26.67**	**k-ft/ft**	***Service II Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_strength\_I}}$ =	**34.16**	**k-ft/ft**	***Strength I Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_strength\_II}}$ =	**24.91**	**k-ft/ft**	***Strength II Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_strength\_III}}$ =	**14.57**	**k-ft/ft**	***Strength III Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_strength\_IV}}$ =	**-6.43**	**k-ft/ft**	***Strength IV Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_strength\_V}}$ =	**45.80**	**k-ft/ft**	***Strength V Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_Extreme\_I}}$ =	**51.02**	**k-ft/ft**	***Extreme I Net Moment About Centerline of Footing***
	$\mathbf{M_{net\_Extreme\_I}}$ =	**8.81**	**k-ft/ft**	***Extreme II Net Moment About Centerline of Footing***

Overturning Calculations:

Service Limit State:

Resultant Location & Crack Length (No Cracking Assumed, Moment about Footing CL)

B =	12.00	ft	*Width of Wall (Toe to Heel Dimension)*
M_{total} =	40.62	k-ft/ft	*Total Moment About CL*
F_v =	40.04	kips/ft	*Vertical Load*
F_h =	5.714	kips/ft	*Lateral Load*
e = M_{total}/F_v =	1.01	ft	*Distance from CL to Resultant Force Intersection*

If e <= B/6 **Full Compression**

Extreme Limit State:

Resultant Location & Crack Length (No Cracking Assumed, Moment about Footing CL)

B =	12.00	ft	*Width of Wall (Toe to Heel Dimension)*
M_{total} =	51.02	k-ft/ft	*Total Moment About CL*
F_v =	37.62	kips/ft	*Vertical Load*
F_h =	5.082	kips/ft	*Lateral Load*
e = M_{total}/F_v =	1.36	ft	*Distance from CL to Resultant Force Intersection*

If e <= B/6 **Full Compression**

Strength Limit State:

Resultant Location & Crack Length (No Cracking Assumed, Moment about 0,0 (toe))

B =	12.00	ft	*Width of Wall (Toe to Heel Dimension)*
M_{total} =	45.80	k-ft/ft	*Total Moment About Toe*
F_v =	52.46	kips/ft	*Vertical Load*
F_h =	9.258	kips/ft	*Lateral Load*
e = M_{total}/F_v =	0.87	ft	*Distance from CL to Resultant Force Intersection*

If e <= B/6 **Full Compression**

****Overall stability and slope stability are included in the geotechnical calculations***

Bearing Pressure Calculations:

Service Limit State:

$$q_{applied} = \frac{\Sigma F_v}{A} \pm \frac{M_{net} * y}{I}$$

$$M_{net} = \Sigma F_v * e$$

$$I = \frac{B^3}{12}$$

$$q_{max} = \frac{\Sigma F_v}{B - 2e}$$

$F_{v\_positive}$ =	40.04	kips/ft	*Maximum Vertical Load*
q_{allow} =	19.80	ksf	*Nominal Bearing Capacity (per Geotechnical Calculations)*
$Ø_b$ =	0.45		*Resistance Factor for Bearing*
q_{max} =	4.02	ksf	*Maximum Bearing Pressure*
$Ø_b$*q_{allow} =	8.91	ksf	*Allowable Bearing Pressure*
	GOOD		***Foundation OK for Bearing at Service Limit State***

Extreme Limit State:

$$q_{applied} = \frac{\Sigma F_v}{A} \pm \frac{M_{net} * y}{I}$$

$$M_{net} = \Sigma F_v * e$$

$$I = \frac{B^3}{12}$$

$$q_{max} = \frac{\Sigma F_v}{B - 2e}$$

$F_{v\_positive}$ =	37.62	kips/ft	*Maximum Vertical Load*
q_{allow} =	19.80	ksf	*Nominal Bearing Capacity (per Geotechnical Calculations)*
$Ø_b$ =	0.45		*Resistance Factor for Bearing*
q_{max} =	4.05	ksf	*Maximum Bearing Pressure*
$Ø_b$*q_{allow} =	8.91	ksf	*Allowable Bearing Pressure*
	GOOD		***Foundation OK for Bearing at Extreme Limit State***

Strength Limit State:

$$q_{applied} = \frac{\Sigma F_v}{A} \pm \frac{M_{net} * y}{I}$$

$$M_{net} = \Sigma F_v * e$$

$$I = \frac{B^3}{12}$$

$$q_{max} = \frac{\Sigma F_v}{B - 2e}$$

$F_{v\_positive}$ =	52.46	kips/ft	*Maximum Vertical Load*
q_{allow} =	19.80	ksf	*Nominal Bearing Capacity (per Geotechnical Calculations)*
$Ø_b$ =	0.45		*Resistance Factor for Bearing*
q_{max} =	5.12	ksf	*Maximum Bearing Pressure*
$Ø_b$*q_{allow} =	8.91	ksf	*Allowable Bearing Pressure*
	GOOD		***Foundation OK for Bearing at Strength Limit State***

Sliding Calculations: Cohesionless Soil per Geotechnical Data

Service Limit State:

F_h =	5.714	kips/ft	*Lateral Load*	
$Ø_f$ =	35.0	deg	*Friction angle*	
μ =	0.47		*Coefficient of friction*	$\mu = tan Ø_f$
$Ø_t$ =	0.8		*Resistance Factor for Sliding*	
C =	1.00		*Concrete Cast Against Soil*	
F_v = V =	40.04	kips/ft	*Force Normal to Base (Vertical Load)*	
R_t =	18.97	kips/ft	*Nominal Sliding Resistance Between Soil and Foundation*	$R_t = C * V * tan Ø_f$
$Ø_tR_t$ =	14.23	kips/ft	*Factor of Safety using coefficient of friction from Geotech*	
If $Ø_tR_t \geq F_h$	**GOOD**		***Foundation OK Against Sliding at Service Limit State***	

Extreme Limit State:

F_h =	5.082	kips/ft	*Lateral Load*	
$Ø_f$ =	35.0	deg	*Friction angle*	
μ =	0.47		*Coefficient of friction*	$\mu = tan Ø_f$
$Ø_t$ =	0.8		*Resistance Factor for Sliding*	
C =	1.00		*Concrete Cast Against Soil*	
F_v = V =	37.62	kips/ft	*Force Normal to Base (Vertical Load)*	
R_t =	17.82	kips/ft	*Nominal Sliding Resistance Between Soil and Foundation*	$R_t = C * V * tan Ø_f$
$Ø_tR_t$ =	13.37	kips/ft	*Factor of Safety using coefficient of friction from Geotech*	
If $Ø_tR_t \geq F_h$	**GOOD**		***Foundation OK Against Sliding at Extreme Limit State***	

Strength Limit State:

F_h =	9.258	kips/ft	*Lateral Load*	
$Ø_f$ =	35.0	deg	*Friction angle*	
μ =	0.47		*Coefficient of friction*	$\mu = tan Ø_f$
$Ø_t$ =	0.8		*Resistance Factor for Sliding*	
C =	1.00		*Concrete Cast Against Soil*	
F_v = V =	52.46	kips/ft	*Force Normal to Base (Vertical Load)*	
R_t =	24.86	kips/ft	*Nominal Sliding Resistance Between Soil and Foundation*	$R_t = C * V * tan Ø_f$
$Ø_tR_t$ =	18.64	kips/ft	*Factor of Safety using coefficient of friction from Geotech*	
If $Ø_tR_t \geq F_h$	**GOOD**		***Foundation OK Against Sliding at Strength Limit State***	

