State of Washington
DEPARTMENT OF FISH AND WILDLIFE

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Main Office Location: Natural Resources Building · 1111 Washington Street SE · Olympia, WA

March 16, 2012

Mr. Arny Stonkus, P.E.
Wild Fish Conservancy
P.O. Box, 15629 Main Street NE
Duvall, WA 98019

Re: Cherry Valley Wildlife Area - Bridge Installation

Dear Mr. Stonkus:

A packet of information regarding the bridge to be installed over Waterwheel Creek at the Cherry Valley Wildlife Area is attached. This packet includes a signed and stamped Geotechnical Report prepared by Landau and Associates. This report was provided to Reid Middleton for design of the foundation. A signed and stamped foundation design is also attached in the packet.

You also requested a signed and stamped bridge design. We are currently soliciting this bridge for purchase and have provided our bridge specifications in the packet. We require that the bridge vendor supply the design and calculation packet upon issuance of the purchase order. We will provide you with that information as soon as we get it. As an agency, we require that all bridges we purchase to be designed and stamped by a Washington State licensed structural engineer. We also require that the engineer have at least 5 years of bridge design experience. Because this will be a steel bridge, we are also requiring that the fabricator is certified in American Institute of Steel Construction in small bridge fabrication. We hold our bridge vendors to the highest of standards. The loading criteria for the bridge are described in the specifications. A set of these specifications was provided to the foundation design engineer as well and used in the calculation of the foundation design.

For reference purposes only, a set of drawings and calculations from a similar bridge is included in the packet. This is the standard of design and calculation we require from our bridge vendors. We expect to have the actual plans and specifications for the Waterwheel Creek Bridge in 4-6 weeks. Understanding that this information is required by King County, we will do our best to expedite the process. We have also provided a cost estimate for the acquisition of the bridge and the construction of the foundation.

The following items are attached in the packet:

1. Geotechnical Report
2. Plan Sheet for Waterwheel Creek
3. Foundation Design
4. Cost Estimate for the Bridge
5. A sample of a typical bridge design that we require of our vendors (Tunk Creek Bridge)

Thank you for your assistance with this project. If you have any questions, please contact Kristen Kuykendall, P.E. at 360.902.8383, or via email at Kristen.Kuykendall@dfw.wa.gov.

Sincerely,

Glenn F. Gerth, P.E.
Program Business Manager

Attachments (5)

cc: Kristen Kuykendall, P.E., Environmental Engineer, CAMP
Read File

Geotechnical Design Recommendations
Cherry Valley Culvert Replacement Project
Duvall, Washington

March 15, 2012

Prepared for

Washington State Department of Fish and Wildlife

LANDAU ASSOCIATES
130 2nd Avenue South
Edmonds, WA 98020
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Appendix	Title
A	Field Explorations and Laboratory Testing
B	Stability Thresholds for Stream Restoration Materials

1.0 INTRODUCTION

This report presents the results of our field investigations and provides our geotechnical engineering conclusions and recommendations for design and construction of bridges at four separate work areas at the Cherry Valley site north of Duvall, Washington. The proposed bridges will replace existing culverts crossing drainage ditches at the project site. The purpose of this study was to complete one soil boring at each proposed bridge location, to characterize subsurface soil and groundwater conditions encountered, and to develop geotechnical conclusions and recommendations for design and construction of the bridges.

The general project location is shown on the Vicinity Map (Figure 1). The locations of the four work areas are shown on Work Area Locations (Figure 2). An exploration plan and proposed bridge section is presented for each of the work areas on Figures 3 through 6. We understand that final bridge superstructure configurations may change somewhat from those shown on Figures 3 through 6. Appendix A presents a description of the field exploration program and summary boring logs. Appendix A also includes a description of the laboratory testing program and the results of the laboratory testing.

This report has been prepared based on our discussions with representatives of Washington State Department of Fish and Wildlife (WDFW); permit level drawings prepared by WDFW; anticipated foundation loads provided by WDFW and Reid Middleton; the results of the explorations completed for this project; our familiarity with geologic conditions within the vicinity of the project area; and our experience on similar projects.

1.1 PROJECT UNDERSTANDING

We understand that WDFW plans to replace four existing 3- to 4-foot diameter culverts at several locations (Work Areas 1 through 4) with single span, voided-slab concrete bridges. A steel girder bridge is also under consideration at Work Area 4. As currently envisioned, the bridges will be approximately 14 ft wide and between 20 and 45 ft long. We further understand that the standard foundation for these bridges is typically about 3 ft wide and 16 ft long. Reactions at the bridge supports at Work Area 4 (the longest bridge) are expected to be about 40 kips for dead loads (DC and DW) and about 20 kips for vehicle live load (LL); somewhat lower dead loads are anticipated for the bridges at the other Work Areas. Due to relatively low live load capacity, the bridges will have a posted 10-ton weight restriction. The bridges are to be designed in general accordance with the 2010 AASHTO *LRFD Bridge Design Specifications* (2010 AASHTO *LRFD Specifications*; AASHTO 2010).

1.2 SCOPE OF SERVICES

Landau Associates was contracted by WDFW to provide geotechnical services to support design of the project. Our services were provided in accordance with our Revised Proposal for Geotechnical Engineering Services dated November 30, 2011 and Amendment No. 3 to the 2011-2013 On-Call geotechnical services contract between WDFW and Landau Associates. Written authorization to proceed was provided by Mr. Bill Phillips on December 13, 2011.

Our scope of services included the following specific tasks:

- Reviewing readily available geological and geotechnical information for the project area.
- Drilling one soil boring at each of the four culvert replacement sites to characterize subsurface soil and groundwater conditions at each location. Borings were advanced to depths ranging from approximately 29 to 49 ft below the existing ground surface (BGS).
- Logging each soil unit observed in the exploratory borings and recording pertinent information, including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence.
- Completing geotechnical laboratory testing consisting of natural moisture content and Atterberg limit determinations on selected soil samples obtained from the borings. One dimensional consolidation testing was also performed on one soil sample. The results of the laboratory analyses are presented in Appendix A.
- Submitting two soil samples for corrosion and sulfide testing, in order to assess the potential for corrosion of buried metallic foundation elements.
- Completing geotechnical engineering analyses and developing geotechnical engineering conclusions and recommendations for design and construction of the proposed bridges.
- Preparing and submitting this written report summarizing our findings, conclusions, and recommendations for the project. The report includes:
    - a site plan showing the locations of the subsurface explorations completed for this investigation
    - descriptive summary logs of the conditions observed in the subsurface explorations completed for this study
    - a summary of surface and subsurface conditions observed in the vicinity of the bridge work areas
    - an evaluation of appropriate foundation types to support the proposed bridges
    - code based seismic design parameters
    - an assessment of the geologic hazards at the site including: evaluation of stream bank slope stability under static and seismic loading conditions and liquefaction and lateral spreading
    - recommendations for deep foundations, including recommended type; anticipated tip elevation; axial capacity; settlement, uplift capacity; L-pile parameters for 12-inch diameter driven steel pipe piles; and parameters for contractor design of helical piers or helical piles

- a discussion of lateral loads due to backfill against buried portions of the bridge foundations
- a discussion of the erosion potential of exposed soil slopes following culvert removal.

2.0 EXISTING CONDITIONS

This section provides a discussion of the general surface and subsurface conditions observed at the work areas at the time of our investigation. Interpretations of the site conditions are based on the results of our review of readily available geologic and geotechnical information, and the results of our site reconnaissance, subsurface explorations, and laboratory testing.

2.1 SURFACE CONDITIONS

The work areas are located within Cherry Valley–a broad, relatively flat, east-west trending valley that extends about 2.5 miles east of the Snoqualmie River about a mile north of Duvall, Washington. The work areas are within the Snoqualmie floodplain. At the time of our site visits, vegetation around each work area consisted of tall, wild grasses and scattered brush. "Spongy" ground, muddy conditions, and areas of standing water were noted in many areas around the project site. The graveled access road that parallels the Lateral A ditch was moderately passable by the 4x4 pickup truck and track-mounted drill rig used during subsurface exploration in December 2011, though some areas of shallow ponded water were present along portions of the wheel paths. At Work Areas 1, 2, and 3, significant depressions were present above the culvert crossings. This suggests that large settlements of the culvert and overlying fill have occurred since these culverts were installed. Large settlements were not evident at Work Area 4, although this could be due to filling of previous depressions.

2.2 GEOLOGIC SETTING

General geologic information for the project area was obtained from the *Geologic Map of the Monroe 7.5-Minute Quadrangle, King and Snohomish Counties, Washington* (Dragovich et al. 2011), published by the Washington Division of Geology and Earth Resources. According to Dragovich, subsurface deposits at the project site consist of Holocene-age alluvial levee deposits and peat deposits. These deposits typically consist of very soft or loose deposits of sand, silt, and mud (levee alluvium) and peat, muck, and organic silt and clay (peat deposits). Subsurface conditions observed in our explorations are consistent with the mapped geologic unit descriptions. None of our explorations encountered firm or dense soil conditions suitable for the support of shallow foundations, although suitable bearing can be provided by lower soils using deep foundations. A discussion of suitable foundation types is discussed in a later section.

2.3 SUBSURFACE SOIL CONDITIONS

Subsurface soil and groundwater conditions at the proposed bridge locations were explored on December 12 and 13, 2011 by advancing and sampling four soil borings using a track-mounted drill rig and the mud-rotary drilling method. Borings B-1 and B-3 were each advanced to a depth of about 49 ft BGS. Borings B-2 and B-4 were each advanced to a depth of about 29 ft BGS. The approximate locations of these exploratory borings are shown on Figures 3 through 6 of this report. A discussion of the field exploration procedures, together with edited logs of the exploratory borings, is presented in Appendix A. A discussion of the laboratory test procedures and the test results are also presented in Appendix A.

At boring B-1 (Work Area 1), we observed approximately 2 ft of gravelly, silty sand (fill) overlying about 8 ft of very soft, highly compressible, organic-rich clay and silt with occasional thin organic fiber mats. From about 10 to 30 ft BGS, we encountered very soft, low- to high-plasticity silt with scattered to occasional organics. Soft to very soft, organic-rich silt was again encountered from about 30 to 35 ft BGS, followed by loose to medium dense silty sand and sand with silt from about 35 to 45 ft BGS. Very soft clayey silt was encountered from about 45 ft to the bottom of the boring at 49 ft BGS.

At borings B-2, B-3, and B-4, about 2 ft of gravelly, silty sand (fill) overlying interbeds of very soft, organic-rich silt, clay and peat to depths ranging from about 20 to 28 ft BGS (note, however, a 6 ft-thick layer of fine sand at boring B-3). Below these depths we observed soft to very soft, low- to high-plasticity silt with relatively few organics.

2.4 GROUNDWATER

Due to the mud rotary drilling method used to advance the exploratory borings and the low-permeability of the fine-grained soils encountered, at-time-of-drilling water levels should be considered approximate. At the time of drilling, groundwater was observed at approximately 2 to 5 ft BGS in each boring. Due to the close proximity of the drainage ditch to the boring locations, these depths appear to correlate with the water level observed in the ditch during drilling. It should be noted that the groundwater conditions reported on the summary logs are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and times. Furthermore, it is anticipated that groundwater conditions will vary depending on subsurface conditions, the weather, the level of the water in the adjacent oxbow, and other factors. Groundwater levels are expected to fluctuate seasonally.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on conditions observed during subsurface exploration, shallow foundations would not be appropriate for support of the proposed replacement bridges. Shallow foundations would experience excessive settlements. Weak soils in the shallow subsurface do not provide a factored resistance exceeding the factored bridge loads. Bridge support should be provided by deep foundations, such as driven or drilled piles or micropiles. Our recommendations for deep foundations are included in a following section.

Permanent cuts of between 1½H:1V and 2H:1V are feasible as shown on the preliminary sections.

3.1 SEISMIC HAZARDS

The Pacific Northwest is seismically active and the site could be subject to ground shaking from a moderate to major earthquake. Consequently, earthquake shaking should be anticipated during the design life of the proposed bridges and they should be designed to resist earthquake loading using appropriate design methodology.

3.1.1 CODE BASED SEISMIC DESIGN

We understand that seismic design of the proposed bridges will be in accordance with the 2010 AASHTO *LRFD Specifications*. Based on the results of the borings completed for the project, the near-surface soil profile is classified as Site Class E. The following spectral accelerations for an earthquake with a 7 percent probability of exceedance in 75 years (USGS 2012) should be used to determine the design response spectrum per Figure 3.10.4.1-1 of the 2010 AASHTO *LRFD Specifications*:

Spectral Acceleration for Short Periods (S_S):	81% of gravity (0.81g)
Spectral Acceleration for a 1-Second Period (S_1):	27% of gravity (0.27g)
Peak Horizontal Ground (Soft Rock) Acceleration Coefficient (PGA):	36% of gravity (0.36g)

The following table summarizes the values of the site coefficients that should be utilized for Site Class E.

SITE COEFFICIENTS FOR SITE CLASS E

	F_a	F_v	F_{pga}
Site Class E	1.12	2.92	1.01

3.1.2 LIQUEFACTION

Liquefaction is defined as a significant rise in pore water pressure within a soil mass caused by earthquake-induced cyclic shaking. The shear strength of liquefiable soil is reduced during large and/or long-duration earthquakes as the soil consistency approaches that of a semi-solid slurry, which can result in significant and widespread structural damage if not properly mitigated. Deposits of loose, granular soil below the water table are most susceptible to liquefaction, although some non-plastic and low-plasticity silts and clays are also considered to be susceptible to liquefaction. Damage caused by foundation rotation, slope failure, seismically induced settlement, and a reduction in bearing capacity are frequently observed in areas where liquefaction has occurred.

Geotechnical data (i.e., Standard Penetration Test blow counts) from the soil borings advanced for the project were analyzed to estimate the factor of safety against liquefaction at the proposed bridge locations. The potential for liquefaction of the granular soils was assessed using the "modified simplified procedure" presented by Youd et al. (2001). Where appropriate, recommendations for liquefaction susceptibility evaluations proposed by Idriss and Boulanger (2008) were incorporated into our analysis.

The liquefaction susceptibility of the cohesive soils was assessed using the method proposed by Bray and Sancio (2006). According to the criteria described by Bray and Sancio, any fine-grained deposit with a plasticity index less than 12 and a water content greater than 85 percent of the liquid limit is potentially susceptible to liquefaction.

The potential for liquefaction was evaluated for an earthquake with a return period of 975 years (7 percent probability of exceedance in 75 years). This seismic event corresponds to a magnitude 6¾ earthquake producing a peak horizontal ground acceleration of 0.37g (F_{pga}*PGA). According to the WSDOT *Geotechnical Design Manual* (WSDOT GDM; WSDOT 2011), a factor of safety of 1.2 or less indicates that liquefaction is likely to occur during the design seismic event.

Our analysis indicates that the coarse-grained alluvium present between 35 and 40 ft BGS in boring B-1 and between 5 and 11 ft BGS in boring B-3 have a factor of safety of less than 1.2, and are therefore considered susceptible to liquefaction during the design seismic event. Based on the results of the laboratory testing of the samples obtained from the borings, about 10 percent of the fine-grained deposits meet the criteria established by Bray and Sancio for being susceptible to liquefaction.

Liquefaction of the near-surface soils will result in ground subsidence. The amount of post-liquefaction ground subsidence during the design earthquake was estimated using empirical methods developed by Tokimatsu and Seed (1987). The empirical method is based on studies of areas that had undergone liquefaction. We estimate that liquefaction induced settlement will be on the order of 0 to about 2 inches. Differential settlement at each bridge abutment could approach one-quarter of the total settlement. Differential settlement between the two bridge abutments could approach one-half of the total settlement.

The actual magnitude and extent of liquefaction and liquefaction induced settlement will depend on many factors. These factors include the duration and intensity of the ground shaking during the seismic event and local soil and groundwater conditions. Therefore, the extent of liquefaction, as well as the magnitude of associated settlement, may vary from that estimated above.

3.2 STABILITY OF PROPOSED CUT SLOPES

As currently envisioned, the existing culverts will be removed at each proposed bridge crossing and the new ditch slopes at those locations will be excavated to inclinations of 1½H:1V to 2H:1V (horizontal to vertical). A total slope height of between 5 and 7½ ft is planned at each crossing. The computer slope stability program, SLIDE Version 5.0 (Rocscience Inc. 2003), was used to evaluate the global stability of cut slopes at the work areas under static and seismic loading conditions. SLIDE evaluates the stability of circular and non-circular failure surfaces in soil or rock using vertical slice limit equilibrium calculations. For this project, several calculation methods were used to calculate forces that would cause slope movement (driving forces) and the forces resisting slope movement (resisting forces) for each selected failure surface. The ratio of resisting force to driving force for a given failure surface is referred to as the factor of safety (FOS). SLIDE uses a searching routine to determine the critical failure surfaces (i.e., those surfaces with the lowest factors of safety) for a given slope.

The potential effect of seismic loading on the proposed cut slopes was analyzed assuming a peak horizontal ground acceleration of 0.37g (F_{pga}*PGA) for a seismic event with a probability of exceedance of 7 percent in a 75-year period (return period of 975 years). The horizontal forces developed during earthquake shaking were represented in the stability analyses by a "pseudo-static" seismic coefficient, k_h. For slopes in natural soils, it is typical to assume that k_h is equal to approximately one-half of the horizontal peak ground acceleration from the design level earthquake. Therefore, for the purpose of our seismic stability analysis, k_h was assumed to be 0.18. The pseudo-static slope stability analysis is generally only completed on the critical surface determined during the static slope-stability analysis.

At Work Areas 1, 2, and 4, soils throughout the depth of interest for slope stability were modeled as very soft, cohesive soil. At Work Area 3, this same soil type was included in the SLIDE model, but an intervening cohesionless soil was modeled between 5 and 11 ft BGS. These soil types are consistent with soil types observed during drilling. For the purpose of the SLIDE analysis, groundwater was assumed to be located 5 ft BGS; this depth to groundwater was observed to produce lower slope stability results than those calculated for other groundwater levels analyzed. Summaries of the SLIDE input parameters and calculated results are tabulated below.

Work Areas 1, 2, and 4 (1½H:1V Slopes)

Depth Range	Slope Height	Static Analysis		Pseudo-Static Seismic (k_h = 0.18g)
		Drained	Undrained	
0-20 ft	6 ft, total	ϕ' = 24° c' = 50 psf	S_u = 150 psf	S_u = 150 psf
Minimum F.o.S.:		1.58	1.64	1.3

Work Area 3 (2H:1V Slope)

Depth Range	Slope Height	Static Analysis		Pseudo-Static Seismic (k_h = 0.18g)
		Drained	Undrained	
0-5 ft	7.5 ft, total	ϕ' = 24° c' = 50 psf	S_u = 150 psf	S_u = 150 psf
5-11 ft		ϕ' = 33° c' = 0 psf	(drained params. used)	(drained params. used)
11-20 ft		ϕ' = 24° c' = 50 psf	S_u = 150 psf	S_u = 150 psf
Minimum F.o.S.:		1.35	1.59	1.05

In a slope stability analysis (either static or pseudo-static seismic loading), slopes with a high potential of failure are represented by a FOS of 1.0 or less. A FOS of 1.0 indicates that the forces driving instability are equal to the forces resisting instability. Washington State Department of Transportation (WSDOT) provides guide recommendations for appropriate factors of safety for cut slope. Cut-slopes are typically considered to be stable under permanent or sustained conditions (i.e., static loading conditions) if the calculated minimum FOS is 1.25 or greater. For seismic loading conditions, a FOS of 1.05 or greater is considered appropriate when evaluating the likelihood of failure. These FOS levels are appropriate where impacts to foundations are likely to be minor or non-existent, such as for the proposed bridges supported by deep foundations.

Our analysis indicates that the proposed 1½H:1V cut slopes at Work Areas 1, 2, and 4 are adequately stable under static and seismic loading conditions. At Work Area 3, the proposed 2H:1V cut-slope is adequately stable under static and seismic loading conditions.

3.3 BRIDGE FOUNDATIONS

Based on the subsurface conditions observed in our borings, weak, highly-compressible, organic-rich soils are present beneath each of the proposed bridge locations to depths ranging from 10 to 28 ft BGS. Below these depths, very soft to soft clayey silt and occasional interbeds of loose to medium dense sand are present. We used the computer program Settle3D (Rocscience 2010), results of consolidation testing, and correlations of soil compressibility with geotechnical index tests to estimate the amount of settlement that would occur beneath 3 ft x 16 ft spread footings supporting the bridge ends. Soils at each work area are unsuitable for the use of shallow footings to resist bridge loads and would experience settlements of about 2 to 4 inches within a year following construction, plus 1 to 3 inches of additional settlement over the next 10 to 20 years. In addition, the soils are not strong enough to provide an adequate factored resistance to support factored bridge reaction loads. Larger, deeper footings or gravel mats would increase the factored foundation resistance; however, we calculated that even a significantly larger footing (5 ft x 16 ft) would not provide adequate resistance. In addition, larger footings would cause additional settlement and would likely require temporary construction de-watering and excavation shoring. Although pre-loading of soft soils can be an effective strategy to strengthen the soil and induce settlement prior to construction, we do not think that pre-loading would be able to strengthen the soil sufficiently to allow spread footings or cause settlement to occur quickly enough to meet the desired project construction schedule. In addition, pre-loading could destabilize the existing drainage ditch slopes and impact the project area unduly due to the large quantity of imported pre-load fill that would be hauled to the site. Finally, pre-loading the soils would likely not mitigate the liquefaction hazard.

Bridge loads should be supported by deep foundations. At Work Area 4, unfactored dead and live load reactions of approximately 40 and 20 kips (respectively) will be resisted by the foundations at each bridge end. Somewhat lower dead loads are anticipated for the bridges at the other Work Areas. Due to the potential for liquefaction, and to reduce settlements of the bridge foundation during service loading and following liquefaction, we recommend that all of the bridge foundations extend to a depth of at least 50 ft BGS. As discussed below, deep foundations could include driven piles, drilled micropiles, or helical piles. Based upon discussion with the project team, we understand that drilled micropiles are the preferred foundation alternative.

3.3.1 PILES AND MICROPILES

Driven piles and drilled micropiles share many similar properties and design considerations, which are discussed here and in several subsections.

Site access constraints, potential pile driving impacts, and crane requirements could make the installation of driven piles difficult. However, driven piles would provide adequate support of the bridge without generating drilling spoils and would allow larger diameter piles to be used, possibly reducing foundation material and labor costs. For initial design, we estimated the capacity of driven 12-inch diameter steel pipe piles. Driven piles can be installed using conventional hammer-impact equipment (air, steam, or diesel). Installation using vibratory methods may also be successful at the project site.

Drilled steel micropiles allow the use of smaller installation equipment and can provide construction advantages where a limited construction footprint is available. Micropiles are typically drilled to the design depth using steel casing segments that are 5 to 10 ft in length. Upon reaching the design depth, the casing is partially retracted while Portland cement grout is injected into the borehole. We recommend that permanent micropile casings extend at least 20 ft BGS in order to reduce downdrag loads resulting from long-term settlement of the shallow organic soils. Depending upon the grout injection method within the bearing layer, the grout-to-ground bond strength can vary widely. Because of their small diameter (typically 3 to 10 inches) micropiles are usually installed in groups. Groups consisting of a greater number of smaller diameter micropiles can be used to reduce negative moments within the pile cap due to point loading of the foundation elements. This approach also allows for greater design redundancy. For initial design, we estimated the capacity of 6- and 10-inch diameter micropiles with low-pressure ("hydrostatic") grout injection. Higher injection pressures usually produce much higher capacities.

Axial Capacity

For 12-inch diameter closed-end steel pipe piles driven to a depth of 50 ft BGS, we estimate an ultimate load capacity of about 45 kips per pile. For 6- and 10-inch micropiles drilled to a depth of 50 ft BGS, we estimate ultimate capacities of 21 and 34 kips, respectively. The estimated ultimate capacity for other depths is presented on Figure 7. Downward axial loads will be carried primarily by skin friction developed along the portion of the pile located below the highly-organic soils at each Work Area. This occurs at depths of between 10 and 28 ft BGS (the values plotted assume a depth of 20 ft). For driven piles in soft clay, a small portion of the total capacity is provided by end-bearing. For micropiles, any contribution from end-bearing is typically ignored. LRFD procedures require using resistance factors that reflect the loading condition (i.e., Limit State) and the foundation capacity calculation method used. For piles and micropiles, we recommend that the following resistance factors be used when evaluating the Service, Strength, and Extreme limit states:

- For driven piles, a resistance factor of 0.35 should be applied for the Strength Limit load condition

- For drilled micropiles, a resistance factor of 0.44 should be applied for the Strength Limit load condition. In addition, load testing of at least one micropile at each Work Area must be performed to validate design assumptions.

- For piles and micropiles, a resistance factor of 1.0 should be used for the Service and Extreme Limit load conditions.

The capacity of a pile or micropile is reduced when installed in proximity with other piles or micropiles. For groups with spacings of less than 6 pile/micropile diameters (center-to-center), a group reduction factor should be applied to each pile or micropile. This factor may be taken as 1.0 for a spacing of 6 diameters and 0.65 at a spacing of 2.5 diameters; linear interpolation of the group reduction factor should be made for intermediate spacings.

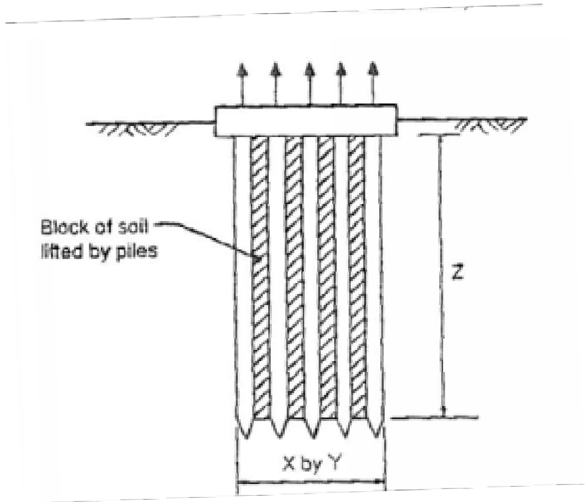
Downdrag Loads

The near-surface high-organic content soils will settle during the design life of the proposed bridges. The settlement of these soils will create downdrag loads on the foundations that should be considered in the design. We estimate that the average downdrag load on 12-inch diameter steel pipe piles will be about 8 kips. For 6- and 10-inch micropiles, we estimate an average downdrag of about 4 and 7 kips, respectively. Downdrag loads at each of the work areas will vary due to differing thickness of the uppermost organic layers, plus variations in the soil from area to area. Downdrag loads should be added to the factored dead and live loads for the Strength and Service Limit state. We recommend a load factor of 1.4 for downdrag loads at Strength Limit state and 1.0 for the Service Limit. Downdrag loads need not be considered for the extreme limit state.

Uplift Capacity

The unfactored (i.e., “ultimate) uplift resistance of a pile or micropile group, R_{ug}, should be taken as the minimum of the nominal uplift resistance of the group considered as a block or the sum of the individual uplift resistances, including the group reduction factor.

The nominal uplift resistance for a single pile due to side resistance, R_s, is presented on Figure 7. The nominal uplift resistance of a group can be taken as the weight of the soil and pile located within a prism as shown in the diagram below. A buoyant unit weight of 42 pounds per cubic foot (pcf) should be utilized in the soil weight calculations.



We recommend that the following resistance factors be used when evaluating the service, strength, and extreme limit states for single piles and pile groups loaded in uplift.

UPLIFT RESISTANCE FACTORS FOR PILE AND MICROPILE DESIGN

Limit State	Resistance Factor, ϕ	
	Single Pile/Micropile	Group
Strength	0.25	0.50
Service	1.00	1.00
Extreme	0.80	0.80

Lateral Capacity

The following tables provide the recommended input soil parameters for an LPILE Plus 5.0 (Ensoft 2005) analysis of lateral load behavior (both static and seismic conditions). These values can be applied for driven piles and drilled micropiles. Generally, lateral resistance behavior is most influenced by soil characteristics a relatively short distance below the pile cap. At Work Areas 1, 2, and 4 the following soil profile inputs should be used for lateral analysis.

RECOMMENDED INPUT SOIL PARAMETERS FOR LATERAL ANALYSIS – WORK AREA 1, 2, 4

Depth BGS[a] (inches)	Soil Type/(Soil Profile Number)	Soil Unit Weight (pci)	Soil Internal Angle of Friction (degrees)	Soil Cohesion (psi)	k-value (pci)	E_{50}
0 to 24	Sand/(4)	0.064	32	---	25	---
24 to 120	Soft Clay/(1)	0.019[b]	---	1.0	---	0.020
>120	Soft Clay/(1)	0.027[b]	---	2.0	---	0.020

(a) Below Ground Surface as measured from boring location
(b) Buoyant unit weight.

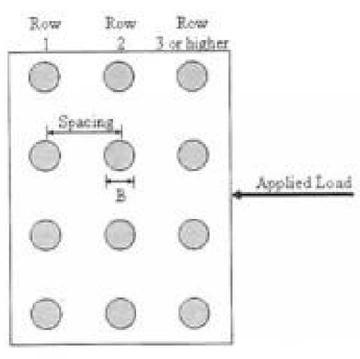
Due to the sand layer present at Work Area three, a modified soil profile should be considered for lateral resistance at that location.

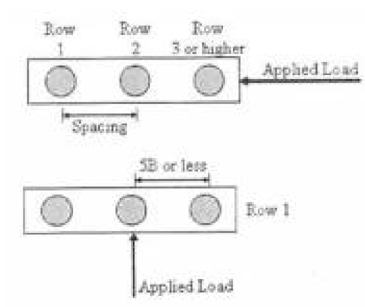
RECOMMENDED INPUT SOIL PARAMETERS FOR LATERAL ANALYSIS – WORK AREA 3

Depth BGS[a] (inches)	Soil Type/(Soil Profile Number)	Soil Unit Weight (pci)	Soil Internal Angle of Friction (degrees)	Soil Cohesion (psi)	k-value (pci)	E_{50}
0 to 24	Sand/(4)	0.064	32	---	25	---
24 to 60	Soft Clay/(1)	0.019[b]	---	1.0	---	0.020
60 to 132	Sand/(4)	0.027[b]	33	---	60	---
>132	Soft Clay/(1)	0.027[b]	---	2.0	---	0.020

(a) Below Ground Surface as measured from boring location
(b) Buoyant unit weight.

To account for group effects, the soil resistance, p, shall be reduced as specified in the following table. The multipliers (P_m) are a function of the center-to-center spacing of the piles (expressed in multiples of pile diameter, D) in the group in the direction of the loading. Sketches showing load and row configurations are presented below.





P_m values should be selected from the following table. For intermediate pile spacings, P_m values may be interpolated.

PILE LOAD MODIFIERS, Pm

Pile Center to Center Spacing[(a)]	Pile Load Modifiers, P_m		
	Row 1	Row 2	Row 3 and Higher
2D	0.45	0.33	0.25
3D	0.70	0.50	0.35
5D	1.00	0.85	0.70

(a) In direction of loading.

Construction Considerations

Deep foundations will be driven or drilled through alluvium composed of very soft to soft organic clayey silt, with layers of peat and loose to medium dense sand. At Work Area 1, dense sand may be encountered near the bottom of the foundations. Wood debris and logs are often encountered in alluvial deposits and may be encountered during culvert removal and foundation construction. During subsurface exploration, we drilled 24 inches through a log at Work Area 2. When encountered within a few feet of the ground surface, logs can often be removed with an excavator or backhoe. Where encountered at depth, logs could obstruct driven piles and possibly result in damage to the piles. Micropiles can often be drilled through wood, although this may require an adjustment in drilling technique. If an obstruction is encountered and a pile cannot be advanced, or if the pile becomes damaged it may be necessary to abandon and relocate the pile. If the pile is obstructed but undamaged, it could possibly be incorporated into the structure at a reduced capacity. Landau Associates should be consulted to determine the allowable reduced capacity. Abandoned piles should either be extracted or cut off at least 3 ft below the bottom of the pile cap.

If driven piles are selected for foundations on this project, pile driving should be accomplished in accordance with Section 6-05 of the 2012 WSDOT *Standard Specifications for Roadway, Bridge, and Municipal Construction* (WSDOT *Standard Specifications)* by the Washington State Department of Transportation (WSDOT 2012). These specifications are typically applied for piles using impact hammers. If foundation piles are installed using a vibratory hammer, axial capacity of the pile should be demonstrated after installation, using either an impact hammer and “re-strike” measurements or static load testing. If load testing is performed, at least one test should be performed at each work area. Test piles should be loaded as described in Section 3.4.

If drilled micropiles are selected for foundations, their final design should be performed by the foundation contractor, whom is in the best position to determine equipment and procedures needed to achieve the required foundation resistance. Final design should be completed under the review of the contractor’s engineer and should be included with a submittal describing installation procedures, materials, and equipment. As described in Section 4.0, we should review these submittals. Test micropiles installed using the selected installation method must be load tested in order to confirm that the required resistance has been achieved. At least one micropile at each work area should be load tested as described in Section 3.4. If a modification to drilling method occurs during construction, additional load tests should be performed to demonstrate the adequacy of the modified installation procedure. We recommend that we review load test results, proposed modifications to installation following testing, and test results following those modifications.

We recommend that the construction and testing of all deep foundations be observed by a qualified geotechnical or civil engineer with experience in these foundation types. The purpose of this observation would be to evaluate the contractor's procedure, to collect and interpret installation data and monitor variations in subsurface conditions, confirm the required penetration depths, and observe “re-strike” or static load tests.

3.3.2 HELICAL PILES

Helical piles (e.g., HEATanchors) or helical piers (e.g., Chance Anchors) could be used to support the proposed bridges. For this project, the cost to install these two types of deep foundation systems will likely be competitive with driven steel pipe piles or drilled micropiles. Helical piles have some advantages over other deep foundation types, including installation using relatively small equipment, no spoils production, and no need for grout mixing. However, they are not typically designed primarily as skin-friction piles in compression, and their small diameter may require many more elements to achieve the required axial and lateral resistances. Because of the installation method and their slender geometry, helical pile/piers are more easily obstructed or deflected by buried objects such as logs or boulders. Also, once installed, the capacity of an individual element cannot normally be increased through grouting, so additional elements, if needed (and as demonstrated by load testing), would require supplemental helical piles. Due to the desire to precast many of the bridge components, including portions of the pile caps, incorporating supplemental foundation elements would result in additional cost and delay.

Helical piles are typically contractor-designed utilizing allowable stress design (ASD) methodologies. The contractor is typically provided the design loads (axial, uplift, and lateral) and the design soil profile. The contractor should be provided with this report, including the borings and laboratory results in Appendices A and B, to complete his design. For lateral resistance, the p-y parameters provided for piles and micropiles may be used for helical piles/piers. As with micropiles, the nominal resistance (including any group reduction) of helical piles/piers must be demonstrated with load testing.

3.4 STATIC LOAD TESTING

The capacity of drilled micropiles and helical piles/piers must be demonstrated by static load testing in compression to at least the nominal (unfactored) required resistance for an individual deep foundation element. Static load testing can also be used in lieu of "re-strike" testing of driven piles. The results of load testing may also be used to estimate the capacity of helical piles/piers during production installation when a torque- to-axial-capacity relationship is established.

Load tests should be completed using the "Quick Load Test Method" described in ASTM D1143. The maximum load in the load test should be the nominal (i.e., "unfactored") load for LRFD design, or 150 percent of the design load for ASD. The pile should sustain the axial design load with no more than 1 inch total vertical movement of the pile head. If more than 1 inch of settlement is measured at the axial design load, construction procedures should be modified and the pile capacity re-tested. Pile load tests should be completed in the presence of an experienced civil or geotechnical engineer who is familiar with the soil conditions at the site and interpreting the results of a pile load test. We would be pleased to offer this service during construction.

3.5 LATERAL LOADS

Below-grade portions of the bridge structure at Work Area 1 will experience lateral loading where soils are retained. Small retained heights may also be necessary at the other work areas. We anticipate that soil to be retained will consist of compacted gravel placed behind the retaining structure following pile cap and wall construction; top-down construction (such as for a soldier pile wall) will not be employed. Due to anticipated strut action of the bridge, restrained rotation of the pile caps, and limited excavation height, we expect that an at-rest lateral condition will exist behind these structures. Lateral earth pressures only need be considered for portions of the foundation located above the ditch invert elevation. We have prepared recommended lateral soil pressures for static and seismic loading against these structures on Figure 8. We understand that final design may eliminate retained soil by adjusting bridge spans.

