SUBMITTED TO:
Forsgren and Associates
1109 W. Mrytle Street, Ste 300
Boise, ID. 83702

BY:
Shannon & Wilson
5260 Chinden Blvd.
Boise, ID. 83714

(202) 658-8700
www.shannonwilson.com

GEOTECHNICAL ENGINEERING REPORT

South Fork Teton River Bridge
Project No. A024(341)
Key No. 24341

FREMONT COUNTY, IDAHO

SHANNON & WILSON

April 8, 2025

Shannon & Wilson No: 112645

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Submitted To: Forsgren and Associates
1109 W. Mrytle Street, Ste 300
Boise, ID. 83702
Attn: Mr. Brent Schiller, P.E.

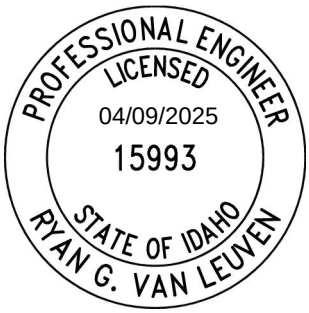
Subject: GEOTECHNICAL ENGINEERING REPORT,
SOUTH FORK TETON RIVER BRIDGE
PROJECT NO. A024(341)
KEY NO. 24341,FREMONT COUNTY, IDAHO

Shannon & Wilson prepared this report and participated in this project as a subconsultant to Forsgren and Associates Inc. Our scope of services was specified in Agreement Number 02-24-0118 with Forsgren and Associates dated July 31, 2024. This report presents the results of our field explorations, laboratory testing, geotechnical analyses and our geotechnical recommendations and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON



Ryan Van Leuven, P.E.
Geotechnical Engineer

Charles Burgert

Charles Burgert, P.E.
Geotechnical Engineer

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GEOTECHNICAL ENGINEERING REPORT

230.01 INTRODUCTION

This Geotechnical Engineering (GE) Report presents the results of a subsurface site exploration and geotechnical recommendations for use in the design and construction of the foundations for the proposed replacement of the existing bridge (Bridge Key 21035) over the South Fork of the Teton River in Fremont County, Idaho. The bridge is located on the Teton Highway (N 2400 E) at approximately MP 7.2, about 0.5 miles north of the city of Teton, Idaho.

A Vicinity Map showing the project location is included in Appendix A. Photographs of the bridge site are also included in Appendix A.

230.01.01 LRFD Design Methodology

Recommendations presented in this report are intended for use with the Load and Resistance Factor Design (LRFD) method outlined in the 10th edition of the *AASHTO LRFD Bridge Design Specifications* (2024), including current interims, the *ITD Standard Specifications for Highway Construction* (2023), the 2024 *ITD Supplemental Specifications for 2023 ITD Standard Specifications for Highway Construction* and the 2024 *ITD Bridge Design LRFD Manual*.

Existing Bridge Information

Idaho Transportation Department (ITD) historical construction drawings (dated 9-10-76) for the existing bridge are included in Appendix A. The existing bridge was constructed after a previous 1936 bridge over the South of the Teton River at this crossing location was damaged in 1976 by the failure of the upstream Teton Dam and the resulting flood.

Idaho Transportation Department (ITD) historical construction drawings (dated 1-10-36) for the previous bridge over the South of the Teton River are also included in Appendix A.

A preliminary drawing provided by Forsgren Associates (Appendix A) shows the approximate location of the previous 1936 bridge superimposed over the existing bridge.

The plan view on Sheet 2 of 14 of the 1976 drawings shows an abutment of the previous 1936 bridge in the northwest section of the existing South Fork Teton bridge, located between the north pier wall and north bridge abutment.

The following summarizes the features of both the existing bridge constructed in 1976 and the previous bridge constructed in 1936.

Existing Bridge (1976 drawings)

Summary of Existing Bridge (1976)	
Item	**Description**
No. of Spans	3
Approximate Out-to-Out Length	150 feet
Approximate Out-to-Out Width	38 feet
Skew	Approx. 45 degrees
Abutment Foundations	Driven Piles
Pier Wall Foundations	Driven Piles

The historical ITD bridge drawings indicate the existing bridge abutments are supported by seven (7) 12x53 steel H-piles, with three (3) piles driven vertically and four (4) piles driven at a front batter of 4H:12V. The piles were designed for a minimum bearing capacity of 50 tons. The abutment pile caps are shown as 2.9 feet in width and 3 feet thick. The bottom of the pile caps is indicated to be about 5.7 feet below the bridge deck. The cantilevered wing walls integrally connected to the bridge abutments are 1.5 feet in width.

The bridge pier walls are supported by ten (10) 12x53 steel H-piles, with eight (8) piles driven vertically and the two (2) end piles driven at an outside batter of 4H:12V. The piles were designed for a minimum bearing capacity of 50 tons. The pier walls are shown as 2.67 feet in width with the H-piles embedded 7 feet into the bottom of the pier walls. The bottom of the pier walls is indicated to be about 2 feet below the bottom of the river channel.

Concrete approach slabs are indicated on both sides of the bridge. The approach slabs are indicated to be 20 feet in length and about 24 feet in width.

Based on the historical ITD bridge drawings, about 6 feet to 8 feet of embankment fill was placed above the ground surface leading up to the south abutment and about 4 feet to 8 feet leading up to the north abutment.

The drawings show the bottom of the riverbed channel to be about 13 to 14 feet below the top of the bridge deck. The high-water level is indicated to be approximately 4.7 feet below the bridge deck.

An elevation datum is not indicated on the ITD historical drawings in Appendix A. The elevations shown on the drawings (dated 9-10-76) would need to be adjusted to convert the elevations to NAVD88 datum elevations.

Previous Bridge (1936 drawings)

Summary of Previous Bridge (1936)	
Item	**Description**
No. of Spans	4
Approximate Out-to-Out Length	101 feet
Approximate Out-to-Out Width	24 feet
Skew	Approx. 26.5 degrees
Abutment Foundations and Wing Walls	Driven Timber Piles
Pier Wall Foundations	Driven Timber Piles

The historical ITD bridge drawings indicate the previous bridge abutments and piers were supported by six (6) timber piles. The piles were designed for a minimum bearing capacity of 15 tons.

Based on the historical ITD bridge drawings, up to approximately 5 feet of embankment fill was placed above the ground surface for the approach embankment at the north bridge abutment and about 2 feet of embankment fill at the south bridge abutment.

The topographic contours on the Location Plan drawing indicate the bottom of the riverbed channel was about 9 feet to 11 feet below the top of the bridge deck.

An elevation datum is not indicated on the ITD historical drawings in Appendix A. The elevations shown on the drawings (dated 1-10-36) would need to be adjusted to convert the elevations to NAVD88 datum elevations.

Rip Rap

Photographs of rip rap at the existing bridge abutments, and along south side of the river channel downstream of the existing bridge, are included in Appendix A. The rip rap generally varied in size from about 1 foot in diameter up to 5 feet in diameter.

Scour

Soil samples from the river channel embankment side slopes for scour analyses were obtained by S&W from hand dug test pits at two accessible channel embankment locations (one on the NW side of river channel and one on the SW side of river channel). A soil sample from the bottom of the river channel was also obtained by Forsgren and Associates east of the existing bridge.

The particle size distribution results for the soil samples are contained in Appendix D.

Proposed Bridge

Based on the information provided by Forsgren and Associates, the proposed bridge will have the following features.

Summary of Proposed Bridge	
Item	**Description**
No. of Spans	2
Approximate Out-to-Out Length	165' 0 '
Approximate Out-to-Out Width	40' – 10"
Skew	45 degrees
Abutment <<and Wing Wall>> Foundation	Rock Socketed Shell Piles
Pier Foundation	Rock Socketed Shell Piles

The proposed bridge will have integral or semi-integral abutments. Three of the wing walls will be cantilevered walls integrally connected to the abutment walls. The wing walls will extend about 6 to 10.5 feet +/- behind the bridge abutments. A pile supported wing wall about 15 feet to 20 feet in length is planned at the northeast corner of the bridge.

The vertical profile of the proposed bridge will be raised up to about 3 feet higher than the existing bridge profile. Proposed finished grade elevation at the bridge abutments is about 4946.2 feet.

Approach slabs are planned for this project. The approach slabs will extend to a minimum of 20 feet beyond each bridge abutment. The bridge abutments will support one end of the approach slabs and a sleeper slab in the approach embankment fill will support the other end.

The approach embankments leading up to the bridge will be widened up to about 10 feet on both sides and raised up to about 3 feet. The dimensions of the new approach embankments are discussed in Section 230.05.06 - Embankments of this report.

Embankment fill should consist of compacted Granular Subbase (703.11) with 2H:1V or flatter side slopes. Granular Subbase is recommended for embankment fill because Granular Subbase is higher quality and more testable than Granular Borrow, and quantity of fill required for project is relatively small.

MSE walls are not planned for this project**.**

Geosynthetic Reinforced Soil (GRS) or Welded Wire Reinforced Soil (WWRS) bridge system is not recommended for this project due to high water levels and potential for scour.

Based on preliminary information provided by Forsgren Associates, the estimated Strength Limit and Service Limit foundation loads at the bridge abutment and pier locations are shown on the following table:

Preliminary Foundation Loads		
Feature	Strength Limit	Service Limit
Abutments	1360 kips	960 kips
Piers	2190 kips	1575 kips

Foundation recommendations for the replacement bridge are discussed in Section 230.05 – Conclusions and Recommendations of this report. Rock socketed shell piles are recommended for support of the bridge abutments and center pier.

Project Reports

This Geotechnical Engineering Report is being prepared along with project reports being prepared by others. A flexible pavement design for pavement reconstruction on this bridge replacement project is provided in Section 230.05.09.09 of this report.

230.02 FIELD EXPLORATION AND LABORATORY TESTING

230.02.01 Borings/Test Pits

230.02.01.01 Borings

Shannon & Wilson (S&W) conducted subsurface explorations in September 2024. This included six borings (B-01 to B-06) to obtain information for the proposed bridge foundations and approach embankments. The boring locations are shown on the Exploration Location Map in Appendix C and summarized on the following table.

Summary of Subsurface Explorations			
Feature	Boring No.	Collar Elevation (Feet)	Boring Depth (Feet)
South Roadway	B- 01	4942.40	6.5
South Approach Embankment	B- 02	4943.68	39.0
South Abutment*	B- 03	4943.81	80.0
North Abutment*	B- 04	4943.75	80.0
North Approach Embankment	B- 05	4943.78	31.5
North Roadway	B- 06	4943.81	6.5

*For up to 200 feet bridge span length

Haz-Tech Drilling of Nampa, Idaho, was contracted by Shannon & Wilson to perform the drilling for the site explorations with a BK-180 drill rig using hollow-stem augers and rock coring to advance the borings. The subsurface soils in the borings were sampled using a Standard Penetration Test (SPT) split-spoon sampler.

Bedrock sampling was accomplished using a rotary core drill and a triple-tube wireline HQ-3 core barrel.

A Shannon & Wilson geotechnical engineer field classified the soils encountered in the borings in general accordance with ASTM D2488 visual-manual classification procedures. Samples of soil and rock materials were packaged and transported to the Shannon & Wilson laboratory in Boise, Idaho, for testing.

Detailed logs of the borings, including sampling depths and descriptions of the materials encountered, are included in Appendix C.

The boring locations were marked in the field by Shannon & Wilson. Boring coordinates and elevations were determined by a Forsgren and Associate, Inc. surveyor after completion

of the site exploration. The elevation datum is North American Vertical Datum 1988 (NAVD88). The locations of the borings, along with the graphic boring logs, are shown on the Foundation Investigation Plat in Appendix B.

230.02.01.02 Test Pits

Soil samples were obtained from hand dug test pits at two accessible channel embankment locations for gradation analyses (one on the NW side of river channel and one on the SW side of river channel). These gradations can be used for scour analyses.

230.02.02 Field Tests

Field testing included Standard Penetration Tests (SPT) performed with an automatic hammer in accordance with ASTM D1586 to evaluate the relative density of cohesionless soils or stiffness of cohesive soils. The SPT N-values are included on the boring logs in Appendix C.

Sections of rock core recovered from the HQ core barrel were measured for Percent Recovery and Rock Quality Designation (RQD) in accordance with ASTM D6032. RQD is measured to assess the degree of jointing or fracturing of a rock mass. The Percent Recovery and RQD values are shown on the boring logs in Appendix C.

230.02.03 Geophysical Exploration

A geophysical survey was not performed.

230.02.04 Laboratory Tests

Soil and rock core samples were selected for laboratory testing to evaluate pertinent physical and engineering properties of the soil and rock. The following lab tests were performed on soil and rock core samples obtained during the subsurface exploration:

- Gradation (ASTM D6913): for grain size distribution
- Classification (ASTM D2487): to classify soil in USCS system
- Moisture Content (ASTM D2216): for in-situ water content of soil
- Atterberg Limits (ASTM D4318): for Liquid Limit (LL), Plastic Limit (PL), and Plastic Index (PI) of soil
- Compressive Strength of Intact Rock (ASTM D7012, Method C): for compressive strength of intact rock
- Sulfate & Chloride (EPA 300): for Sulfate and Chloride in soils

- Resistivity (ASTM D6439): for corrosion testing
- pH (ASTM (D4972): for corrosion testing
- R-Value (ASTM D2844): for pavement structure design
- Sand Equivalent (ASTM D2419): for pavement structure design

The laboratory test results are shown on the boring logs in Appendix C and on the laboratory test data sheets in Appendix D. The soil classifications after laboratory testing are per ASTM D2487.

230.03 SURFACE CONDITIONS

The project is situated in the Teton River Valley on the Eastern Snake River Plain.

The ground surface of the farm field areas located northeast, northwest and southeast of the South Fork of the Teton River bridge crossing are relatively flat with ground surface elevations varying from about 4940 feet to 4945 feet.

A large site area to the southwest of the bridge, and a smaller site area to the southeast of the bridge, appear to be previous excavated borrow areas. The ground surface elevation of the smaller excavation area to the southeast of the bridge is about 4930 feet and the ground surface elevation of the large excavation area to the southwest of the bridge is about 4920 feet.

An earthen berm runs along the south side of the river channel between the South Fork of the Teton River and the excavated borrow areas. The earthen berm may be native soil left in place when the borrow areas south of the river were excavated.

A small pond area northwest of the bridge also may have possibly been an excavated borrow area.

Based on the USDA NRCS Soil Resource Report for Fremont County, Idaho, near surface soils in the vicinity of the proposed project are gravelly river wash in the excavated borrow areas and St. Anthony gravelly sandy loam and Labenzo loamy sand in the existing farm field areas.

Manmade features in the project vicinity include the existing bridge and the approach embankments leading up to the bridge. The elevation of the existing bridge deck is approximately 4944 feet.