Abutment Wall Check: Worst-Case Limit State Loads and Reactions Used

Properties of Abutment:

t_{bw} =	6.00	ft	*Thickness of Abutment*
H_{bw} =	8.50	ft	*Height of Abutment*
d =	69.00	in	*Distance from Extreme Compression Fiber to CL of Tension Reinforcement (3" Cover)*
f'c =	4.50	ksi	*Compressive Strength of Concrete*
β =	0.83		*ACI 318-14 Table 22.2.2.4.3*
λ =	1.00		*Normalweight Concrete per ACI 318-14 Table 19.2.4.2*
A_s =	0.44	in^2	*Area of #6 bar*
f_y =	60.00	ksi	
s =	12.00	in	*Spacing of Vertical bars*
$Ø_m$ =	0.9		*AASHTO Section 5.5.4.2 for Tension-Controlled Reinforced Sections*
$Ø_v$ =	0.9		*AASHTO Section 5.5.4.2 for Shear in Reinforced Concrete Sections*
$\rho_b = 0.85*\beta*f'c/f_y*(87/(87+f_y))$ =	0.053		*Balanced Percentage of Steel*
ρ_{max} =	0.033		*Maximum Percentage of Steel*
ρ_{min} =	0.003		*Minimum Percentage of Steel*
As_{min} =	0.406	in^2/ft	*Minimum Area of Steel Required (AASHTO Eq. 5.10.6-1)* $A_{s\_min} = \frac{1.3 * b * h}{2 * (b + h) * F_y}$
$A_{s\_min} < A_s$ =	GOOD		
a =	0.58	in	*Depth of Equivalent Rectangular Stress Block* $a = \frac{A_s * f_y}{0.85 * f_c' * b_w}$
M_u =	51.02	k-ft/ft	*LRFD Moment Applied to Abutment Wall (Taken at Wall CL)*
V_u =	11.71	k/ft	*LRFD Shear Applied to Abutment Wall*
$ØM_n$ =	136.05	k-ft/ft	*Allowable Moment Capacity of Abutment Wall* $\phi M_n = \phi * A_s * f_y * \left(d - \frac{a}{2}\right)$
IF $ØM_n \geq M_u$	GOOD		
$ØV_c$ =	100.0	k/ft	*Allowable Shear Capacity of Back Wall* $ØV_c = \phi * 2 * \lambda * \sqrt{f_c'} * b * d$
IF $ØV_c \geq V_u$	GOOD		
$ØV_n$ =	28.51	k/ft	*Allowable Shear Friction (ACI 318-14 EQ. 22.5.5.1)* $ØV_n = \phi * \mu * A_{vf} * f_y$
IF $ØV_n$ => V_u	GOOD		***Reinforcing Capable of Resisting Shear at Abutment Interface***

#6 vertical bars at 12" oc. Each Face OK

Abutment Footing Check:

b_f =	12.00	ft	*Footing Width*
t_f =	2.00	ft	*Thickness of Footing*
d =	21.00	in	*Distance from Extreme Compression Fiber to CL of Tension Reinforcement (3" Cover)*
f'c =	4.50	ksi	*Compressive Strength of Concrete*
β =	0.83		*ACI 318-14 Table 22.2.2.4.3*
λ =	1.00		*Normalweight Concrete per ACI 318-14 Table 19.2.4.2*
A_s =	0.44	in^2	*Area of #6 bar*
f_y =	60.00	ksi	
s =	12.00	in	*Spacing of Horizontal Bars*
$Ø_m$ =	0.9		*AASHTO Section 5.5.4.2 for Tension-Controlled Reinforced Sections*
$Ø_v$ =	0.9		*AASHTO Section 5.5.4.2 for Shear in Reinforced Concrete Sections*
μ =	0.60		*Coefficient of friction per ACI 318 Table 22.9.4.2*
$\rho_b = 0.85*\beta*f'c/f_y*(87/(87+f_y))$ =	0.053		*Balanced Percentage of Steel*
ρ_{max} =	0.033		*Maximum Percentage of Steel*
ρ_{min} =	0.003		*Minimum Percentage of Steel*
A_{s_min} =	0.223	in^2/ft	*Minimum Area of Steel Required (AASHTO Eq. 5.10.6-1)* $A_{s\_min} = \frac{1.3*b*h}{2*(b+h)*F_y}$
$A_{s\_min} < A_s$ =	GOOD		
a =	0.58	in	*Depth of Equivalent Rectangular Stress Block* $a = \frac{A_s * f_y}{0.85 * f'_c * b_w}$
M_u =	18.07	k-ft/ft	*LRFD Moment Applied at Footing*
V_u =	5.12	k/ft	*LRFD Shear Applied at Footing*
$ØM_n$ =	41.01	k-ft/ft	*Allowable Moment Capacity of Footing* $\phi M_n = \phi * A_s * f_y * \left(d - \frac{a}{2}\right)$
IF $ØM_n \geq M_u$	**GOOD**		
$ØV_c$ =	30.4	k/ft	*Allowable Shear Capacity of Footing* $ØV_c = \phi * 2 * \lambda * \sqrt{f'_c} * b * d$
IF $ØV_c \geq V_u$	**GOOD**		
$ØV_n$ =	14.26	k/ft	*Allowable Shear Friction (ACI 318-14 EQ. 22.5.5.1)* $ØV_n = \phi * \mu * A_{vf} * f_y$
IF $ØV_n$ => V_u	**GOOD**		***Reinforcing Capable of Resisting Shear at Abutment Interface***

#6 bars at 12" oc. Top and Bottom Longitudinal and #6 Bars at 12"oc. Transverse OK

McMILLEN JACOBS ASSOCIATES

SUBJECT:	Klamath River Renewal Corporation	**BY:** KNH/GAC	**CHK'D BY:** 0
	Daggett Road Bridge at Iron Gate Reservoir	**DATE:** 10/15/2021	
	Daggett Road Bridge Abutment and Yreka Waterline Support Calculations	**PROJECT NO.:** 21-067	

Purpose

Design of connections between bridge superstructure and abutments with superstructure loads provided by Acrow.

Information: Loads

Loads Provided by Acrow:

Load Factors *(Per AASHTO LRFD)*

MPF = 1.00 (2 Lanes)

	HL-93		EV-2		DC	DW	WS	WL	EQ
LIMIT STATE	LL	IM	LL	IM					
Strength I	1.75	33%	-	-	1.25	1.50	-	-	-
Strength II	1.35	33%	1.35	33%	1.25	1.50	-	-	-
Strength III	-	-	-	-	1.25	1.50	1.00	-	-
Strength V	1.35	33%	-	-	1.25	1.50	1.00	1.00	-
Service I	1.00	33%	-	-	1.00	1.00	1.00	1.00	-
Extreme Event I	-	-	-	-	1.00	1.00	-	-	1.00
Extreme Event II	0.50	33%	-	-	1.00	1.00	-	-	-

ACROW PANEL BRIDGE - REACTIONS

ALL REACTIONS ARE PER PER CORNER OF BRIDGE EXCEPT WIND

NOTE: - *All values are in kips (1,000 lbs)*
- *Multiple Presence Factor is included*
- *Eccentricity is included*
- *Dynamic Load Allowance is not included*

STRENGTH I REACTIONS:

LOADS	1 Lane HL-93
DC	246
DW Epoxy	70
HL-93 Truck	131
HL-93 Lane	157
TOTAL	**604**
Braking (longitudinal)	34

STRENGTH II REACTIONS:

LOADS	1 Lane P5
DC	246
DW Epoxy	70
P5 Truck	24
TOTAL	**340**

STRENGTH III REACTIONS:

LOADS	
DC	246
DW Epoxy	70
TOTAL	316
Transverse Wind	73

STRENGTH V REACTIONS:

LOADS	1 Lane HL-93
DC	246
DW Epoxy	70
HL-93 Truck	101
HL-93 Lane	121
TOTAL	**538**
Braking (longitudinal)	27
Transverse Wind	73
Wind on Live Load	13

EXTREME EVENT I REACTIONS:

LOADS	
DC	197
DW Epoxy	47
Total	**244**
Seismic	62

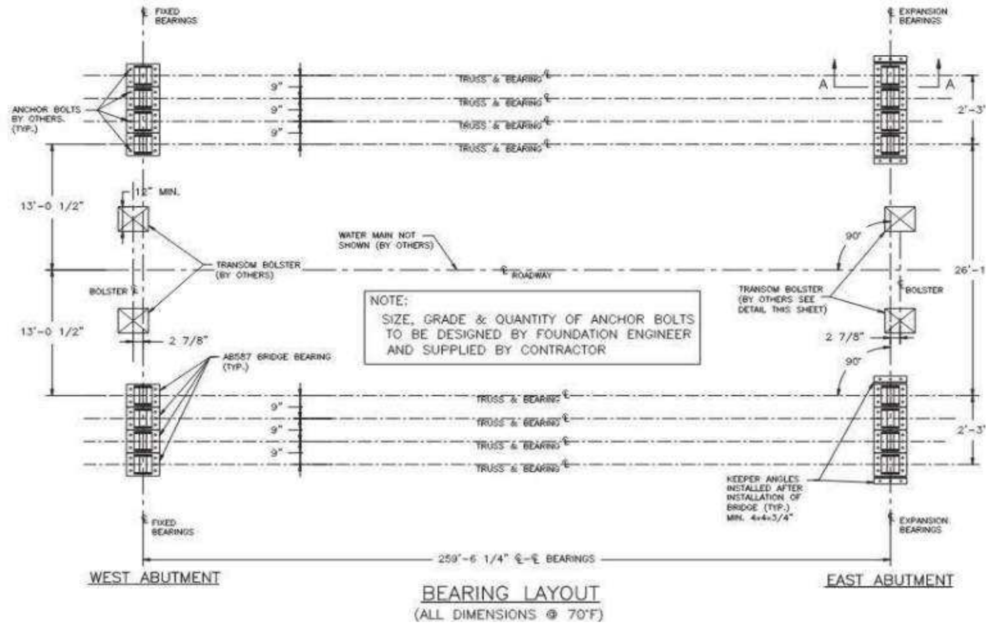
SERVICE I REACTIONS:

LOADS	1 Lane HL-93
DC	197
DW Epoxy	47
HL-93 Truck	75
HL-93 Lane	90
TOTAL	**409**
Braking (longitudinal)	20
Transverse Wind	73
Wind on Live Load	13

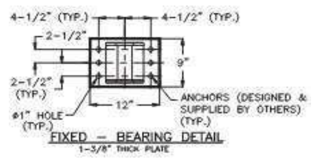
P_{dead} =	244	kips	*Dead Load Reaction from Bridge Superstructure*
A_s =	0.254		*Peak seismic ground accceleration coefficient modified by short-period site factor*
P_{eq_min} =	61.9	kips	*Minimum design connection force for single-span bridges per AASHTO Section 3.10.9.1*
R =	0.80		*Response Modification Factor for Connections (AASHTO Table 3.10.7.1-2)*
F_{eq} =	77.5	kips	*Lateral seismic load at each abutment*
F_{wind} =	86	kips	*Lateral wind load at each abutment*

Calculations: Bridge Superstructure Attachment to Abutment

Bearing Plate Information Provided by Acrow:



Fixed End Connection: Bearing Plate Attachment



N =	8		*Number of Bearing Plates on Abutment*
F_{eq} =	12.109	kips	*Lateral seismic load at each bearing connection in X direction*
F_{wind} =	10.75	kips	*Lateral wind load at each bearing connection in Y direction*

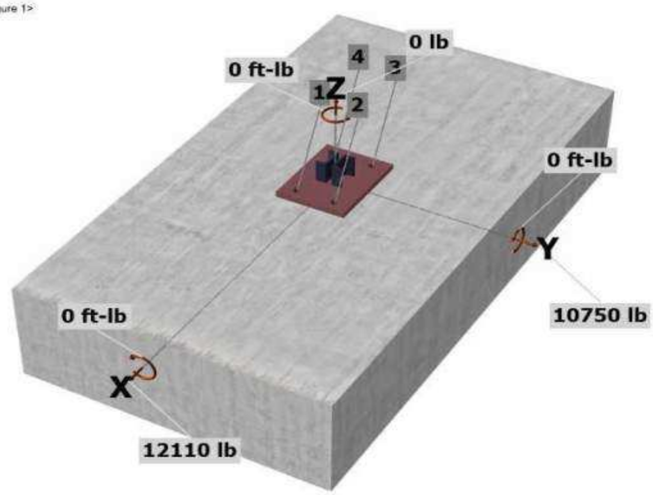
Wind load controls perpendicular to bridge span (Y direction) and seismic load controls parallel to bridge span (X direction). The LRFD factored bridge reactions are assumed to be equal at each be location (8 total) so the reaction at each abutment is divided by the number of connections. The base plate is assumed to act rigidly, equally distributing the shear loads between anchors. An arbitra was used for the following anchor calculations, however, the analysis is unaffected by the profile selection. Although the Acrow bearing plate can accommodate (6) anchors, a single plate with (4) a was analyzed based on edge distance and minimum spacing requirements. Spacing between anchors across abutment has been considered in the following anchor calcs

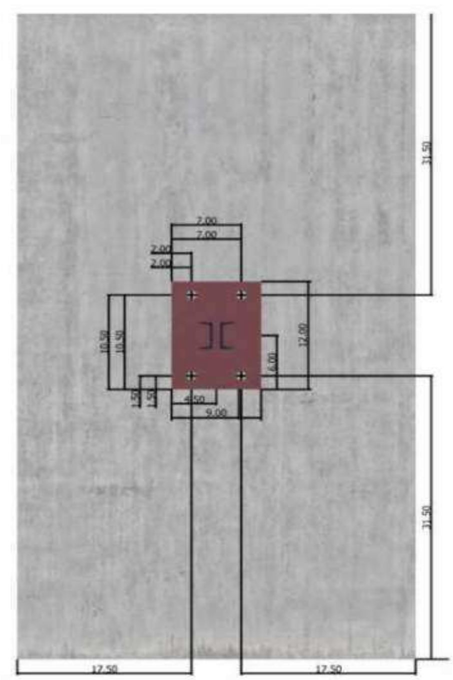
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: No
Anchors subjected to sustained tension: No
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 12110
V_{uay} [lb]: 10750
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>





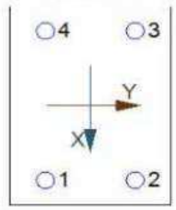
3. Resulting Anchor Forces

Anchor	Tension load	Shear load x	Shear load y	Shear load combined

	N_{ua} (lb)	V_{uax} (lb)	V_{uay} (lb)	$\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	3027.5	2687.5	4048.3
2	0.0	3027.5	2687.5	4048.3
3	0.0	3027.5	2687.5	4048.3
4	0.0	3027.5	2687.5	4048.3
Sum	0.0	12110.0	10750.0	16193.0

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
21090	1.0	0.65	0.68	9322

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	4500	21.00	58100

$\phi V_{cbgy} = \phi(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
1728.00	1984.50	1.000	1.000	1.000	1.146	58100	0.70	40571

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	4500	16.00	38639

$\phi V_{cbgx} = \phi(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgx} (lb)
960.00	1152.00	1.000	0.919	1.000	1.000	38639	0.70	20708

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	4500	16.00	38639

$\phi V_{cbgx} = \phi(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgx} (lb)
960.00	1152.00	1.000	1.000	1.000	1.000	38639	0.70	45079

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	4500	17.50	44198

$\phi V_{cbgy} = \phi(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
1476.00	1378.13	1.000	1.000	1.000	1.046	44198	0.70	69309

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi\min|k_{cp}N_{ag};\ k_{cp}N_{cbg}| = \phi\min|k_{cp}(A_{Na}/A_{Na0})\Psi_{ec,Na}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba};\ k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b|$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Na} (in²)	A_{Na0} (in²)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	850.31	494.11	1.000	1.000	1.000	20581	35418

A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
1800.00	1296.00	1.000	0.992	1.000	1.000	35334	48666	0.70

ϕV_{cpg} (lb)
49585

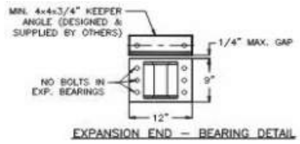
11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	4048	9322	0.43	Pass
T Concrete breakout y+	10750	40571	0.26	Pass
T Concrete breakout x+	12110	20708	0.58	Pass
ǁ Concrete breakout x-	5375	45079	0.12	Pass
ǁ Concrete breakout y+	6055	69309	0.09	Pass
Concrete breakout, combined	-	-	**0.64**	**Pass (Governs)**
Pryout	16193	49585	0.33	Pass

SET-XP w/ 1"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

Expansion End Connection: Keeper Angle Attachment



N =	4		*Number of Keeper Angles on Abutment*
F_{eq} =	24.219	kips	*Lateral seismic load at each keeper angle in X direction*
F_{wind} =	21.5	kips	*Lateral wind load at each keeper angle in Y direction*