3.6 CHANNEL SCOUR

The drainage ditches at the project site are subject to minimum flow rates to maintain fish passage, and much higher flow rates during and following a 100-year storm event. Flow rates and cross sectional areas for the drainage ditch were not available at the bridge locations during our review of project documents (WDFW 2011). However, flow velocities may be determined from these data and compared with the permissible flow velocity appropriate for soils at the work areas. Permissible flow velocities for a variety of streambed substrates are tabulated by Fischenich 2001, which is included in Appendix B. At Work Areas 1, 2, and 4, a permissible flow velocity of no more than 2 ft/sec is appropriate (noncolloidal alluvial silt). At Work Area 3, a permissible flow velocity of no more than 1.5 ft/sec is appropriate for the fine sand present in the ditch invert at that location. If higher flow velocities are anticipated, this could be accommodated by either placing an erosion armoring layer of coarser material, such as a gravel/cobble mix, or incorporating bioengineered methods. However, bioengineered methods may not be suitable for permanent or long-term submergence, which occurs at least below the fish passage flow elevation.

3.7 CORROSION POTENTIAL OF SOILS

Two samples were submitted to a local analytical laboratory for chemical testing to assess the overall corrosive potential for soils in the shallow subsurface of the project site. Testing was performed in accordance with the American Water Works Association (AWWA) recommendations for assessing the corrosion potential of soils, including a determination of minimum resistivity, moisture content, pH, redox potential, and sulfide content of both samples. The results of analytical testing are presented in Appendix A.

Using the AWWA, Procedure C105 points ranking system for the above parameters, the soils in the shallow subsurface at the project site have a mildly aggressive to aggressive potential for corrosion of buried elements of the bridge structures. However, additional parameters (sulfate and chloride content) were not determined, the results of which could indicate a greater corrosion potential. The design of steel pipe piles and micropiles is often controlled by installation stresses experienced during driving or drilling, and a greater amount of steel is included for constructability reasons than is necessary to resist post-construction loads. However, if long-term foundation capacity requires it, either additional steel should be provided to counter the loss of steel section due to corrosion, or else epoxy paint, galvanization, or other anti-corrosion measures should be included in design.

4.0 DOCUMENT REVIEW AND CONSTRUCTION OBSERVATIONS

Landau Associates recommends that we review the geotechnical-related portions of the plans and specifications for the proposed project in advance of project bidding. The purpose of the review is to verify that the recommendations presented in this geotechnical report have been properly interpreted and implemented in the design and specifications.

We recommend that monitoring, testing, and consultation be provided during construction to confirm that the conditions encountered are consistent with those indicated by our exploration, to provide expedient recommendations should conditions be revealed during construction that differ from those anticipated, and to evaluate whether geotechnical-related activities comply with project plans and specifications and the recommendations contained in this report. Such activities include pile installation, pile load testing, and other geotechnical-related activities. Landau Associates would be pleased to provide these construction related services.

5.0 USE OF THIS REPORT

This report was prepared for the exclusive use of WDFW for specific application to the proposed Cherry Valley Culvert Replacement Project. The use by others, or for purposes other than intended, is at the user's sole risk. The findings, conclusions, and recommendations presented herein are based on our understanding of the project, review of readily available geologic information in the project vicinity, and on subsurface conditions observed during a field investigation performed on December 12 and 13, 2011. Within the limitations of scope, schedule, and budget, the conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering principles and practices in the area at the time the report was prepared. We make no other warranty, either express or implied.

The conclusions and recommendations contained in this report are based in part upon the subsurface data obtained from the explorations completed for this study. There may be some variation in subsurface soil and groundwater conditions at the site, and the nature and extent of the variations may not become evident until construction. Accordingly, a contingency for unanticipated conditions should be included in the construction budget and schedule.

If variations in subsurface conditions are encountered during construction, Landau Associates should be notified for review of the recommendations in this report, and revision of such if necessary. If there is a substantial lapse of time between submission of this report and the start of construction, or if conditions change due to construction operations at or adjacent to the project site, we recommend that we review this report to determine the applicability of the conclusions and recommendations contained herein.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the bidding and construction phases of the project. If you have any questions or comments regarding the information contained in this report, or if we may be of further service, please call.

LANDAU ASSOCIATES, INC.

Chad T. McMullen, P.E.
Senior Geotechnical Engineer

Steven R. Wright, P.E.
Senior Associate

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CTM/SZW/rgm

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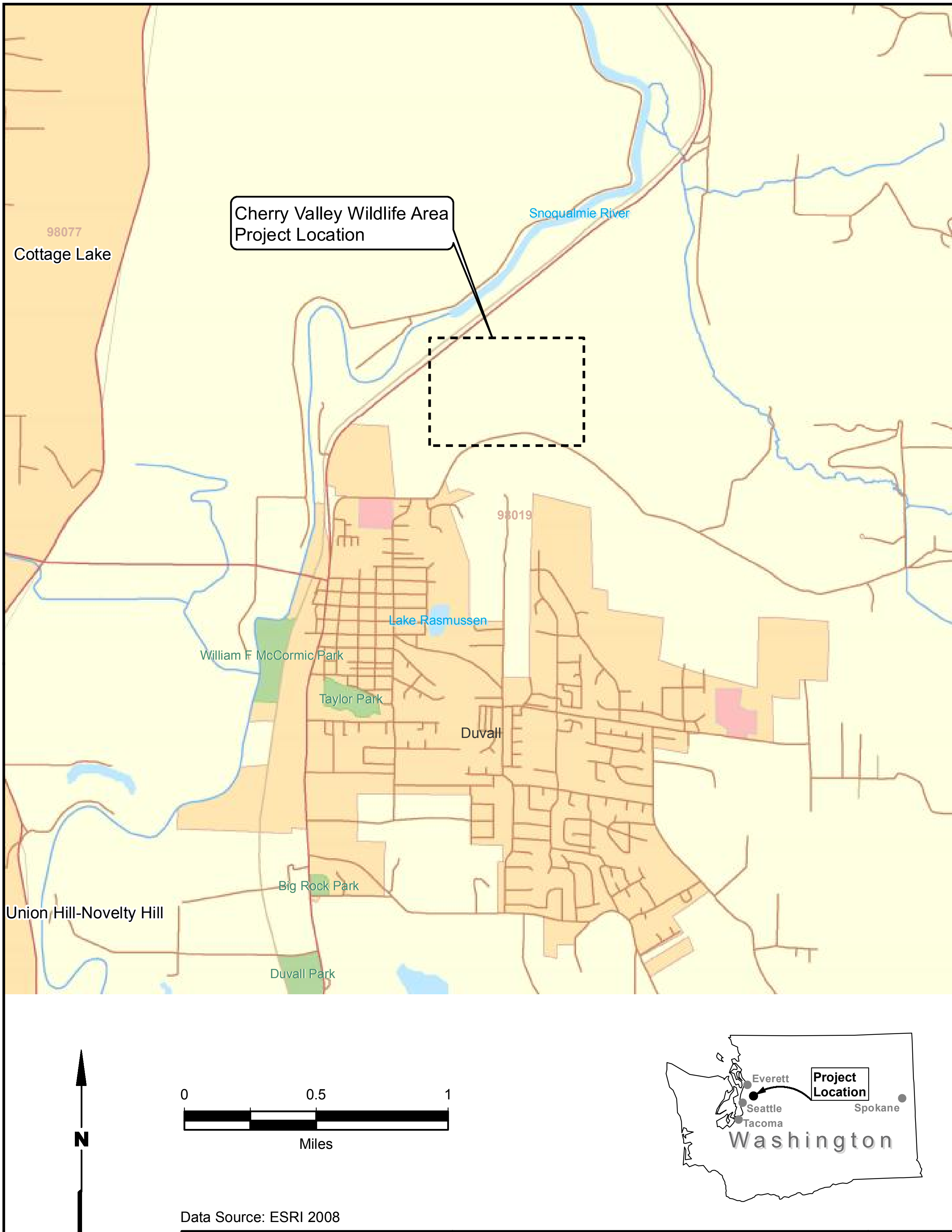

Data Source: ESRI 2008

Y:\Projects\1213006\010.011\Mapdocs\Figure 1.mxd

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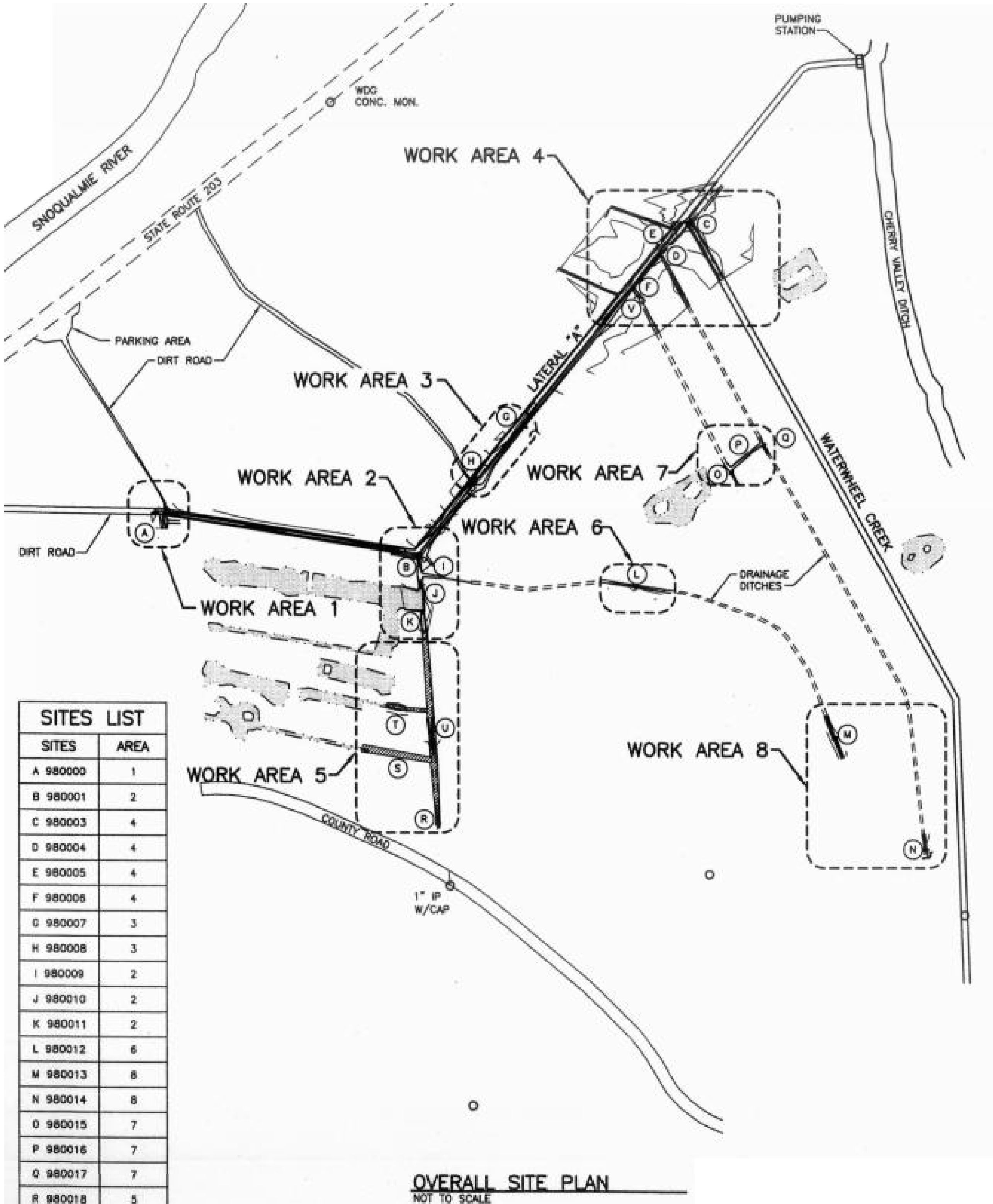
WDFW Cherry Valley Culverts Replacement Project Duvall, Washington	**Vicinity Map**	Figure 1



OVERALL SITE PLAN
NOT TO SCALE

SITES LIST	
SITES	AREA
A 980000	1
B 980001	2
C 980003	4
D 980004	4
E 980005	4
F 980006	4
G 980007	3
H 980008	3
I 980009	2
J 980010	2
K 980011	2
L 980012	6
M 980013	8
N 980014	8
O 980015	7
P 980016	7
Q 980017	7
R 980018	5
S 980019	5
T 980020	5
U 980021	5
V 980071	4

TOTAL CUT/FILL QUANTITIES		
TOTAL CUT	TOTAL FILL	TOTAL NET
−3609.92cy	269.02cy	−3340.90cy

Source: Washington Department of Fish & Wildlife 10/31/11

LANDAU ASSOCIATES, INC. | V:\1213\006\010.011\CV_workareas.dwg (A) "Figure 2" 3/15/2012

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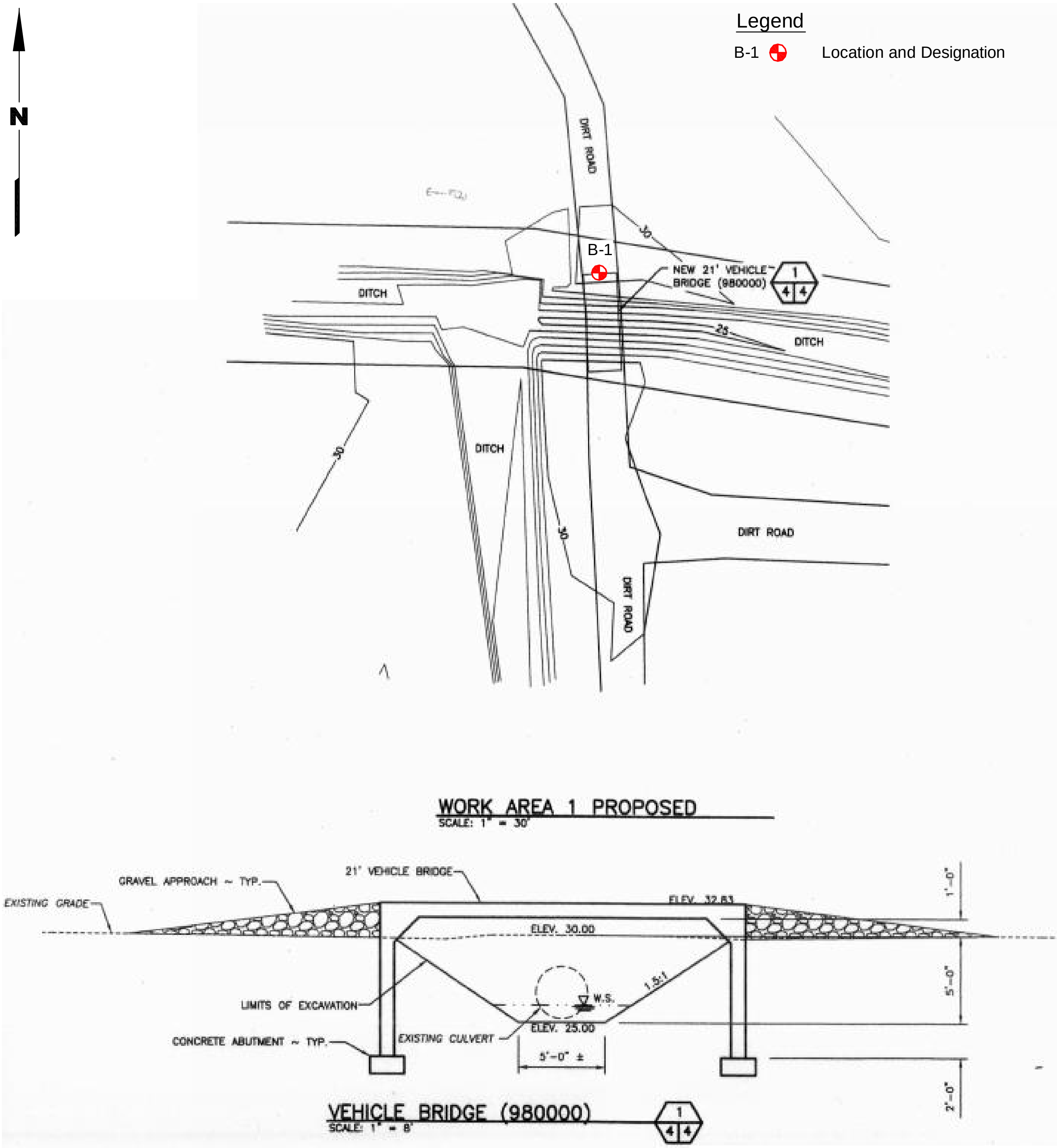
WDFW Cherry Valley Culverts Replacement Project Duvall, Washington	**Work Area Locations**	Figure 2

Legend

B-1 Location and Designation



QUANTITIES			
SITE #	CUT	FILL	NET
980000	-55.58cy	8.80cy	-46.78cy

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

Source: Washington Department of Fish & Wildlife 10/31/11

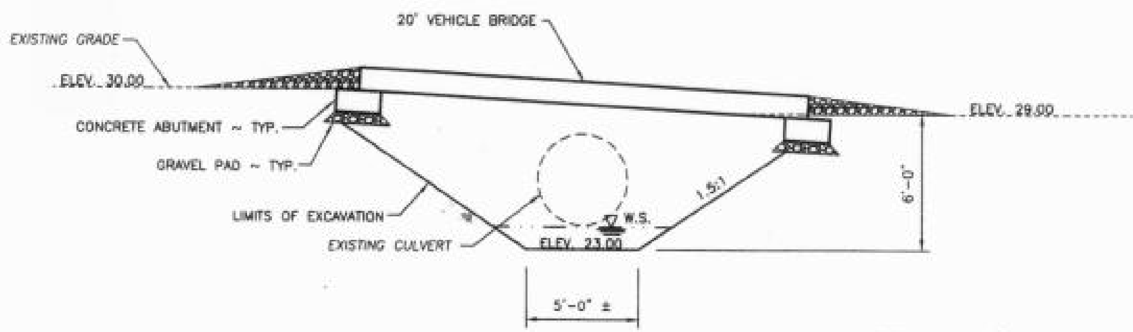
LANDAU ASSOCIATES, INC. | V:\1213\006\010.011\CV_workareas.dwg (A) "Figure 3" 3/15/2012

LANDAU ASSOCIATES

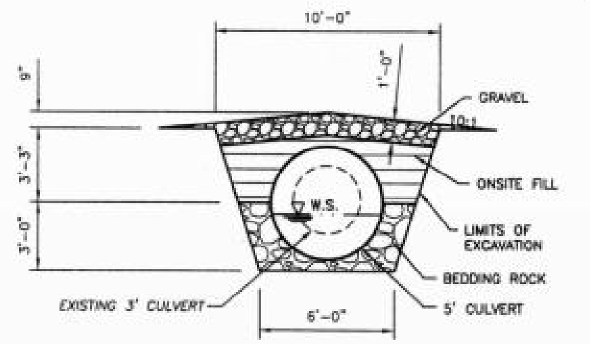
WDFW Cherry Valley Culverts Replacement Project
Duvall, Washington

Work Area 1 Plan and Section

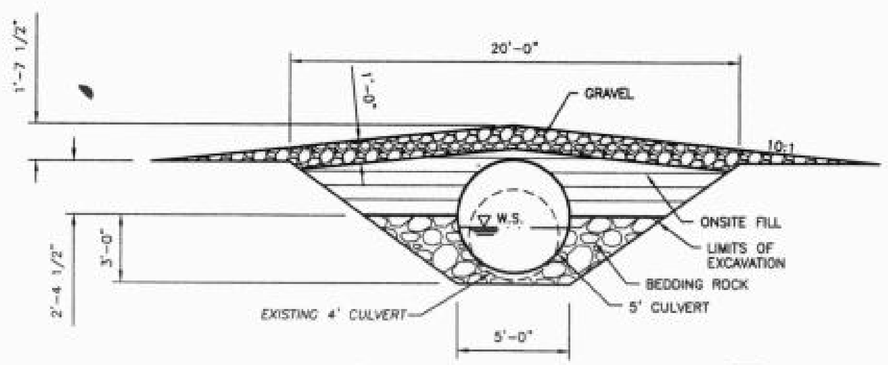
Figure
3



PEDESTRIAN FOOTBRIDGE (980001)
SCALE: 1" = 8'

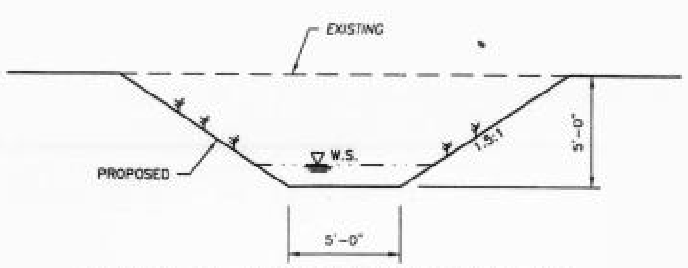


5' CULVERT (980009)
SCALE: 1" = 8'



5' CULVERT (980011)
SCALE: 1" = 8'

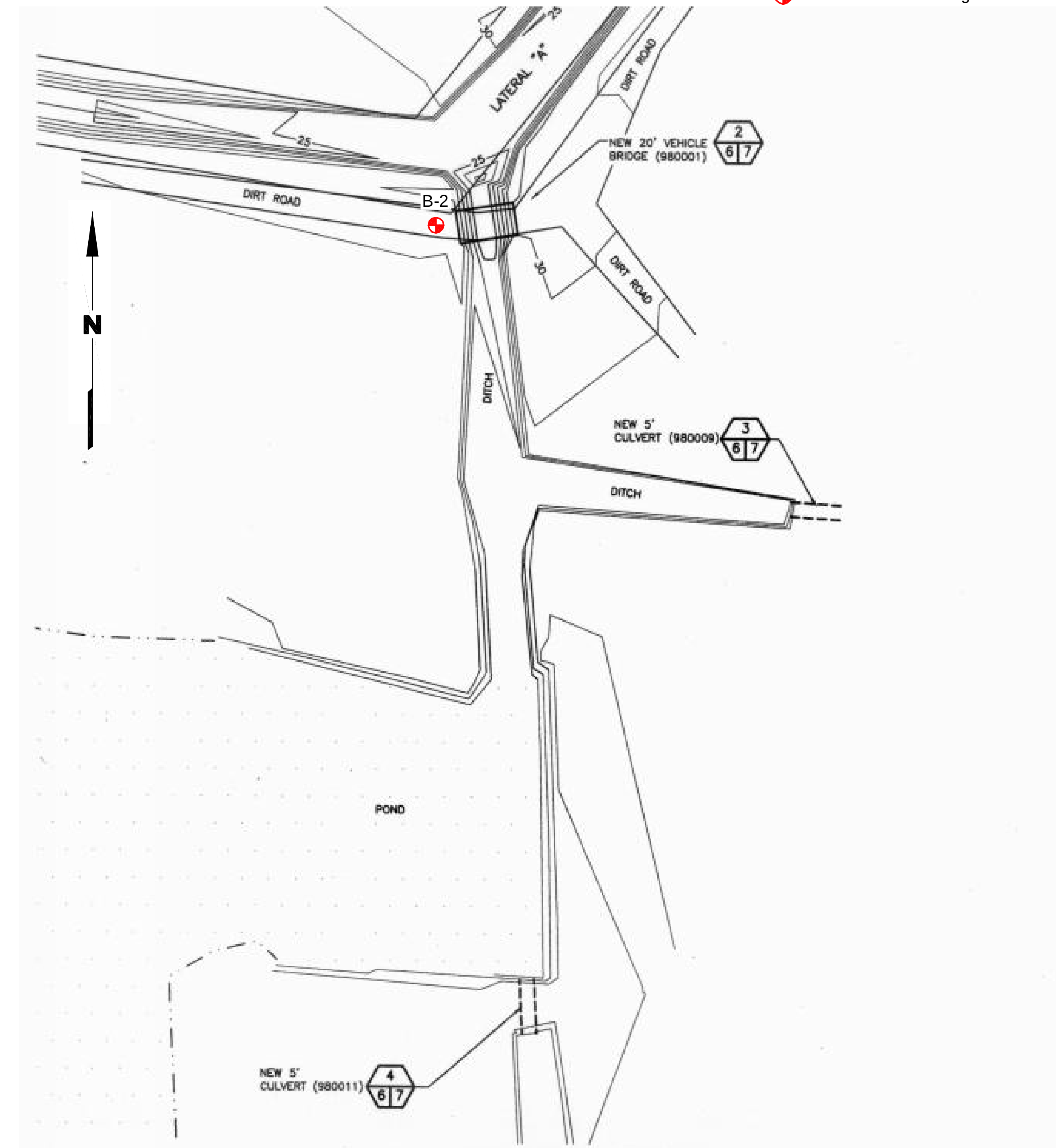
QUANTITIES			
SITE #	CUT	FILL	NET
980001	−75.40cy	3.91cy	−71.49cy
980009	−25.91cy	25.50cy	−.41cy
980010	−90.58cy	0cy	−90.58cy
980011	−38.04cy	54.11cy	+16.07cy



TYPICAL EXCAVATION DETAIL (980010)
SCALE: 1" = 8'

Legend

B-2 Location and Designation



WORK AREA 2 PROPOSED
SCALE: 1" = 50'

Note

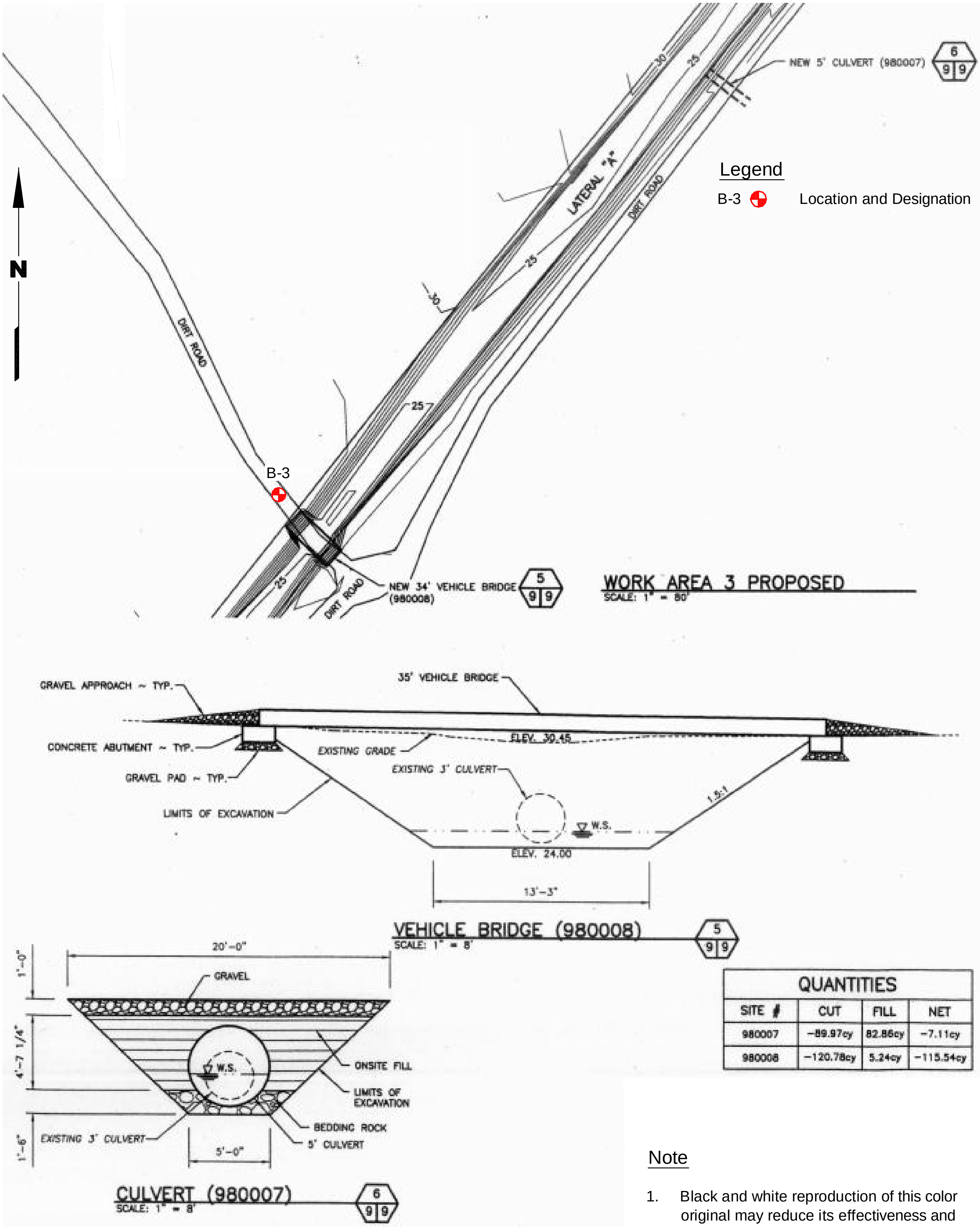
1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

Source: Washington Department of Fish & Wildlife 10/31/11

LANDAU ASSOCIATES, INC. | V:\1213\006\010.011\CV_workareas.dwg (A) "Figure 4" 3/15/2012

LANDAU ASSOCIATES

WDFW Cherry Valley Culverts Replacement Project Duvall, Washington	**Work Area 2 Plan and Sections**	Figure 4



QUANTITIES

SITE #	CUT	FILL	NET
980007	−89.97cy	82.86cy	−7.11cy
980008	−120.78cy	5.24cy	−115.54cy

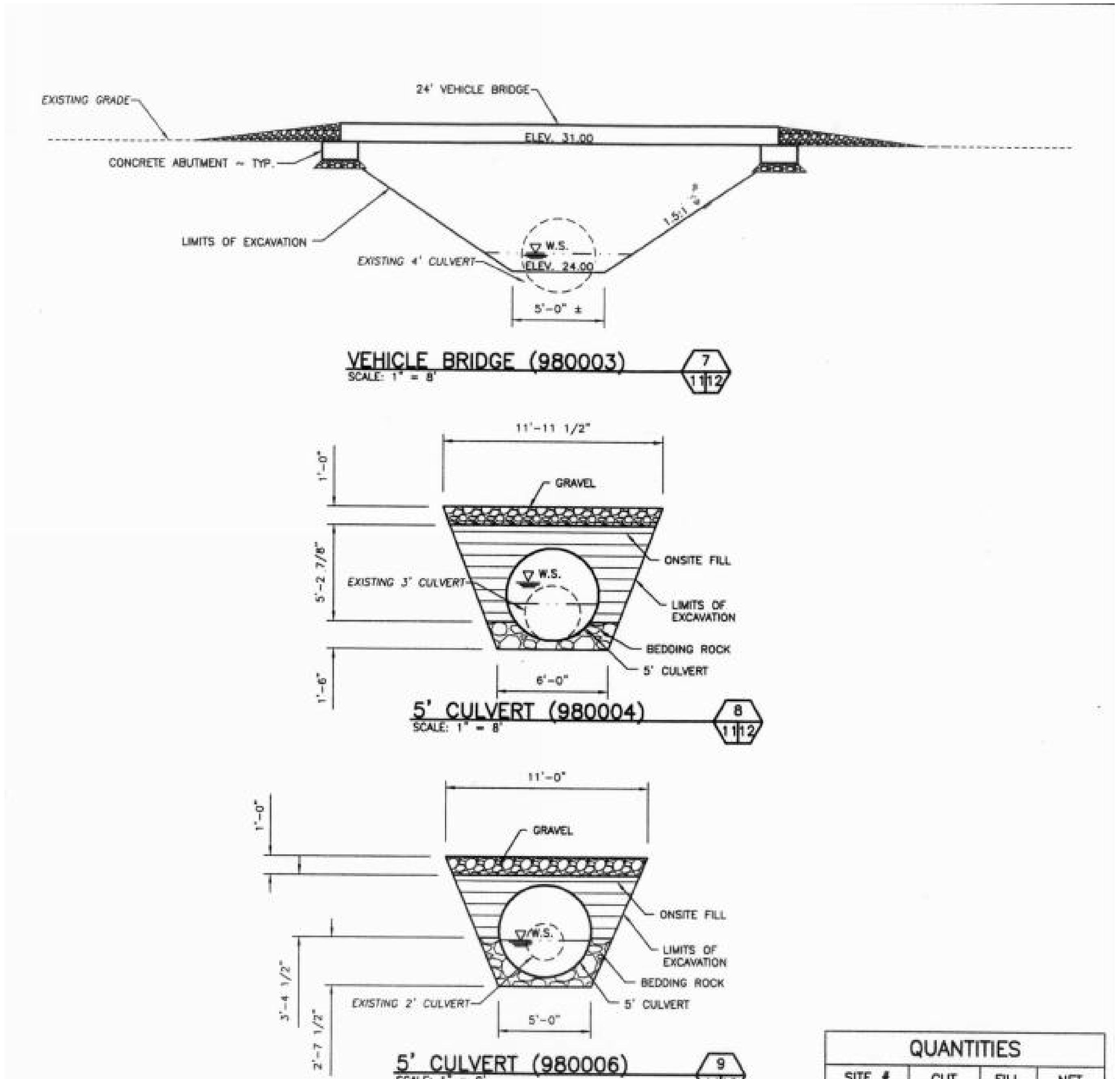
Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

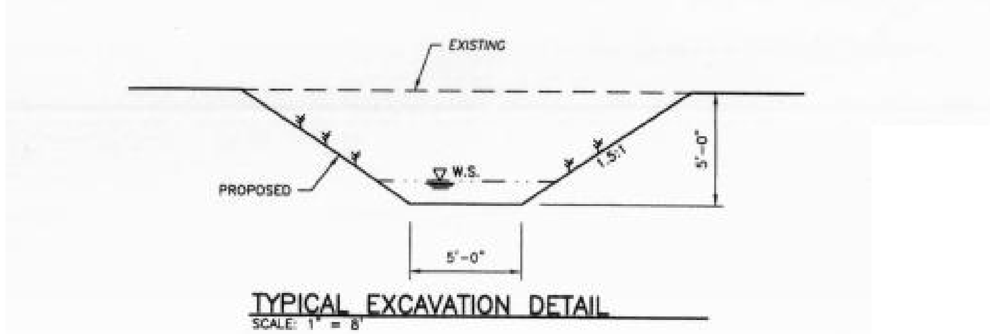
Source: Washington Department of Fish & Wildlife 10/31/11