Overhead utility lines are present along the east side of the highway leading up to the bridge. A buried communication line (possibly abandoned) is located along the east side of the roadway. There is a metal pipe anchored to the east side of the bridge where this line crosses over the river.

The width of the river channel at the bridge location is about 125 feet. The elevation of the river channel bottom is about 4928.5 feet. The sides of the river channel are about 15.5 feet in height, with side slope inclinations varying from about 1.5H:1V to 3H: 1V. The Q_{50} water level elevation in the river is about 4939 feet.

Land use is primarily agricultural farmland with some commercial businesses and residential houses.

Stormwater drainage in the immediate vicinity of the existing bridge consists of sheet flow down the sides of the existing Teton Highway roadway embankments.

Vegetation in the project area consists of intermittent trees with grass and some shrubs. Wetland vegetation appears to be present along the river channel.

An aerial view of the surface conditions at the project site is shown below.



230.04 SUBSURFACE CONDITIONS

Subsurface conditions encountered at the boring locations are indicated on the Foundation Investigation Plat in Appendix B and on the boring logs in Appendix C.

Stratification boundaries shown on the Plat and boring logs indicate the approximate location of changes in the subsurface strata. In situ, the transition between strata may be gradual or abrupt. Variations in the subsurface conditions may occur between the boring locations.

Based on the results of the site exploration, a generalized subsurface profile has been developed to provide a basis for discussion of the subsurface conditions for the proposed project. The generalized subsurface profile is described in the following sections.

230.04.01 Soil Profiles

Soil Profile for Roadway Approaches and Bridge Abutments (Borings B-01 to B-06)

Asphalt Concrete

Borings B-01 to B-06 were drilled through the existing flexible pavement on both sides of the bridge. The thickness of asphalt concrete encountered at these boring locations was 0.42 to 0.50 feet (5 to 6 inches). An aggregate base layer was not apparent at the boring locations.

Silty Sand with Gravel to Poorly Graded Gravel with Silt and Sand (Embankment Fill)

Below the asphalt concrete was granular embankment fill consisting of medium dense to very dense silty sand with gravel to poorly graded gravel with silt and sand. This embankment fill extended to a depth ranging from about 5 feet (about EL 4939 feet) below the ground surface (BGS) at Boring B-06 to about 10 feet (about EL 4933 feet) at Boring B-05.

Standard Penetration Test (SPT) N-values ranged from 21 blows per foot of penetration to 50 blows for 5 inches of penetration, with an average N-value greater than 50 blows per foot of penetration. The SPT N-values may have been affected by the larger diameter gravel in the stratum.

Laboratory test results indicated moisture contents of 3% to 5%.

Silt with Sand to Silty Sand with Gravel (Native)

Below the granular embankment fill at Borings B-02 to B-04 was loose to dense silt with sand to silty sand with gravel. This native soil extended to a depth of about 10 feet (about EL 4934 feet) at Boring B-04 and to a depth of about 12.5 feet (EL 4931 feet) at Borings B-02 and B-03.

Standard Penetration Test N-values ranged from 5 to 38 blows per foot of penetration, with an average N-value of 20 blows per foot of penetration.

Laboratory test results indicated moisture contents of 7% to 19%, liquid limit (LL) values of NV (No Value), and plastic index (PI) values of NP (Non-plastic).

Poorly Graded Sand to Well- Graded Gravel with Silt and Sand

Below the silt with sand to silty sand with gravel was loose to very dense sand to gravel with silt and sand. This stratum was present to a depth of 30 feet (about EL 4913 feet) at Boring B-05 to 38.5 feet (about EL 4905 feet) at Boring B-03.

Standard Penetration Test N-values ranged from 6 blows per foot of penetration to 55 blows for 6 inches of penetration, with an average N-value of about 42 blows per foot of penetration. The SPT N-values may have been affected by the larger diameter gravel in the stratum.

Laboratory test results indicated moisture contents of 4% to 15%.

Upper Basalt Bedrock

Basalt bedrock was encountered at Borings B-02 to B-05. The top of the bedrock was encountered at the following approximate depths and elevations.

	Top of Bedrock		
Feature	Boring No.	Depth (Feet)	Elevation (Feet)
South Approach Embankment	B-02	37.5	4906.2
South Abutment	B- 03	38.5	4905.3
North Abutment	B- 04	32.5	4911.2
North Approach Embankment	B-05	30.0	4913.8

The upper part of the bedrock was weathered, and augers advanced to a depth of about 1.5 feet to 2.5 feet into the top of the bedrock before auger refusal occurred or rock coring commenced.

The bedrock extended to a depth of 67 feet (about EL 4877 feet) in Boring B-03 and to a depth of 50 feet (about EL 4894 ft) in Boring B-04.

Rock core recoveries from 40 feet to 65 feet in Boring B-03 were 100% and RQD values ranged from 43% to 98%. Rock core recovery from 65 feet to 70 feet was 67% and RQD was 15% because a soil layer was encountered from about 67 feet to 70 feet in the rock core.

Rock core recoveries from 35 feet to 45 feet in Boring B-04 varied from 93% to 100% and RQD values ranged from 45% to 63%. Rock core recovery from 45 feet to 50 feet was 73% and RQD value was 33% because interbedded soil seams were encountered in the rock core.

Unconfined compressive strengths of intact samples of the bedrock ranged from 4,702 to 18,586 pounds per square inch (psi), indicating the rock classified as medium strong (R3) to very strong (R5).

Photographs of the rock cores recovered from the site exploration borings are included in Appendix C.

Sandy Silt

Below the upper basalt bedrock at boring B-03 was a 5.5 feet thick stratum of sandy silt to a depth of 72.5 feet (about EL. 4871.5 feet).

Sand, Basalt Rubble and Gravel with Sand and Cobbles

Below the upper basalt bedrock at Boring B-04 was a 20 feet thickness of sand, basalt rubble and subrounded to rounded gravel with sand and cobbles to a depth of 70 feet (about EL. 4874 feet).

Lower Basalt Bedrock

Below the sandy silt in Boring B-03 was highly vesicular basalt bedrock to the bottom of the boring at a depth of 80 feet (EL. 4863.8 feet). Rock core recoveries from 70 feet to 80 feet were 77% to 100%, and RQD values were from 42% to 78%. Rock core recovery and RQD from 70 feet to 75 feet in Boring B-03 was low because a soil layer was encountered from 70 feet to about 72.5 feet in the rock core.

Below the gravel with sand and cobbles in Boring B-04 was highly vesicular basalt bedrock with interbedded soil layers to the bottom of the boring at a depth of 80 feet (EL. 4863.8 feet). Rock core recoveries from 70 feet to 80 feet were 70% to 93%, and RQD values were from 43% to 80%.

Photographs of the rock cores recovered from the site exploration borings are included in Appendix C.

Comments Regarding Subsurface Profile

The subsurface soils represent glacial outwash and flood deposits that can be highly variable.

Auger advancement during drilling was difficult at times though the sands and gravel at this site. Large cobbles and boulders did not appear to be encountered in the site investigation borings. However, based on visual observations of the borrow areas in the project vicinity cobbles (3 to 12 inches in nominal size - see Photo 5 in Appendix A) and possibly boulders (greater than 12 inches in diameter) can be encountered at any depth or location in these types of glacial outwash and flood deposits.

230.04.02 Groundwater

Water was flowing in the South Fork of the Teton River at the time of the site exploration in September 2024 at about 10 feet below the bridge deck (about EL. 4934 feet).

Groundwater was not encountered in the shallow roadway Borings B-01 and B-06 to a depth of 6.5 feet. Groundwater was observed in Borings B-02 to B-05 at depths ranging from 24.9 feet to 31.6 feet (EL 4918.9 feet to 4912.1 feet). Borings B-02 to B-05 were drilled about 30 feet to 150 feet south and north of the existing bridge abutments.

Based on Idaho Department of Water Resources (IDWR) Well Driller reports in the site vicinity, groundwater was encountered in the basalt bedrock at a depth of 80 feet to 96 feet, and the well logs indicate groundwater was not encountered in the soils above the basalt bedrock.

Fluctuations of the groundwater level at this site can occur due to seasonal water flows in the river, precipitation or other factors. Observing the actual changes in the seasonal groundwater level fluctuations is beyond the scope of this report.

230.04.03 Frost

Recommended frost depth in the project area is 3 feet.

230.04.04 Corrosion

According to the USDA NRCS Soil Resource Report, the corrosion potential of the surface soils is low for concrete and moderate for uncoated steel.

230.05 CONCLUSIONS AND RECOMMENDATIONS

230.05.01 General

Based on the subsurface conditions encountered at the boring exploration locations, the proposed replacement bridge can be supported by rock socketed shell piles.

The bridge abutments for the replacement bridge will be located behind the existing bridge abutments and the center pier constructed in the river. The structural designer should check that the foundations for the replacement bridge will be located such that the replacement bridge can be constructed without interference from the H-pile foundations for the existing bridge structure constructed in 1976 and the timber pile foundation locations for the bridge structure constructed in 1936.

The following sections recommend subgrade preparation, design pile resistances, anticipated performance, and construction considerations for the replacement bridge foundations. If revisions in the nature, design, location, or construction of the proposed bridge occur after this report has been submitted, the recommendations within the following sections are subject to review by Shannon & Wilson and may need to be modified as deemed necessary.

230.05.02 Foundations

230.05.02.01 Spread Footings

Due to potential scour from river flows, spread footing foundations are not recommended.

230.05.02.02 Deep Foundations

Abutments and Center Pier

<u>Rock Socketed Shell Piles</u>

It is recommended the bridge abutments, wing wall at the northeast corner of the bridge and center pier for the replacement bridge be supported on open end shell piles installed in concrete filled rock sockets.

At the bridge abutments and wing wall at the northeast corner of the bridge, 18-inch outside diameter (O.D.) shell piles will be installed into 24-inch diameter rock sockets excavated to a depth of 8 feet below the top of bedrock.

At the center pier, 24-inch outside diameter (O.D.) shell piles will be installed into 30-inch diameter rock sockets excavated to a depth of 8.5 feet below the top of bedrock.

The installation recommendations for the rock socketed shell piles are provided in Section 230.05.09 – Construction.

Pile Type – Spacing

Open-end, 18-inch and 24-inch O.D. steel shell piles with a wall thickness of ½ inch and meeting the requirement of ASTM A252 Grade 3 (45 ksi yield strength steel) are recommended. The shell piles can be filled with Class 30 concrete.

A minimum center-to-center spacing of 4.5B between shell piles is recommended, where B is the outside diameter of the shell piles.

Axial Pile Resistance

The axial pile resistance is based on the side resistance of the rock socketed shell piles.

For properly constructed rock socketed shell piles at the bridge abutments and wing wall at the northeast corner of the bridge installed to the bottom of excavated rock sockets filled with 8 feet of Class 40 concrete, the estimated nominal axial resistance of an 18-inch O.D. shell pile is 475 kips.

For properly constructed rock socketed shell piles at the center pier installed to the bottom of excavated rock sockets filled with 8.5 feet of Class 40 concrete, the estimated nominal axial resistance of a 24-inch O.D. shell pile is 745 kips.

A resistance factor of 0.5 may be assumed for the nominal axial resistance of the rock socketed shell piles for Strength Limit State design. Test piles are not required.

The nominal downward axial group capacity for the rock socketed piles may be taken as the sum of the individual pile capacities.

Settlement of Pile Group

Settlement is not a concern for properly constructed rock socketed shell piles.

Down Drag Load

Down drag is not a concern for the rock socketed shell piles.

Pile Uplift Resistance

It is assumed uplift loads will not act on the piles at the abutments or center pier.

Lateral Pile Resistance – Bridge Abutments

Preliminary pile lateral resistance analyses were performed using the computer program LPILE 6.0 and the subsurface conditions described in Section 230.04. The axial pile resistance for the LPILE analyses was assumed to be 50 percent of the recommended nominal axial resistance.

Based on the site exploration results, the parameters provided in the following table may be assumed for analyses of the piles at the bridge abutments and wing wall at the northeast corner of the bridge under lateral loading. It is assumed the subsurface condition at north abutment will control the lateral load analyses.

Soil Parameters for Lateral Pile Load Analysis (Abutments)		
Soil Parameter	Stratum	
	Sand and Gravel (below water table)	Concrete Filled Rock Socket (below water table)
Depth (ft) (Elevation (ft))	0 to 26 (4936 to 4910)	26 to 34 (4910 to 4902)
Consistency	Medium Dense to Very Dense	Weak Rock
Effective Unit Weight (pcf)	58[1]	150
Friction Angle (degrees)	32[1]	--
Unconfined Compressive Strength (psi)	--	10,000
RQD (%)	--	55
Young's Modulus for Intact Rock (Ei)	--	3,470 ksi
Young's Modulus for Rock Mass (E_m)	--	520 ksi
Constant (k_m)	--	0.0005
Initial Lateral Subgrade Modulus, k[2] (pci)	75[1]	--

NOTES:

1 Estimated from SPT N-values and visual examination of recovered soil sample and lab test results. Values are reduced to account for 6-inches of pea gravel placed between 30-inch temporary casing and 18-inch permanent shell pile.

2 For use with LPILE Program only. May not be used with other programs.

For fixed head conditions, the LPILE analyses were performed for 18-inch O.D. closed-end shell piles with ½ inch wall thickness, a yield stress of 45 ksi, and filled with Class 30 concrete. A 2H:1V front slope was assumed in front of the abutment piles.

Appendix E includes illustrated plots showing the estimated lateral deflection, moment, and shear caused by lateral loading on the piles. Calculated lateral pile resistances for typical lateral pile top deflections from the LPILE analyses results are shown in the table below.

Location - Pile Axis	Lateral Pile Resistances (kips) .25" Pile Top Deflection	2.0" Pile Top Deflection
Abutments	28	100

Lateral Pile Resistance – Center Pier

The parameters provided in the following table may be assumed for analyses of the rock socketed shell piles at the center pier under lateral loading.

Soil Parameter	Soil Parameters for Lateral Pile Load Analysis (Center Pier) Stratum		
	Cantilever Shell Pile	Sand and Gravel (below water table)	Concrete Filled Rock Socket (below water table)
Depth (ft) (Elevation (ft))	0 to 9 (4937 to 4928)	9 to 30 (4928 to 4907)	30 to 38.5 (4907 to 4998.5)
Consistency	--	Medium Dense to Very Dense	Weak Rock
Effective Unit Weight (pcf)	--	58[1]	150
Friction Angle (degrees)	--	32[1]	--
Unconfined Compressive Strength (psi)	--	--	10,000
RQD (%)	--	--	55
Young's Modulus for Intact Rock (Ei)	--	--	3,470 ksi
Young's Modulus for Rock Mass (E_m)	--	--	520 ksi
Constant (k_m)	--	--	0.0005
Initial Lateral Subgrade Modulus, k^2 (pci)	--	75[1]	--

NOTES:

1 Estimated from SPT N-values and visual examination of recovered soil sample and lab test results. Values are reduced to account for 6-inches of pea gravel placed between 36-inch temporary casing and 24-inch permanent shell pile.
2 For use with LPILE Program only. May not be used with other programs.