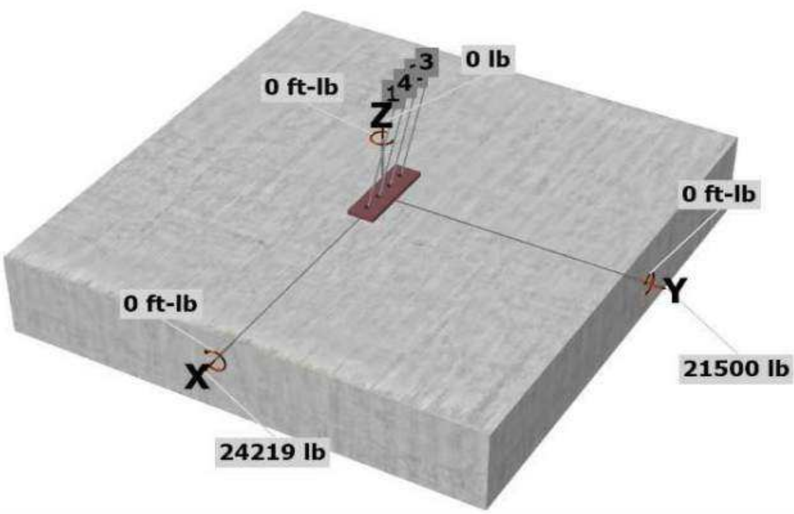
Wind load controls perpendicular to bridge span (Y direction) and seismic load controls parallel to bridge span (X direction). The LRFD factored bridge reactions are assumed to be equal at each ke location (4 total) so the reaction at each abutment is divided by the number of connections. The angle is assumed to act rigidly, equally distributing the shear loads between anchors. A single angle anchors was analyzed.

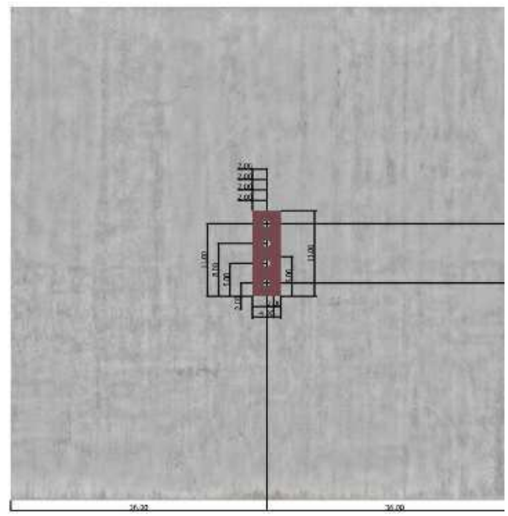
Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: No
Ductility section for tension: 17.2.3.4.2 not applicable
Ductility section for shear: 17.2.3.5.3 (a) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

N_{ua} [lb]: 0
V_{uax} [lb]: 24219
V_{uay} [lb]: 21500
M_{ux} [ft-lb]: 0
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



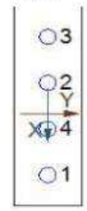


3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	6054.8	6450.0	8846.6
2	0.0	6054.8	5016.7	7863.0
3	0.0	6054.8	4300.0	7426.3
4	0.0	6054.8	5733.3	8338.5
Sum	0.0	24219.0	21500.0	32474.5

Maximum concrete compression strain (‰): 0.00
Maximum concrete compression stress (psi): 0
Resultant tension force (lb): 0
Resultant compression force (lb): 0
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.25
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.22

<Figure 3>



8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\alpha_{V,seis}$	$\phi_{grout}\alpha_{V,seis}\phi V_{sa}$ (lb)
21090	1.0	0.65	0.68	9322

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	4500	36.00	130407

$\phi V_{cbgy} = \phi(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
4050.00	5832.00	0.991	0.883	1.000	1.000	130407	0.70	55483

Shear perpendicular to edge in x-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	4500	42.00	164332

$\phi V_{cbx} = \phi(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1 & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbx} (lb)
4536.00	7938.00	0.871	1.000	1.000	164332	0.70	57282

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
8.00	1.000	1.00	4500	36.00	130407

$\phi V_{cbgy} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
4050.00	5832.00	1.000	1.000	1.000	1.000	130407	0.70	126785

Shear parallel to edge in x-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5};\ 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
8.00	1.000	1.00	4500	33.00	114451

$\phi V_{cbx} = \phi\,(2)(A_{Vc}/A_{Vco})\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1a)

A_{Vc} (in²)	A_{Vco} (in²)	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbx} (lb)
3564.00	4900.50	1.000	1.000	1.000	114451	0.70	116532

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi\min|k_{cp}N_{ag};\ k_{cp}N_{cbg}| = \phi\min|k_{cp}(A_{Na}/A_{Na0})\Psi_{ec,Na}\Psi_{ed,Na}\Psi_{cp,Na}N_{ba};\ k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b|$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Na} (in²)	A_{Na0} (in²)	$\Psi_{ed,Na}$	$\Psi_{ec,Na}$	$\Psi_{cp,Na}$	N_{ba} (lb)	N_a (lb)
2.0	694.17	494.11	1.000	0.959	1.000	20581	27732

A_{Nc} (in²)	A_{Nco} (in²)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	N_{cb} (lb)	ϕ
1620.00	1296.00	0.974	1.000	1.000	1.000	35334	43040	0.70

ϕV_{cpg} (lb)
38825

11. Results

11. Interaction of Tensile and Shear Forces (Sec. D.7)?

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	**8847**	**9322**	**0.95**	**Pass (Governs)**
T Concrete breakout y+	21500	55483	0.39	Pass
T Concrete breakout x+	24219	57282	0.42	Pass
|| Concrete breakout y+	24219	126785	0.19	Pass
|| Concrete breakout x+	6450	116532	0.06	Pass
Concrete breakout, combined	-	-	0.57	Pass
Pryout	32385	38825	0.83	Pass

SET-XP w/ 1"Ø F1554 Gr. 36 with hef = 12.000 inch meets the selected design criteria.

McMILLEN JACOBS ASSOCIATES	PROJECT KRRC		SHEET 1
	SUBJECT Daggett Bridge Pipe - Pipe Reactions		DATE
	BY GAC	CHECKED	PROJECT NO.

Reactions @ Pipe Support Brace

Dead Load

- Pipe (empty)

$$495 \text{ \#/ft}^3 (\pi)(2')(3/4/12)(10') = 1{,}942^{\#}$$

- Steel Saddle and Web

$$495 \text{ \#/ft}^3 (3/8/12)(.5)(3.4') = 26^{\#} \quad \text{(web)}$$

$$+\ 495 \text{ \#/ft}^3 (.5/12)(.5')(2.75') = 28^{\#} \quad \text{(saddle plate)}$$

$$+\ 495 \text{ \#/ft}^3 (4)(3/8/12)(.40)(.5) = 12^{\#} \quad \text{(stiffeners)}$$

- Bottom Plate & side Plates

$$495 \text{ \#/ft}^3 (1''/12)(6''/12)[(2)(6.16') + 3.5'] = 326^{\#}$$

- B-Deck (Verco)

$$3.0 \text{ psf}[(6.16')(2) + 3.5'](10') = 474^{\#}$$

- Water

$$62.4 \text{ \#/ft}^3 \left(\frac{\pi(2)^2}{4}\right)(10') = \underline{1960^{\#}}$$

$$F_{y_1} = F_{y_2} = 2{,}384^{\#} \qquad \Sigma_1 = 4{,}768^{\#}$$

combined tension & shear in bolts

$F_u = 120\text{ksi}$

A325 Bolts

$3/8"\ \phi$ Bolts

$$\left(\frac{R_{uT}}{\phi_t R_{nt}}\right)^2 + \left(\frac{R_{uV}}{\phi_V R_{nV}}\right)^2 \leq 1.0$$

$$R_{uT} = 1.0(3{,}354^{\#}) = 3{,}354^{\#}$$

$$\phi_t R_{nt} = .75(.75)(120\text{ksi})\left(\frac{\pi(3/8)^2}{4}\right) = 7{,}450^{\#}$$

threads

$$R_{uV} = 1.0(1{,}663^{\#}) = 1{,}663$$

$$\phi_V R_{nV} = .75(.4)(120\text{ksi})(2)\left(\frac{\pi(3/8)^2}{4}\right) = 7{,}950^{\#}$$

unity Check

$$\left(\frac{3{,}354}{7{,}450}\right)^2 + \left(\frac{1{,}663}{7{,}950}\right)^2 = \underline{.24} << 1.0$$

$\therefore$ $\underline{3/8" \text{ Bolts OK}}$

McMILLEN JACOBS ASSOCIATES	PROJECT KRRC		SHEET 6
	SUBJECT		DATE
	BY GAC	CHECKED	PROJECT NO.