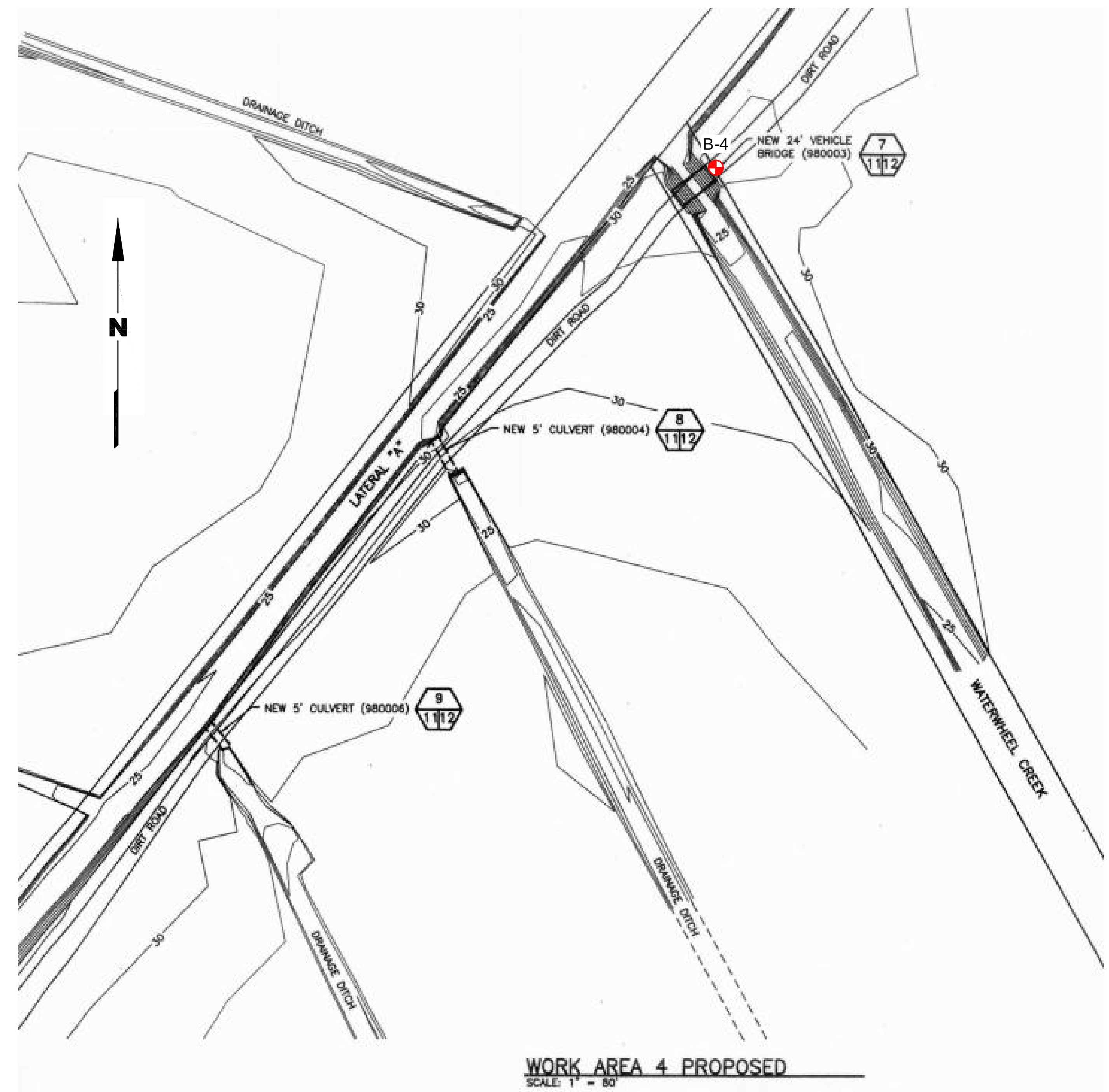
LANDAU ASSOCIATES

WDFW Cherry Valley Culverts Replacement Project Duvall, Washington	**Work Area 3 Plan and Section**	Figure 5



QUANTITIES			
SITE #	CUT	FILL	NET
980003	−113.55cy	5.82cy	−107.73cy
980004	−42.33cy	38.84cy	−3.49cy
980005	−37.11cy	0cy	−37.11cy
980006	−31.87cy	25.83cy	−6.04cy
980071	−36.94cy	0cy	−36.94cy





Legend

B-4 Location and Designation

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

Source: Washington Department of Fish & Wildlife 10/31/11

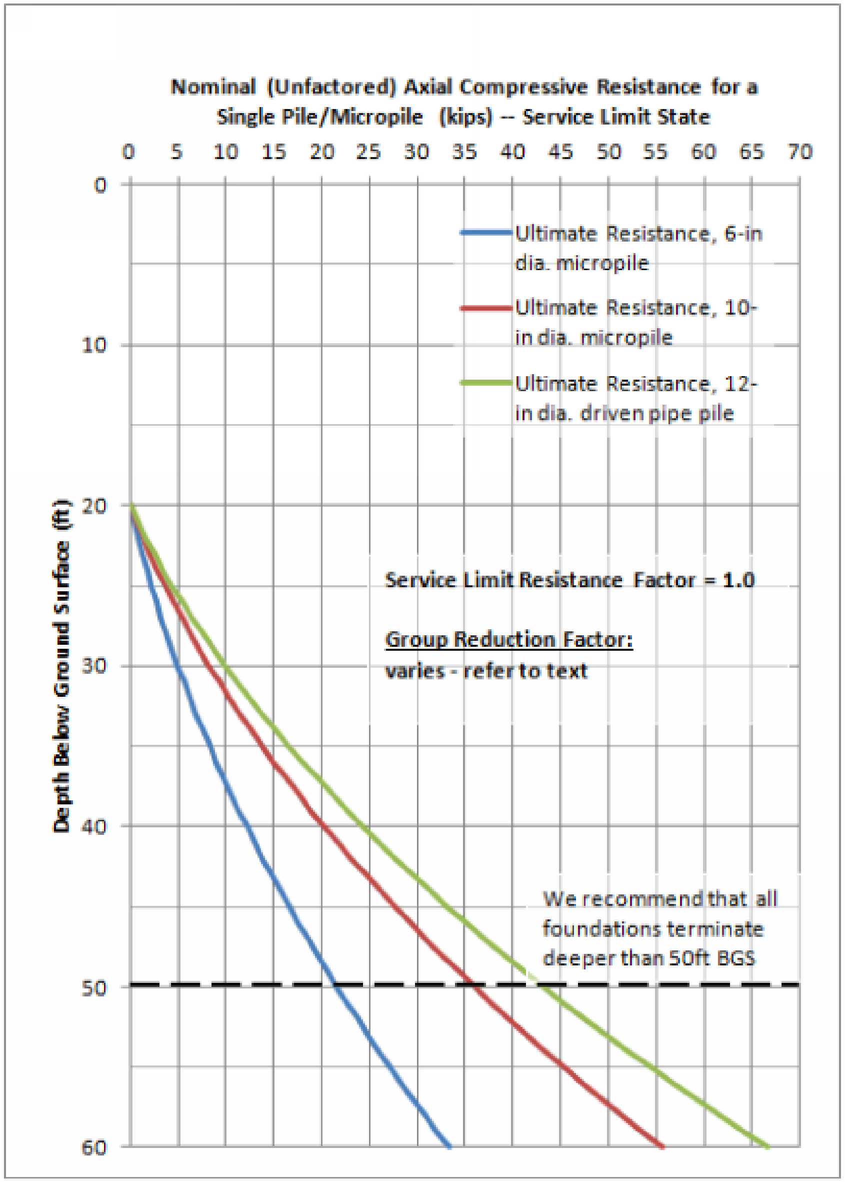
WDFW Cherry Valley Culverts Replacement Project
Duvall, Washington

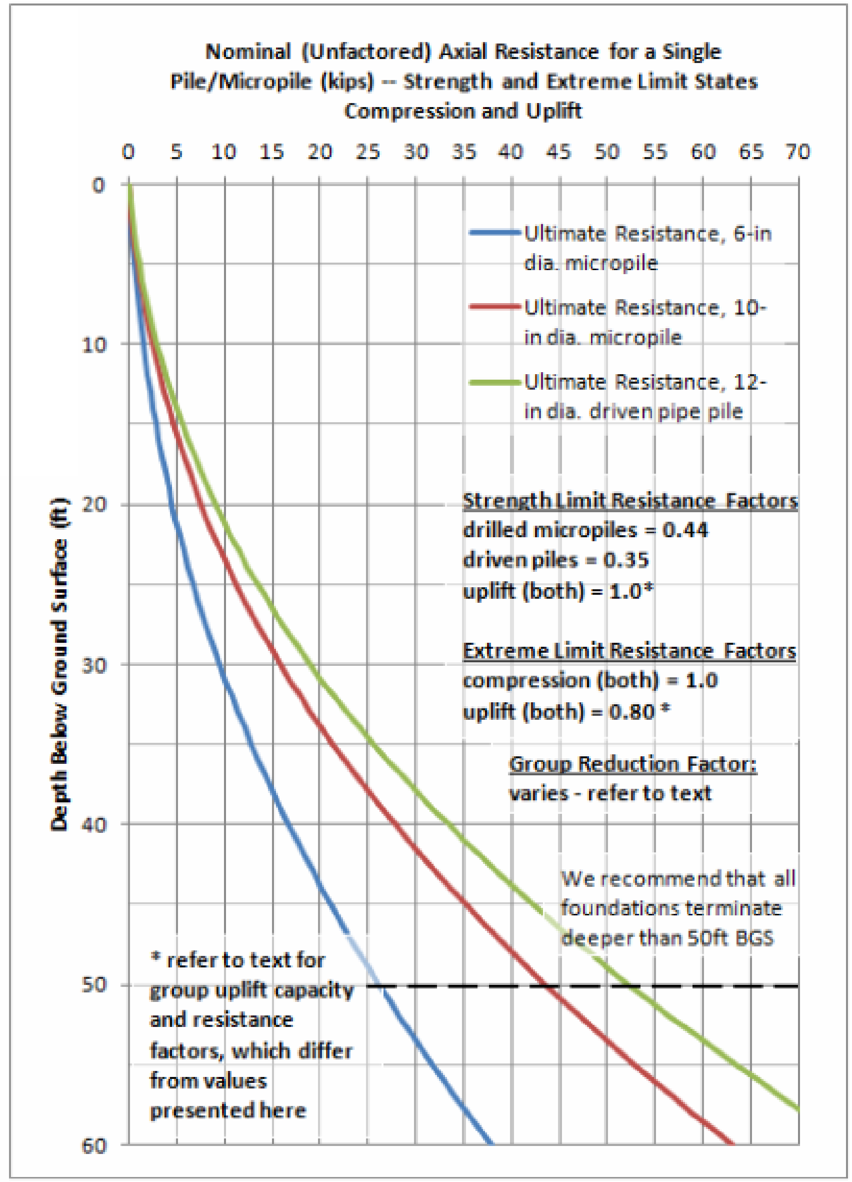
Work Area 4 Plan and Sections

Figure 6

LANDAU ASSOCIATES

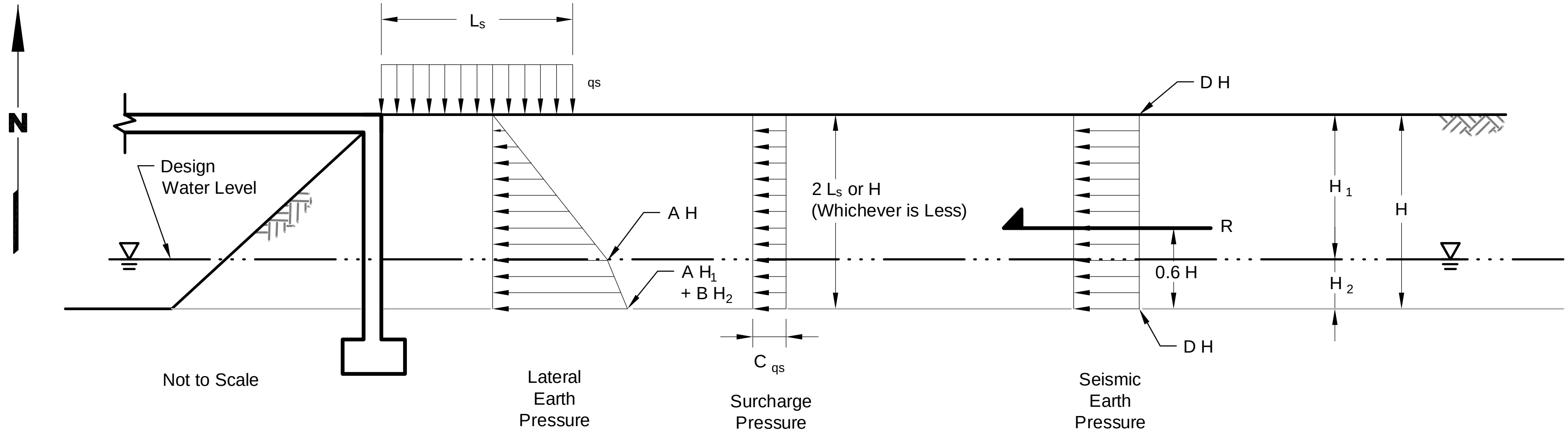
LANDAU ASSOCIATES, INC. | V:\1213\006\010.011\CV_workareas.dwg (A) "Figure 6" 3/15/2012





LANDAU ASSOCIATES

WDFW Cherry Valley Culvert Replacement Project Duvall, Washington	6-, 10-, & 12-inch Diameter Pile/Micropile Capacity	Figure 7



Notes

1. The below grade wall for Work Area 1 should be designed in accordance with the 2010 AASHTO LRDGE Bridge Design Specifications (AASTH 2010).
2. All earth pressures are presented in terms of pounds per square foot (psf), with all dimensions in feet.
3. Lateral earth pressure, surcharge pressure, and seismic earth pressure are dependent on the ability of the wall to rotate or translate. If the wall is able to rotate or translate at least 0.001H, where H is the height of the wall, active conditions are assumed to develop. If the wall is unable to rotate or translate, at-rest conditions should be assumed in the design. The coefficients A, B, C, and D provided on the figure are dependent on whether active or at-rest conditions develop. See table for value of coefficients.
4. The design water level should be set at the 100-year flood water level. It is assumed that free draining material will be provided behind the wall such that uneven hydrostatic earth pressures do not develop.
5. It is typical practice to accommodate traffic and construction equipment with a uniform surcharge pressure, qs (psf), of 250 psf over the distance, Ls (ft). Large construction equipment or large surcharge loads should be addressed by use of a higher surcharge pressure.
6. The earth pressure, surcharge pressure, and seismic earth pressure can be assumed to act over the width of the below-grade wall.
7. Resistance to lateral loads will be provided by the pile foundations. Passive pressure should be neglected in the design of the below-grade wall.
8. A load factor of 1.0 is included in the earth pressure, surcharge pressure, and seismic earth pressure.

	Active Conditions	At-Rest Conditions
A	35	55
B	19	30
C	0.26	0.41
D	6	15

LANDAU ASSOCIATES

Cherry Valley Culvert Replacement Project King County, Washington	**Lateral Earth Pressures**	Figure 8

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface conditions within the limits of the project area were explored on December 12 and 13, 2011. The exploration program consisted of advancing and sampling one exploratory boring at each of the four bridge locations (borings B-1 through B-4) at the approximate locations shown on the Work Area Plans (Figures 3 through 6). The exploratory borings were advanced to depths ranging from about 29 to 49 ft below existing ground surface (BGS) using a track-mounted drill rig and the open-hole, mud-rotary drilling method. Holocene Drilling, Inc. of Puyallup, Washington advanced the borings under subcontract to Landau Associates. The explorations were located in the field by pacing distances from existing features referenced on a site plan provided by the Washington State Department of Fish and Wildlife. The approximate ground surface elevations at the exploratory boring locations were not determined.

The field exploration program was coordinated and monitored by a geotechnical engineer from our staff, who also obtained representative soil samples, maintained a detailed record of the observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed in our exploratory borings was described using the soil classification system shown on Figure A-1, in general accordance with ASTM D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Logs of the exploratory borings are presented on Figures A-2 through A-5. These logs represent our interpretation of subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times. A further discussion of the soil and groundwater conditions observed is contained in the text portion of this report.

Most soil samples were obtained using a 2-inch outside-diameter (O.D.) split-spoon sampler and the Standard Penetration Test (SPT) procedure (ASTM D1586). Selected samples were collected using either a larger, 3.0-inch OD split-spoon sampler (for additional collected sample volume) or a 3.0-inch O.D. thin-wall sampler. Split-spoon samplers were driven 18 inches using a 140-pound automatic trip hammer falling from a height of 30 inches. The number of blows required to drive the sampler for each 6-inch interval of the 18-inch drive was recorded on the field log. The number of blows required to drive the sampler the last 12 inches of the 18-inch drive is termed the Standard Penetration Resistance and is shown on the summary log. Thin-wall samples were obtained using a piston sampler. Upon completion of drilling and sampling, the boreholes were abandoned in accordance with the requirements of WAC 173-160.

GEOTECHNICAL LABORATORY TESTING

The samples were checked against the field log descriptions, which were updated where appropriate in general accordance with ASTM D2487, *Standard Test Method for Classification of Soils for Engineering Purposes*. Natural moisture content determinations were performed in general accordance with ASTM D2216 on soil samples obtained from the borings. The natural moisture content is shown as “W=xx” (percent of dry weight) at the respective sample depth in the column labeled “Test Data” on the boring logs. Atterberg Limit determinations were performed on representative soil samples obtained from the borings in accordance with ASTM D4318. Samples selected for Atterberg Limits are designated with an “AL” in the column labeled “Test Data” on the boring logs. The results of the Atterberg Limits are shown on Figure A-6 in this appendix. 1-dimensional consolidation testing was performed on one thin-wall sample obtained from boring B-2 at a depth of about 23 ft, in accordance with the incremental loading procedure in ASTM D 2435. The purpose of consolidation testing was to determine the compressibility of soils within the foundation bearing layer, in order to complete settlement calculations. The sample selected for consolidation testing is denoted by “Consol” in the column labeled “Test Data” on Figure A-3. Results of consolidation testing are presented on Figure A-7.

ANALYTICAL LABORATORY TESTING

Two samples were submitted to Analytical Resources Incorporated (ARI) of Tukwila, Washington for corrosion and sulfide testing. Samples submitted were obtained from the shallow subsurface at Work Areas 1 and 4. A suite of corrosion tests in accordance with American Water Works Association (AWWA) procedure C105 was performed, which includes a determination of the minimum resistivity, moisture content, pH, and redox potential of the soil samples. Sulfide content determinations were made on both specimens. Corrosion suite and sulfide content test results are presented in a packet from ARI following Figure A-7.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL(1)	TYPICAL DESCRIPTIONS (2)(3)
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

Notes:
1. USCS letter symbols correspond to symbols used by the Unified Soil Classification System and ASTM classification methods. Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil descriptions are based on the general approach presented in the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the Standard Test Method for Classification of Soils for Engineering Purposes, as outlined in ASTM D 2487.
3. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50%	- "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50%	- "very gravelly," "very sandy," "very silty," etc.
	> 15% and ≤ 30%	- "gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 15%	- "with gravel," "with sand," "with silt," etc.
	≤ 5%	- "with trace gravel," "with trace sand," "with trace silt," etc., or not noted.

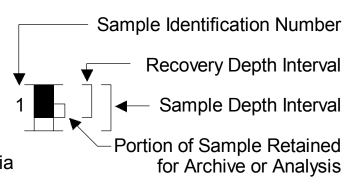
4. Soil density or consistency descriptions are based on judgement using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.

Drilling and Sampling Key

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Single-Tube Core Barrel
f	Double-Tube Core Barrel
g	2.50-inch O.D., 2.00-inch I.D. WSDOT
h	3.00-inch O.D., 2.375-inch I.D. Mod. California
i	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Vibrocore (Rotosonic/Geoprobe)
5	Other - See text if applicable

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

▽ Approximate water level at time of drilling (ATD)

▼ Approximate water level at time other than ATD

LANDAU ASSOCIATES

WDFW Cherry Valley
King County, Washington

Soil Classification System and Key

Figure A-1

B-1

LAI Project No: 1213006.010

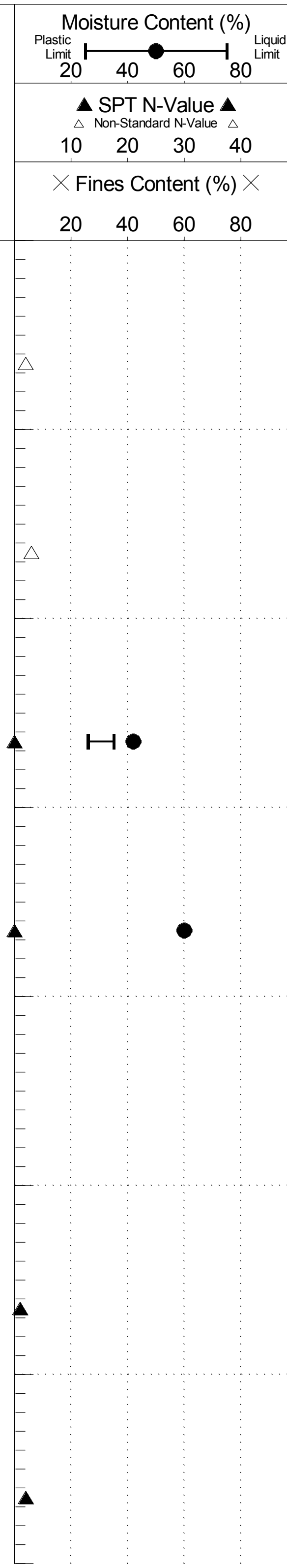
SAMPLE DATA / SOIL PROFILE

Drilling Method: Mud Rotary
Ground Elevation (ft): Not determined
Drilled By: Holocene Drilling Inc.
Logged By: CTM Date: 12/13/11

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
~3	S-1	h2	2	Corrosion Sulfides
~8	S-2a S-2b	h2	3	
~13	S-3	b2	0	W = 42 AL
~18	S-4	b2	0	W = 60
~23	S-5	c3		
~28	S-6	b2	1	
~33	S-7	b2	2	

Depth (ft)	USCS Symbol	Description
0	SM	Brown, gravelly, silty SAND (loose, moist) **(FILL)**
~1.5	OH	Brown, organic-rich, clayey SILT **(ALLUVIUM)**
5	CH/OH	Gray-brown, silty, organic-rich CLAY, numerous fine to coarse plant fibers (very soft, wet)
~8	OH	Brown, silty organic CLAY, numerous fine to coarse fibers (very soft, wet) -- gray silty fine sand in sampler tip
10	ML	Gray, clayey SILT, trace of fine sand, scattered organics
15	MH	Gray, clayey SILT with occasional organics throughout (very soft, wet) -- no sample recovery; gray cuttings per driller
30	MH/OH	Closely laminated (distorted), gray-brown and brown, organic-rich clayey SILT, fine organics throughout (soft, wet)

Groundwater: ∇ (estimated)



Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

1213006.01 3/14/12 \\TACOMA1\DATA\DATA\PROJECTS\1213006.010 CHERRY VALLEY\T1213006.010.011.GPJ SOIL BORING LOG WITH GRAPH

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WDFW Cherry Valley
King County, Washington

Log of Boring B-1

Figure A-2 (1 of 2)

B-1

LAI Project No: 1213006.010

Drilling Method: Mud Rotary
Ground Elevation (ft): Not determined
Drilled By: Holocene Drilling Inc.
Logged By: CTM Date: 12/13/11

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Description
35						SM	Gray, very silty fine SAND, scattered fine organics (loose, wet)
		S-8	b2	5			
40						SP-SM	Gray, fine to medium SAND with silt, trace of fine gravel (medium dense, wet)
		S-9	b2	21			
45						MH	Gray, clayey SILT with trace fine mica flakes (very soft, wet)
		S-10	b2	0	W = 41		

Boring Completed 12/13/11
Total Depth of Boring = 49.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

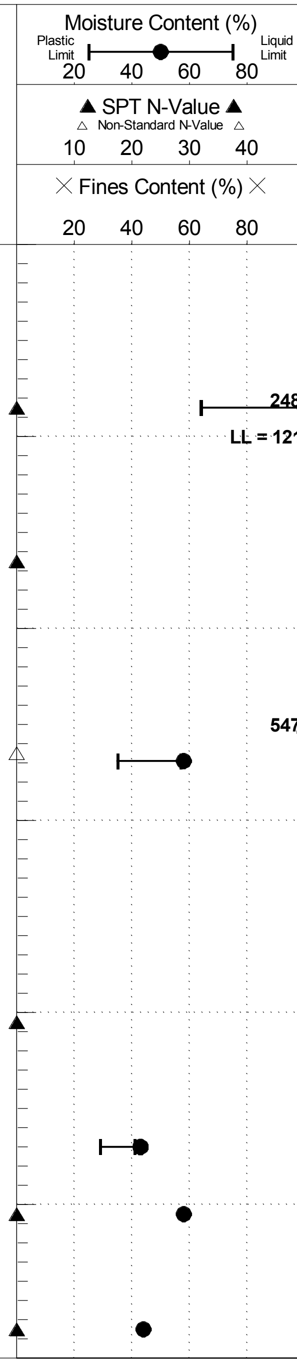
B-2

LAI Project No: 1213006.010

Drilling Method: Mud Rotary
Ground Elevation (ft): Not determined
Drilled By: Holocene Drilling Inc.
Logged By: CTM Date: 12/13/11

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile
0					SM	Brown, gravelly, silty SAND (loose, moist) **(FILL)**
					WD	Fresh wood, cedar odor, suspect log covered by fill
						-- skip sampling at 2.5 ft
	S-1	b2	0	W = 248 AL	MH/ OH	Brown, organic-rich clayey SILT, fibrous, with scattered wood fragments (very soft, wet) **(ALLUVIUM)**
	S-2	b2	0			-- no recovery, cuttings are similar to above
10					PT	Brown, silty PEAT, fibrous (very soft, wet)
	S-3A S-3B	h2	0	W = 547 W = 58 AL	MH/ OH	Gray-brown, organic clayey SILT, fibrous (very soft, wet)
						-- per driller, gray at 15.5ft
	S-4	c3				-- no recovery
20	S-5	b2	0			-- alternating bands of brown, clayey PEAT and gray-brown, clayey organic SILT (very soft, wet)
					ML	Gray, clayey SILT with occasional organic fibers, trace of fine sand
	S-6	c3		Consol W = 43 W = 43 AL		
25	S-7	b2	0	D = 77 W = 97 W = 58		
	S-8	b2	0	W = 44		

Groundwater: (estimated)



Boring Completed 12/13/11
Total Depth of Boring = 29.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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WDFW Cherry Valley
King County, Washington

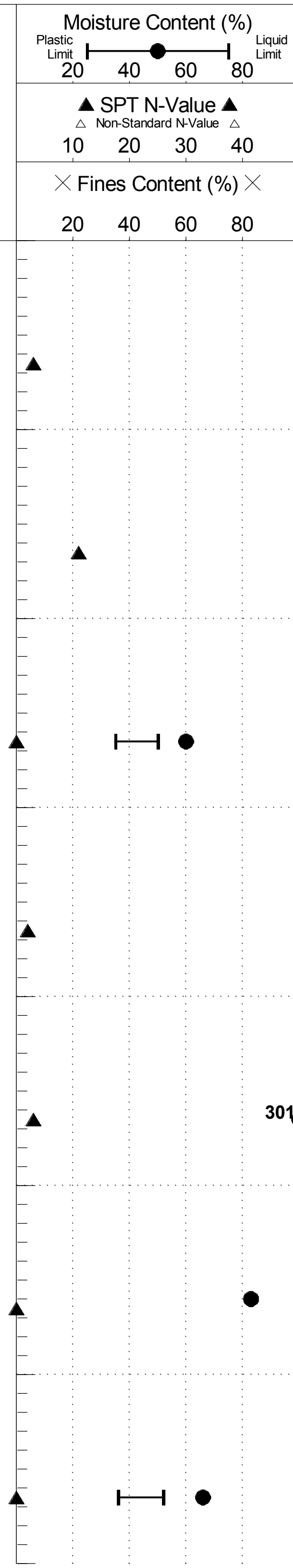
Log of Boring B-2

Figure A-3

B-3

LAI Project No: 1213006.010

SAMPLE DATA						SOIL PROFILE			
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Mud Rotary; Ground Elevation (ft): Not determined; Drilled By: Holocene Drilling Inc.; Logged By: CTM; Date: 12/12/11	Groundwater
0							SM	Brown, gravelly, silty SAND (loose, moist) **(FILL)**	
							ML/ MH	Gray, clayey SILT with organics (soft, wet) **(ALLUVIUM)**	(estimated)
		S-1	b2	3					
5							SP-SM	Gray, fine SAND, trace of silt, occasional wood debris (medium dense, wet)	
		S-2	b2	11					
10								-- slight gravelly drill action at 10 - 11 ft BGS	
							ML/ MH	Gray, clayey SILT with fine organic fibers throughout (very soft, wet)	
		S-3	b2	0	W = 60 AL				
15							OH	Dark brown to brown, very organic-rich SILT, numerous fibers (very soft, wet)	
		S-4	b2	2					
20									
		S-5	b2	3	W = 301			-- 1/2-inch band of gray silt at sample top	
25									
		S-6	b2	0	W = 83		MH	Gray, clayey SILT, scattered to occasional organic fibers and wood debris (very soft, wet)	
30									
		S-7	b2	0	W = 66 AL				
35									



Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

1213006.01 3/14/12 \\TACOMA1\DATA\DATA\PROJECT\1213006.010\T\1213006.010.011.GPJ SOIL BORING LOG WITH GRAPH

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WDFW Cherry Valley
King County, Washington

Log of Boring B-3

Figure A-4 (1 of 2)

B-3

LAI Project No: 1213006.010

SAMPLE DATA						SOIL PROFILE			Groundwater	Moisture Content (%) / SPT N-Value / Fines Content (%)
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Mud Rotary; Ground Elevation (ft): Not determined; Drilled By: Holocene Drilling Inc.; Logged By: CTM; Date: 12/12/11		
35							ML	Gray, fine sandy, SILT with organics and wood debris (very soft to soft, wet)		
		S-8	b2	2						
40							MH	Gray, clayey SILT with occasional fine organics (very soft, wet)		
		S-9	b2	0						
45										
		S-10	b2	0	W = 59					

Moisture Content (%): Plastic Limit, Liquid Limit; 20 40 60 80

▲ SPT N-Value ▲; △ Non-Standard N-Value △; 10 20 30 40

× Fines Content (%) ×; 20 40 60 80

50

Boring Completed 12/12/11
Total Depth of Boring = 49.0 ft.

55

60

65

70

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

B-4

LAI Project No: 1213006.010

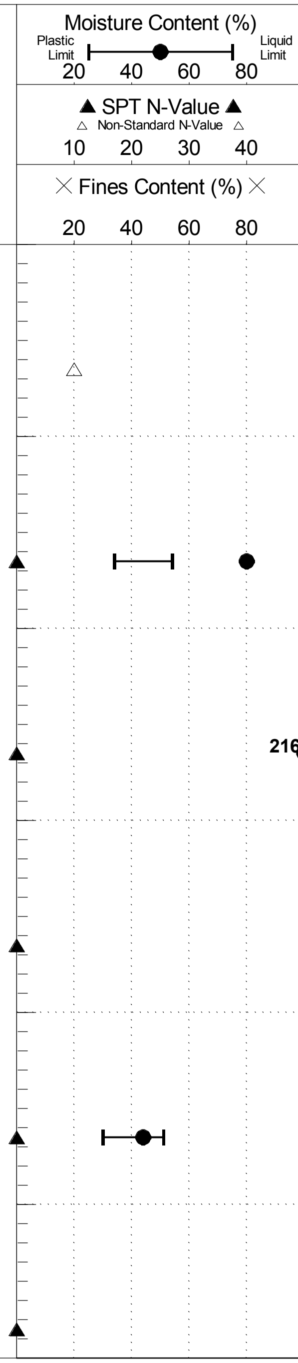
SAMPLE DATA / SOIL PROFILE

Drilling Method: Mud Rotary
Ground Elevation (ft): Not determined
Drilled By: Holocene Drilling Inc.
Logged By: CTM   Date: 12/12/11

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
~3		S-1	h2	10	Corrosion Sulfides
~8		S-2	b2	0	W = 80 AL
~13		S-3	b2	0	W = 216
~18		S-4	b2	0	
~23		S-5	b2	0	W = 44 AL
~28		S-6	b2	0	

Depth (ft)	USCS Symbol	Description
0	SM	Brown, gravelly, silty SAND (loose, moist) **(FILL)**
~1.5	ML	Dark brown with dark-gray, clayey SILT with sand and fine gravel (medium stiff, moist) **(ALLUVIUM)**
5	MH	Gray, clayey SILT, trace of fine sand, scattered fine organics (very soft, wet) -- fibrous at top of sample
10	OH	Brown, organic SILT, fibrous (very soft, wet)
20	MH	Gray, clayey SILT, occasional organic fiber mats (very soft, wet)

Groundwater: ▽ (estimated) at approximately 3.5 ft



Boring Completed 12/12/11
Total Depth of Boring = 29.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

1213006.01 3/14/12 \\TACOMA1\DATA\DATA\PROJECT\1213006.010 CHERRY VALLEY\T1213006.010.011.GPJ SOIL BORING LOG WITH GRAPH

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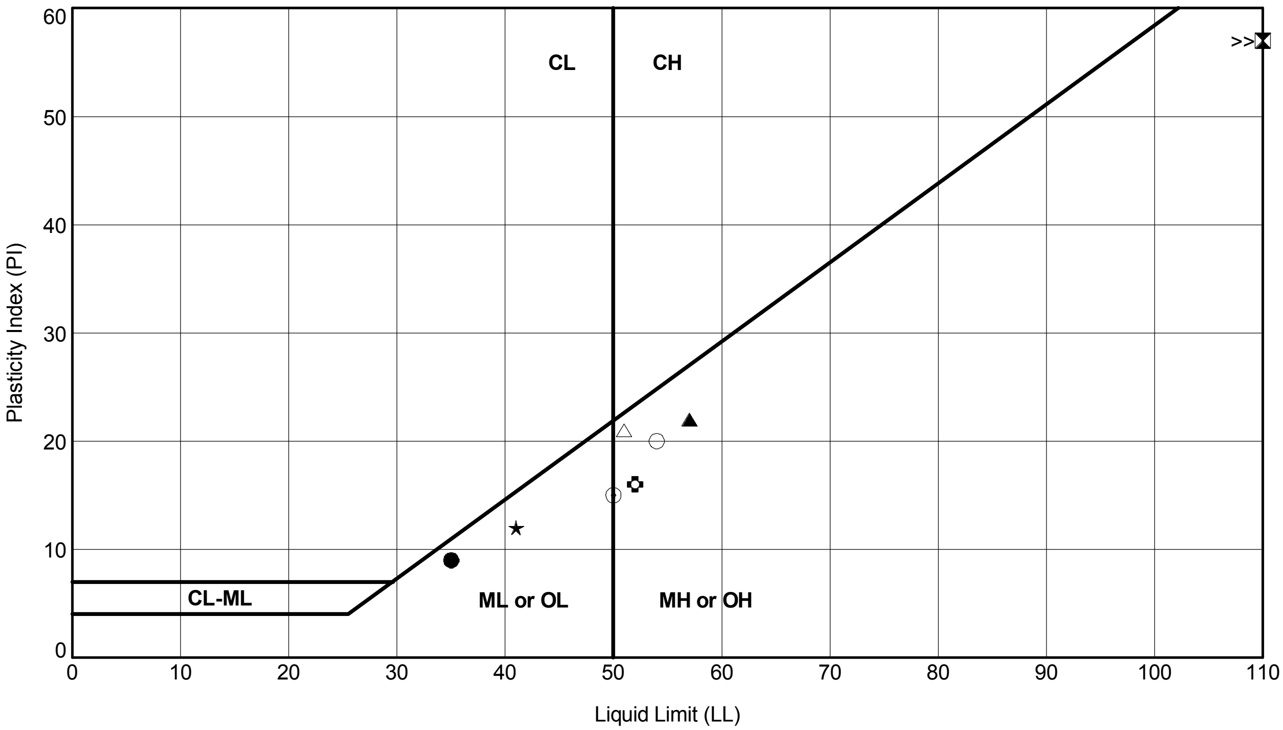
WDFW Cherry Valley King County, Washington	Log of Boring B-4	Figure A-5



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-3	12.5	35	26	9	42	Gray, clayey SILT with occasional organics	ML
⧗	B-2	S-1	3.5	121	64	57	248	Brown, organic-rich, clayey SILT	MH/OH
▲	B-2	S-3B	12.9	57	35	22	58	Gray-brown, organic-rich, clayey SILT	MH/OH
★	B-2	S-6	23.5	41	29	12	43	Gray, clayey SILT with occasional organics	ML
⊙	B-3	S-3	12.5	50	35	15	60	Gray, clayey SILT with fine organics	ML/MH
✿	B-3	S-7	32.5	52	36	16	66	Gray, clayey SILT with occasional organics	MH
○	B-4	S-2	7.5	54	34	20	80	Gray, clayey SILT, trace of sand, scatt. organics	MH
△	B-4	S-5	22.5	51	30	21	44	Gray, clayey SILT	MH

ASTM D 4318 Test Method

1213006.01 3/14/12 \\TACOMA1\DATA\DATA\PROJECT\1213\006.010 CHERRY VALLEY\T\1213006.010.011.GPJ ATTERBERG LIMITS FIGURE

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WDFW Cherry Valley
King County, Washington

Plasticity Chart

Figure
A-6

Consolidation Test
Consolidation Specimen Information

Project: WDFW Cherry Creek
Location: Boring B-2, Sample S-6
Job Number: 1213006.010.011