For fixed head conditions, the LPILE analyses were performed for 24-inch O.D. shell piles with ½ inch wall thickness installed in a concrete filled rock socket, with a yield stress of 45 ksi, and the shell pile filled with Class 30 concrete.

Appendix E includes illustrated plots showing the estimated lateral deflection, moment, and shear caused by lateral loading on the piles. Calculated lateral pile resistances for typical lateral pile top deflections from the LPILE analyses results are shown in the table below.

	Lateral Pile Resistances (kips)	
Location - Pile Axis	.25" Pile Top Deflection	2.0" Pile Top Deflection
Center Pier	25	110

The LPILE analysis indicates the point of fixity under lateral loading is near the top of the bedrock for shell piles in concrete filled rock sockets at the abutments and center pier.

The LPILE analyses for the abutments and center pier assumed only vertical and lateral loads acting at the top of the piles and did not include analyses of any overturning moments acting on the piles. If overturning moments are acting on the piles, the structural designer should perform additional LPILE analyses using the recommended soil parameters.

Group effects do not need to be included in the LPILE analyses for rock socketed shell piles.

Estimated Pile Tip Elevation and Length

At the north bridge abutment and wing wall at the northeast corner of the bridge, it is assumed the bottom of the pile cap will be at about EL 4936.2 feet, top of bedrock is about EL. 4910 feet and the piles will be installed 8 feet into bedrock to about EL. 4902 feet. Assuming piles are embedded about 2 feet into the bottom of the pile cap, the estimated pile length at the north abutment is about 36.2 feet.

At the south bridge abutment, it is assumed the bottom of the pile cap will be at about EL 4936.1 feet, top of bedrock is about EL. 4904 feet and the piles will be installed 8 feet into bedrock to about EL. 4896 feet. Assuming piles are embedded about 2 feet into the bottom of the pile cap, the estimated pile length at the south abutment is about 42.1 feet.

At the center pier, it is assumed the bottom of the pier column cap will be at about EL 4937.0 feet, top of bedrock is about EL. 4907 feet and the piles will be installed 8.5 feet into bedrock

to about EL. 4898.5 feet. Assuming piles are embedded about 2 feet into the bottom of the pile cap, the estimated pile length at the center pier is about 40.5 feet.

Highest pile tip elevations are the same as the estimated pile tip elevations mentioned above.

The bedrock at this site was sloping and undulating. It is emphasized that the top of bedrock elevations assumed are only estimates based on the exploration borings. The actual top of bedrock elevation encountered at the pile locations during construction could vary from the assumed elevations, and the installed pile lengths must be adjusted accordingly in the field. Piling must not be brought to the site precut to the estimated pile lengths.

If the design pile cap bottom elevations or pile embedment into the bottom of the pile caps are different from the above assumed values, the estimated pile lengths will also need to be adjusted accordingly.

230.05.03 Lateral Pressures and Backfill

230.05.03.01 Concrete Abutment Walls and Wing Walls

The soil parameters provided in the following table are recommended for design of concrete abutment walls and wing walls with approach slabs for backfill composed of ¾" Aggregate for Untreated Base (703.04). The ¾" Aggregate for Untreated Base backfill should extend below the approach slabs to a distance of 2 feet beyond the end of the approach slabs.

Soil Parameters & Earth Pressure Values for Design							
Parameter	Horizontal Backfill	2.5H:1V Slope Backfill		2.25H:1V Slope Backfill		2.0H:1V Slope Backfill	
Unit Weight (moist, compacted, γ_m (pcf.))	130	130		130		130	
Friction Angle, φ (deg.)	34	34		34		34	
Parameter	**Horizontal Backfill**	**Upward Slope**	**Downward Slope**	**Upward Slope**	**Downward Slope**	**Upward Slope**	**Downward Slope**
Active Earth Pressure, Coefficient, Ka	0.27	0.37	0.24	0.39	0.23	0.42	0.22
Active Equivalent Fluid Pressure (pcf.)	35	48	31	51	30	55	28
At-Rest Earth Pressure Coefficient, Ko	0.44	0.60	0.28	0.62	0.26	0.64	0.24
At-Rest Equivalent Fluid Pressure (pcf.)	57	78	36	81	34	83	31
Passive Earth Pressure Coefficient, Kp	3.70	7.94	1.66	8.79	1.51	10.0	1.34
Passive Equivalent Passive Fluid Pressure (pcf.)	480	1030	215	1140	195	1300	175

The earth pressure values provided in the table above are nominal values, and the designer should apply AASHTO LRFD Bridge Design Specifications resistance factors for design.

The nominal lateral earth pressure values provided in the table are for walls less than 20 feet in height with properly drained backfill and do not include hydrostatic pressures that can develop if groundwater or surface water is trapped behind a structure. The values in the table assume the back of walls are vertical and do not include friction between the back of the wall and backfill material.

Additional earth pressures resulting from loads applied at the ground surface should also be included in the design of abutment walls and wing walls. The additional earth pressures may be due to construction equipment loads, traffic loads, surface surcharge loads, point loads, line loads, and strip loads. Traffic loads should be evaluated in accordance with Tables 3.11.6.4-1 and 3.11.6.4-2 of the AASHTO LRFD Bridge Design Specifications using the estimated soil unit weight and active or at-rest earth pressure coefficients as appropriate in the table above.

Seismic considerations may also need to be included in the wall design. The Mononobe-Okabe Analysis, as described in Appendix A11 of the AASHTO LRFD Bridge Design Specifications, may be used to calculate the seismic soil forces acting on the walls.

Walls that are restrained against any movement should be designed for at-rest earth pressure. Refer to Table C3.11.1-1 in the AASHTO LRFD Bridge Design Specifications for required relative wall movements to activate the active or passive earth pressure, assuming wall backfill as dense sand. For wall movements between at-rest and active or passive conditions, the lateral earth pressures can be linearly interpolated between the at-rest and active or passive earth pressure values.

230.05.04 MSE Walls

Mechanically Stabilized Earth (MSE) walls are not planned for this project.

230.05.05 Anchors

Anchors are not anticipated on this project.

230.05.06 Drainage

Internal Drainage of Concrete Abutment Walls and Wing Walls

For concrete abutment walls and wing walls with approach slabs, it is recommended that ¾" Aggregate for Untreated Base (703.04) compacted per Section 301 requirements be used in the zone behind the wall face and below the approach slabs to a distance of 2 feet beyond the end of the approach slabs. This will facilitate drainage of water that may infiltrate into the backfill and reduce the potential development of hydrostatic forces acting on the walls.

Weep holes near the wall bottom or perforated pipe installed at the wall bottom are recommended to drain water from the wall backfill.

230.05.07 Embankments

General

Based on cross sections provided by Forsgren and Associates, the existing approach embankments leading up to the bridge will be widened up to about 10 feet on each side and raised up to about 3 feet.

New embankment fills for widening or raising placed outside of specified backfill areas behind concrete abutment and wings walls with approach slabs should consist of Granular Subbase (703.11) compacted to Class A requirements. Side slopes should be 2H:1V or flatter.

Section 230.05.09 – Construction in this report provides recommendations for embankment construction.

Global Stability Analyses

Approach Embankment (Station 29+00)

A global stability analysis was performed for the side slope of the approach embankments leading up to the bridge at Station 29+00 using the computer program SLOPE/W. This analysis assumed the approach embankments to have a width at the top of 40 feet, a height of 25 feet, and embankment side slopes of 2H:1V or flatter.

Traffic surcharge of 250 psf (2 feet of soil at unit weight of 125 pcf.) was included in the analysis.

The global stability analysis results (Appendix F) indicate an estimated external global stability factor of safety greater than 1.5 for static conditions and greater than 1.1 for seismic conditions. These factors of safety are considered adequate.

Approach Embankment (Station 32+50)

A global stability analysis was performed for the side slope of the approach embankments leading up to the bridge at Station 32+50 using the computer program SLOPE/W. This analysis assumed the approach embankments to have a width at the top of 40 feet, a height of up to 20 feet, and embankment side slopes of 2H:1V or flatter.

Traffic surcharge of 250 psf (2 feet of soil at unit weight of 125 pcf.) was included in the analysis.

The global stability analysis results (Appendix F) indicate an estimated external global stability factor of safety greater than 1.5 for static conditions and greater than 1.1 for seismic conditions. These factors of safety are considered adequate.

Settlement

Soils that exist below the approach embankments are medium dense to dense cohesionless soils. If the soils below the approach embankment fill areas are properly stripped, compacted, and prepared in accordance with Section 205.03.F of the ITD Standard Specifications, then settlement of the embankment fill is not expected to be a concern. Most of the settlement is expected to occur during construction of the approach embankments.

230.05.08 Erosion Control

Abutments

Embankment widening areas and areas in front of or behind bridge abutments and wing walls should be protected with appropriate Best Management Practices (BMPs) to reduce the potential for erosion. BMPs may include slope flattening, covering soils with topsoil and applying seeding, applying slope armor or rock mulch, or constructing concrete slope paving.

Surface runoff from paved surfaces and the bridge deck should not be allowed to discharge in an uncontrolled manner onto the embankment slopes or into areas in front of or behind the retaining walls. Curbs may be used to direct surface runoff to drop inlets or other collection and discharge features.

Riprap revetments or concrete paved ditches may be placed at the ends of walls, groin areas, or discharge drainage areas where surface water may be directed or concentrated. Discharge areas should be protected with riprap underlain by a Riprap/Erosion Control Geotextile (718.06) or lined with concrete to prevent soil erosion.

The civil designer should size and situate riprap and show the layout of drainage features on the project drawings. A modification to Table 711.04-2, with the appropriate Class, may be required, as provided in the following table:

On Page 677, Subsection 711.04 Riprap
Add the following rows of text to Table 711.04-2

Class	Nominal Riprap Size, D50 (b) (inches)	Percent of Rock Equal or Smaller, Dx	Range of Intermediate Dimensions © (inches)
I	6	100	12
		85	7-9
		50	5-7
		15	3-5
II	9	100	18
		85	11-14
		50	8-11
		15	5-8

Temporary Sediment and Erosion Controls

An erosion and sediment control plan should be prepared for this project.

230.05.09 Seismic Design

The location of the proposed bridge is approximately Latitude: 43.894504°N and Longitude: -111.678088°W. Based on the field exploration results, the site geology, and Table 3.10.3.1-1 of the AASHTO LRFD Bridge Design Specifications, the subsurface profile at the site can be designated as Site Class D.

The table below shows peak ground acceleration coefficient (PGA), short- and long-period spectral acceleration coefficients (S_s and S_1) for Site Class B, Site Factors (F_{pga}, F_a, and F_v) and modified acceleration coefficients for Site Class D. These values were obtained following the AASHTO LRFD Bridge Design Specifications based on a 7 percent probability of being exceeded in 75 years.

Acceleration Coefficients For Response Spectrum (7% probability of being exceeded in 75 years) (1,000 year return and 5% critical damping)			
Period (Seconds)	Standard Acceleration Coefficients for Site Class B	Site Factor for Site Class D	Modified Acceleration Coefficients for Site Class D
0.0	PGA = 0.139	F_{pga} = 1.522	A_S = 0.212
0.2	S_s = 0.336	F_a = 1.531	S_{DS} = 0.515
1.0	S_1 = 0.121	F_v = 2.317	S_{D1} = 0.280

Based on the ITD Materials Manual, Figure 630.05.01.01 "Active Faults in Idaho," active faults are not indicated as present within 6 miles of the bridge location. Therefore, a site-specific seismic analysis is not required in accordance with the AASHTO LRFD Bridge Design Specifications.

The risk of liquefaction or seismic settlement at this project site is low due to the generally dense coarse-grained subsurface soils, depth to bedrock, and low seismic acceleration values.

230.05.10 Construction

The roadway leading up to the proposed replacement bridge will be closed during construction, and traffic detoured. The stages of construction are:

Stage 1: Remove ½ of the existing bridge and install rock socketed piles at abutment and pier locations. Construct first ½ of bridge.

Stage 2: Remove the remaining section of the existing bridge and install rock socketed piles at abutment and pier locations. Construct remaining ½ of the new bridge.

230.05.10.01 Rock Socketed Pile Installation

The following installation procedure is recommended for rocketed socketed shell piles. Full-time observation of the rock socketed shell pile installation by the Engineer is required.

1. Advance a temporary casing with cutting teeth (30-inch diameter for abutments and wing wall at the northeast corner of the bridge, 36-inch diameter for pier) through the site soils to the top of the bedrock and twist the temporary casing 1 foot into the top of the bedrock.
2. Excavate soil within temporary casing to top of bedrock.

3. Drill a rock socket (24-inch-diameter for abutment and wing wall at the northeast corner of the bridge, 30-inch diameter for pier) from the top of the bedrock to a depth of 8 feet at abutments and to a depth of 8.5 feet at pier. Bedrock at the site has a medium strong (R3) to very strong (R5) with measured unconfined compressive strengths generally ranging from 4,702 to 18,586 pounds per square inch (psi).
4. Clean out all broken rock or loose sediment out of the bottom of the rock sockets prior to concrete placement. Thoroughly clean the sidewalls of the rock socket to allow for adequate side resistance to develop. Remove any sediment laden water from the soil and rock excavation and replace with clean water prior to placement of the concrete.
5. Place Class 40 concrete to the top of the rock socket. Limit coarse aggregate for Class 40 concrete to Size No 2b. Class 40 concrete for rock socket shall have a slump ranging from 7 to 8.5 inches.
6. Immediately install open-ended shell piles (18-inch diameter for abutments and wing wall at the northeast corner of the bridge, 24-inch diameter for pier) into the fresh concrete all the way to the bottom of the rock socket. Temporarily brace open-end shell pile in plumb, vertical position until concrete sets.
7. Cover top of shell pile after concrete has set and fill the annular space between the outside perimeter of the shell pile and the temporary steel casing with Pea Gravel (703.09), then remove the temporary casing. The casing can be removed during filling of the annular space with pea gravel, provided the bottom of the casing is always kept at a depth of at least 5 feet below the top of the pea gravel during removal.
8. Pump out any water within the shell piles after the piles have been installed and fill the inside of the shell piles with Class 30 concrete (Class 40 concrete can be utilized at Contractor discretion and cost).

<u>Concrete Placement in Rock Socket</u>

1. Use a rigid steel tremie pipe if concrete is to be placed through water into the rock socket excavation. Ensure the rigid steel tremie pipe extends all the way to the bottom of the rock socket prior to placement of the concrete. Prevent any water entering the bottom of the pipe prior to concrete placement by effectively plugging the end of the tremie pipe.