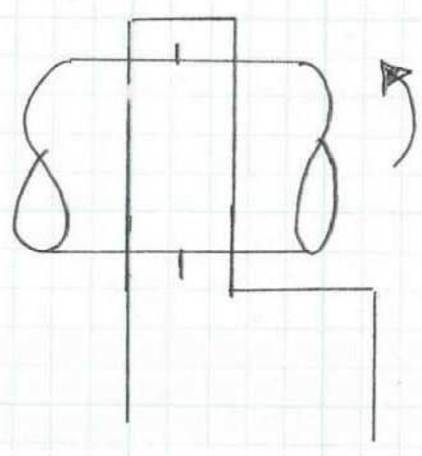
$$f_b = \frac{My}{I} = \frac{6{,}055{,}000 \text{ in\#} (12.50")}{4{,}201 \text{ in}^4}$$

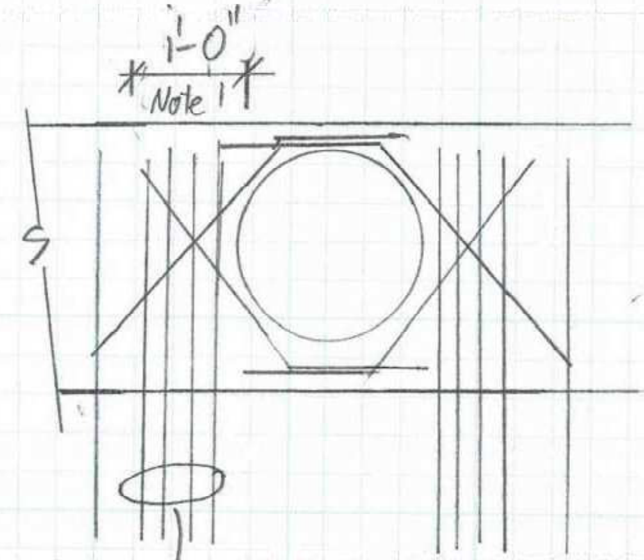
$$= \underline{18{,}016 \text{ psi}} < .6(f_y)$$

Note:
Moment associated w/ self wt of Pipe + H_2O ignored

∴ No problem w/ bending

Moment @ concrete stemwall

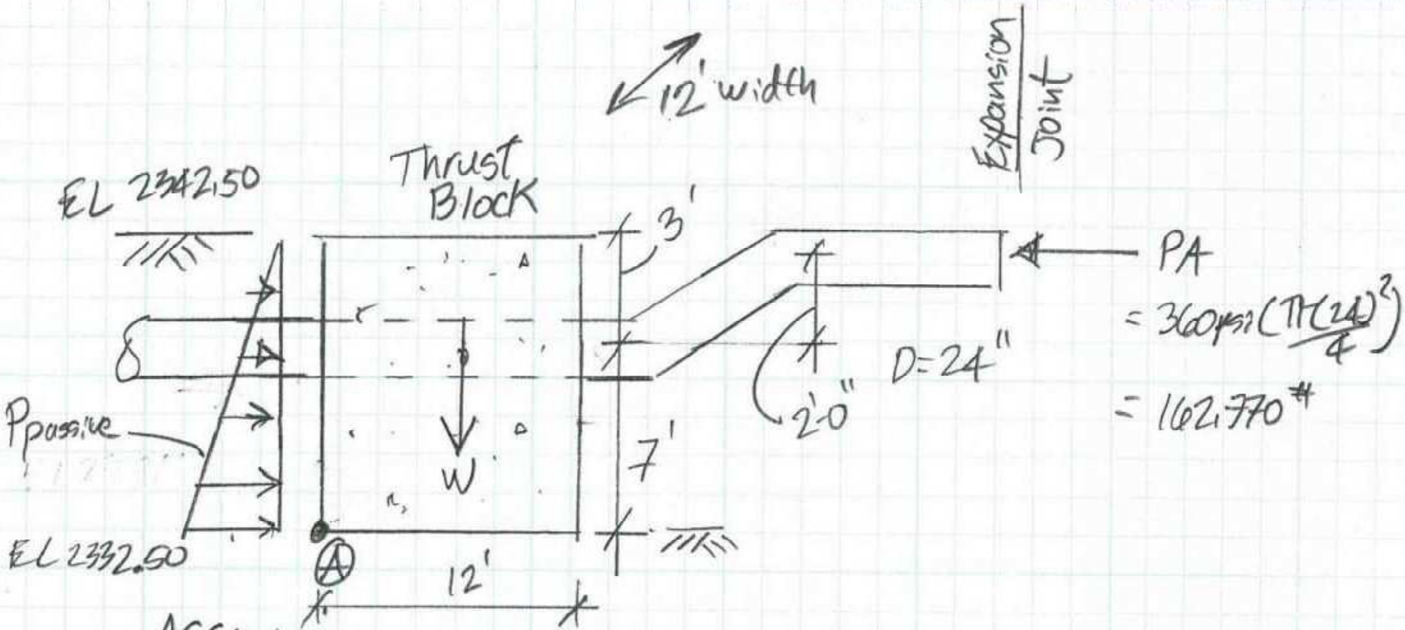




(4) #5 bars
EA SIDE
OF Pipe Penetration

Note 1: Assume 1'-0"
Tributary width
each side of
pipe

McMILLEN JACOBS ASSOCIATES	PROJECT KRRd		SHEET 7
	SUBJECT Thrust Block Calc		DATE
	BY GJC	CHECKED	PROJECT NO.



Assume:

1) concrete block resists the overturning moment

2) Axial Load (Longitudinal) is resisted by frictional forces around the pipe (soil surrounding pipe)
Also resisted by "passive" pressure at Block

Try block 10' X 12' X 12'

$$Kp = \frac{1+\sin 30^\circ}{1-\sin 30^\circ} = 3.0$$

$$wt = 150pcf(10')(12')(12')$$
$$= 216,000^{\#}$$

$$P_{passive} = \tfrac{1}{2}(120pcf)(3.0)(10')^2(10') = 180,000$$

$$P_{acting} = 162,770^{\#}$$

$$180,000 > 162,770^{\#}$$

∴ No problem w/ sliding

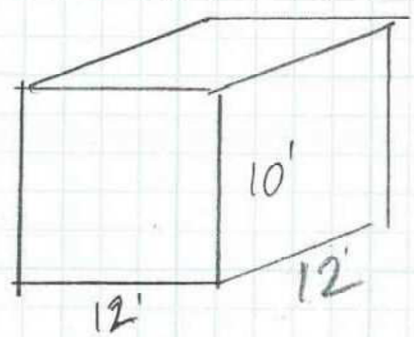
Check OT'ing ΣM_{A}

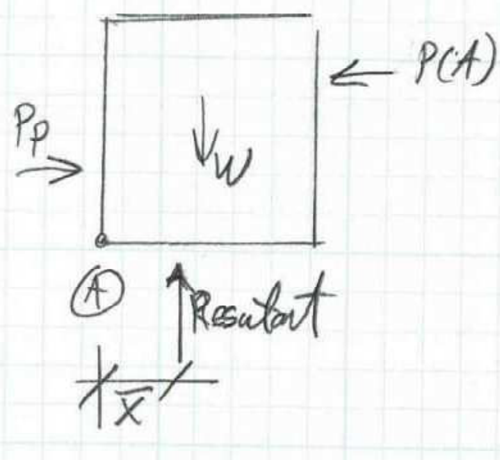
$$M_{OT'ing} = 162{,}770^{\#}(3.1') = 504{,}587 \text{ ft}\cdot\#$$

$$M_{Righting} = 150\text{pcf}(10')(12)(12)(12/2) = 1{,}296{,}000' \text{ ft}\cdot\#$$

$M_R > M_{OT}$ $\therefore$ $\underline{\underline{\text{OK}}}$

Determine Resultant Location





$$\bar{x} = \frac{M_R - M_{OT}}{\Sigma F_V}$$

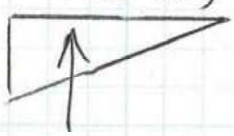
$$= \frac{1{,}296{,}000 - 504{,}587}{216{,}000} = 3.66'$$ ← Note, this is outside the Kern.