Project Number: 1213006.010.011

Test Date: 12-19-2011

Sample Number: S-6
Boring Number: B-2
Depth: 23.8
Sample Type: Undisturbed

Sample Description:
Gray, clayey SILT, scattered fine organics
Remarks:
Boring B-2, Sample S-6

Test Number:
Liquid Limit: 41
Plastic Limit: 29
Specific Gravity: 2.65 Assumed

Initial Void Ratio: 1.155
Plasticity Index (%): 12
Weight of Ring (g): 109.7

Initial Height (in): 1.000
Initial Diameter (in): 2.500

Parameters	Initial Specimen	Final Specimen
Moist Weight + Container (g)	209.20	450.18
Dry Soil + Container (g)	174.69	413.58
Weight of Container (g)	93.53	315.48
Moisture Content (%)	42.52	37.31
Void Ratio	1.1549	0.9566
Saturation (%)	97.36	103.15
Dry Density (pcf)	76.68	84.47

Tested By: C McMullen

Checked By:

Consolidation Test Results
Summary

Project: WDFW Cherry Creek
Location: Boring B-2, Sample S-6
Job Number: 1213006.010.011

Project Number: 1213006.010.011

Sample Number: S-6
Boring Number: B-2
Depth: 23.8
Sample Type: Undisturbed

Sample Description:
Gray, clayey SILT, scattered fine organics
Remarks:
Boring B-2, Sample S-6

Test Number: 1
Test Date: 12-19-2011

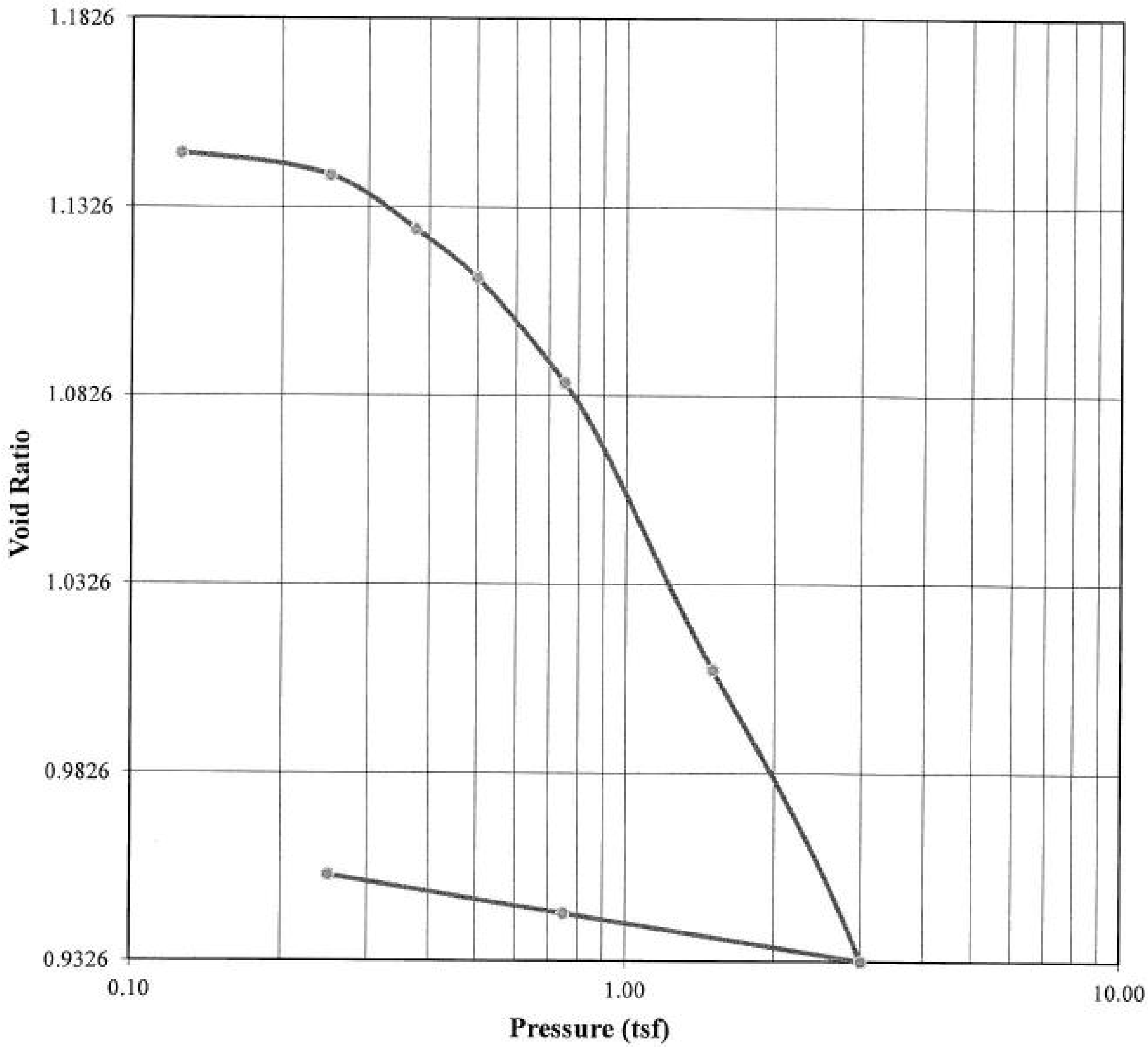
Index	Load Sequence (tsf)	Cummulative Change in Height (in)	Specimen Height (in)	Height of Void (in)	Vertical Strain (%)	Void Ratio	t90 Fitting Time (min)	t50 Fitting Time (min)	t90 Cv (ft2/year)	t50 Cv (ft2/year)
0	0.000	0.0000	1.0000	0.5357	0.00	1.1536	0.000	0.000	0.000	0.000
1	0.125	0.0031	0.9969	0.5326	0.31	1.1469	0.985	0.470	780.682	380.233
2	0.250	0.0058	0.9942	0.5299	0.58	1.1411	1.045	1.078	731.672	164.881
3	0.375	0.0124	0.9876	0.5233	1.24	1.1268	1.772	2.363	426.003	74.196
4	0.500	0.0184	0.9816	0.5173	1.84	1.1139	3.650	4.321	204.269	40.088
5	0.750	0.0313	0.9687	0.5044	3.13	1.0861	3.684	3.283	197.100	51.387
6	1.500	0.0667	0.9333	0.4690	6.67	1.0099	4.262	1.968	158.149	79.563
7	3.000	0.1026	0.8974	0.4331	10.26	0.9326	3.130	1.673	199.072	86.557
8	0.750	0.0967	0.9033	0.4390	9.67	0.9453				
9	0.250	0.0920	0.9080	0.4437	9.20	0.9554				

Predicted value indicated with *

Tested By: C. McMullen

Checked By: C. McMullen

Consolidation Test
Test Results



	Before	After	Liquid Limits:	41	Test Date:	12-19-2011
Moisture (%):	42.52	37.31	Plastic Limits:	29		
Dry Density (pcf):	76.68	84.47	Plasticity Index (%):	12		
Saturation (%):	97.36	103.15				
Void Ratio:	1.1549	0.9566	Specific Gravity:	2.650	Assumed	

Soil Description:	Gray, clayey SILT, scattered fine organics		
Project Number:	1213006.010.011	Depth: 23.8	Remarks:
Sample Number:	S-6	Boring Number: B-2	Boring B-2, Sample S-6
Project:	WDFW Cherry Creek		
Client:	WDFW		
Location:	Boring B-2, Sample S-6		

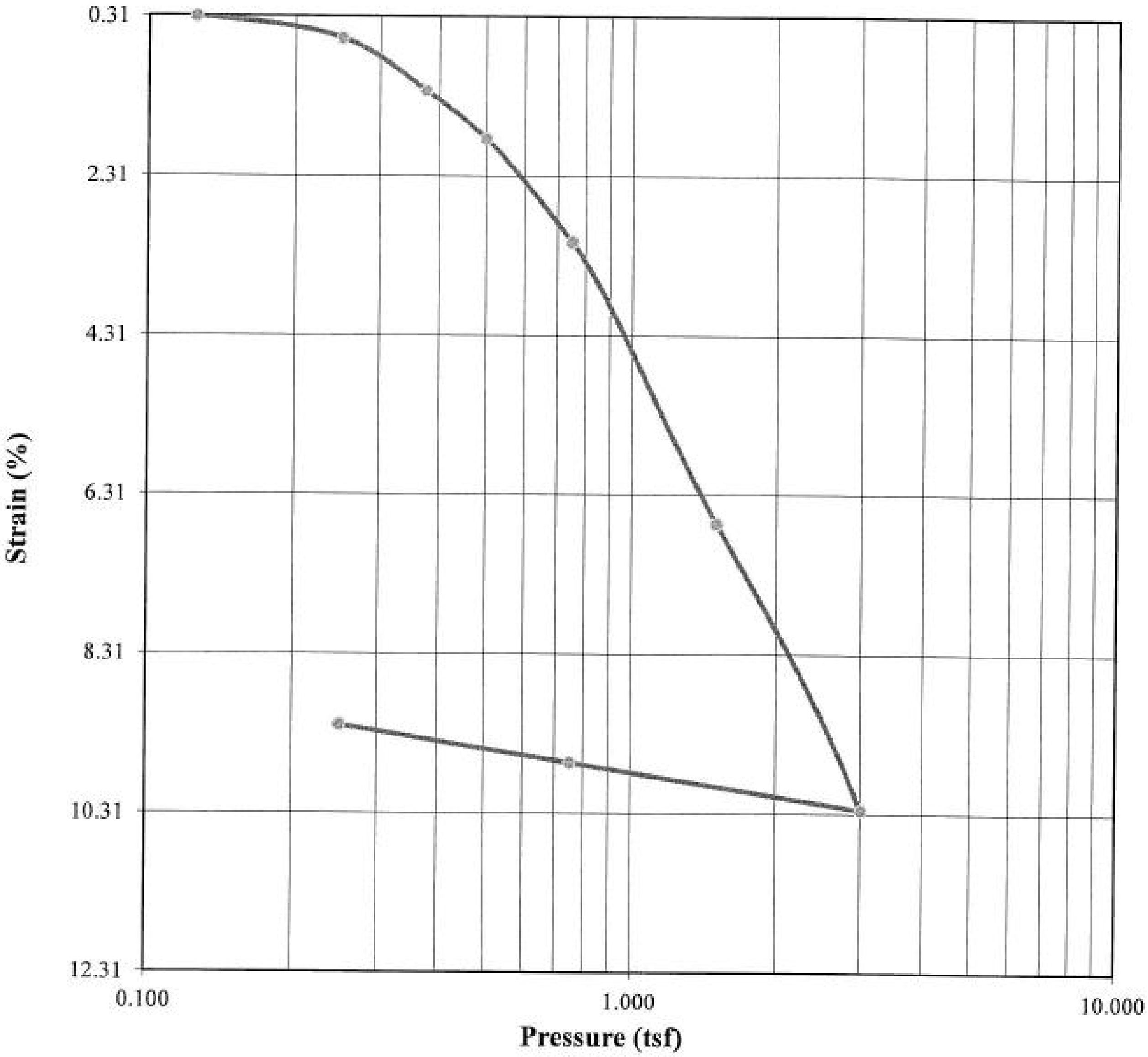
Landau Associates

Figure

A-7

(3 of 4)

Consolidation Test
Test Results



	Before	After				
			Liquid Limits:	41	**Test Date:**	12-19-2011
Moisture (%):	42.52	37.31	**Plastic Limits:**	29		
Dry Density (pcf):	76.68	84.47	**Plasticity Index (%):**	12		
Saturation (%):	97.36	103.15				
Void Ratio:	1.1549	0.9566	**Specific Gravity:**	2.650	Assumed	

Sample Description:	Gray, clayey SILT, scattered fine organics		
Project Number:	1213006.010.011	**Depth:** 23.8	**Remarks:**
Sample Number:	S-6	**Boring Number:** B-2	Boring B-2, Sample S-6
Project:	WDFW Cherry Creek		
Client:	WDFW		
Location:	Boring B-2, Sample S-6		

APPENDIX B

Stability Thresholds for Stream Restoration Materials

Stability Thresholds for Stream Restoration Materials

by Craig Fischenich[1]

May 2001

Complexity	***Value as a Planning Tool***	***Cost***
Low Moderate High	Low Moderate High	Low Moderate High

OVERVIEW

Stream restoration projects usually involve some modification to the channel or the banks. Designers of stabilization or restoration projects must ensure that the materials placed within the channel or on the banks will be stable for the full range of conditions expected during the design life of the project. Unfortunately, techniques to characterize stability thresholds are limited. Theoretical approaches do not exist and empirical data mainly consist of velocity limits, which are of limited value.

Empirical data for shear stress or stream power are generally lacking, but the existing body of information is summarized in this technical note. Whereas shear thresholds for soils found in channel beds and banks are quite low (generally < 0.25 lb/sf), those for vegetated soils (0.5 – 4 lb/sf), erosion control materials and bioengineering techniques (0.5 – 8 lb/sf), and hard armoring (< 13 lb/sf) offer options to provide stability.

STABILITY CRITERIA

The stability of a stream refers to how it accommodates itself to the inflowing water and sediment load. In general, stable streams may adjust their boundaries but do not exhibit trends in changes to their geometric character. One form of instability occurs when a stream is unable to transport its sediment load (i.e., sediments deposited within the channel), leading to the condition referred to as aggradation.

When the ability of the stream to transport sediment exceeds the availability of sediments within the incoming flow, and stability thresholds for the material forming the boundary of the channel are exceeded, erosion occurs. This technical note deals with the latter case of instability and distinguishes the presence or absence of erosion (threshold condition) from the magnitude of erosion (volume).

Erosion occurs when the hydraulic forces in the flow exceed the resisting forces of the channel boundary. The amount of erosion is a function of the relative magnitude of these forces and the time over which they are applied. The interaction of flow with the boundary of open channels is only imperfectly understood. Adequate analytical expressions describing this interaction have not yet been developed for conditions associated with natural channels. Thus, means of characterizing erosion potential must rely heavily upon empiricism.

Traditional approaches for characterizing erosion potential can be placed in one of two categories: maximum permissible velocity, and tractive force (or critical shear stress). The former approach is advantageous in that velocity is a parameter that can be measured within the flow. Shear stress cannot be directly measured – it must be computed from other flow parameters. Shear stress is a better measure of the fluid force on the channel boundary than is velocity. Moreover, conventional guidelines, including ASTM standards, rely upon the shear stress as a

[1] USAE Research and Development Center, Environmental Laboratory, 3909 Halls Ferry Rd., Vicksburg MS 39180

means of assessing the stability of erosion control materials. Both approaches are presented in this paper.

Incipient Motion (Threshold Condition)

As flow over the bed and banks of a stream increases, a condition referred to as the threshold state is reached when the forces tending to move materials on the channel boundary are in balance with those resisting motion. The forces acting on a noncohesive soil particle lying on the bed of a flowing stream include hydrodynamic lift, hydrodynamic drag, submerged weight ($F_w - F_b$), and a resisting force F_r. as seen in Figure 1. The drag is in the direction of the flow and the lift and weight are normal to the flow. The resisting force depends on the geometry of the particles. At the threshold of movement, the resultant of the forces in each direction is zero. Two approaches for defining the threshold state are discussed herein, initial movement being specified in terms of either a critical velocity (v_{cr}) or a critical shear stress (τ_{cr}).

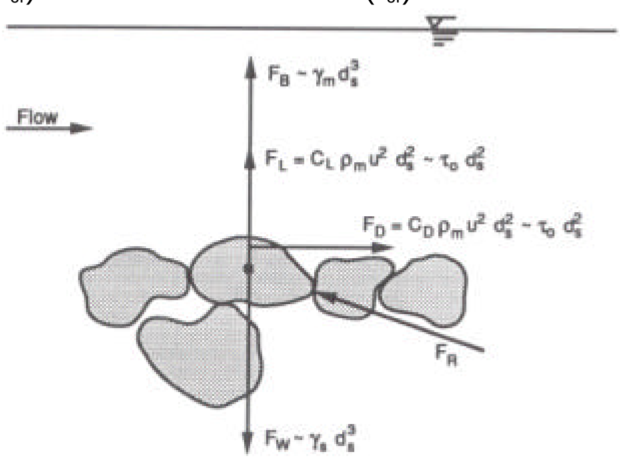


Figure 1. Forces acting on the boundary of a channel (adapted from Julien (1995)).

Critical Velocity

Figure 1 shows that both the lift and the drag force are directly related to the velocity squared. Thus, small changes in the velocity could result in large changes in these forces. The permissible velocity is defined as the maximum velocity of the channel that will not cause erosion of the channel boundary. It is often called the critical velocity because it refers to the condition for the initiation of motion. Early works in canal design and in evaluating the stability of waterways relied upon this method. Considerable empirical data exist relating maximum velocities to various soil and vegetation conditions.

However, this simple method for design does not consider the channel shape or flow depth. At the same mean velocity, channels of different shapes or depths may have quite different forces acting on the boundaries. Critical velocity is depth-dependent, and a correction factor for depth must be applied in this application. Despite these limitations, maximum permissible velocity can be a useful tool in evaluating the stability of various waterways. It is most frequently applied as a cursory analysis when screening alternatives.

Critical Shear Stress

The forces shown in Figure 1 can also be expressed in terms of the shear stress. Shear stress is the force per unit area in the flow direction. Its distribution in steady, uniform, two-dimensional flow in the channel can be reasonably described. An estimate of the average boundary shear stress (τ_o) exerted by the fluid on the bed is:

$$\tau_o = \gamma D S_f \qquad (1)$$

where γ is the specific weight of water, D is the flow depth (~ hydraulic radius), and S_f is the friction slope. Derived from consideration of the conservation of linear momentum, this quantity is a spatial average and may not provide a good estimate of bed shear at a point.

Critical shear stress (τ_{cr}) can be defined by equating the applied forces to the resisting forces. Shields (1936) determined the threshold condition by measuring sediment transport for values of shear at least twice the critical value and then extrapolating to the point vanishing sediment transport. His laboratory experiments have since served as a basis for defining critical shear stress. For soil grains of diameter d and angle of repose ϕ on a flat bed, the following relations can approximate the critical shear for various sizes of sediment:

$$\tau_{cr} = 0.5(\gamma_s - \gamma_w) d \, Tan\phi \quad \text{For clays} \qquad (2)$$

$$\tau_{cr} = 0.25 d_*^{-0.6} (\gamma_s - \gamma_w) d \, Tan\phi \quad \text{For silts and sands} \qquad (3)$$

$t_{cr} = 0.06(l_s - l_w)d\ Tan\, f$ For gravels and cobbles (4)

Where

$$d_* = d\left[\frac{(G-1)g}{n^2}\right]^{1/3} \qquad (5)$$

γ_s = the unit weight of the sediment
γ_w = the unit weight of the water/sediment mixture
G = the specific gravity of the sediment
G = gravitational acceleration
ν = the kinematic viscosity of the water/sediment mixture

The angle of repose ϕ for noncohesive sediments is presented in Table 1 (Julien 1995), as are values for critical shear stress. The critical condition can be defined in terms of shear velocity rather than shear stress (note that shear velocity and channel velocity are different). Table 1 also provides limiting shear velocity as a function of sediment size. The V_{*c} term is the critical shear velocity and is equal to

$$V_{*c} = \sqrt{gR_hS_f} \qquad (6)$$

Table 1. Limiting Shear Stress and Ve locity for Uniform Noncohesive Sediments

Class name	d_s (in)	*f* (deg)	t_{*c}	t_{cr} (lb/sf)	V_{*c} (ft/s)
Boulder					
Very large	>80	42	0.054	37.4	4.36
Large	>40	42	0.054	18.7	3.08
Medium	>20	42	0.054	9.3	2.20
Small	>10	42	0.054	4.7	1.54
Cobble					
Large	>5	42	0.054	2.3	1.08
Small	>2.5	41	0.052	1.1	0.75
Gravel					
Very coarse	>1.3	40	0.050	0.54	0.52
Coarse	>0.6	38	0.047	0.25	0.36
Medium	>0.3	36	0.044	0.12	0.24
Fine	>0.16	35	0.042	0.06	0.17
Very fine	>0.08	33	0.039	0.03	0.12
Sands					
Very coarse	>0.04	32	0.029	0.01	0.070
Coarse	>0.02	31	0.033	0.006	0.055
Medium	>0.01	30	0.048	0.004	0.045
Fine	>0.005	30	0.072	0.003	0.040
Very fine	>0.003	30	0.109	0.002	0.035
Silts					
Coarse	>0.002	30	0.165	0.001	0.030
Medium	>0.001	30	0.25	0.001	0.025

Table 1 provides limits best applied when evaluating idealized conditions, or the stability of sediments in the bed. Mixtures of sediments tend to behave differently from uniform sediments. Within a mixture, coarse sediments are generally entrained at lower shear stress values than presented in Table 1. Conversely, larger shear stresses than those presented in the table are required to entrain finer sediments within a mixture.

Cohesive soils, vegetation, and other armor materials can be similarly evaluated to determine empirical shear stress thresholds. Cohesive soils are usually eroded by the detachment and entrainment of soil aggregates. Motivating forces are the same as those for noncohesive banks; however, the resisting forces are primarily the result of cohesive bonds between particles. The bonding strength, and hence the soil erosion resistance, depends on the physio-chemical properties of the soil and the chemistry of the fluids. Field and laboratory experiments show that intact, undisturbed cohesive soils are much less susceptible to flow erosion than are non-cohesive soils.

Vegetation, which has a profound effect on the stability of both cohesive and noncohesive soils, serves as an effective buffer between the water and the underlying soil. It increases the effective roughness height of the boundary, increasing flow resistance and displacing the velocity upwards away from the soil, which has the effect of reducing the forces of drag and lift acting on the soil surface. As the boundary shear stress is proportional to the square of the near-bank velocity, a reduction in this velocity produces a much greater reduction in the forces responsible for erosion.

Vegetation armors the soil surface, but the roots and rhizomes of plants also bind the soil and introduce extra cohesion over and above any intrinsic cohesion that the bank material may have. The presence of vegetation does not render underlying soils immune from erosion, but the critical condition for erosion of a vegetated bank is usually the threshold of failure of the plant stands by snapping, stem scour, or uprooting, rather than for detachment and entrainment of the soils themselves. Vegetation failure usually occurs at much higher levels of flow intensity than for soil erosion.

Both rigid and flexible armor systems can be used in waterways to protect the channel bed from erosion and to stabilize side slopes. A wide array of differing armor materials are available to accomplish this. Many manufactured products have been evaluated to determine their failure threshold. Products are frequently selected using design graphs that present the flow depth on one axis and the slope of the channel on the other axis. Thus, the design is based on the depth/slope product (i.e., the shear stress). In other cases, the thresholds are expressed explicitly in terms of shear stress. Notable among the latter group are the field performance testing results of erosion control products conducted by the TXDOT/TTI Hydraulics and Erosion Control Laboratory (TXDOT 1999).

Table 2 presents limiting values for shear stress and velocity for a number of different channel lining materials. Included are soils, various types of vegetation, and number of different commonly applied stabilization techniques. Information presented in the table was derived from a number of different sources. Ranges of values presented in the table reflect various measures presented within the literature. In the case of manufactured products, the designer should consult the manufacturer's guidelines to determine thresholds for a specific product.

Uncertainty and Variability

The values presented in Table 2 generally relate to average values of shear stress or velocity. Velocity and shear stress are neither uniform nor steady in natural channels. Short-term pulses in the flow can give rise to instantaneous velocities or stresses of two to three times the average; thus, erosion may occur at stresses much lower than predicted. Because limits presented in Table 2 were developed empirically, they implicitly include some off this variability. However, natural channels typically exhibit much more variability than the flumes from which these data were developed.

Sediment load can also profoundly influence the ability of flow to erode underlying soils. Sediments in suspension have the effect of damping turbulence within the flow. Turbulence is an important factor in entraining materials from the channel boundaries. Thus, velocity and shear stress thresholds are 1.5 to 3 times that presented in the table for flows carrying high sediment loads.

In addition to variability of flow conditions, variation in the channel lining characteristics can influence erosion predictions. Natural bed material is neither spherical nor of uniform size. Larger particles may shield smaller ones from direct impact so that the latter fail to move until higher stresses are attained. For a given grain size, the true threshold criterion may vary by nearly an order of magnitude depending on the bed gradation. Variation in the installation of erosion control measures can reduce the threshold necessary to cause erosion.

Table 2. Permissible Shear and Velocity for Selected Lining Materials[1]

Boundary Category	Boundary Type	Permissible Shear Stress (lb/sq ft)	Permissible Velocity (ft/sec)	Citation(s)
Soils	Fine colloidal sand	0.02 - 0.03	1.5	A
	Sandy loam (noncolloidal)	0.03 - 0.04	1.75	A
	Alluvial silt (noncolloidal)	0.045 - 0.05	2	A
	Silty loam (noncolloidal)	0.045 - 0.05	1.75 – 2.25	A
	Firm loam	0.075	2.5	A
	Fine gravels	0.075	2.5	A
	Stiff clay	0.26	3 – 4.5	A, F
	Alluvial silt (colloidal)	0.26	3.75	A
	Graded loam to cobbles	0.38	3.75	A
	Graded silts to cobbles	0.43	4	A
	Shales and hardpan	0.67	6	A
Gravel/Cobble	1-in.	0.33	2.5 – 5	A
	2-in.	0.67	3 – 6	A
	6-in.	2.0	4 – 7.5	A
	12-in.	4.0	5.5 – 12	A
Vegetation	Class A turf	3.7	6 – 8	E, N
	Class B turf	2.1	4 - 7	E, N
	Class C turf	1.0	3.5	E, N
	Long native grasses	1.2 – 1.7	4 – 6	G, H, L, N
	Short native and bunch grass	0.7 - 0.95	3 – 4	G, H, L, N
	Reed plantings	0.1-0.6	N/A	E, N
	Hardwood tree plantings	0.41-2.5	N/A	E, N
Temporary Degradable RECPs	Jute net	0.45	1 – 2.5	E, H, M
	Straw with net	1.5 – 1.65	1 – 3	E, H, M
	Coconut fiber with net	2.25	3 – 4	E, M
	Fiberglass roving	2.00	2.5 – 7	E, H, M
Non-Degradable RECPs	Unvegetated	3.00	5 – 7	E, G, M
	Partially established	4.0-6.0	7.5 – 15	E, G, M
	Fully vegetated	8.00	8 – 21	F, L, M
Riprap	6 – in. d_{50}	2.5	5 – 10	H
	9 – in. d_{50}	3.8	7 – 11	H
	12 – in. d_{50}	5.1	10 – 13	H
	18 – in. d_{50}	7.6	12 – 16	H
	24 – in. d_{50}	10.1	14 – 18	E
Soil Bioengineering	Wattles	0.2 – 1.0	3	C, I, J, N
	Reed fascine	0.6-1.25	5	E
	Coir roll	3 - 5	8	E, M, N
	Vegetated coir mat	4 - 8	9.5	E, M, N
	Live brush mattress (initial)	0.4 – 4.1	4	B, E, I
	Live brush mattress (grown)	3.90-8.2	12	B, C, E, I, N
	Brush layering (initial/grown)	0.4 – 6.25	12	E, I, N
	Live fascine	1.25-3.10	6 – 8	C, E, I, J
	Live willow stakes	2.10-3.10	3 – 10	E, N, O
Hard Surfacing	Gabions	10	14 – 19	D
	Concrete	12.5	>18	H

[1] Ranges of values generally reflect multiple sources of data or different testing conditions.

A. Chang, H.H. (1988).
B. Florineth. (1982)
C. Gerstgraser, C. (1998).
D. Goff, K. (1999).
E. Gray, D.H., and Sotir, R.B. (1996).
F. Julien, P.Y. (1995).
G. Kouwen, N.; Li, R. M.; and Simons, D.B., (1980).
H. Norman, J. N. (1975).
I. Schiechtl, H. M. and R. Stern. (1996).
J. Schoklitsch, A. (1937).
K. Sprague, C.J. (1999).
L. Temple, D.M. (1980).
M. TXDOT (1999)
N. Data from Author (2001)
O. USACE (1997).

Changes in the density or vigor of vegetation can either increase or decrease erosion threshold. Even differences between the growing and dormant seasons can lead to one- to twofold changes in erosion thresholds.

To address uncertainty and variability, the designer should adjust the predicted velocity or shear stress by applying a factor of safety or by computing local and instantaneous values for these parameters. Guidance for making these adjustments is presented in the section titled “Application” below.

EROSION MAGNITUDE

The preceding discussion dealt with the presence or absence of erosion, but did not address the extent to which erosion might occur for a given flow. If the thresholds presented in Table 2 are exceeded, erosion should be expected to occur. In reality, even when those thresholds are not exceeded, some erosion in a few select locations may occur. The extent to which this minor erosion could become a significant concern depends in large measure on the duration of the flow, and upon the ability of the stream to transport those eroded sediments.

Flow Duration

Although not stated, limits regarding erosion potential published by manufacturers for various products are typically developed from studies using short flow durations. They do not reflect the potential for severe erosion damage that can result from moderate flow events over several hours. Studies have shown that duration of flow reduces erosion resistance of many types of erosion control products, as shown in Figures 2 - 4. A factor of safety should be applied when flow duration exceeds a couple of hours.

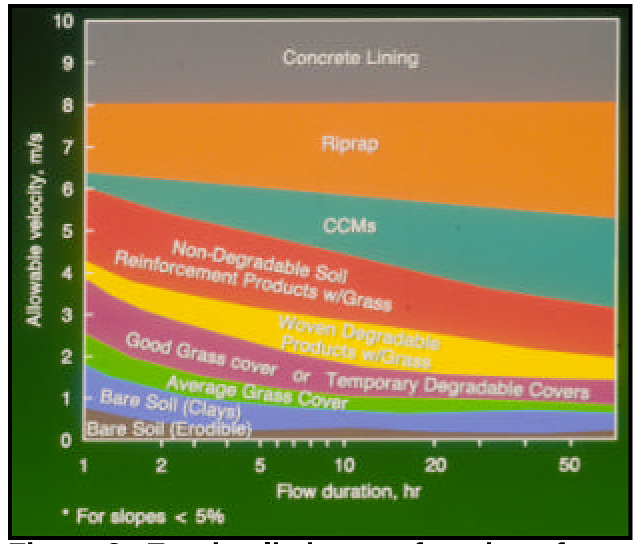


Figure 2. Erosion limits as a function of flow duration (from Fischenich and Allen (2000)).

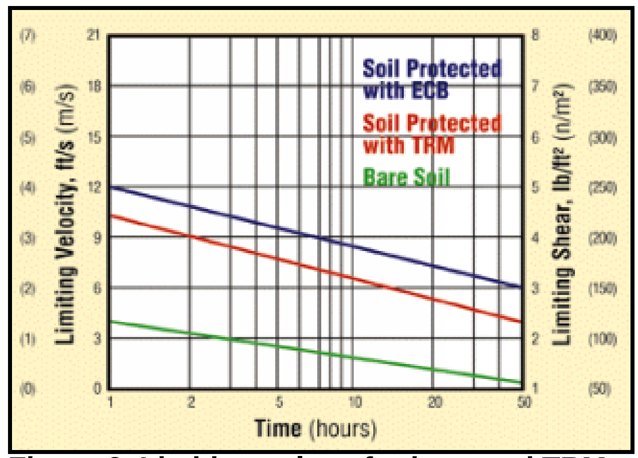


Figure 3. Limiting values for bare and TRM protected soils (from Sprague (1999))

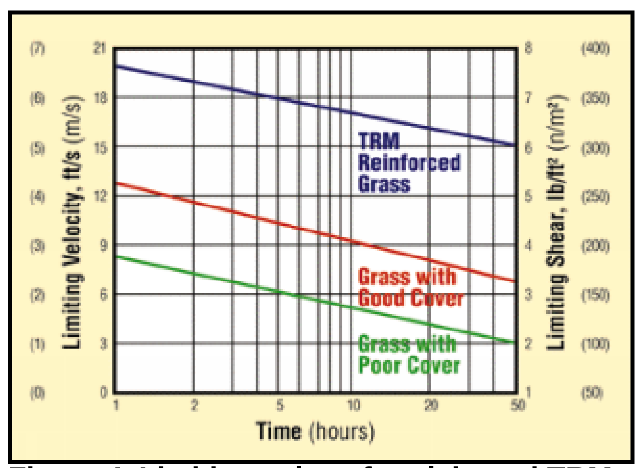


Figure 4. Limiting values for plain and TRM reinforced grass (from Sprague (1999))

Correlations between flow volume and amount of erosion tend to be poor. Multi-peaked flows may be more effective than single flows of comparable or greater magnitude because of the increased incidence of wetting. Flows with long durations often have a more significant effect on erosion than short-lived flows of higher magnitude. Sediment transport analysis can be used to gauge the magnitude of erosion potential in the channel design, but predictive capability is limited.

Sediment Transport

A number of flow measures can be used to assess the ability of a stream to transport sediment. The unit stream power (P_m) is one common approach, and is related to the earlier discussion in that stream power includes both velocity and shear stress as components. Sediment transport (Q_s) increases when the unit stream power (P_m) increases. Unit stream power in turn is controlled by both tractive stress and flow velocity:

$$P_m = v \cdot \tau = v \cdot \gamma_w \cdot D \cdot S_f \quad (7)$$

The total power (P_t) is the product of the unit power times the channel width (W):

$$P_t = P_m \cdot W = v \cdot W \cdot D \cdot \gamma_w \cdot S_f = v \cdot A \cdot \gamma_w \cdot S_f = Q_w \cdot \gamma_w \cdot S_f \quad (8)$$

Stream power assessments can be useful in evaluating sediment discharge within a stream channel and the deposition or erosion of sediments from the streambed. However, their utility for evaluating the stability of measures applied to prevent erosion is limited because of the lack of empirical data relating stream power to stability. The analysis of general streambank erosion is not a simple extension of the noncohesive bed case with an added downslope gravity component. Complication is added by other influencing variables, such as vegetation, whose root system can reinforce bank material and increase erosion resistance. Factors influencing bank erosion are summarized in Table 3.

Table 3. Factors Influencing Erosion

actor	*elevant characteristics*
low properties	lagnitude, frequency and variability of stream discharge; Magnitude and distribution of elocity and shear stress; Degree of turbulence
ediment composition	ediment size, gradation, cohesion and stratification
limate	ainfall amount, intensity and duration; Frequency and duration of freezing
ubsurface conditions	eepage forces; Piping; Soil moisture levels
hannel geometry	Vidth and depth of channel; Height and angle of bank; Bend curvature
iology	'egetation type, density and root character; Burrows
nthropogenic factors	Irbanization, flood control, boating, irrigation

APPLICATION

The stability of a waterway or the suitability of various channel linings can be determined by first calculating both the mean velocity and tractive stress (by the previous equations). These values can then be compared with allowable velocity and tractive stress for a particular ground cover or lining system under consideration (e.g., existing vegetation cover, an erosion control blanket, or bioengineering treatment). Allowable tractive stresses for various types of soil, linings, ground covers, and stabilization measures including soil bioengineering treatments, are listed in Table 2. Additionally, manufacturers' product literature can provide allowable tractive stresses or velocities for various types of erosion control products.

An iterative procedure may be required when evaluating channel stability because various linings will affect the resistance coefficient, which in turn may change the estimated flow conditions. A general procedure for the application of information presented in this paper is outlined in the following paragraphs.

Step 1-Estimate Mean Hydraulic Conditions.
Flow of water in a channel is governed by the discharge, hydraulic gradient, channel geometry, and roughness coefficient. This functional relationship is most frequently evaluated using normal depth or backwater computations that take into account principles of conservation of linear momentum. The latter is preferable because it accounts for variations in momentum slope, which is directly related to shear stress. Several models are available to aid the hydraulic engineer in assessing hydraulic conditions. Notable examples include HEC-2, HEC-RAS, and WSP2. Channel cross sections, slopes, and Manning's coefficients should be determined based upon surveyed data and observed or predicted channel boundary conditions. Output from the model should be used to compute main channel velocity and shear stress at each cross section.