2. Provide an initial surge of concrete sufficient to displace water and embed the bottom of the tremie pipe within the concrete. Keep the bottom of the tremie pipe at least 5 feet below the top of the concrete being placed in a rock socket with water.
3. Pump and direct any water that is displaced or removed during the rock socketed pile construction to a collection area.

The above installation procedures and requirements for rock socketed pile installation, including full-time observation of the rock socketed pile construction by qualified inspection personnel, should be included on the Plan Sheets for this project.

230.05.10.02 Embankment Construction

Embankments should be constructed in accordance with Section 205.03.F of the ITD Standard Specifications.

Prior to placement of embankment fill, the ground surface in embankment widening areas should be stripped of existing vegetation, organic soils, loose soils, and debris. The depth of stripping is estimated to be about 1 foot.

Following stripping of the site soils, the subgrade surface should be scarified, moisture conditioned and compacted to Class A requirements in accordance with Section 205.03.G of the ITD Standard Specifications.

If loose or soft soils are encountered at the surface of the excavated subgrade that cannot be moisture conditioned and effectively compacted by repeated passes of an approved vibratory compactor, those soils, up to 2 feet in depth, or as directed by the Engineer, should be over excavated and replaced with Granular Borrow (205.02.B) or an approved material compacted to Class A requirements.

The excavation and preparation of the subgrade must be observed and approved by the Engineer prior to placement of the compacted embankment fill material.

230.05.10.03 Staged Construction for Settlement Control

Staged construction for settlement control is not expected to be required for the construction of the proposed bridge and approach embankments.

230.05.10.04 Excavation Slopes

Construction site safety and evaluating appropriate temporary excavation and construction slopes is the sole responsibility of the Contractor, who selects and directs the means,

methods, and sequencing of the construction operations. The Contractor must be familiar with, and comply with the applicable local, state, and federal safety regulations (including the current OSHA Excavation and Trench Safety Standards) for temporary excavations and construction slopes.

230.05.10.05 Temporary Shoring

If required, the Contractor must provide temporary shoring in accordance with Section 584 of the ITD Standard Specifications for Highway Construction.

The design, construction, maintenance, and removal of temporary shoring for the project are the sole responsibility of the Contractor. Contractor should conduct a subsurface exploration that is relevant to the Contractor's temporary shoring system

230.05.10.06 Cofferdam

Cofferdams will be constructed in the river to access and install rock socketed piles at the center pier.

The design, construction, maintenance, and removal of cofferdams for the project are the sole responsibility of the Contractor. Contractor should conduct a subsurface exploration that is relevant to the Contractor's cofferdam system.

230.05.10.07 Dewatering

The Q_{50} water surface level for the South Fork of the Teton River at the bridge location is EL 4939.2 feet. It is recommended that the proposed bridge be constructed during a period when water flow in the South Fork of the Teton River is seasonally low.

At the time of the site exploration in September 2024, water was flowing in the river at about El. 4935 feet. Groundwater at the abutment locations for the replacement bridge is anticipated to be the same as the water level in the river during the time of construction.

Groundwater was observed in site exploration Borings B-02 to B-05 at depths ranging from 24.9 feet to 31.6 feet (EL 4918.9 feet to 4912.1 feet). However, these borings were drilled about 30 feet to 150 feet south and north of the existing bridge abutments.

Idaho Department of Water Resources (IDWR) Well Driller reports beyond the bridge and river location indicate groundwater was encountered in the basalt bedrock at a depth of 80 feet to 96 feet, and the well logs indicated groundwater was not encountered in the soils above the basalt bedrock.

The design, construction, maintenance, and removal of dewatering systems needed for the project are the sole responsibility of the Contractor.

230.05.10.08 Utilities

Overhead power lines are present along the east side of the highway at the bridge location. Buried utility lines may also be present at the site.

Existing utility lines may need to be protected, relocated, replaced, or otherwise monitored by the Contractor during construction, so that subgrade excavations, compaction, fill placement, pile installation, ground settlement due to fill placement, or other construction operations do not cause damage to the existing utility lines.

230.05.10.09 Flexible Pavement Design

Existing Asphalt Concrete

Borings B-01 and B-06 were drilled through the Teton Highway pavement about 500 feet south and north of the existing bridge. The two borings encountered 0.46 feet to 0.50 feet (5.5 inches to 6 inches) of asphalt concrete overlying granular embankment fill.

Borings B-02 and B-05 were drilled through the Teton Highway pavement about 150 feet south and north of the existing bridge. The two borings encountered 0.42 feet to 0.46 feet (5 inches to 5.5 inches) of asphalt concrete overlying granular embankment fill.

Borings B-03 and B-04 were drilled through the Teton Highway pavement about 30 feet south and north of the existing bridge. The two borings encountered 0.42 feet (5 inches) of asphalt concrete overlying granular embankment fill.

Embankment Fill

Granular embankment fill encountered below the asphalt concrete to a depth of about 5 feet to 10 feet at the boring locations was medium dense to very dense silty sand with gravel to poorly graded gravel with silt and sand.

The following table summarizes the subgrade embankment fill soil characteristics.

Subgrade Soils Laboratory Test Results											
Sample		Moisture (%)	Atterberg Limits			Particle Size			R- Value	Sand Equiv.	USCS
Location	Depth (ft)		LL- (%)	PL- (%)	PI	Gravel (%)	Sand (%)	Fine (%)			
Subgrade (Embankment Fill)											
B-02	2 – 4	--	--	--	--	48	37	15	78	25	GM
B-05	2 – 4	--	--	--	--	53	37	10	81	33	GP-GM

The granular embankment material generally met ITD requirements for Granular Subbase (703.11) with adequate R-value properties.

Total Design Pavement Thickness

Flexible reconstruction is recommended for the roadway approaches on either side of the bridge. The Idaho R-value design methodology was used to prepare this report.

Traffic Load

ITD Roadway Data Section provided traffic data projections. The Traffic Index (TI) for the 20-year design traffic load is 8.7, assuming a 60/40 directional split and 100% of the truck traffic in the design lane. Appendix G contains the ITD traffic data and the traffic projection worksheet.

Flexible Pavement Design

Shannon & Wilson performed R-value testing on roadway embankment fill subgrade samples. An R-value of 60, along with a TI of 8.7, was assumed for the design.

The following table summarizes the flexible pavement layer thicknesses and the flexible pavement reconstruction design recommendations. The flexible pavement design calculations are in Appendix G.

Summary of Flexible Pavement Reconstruction (20-Year Analysis) Design	
Pavement Layer	Minimum Layer Thickness (feet)
HMA	0.35
Untreated Base	0.65
Granular Subbase	1.05
Total	2.05

Pavement Type and Surface Smoothness

For the Teton highway, the recommended pavement type is Superpave Hot Mix Asphalt (HMA), Class SP-3 (405).

The project should be constructed to Schedule II Surface Smoothness (405.03.P).

Typical Section

The following table summarizes the typical section for the project.

Flexible Pavement Reconstruction – South Fork Teton Bridge Traffic Loading – TI: 8.7, Flexible ESALs: 768,980			
	Thickness (feet)	Layer	Materials
	0.35	Superpave Hot Mix Asphalt Class SP-3 (405), 2 lifts	Aggregate for Superpave HMA Pavement, ½-inch (703.05) Asphalt, PG 58-34 (702.01)
	0.65	Aggregate Base (303)	Aggregate for Untreated Base, ¾-inch Type A (703.04)
	1.05	Granular Subbase (301)	Aggregate for Granular Subbase (703.11)

The asphalt PG grade for Superpave HMA is based on LTPPBind software (version 3.1) (2005) with a 98 percent desired reliability value. Appendix G includes the LTPPBind analysis results.

The existing embankment fill may be considered as Granular Subbase material. Depending on the vertical profile of the reconstructed Teton Highway, the aggregate base may be placed directly on the existing embankment fill, or on additional granular subbase imported and placed over the existing embankment fill.

If the HMA is carried across the concrete deck, then 0.35 feet of HMA (2 lifts) may be considered to facilitate future HMA rehabilitation strategies, such that future milling does not damage the deck membrane.

A tack coat should be placed between asphalt lifts.

230.06 APPENDICES

- Appendix A: Vicinity Map, Site Photographs, Historical ITD Bridge Drawings, Geosynthetic Reinforced Abutment Backfill
- Appendix B: Foundation Investigation Plat
- Appendix C: Subsurface Exploration Plan and Boring Logs
- Appendix D: Laboratory Testing
- Appendix E: LPILE Analyses
- Appendix F: SLOPE/W Stability Analysis
- Appendix G: Flexible Pavement Design Analyses

230.07 FOUNDATION INVESTIGATION PLAT

The Foundation Investigation Plat was prepared by Shannon & Wilson for this Geotechnical Engineering Report. A signed and sealed PDF of the Foundation Investigation Plat will be provided upon approval of this Geotechnical Engineering Report.

230.08 CURRENT SPECIFICATIONS AND MINIMUM TESTING REQUIREMENTS

This Geotechnical Engineering Report is based upon the following documents:

Idaho Transportation Department (ITD). 2020. Materials Manual.

Idaho Transportation Department (ITD). 2019. Quality Assurance Manual.

Idaho Transportation Department (ITD). 2023. Standard Specifications for Highway Construction.

Idaho Transportation Department (ITD). 2024. Supplemental Specifications for the 2023 Idaho Standard Specifications for Highway Construction.

230.09 MODIFICATIONS TO STANDARD SPECIFICATIONS

On Page 677, Subsection 711.04 Riprap
Add the following rows of text to Table 711.04-2

Class	Nominal Riprap Size, D50 (b) (inches)	Percent of Rock Equal or Smaller, Dx	Range of Intermediate Dimensions © (inches)
I	6	100	12
		85	7-9
		50	5-7
		15	3-5
II	9	100	18
		85	11-14
		50	8-11
		15	5-8

On Page 627, Subsection 703.02C Coarse Aggregate for Concrete

Limit coarse aggregate for Class 40 concrete to Size No 2b. Class 40 concrete for rock socket shall have a slump ranging from 7 to 8.5 inches.

On Page 393, Section 519 – Drilled-In-Concreted Piles

There are project specific geotechnical modifications that need to be made to the above Standard Specification. Shannon and Wilson will coordinate with ITD Anthony Beauchamp, State Geotechnical Engineer, phone number (208) 334-8554 or email anthony.beauchamp@itd.idaho.gov. or Tyler Coy, phone number (208) 334-8436 or email tyler.coy@itd.idaho.gov. and with Forsgren and Associates to modify and revise Section 519 for this project.

As previously mentioned, the installation procedures and requirements for rock socketed pile installation, including full-time observation of the rock socketed pile construction by qualified inspection personnel, should be included on the Plan Sheets for this project.

Notes to Contractor:

- Axial pile resistance is based on side resistance of the rock socket sidewalls. Thoroughly clean the sidewalls of the rock socket to allow for adequate side resistance to develop.

- Remove any sediment laden water from the soil and rock excavation and replace with clean water prior to placement of the concrete.

230.10 SPECIAL PROVISION ITEMS

There are no geotechnical Special Provisions needed for this project.

230.11 REFERENCES

Guides, Specifications, and Textbooks

American Association of State Highway and Transportation Officials. 2024. AASHTO LRFD Bridge Design Specifications. 10th Edition.

Hough, B.K. 1957. Basic Soils Engineering. New York: Ronald Press.

ITD Sources

Idaho Transportation Department (ITD). 2023. Standard Specifications for Highway Construction.

Idaho Transportation Department (ITD). 2024. Supplemental Specifications for 2023 Standard Specifications for Highway Construction.

Idaho Transportation Department (ITD). 2020. Materials Manual.

Idaho Transportation Department (ITD). 2024. Bridge Design LRFD Manual.

Reports

USDA NRCS. 1993. "Soil Resource Report for Fremont County Area, Idaho." Web Soil Survey. <https://www.nrcs.usda.gov/wps/portal/nrcs/site/national/home/>

Software

LPILE by Ensoft, 2022. Analysis of Piles and Drilled Shafts under Lateral Loads.

SLOPE/W by GeoStudio. Slope Stability Analysis. Version 2021.3

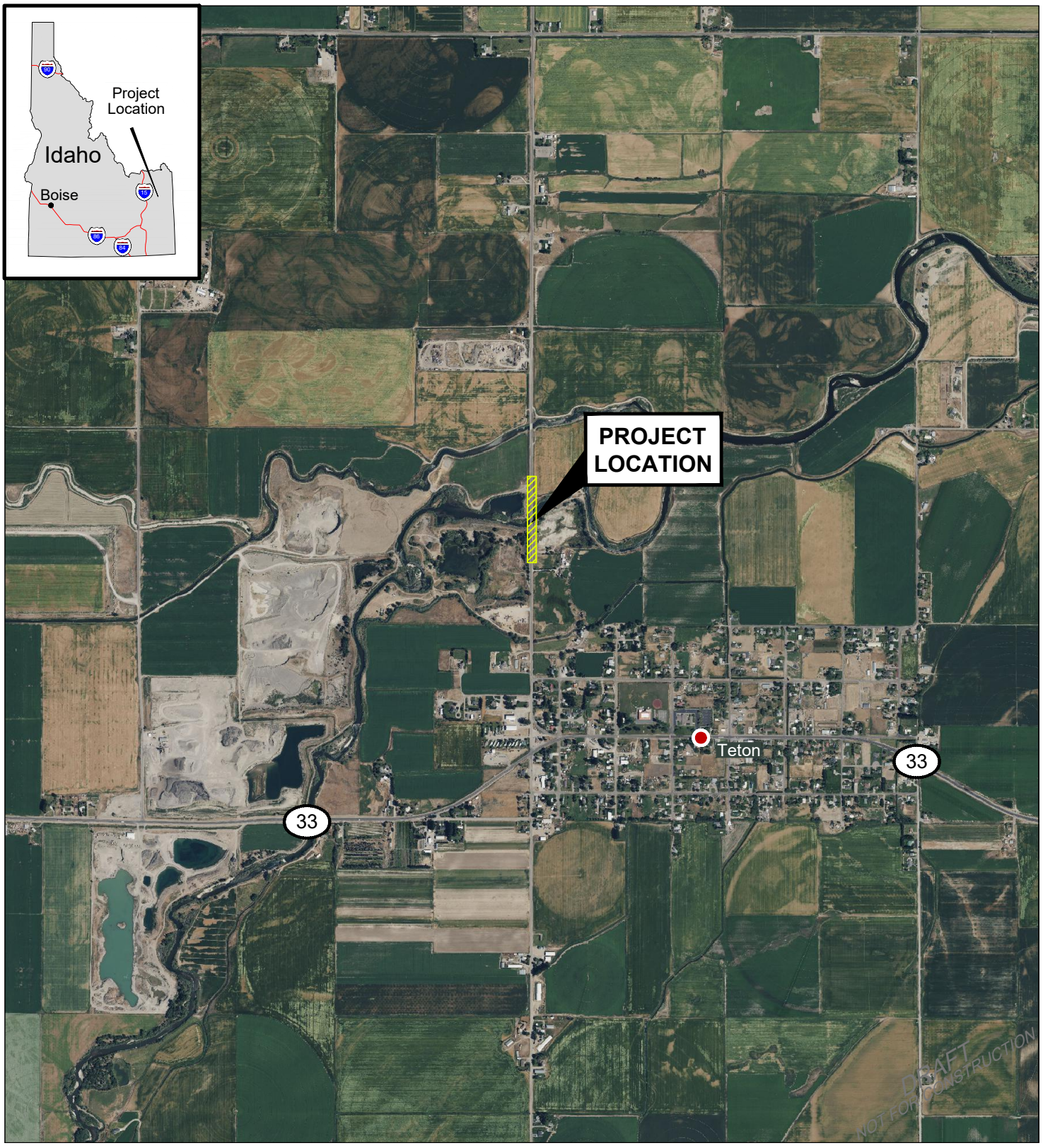
APPENDIX A – LOCATION, PHOTOS, HISTORICAL DRAWINGS