For evaluation of bearing pressure, assume $\bar{x} = 3.66'$ ft

McMILLEN JACOBS ASSOCIATES	PROJECT	SHEET 9
	SUBJECT	DATE
	BY / CHECKED	PROJECT NO.

$3.66(3') = 11'$

q_{max}



$\frac{q_{max}(11')}{2}(10') = 216{,}000$

short term loading due to surge per geotech 6/24/21

$q_{max} = \underline{3{,}927} \text{ psf} < 3{,}500 \text{ psf}\,(1.33)$

Allowable bearing pressure

Wind Loading

ASCE 7-16, Eqn 26.10-1

$$q_Z = .00256 (K_Z)(K_{ZT})(K_d)(K_e)(V^2)$$

where $K_Z = 0.85$ Velocity Pressure Exposure

$K_{ZT} = 1.0$ Topographic Factor

$K_d = 0.95$ Wind Directionality Factor

$K_e = 1.0$ Ground Elevation Factor

Velocity = 115 mph

$$q_Z = .00256(.85)(1.0)(.95)(1.0)(115)^2$$

$= 27$ psf

Note: 27 psf is the total wind pressure acting on the shroud. It would be logical to divide by two to obtain the windward and leeward pressures. However, to be conservative, it is decided to use 27 psf acting on each of the (3) surfaces (top, Left, Right)

McMILLEN JACOBS ASSOCIATES	PROJECT Daggett Road Bridge		SHEET 11
	SUBJECT Pipe Shroud		DATE
	BY GAC	CHECKED	PROJECT NO.

Determine: Design of shroud assembly

Given:

- Assumed wind load
  - 27 psf windward
  - 27 psf leeward
  - 27 psf uplift
- Snow = 40 psf

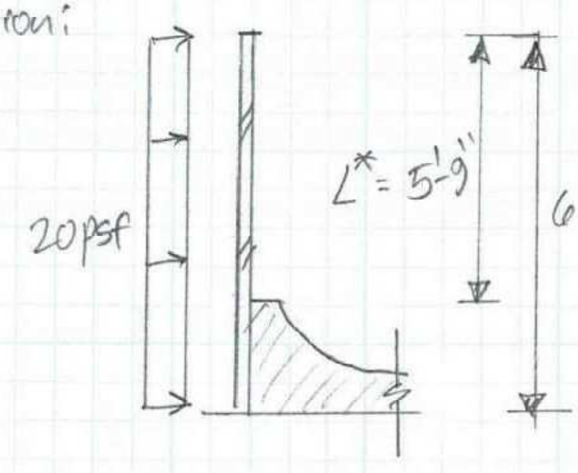
Trib Wall width = 10'-0"

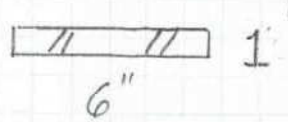
$U = 1.2D + 1.6S + .5W$

or

$U = 1.2D + 1.0W + .5S$

Solution:





$Z_x = \frac{bh^2}{4} = \frac{6"(1)^2}{4}$

$= 1.50 \text{ in}^3$

Wind Load = 27 psf (10') = 270 plf

$\phi M_n \geq M_u$

$W_u = 1.0(27 psf)(10')$

$= 270$ plf

$M_u = W_u(L^*)(L^*/2) = 279 plf(5.75)^2/2 = 4,463$ ft·#

McMILLEN JACOBS ASSOCIATES	PROJECT Daggett Road Bridge		SHEET B 12
	SUBJECT Pipe Shroud		DATE
	BY GAC	CHECKED	PROJECT NO.

$$M_u = 4{,}463 \text{ ft}\cdot\# \left(\frac{12 \text{ in}}{\text{ft}}\right) = 53{,}561 \text{ in}\cdot\text{lbs}$$

$$\phi M_n = \phi (f_y)(Z_x)$$

$$= .9(50{,}000)(1.50 \text{ in}^3)$$

$$= 67{,}500 \text{ in}\#$$

$$67{,}500 > 53{,}561$$

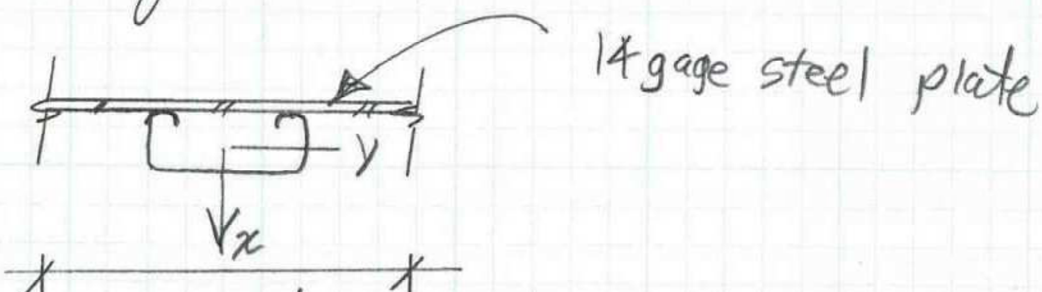
$\therefore$ use 1" x 6" vertical plates

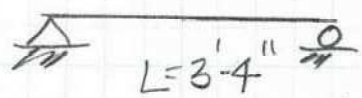
McMILLEN JACOBS ASSOCIATES	PROJECT Daggett Road Bridge	SHEET ~~004~~ 13
	SUBJECT Pipe Shroud	DATE
	BY GAC / CHECKED	PROJECT NO.

section Modulus for 600S162-54

$I_{yy} = 0.180\ in^4$ from SSMA Manual

Maximum span = 3'-4"
Spacing of stud (Horiz) = 2'-0"





- Try making the stud non-composite with the 14 gage steel plate

$W = 40psf\,(2') = 40\,plf$

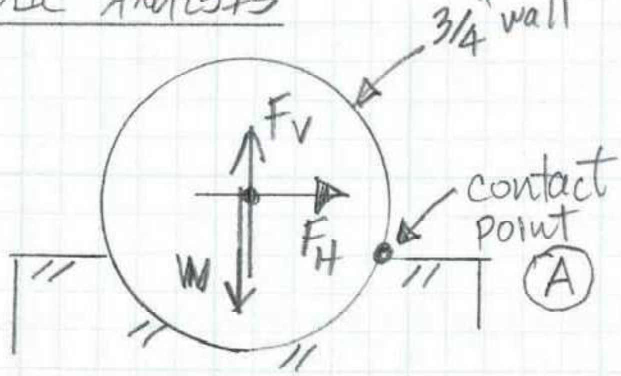
$M = \dfrac{wL^2}{8} = \dfrac{80plf\,(3.33)^2}{8} = 110\ ft\#$

$= 1{,}320\ in \cdot lbs$

$f_b = \dfrac{M}{S} = \dfrac{My}{I_{yy}} = \dfrac{1{,}320\ in \cdot lbs\,(1.62 \cdot \frac{1}{2})}{0.18\ in^4} = \qquad psi$

5,940 psi < fy ∴ ok, use
600S162-54 @ 24" OC

SEISMIC ANALSIS



Determine:
whether strap is needed at saddle to restain pipe.

$$F_p = \frac{0.4(a_p)S_{DS}W_p}{R_p/I_p}\left(1+2\frac{z}{h}\right)$$

where: $a_p = 2.5$ $\quad z/h = 0$
$R_p = 6.0$
$I_p = 1.50$
$S_{DS} = .594$

$$W_p = \underbrace{495\ \#/ft^3(\pi)(2')(3/4/12)}_{\text{Steel}} + \underbrace{62.4\ \#/ft^3\left(\frac{\pi(2)^2}{4}\right)}_{H_2O}$$