Step 2- Estimate Local/Instantaneous Flow Conditions.
The computed values for velocity and shear stress may be adjusted to account for local variability and instantaneous values higher than mean. A number of procedures exist for this purpose. Most commonly applied are empirical methods based upon channel form and irregularity. Several references at the end of this paper present procedures to make these adjustments. Chang (1988) is a good example. For straight channels, the local maximum shear stress can be assumed from the following simple equation:

$$\boldsymbol{t}_{\max} = 1.5\boldsymbol{t} \tag{9}$$

for sinuous channels, the maximum shear stress should be determined as a function of the planform characteristics using Equation 10:

$$\boldsymbol{t}_{\max} = 2.65\,\boldsymbol{t}\left(\frac{R_c}{W}\right)^{-0.5} \tag{10}$$

where R_c is the radius of curvature and W is the top width of the channel. Equations 9 and 10 adjust for the spatial distribution of shear stress; however, temporal maximums in turbulent flows can be 10 – 20 percent higher, so an adjustment to account for instantaneous maximums should be added as well. A factor of 1.15 is usually applied.

Step 3- Determine Existing Stability.
Existing stability should be assessed by comparing estimates of local and instantaneous shear and velocity to values presented in Table 2. Both the underlying soil and the soil/vegetation condition should be assessed. If the existing conditions are deemed stable and are in consonance with other project objectives, then no further action is required. Otherwise, proceed to step 4.

Step 4- Select Channel Lining Material.
If existing conditions are unstable, or if a different material is needed along the channel perimeter to meet project objectives, a lining material or stabilization measure should be selected from Table 2, using the threshold values as a guideline in the selection. Only material with a threshold exceeding the predicted value should be selected. The other project objectives can also be used at this point to help select from among the available alternatives. Fischenich and Allen (2000) characterize attributes of various protection measures to help in the selection.

Step 5- Recompute Flow Values.
Resistance values in the hydraulic computations should be adjusted to reflect the selected channel lining, and hydraulic condition should be recalculated for the channel. At this point, reach- or section-averaged hydraulic conditions should be adjusted to account for local and instantaneous extremes.
Table 4 presents velocity limits for various channel boundaries conditions. This table is useful in screening alternatives, or as an alternative to the shear stress analysis presented in the preceding sections.

Table 4. Stability of Channel Linings for Given Velocity Ranges

Lining	0 – 2 fps	2 – 4 fps	4 – 6 fps	6 – 8 fps	> 8 fps
Sandy Soils	Use Caution	Not Appropriate	Not Appropriate	Not Appropriate	Not Appropriate
Firm Loam	Use Caution	Use Caution	Not Appropriate	Not Appropriate	Not Appropriate
Mixed Gravel and Cobbles	Appropriate	Use Caution	Use Caution	Not Appropriate	Not Appropriate
Average Turf	Appropriate	Use Caution	Use Caution	Not Appropriate	Not Appropriate
Degradable RECPs	Appropriate	Use Caution	Use Caution	Use Caution	Not Appropriate
Stabilizing Bioengineering	Appropriate	Use Caution	Use Caution	Use Caution	Not Appropriate
Good Turf	Appropriate	Appropriate	Use Caution	Use Caution	Use Caution
Permanent RECPs	Appropriate	Appropriate	Appropriate	Appropriate	Use Caution
Armoring Bioengineering	Appropriate	Appropriate	Appropriate	Appropriate	Use Caution
CCMs & Gabions	Appropriate	Appropriate	Appropriate	Appropriate	Use Caution
Riprap	Appropriate	Appropriate	Appropriate	Appropriate	Appropriate
Concrete	Appropriate	Appropriate	Appropriate	Appropriate	Appropriate

Key:

(green)	Appropriate
(yellow)	Use Caution
(red)	Not Appropriate

Step 6– Confirm Lining Stability.

The stability of the proposed lining should be assessed by comparing the threshold values in Table 2 to the newly computed hydraulic conditions. These values can be adjusted to account for flow duration using Figures 2-4 as a guide. If computed values exceed thresholds, step 4 should be repeated. If the threshold is not exceeded, a factor of safety for the project should be determined from the following equations:

$$FS = \frac{t_{max}}{t_{est}} \quad or \quad FS = \frac{V_{max}}{V_{est}} \qquad (11)$$

In general, factors of safety in excess of 1.2 or 1.3 should be acceptable. The preceding five steps should be conducted for every cross section used in the analysis for the project. In the event that computed hydraulic values exceed thresholds for any desirable lining or stabilization technique, measures must be undertaken to reduce the energy within the flow. Such measures might include the installation of low-head drop structures or other energy-dissipating devices along the channel. Alternatively, measures implemented within the watershed to reduce total discharge could be employed.

APPLICABILITY AND LIMITATIONS

Techniques described in this technical note are generally applicable to stream restoration projects that include revegetation of the riparian zone or bioengineering treatments.

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Fischenich, C. (2001). "Stability Thresholds for Stream Restoration Materials," EMRRP Technical Notes Collection (ERDC TN-EMRRP-SR-29), U.S. Army Engineer Research and Development Center, Vicksburg, MS. *www.wes.army.mil/el/emrrp*

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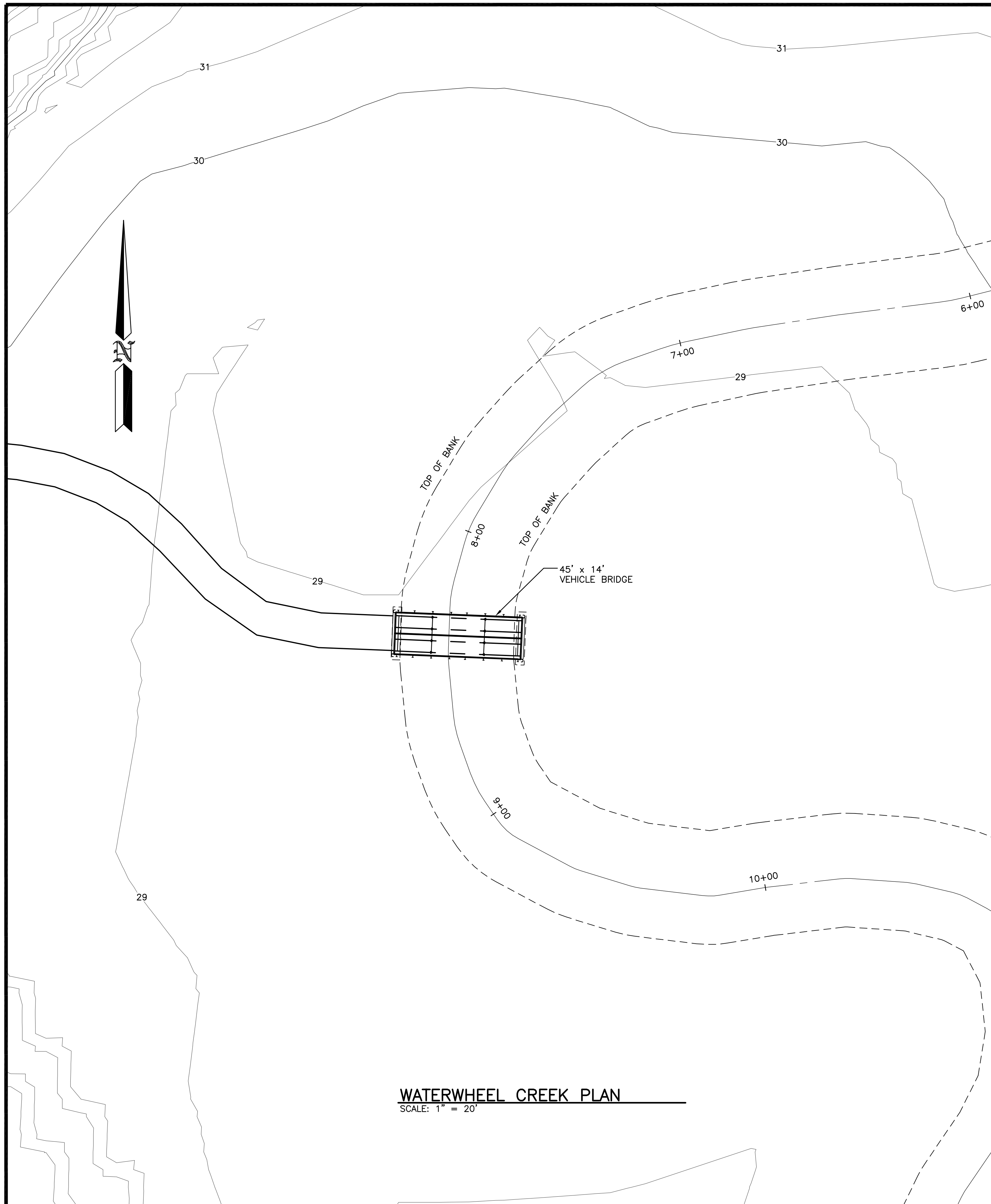
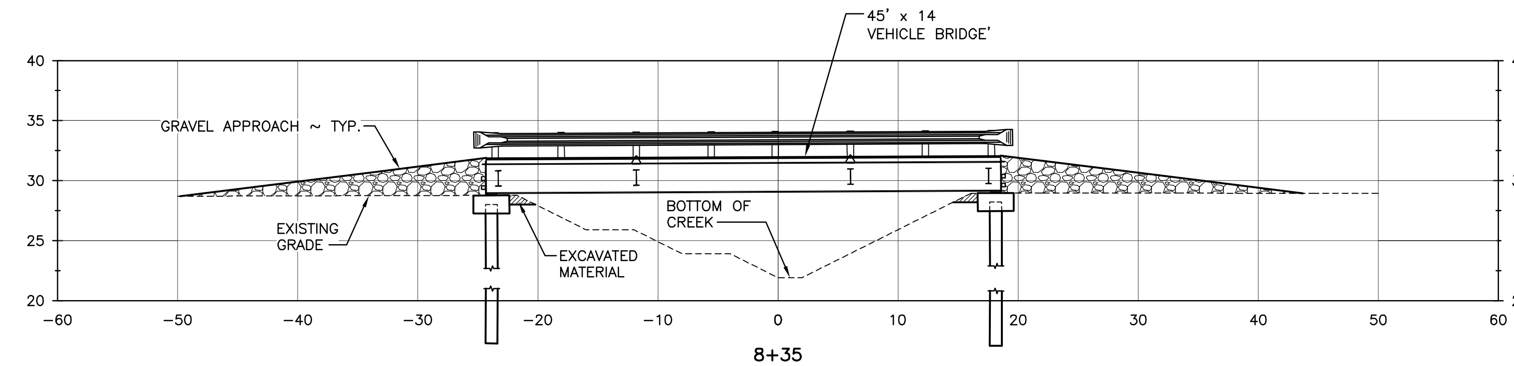
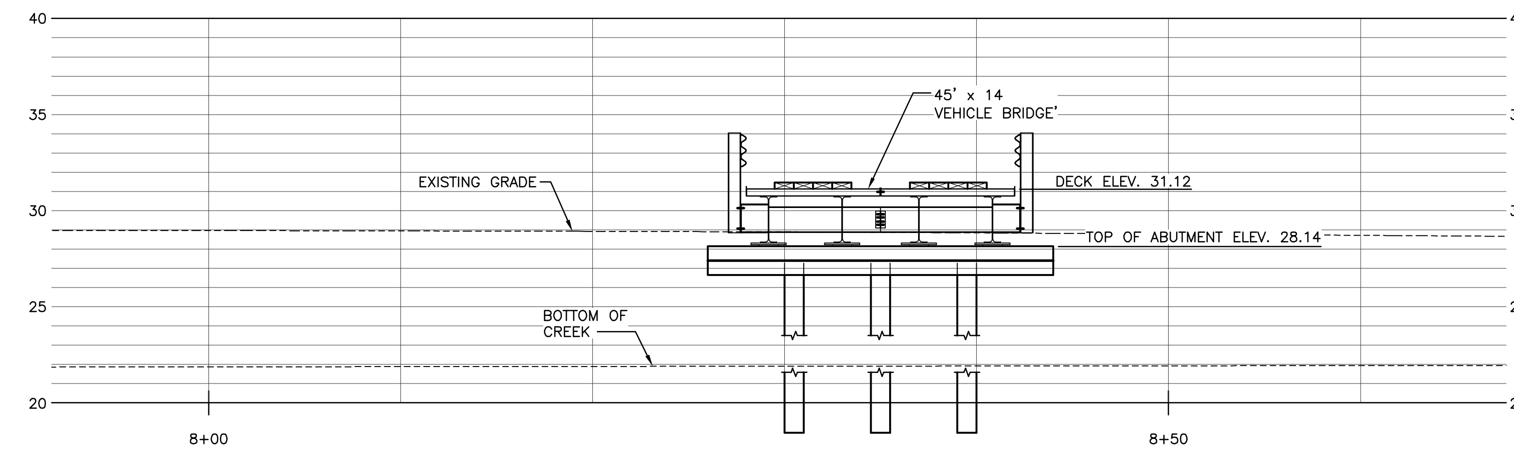


WATERWHEEL CREEK PLAN
SCALE: 1" = 20'



WATERWHEEL CREEK SECTION
SCALE: 1" = 8'



WATERWHEEL CREEK PROFILE
SCALE: 1" = 5'

LOADING

DEAD LOAD:	19 TONS
LIVE LOAD:	10 TONS

COUNTY:	KING
PARCEL NUMBER:	0726079031
TOWNSHIP:	26N
RANGE:	7E, W.M.
SECTION:	7

WASHINGTON STATE
DEPARTMENT OF FISH AND WILDLIFE

SYM	DATE	REVISION DESCRIPTION	BY

APPROVED AND RELEASED FOR CONSTRUCTION
CHIEF ENGINEER .... DATE: ....
PROGRAM .... DATE: ....

0 ——— 1"
BAR MEASURES ONE INCH ON ORIGINAL DRAWINGS

DESIGNED BY K. KUYKENDALL
CHECKED BY
DRAWN BY A. JOHNSON
DATE 03/15/2012

CHERRY VALLEY WILDLIFE AREA
WATERWHEEL CREEK BRIDGE
PLAN, PROFILE, & SECTION

PROJECT NO. KG:R37:11-1
SHEET 1 OF 3

GENERAL STRUCTURAL NOTES

GENERAL

BUILDING CODE: ALL MATERIALS, WORKMANSHIP, DESIGN, AND CONSTRUCTION SHALL CONFORM TO THE DRAWINGS, SPECIFICATIONS, AND THE 2010 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS.

STANDARDS: REFERENCE TO ASTM AND OTHER STANDARDS SHALL MEAN THE LATEST EDITION IN EFFECT ON THE BID DATE, UNLESS NOTED IN THESE DOCUMENTS OR DESIGNATED BY THE GOVERNING CODE.

GENERAL: THESE NOTES CONTAIN GENERAL INFORMATION AND ARE NOT COMPLETE FOR CONSTRUCTION PURPOSES. THE CONTRACTOR SHALL VERIFY INFORMATION (INCLUDING ALL DIMENSIONS) PROVIDED IN THE DRAWINGS WITH THE PROJECT SPECIFICATIONS AND BRING ANY CONFLICTS TO THE ATTENTION OF THE OWNERS REPRESENTATIVE BEFORE BEGINNING THE AFFECTED WORK. THE OWNERS REPRESENTATIVE WILL RESOLVE ANY SUCH CONFLICT.

LOADS

IN ADDITION TO THE BRIDGE SELF WEIGHT, THE FOLLOWING LOADS ARE USED FOR DESIGN:
10 T SERVICE TRUCK

CONTRACTOR RESPONSIBILITIES: DRAWINGS REPRESENT DESIGN OF STRUCTURE IN COMPLETED FORM. CONTRACTOR SHALL BE RESPONSIBLE FOR METHODS, SEQUENCES, AND SAFETY PRECAUTIONS REQUIRED TO PERFORM WORK.

CONTRACTOR SHALL PROVIDE TEMPORARY SHORING AND BRACING OF STRUCTURAL MEMBERS AS REQUIRED. SHORING AND BRACING SHALL NOT BE REMOVED UNTIL ALL FINAL CONNECTIONS HAVE BEEN COMPLETED IN ACCORDANCE WITH THE DRAWINGS, AND MATERIALS HAVE ACHIEVED DESIGN STRENGTH.

DISCREPANCIES: IN CASE OF DISCREPANCIES BETWEEN DRAWINGS, SPECIFICATIONS, REFERENCE STANDARDS, OR GOVERNING CODE, THE MORE STRINGENT REQUIREMENTS SHALL GOVERN. CONTRACTOR SHALL NOTIFY THE CONTRACTING OFFICER OF DISCREPANCIES AND OBTAIN DIRECTION PRIOR TO PROCEEDING. NOTES ON INDIVIDUAL STRUCTURAL DRAWINGS SHALL TAKE PRIORITY OVER STRUCTURAL NOTES. NOTED DIMENSIONS TAKE PRECEDENCE OVER SCALED DIMENSIONS.

SCALING OF DRAWINGS: DRAWINGS ARE NOT TO BE SCALED.

SPECIFICATIONS: REFER TO SPECIFICATIONS FOR INFORMATION IN ADDITION TO THESE NOTES AND DRAWINGS.

GEOTECHNICAL

FOUNDATION DESIGN IS BASED ON THE SOIL PARAMETERS OF THE FOLLOWING GEOTECHNICAL REPORT.

GEOTECHNICAL REPORT REFERENCE: TITLE: GEOTECHNICAL DESIGN RECOMMENDATIONS
CHERRY VALLEY CULVERT REPLACEMENT PROJECT
DUVALL, WA

PREPARED BY: LANDAU ASSOCIATES
DATE: FEBRUARY 24, 2012

SUBMITTALS

SHOP DRAWINGS SHALL BE SUBMITTED TO THE OWNERS REPRESENTATIVE FOR REVIEW PRIOR TO FABRICATION OR CONSTRUCTION OF THESE ITEMS:

CONCRETE MIX DESIGN
CONCRETE REINFORCING
BRIDGE SUPER STRUCTURE

CONTRACTOR SHALL REVIEW AND STAMP SUBMITTALS PRIOR TO SUBMISSION. IF SHOP DRAWINGS DIFFER FROM DESIGN SHOWN ON STRUCTURAL DRAWINGS, THEY SHALL BE SEALED BY THE WASHINGTON STATE REGISTERED PROFESSIONAL, ENGINEER RESPONSIBLE FOR THE DESIGN. DIMENSIONS AND QUANTITIES ARE CONTRACTOR'S RESPONSIBILITY AND WILL NOT BE REVIEWED. CONTRACTOR SHALL BE RESPONSIBLE FOR ALL MATERIALS PLACED PRIOR TO RECEIPT OF REVIEWED SHOP DRAWINGS. CONTRACTOR SHALL ALLOW A MINIMUM OF 10 WORKING DAYS FOR REVIEW.

DEFERRED SUBMITTALS: DRAWINGS AND CALCULATIONS FOR BIDDER-DESIGNED COMPONENTS, SEALED BY THE WASHINGTON STATE REGISTERED PROFESSIONAL ENGINEER RESPONSIBLE FOR THE DESIGN, SHALL BE SUBMITTED TO THE OWNERS REPRESENTATIVE FOR REVIEW OF GENERAL CONFORMANCE WITH THE DESIGN OF THE BRIDGE. DEFERRED SUBMITTALS INCLUDE:

BRIDGE SUPER STRUCTURE
MICROPILE, IF USED

SUBMITTALS OF BIDDER-DESIGNED COMPONENTS SHALL INCLUDE LOCATIONS, MAGNITUDES, AND DIRECTIONS OF ALL FORCES TRANSFERRED TO THE STRUCTURE. CALCULATIONS SUBMITTED FOR BIDDER-DESIGNED COMPONENTS ARE FOR INFORMATION ONLY AND WILL NOT BE REVIEWED.

EXISTING CONDITIONS: CONTRACTOR SHALL VERIFY ALL EXISTING DIMENSIONS, MEMBER SIZES, AND CONDITIONS PRIOR TO COMMENCING ANY WORK. THIS IS CRITICAL FOR FABRICATION AND INSTALLATION OF NEW ELEMENTS AND MODIFICATIONS TO EXISTING ELEMENTS. ALL DIMENSIONS OF EXISTING CONSTRUCTION SHOWN ON THE DRAWINGS ARE INTENDED AS GUIDELINES ONLY AND MUST BE VERIFIED. EXISTING CONDITIONS SHOWN ON DRAWINGS ARE BASED EITHER ON SITE OBSERVATIONS, ORIGINAL DRAWINGS, OR WERE ASSUMED BASED ON EXPECTED CONDITIONS. ACTUAL CONDITIONS MAY VARY DUE TO CONSTRUCTION ERRORS AND TOLERANCES, ERRORS IN THE RECORD DRAWINGS, DAMAGE, DETERIORATION, UNDOCUMENTED MODIFICATIONS OR OTHER FACTORS. IF EXISTING CONDITIONS DO NOT CLOSELY MATCH CONDITIONS SHOWN ON DRAWINGS, OR IF EXISTING MATERIALS ARE OF QUESTIONABLE OR SUBSTANDARD QUALITY, NOTIFY OWNERS REPRESENTATIVE PRIOR TO COMMENCING ANY WORK.

ANCHORAGE

POST-INSTALLED ANCHORS SHALL BE INSTALLED PER MANUFACTURER'S INSTRUCTIONS AND NOTED ICC-ES REPORTS. ANCHORS SHALL BE THE FOLLOWING TYPES.

ADHESIVE ANCHORS ICBO APPROVED, ASTM C881 TYPE IV, GRADE 3, CLASS B/C, TWO-COMPONENT, NON-SAG, EPOXY TYPE ADHESIVE. MIX AND INSTALL IN ACCORDANCE WITH THE MANUFACTURER'S WRITTEN INSTRUCTIONS AND SPECIFICATIONS. CONCRETE DRILL BITS SHALL CONFORM TO ANSI B212.15.

NO REINFORCING BARS SHALL BE CUT TO INSTALL ANCHORS IF EXISTING REINFORCING IS ENCOUNTERED. ANCHOR HOLES SHALL BE FILLED WITH EPOXY ADHESIVE AND A NEW HOLE DRILLED A MINIMUM OF 3 BOLT DIAMETERS AWAY.

CONCRETE

CONCRETE SHALL BE PROPORTIONED TO ACHIEVE A WORKABLE MIX THAT CAN BE PLACED WITHOUT SEGREGATION OR EXCESS FREE SURFACE WATER. MIX DESIGNS SHALL BE SUBMITTED FOR REVIEW PRIOR TO USE. COMPLY WITH IBC SECTION 1905. MIXES SHALL MEET OR EXCEED THE FOLLOWING CRITERIA:

TYPE OF CONSTRUCTION	COMPRESSIVE STRENGTH (f'c)	TEST AGE	MAXIMUM WATER/CEMENT RATIO
CONCRETE	4,000 PSI	28 DAYS	0.40

EMBEDDED ITEMS: CONDUIT AND SLEEVES SHALL NOT BE EMBEDDED IN OR PASS THROUGH CONCRETE WITHOUT APPROVAL. ALUMINUM ITEMS SHALL NOT BE EMBEDDED IN CONCRETE. SUBMIT CONDUIT LAYOUT AND EMBEDDED ITEM PLANS FOR REVIEW PRIOR TO PLACING CONCRETE.

CONCRETE CAST AGAINST EXISTING CONCRETE: THE SURFACE OF THE EXISTING CONCRETE SHALL BE PREPARED IN ACCORDANCE WITH THE FOLLOWING:

1. SURFACE OF JOINT SHALL BE SAND BLASTED OR ROUGHENED WITH A CHIPPING HAMMER TO EXPOSE AGGREGATE EMBEDDED IN PREVIOUS POUR.
2. EXPOSED AGGREGATE SHALL PROTRUDE ¼" MINIMUM.
3. JOINT SURFACE SHALL BE CLEANED AND LAITANCE REMOVED.
4. JOINT SHALL BE WETTED AND STANDING WATER REMOVED IMMEDIATELY BEFORE NEW CONCRETE IS PLACED.

CONCRETE REINFORCEMENT

REINFORCING STEEL SHALL BE PLACED AND SUPPORTED IN ACCORDANCE WITH CRSI MSP-1. REINFORCING STEEL SHALL BE DETAILED IN ACCORDANCE WITH ACI SP-66. NO BENDING OR STRAIGHTENING OF REINFORCEMENT WILL BE PERMITTED AFTER PARTIAL EMBEDMENT IN CONCRETE.

LAP ALL CONTINUOUS REINFORCEMENT IN ACCORDANCE WITH REINFORCEMENT SPLICE AND DEVELOPMENT LENGTH SCHEDULE BELOW, PROVIDE CORNER BARS AT ALL WALL AND FOOTING INTERSECTIONS. LAP ADJACENT MATS OF WELDED WIRE FABRIC A MINIMUM OF 1 CROSS WIRE SPACING + 2" OR 8" WHICHEVER IS GREATER.

F'c = 4,000 PSI			
SIZE	LA (L_a)$^{(2,3,4)}$ & LD (L_d)$^{(2,3,4)}$	LB (L_b)$^{(2,3,4)}$	LDH (L_{dh})
3	15	19	8
4	19	25	10
5	24	31	12
6	29	37	15
7	42	54	17
8	48	62	19
9	54	70	22
10	61	79	25
11	67	87	27

NOTES:

1. USE THE LENGTHS IN THIS SCHEDULE, UNLESS NOTED OTHERWISE.
2. FOR BARS WITH LOW COVER OR SPACING, INCREASE LENGTH BY 50%. IF BARS ARE ENCLOSED IN STIRRUPS OR TIES, INCREASE APPLIES WHEN BAR COVER OR CLEAR BAR SPACING IS LESS THAN db. IF BARS ARE NOT ENCLOSED IN STIRRUPS OR TIES, INCREASE APPLIES WHEN BAR COVER IS LESS THAN db OR CLEAR BAR SPACING IS LESS THAN 2db.
3. FOR TOP BARS INCREASE LENGTH BY 30%. A TOP BAR IS A HORIZONTAL BAR WITH MORE THAN 12 INCHES OF FRESH CONCRETE CAST BELOW IT.

ABBREVIATIONS:

DB (db) BAR DIAMETER
LD (L_d) TENSION DEVELOPMENT LENGTH
LA (L_a) CLASS A LAP SPLICE LENGTH
LB (L_b) CLASS B LAP SPLICE LENGTH
LDH (L_{dh}) TENSION DEVELOPMENT LENGTH FOR A STANDARD HOOK

CONCRETE COVER: UNLESS NOTED OTHERWISE, MINIMUM COVER FOR REINFORCING SHALL BE:

ABUTMENT 3" (EXCEPT 2" TOP AND FORMED SIDES)

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Ph: 425 741-3800

WASHINGTON STATE
DEPARTMENT OF FISH AND WILDLIFE

BRIAN C. MOON
STATE OF WASHINGTON
35945
PROFESSIONAL ENGINEER
3-15-12

SYM	DATE	REVISION DESCRIPTION	BY

APPROVED AND RELEASED FOR CONSTRUCTION

CHIEF ENGINEER .... DATE: ....

PROGRAM .... DATE: ....

0 — 1"
BAR MEASURES ONE INCH ON ORIGINAL DRAWINGS

DESIGNED BY LKL
CHECKED BY BCM
DRAWN BY DJO
DATE 03/15/12

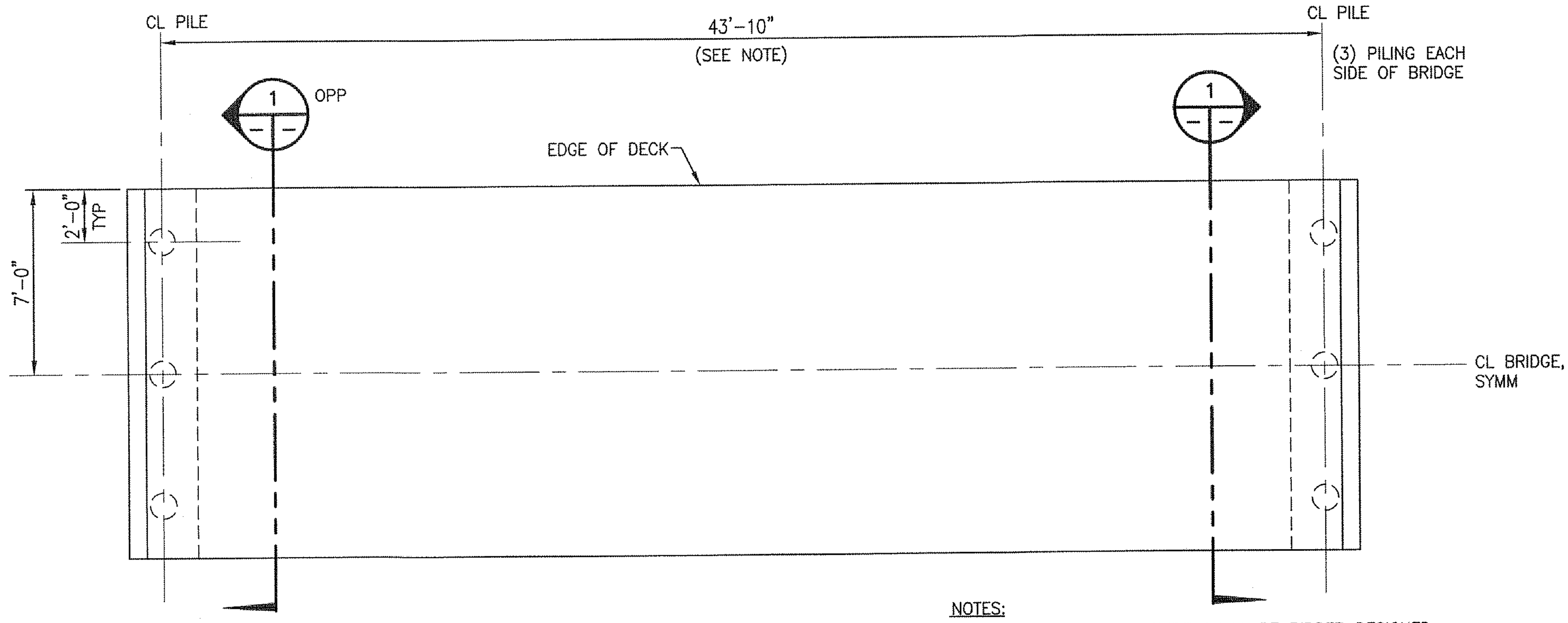
CHERRY VALLEY WILDLIFE AREA

WATERWHEEL CREEK BRIDGE

STRUCTURAL NOTES

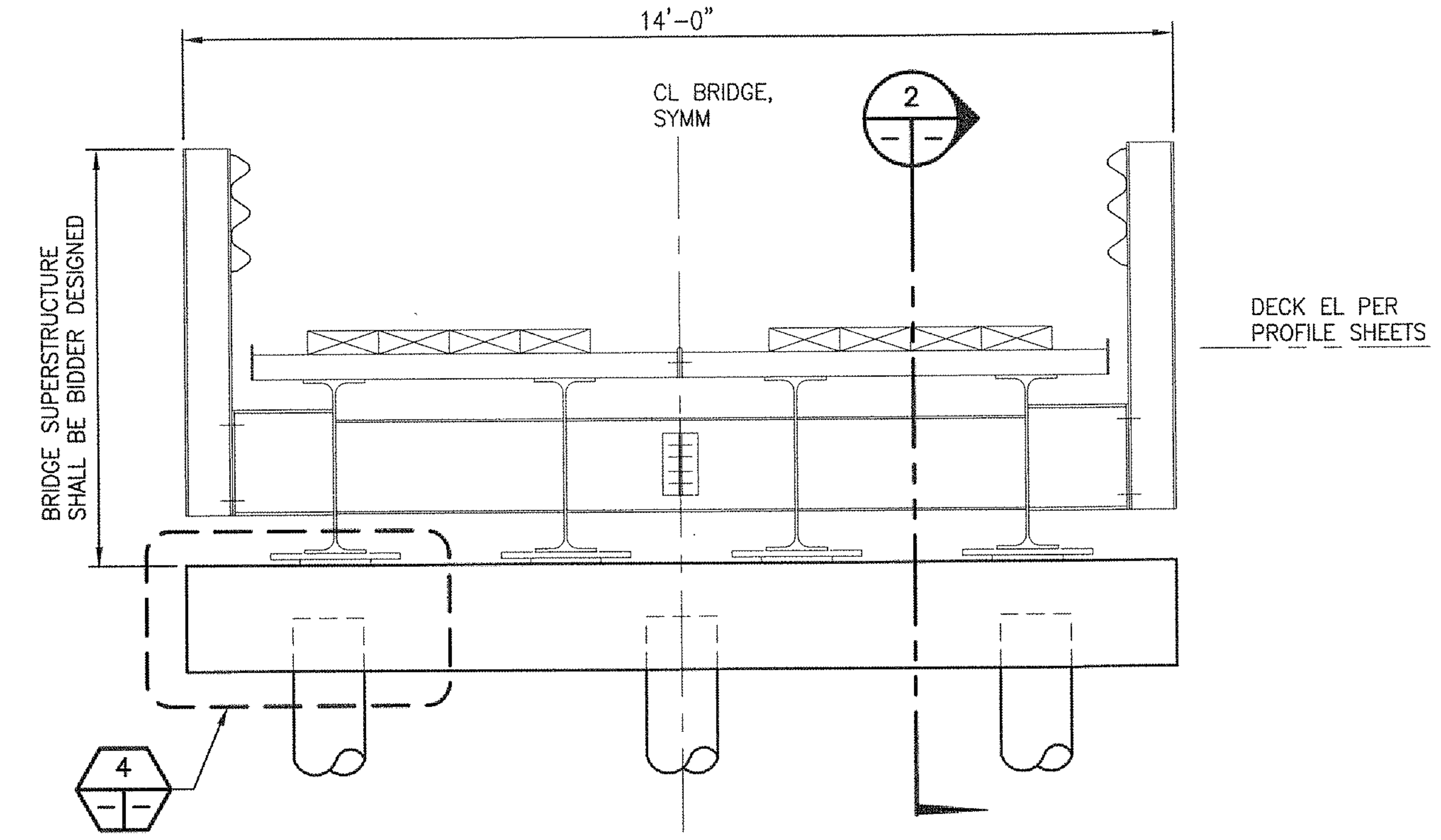
PROJECT NO. WM: R6: 10-1

SHEET 2 OF 3

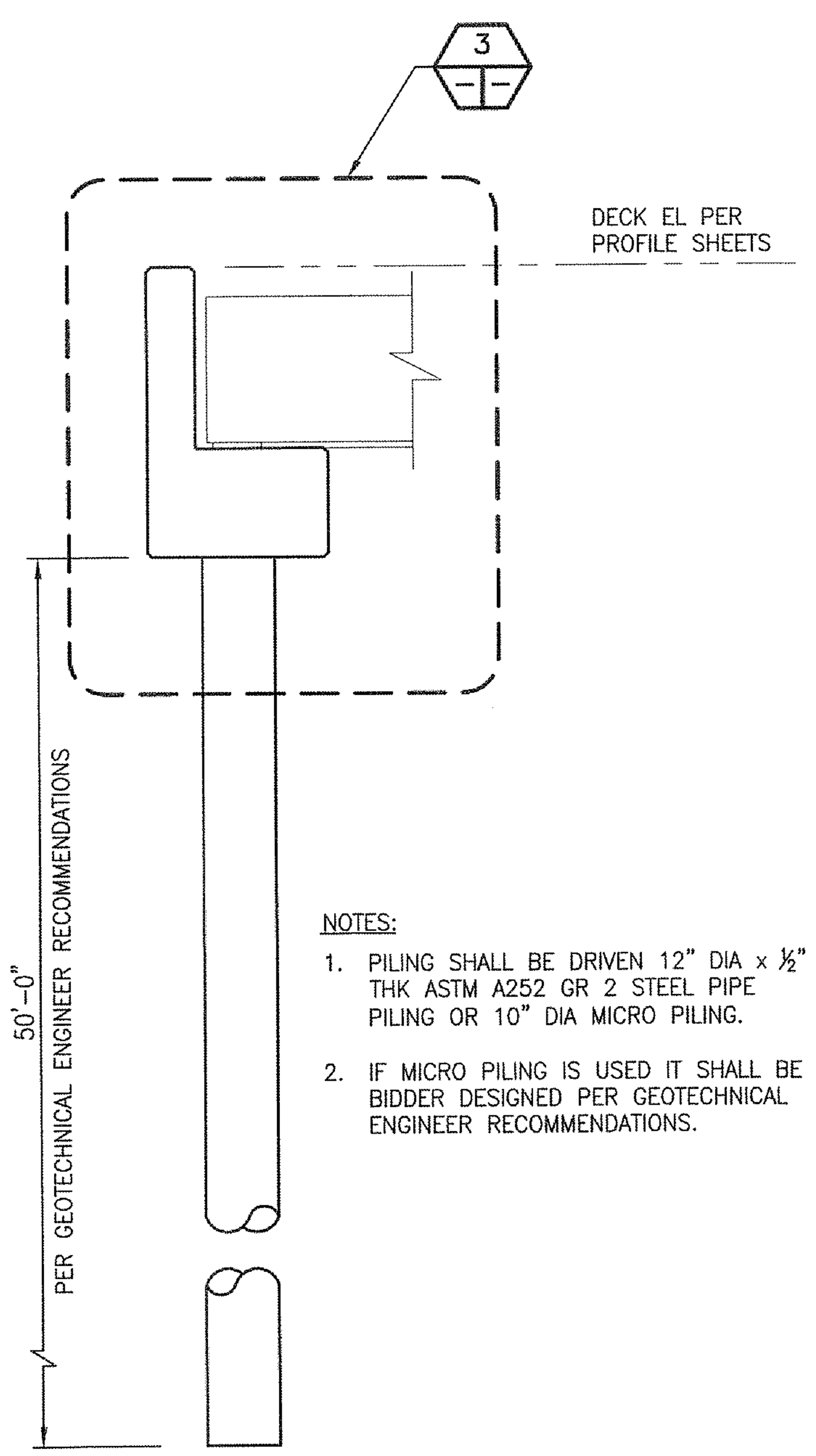


NOTES:
1. BRIDGE SUPERSTRUCTURE SHALL BE BIDDER DESIGNED.
2. GUARDRAIL AT EACH SIDE OF BRIDGE NOT SHOWN FOR CLARITY.
3. BRIDGE SUPERSTRUCTURE SHALL BE COORDINATED WITH THE DISTANCE BETWEEN PILING.