Appendix A

Location, Photos, Historical Drawings

CONTENTS

- Vicinity Map
- Site Photographs
- Historical Bridge Drawings



South Fork Teton River Bridge
Fremont County, Idaho

VICINITY MAP

November 13, 2024 71-112645

SHANNON & WILSON FIG. 1

112645 - Vic Map PREP 13-Nov-24.dwg
11/13/2024 11:02 AM
ACM


25'-7¼"
25'-2⅝"
25'-2⅝"
25'-7¼"
Sta 30+57.18
Sta 31+08
Sta 31+58.82
Widen Shoulder
Widen Shoulder
Widen Shoulder
30
31
32
KN24341 S. Fk Teton R. Bridge
21035
Image © 2024 Airbus



Photo 1: Looking South on Teton Hwy towards Bridge (Overhead Lines on East Side of Hwy)



Photo 2: West Side of Bridge Looking East (Upstream)



Photo 3: East Side of Bridge (Utility Line Connected to Bridge)



Photo 4: Borrow Area Southeast of Bridge



Photo 5: Cobbles in Borrow Area Southeast of Bridge



Photo 6: Looking Downstream (West) from Bridge (Riprap along South Bank)



Photo 7: Riprap - South Bridge Abutment



Photo 8: Riprap - North Bridge Abutment



Photo 9 – Scanline Area – North Channel Bottom Area



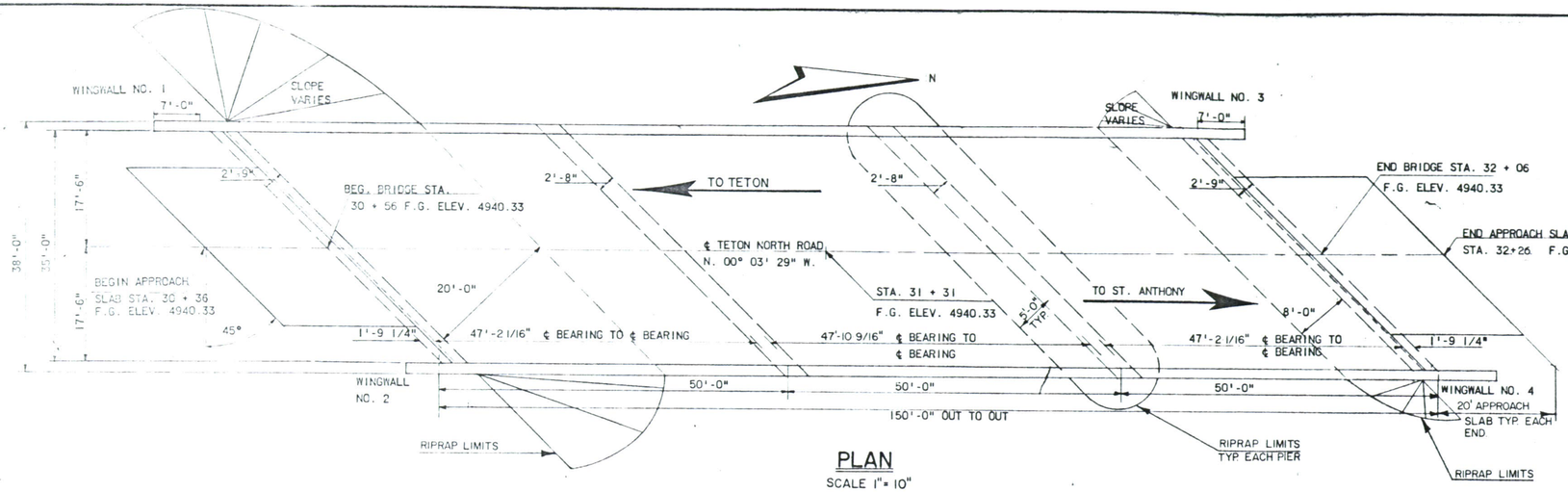
Photo 10 – Scanline Area – South Channel Bottom Area

FEDERAL ROAD REGION NO.	STATE	FEDERAL AID PROJECT NO.
10	IDAHO	ER-6774(7)

SEC. A

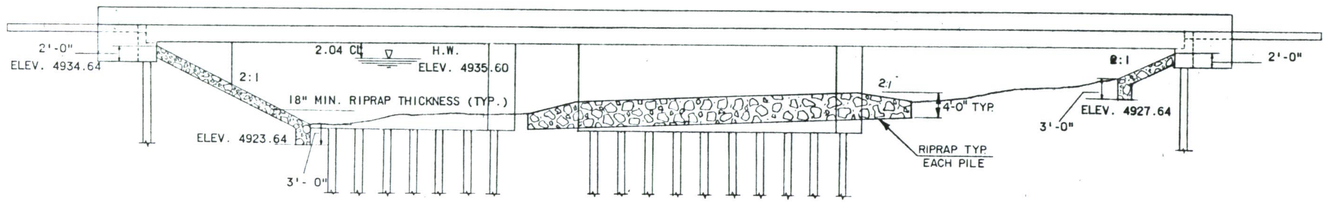
DRAWING INDEX

SITUATION AND LAYOUT	SHEET 1 OF 14
FOUNDATION INVESTIGATION	" 2 " "
PILE LOCATION PLAN	" 3 " "
DECK PLAN	" 4 " "
PRESTRESSED VOIDED SLAB	" 5 " "
ABUTMENT DETAILS	" 6 " "
WINGWALL DETAILS	" 7 " "
PIER DETAILS	" 8 " "
STANDARD BRIDGE DRAIN DETAILS	" 9 " "
STANDARD CONCRETE PARAPET - TYPE IV SHEET 1	" 10 " "
STANDARD CONCRETE PARAPET - TYPE IV SHEET 2	" 11 " "
STEEL PILE DETAILS	" 12 " "
APPROACH SLAB	" 13 " "
METAL REINFORCEMENT DETAILS	" 14 " "



PLAN

SCALE 1"= 10"



ELEVATION

SCALE 1"= 10"

HYDRAULIC DATA

Q_{50} = 4980 CFS
V_{50} = 6.4 FPS
A = 1010 SQ. FT.

TRAFFIC DATA

1978 ADT 640
1978 ADT 670
1998 ADT 930
1998 DHV 112
DESIGN SPEED 50 MPH

QUANTITIES

ITEM NO.	ITEM	UNIT	QUANTITY
210-B-1	STRUCTURE EXCAVATION - SCHED. NO.1	CY	105
210-C	COMPACTING BACKFILL	CY	1150
502-F-1	2'-2" X 3' WIDE PRESTRESSED VOIDED SLAB	LF	293
502-F-2	2'-2" X 4' WIDE PRESTRESSED VOIDED SLAB	LF	1174
502-B-1	CONCRETE CLASS 30 - SCHED. NO. 1	CY	166
502-C-7	CONCRETE CLASS 50 - SCHED. NO. 2	CY	8
502-G	CONCRETE PARAPET - TYPE IV	LF	328
502-H	APPROACH SLAB	SY	113
503-B	METAL REINFORCEMENT - SCHED. NO. 1	LB	16697
505-C	FURNISHING AND DRIVING H-BEAM PILES	LF	978
505-E	FURNISHING AND DRIVING TEST PILES	LF	30
624	RIPRAP	CY	340
5PB-1	DECK SEAL CONCRETE	SY	577

GENERAL NOTES

MATERIALS, CONSTRUCTION AND WORKMANSHIP SHALL CONFORM WITH IDAHO DEPARTMENT OF HIGHWAYS STANDARD SPECIFICATIONS, 1967 EDITION, & 1974 SUPPLEMENTAL SPECIFICATIONS, UNLESS NOTED OTHERWISE.

METAL REINFORCEMENT SHALL CONFORM WITH ASTM SPECIFICATION A615, GRADE 40 OR 60.

METAL REINFORCEMENT SHALL HAVE A 2" COVERING OUTSIDE OF BARS UNLESS NOTED OTHERWISE.

ALL ITEMS SHOWN OR NOTED ON THE PLANS WHICH ARE NOT SPECIFICALLY A BID ITEM ARE CONSIDERED INCIDENTAL ITEMS. THE COST OF FURNISHING AND INSTALLING ALL INCIDENTAL ITEMS WILL NOT BE PAID SEPARATELY, BUT SHALL BE INCLUDED IN THE UNIT PRICES BID FOR OTHER ITEMS, UNLESS NOTED OTHERWISE.

CONCRETE IN PRESTRESSED DECK SLABS SHALL CONFORM TO CLASS 50.

CONCRETE CLASS 40A SHALL BE USED IN THE PARAPETS. ALL OTHER CONCRETE SHALL CONFORM TO CLASS 30.

EXPOSED CONCRETE SURFACES SHALL NOT BE PAINTED.

COMPACTED BACKFILL SHALL BE FREE DRAINING.

DESIGN NOTES

STRUCTURE DESIGN IN ACCORDANCE WITH 1973 AASHO SPECIFICATIONS, & 1974-1976 INTERIM SPECIFICATIONS.

LIVE LOAD; HS 20-44. IMPACT IN ACCORDANCE WITH SPECIFICATIONS.

UNIT STRESSES:

CONCRETE:	CLASS 30	f'c = 3,000 PSI IN 28 DAYS
		fc = 1,200 PSI
	CLASS 40A	f'c = 4,000 PSI IN 28 DAYS
		fc = 1,600 PSI
	CLASS 50	f'c = 5,000 PSI IN 28 DAYS
		fc = 2,000 PSI
	METAL REINFORCEMENT:	fs =20,000 PSI

DESIGNED FOR 22 PSF F.W.S.

75% OF RIPRAP SHALL BE 10"-18" IN DIAMETER.

PILES SHALL BE DRIVEN TO A MINIMUM BEARING CAPACITY OF 50 TONS.

PRESTRESSED SLAB DESIGNED FULLY CONTINUOUS FOR LIVE LOAD

James D. Kelly
PROFESSIONAL ENGINEER
STATE OF IDAHO

SCALES SHOWN ARE FOR 22" X 36" PRINTS ONLY

SITUATION AND LAYOUT

150 FOOT PRESTRESSED CONCRETE BRIDGE OVER
SOUTH FORK TETON RIVER — STATION 31+31
NORTH TETON ROAD — FREMONT COUNTY

DESIGNED BY: BGH
DESIGN CHECKED BY: JLK
DRAWN BY: MV
DRAWING CHECKED BY: BGH
ENGINEERING FEATURES
APPROVED BY: Harry Perkins
DATE 9/10/76 PRES. OR SENIOR PARTNER

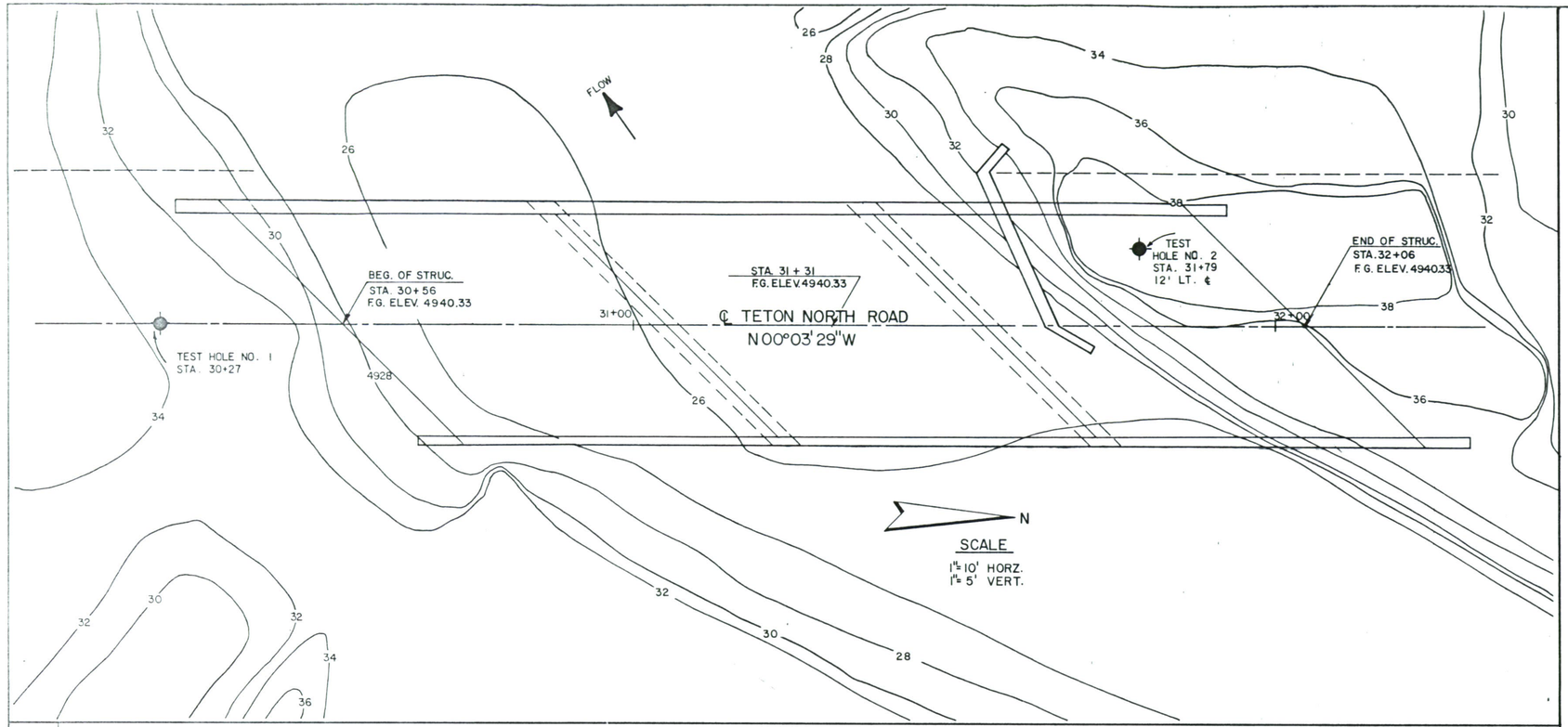
APPROVED AS TO GENERAL USABILITY
CHAIRMAN FREMONT COUNTY COMMISSIONERS
STATE OF IDAHO