$= 390$ plf

$$F_p = \frac{.4(2.5)(.594)}{\frac{6.0}{1.5}}(390\text{ plf})$$

$F_V = .2S_{DS}W_p$

$= 57$ plf ← strength level

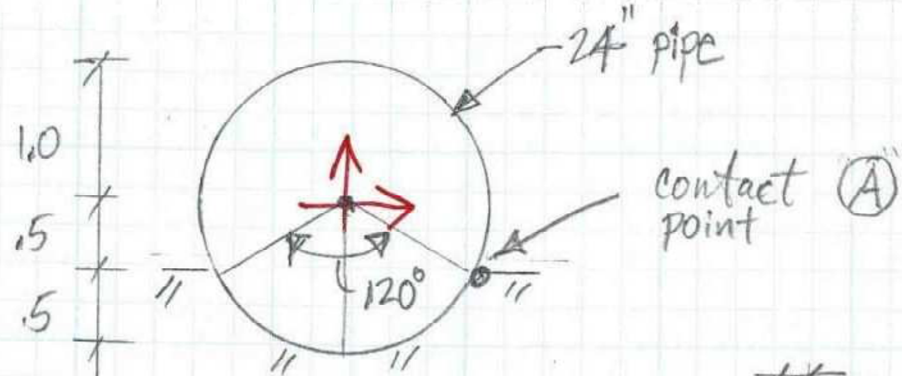
McMILLEN JACOBS ASSOCIATES

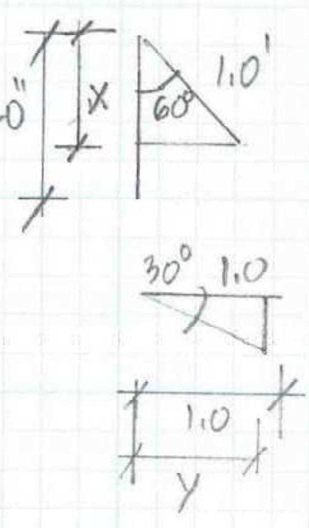
PROJECT		SHEET 15
SUBJECT		DATE
BY	CHECKED	PROJECT NO.

$F_H = 57\text{plf}/1.4 = \underline{\underline{41\text{ plf}}}$ (Service level, lateral seismic force)

$F_V = .20(.594)(390\text{plf}) = 46\text{ plf}$ ← strength level

$= 46/1.4 = \underline{\underline{33\text{ plf}}}$ (service level, vertical seismic force)





$\cos 60° = \frac{X}{1.0}$

$X = \underline{1.0(\cos 60°)}$

$= 0.50$

$y = 1.0(\cos 30°)$

$= .86'$

$\Sigma M_{OTing} @ (A)$

$33\text{plf}(.86') + 41\text{plf}(.5') = 48\text{ ft}\#$

$M_{Righting} = 390\text{plf}(.86') = 335\text{ ft}\#$

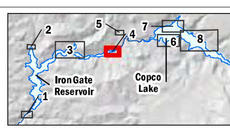
$M_R >> M_{OT}$ ∴ No problem w/ rolling out

Appendix E
Geotechnical Boring Logs

0 500 Feet

Imagery, NAIP 2014

AECOM
Klamath River Renewal Corporation
Klamath River Renewal Project



As-Drilled Boring
Project Boundary

FIGURE 1
Planned and Completed Geotechnical Borings
Sheet 4 of 8

Date(s) Drilled	**1/22/2019-1/23/209**	Logged By	**S. Janowski**	Checked By	**P. Respess**
Drilling Method	**Solid Stem Auger, HQ-3 Rock Core**	Drill Bit Size/Type	**4-inch solid stem auger, 4-inch diamond coring bit**	Total Depth of Borehole	**51.5 feet**
Drill Rig Type	**Truck Mounted CME 75**	Drilling Contractor	**Taber Drilling**	NAVD 88 Ground Surface Elevation	**2344 feet**
Groundwater Level	**11.7' 1/23/2019**	Sampling Methods	**2.5-inch ID ModCal, SPT, HQ Core Barrel**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**North end of Daggett Road Bridge**	Coordinate Location	**N 2602349 E 6462482**

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery,%	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2344	0								SANDY LEAN CLAY with GRAVEL (CL); very stiff; moist; dark brown (10yr3/3); 20% subrounded to rounded GRAVEL to 3/4"; 20% fine- to medium-grained SAND; 60% medium plasticity FINES --FILL--						
	1														
2342	2														
	3														
2340	4														
	5											6			pp=3.0 tsf
2338	6										1-1 1-2	6	78		
												8			
	7														
2336	8								CLAYEY GRAVEL with SAND (SC); very dense; moist; yellowish brown to dark brown; interbedded layers of gravel with clay and sand --ALLUVIUM--						Fill estimate based on height of slope embankment
	9														
2334	10										2	100/1"	100		
	11														
2332	12														
	13														

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	MATERIAL DESCRIPTION	Soil Samples Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13							CLAYEY GRAVEL with SAND (SC); very dense; moist; yellowish brown to dark brown; interbedded layers of gravel with clay and sand --ALLUVIUM-- *(continued)*						
2330	14													
	15									3	16 20 34	78		SA: G=42%; S=27%; F=31%
2328	16													
	17							BASALT BOULDERS and COBBLES in SAND & GRAVEL matrix; medium dark gray (N4) to dark gray (N3), strong, some boulders are scoriaceous, matrix washed out --ALLUVIUM--						Rig chatter
2326	18													
	19													
2324	20									4	17 50 50	61		
	21	1	1	100	NA	0							0921 [6] 0926	End of day 1/22/2019
													0933	Begin day 1/23/2019; AM water level=11.7' bgs Switch to HQ rock core
2322	22				NA									
	23				NA		NR							
2320	24	2		2	NA	0							[100]	
	25				NA									
2318	26				NA					5	65	100	0936	
	27												0956	
2316	28	3		71	NA	27*	NR							*Rock does not meet soundness criteria for RQD calculation
					NA			VOLCANICLASTIC BRECCIA --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--						
	29													

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
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Log of Soil and Core Boring B-15

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Number	Lithology	MATERIAL DESCRIPTION	Soil Samples: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29	3	1	71	5	27*	1,1; 1; 1	△ △	VOLCANICLASTIC BRECCIA; light olive gray (5Y5/2); moderately weathered; weak; highly to intensely fractured; angular clasts to 1/2" --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--					[68]	
2314	30				6		1; 1	△ △	1: 20°, J, MW, Sd, Sp, Wa, R Becomes grayish blue-green (5BG5/2); slightly weathered; moderately strong						*Rock does not meet soundness criteria for RQD calculation
	31				0		M	△ △						1000 / 1008	
2312	32				3		1; 1; M; 1; M	△ △	1: 30°, J, MW, Sd+Fe, Sp+Su, Wa, R						
	33				1		M; 1	△ △							
2310	34	4		100	0	82		△ △	Becomes slightly fractured					[60]	
	35				0			△ △							
2308	36				0		M	△ △						1013 / 1017	
	37				0		M; M	△ △							
2306	38				0		M; M	△ △	Becomes light olive gray (5Y5/2); moderately weathered; weak; highly fractured						
	39	5		100	1	96*	1	△ △	1: 15°, J, MW, Fe, Su, Wa, VR					[75]	*Rock does not meet soundness criteria for RQD calculation
2304	40		2		1		M; M; 2	△ △	2: 60°, J/Sh, MW, Fe+Mn+Sd, Su+Sp, Wa, R						
	41				2		1	△ △	Becomes grayish blue-green (5BG5/2); slightly weathered 1: 20°, J, MW, No, No, Wa-St, VR					1021 / 1024	
2302	42				1		1; M	△ △	Becomes moderately fractured						
	43				0		M	△ △							
2300	44	6		100	0	72	M	△ △	Becomes weak to very weak					[43]	
	45														

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-15

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing	Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	45	6	2	100	4	72		1, 1, M, M		VOLCANICLASTIC BRECCIA; grayish blue-green (5BG5/2); slightly weathered; weak to very weak; highly fractured; angular clasts to mostly to 1/2", occasionally to 1.5" --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*						
2298	46				0			M							1031	
	47	7		100	0	94		M							1035	UCS = 1546 psi
2296	48				1			1		1: 20°, J, MW, No, No, Wa, R 2: 15°, J, N-VN, Sd+Si, So-Pa, Pl, R-SR						
	49				2			2,2							[43]	
2294	50				2			2, 2								
	51				0			M							1042	
2292	52									**TOTAL DEPTH = 51.5 FEET** **Grout mix: 30 gallons of water, six 47# bags of cement, no bentonite**						
	53															
2290	54															
	55															
2288	56															
	57															
2286	58															
	59															
2284	60															
	61															