VEHICLE BRIDGE PLAN
SCALE: 1/4"=1'-0"

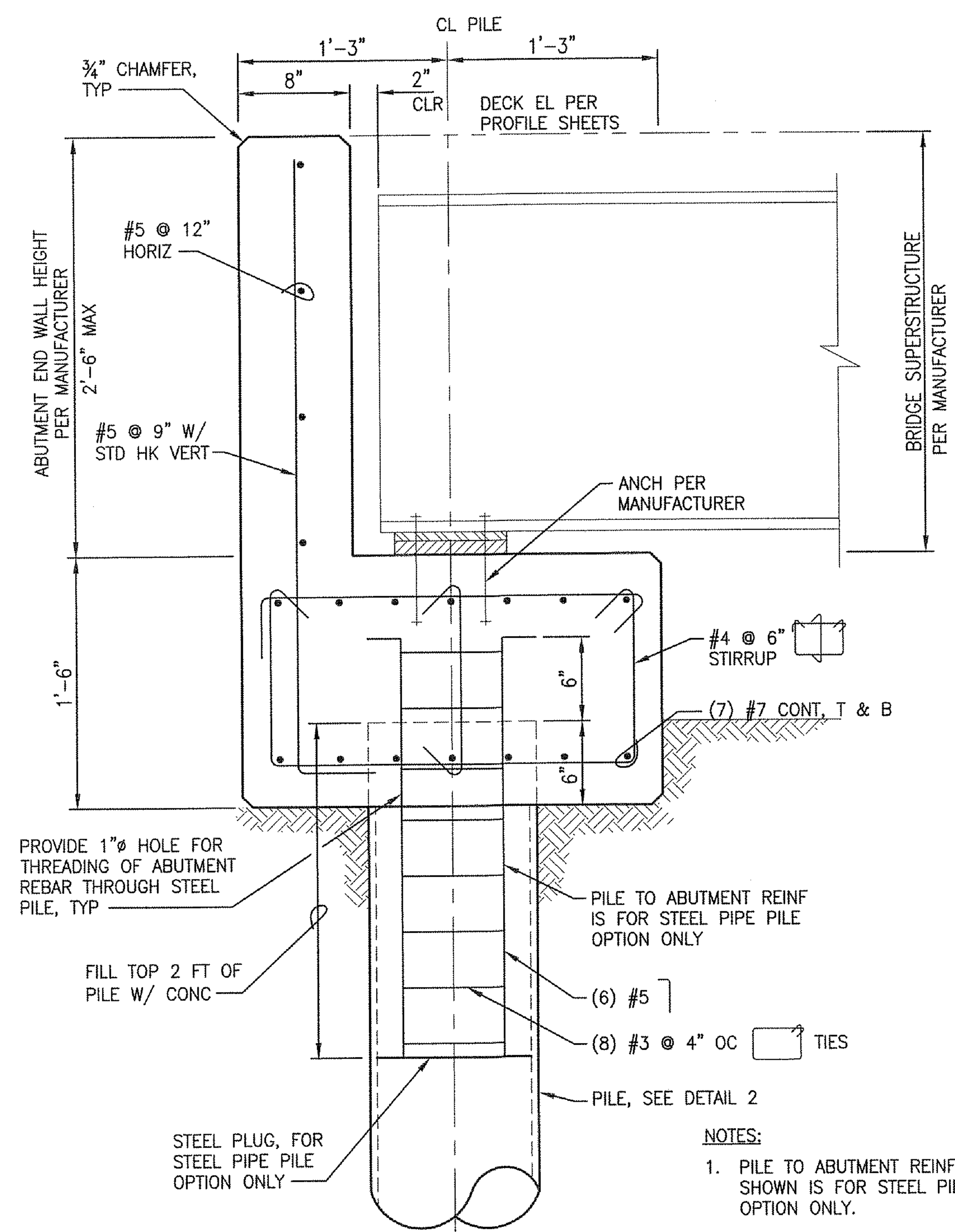


BRIDGE SECTION 1
SCALE: 1/2"=1'-0"



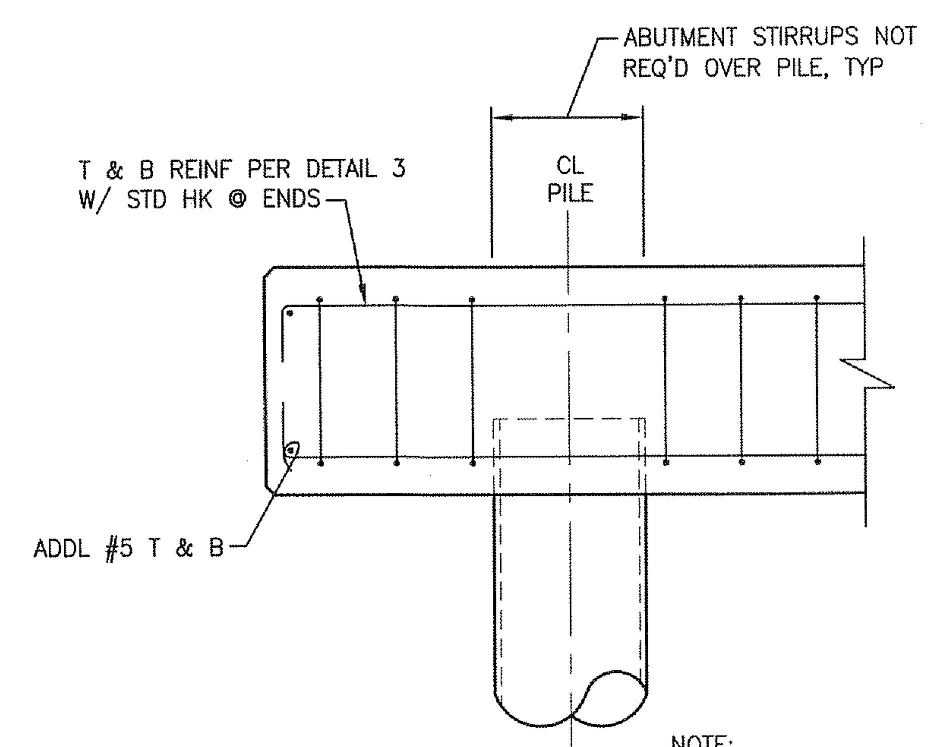
NOTES:
1. PILING SHALL BE DRIVEN 12" DIA x ½" THK ASTM A252 GR 2 STEEL PIPE PILING OR 10" DIA MICRO PILING.
2. IF MICRO PILING IS USED IT SHALL BE BIDDER DESIGNED PER GEOTECHNICAL ENGINEER RECOMMENDATIONS.

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



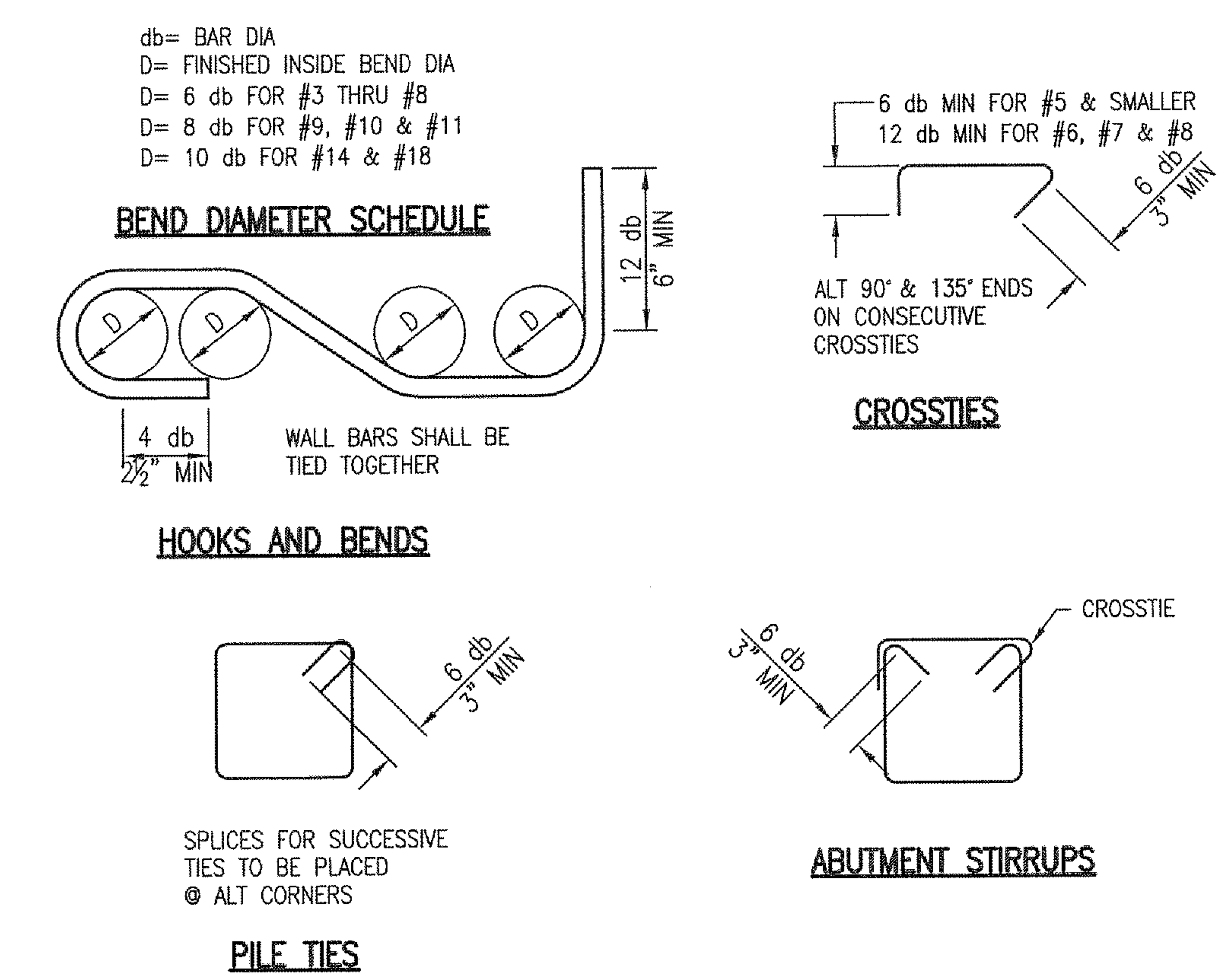
NOTES:
1. PILE TO ABUTMENT REINFORCEMENT SHOWN IS FOR STEEL PIPE PILE OPTION ONLY.
2. IF MICRO PILING IS USED, PILE TO ABUTMENT REINFORCEMENT SHALL BE BIDDER DESIGNED.

ABUTMENT DETAIL 3
SCALE: 1 1/2"=1'-0"



NOTE:
ABUTMENT END WALL NOT SHOWN FOR CLARITY.

ABUTMENT END 4
SCALE: 1"=1'-0"



TYPICAL REINFORCING BAR BEND AND LAP DETAILS 5
NOT TO SCALE

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WASHINGTON STATE
DEPARTMENT OF FISH AND WILDLIFE

BRIAN C. MOON — STATE OF WASHINGTON — PROFESSIONAL ENGINEER — 3-15-12

SYM	DATE	REVISION DESCRIPTION	BY
		APPROVED AND RELEASED FOR CONSTRUCTION	

CHIEF ENGINEER .... DATE: ....
PROGRAM .... DATE: ....

0 — 1"
BAR MEASURES ONE INCH ON ORIGINAL DRAWINGS

DESIGNED BY JJS
CHECKED BY HCT
DRAWN BY DJO
DATE 03/15/12

CHERRY VALLEY WILDLIFE AREA
WATERWHEEL CREEK BRIDGE
TYPICAL BRIDGE PLAN, SECTIONS AND DETAILS

PROJECT NO. WM: R6: 10-1
SHEET 3 OF 3

Waterwheel Creek Bridge

Bridge Abutment Design – Permit Submittal

March 2012

Prepared for:
Washington State Department of Fish and Wildlife

STRUCTURAL CALCULATIONS

BRIAN C. MOON
STATE OF WASHINGTON
35943
STRUCTURAL ENGINEER
PROFESSIONAL ENGINEER

Prepared By:

Brian C. Moon, P.E., S.E.
Bradley A. Martin, P.E., S.E.

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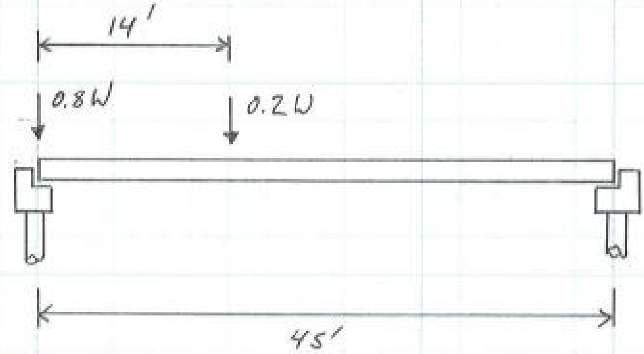
File No. 262012.024

ABUTMENT DESIGN

LOADS:



$W = 10T = 20k$ (10T 2-AXLE TRUCK)

$P_{LL} = 0.8W + 31/45 \times 0.2W = 18.8\ k$

$P_{DC} = 13.4\ k$

$P_{DW} = 37.4\ k$

NOTE: BRIDGE DECK & WEARING SURFACE WT EST BASED ON PRELIMINARY CONSTRUCTION

$IM = 33\%$

LOAD MODIFIERS:

$\eta_D = 1.0$ (CONVENTIONAL DESIGN)

$\eta_R = 1.0$ (CONVENTIONAL REDUNDANCY)

$\eta_I = 1.0$ (TYPICAL BRIDGE)

$\eta_i = \eta_D \eta_R \eta_I = 1.0$

PILE DESIGN:

$W_{CAP} \approx [(1ft \times 2.5ft) + (0.67ft \times 2ft)] \times 14ft \times 0.15kcf = 8k$

$Q = [1.0 \times (8k + 13.4k) + 1.0 \times 37.4k + 1.0 \times 18.8k \times 1.33]\eta_i$
$= 84k$

$\phi R_n = 1.0 \times 45k = 45k$ (SOIL CAPACITY FOR 12" PIPE PILE)

$N = Q/(\phi R_n - 8k) = 2.3$
DOWN-DRAG

NOTE: EST LOADS BASED ON LARGER BRIDGE.
ASSUME 2 PILES O.K.
INSTALL PILES TO DEEPER SOILS TO OBTAIN CAPACITY.

ALTERNATIVE 10" MICRO-PILE DESIGN:

$\phi R_n = 34k$

$N = Q/(\phi R_n - 7k) = 3.1$
DOWN-DRAG

∴ 3 PILES @ MIN 5' SPC

$P_{ALL} = 187k$ (STD 10" PIPE, $F_y = 35ksi$, $H_{EFF} = 20'$)

ALTERNATIVE 6" MICRO-PILE DESIGN:

$\phi R_n = 21k$

$N = Q/(\phi R_n - 4k) = 4.9$

∴ 5 PILES @ MIN 3' SPC

NOTE: 6" PILES LIKELY LACK LAT CAPACITY FOR DESIGN.
EST NUMBER IS FOR COMPARISON W/ LARGER PILES.

PILE LATERAL CHECK:

NOTE: ASSUME FIXITY OF PILE @ DEPTH OF 15D = 15 ft.

$(EQ)_{max} = 33\,k$ → DIVIDE TO (4) PILES, (2) EA ABUTMENT.

$M_u = (EQ)_{max} \times 15\,ft = 124\,k$.

$\phi M_n = 141\,ft \cdot k > M_u$

PILE COMBINED LOADING:

$P_u = [1.25 \times (8k + 13.4k) + 1.5 \times 37.4k + 1.0 \times 1.33 \times 18.8k] \times 1/2$
$= 54\,k$

$H_{EFF} = K \times 15\,ft = 12\,ft$

$k = 0.8$ (FIXED BOT, FREE TOP W/ TRANSLATIONAL RESTRAINT)

$\phi P_n = 405\,k$

$P_u/\phi P_n = 0.13 \Rightarrow \frac{1}{2} P_u/\phi P_n + M_u/\phi M_n = 0.94$ ∴ O.K.

∴ DUE TO POSSIBLE VARIATION IN ACTUAL DEAD LOAD FROM PROSPECTIVE BRIDGE MANUFACTURERS, USE 3-PILE ABUTMENT.

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Client	DFW	Sheet	4 of 7
Project	WATERWHEEL CREEK BRIDGE	Design by	BAM
		Date	03/14/2012
		Checked by	BCM
Project No.	26 2012.024	Date	3/15/12

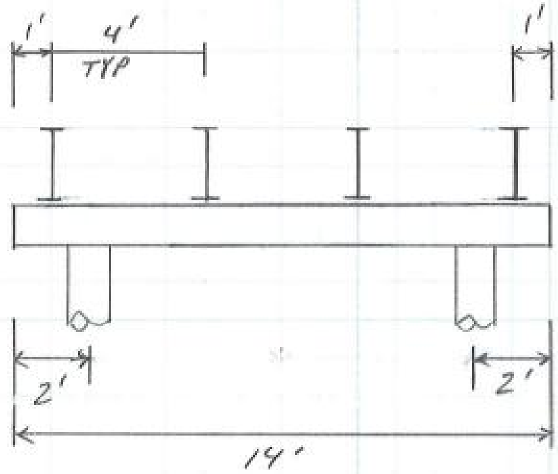
2006 IBC/ASCE 7-05 EARTHQUAKE FORCES

I.D.:

INPUT:				**Ref:**
Mapped short period accel. (T=0.2 s):	S_s =	0.81	g	11.4.1
Mapped acceleration @ T=1 s:	S_1 =	0.27	g	11.4.1
Redundancy Factor (SDC D, E, F)	ρ =	1	(1.0 or 1.3)	12.3.4
Site class:		E		11.4.2
Effective seismic weight:	W =	67	k	
Basic seismic-force-resisting system:	G. Cantilevered column system conforming to:			T12.2-1
Lateral system:	Cantilever Col-Detailed to Ordinary steel moment frames			T12.2-1
Occupancy Category:	I or II (All Buildings not III or IV, or Low Hazard (agriculature)			T 1-1
Building height:	h_n =	15	ft	12.8.2.1
Calculated fundamental period:	Tc =		sec (leave blank for auto calculation of T)	12.8.2
Response modification factor:	R =	1.25		T 12.2-1
System Overstrength Factor:	Ω =	1.25		T 12.2-1
Deflection Amplification Factor:	C_d =	1.25		T 12.2-1
ASCE 7 Section for detailing requirements:		14.1,12.2.5.2		T 12.2-1

BASE SHEAR CALCULATION:			**Ref:**
Acceleration site coefficient:	F_a =	1.13	T 11.4-1
Velocity site coefficient:	F_v =	2.92	T 11.4-2
Adjusted short period acceleration:	$S_{MS} = F_a{*}S_s$ =	0.914 g	EQ 11.4-1
Adjusted 1 sec. period acceleration:	$S_{M1} = F_v{*}S_1$ =	0.788 g	EQ 11.4-2
Design short period acceleration:	$S_{DS} = 2/3{*}S_{MS}$ =	0.609 g	EQ 11.4-3
Design 1 sec. period accel.:	$S_{D1} = 2/3{*}S_{M1}$ =	0.526 g	EQ 11.4-4
Control period 1:	$T_O = 0.2{*}S_{D1}/S_{DS}$ =	0.173 s	11.4.5
Control period 2:	$T_S = S_{D1}/S_{DS}$ =	0.863 s	11.4.5
Seismic importance factor:	I_E =	1	T 11.5-1
Seismic design category:	S_{DC} =	D	T 11.6-1,-2
Period coefficient:	C_t =	0.028	T 12.8-2
Period exponent:	x =	0.80	T 12.8-2
Approximate fundamental period:	$T_a = C_t{*}h_n^x$ =	0.244 s	EQ 12.8-7
Upper bound period coefficient:	C_u =	1.4	T 12.8-1
Upper bound period:	$T_u = C_u{*}T_a$ =	0.342 s	12.8.2
Fundamental period:	T =	0.244 s	12.8.2
Calculated seismic response coefficient:	$C_s = S_{DS}/(R/I_E)$ =	0.487	EQ 12.8-2
Upper bound response coefficient:	$C_s = S_{D1}/(R/I_E{*}T)$ =	1.721	EQ 12.8-3,-4
Lower bound response coefficient:	C_s =	0.010	EQ 12.8-5
For S1 ≥ 0.6 lower bound coefficient:	$C_s = 0.5{*}S_1/(R/I_E)$ =	DNA	EQ 12.8-6
Controlling lower bound response coeff	C_s =	0.010	
Controlling response coefficient:	C_s =	0.487	
Seismic base shear:	**$\rho V = \rho C_s W$ =**	**33 k**	EQ 12.8-1

CAP DESIGN:



NOTE: NUMBER & LOC OF BRIDGE GIRDERS ESTIMATED. APPLY FULL LIVE LOAD TO CENTER TWO GIRDERS.

$$Q_{PILE} = \left[0.6\left[1.25\times13.4k + 1.5\times37.4k\right] + 0.7\times1.25\times8k + 1.75\times1.33\times18.8k\right]\times 1/2$$

$$= 47\ k$$

$$V_u = 47\ k$$

$$M_u = 3\,ft \times 47k = 142\ ft\cdot k$$

FLEX REBAR:

$$(A_s)_{MIN} = \frac{12\,in/ft \times M_u}{(0.9)^2 \times 15\,in \times 60\,ksi} = 2.34\,in^2$$

∴ USE MIN (5) #7 T & B

SHEAR REBAR:

$$A_V = \frac{V_u \times 6\,in}{0.75\times15\,in\times60\,ksi} = 0.42\,in^2$$

∴ USE #4 @ 6" SPC w/ 3rd CTR BAR

Client	DFW	Sheet	6 of 7
Project	WATERWHEEL CREEK BRIDGE	Design by	BAM
		Date	03/14/2012
		Checked by	BCM
Project No.	262012.024	Date	3/15/12

DEFLECTION CHECK:

NOTE: CALC w/ ENTIRE TRIB LOAD @ MID-PT. $\Delta = PL^3/48EI$

$P = 0.7 \times 8k + 0.6 \times (13.4k + 37.4k) + 18.8k = 55k$

$L = 120\ in$

$E = 57\sqrt{f'_c} = 3{,}600\ ksi$

$f'_c = 4{,}000\ psi$

$I_{CR} = 0.5 I_g = 5{,}830\ in^4$

$I_g = 24\ in \times (18\ in)^3/12 = 11{,}660\ in^4$

$\Delta = \dfrac{PL^3}{48EI} = 0.09\ in \approx 1/8\ in$ $\therefore$ O.K.

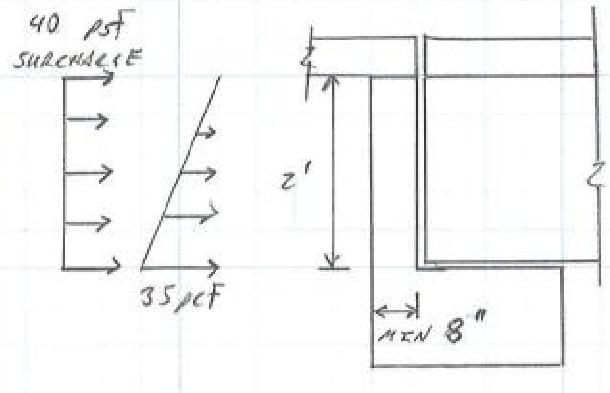
RETAINING WALL:

$F_a = 35_{pcf} \times (2ft)^2 \times 1/2$

$= 70$ #/ft (ACTIVE SOIL PRESSURE)

$F_s = 40_{psf} \times 2ft$

$= 80$ #/ft (VEHICLE SURCHARGE)



$M_u = 2ft \times [(1/3 \times F_a \times 1.5) + (1/2 \times F_s \times 1.75 \times 1.33)]$

$= 0.266$ ft·k/ft

$(A_s)_{MIN} = \dfrac{12\,in/ft \times M_u}{(0.9)^2 \times 60\,ksi \times 4\,in} = 0.016\ in^2/ft$ ∴ USE MIN #4 EW

CHECK SHEAR:

$V_u = 1.5F_a + 1.75 \times 1.33 F_s = 0.29$ k/f ∴ LOAD IS LOW - WALL STRENGTH O.K.

CHECK DEFLECTION @ TOP OF WALL:

NOTE: CALC I AS $\frac{1}{2} I_g$ USING $H = d = 4in$

$\Delta = \dfrac{Pb^2}{6EI}(3L - b) = 0.002$ in ∴ DEFLECTION SMALL → O.K.

$P = 0.15$ k

$I = 1/2 \times 12_{in} \times (4_{in})^3/12 = 32\ in^4$

$E = 3{,}600$ ksi

$L = 24$ in ; $b = 12$ in

COST ESTIMATE FOR WATERWHEEL CREEK BRIDGE

ITEMS	COST	UNIT	QTY	TOTAL
14’ x 45’ bridge (including deck and rails)	62,000	ea	1	62,000
Piling (installed)	8,000	ea	6	48,000
Concrete abutment	7,500	ea	2	15,000
Gravel	15	ton	45	675
Bridge Installation (crane or 2 excavators)	7,000	ls	1	7,000
Mob/Demob	7,500	ls	1	7,500
Subtotal for Construction				140,175
Contingency			10%	14,017.5
Tax			9%	12615.75
			TOTAL	**166,808.25**

Washington Fish & Wildlife
Tunk Creek Bridge
70'-0" x 14'-0" Bridge

Engineering Design Calculations

Prepared By:

John W. Stiles, P.E.
N6940 Warren Drive
Hartland, WI 53029
262-538-2668

August 30, 2005

BIG R MANUFACTURING, LLC
GREELEY, CO
BR4225

Input Calcs - Pages D1, D2
AISI Design Check 1 – 12
Drawings 1 - 3

JOHN W. STILES
STATE OF WASHINGTON
9/23/05
16496
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 12/23/06

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262-538-2668
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JOB No.	BR4225	SHEET No. D- 1	
CLIENT	Big R Manufacturing, LLC		
PROJECT	Wash Fish & Wildlife, Tunk Creek		
SUBJECT	AISI Input		
CALCS BY	JWS	DATE	08/29/05
CHK'D BY		DATE	
70'-0" x 14'-0"			

Design of Corrugated Metal Deck and Input for AISI Bridge Program

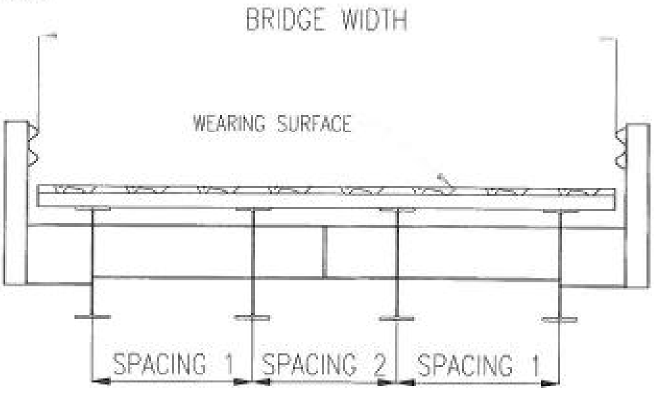
Written By	JWS	April-01	Input	Check	Critical Output

GENERAL INPUT:

	AASHTO		
Roadway Width :	14.00	feet	
Number of Beams :	4		
Number of Modules :	2		
Span Length :	70.00	feet	
Design Truck :	HS20		
Skew :	0.00	Degrees	
Wt of Bridge Deck (if nor BigR deck)		lb/ft^2	
Depth Wearing Surf (Asp, Gra)	12.00	inches	
Wear. Surf Unit Wt :	7.20	lb/ft^3	
End of Beam - CL Brg	5	inches	
Est. Beam Flg width	12	inches	
Number of traffic Lanes	1		

STANDARD ITEMS USED - CHANGE IF REQ'D

Guard Rail :	12 Ga W-Beam		6.82	lb/ft
Guard Posts:	W8x24	Input Wt=>	24	lb/ft
Post Blocks:	W12x26	Input Wt=>	26.0	lb/ft
Post Spa.	6.25	Std	0.0	<=Other
Top Rail Ht	2.67	Ft Std	0.0	<=Other
Diaph Size	W16x26	Input Wt=>	26.0	<=Other
No Diaph's	3.80	4		
Additional DL	20	psf (req'd)		
Post Length	5.1045	ft		
Overhang	1.1667	ft		
Post Blk	1.4480	ft		



BRIDGE CROSS SECTION

14-ft Width	3.8333	4.0000	3.8333	14.00	1.1667
16-ft Width	4.1667	4.6667	4.1667	16.00	1.5000
Beam Spacing	4.9167	No Spa =	2	OH =	1.1650

Other Width Input

Check Deck Thickness:

Design Truck	HS20	Wheel Load	20.80	kips
Net Span	2.833	A @ NA_{DECK}	1921.23	in^2
		Tire Load(ksf)	1.56	
$M_{LL}=R_1(L/2)-(wL_w^2/2)$ =		1.16	ft-kips (w/ 1.3 Impact)	
Use BigR Deck (Yes or No)		Yes		
Area of deck corrugation		23.07	in^2	
Wt of Deck + Corr Fill		10.75	lb/ft	Use 7Ga
Wt of Wear Surface		7.20	lb/ft	
Wt of Future WS		20.00	lb/ft	
Total Dead Load		37.95	lb/ft	
$M_{DL}=wL^2/8$ =		0.04	ft-kips	
$\Sigma M_{DL}+M_{LL}$ =		1.20	ft-kips	
$S_{REQ'D}=M*12/(0.60*F_y)$		0.481	in^3	
Use What Deck Thick		9 Ga	OK	

Corrugated Deck Design Properties:

Material	Thickness	Yield Stress F_y	Weight (psf)	S_x (in^3)
9 Ga	0.149	50.0	9.6	3.88
7 Ga	0.179	50.0	11.5	4.63

Truck Type:	Front Wheel (kips)[1]	Rear Whls(kips)[1]	Rear Whl Area[1]	L^2 (in)	W^2 (in)
H10	2.6	10.4	104	34.85	44.52
H15	3.99	15.6	156	36.30	48.15
H20	5.2	20.8	208	37.52	51.20
HS15	3.9	15.6	156	36.30	48.15
HS20	5.2	20.8	208	37.52	51.20
HS25	6.5	26	260	38.60	53.90
HS30	7.8	31.2	312	39.57	56.33
U80	7.8	24.05	240.5	38.21	52.92
U102	5.2	36.4	364	40.47	58.57
U150	7.8	53.3	533	43.00	64.90

Note:[1] Includes 1.3 Impact Factor, [2] Includes wearing surface + 2.2" to CG of Deck Section

Compute Dead Load of Bridge:

			Total Wt (lb)	Per Ft	
Corrugated Deck Wt			9408	136.02	
Fill & Wear Surface			8186	118.36	
				0.00	
				0.00	<=Input weight per foot of backing tubes or other rail weights (curbs, etc)
Rail Posts	Est No =	13	3185	46.05	Input Actual if more
Post Block Bms Est=		13	979	14.15	Input Actual if more
Diaphragms			1153	16.66	
Dead Load per Foot of Bridge =				331.24	lb/ft
Dead Load per Foot of Bridge per Beam =				82.81	lb/ft Span C-C Brg = 69.1667 Ft
Future Wearing Surface Load per Beam =				70.00	lb/ft

6.92	0.1 L
13.83	0.2 L
20.75	0.3 L
27.67	0.4 L
29.95	0.433 L
32.28	0.4667 L
34.58	0.5 L
36.87	0.533 L
39.20	0.5667 L
41.50	0.6 L
48.42	0.7 L
55.33	0.8 L
62.25	0.9 L

LIVE LOAD DISTRIBUTION FACTORS:

Ref: Table 3.23.1 from AASHTO for Corrugated Steel Deck, One Lane Bridge

Momemt Distribution Factor for 1 Lane = S/5.5			2 Lanes = S/4.5	Lane Reductions 1.00
Average Beam Spacing	3.89	ft		
Moment Dist Factor = S/	5.5	0.707		
Shear Distribution Factor =		1.00	Bm Spa < 6.0-ft use 1	
Deflection Distribution Factor =		0.50	# Lines of Wheels/# Beams	

BRACING:

Since the top flange of the beams are welded to each corrugation of the bridge deck, it can be assumed that the top flange is continuously braced. The AISI Bridge Design Program requires a finite length for input for unbraced section calculations. For simplicity, use 1/10 of span for a point of bracing.