FILE NO.	DWG. NO.	SHEET NO.
586	13708	1 OF 14

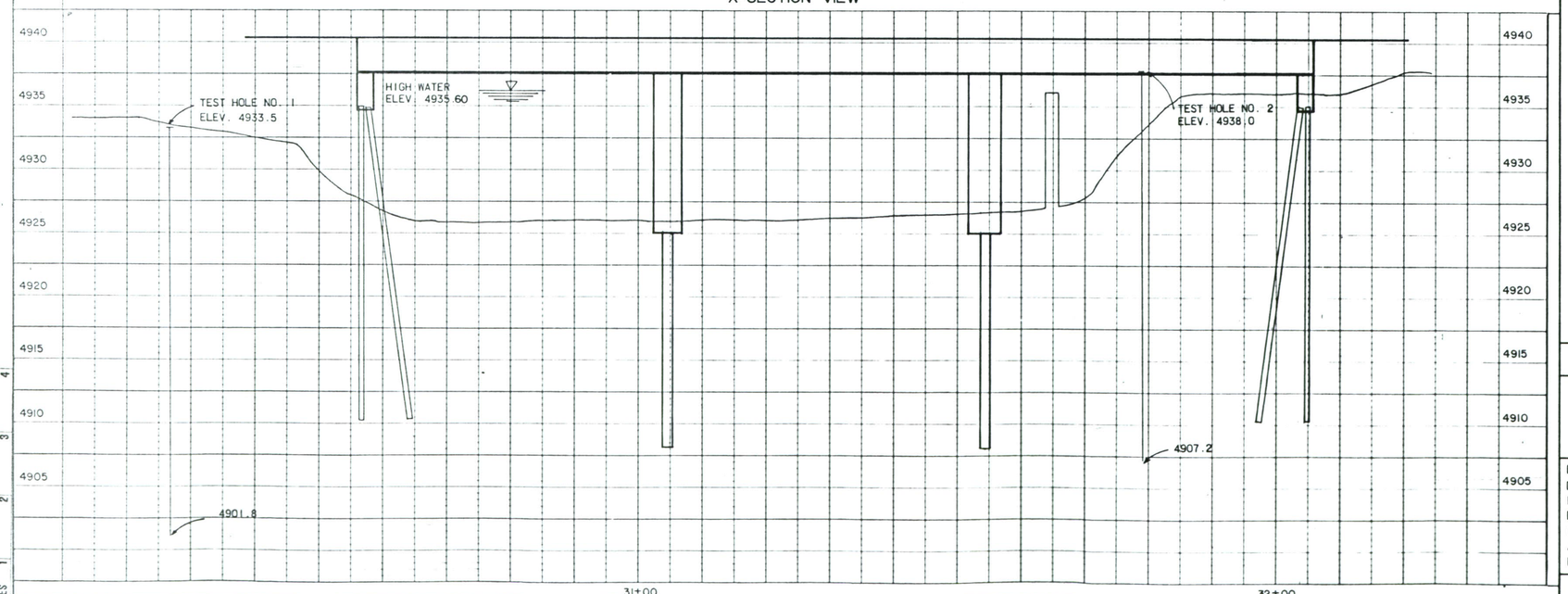
MICROFILM RECORD
DATE 11-19-82
ROLL NO. 538
AP-CARD or FRAME NO. 41
METRIC 2 3 4 5 6 7 8 9 10
INCHES 1 2 3 4

FEDERAL ROAD REGION NO.	STATE	FEDERAL AID PROJECT NO.
10	IDAHO	ER-6774(7)

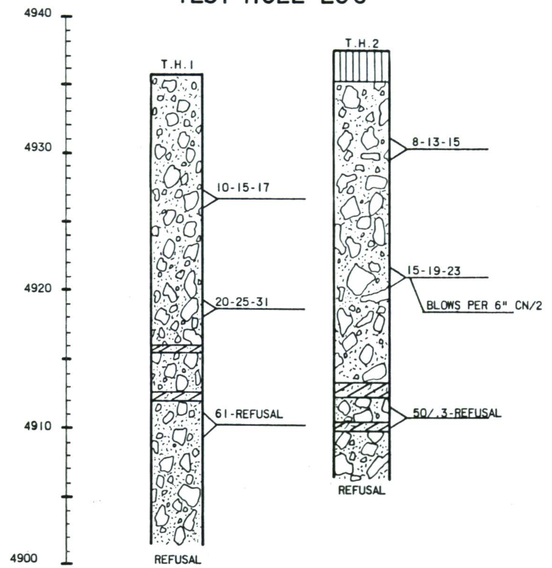
SEC. A



X-SECTION VIEW



TEST HOLE LOG



LEGEND	CLASSIFICATION	NOMENCLATURE
		ASPHALT, ROAD MIX
	7	SAND AND GRAVEL (CLAY) MAX. PARTICLE SIZE = 3"
	13	SILTY SAND

REGISTERED PROFESSIONAL ENGINEER — STATE OF IDAHO — JAMES L. KELLER

N = NUMBER OF BLOWS REQUIRED TO DRIVE 2" O.D. SPLIT SPOON SAMPLER = 1 FOOT INTO MATERIAL, USING A 140 lb. HAMMER WITH A 30" DROP.

HOLLOW STEM AUGER TEST HOLE

SCALES SHOWN ARE FOR 36" x 22" PRINTS ONLY

FOUNDATION INVESTIGATION

150 FOOT PRESTRESSED CONCRETE BRIDGE OVER

SOUTH FORK TETON RIVER — STA. 31+31

NORTH TETON ROAD — FREMONT COUNTY

DESIGNED BY	BGH
DESIGN CHECKED BY	JLK
DRAWN BY	GKM
DRAWING CHECKED	BGH
ENGINEERING FEATURES APPROVED BY	PRES. OR SENIOR PARTNER
DATE	9/10/76

APPROVED AS TO GENERAL USABILITY

CHAIRMAN FREMONT COUNTY COMMISSIONERS

STATE OF IDAHO

FILE NO.	DWG. NO.	SHEET NO.
586	13708	2 OF 14

PLATE 1, PLAN-PROFILE

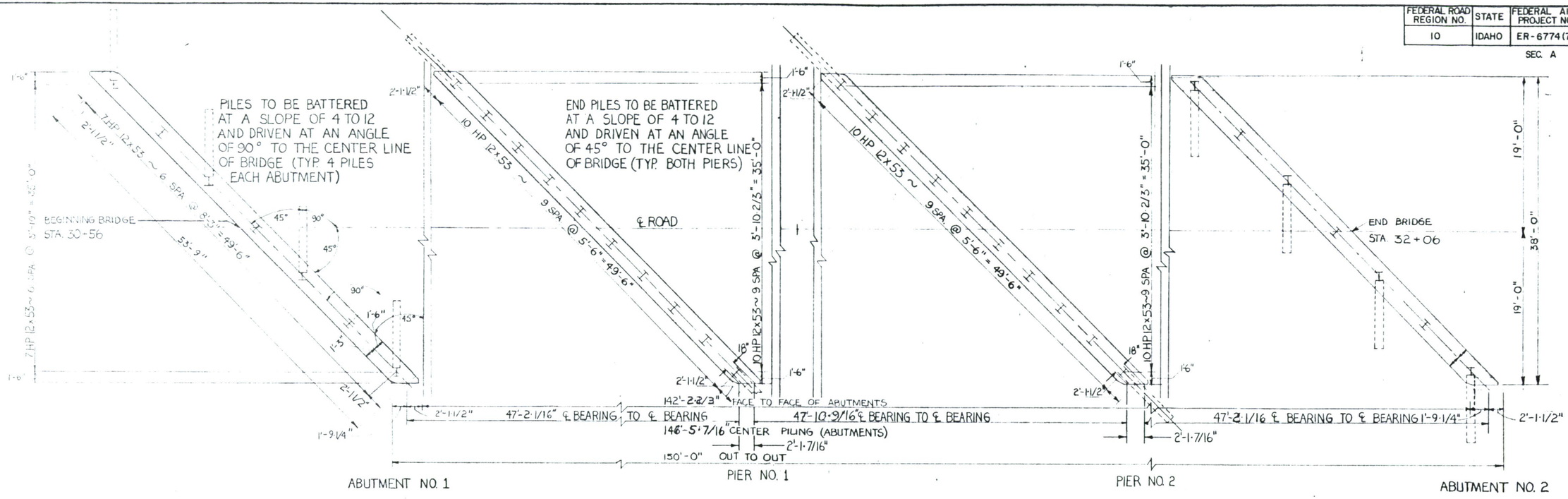
KEUFFEL & ESSER CO.

MICROFILM RECORD — DATE 11-19-82 — ROLL NO. 8-33 — AP-CARD or FRAME NO. 42

PROFILE — NOTE BOOK 1014

FEDERAL ROAD REGION NO.	STATE	FEDERAL AID PROJECT NO.
10	IDAHO	ER-6774(7)

SEC. A



PILE LAYOUT PLAN

SCALE: 1/6" = 1'-0"

NOTES:

PILING SHALL BE DRIVEN TO A BEARING VALUE OF 50 TONS PER PILE MIN..

PILE PENETRATION SHALL NOT BE LESS THAN 28 FEET.

FOR PILING DETAILS SEE SHEET NO. 12 OF 14.

MICROFILM RECORD

DATE	ROLL NO.	AP-CARD or FRAME NO.
11-19-82	5233	43

METRIC 2 3 4 5 6 7 8 9 10

INCHES 1 2 3 4

REGISTERED PROFESSIONAL ENGINEER STATE OF IDAHO JAMES L. KELLER

James L. Keller

PILE LOCATION PLAN

150 FOOT PRESTRESSED CONCRETE BRIDGE OVER SOUTH FORK TETON RIVER STA. 31+31

NORTH TETON ROAD FREMONT COUNTY

DESIGNED BY	BGH
DESIGN CHECKED BY	JLK
DRAWN BY	GKO
DRAWING CHECKED	BGH
ENGINEERING FEATURES	
APPROVED BY	PRES. OR SENIOR PARTNER
DATE	9/10/76

APPROVED AS TO GENERAL USABILITY

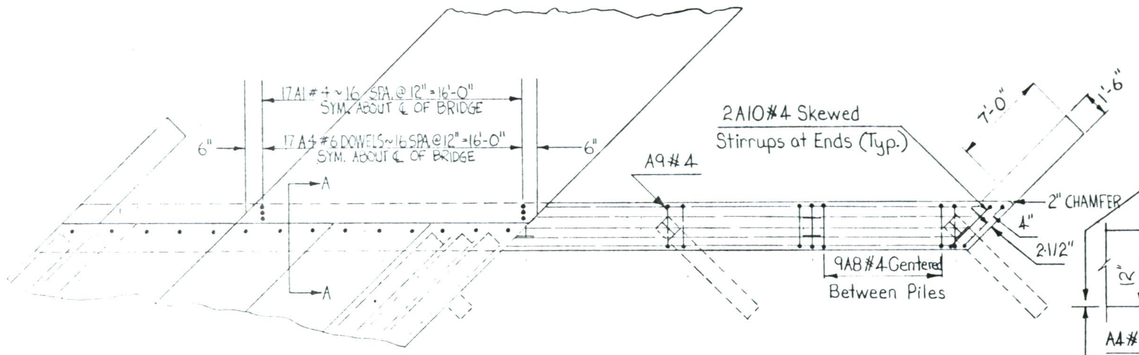
CHAIRMAN FREMONT COUNTY COMMISSIONERS

STATE OF IDAHO

FILE NO.	DWG. NO.	SHEET NO.
586	13708	3 of 14

FEDERAL ROAD REGION NO.	STATE	FEDERAL AID PROJECT NO.
10	IDAHO	ER-6774(7)

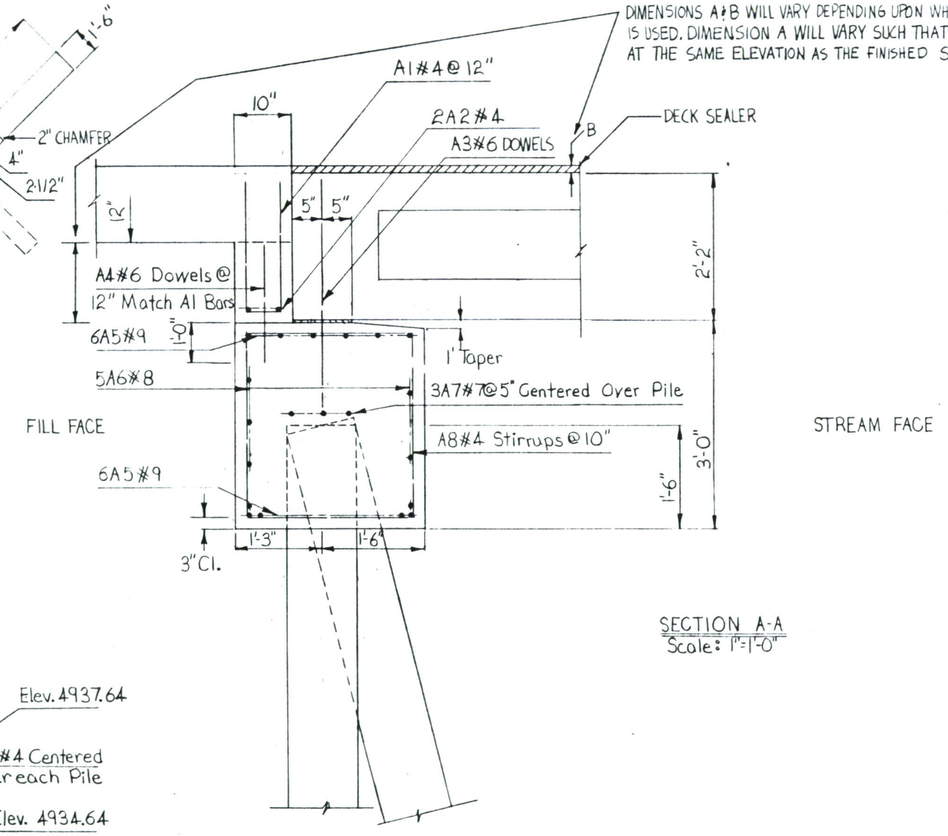
SEC. A



ABUTMENT PLAN
Scale: 1/4"=1'-0"

ABUTMENT ELEVATION
Scale: 1/4"= 1'-0"

DIMENSIONS A & B WILL VARY DEPENDING UPON WHICH METHOD OF DECK SEALER IS USED. DIMENSION A WILL VARY SUCH THAT TOP OF APPROACH SLAB IS AT THE SAME ELEVATION AS THE FINISHED SURFACE.