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-16

Date(s) Drilled	1/12/2019	Logged By	P. Respess	Checked By	S. Janowski
Drilling Method	Rotary Wash, HQ-3 Rock Core	Drill Bit Size/Type	3-7/8-inch tricone, 3 3/4-inch diamond coring bit	Total Depth of Borehole	24.5 feet
Drill Rig Type	Barge Mounted CME-45	Drilling Contractor	Taber Drilling	NAVD 88 Ground Surface Elevation	2319 feet
Groundwater Level	12 feet above ground surface	Sampling Methods	SPT, HQ Core Barrel	Hammer Data	Automatic hammer; 140 lbs, 30-inch drop
Borehole Backfill	Bentonite cement grout to ground surface	Borehole Location	12' downstream of Daggett Road bridge	Coordinate Location	N 2602237 E 6462573

Rock Core

Depth, feet	Run No.	Box No.	Recovery, %	R Q D, %	Fractures per Foot	Fracture Drawing Number	Drill Time [Rate, ft/hr]
3.0–4.5	1	1	100	100	3–4: 0; 4–4.5: 0	M, M, M, M	1024; [90]; 1025
4.5–9.5	2		100	100	4.5–5: 0; 5–6: 0; 6–7: 0; 7–8: 1; 8–9: 0; 9–9.5: 0	M, M, M, 1, M	1029; [150]; 1031
9.5–13	3		100	100	9.5–10: 0; 10–11: 0; 11–12: 0; 12–13: 0	M, M, M, M	1034; [100]

Soil Samples

Type	Number	Blows / 6 in.	Recovery, %
SPT (0–1.5 ft)	1	3, 5, 15	

Elevation, feet: 2318 (1 ft), 2316 (3 ft), 2314 (5 ft), 2312 (7 ft), 2310 (9 ft), 2308 (11 ft)

MATERIAL DESCRIPTION

0 ft: VOLCANICLASTIC BRECCIA; gray-green; completely weathered; extremely weak; fine-grained matrix; dark gray-black angular clasts up to 1/4"-1/2"; slightly fractured with widely-spaced natural fractures; numerous mechanical breaks
--TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)--

3 ft: Becomes moderately to slightly weathered; moderately strong; slightly fractured; multi-colored clasts up to 2"

Broken mechanical

7 ft: 1: 20°, J, N, No, No, Wa, SR

FIELD NOTES AND TEST RESULTS

12' of water in river at time of drilling

5" HWT casing driven to 14' (refusal)
Tricone to 15' and continue with HQ core
High Water Circulation Return (WCR)

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	MATERIAL DESCRIPTION	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
2306	13	3	1	100	0	100	M	VOLCANICLASTIC BRECCIA; gray-green; moderately to slightly weathered; moderately strong; slightly fractured; multi-colored clasts up to 2"; numerous mechanical breaks. Becomes clasts up to 3-4" at 13.8' --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*	1037	High WCR
	14				0		M		1040	
2304	15	4	2	100	0	100	M			
	16				1		M, 1	1: 30°, J, N, No, No, Pl-Wa, SR		
2302	17				0		M		[150]	
	18				0		M, M			
2300	19						M, M		1042	
		5		96	0	96			1045	
	20				0					
2298	21				0		M			
	22				0		M, M		[75]	
2296	23				0		1			
	24				NA		NR		1049	
	24.5							**TOTAL DEPTH = 24.5 FEET** **15 gallons of grout: 6 sack mix with 5% bentonite**		
2294	25									
	26									
2292	27									
	28									
	29									

Project: Klamath River Renewal Project
Project Location: Copco and Iron Gate Reservoirs
Project Number: 60537920

Log of Soil and Core Boring B-17

Date(s) Drilled	**1/22/2019**	Logged By	**S. Janowski**	Checked By	**P. Respess**
Drilling Method	**Solid Stem Auger, HQ-3 Rock Core**	Drill Bit Size/Type	**4-inch solid stem auger, 4-inch diamond coring bit**	Total Depth of Borehole	**41.5 feet**
Drill Rig Type	**Truck Mounted CME 75**	Drilling Contractor	**Taber Drilling**	NAVD 88 Ground Surface Elevation	**2341 feet**
Groundwater Level	**Not encountered before HQ rock coring**	Sampling Methods	**2.5-inch ID ModCal, SPT, HQ Core Barrel**	Hammer Data	**Automatic hammer; 140 lbs, 30-inch drop**
Borehole Backfill	**Cement grout to ground surface**	Borehole Location	**South end of Daggett Road Bridge**	Coordinate Location	**N 2602195 E 6462721**

Elevation, feet	Depth, feet	ROCK CORE: Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	SOIL SAMPLES: Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	0								GRAVELLY CLAY with SAND (CL); stiff; moist; dark brown (7.5YR3/3); subangular to subrounded GRAVEL to 1/2"; medium-grained SAND; medium plasticity FINES --FILL--						
2340	1														
	2														
2338	3														
	4														
2336	5										1-1 1-2	5 50/5"	100		pp=1.0 tsf Driller adds water to facilitate advancement
	6														
2334	7														
	8														
2332	9								SANDY GRAVEL (GP); very dense; moist; brown; subangular to subrounded GRAVEL to 2.25"; medium- to coarse-grained SAND --ALLUVIUM--						
	10										2-1 2-2	42 50/4"	100		
2330	11														
	12														
2328	13								VOLCANICLASTIC BRECCIA --TERTIARY VOLCANICS--						Driller felt change during advancement

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Soil Samples Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	13							△ △	VOLCANICLASTIC BRECCIA; greenish-gray (5G6/1); slightly weathered; moderately strong; slightly fractured; angular clasts to 1/2" in fine matrix --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*						
	14							△ △							
2326								△ △							
	15	1	1	100	0	100	M	△ △		▯	3	50/4"	100	1110 [45] 1112	Switch to HQ core
2324	16				0		M	△ △							
	17	2		100	0	100		△ △						1147	
2322	18				0		M, M, M, M	△ △						[75]	UCS = 2130 psi
	19				0			△ △							
2320	20				0			△ △							
	21				0			△ △						1151	
2318	22	3		100	0	84		△ △						1216	
	23				1		1	△ △	Very weak 1: 20°, J, VN, Cl+Sd, Sp, Pl, S-SR						
2316	24				2		1, 1	△ △	Weak					[100]	
	25				0		M	△ △							
2314	26				0		M, M	△ △						1219	
	27	4		100	0	100		△ △						1223	
2312	28				0		M	△ △						[75]	
	29														

Project: Klamath River Renewal Project	Log of Soil and Core Boring B-17
Project Location: Copco and Iron Gate Reservoirs	Sheet 3 of 3
Project Number: 60537920	

Elevation, feet	Depth, feet	Run No.	Box No.	Recovery, %	Fractures per Foot	R Q D, %	Fracture Drawing Number	Lithology	MATERIAL DESCRIPTION	Type	Number	Blows / 6 in.	Recovery, %	Drill Time [Rate, ft/hr]	FIELD NOTES AND TEST RESULTS
	29		2		0				VOLCANICLASTIC BRECCIA; greenish-gray (5G6/1); slightly weathered; moderately strong; slightly fractured; angular clasts to 1/2" in fine matrix --TERTIARY VOLCANICS (BOGUS MOUNTAIN BEDS, undifferentiated)-- *(continued)*						
	30	4		100	0	100								[75]	
2310	31								Coarser clasts to 2"						
					0		M							1227 / 1231	
	32				0										
2308	33				0										
	34	5		100		86								[100]	
2306					1										
	35				3		1		Light brownish gray (5YR6/1); moderately weathered; weak; highly fractured 1: 60°, J/Sh, W, Ca+Mn+Fe, Pa+Su IR, VR-Slk (rough grooves)						
	36						M		Abundant mechanical fractures						
2304					0									1234 / 1239	
	37				0		M, M								
	38				0		M								
2302	39	6		100		100								[75]	
					0										
	40														UCS = 2985 psi
2300					0		M		1.5" Clast						
	41				0									1243	
	41.5								**TOTAL DEPTH = 41.5 FEET** **Grout mix: 20 gallons of water, five 47# bags of cement, no bentonite**						
	42														
2298	43														
	44														
2296	45														

Report: GEO_CORE+SOIL_NO PACK_WITH LITH; File: KLAMATH_MASTER.GPJ; 6/20/2019 B-17