Unbraced Length = L/10 = 6.917 ft

6.9167	13.8333	20.7500	27.6667	34.5833	41.5000	48.4167	55.3333	62.2500	69.1667

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JOB No.	BR4225	SHEET No. D- 1	
CLIENT	**Big R Manufacturing, LLC**		
PROJECT	Wash Fish & Wildlife, Tunk Creek		
SUBJECT	AISI Input		
CALCS BY	JWS	DATE	08/29/05
CHK'D BY		DATE	
70'-0" x 14'-0"			

RESULT SUMMARY FROM AISI BRIDGE DESIGN PROGRAM:

Specified Deflection = L/	500	=	1.680	inches	W36x230
Actual LL Defl (AISI) =	1.418	inches = L/	592	OK	
Maximum DL Defl =	0.604	inches =	3/5		

Computed Impact =	1.256			
Live Load + Impact Reaction From AISI:	29.64	kips Shear	30.30	kips Reaction
Dead Load Reaction From AISI :	6.94	kips		
Superimposed DL From AISI :	0.00	kips		
Live Load Reaction (No Impact):	23.59	kips		

Bridge Ratings:

Inventory Rating	0.00	Tons
Operating Rating	0	Tons

ELASTOMERIC BEARING PAD DESIGN:

Design is in accordance with AASHTO Section 14.6.6.1, Method A

Shore Durometer Hardness Used:	60	
Shear Modulus (minimum) :	130	psi
Shear Modulus (maximum) :	200	psi
Span Length : c-c Bearing	69.1667	Ft

Durometer Hardness	G_{MIN}	G_{MAX}
50	90	150
60	130	200
70	150	250

		Min °F		Max °F
Temperature Range to use	Extreme	-30	TO	120

Moderate (0° to 120°)
Extreme (-30° to 120°)

Coefficient of Thermal Expansion for Steel 6.50E-06 Inches per Degree F

Temperature Movement = $\Delta_s = 0.0000065L(\delta T) =$ 0.405 inches Minimum Pad Thickness = $h_{rt} = 2\Delta_s$ 0.809 inches

Use reinforced pad ? (yes or no) Yes

		0	inches
Reinforced Pads Input =>	Thick Outside Layers	0.5	inches
	Thick Inside Layers	0	inches
	Estimate # Inside Layers	0	
	Thickness of steel shims	0.125	inches
	Total Pad Thickness =	1.125	inches

Total Elastomer thickness = 1 OK

Estimated Pad Dimensions:

Length in direct of span	8	inches
Width (Transverse to span)	12	inches
Layer Thick	0.5	inches
Total Thick	0.5	inches

Shape Factor = $S = \dfrac{LW}{2t_{MAX}(L+W)} =$ 4.80

t in Formula = 0.5

Beam Reactions: (From above)				Pad Stress
	Dead Load	6.94	kips	0.072
	Future DL	0	kips	0.000
	LL + I	30.30	kips	0.316

Reinforced Pad Stresses
Allowable Stress <= 1000 psi or 1000GS psi
Use 624.00 psi

Compressive Stress Design:

Allowable Pad Stress =	624.00	psi	
Actual Pad Stress =	387.95	psi	OK

Rotational Capacity Check:

At placement of beams dead load rotation is zero, since it has already taken place, design for LL only

Max LL Defl = L/	592	=	1.40	inches
$\phi = \dfrac{4\Delta}{L} =$ Rotation of Pad =	0.0068	radians		
Construction Rotation	0.0004	radians (Estimated)		
Total Rotation =	0.0072	radians		
$\sigma_{TL} \geq 0.5GS\left(\dfrac{L}{h_{ri}}\right)^2 \dfrac{\theta_{m,s}}{n}$ = Rotational Stress =	571	psi	< Allowable, OK	Number of effective layer, "n" = 1

Stability Check per 14.6.6.3.6 — Stability Checks OK

$h_s > \dfrac{3.0h_{r\,max}\sigma_{TL}}{F_y}$ 0.026 inches or $h_s > \dfrac{2.0h_{r\,max}\sigma_{LL}}{F_{sr}}$ 0.013 inches F_{sr} used = 24000 Shim thickness OK

Approximate Modulus of Elasticity of pad $E_c = 6GS^2$ 17971 psi

Moment transferred by pad to superstruct $M_m = \dfrac{0.5E_c I\theta_m}{h_{rt}}$ = 32904 in-lbs

Longit Force transferred to substructure $H_m = \dfrac{GA\Delta_s}{h_{rt}}$ = 5050 lbs

```
Date:          08-30-2005
Time:          10:48:36
INPUT FILE NAME:C:\Program Files\AisiBeam\BR4225 Wash
OUTPUT FILE NAME:C:\Program Files\AisiBeam\BR4225 Wash.out

     *********************************************************************
     *                                                                   *
     *                 AISI SHORT SPAN BRIDGE DESIGN PROGRAM             *
     *     AASHTO SPECIFICATION FOR HIGHWAY BRIDGES (96,97,98 and 99)    *
     *     LOAD FACTOR DESIGN FOR COMPOSITE AND NON-COMPOSITE GIRDERS    *
     *            SOFTWARE USES GROUP I LFD LOAD COMBINATION             *
     *                            VERSION 3.0                            *
     *          COPYRIGHT 2000 AMERICAN IRON AND STEEL INSTITUTE         *
     *                                                                   *
     *********************************************************************


WAshington Fish & Wildlife,Tunk Creek Bridge
70'-0" x 14'-0"    U80
BigR Manufacturing, LLC     BR4225
John W Stiles, PE    8/29/05


****************
* INPUT DATA:  *
****************

    INPUTS AND OUTPUTS ARE IN U.S. CUSTOMARY UNITS       U.S. UNITS
    EFFECTIVE WIDTH IS SUPPLIED BY:                      THE USER
    DISTANCE BETWEEN CROSS BRACINGS ARE SUPPLIED BY:     THE USER
    VALUE OF MODULAR RATIO (N)IS SUPPLIED BY:            THE USER
    DISTRIBUTION FACTORS ARE SUPPLIED BY:                THE USER
    IMPACT FACTOR IS CALCULATED BY:                      THE PROGRAM
    SPAN LENGTH  (FT):                                   69.1667
    MOMENT DISTRIBUTION FACTOR:                          0.5
    SHEAR DISTRIBUTION FACTOR:                           1
    DEFLECTION DISTRIBUTION FACTOR:                      0.5
    NUMBER OF SEGMENTS IN SPAN:                          10
    DEAD LOAD (EXCLUDING SELF WEIGHT) PER GIRDER (K/FT) 0.0828
    SUPERIMPOSED DEAD LOAD  (K/FT):                      0
    FATIGUE VEHICLE:                                     HS - 20
    LIVE LOAD VEHICLE :                                  HS - 20
    TYPE OF STEEL:                                       M 270 GRADE 50W
    BRIDGE TYPE:                                         NON-COMPOSITE
    GIRDER TYPE:                                         ROLLED BEAM
     DEFLECTION CRITERIA WITH DEFLECTION/SPAN RATIO OF     1/ 500 IS CHECKED
    ROAD CASE:                                           CASE II
    CALCULATION TYPE:                                    RATING
    AASHTO CONSTRUCTIBILITY CRITERIA:                    IS NOT APPLICABLE
    ROLLED BEAM SIZE:                                    W33X118
    LENGTH OF COVER PLATE  (FOOT):                       0
    WIDTH OF COVER PLATE  (IN.):                         0
    COVER PLATE THICKNESS  (IN.):                        0

                    BRACING INFORMATION TABLE

    BRACE                                          DISTANCE FROM
    NUMBER                                         LEFT SUPPORT (FT)

     1                                             0.00
     2                                             6.92
     3                                             13.83
     4                                             20.75
     5                                             27.67
     6                                             29.95
     7                                             32.28
     8                                             34.58
     9                                             36.87
```

10	39.20
11	41.50
12	48.42
13	55.33
14	62.25
15	69.17

PRINT THE FOLLOWING SECTIONS OF THE OUTPUT FILE

PART I
PART II
PART V
PART VI

****
* OUTPUT DATA: *
****

TOTAL DEAD LOAD (INCLUDING SELF WEIGHT) PER GIRDER (K/FT) 0.20

****

PART I

INFORMATION REQUIRED FOR DESIGN DOCUMENTS

****

^^^

VEHICLE INVENTORY RATING IN TONS = 76.50

VEHICLE OPERATING RATING IN TONS = 127.76

^^^

SERVICE DEFLECTION PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM LIVE LOAD DEF. (IN.)	DEAD LOAD TOTAL (IN.)	DEAD LOAD STEEL ONLY (IN.)	DEAD LOAD SLAB ONLY (IN.)	SUPER-IMPOSED LOAD (IN.)	TOTAL DEF. PER GIRDER (IN.)
0	0.00	0.000	0.000	0.000	0.000	0.000	0.000
1	6.92	0.440	0.190	0.111	0.078	0.000	0.630
2	13.83	0.838	0.359	0.211	0.148	0.000	1.197
3	20.75	1.150	0.491	0.289	0.203	0.000	1.641
4	27.67	1.352	0.576	0.338	0.237	0.000	1.928
5	34.58	1.418	0.604	0.355	0.249	0.000	2.022
6	41.50	1.352	0.576	0.338	0.237	0.000	1.928

7	48.42	1.150	0.491	0.289	0.203	0.000	1.641
8	55.33	0.838	0.359	0.211	0.148	0.000	1.197
9	62.25	0.440	0.190	0.111	0.078	0.000	0.630
10	69.17	0.000	0.000	0.000	0.000	0.000	0.000

SERVICE LOAD REACTIONS PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD REACTION PER GIRDER (KIPS)	SUPER-IMPOSED LOAD PER GIRDER (KIPS)	LIVE LOAD PER PER GIRDER (KIPS)	MAXIMUM TOTAL LOAD PER GIRDER (KIPS)	MINIMUM TOTAL LOAD PER GIRDER (KIPS)
0	6.94	0.00	29.64	36.59	6.94
69.1667	6.94	0.00	29.64	36.59	6.94

TOTAL GIRDER WEIGHT IS 4.081 TONS

PART II
MOMENT AND SHEAR INFORMATION

SERVICE MOMENT PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM LIVE LOAD MOMENT PER GIRDER (K-FT)	DEAD LOAD MOMENT PER GIRDER (K-FT)	SUPERIMPOSED DEAD LOAD MOMENT (K-FT)	TOTAL MOMENT PER GIRDER (K-FT)
0	0.00	0.00	0	0	0.00
1	6.92	119.78	43.23	0	163.01
2	13.83	208.24	76.85	0	285.09
3	20.75	265.40	100.87	0	366.27
4	27.67	298.28	115.28	0	413.56
5	34.58	303.37	120.08	0	423.45
6	41.50	298.28	115.28	0	413.56
7	48.42	265.40	100.87	0	366.27
8	55.33	208.24	76.85	0	285.09
9	62.25	119.78	43.23	0	163.01
10	69.17	0.00	0	0	0.00

SERVICE TRUCK OR LANE LOAD MOMENT PER GIRDER INFORMATION TABLE

(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM TRUCK MOMENT (K-FT)	MINIMUM TRUCK MOMENT (K-FT)	TRUCK LIVE MOMENT RANGE (K-FT)	MAXIMUM LANE MOMENT (K-FT)	MINIMUM LANE MOMENT (K-FT)	LANE LIVE MOMENT RANGE (K-FT)	LIVE LOAD IMPACT
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26
6.92	119.78	0.00	119.78	78.54	0.00	78.54	0.26
13.83	208.24	0.00	208.24	139.63	0.00	139.63	0.26
20.75	265.40	0.00	265.40	183.26	0.00	183.26	0.26
27.67	298.28	0.00	298.28	209.44	0.00	209.44	0.26
34.58	303.37	0.00	303.37	218.17	0.00	218.17	0.26
41.50	298.28	0.00	298.28	209.44	0.00	209.44	0.26
48.42	265.40	0.00	265.40	183.26	0.00	183.26	0.26
55.33	208.24	0.00	208.24	139.63	0.00	139.63	0.26
62.25	119.78	0.00	119.78	78.54	0.00	78.54	0.26
69.17	0.00	0.00	0.00	0.00	0.00	0.00	0.26

SERVICE FATIGUE TRUCK AND LANE LOAD MOMENT RANGES PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM FATIGUE TRUCK MOMENT (K-FT)	MINIMUM FATIGUE TRUCK MOMENT (K-FT)	FATIGUE TRUCK MOMENT RANGE (K-FT)	MAXIMUM FATIGUE LANE MOMENT (K-FT)	MINIMUM FATIGUE LANE MOMENT (K-FT)	FATIGUE LANE MOMENT RANGE (K-FT)	LIVE LOAD IMPACT
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.26
6.92	119.78	0.00	119.78	78.54	0.00	78.54	0.26
13.83	208.24	0.00	208.24	139.63	0.00	139.63	0.26
20.75	265.40	0.00	265.40	183.26	0.00	183.26	0.26
27.67	298.28	0.00	298.28	209.44	0.00	209.44	0.26
34.58	303.37	0.00	303.37	218.17	0.00	218.17	0.26
41.50	298.28	0.00	298.28	209.44	0.00	209.44	0.26
48.42	265.40	0.00	265.40	183.26	0.00	183.26	0.26
55.33	208.24	0.00	208.24	139.63	0.00	139.63	0.26
62.25	119.78	0.00	119.78	78.54	0.00	78.54	0.26
69.17	0.00	0.00	0.00	0.00	0.00	0.00	0.26

SERVICE FATIGUE TRUCK AND LANE LOAD MOMENT RANGES PER GIRDER AT DIAPHRAGMS INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

CROSS FRAME OR DIA-PHRAGM NUMBER	DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM FATIGUE TRUCK MOMENT (K-FT)	MINIMUM FATIGUE TRUCK MOMENT (K-FT)	FATIGUE TRUCK MOMENT RANGE (K-FT)	MAXIMUM FATIGUE LANE MOMENT (K-FT)	MINIMUM FATIGUE LANE MOMENT (K-FT)	FATIGUE LANE MOMENT RANGE (K-FT)
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	6.92	119.78	0.00	119.78	78.54	0.00	78.54
3	13.83	208.20	0.00	208.20	139.60	0.00	139.60
4	20.75	265.40	0.00	265.40	183.26	0.00	183.26
5	27.67	298.28	0.00	298.28	209.44	0.00	209.44
6	29.95	299.96	0.00	299.96	212.32	0.00	212.32
7	32.28	301.68	0.00	301.68	215.26	0.00	215.26
8	34.58	303.37	0.00	303.37	218.17	0.00	218.17
9	36.87	301.69	0.00	301.69	215.28	0.00	215.28
10	39.20	299.98	0.00	299.98	212.34	0.00	212.34

11	41.50	298.28	0.00	298.28	209.44	0.00	209.44
12	48.42	265.40	0.00	265.40	183.26	0.00	183.26
13	55.33	208.27	0.00	208.27	139.65	0.00	139.65
14	62.25	119.78	0.00	119.78	78.54	0.00	78.54
15	69.17	0.00	0.00	0.00	0.00	0.00	0.00

OVERLOAD FACTORED MOMENT PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	1.67*MAX. LIVE LOAD MOMENT (K-FT)	DEAD LOAD MOMENT PER GIRDER (K-FT)	SUPERIMPOSED DEAD LOAD MOMENT (K-FT)	TOTAL FACTORED MOMENT (K-FT)
0	0.00	0.00	0	0	0.00
1	6.92	200.03	43.23	0	243.26
2	13.83	347.76	76.85	0	424.61
3	20.75	443.22	100.87	0	544.09
4	27.67	498.13	115.28	0	613.41
5	34.58	506.63	120.08	0	626.71
6	41.50	498.13	115.28	0	613.41
7	48.42	443.22	100.87	0	544.09
8	55.33	347.76	76.85	0	424.61
9	62.25	200.03	43.23	0	243.26
10	69.17	0.00	0	0	0.00

MAXIMUM FACTORED MOMENT PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	1.67*1.3* MAX. LIVE LOAD MOMENT (K-FT)	1.3*DEAD LOAD MOMENT (K-FT)	1.3*SUPER-IMPOSED MOMENT (K-FT)	TOTAL MAX. FACTORED LOAD MOMENT (K-FT)
0	0.00	0.00	0.00	0.00	0.00
1	6.92	260.04	56.20	0.00	316.24
2	13.83	452.09	99.90	0.00	551.99
3	20.75	576.18	131.13	0.00	707.31
4	27.67	647.57	149.86	0.00	797.43
5	34.58	658.62	156.10	0.00	814.72
6	41.50	647.57	149.86	0.00	797.43
7	48.42	576.18	131.13	0.00	707.31
8	55.33	452.09	99.90	0.00	551.99
9	62.25	260.04	56.20	0.00	316.24
10	69.17	0.00	0.00	0.00	0.00

SERVICE SHEAR PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	MAXIMUM LIVE LOAD SHEAR (KIPS)	MINIMUM LIVE LOAD SHEAR (KIPS)	LIVE LOAD SHEAR RANGE (KIPS)	DEAD LOAD SHEAR (KIPS)	SUPERIMPOSED LOAD SHEAR (KIPS)
0	29.64	0.00	29.64	6.94	0
1	17.32	-1.01	18.32	5.56	0
2	15.05	-2.01	17.07	4.17	0
3	12.79	-4.00	16.79	2.78	0
4	10.53	-6.01	16.54	1.39	0
5	8.26	-8.26	16.53	0	0
6	6.01	-10.53	16.54	-1.39	0
7	4.00	-12.79	16.79	-2.78	0
8	2.01	-15.05	17.07	-4.17	0

9	1.01	-17.32	18.32	-5.56	0
10	0.00	-29.64	29.64	-6.94	0

SERVICE TRUCK AND OR LANE LOAD SHEAR RANGES PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM TRUCK SHEAR (KIPS)	MINIMUM TRUCK SHEAR (KIPS)	TRUCK LIVE SHEAR RANGE (KIPS)	MAXIMUM LANE SHEAR (KIPS)	MINIMUM LANE SHEAR (KIPS)	LANE LIVE SHEAR RANGE (KIPS)	LIVE LOAD IMPACT
0.00	29.64	0.00	29.64	23.31	0.00	23.31	0.26
6.92	17.32	-1.01	18.32	12.99	-0.89	13.88	0.26
13.83	15.05	-2.01	17.07	10.99	-1.91	12.91	0.26
20.75	12.79	-4.00	16.79	9.13	-3.08	12.21	0.26
27.67	10.53	-6.01	16.54	7.41	-4.38	11.79	0.26
34.58	8.26	-8.26	16.53	5.83	-5.83	11.65	0.26
41.50	6.01	-10.53	16.54	4.38	-7.41	11.79	0.26
48.42	4.00	-12.79	16.79	3.08	-9.13	12.21	0.26
55.33	2.01	-15.05	17.07	1.91	-10.99	12.91	0.26
62.25	1.01	-17.32	18.32	0.89	-12.99	13.88	0.26
69.17	0.00	-29.64	29.64	0.00	-23.31	23.31	0.26

SERVICE FATIGUE TRUCK AND LANE LOAD SHEAR RANGES PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	MAXIMUM FATIGUE TRUCK SHEAR (KIPS)	MINIMUM FATIGUE TRUCK SHEAR (KIPS)	FATIGUE TRUCK LIVE SHEAR RANGE (KIPS)	MAXIMUM FATIGUE LANE SHEAR (KIPS)	MINIMUM FATIGUE LANE SHEAR (KIPS)	FATIGUE LANE LIVE SHEAR RANGE (KIPS)	LIVE LOAD IMPACT
0.00	29.64	0.00	29.64	23.31	0.00	23.31	0.26
6.92	17.32	-1.01	18.32	12.99	-0.89	13.88	0.26
13.83	15.05	-2.01	17.07	10.99	-1.91	12.91	0.26
20.75	12.79	-4.00	16.79	9.13	-3.08	12.21	0.26
27.67	10.53	-6.01	16.54	7.41	-4.38	11.79	0.26
34.58	8.26	-8.26	16.53	5.83	-5.83	11.65	0.26
41.50	6.01	-10.53	16.54	4.38	-7.41	11.79	0.26
48.42	4.00	-12.79	16.79	3.08	-9.13	12.21	0.26
55.33	2.01	-15.05	17.07	1.91	-10.99	12.91	0.26
62.25	1.01	-17.32	18.32	0.89	-12.99	13.88	0.26
69.17	0.00	-29.64	29.64	0.00	-23.31	23.31	0.26

MAXIMUM FACTORED SHEAR PER GIRDER INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

DISTANCE FROM LEFT SUPPORT (FT)	1.67*1.3* MIN. LIVE LOAD SHEAR (KIPS)	1.67*1.3* MAX. LIVE LOAD SHEAR (KIPS)	1.3*DEAD LOAD SHEAR (KIPS)	1.3*SUPER-IMPOSED SHEAR (KIPS)	TOTAL ABSOLUTE SHEAR (KIPS)
0.00	0.00	64.35	9.02	0.00	73.37
6.92	-2.19	37.60	7.23	0.00	44.83
13.83	-4.36	32.67	5.42	0.00	38.09
20.75	-8.68	27.77	3.61	0.00	31.38
27.67	-13.05	22.86	1.81	0.00	24.67
34.58	-17.93	17.93	0.00	0.00	17.93
41.50	-22.86	13.05	-1.81	0.00	24.67
48.42	-27.77	8.68	-3.61	0.00	31.38

55.33	-32.67	4.36	-5.42	0.00	38.09
62.25	-37.60	2.19	-7.23	0.00	44.83
69.17	-64.35	0.00	-9.02	0.00	73.37

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*                      PART V                       *
*     STRESS AND LOAD/CAPACITY RATIO INFORMATION    *
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OVERLOAD MOMENT CAPACITY TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE
(TOP OF STEEL OVERLOAD STRESS CAPACITY RATIO)

SECTION POINT		DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD STRESS (KSI)	SUPER-IMPOSED LOAD STRESS (KSI)	1.67*LIVE LOAD & IMPACT STRESS (KSI)	TOTAL LOAD STRESS (KSI)	ALLOW-ABLE STRESS KSI	OVERLOAD TOP STEEL RATIO
0	RIGHT	0.00	0.00	0.00	0.00	0.00	40.00	0.000
1	LEFT	6.92	1.45	0.00	6.69	8.13	40.00	0.203
1	RIGHT	6.92	1.45	0.00	6.69	8.13	40.00	0.203
2	LEFT	13.83	2.57	0.00	11.62	14.19	40.00	0.355
2	RIGHT	13.83	2.57	0.00	11.62	14.19	40.00	0.355
3	LEFT	20.75	3.37	0.00	14.82	18.19	40.00	0.455
3	RIGHT	20.75	3.37	0.00	14.82	18.19	40.00	0.455
4	LEFT	27.67	3.85	0.00	16.65	20.50	40.00	0.513
4	RIGHT	27.67	3.85	0.00	16.65	20.50	40.00	0.513
5	LEFT	34.58	4.01	0.00	16.93	20.95	40.00	0.524
5	RIGHT	34.58	4.01	0.00	16.93	20.95	40.00	0.524
6	LEFT	41.50	3.85	0.00	16.65	20.50	40.00	0.513
6	RIGHT	41.50	3.85	0.00	16.65	20.50	40.00	0.513
7	LEFT	48.42	3.37	0.00	14.82	18.19	40.00	0.455
7	RIGHT	48.42	3.37	0.00	14.82	18.19	40.00	0.455
8	LEFT	55.33	2.57	0.00	11.62	14.19	40.00	0.355
8	RIGHT	55.33	2.57	0.00	11.62	14.19	40.00	0.355
9	LEFT	62.25	1.45	0.00	6.69	8.13	40.00	0.203
9	RIGHT	62.25	1.45	0.00	6.69	8.13	40.00	0.203
10	LEFT	69.17	0.00	0.00	0.00	0.00	40.00	0.000

OVERLOAD MOMENT CAPACITY TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE
(BOTTOM OF STEEL OVERLOAD STRESS CAPACITY RATIO)

SECTION POINT		DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD STRESS (KSI)	SUPER-IMPOSED LOAD STRESS (KSI)	1.67*LIVE LOAD & IMPACT STRESS (KSI)	TOTAL LOAD STRESS (KSI)	ALLOW-ABLE STRESS KSI	OVERLOAD BOTTOM STEEL RATIO
0	RIGHT	0.00	0.00	0.00	0.00	0.00	40.00	0.000
1	LEFT	6.92	1.45	0.00	6.69	8.13	40.00	0.203
1	RIGHT	6.92	1.45	0.00	6.69	8.13	40.00	0.203
2	LEFT	13.83	2.57	0.00	11.62	14.19	40.00	0.355
2	RIGHT	13.83	2.57	0.00	11.62	14.19	40.00	0.355
3	LEFT	20.75	3.37	0.00	14.82	18.19	40.00	0.455
3	RIGHT	20.75	3.37	0.00	14.82	18.19	40.00	0.455
4	LEFT	27.67	3.85	0.00	16.65	20.50	40.00	0.513

4 RIGHT	27.67	3.85	0.00	16.65	20.50	40.00	0.513
5 LEFT	34.58	4.01	0.00	16.93	20.95	40.00	0.524
5 RIGHT	34.58	4.01	0.00	16.93	20.95	40.00	0.524
6 LEFT	41.50	3.85	0.00	16.65	20.50	40.00	0.513
6 RIGHT	41.50	3.85	0.00	16.65	20.50	40.00	0.513
7 LEFT	48.42	3.37	0.00	14.82	18.19	40.00	0.455
7 RIGHT	48.42	3.37	0.00	14.82	18.19	40.00	0.455
8 LEFT	55.33	2.57	0.00	11.62	14.19	40.00	0.355
8 RIGHT	55.33	2.57	0.00	11.62	14.19	40.00	0.355
9 LEFT	62.25	1.45	0.00	6.69	8.13	40.00	0.203
9 RIGHT	62.25	1.45	0.00	6.69	8.13	40.00	0.203
10 LEFT	69.17	0.00	0.00	0.00	0.00	40.00	0.000

MOMENT STRENGTH CAPACITY TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE
(MOMENT CAPACITY RATIO)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	1.3*(DEAD + SUPER- IMPOSED+ 1.67*LIVE) MOMENT (K-FT)	Rb IN AASHTO 10.48.4.1	Mr IN AASHTO 10.48.4.1 (K-FT)	MU MOMENT CAPACITY (K-FT)	MOMENT STRENGTH RATIO	GOVER- NING AASHTO DESIGN EQUA- TION	IS SEC- TION COM- PACT?
0 RIGHT	0.00	0.00	NA	NA	1,729.17	0.000	(10-92)	Yes
1 LEFT	6.92	316.24	NA	NA	1,729.17	0.183	(10-92)	Yes
1 RIGHT	6.92	316.24	NA	NA	1,729.17	0.183	(10-92)	Yes
2 LEFT	13.83	551.99	NA	NA	1,729.17	0.319	(10-92)	Yes
2 RIGHT	13.83	551.99	NA	NA	1,729.17	0.319	(10-92)	Yes
3 LEFT	20.75	707.31	NA	NA	1,729.17	0.409	(10-92)	Yes
3 RIGHT	20.75	707.31	NA	NA	1,729.17	0.409	(10-92)	Yes
4 LEFT	27.67	797.43	NA	NA	1,729.17	0.461	(10-92)	Yes
4 RIGHT	27.67	797.43	NA	NA	1,729.17	0.461	(10-92)	Yes
5 LEFT	34.58	814.72	NA	NA	1,729.17	0.471	(10-92)	Yes
5 RIGHT	34.58	814.72	NA	NA	1,729.17	0.471	(10-92)	Yes
6 LEFT	41.50	797.43	NA	NA	1,729.17	0.461	(10-92)	Yes
6 RIGHT	41.50	797.43	NA	NA	1,729.17	0.461	(10-92)	Yes
7 LEFT	48.42	707.31	NA	NA	1,729.17	0.409	(10-92)	Yes
7 RIGHT	48.42	707.31	NA	NA	1,729.17	0.409	(10-92)	Yes
8 LEFT	55.33	551.99	NA	NA	1,729.17	0.319	(10-92)	Yes
8 RIGHT	55.33	551.99	NA	NA	1,729.17	0.319	(10-92)	Yes
9 LEFT	62.25	316.24	NA	NA	1,729.17	0.183	(10-92)	Yes
9 RIGHT	62.25	316.24	NA	NA	1,729.17	0.183	(10-92)	Yes
10 LEFT	69.17	0.00	NA	NA	1,729.17	0.000	(10-92)	Yes

SHEAR CAPACITY AT SECTION POINTS INFORMATION TABLE
(SHEAR CAPACITY RATIO)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	Vp IN AASHTO (10-115) (KIPS)	C IN AASHTO (10-116) (10-117)	Vu IN AASHTO (10-113) (KIPS)	TOTAL ABSOLUTE FACTORED SHEAR (KIPS)	SHEAR CAPACITY RATIO	GOVERN- ING AASHTO DESIGN EQUATION
0	0.00	500.51	1.00	500.51	73.37	0.147	(10-113)
1	6.92	500.51	1.00	500.51	44.83	0.090	(10-113)
2	13.83	500.51	1.00	500.51	38.09	0.076	(10-113)
3	20.75	500.51	1.00	500.51	31.38	0.063	(10-113)
4	27.67	500.51	1.00	500.51	24.67	0.049	(10-113)
5	34.58	500.51	1.00	500.51	17.93	0.036	(10-113)
6	41.50	500.51	1.00	500.51	24.67	0.049	(10-113)
7	48.42	500.51	1.00	500.51	31.38	0.063	(10-113)
8	55.33	500.51	1.00	500.51	38.09	0.076	(10-113)
9	62.25	500.51	1.00	500.51	44.83	0.090	(10-113)
10	69.17	500.51	1.00	500.51	73.37	0.147	(10-113)

DEFLECTION AND DEFLECTION RATIO INFORMATION TABLE
(LIVE LOAD IMPACT INCLUDED)

SECTION POINT	DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD DEF. TOTAL (IN.)	SUPER IMPOSED LOAD DEF. (IN.)	LIVE LOAD DEF. (IN.)	ALLOWABLE LIVE LOAD DEF. (IN.)	DEF. RATIO
0	0.00	0.00	0.00	0.00	1.66	0.000
1	6.92	0.19	0.00	0.44	1.66	0.265
2	13.83	0.36	0.00	0.84	1.66	0.505
3	20.75	0.49	0.00	1.15	1.66	0.693
4	27.67	0.58	0.00	1.35	1.66	0.815
5	34.58	0.60	0.00	1.42	1.66	0.854
6	41.50	0.58	0.00	1.35	1.66	0.815
7	48.42	0.49	0.00	1.15	1.66	0.693
8	55.33	0.36	0.00	0.84	1.66	0.505
9	62.25	0.19	0.00	0.44	1.66	0.265
10	69.17	0.00	0.00	0.00	1.66	0.000

FATIGUE STRESS AT WEB AND BOTTOM FLANGE JUNCTION INFORMATION TABLE
(FATIGUE CAPACITY RATING FOR CATEGORY C, CROSS FRAMES AND DIAPHRAGMS DETAILS)

SECTION POINT	SECTION MODULUS AT WEB & BOTTOM FLANGE (IN^3)	FATIGUE TRUCK STRESS RANGE AT WEB & FLANGE (KSI)	FATIGUE CATEGORY C STRESS RANGE FOR TRUCK (KSI)	TRUCK CATEGORY C FATIGUE RATIO	FATIGUE LANE STRESS RANGE AT WEB & FLANGE (KSI)	FATIGUE CATEGORY C STRESS RANGE FOR LANE (KSI)	LANE CATEGORY C FATIGUE RATIO
0 RIGHT	376.04	0.00	21.00	0.000	0.00	35.50	0.000
1 LEFT	376.04	3.82	21.00	0.182	2.51	35.50	0.071
1 RIGHT	376.04	3.82	21.00	0.182	2.51	35.50	0.071
2 LEFT	376.04	6.65	21.00	0.316	4.46	35.50	0.126
2 RIGHT	376.04	6.65	21.00	0.316	4.46	35.50	0.126
3 LEFT	376.04	8.47	21.00	0.403	5.85	35.50	0.165
3 RIGHT	376.04	8.47	21.00	0.403	5.85	35.50	0.165
4 LEFT	376.04	9.52	21.00	0.453	6.68	35.50	0.188
4 RIGHT	376.04	9.52	21.00	0.453	6.68	35.50	0.188
5 LEFT	376.04	9.68	21.00	0.461	6.96	35.50	0.196
5 RIGHT	376.04	9.68	21.00	0.461	6.96	35.50	0.196
6 LEFT	376.04	9.52	21.00	0.453	6.68	35.50	0.188
6 RIGHT	376.04	9.52	21.00	0.453	6.68	35.50	0.188
7 LEFT	376.04	8.47	21.00	0.403	5.85	35.50	0.165
7 RIGHT	376.04	8.47	21.00	0.403	5.85	35.50	0.165
8 LEFT	376.04	6.65	21.00	0.316	4.46	35.50	0.126
8 RIGHT	376.04	6.65	21.00	0.316	4.46	35.50	0.126
9 LEFT	376.04	3.82	21.00	0.182	2.51	35.50	0.071
9 RIGHT	376.04	3.82	21.00	0.182	2.51	35.50	0.071
10 LEFT	376.04	0.00	21.00	0.000	0.00	35.50	0.000

FATIGUE STRESS AT WEB AND CROSS FRAME JUNCTION INFORMATION TABLE
(FATIGUE AT CROSS FRAMES AND DIAPHRAGMS WELD DETAILS)

CROSS FRAME OR DIA- PHRAGM NUMBER	SECTION MODULUS AT WEB & BOTTOM FLANGE	FATIGUE TRUCK MOMENT RANGE AT DIA-	FATIGUE TRUCK STRESS RANGE AT WEB	IS FATIGUE TRUCK STRESS AT DIA-	FATIGUE LANE MOMENT RANGE AT DIA	FATIGUE LANE STRESS RANGE AT WEB	IS FATIGUE LANE STRESS AT DIA-

		PHRAGM	& FLANGE	PHRAGM WELD OK?	PHRAGM	& FLANGE	PHRAGM WELD OK?
	(IN^3)	(K-FT)	(KSI)		(K-FT)	(KSI)	
1 Right	376.04	0.00	0.00	Yes	0.00	0.00	Yes
2 LEFT	376.04	119.78	3.82	Yes	78.54	2.51	Yes
2 Right	376.04	119.78	3.82	Yes	78.54	2.51	Yes
3 LEFT	376.04	208.20	6.64	Yes	139.60	4.45	Yes
3 Right	376.04	208.20	6.64	Yes	139.60	4.45	Yes
4 LEFT	376.04	265.40	8.47	Yes	183.26	5.85	Yes
4 Right	376.04	265.40	8.47	Yes	183.26	5.85	Yes
5 LEFT	376.04	298.28	9.52	Yes	209.44	6.68	Yes
5 Right	376.04	298.28	9.52	Yes	209.44	6.68	Yes
6 LEFT	376.04	299.96	9.57	Yes	212.32	6.78	Yes
6 Right	376.04	299.96	9.57	Yes	212.32	6.78	Yes
7 LEFT	376.04	301.68	9.63	Yes	215.26	6.87	Yes
7 Right	376.04	301.68	9.63	Yes	215.26	6.87	Yes
8 LEFT	376.04	303.37	9.68	Yes	218.17	6.96	Yes
8 Right	376.04	303.37	9.68	Yes	218.17	6.96	Yes
9 LEFT	376.04	301.69	9.63	Yes	215.28	6.87	Yes
9 Right	376.04	301.69	9.63	Yes	215.28	6.87	Yes
10 LEFT	376.04	299.98	9.57	Yes	212.34	6.78	Yes
10 Right	376.04	299.98	9.57	Yes	212.34	6.78	Yes
11 LEFT	376.04	298.28	9.52	Yes	209.44	6.68	Yes
11 Right	376.04	298.28	9.52	Yes	209.44	6.68	Yes
12 LEFT	376.04	265.40	8.47	Yes	183.26	5.85	Yes
12 Right	376.04	265.40	8.47	Yes	183.26	5.85	Yes
13 LEFT	376.04	208.27	6.65	Yes	139.65	4.46	Yes
13 Right	376.04	208.27	6.65	Yes	139.65	4.46	Yes
14 LEFT	376.04	119.78	3.82	Yes	78.54	2.51	Yes
14 Right	376.04	119.78	3.82	Yes	78.54	2.51	Yes
15 LEFT	376.04	0.00	0.00	Yes	0.00	0.00	Yes

NOTE: IF FATIGUE STRESS FOR WELD AT ANY CROSS FRAME OR DIAPHRAGM IS NOT ACCEPTABLE, USE BOLTED CONNECTION INSTEAD OF WELDED CONNECTION, OR REDESIGN THE BEAM, OR CHANGE THE DIAPHRAGM LOCATION IF POSSIBLE

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*                    PART VI                        *
*             LOAD RATING INFORMATION               *
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OVERLOAD MOMENT INVENTORY AND OPERATING RATING TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE

(TOP OF STEEL OVERLOAD STRESS CAPACITY RATING)