SECTION A-A
Scale: 1"=1'-0"

SEE PILE LAYOUT SHEET FOR BATTER OF PILES.

REGISTERED PROFESSIONAL ENGINEER — STATE OF IDAHO — JAMES L. KELLER — 2394

MICROFILM RECORD

DATE	ROLL NO.	AP-CARD or FRAME NO.
11-19-82	33	46

METRIC 2 3 4 5 6 7 8 9 10
INCHES 1 2 3 4

ABUTMENT DETAIL

150 FOOT PRESTRESSED CONCRETE BRIDGE OVER
SOUTH FORK TETON RIVER — STATION 31+31
NORTH TETON ROAD — FREMONT COUNTY

DESIGNED BY: BGH
DESIGN CHECKED BY: JLK
DRAWN BY: DRK
DRAWING CHECKED BY: BGH
ENGINEERING FEATURES APPROVED BY: Darryl Perkins
DATE: 9/10/76

APPROVED AS TO GENERAL USABILITY

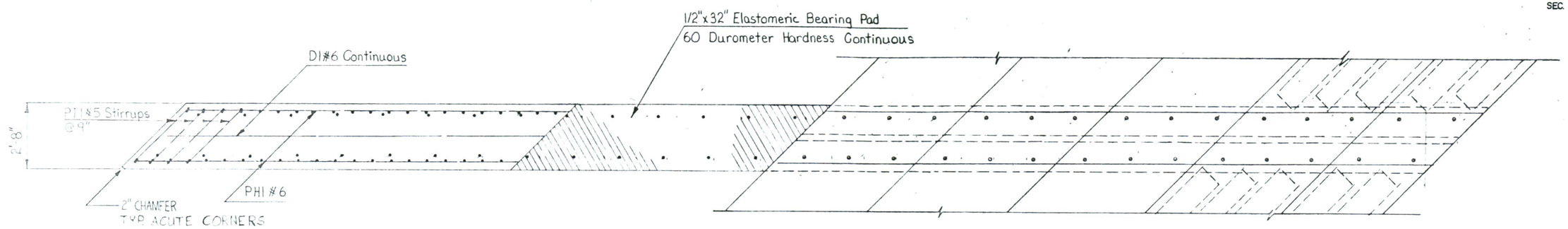
CHAIRMAN FREMONT COUNTY COMMISSIONERS

STATE OF IDAHO

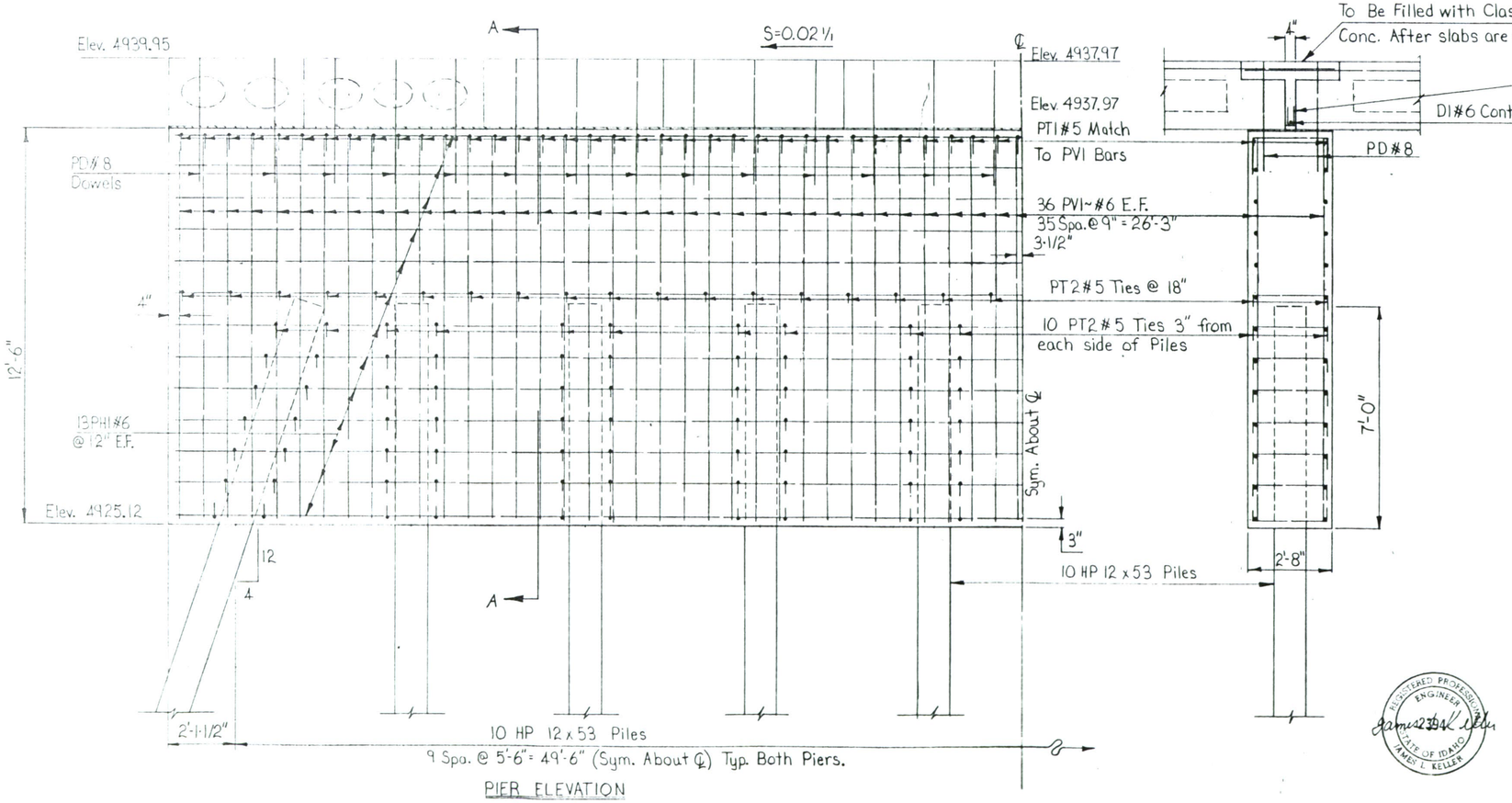
FILE NO.	DWG. NO.	SHEET NO.
586	13708	6 of 14

FEDERAL ROAD REGION NO.	STATE	FEDERAL AID PROJECT NO.
10	IDAHO	ER-6774(7)

SEC. A



PIER PLAN
Scale: 1/2"=1'-0"



PIER ELEVATION
Scale: 1/2"=1'-0"

SECTION A-A
Scale: 1/2"=1'-0"

SCALES SHOWN ARE FOR 36"x 22" PRINTS ONLY

PIER DETAILS

150 FOOT PRESTRESSED CONCRETE BRIDGE OVER SOUTH FORK TETON RIVER

STATION 31+31

NORTH TETON ROAD

FREMONT COUNTY

DESIGNED BY:	BGH
DESIGN CHECKED BY:	JLK
DRAWN BY:	DRK
DRAWING CHECKED BY:	BGH
ENGINEERING FEATURES APPROVED BY:	PRES. OR SENIOR PARTNER
DATE:	9/10/76

APPROVED AS TO GENERAL USABILITY

CHAIRMAN FREMONT COUNTY COMMISSIONERS

STATE OF IDAHO

FILE NO.	DWG. NO.	SHEET NO.
586	13708	8 OF 14

REGISTERED PROFESSIONAL ENGINEER — JAMES L. KELLER — STATE OF IDAHO

MICROFILM RECORD
DATE 11-12-82
ROLL NO. 35 33
AP-CARD or FRAME NO. 48
METRIC 2 3 4 5 6 7 8 9 10
INCHES 1 2 3 4

W.P.S.O.

Fed. Road Dist. No.	State	Fed. Aid Proj. No.	Total Sheets	Sheet No.
12	IDAHO	217A		

101' TIMBER BRIDGE
TIMBER SCHEDULE

Sta 31+08 — W.P.S.O. 217-A

DESCRIPTION	FINISH TO	FINISH	SIZE	LENGTH	NO	F.B.M. UNTREAT	F.B.M. TREATED
Rail Post	7½"×7½"	S4S	8"×8"	16'-0"	2	171	
" "	7½"×7½"	S4S	8"×8"	12'-0"	5	320	
" "	5½"×7½"	S4S	6"×8"	18'-0"	8	576	
Rail	2⅝"×7½"	S4S	3"×8"	20'-0"	1	40	
"	2⅝"×7½"	S4S	3"×8"	26'-0"	8	416	
"	3⅝"×7½"	S4S	4"×8"	20'-0"	1	53	
"	3⅝"×7½"	S4S	4"×8"	26'-0"	8	555	
Curb	7½"×7½"	S4S	8"×8"	20'-0"	1		107
"	7½"×7½"	S4S	8"×8"	26'-0"	8		1109
Pavement Stop	1½"×7½"	S4S	2"×8"	26'-0"	8		278
Curb Blocks	2"×7½"	S2E	2"×8"	20'-0"	4		107
Floor	1⅝"×5⅞"	S4S	2"×6"	24'-0"	751		18024
Stringers	full Sawn	Rough	8"×22"	26'-0"	36		13728
Stringer Splices	" "	"	8"×22"	24'-0"	1		352
Caps		"	12"×12"	28'-0"	5		1680
Blocks		"	6"×22"	12'-0"	4		528
Bridging		"	2"×4"	14'-0"	26		243
Shear Blocks		"	6"×6"	10'-0"	5		150
Backing Plank		"	3"×12"	14'-0"	16		672
" "		"	3"×12"	16'-0"	14		672
Wing Plank		"	3"×12"	12'-0"	4		144
"		"	3"×12"	10'-0"	12		360
Nailing Strip		"	3"×8"	14'-0"	2		56
" "		"	3"×6"	12'-0"	2		36
Braces		"	3"×8"	30'-0"	6		360
			TOTAL			2131	38,606

Piling Treated - 1080 Lin Ft.

To accompany Dwgs. No. 2909 to 2911

101' TIMBER BRIDGE
HARDWARE SCHEDULE

Sta. 31+08 — W.P.S.O. 217-A

DESCRIPTION	SIZE	LENGTH	NO	WT.	
Carriage Bolts	⅜"	10½"	176	59	
" "	⅜"	12"	16	6	
Bolts	¾"	13"	4	8	
"	¾"	16"	46	106	
"	¾"	18"	124	315	
"					
"	¾"	24"	36	118	
"					
"	¾"	35"	14	65	
"	¾"	42"	64	353	
"	¾"	26"	12	43	
Lag Screws	¾"	14"	4	7	
" "	¾"	30	4	15	
Drift Bolts	¾"	24	30	91	
⅜" Washers for ⅜" Bolt			192	3	
½" " " " "			192	8	
Pl. Washers for ¾" Bolt			56	6	
O.G. Washers for ¾" Bolts			552	690	
10d Nails				8	
40d Nails				675	
60d Nails				76	
			TOTAL LBS.	2652	

Contractor to verify bolt lengths before ordering.

To accompany Dwgs. No. 2909 to 2911

MICROFILM RECORD — DATE 8-12-77 — ROLL NO. 7 — AP-CARD or FRAME NO. 149

METRIC 2" 3 4 5 6 7 8 9 10 4 — INCHES 1 2 3

GENERAL NOTES:-

Structure designed in accordance with AASHO Specification of Dec. 1, 1926. H-15 Loading, no impact.

Floor and Stringers designed for a future 2½" Oil Mat wearing surface.

All lumber shall pass the requirements for select structural grade of Coast Region Douglas Fir, as defined in the grading rules of the West Coast Lumberman's Association dated July 1, 1934.

All lumber except railing and rail posts, including piling, shall be pressure treated to contain not less than 8 Lbs. of creosote oil per cu. ft by the empty cell process.

Cut timber surfaces and bolt holes drilled after treatment shall be given two coats of hot tar or asphalt

Portion of rail posts buried in the ground shall be given two coats of hot tar or as asphalt

Railing, rail posts, pavement stop, flooring and curb to be S4S. Curb blocks to be S2E. All other lumber to be rough.

Railing and rail posts above top of curb shall be given two coats of white lead and oil. Portion of rail posts below top of curb shall be given two coats of a good black paint

Bearing value of piling shall not be less than 15 tons per pile.

Any excess length of bolts in the rail or curb shall be cut off.

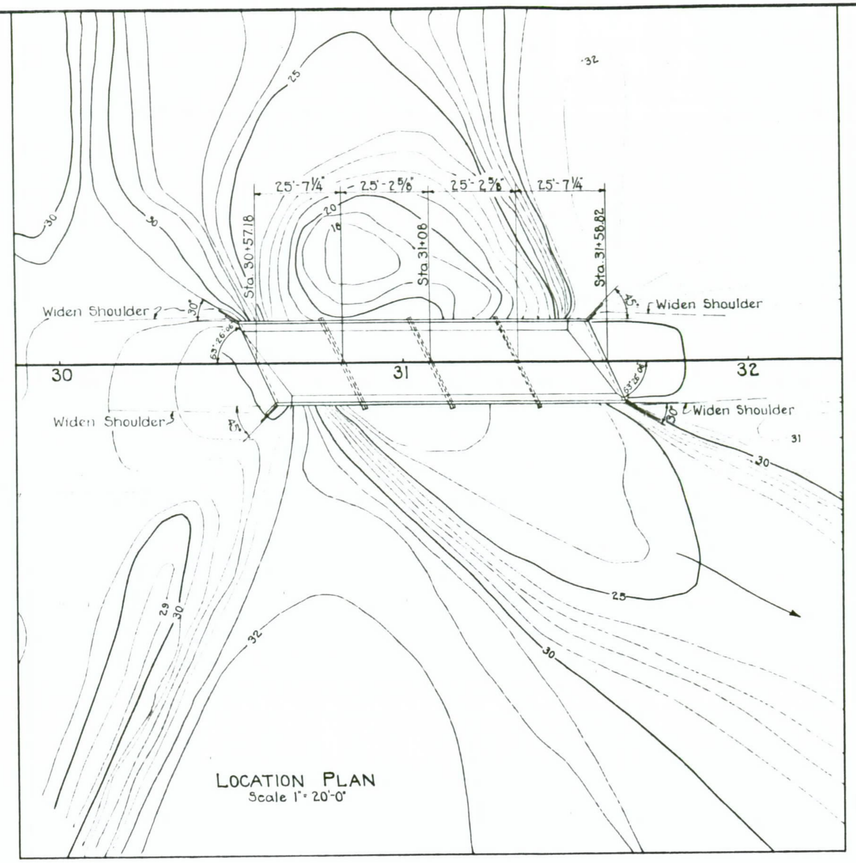
Use Bureau of Highways Standard Specification except as otherwise noted.

Payment for hardware to be included in the bid price for lumber.