SECTION POINT		DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD STRESS (KSI)	SUPER-IMPOSED LOAD STRESS (KSI)	1.67*LIVE LOAD & IMPACT STRESS (KSI)	ALLOW-ABLE STRESS (KSI)	INVEN-TORY RATING (TONS)	OPERA-TING RATING (TONS)
0	RIGHT	0.00	0.00	0.00	0.00	40.00	INFINITY	INFINITY
1	LEFT	6.92	1.45	0.00	6.69	40.00	207.58	346.67
1	RIGHT	6.92	1.45	0.00	6.69	40.00	207.58	346.67
2	LEFT	13.83	2.57	0.00	11.62	40.00	115.92	193.59
2	RIGHT	13.83	2.57	0.00	11.62	40.00	115.92	193.59
3	LEFT	20.75	3.37	0.00	14.82	40.00	89.01	148.64
3	RIGHT	20.75	3.37	0.00	14.82	40.00	89.01	148.64
4	LEFT	27.67	3.85	0.00	16.65	40.00	78.15	130.51
4	RIGHT	27.67	3.85	0.00	16.65	40.00	78.15	130.51
5	LEFT	34.58	4.01	0.00	16.93	40.00	76.50	127.76
5	RIGHT	34.58	4.01	0.00	16.93	40.00	76.50	127.76
6	LEFT	41.50	3.85	0.00	16.65	40.00	78.15	130.51
6	RIGHT	41.50	3.85	0.00	16.65	40.00	78.15	130.51
7	LEFT	48.42	3.37	0.00	14.82	40.00	89.01	148.64
7	RIGHT	48.42	3.37	0.00	14.82	40.00	89.01	148.64
8	LEFT	55.33	2.57	0.00	11.62	40.00	115.92	193.59
8	RIGHT	55.33	2.57	0.00	11.62	40.00	115.92	193.59
9	LEFT	62.25	1.45	0.00	6.69	40.00	207.58	346.67
9	RIGHT	62.25	1.45	0.00	6.69	40.00	207.58	346.67
10	LEFT	69.17	0.00	0.00	0.00	40.00	INFINITY	INFINITY

OVERLOAD MOMENT INVENTORY AND OPERATING RATING TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE

(BOTTOM OF STEEL OVERLOAD STRESS CAPACITY RATING)

SECTION POINT		DISTANCE FROM LEFT SUPPORT (FT)	DEAD LOAD STRESS (KSI)	SUPER-IMPOSED LOAD STRESS (KSI)	1.67*LIVE LOAD & IMPACT STRESS (KSI)	TOTAL VEHICLE LOAD (TONS)	INVEN-TORY RATING (TONS)	OPERA-TING RATING (TONS)
0	RIGHT	0.00	0.00	0.00	0.00	36.00	INFINITY	INFINITY
1	LEFT	6.92	1.45	0.00	6.69	36.00	207.58	346.67
1	RIGHT	6.92	1.45	0.00	6.69	36.00	207.58	346.67
2	LEFT	13.83	2.57	0.00	11.62	36.00	115.92	193.59
2	RIGHT	13.83	2.57	0.00	11.62	36.00	115.92	193.59
3	LEFT	20.75	3.37	0.00	14.82	36.00	89.01	148.64
3	RIGHT	20.75	3.37	0.00	14.82	36.00	89.01	148.64
4	LEFT	27.67	3.85	0.00	16.65	36.00	78.15	130.51
4	RIGHT	27.67	3.85	0.00	16.65	36.00	78.15	130.51
5	LEFT	34.58	4.01	0.00	16.93	36.00	76.50	127.76
5	RIGHT	34.58	4.01	0.00	16.93	36.00	76.50	127.76
6	LEFT	41.50	3.85	0.00	16.65	36.00	78.15	130.51
6	RIGHT	41.50	3.85	0.00	16.65	36.00	78.15	130.51
7	LEFT	48.42	3.37	0.00	14.82	36.00	89.01	148.64
7	RIGHT	48.42	3.37	0.00	14.82	36.00	89.01	148.64
8	LEFT	55.33	2.57	0.00	11.62	36.00	115.92	193.59
8	RIGHT	55.33	2.57	0.00	11.62	36.00	115.92	193.59
9	LEFT	62.25	1.45	0.00	6.69	36.00	207.58	346.67
9	RIGHT	62.25	1.45	0.00	6.69	36.00	207.58	346.67
10	LEFT	69.17	0.00	0.00	0.00	36.00	INFINITY	INFINITY

MOMENT STRENGTH INVENTORY AND OPERATING RATING TO LEFT AND RIGHT OF SECTION POINTS INFORMATION TABLE

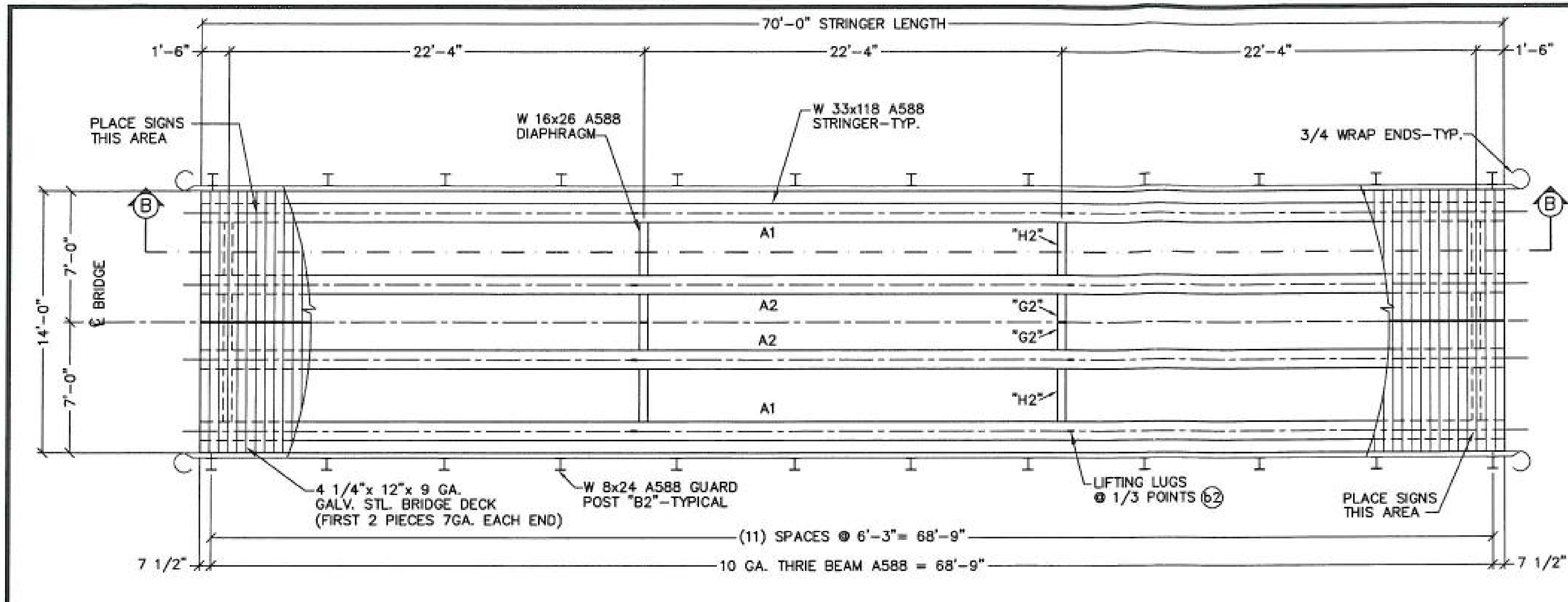
(MOMENT CAPACITY RATING)

SECTION POINT	1.3*DEAD LOAD	1.3*SUPER IMPOSED	2.17* LIVE &	MOMENT STRENGTH	TOTAL VEHICLE	INVEN-TORY	OPERA-TING

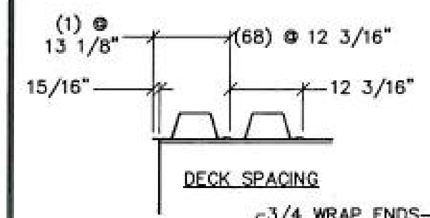
		MOMENT	LOAD MOMENT	IMPACT MOMENT		LOAD	RATING	RATING
		(K-FT)	(K-FT)	(K-FT)	(K-FT)	(TONS)	(TONS)	(TONS)
0	RIGHT	0.00	0.00	0.00	1,729.17	36.00	INFINITY	INFINITY
1	LEFT	56.20	0.00	259.92	1,729.17	36.00	231.71	386.96
1	RIGHT	56.20	0.00	259.92	1,729.17	36.00	231.71	386.96
2	LEFT	99.90	0.00	451.88	1,729.17	36.00	129.80	216.76
2	RIGHT	99.90	0.00	451.88	1,729.17	36.00	129.80	216.76
3	LEFT	131.13	0.00	575.92	1,729.17	36.00	99.89	166.82
3	RIGHT	131.13	0.00	575.92	1,729.17	36.00	99.89	166.82
4	LEFT	149.86	0.00	647.27	1,729.17	36.00	87.84	146.69
4	RIGHT	149.86	0.00	647.27	1,729.17	36.00	87.84	146.69
5	LEFT	156.10	0.00	658.31	1,729.17	36.00	86.02	143.66
5	RIGHT	156.10	0.00	658.31	1,729.17	36.00	86.02	143.66
6	LEFT	149.86	0.00	647.27	1,729.17	36.00	87.84	146.69
6	RIGHT	149.86	0.00	647.27	1,729.17	36.00	87.84	146.69
7	LEFT	131.13	0.00	575.92	1,729.17	36.00	99.89	166.82
7	RIGHT	131.13	0.00	575.92	1,729.17	36.00	99.89	166.82
8	LEFT	99.90	0.00	451.88	1,729.17	36.00	129.80	216.76
8	RIGHT	99.90	0.00	451.88	1,729.17	36.00	129.80	216.76
9	LEFT	56.20	0.00	259.92	1,729.17	36.00	231.71	386.96
9	RIGHT	56.20	0.00	259.92	1,729.17	36.00	231.71	386.96
10	LEFT	0.00	0.00	0.00	1,729.17	36.00	INFINITY	INFINITY

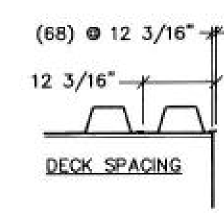
SHEAR INVENTORY AND OPERATING RATING AT SECTION POINTS INFORMATION TABLE
(SHEAR CAPACITY RATING)

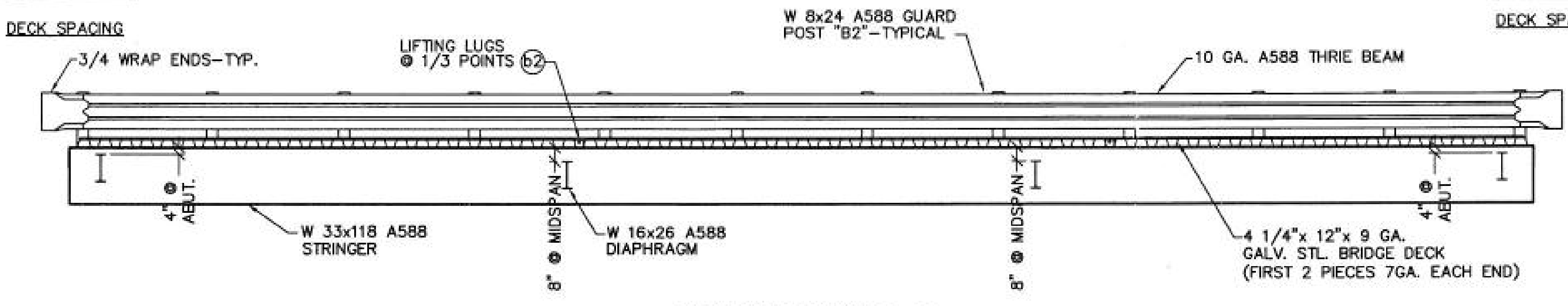
SECTION POINT	1.3* DEAD LOAD SHEAR	1.3* SUPER-IMPOSED LOAD SHEAR	2.17* MIN LIVE LOAD SHEAR	2.17* MAX LIVE LOAD SHEAR	SHEAR CAPACITY	INVEN-TORY RATING	OPERA-TING RATING
	(KIPS)	(KIPS)	(KIPS)	(KIPS)	(KIPS)	(TONS)	(TONS)
0	9.02	0.00	0.00	64.32	500.51	275.09	459.40
1	7.23	0.00	-2.19	37.58	500.51	472.49	789.06
2	5.42	0.00	-4.36	32.66	500.51	545.75	911.40
3	3.61	0.00	-8.68	27.75	500.51	644.52	1,076.35
4	1.81	0.00	-13.04	22.85	500.51	785.70	1,312.12
5	0.00	0.00	-17.92	17.92	500.51	1,005.26	1,678.78
6	-1.81	0.00	-22.85	13.04	500.51	785.70	1,312.12
7	-3.61	0.00	-27.75	8.68	500.51	644.52	1,076.35
8	-5.42	0.00	-32.66	4.36	500.51	545.75	911.40
9	-7.23	0.00	-37.58	2.19	500.51	472.49	789.06
10	-9.02	0.00	-64.32	0.00	500.51	275.09	459.40



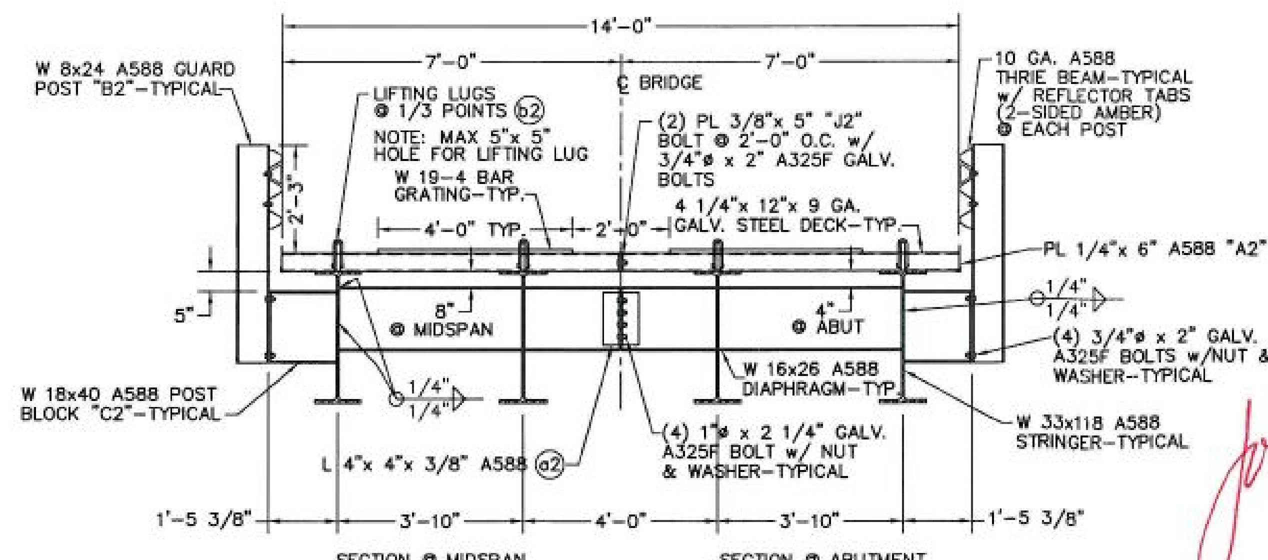
PLAN VIEW



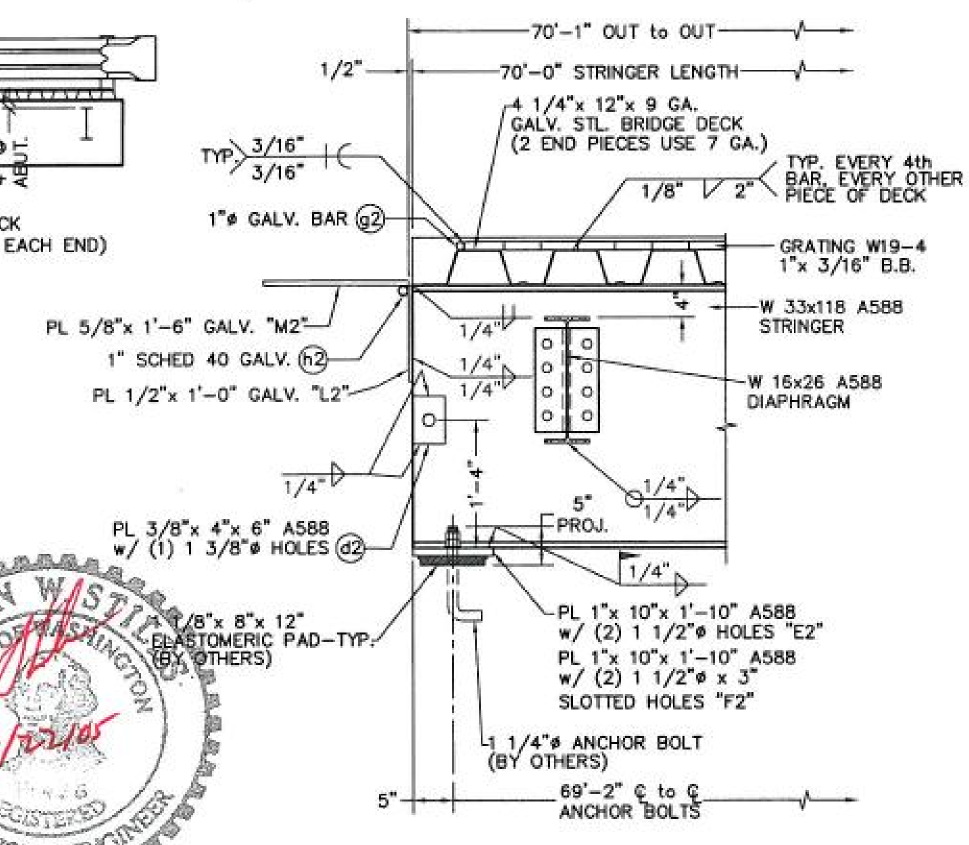




ELEVATION VIEW B-B



CROSS SECTION



BEARING DETAIL

GENERAL NOTES

ALL WORK SHALL BE DONE ACCORDING TO THE CURRENT STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES ADOPTED BY AASHTO, APPLICABLE TO THE PROJECT.
STRINGERS: A588
ALL STRUCTURAL STEEL NOT OTHERWISE NOTED SHALL BE AASHTO SPEC. A36.
WELDING SHALL CONFORM TO THE LATEST EDITION OF THE A.W.S. STANDARD SPECIFICATIONS FOR WELDING HIGHWAY BRIDGES AS AMENDED.
WELDING ELECTRODES USED SHALL BE:

TYPE OF STEEL	WELDING ELECTRODE
A709, A36/A36M, A572/A572M & A500	E7018, E7018-H8R, E7018-H4R
A709M, 250M, 345M	E71T1-1M
A709, A588/A588M & A847	E8018-C1, E8018-C3
A709M, 50W, 345W	E81T1-Ni2

PAINTED STEEL THAT IS TO BE FIELD WELDED SHALL HAVE PAINT SCRAPED or GROUND OFF WELDING AREA BEFORE WELDING.

BOLTS SHALL BE FURNISHED IN THE AMOUNT OF TWO PERCENT IN EXCESS OF THE NOMINAL NUMBER REQUIRED FOR EACH SIZE AND LENGTH.

LETTER SUFFIXES SHOWN ON DRAWINGS FOLLOWING THE BOLT DESIGNATIONS REPRESENT THE FOLLOWING.
- F – FRICTION TYPE CONNECTION.
- N – BEARING TYPE CONNECTION WITH THREADS PERMITTED IN SHEAR PLANE.
- X – BEARING TYPE CONNECTION WITH THREADS NOT PERMITTED IN SHEAR PLANE.
- P – PRETENSION

OUTSIDE SURFACE OF OUTSIDE STRINGERS SHALL BE BRUSH BLASTED SPPC-SP7.

LOADING DATA

LIVE LOAD:	DESIGN LOAD HS-20
DEAD LOAD:	ASSUMES 20 lb., PER SQ. FT. ADDITIONAL WEARING SURFACE.
LIVE LOAD DEFLECTION CRITERIA	Δ = L/500

NOTE: DO NOT APPLY DEICING OR DUST PROHIBITIVE CHEMICALS OR SALTS TO ANY AREA OF THE BRIDGE.

APPROXIMATE 1/2 BRIDGE WEIGHT = 29,000 lb.

JOHN W. STILES
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER
9/22/05
EXPIRES 12/23/06

BR4225

BIG R Manufacturing LLC
P.O. BOX 1290, GREELEY, COLORADO 80632
PHONE: (970) 356-9600 FAX: (970) 356-9621
TOLL FREE: 1-800-234-0734 www.bigrmfg.com

14'-0" x 70'-0" BRIDGE

WASHINGTON DEPT. OF FISH AND WILDLIFE
TUNK CREEK PORTABLE STEEL BRIDGE

DATE	DRAWN BY	CHECKED BY	REVISIONS	REVISIONS	REVISIONS
8-21-2005	R. Haworth	MC			

SHEET NO. 1 OF 3

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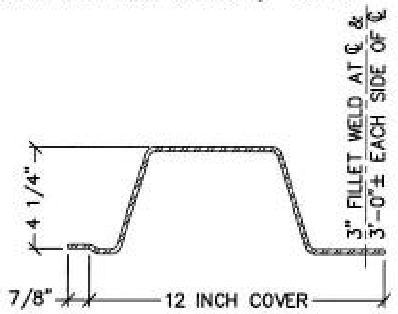
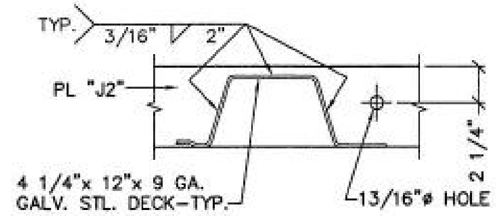
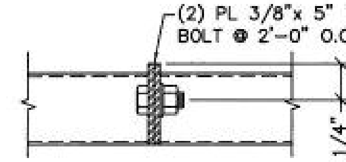
ASPHALT SURFACE: (IF APPLICABLE)
A. INSTALLATION
1. CLEAN METAL SURFACES OF ALL FOREIGN MATTER.
2. APPLY A TACK COAT OVER FLOOR SURFACE.
3. FILL AND COMPACT ALL CORRUGATIONS w/ASPHALT TO THE TOP OF THE FLOOR.
4. OVERLAY A LEVELING COURSE AND ADDITIONAL COURSES AS NECESSARY TO FINAL WEARING COURSE AND COMPACT TO REQUIRED DENSITY.

BRIDGE FLOOR FASTENING:
A. WELDING
1. WELD EACH PIECE TO ALL STRINGERS w/ PLATE THICKNESS x 3" FILLET WELDS.
2. WELD OVERLAPPING w/ PLATE THICKNESS FILLET WELDS.
3. PLACE 1 1/2" SKIP WELD ON EDGE OF DECKING AT EACH SIDE OF LIFTING LUG.
4. 4 1/4"x 12"x 7 or 9 GA. A607 DECKING Fy = 50 KSI.

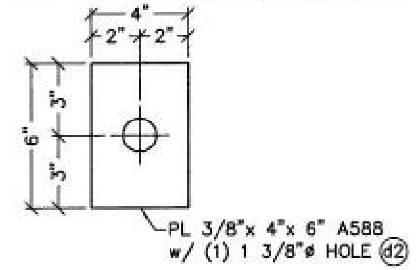


STEEL FLOOR SECTION

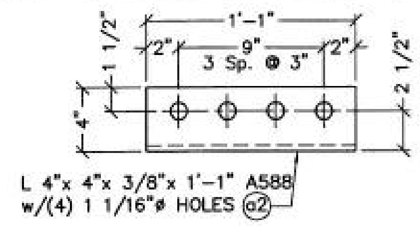




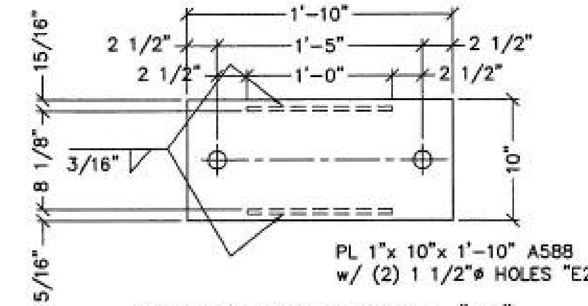
DECK SPLICE BAR



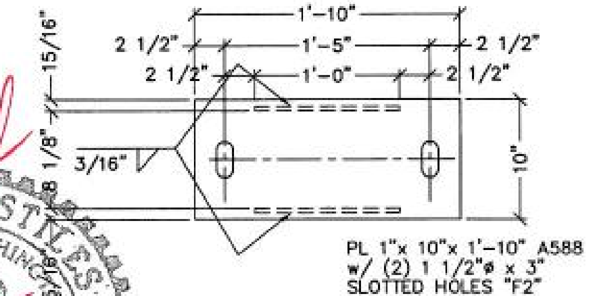
END REINFORCEMENT DETAIL "d2"



DIAPHRAGM SPLICE ANGLE "a2"



BEARING PLATE DETAIL "E2"



BEARING PLATE DETAIL "F2"

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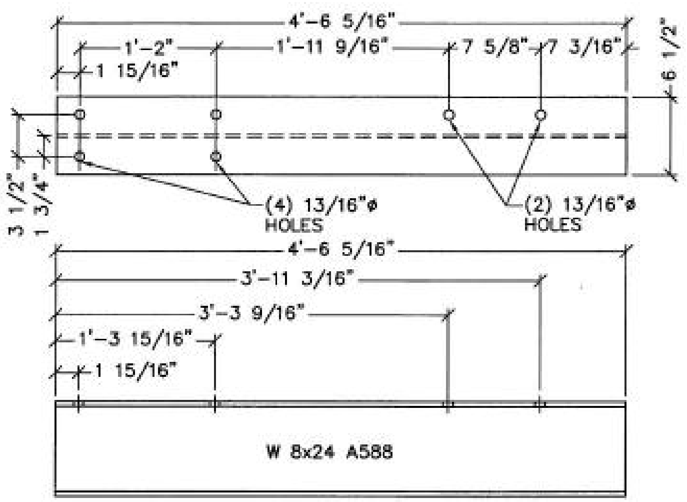
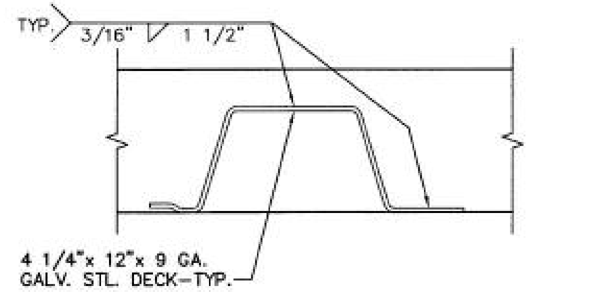
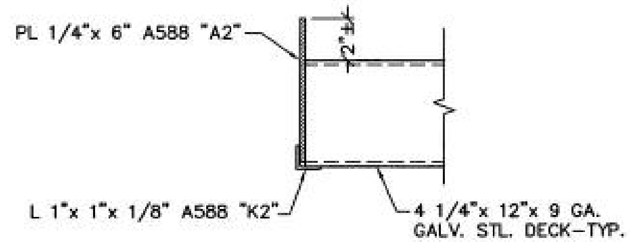
STATE OF WASHINGTON
REGISTERED PROFESSIONAL ENGINEER
9/12/05
EXPIRES 12/22/06

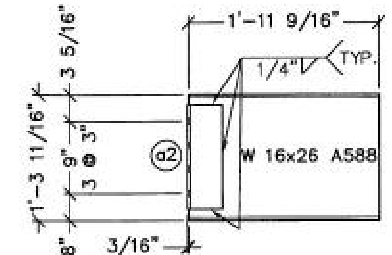


GUARDRAIL POST DETAIL "B2"

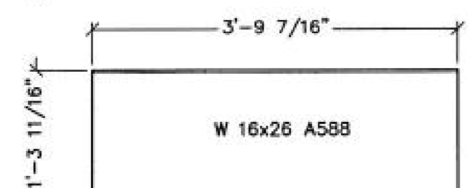




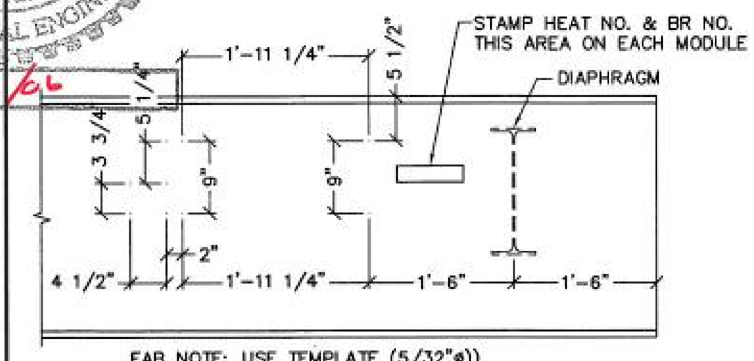
SIDE DAM DETAIL



DIAPHRAGM DETAIL "G2"

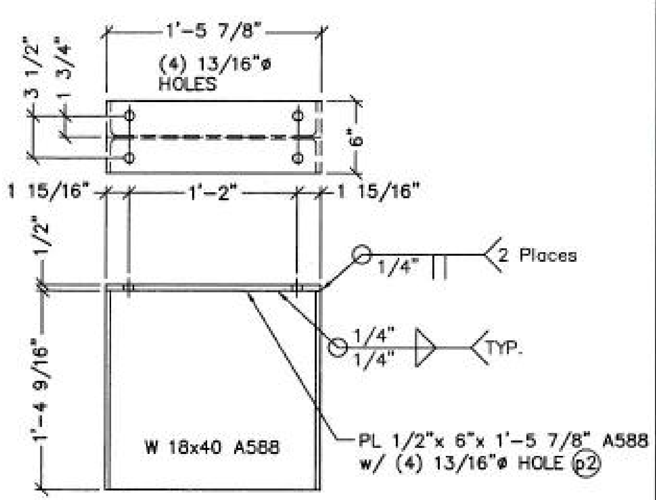


DIAPHRAGM DETAIL "H2"

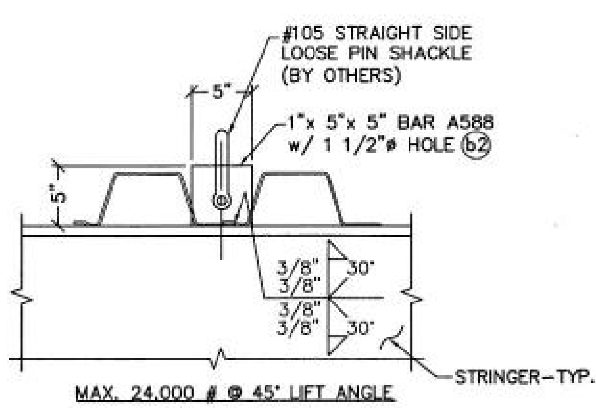


FAB NOTE: USE TEMPLATE (5/32"ø))

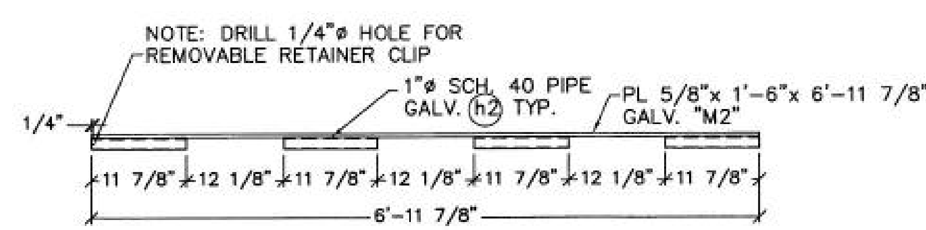
SIGN HOLE LOCATION DETAIL

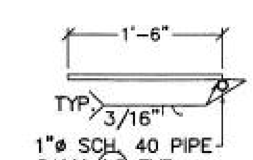


POST BLOCK DETAIL "C2"

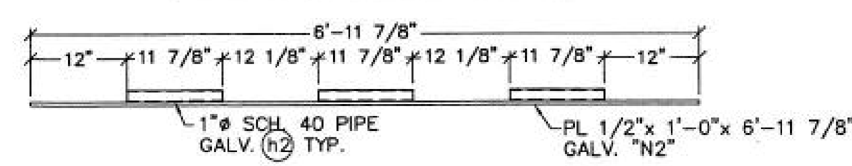


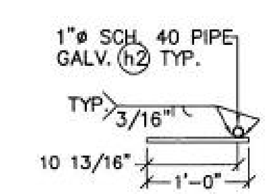
LIFTING LUG DETAIL





TRANSITION PLATE DETAIL "M2"





END PLATE DETAIL "L2"

BR4225

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TOLL FREE: 1-800-234-0734 www.bigrmfg.com

14'-0" x 70'-0" BRIDGE
WASHINGTON DEPT. OF FISH AND WILDLIFE
TUNK CREEK PORTABLE STEEL BRIDGE

DATE	DRAWN BY	CHECKED BY	REVISIONS	REVISIONS
8-21-2005	R. Haworth	WL		

SHEET NO. 2 OF 3

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BILL OF MATERIALS

	Qty.	Shape	Description	Length	MK	Finish
	2	W	33x118 A588	70'-0"	A1	–
	2	W	33x118 A588	70'-0"	B1	–
	2	Plate	1/4"x 6" A588	70'-0"	A2	–
	24	W	8x24 A588	4'-6 5/16"	B2	–
	24	W	18x40 A588	1'-4 9/16"	C2	–
Loose	4	Plate	1"x 10" A588	1'-10"	E2	–
Loose	4	Plate	1"x 10" A588	1'-10"	F2	–
	8	W	16x26 A588	1'-11 9/16"	G2	–
	8	W	16x26 A588	3'-9 7/16"	H2	–
	2	Plate	3/8"x 5" A588	70'-0"	J2	–
	2	Angle	1"x 1"x 1/8" A588	70'-0"	K2	–
	4	Plate	1/2"x 12" A572-50	6'-11 7/8"	L2	GALVANIZED
	4	Plate	5/8"x 1'-6" A572-50	6'-11 7/8"	M2	GALVANIZED
	16	Angle	4"x 4"x 3/8" A588	1'-1"	a2	–
	8	Bar	1"x 5" A588	5"	b2	–
	16	Bar	3/8"x 3/8" Square	1'-0"	c2	–
	8	Bar	3/8"x 4" A588	0'-6"	d2	–
	4	Rod	1" ø	4'-0"	g2	GALVANIZED
	28	Pipe	1" Sched 40	0'-11 7/8"	h2	GALVANIZED
	4	Rod	7/8" ø	6'-11 7/8"	r2	STAINLESS
	4	Clips	Retainer Clips	–	–	–
	24	Plate	1/2"x 6" A588	1'-5 7/8"	p2	–
	132	Deck	4 1/4"x 12"x 9ga	6'-11 5/8"	–	GALVANIZED
	8	Deck	4 1/4"x 12"x 7ga	6'-11 5/8"	–	GALVANIZED
	2	Grating	W 19-4 1"x 3/16"x 4'-0" Bar Grating	70'-0"	–	GALVANIZED
	4	End Section	10 GA 3/4 Wrap End Sections	–	–	A588
	2	Thrie Beam	10 GA Rail (3 spa @ 6'-3")	18'-9"	–	A588
	4	Thrie Beam	10 GA. Rail (Std Punch)	25'-0"	–	A588
	24	Reflector	2 Sided Amber Reflector Tabs	–	–	Galvanize
	98	Bolts	3/4"ø A325 W/ Nut & Washer Post to Block	2"	–	Galvanize
	37	Bolts	3/4"ø A325 W/ Nut & Washer Center Stitch	2"	–	Galvanize
	33	Bolts	1"ø A325 W/ Nut & Washer Diaphragm Splice	2 1/4"	–	Galvanize
	49	Bolts	5/8"ø Button Head w/ A 563 Nut Flared End Sections	1 1/4"	–	Galvanize
	50	Bolts	5/8"ø Button Head w/ A 563 Nut & 5/8" Rectangular Washer Rail to Post	2"	–	Galvanize
	49	Bolts	5/8"ø Button Head W/ A563 Nut Guard Rail Splice	1 1/4"	–	Galvanize
	2	Signs	Big R Manufacturing Signs	–	–	–
	2	Signs	Bridge Data Plates	–	–	–

9/22/05

EXPIRES 12/23/06

BR4225

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14'-0" x 70'-0" BRIDGE
WASHINGTON DEPT. OF FISH AND WILDLIFE
TUNK CREEK PORTABLE STEEL BRIDGE

DATE	DRAWN BY	CHECKED BY	REVISIONS	REVISIONS
8-21-2005	R. Haworth	MC		

SHEET NO. 3 OF 3

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