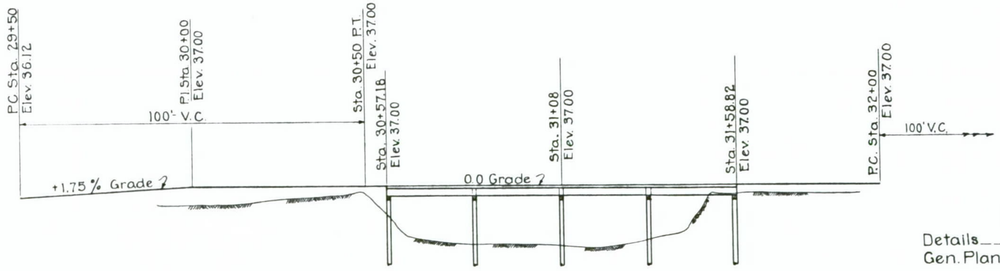
Lengths of piling are for estimating purposes only.

Excavation to be computed from measurements taken after old structure is removed

Stringers to be full sawn.



LOCATION PLAN
Scale 1" = 20'-0"



℄ PROFILE
Scale: 1"=20'-0"
NOTE:- Elevation given are to finished grades.

DRAWINGS

Details _____ Drwg. No. 2910 - 11
Gen. Plan & Quantities _____ " _____ 2909

BRIDGE HAS BEEN REMOVED. ORIGINAL DRAWINGS HAVE BEEN DESTROYED. SEE NEW DRAWING # 13208

Destroyed by Teton Dam Failure

QUANTITIES

C. D. Fir Treated	38,606	M.F.B.M.
C. D. Fir Untreated	2,131	" " "
Piling Treated	1080	Lin. Ft
Remove Present Structure	Lump Sum	
Excavation - Structure	20	Cu. Yds.

Revised 1-28-36

101' TIMBER BRIDGE OVER TETON RIVER
Sta. 31+08
TETON CITY - ST ANTHONY HWY. FREMONT CO.

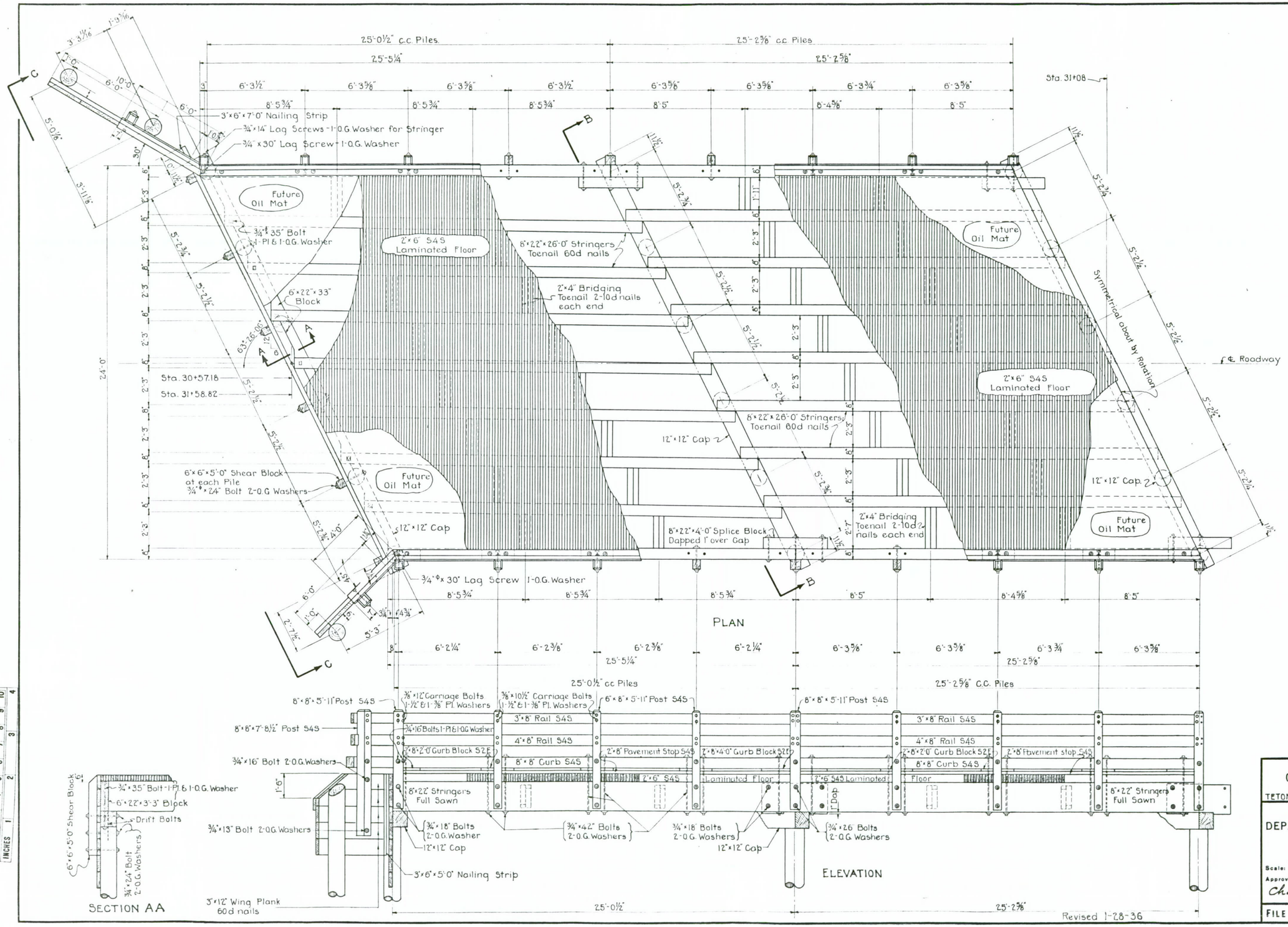
STATE OF IDAHO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF HIGHWAYS
BOISE-IDAHO

Scale: As Shown — Designed by EDC
Approved: Chas. A. Kyle Bridge Eng'r. — Checked by WLL — Date 1-10-36

FILE NO. 586 DRAWING NO. 2909

W.P.S.O.

Fed. Road Dist. No.	State	Fed. Aid Proj. No.	Total Sheets	Sheet No.
12	IDAHO	217A		



Revised 1-28-36

MICROFILM RECORD										
ROLL NO.	AP-CARD or FRAME NO.									
7	170									
9-2-22										
METRIC 2	3	4	5	6	7	8	9	10		4
INCHES	1								3	2

101' TIMBER BRIDGE OVER TETON RIVER

Sta. 31+08

TETON CITY - ST. ANTHONY HWY. FREMONT CO.

STATE OF IDAHO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF HIGHWAYS
BOISE-IDAHO

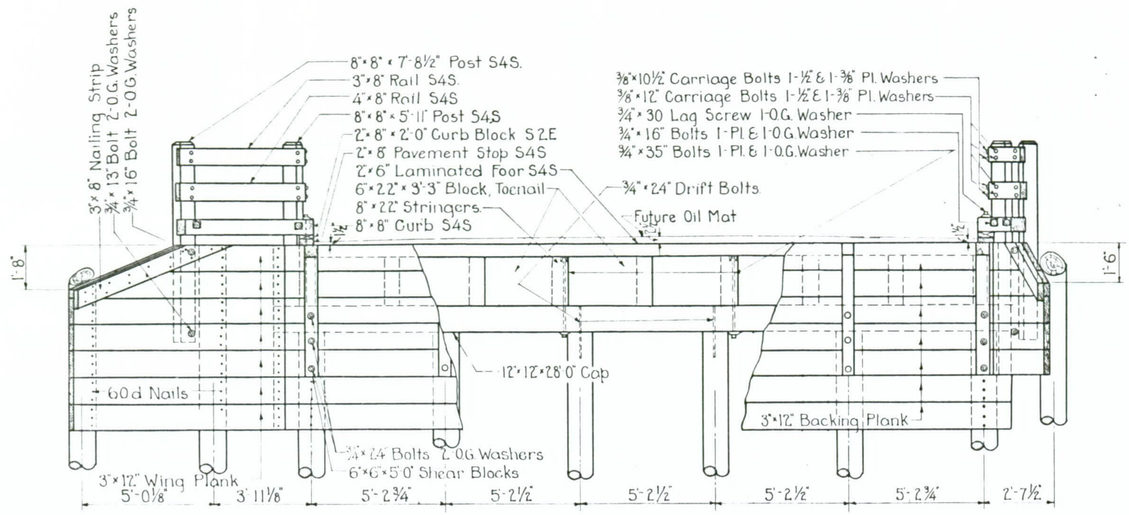
Scale: ⅜" = 1'-0" Designed by L.E.D.

Approved: Chas. A. Kyle Bridge Eng'r. Date 1-10-36 Checked by H.L.L.

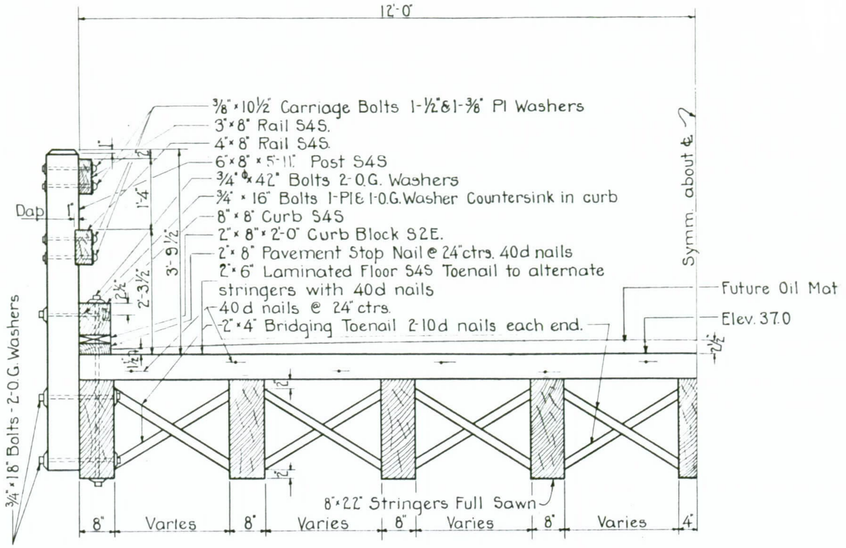
FILE NO. 586 DRAWING NO. 2910

W.P.S.O.

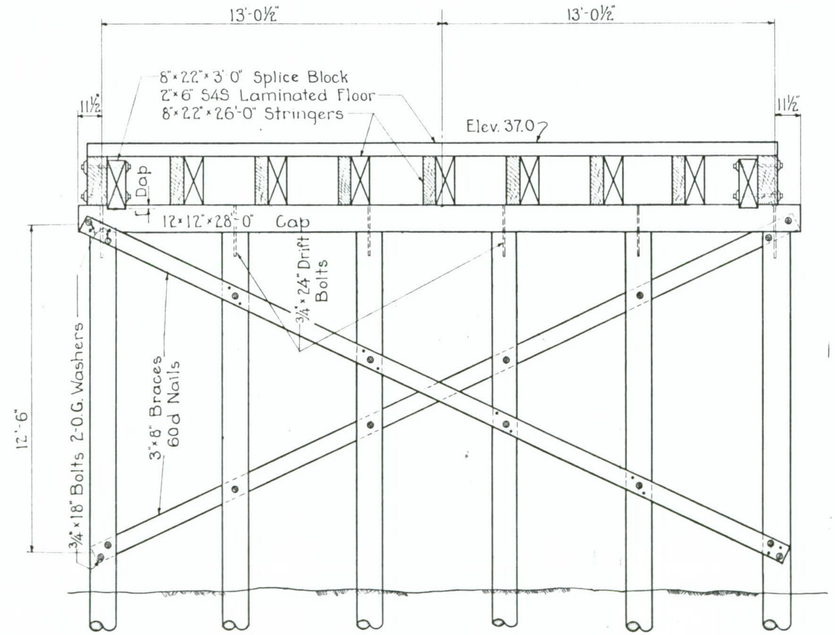
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	TOTAL SHEETS	SHEET NO.
12	IDAHO	217-A		



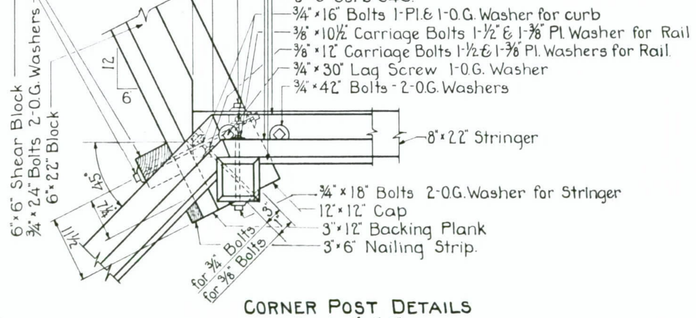
VIEW CC
Scale ⅜" = 1'-0"



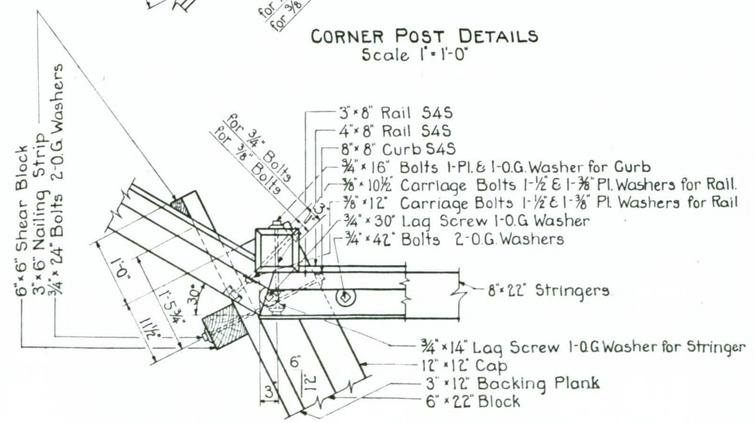
HALF ROADWAY SECTION
Scale ¾" = 1'-0"



VIEW BB
Curb and Rail Removed
Scale ⅜"-1'-0"



CORNER POST DETAILS
Scale 1" = 1'-0"



Revised 1-28-36

101' TIMBER BRIDGE
OVER TETON RIVER
Sta 31+08
TETON CITY - ST ANTHONY HWY. FREMONT CO.

STATE OF IDAHO
DEPARTMENT OF PUBLIC WORKS
BUREAU OF HIGHWAYS
BOISE-IDAHO

Scale: As Shown — Designed by EDC
Approved: Chas F Kyle Bridge Eng'r. — Checked by WLL — Date 1/20/36

FILE NO. 586 DRAWING NO. 2911

MICROFILM RECORD
DATE 9-12-77
ROLL NO. 7
AP-CARD or FRAME NO. 111
METRIC 2 3 4 5 6 7 8 9 10
INCHES 1 2 3 4

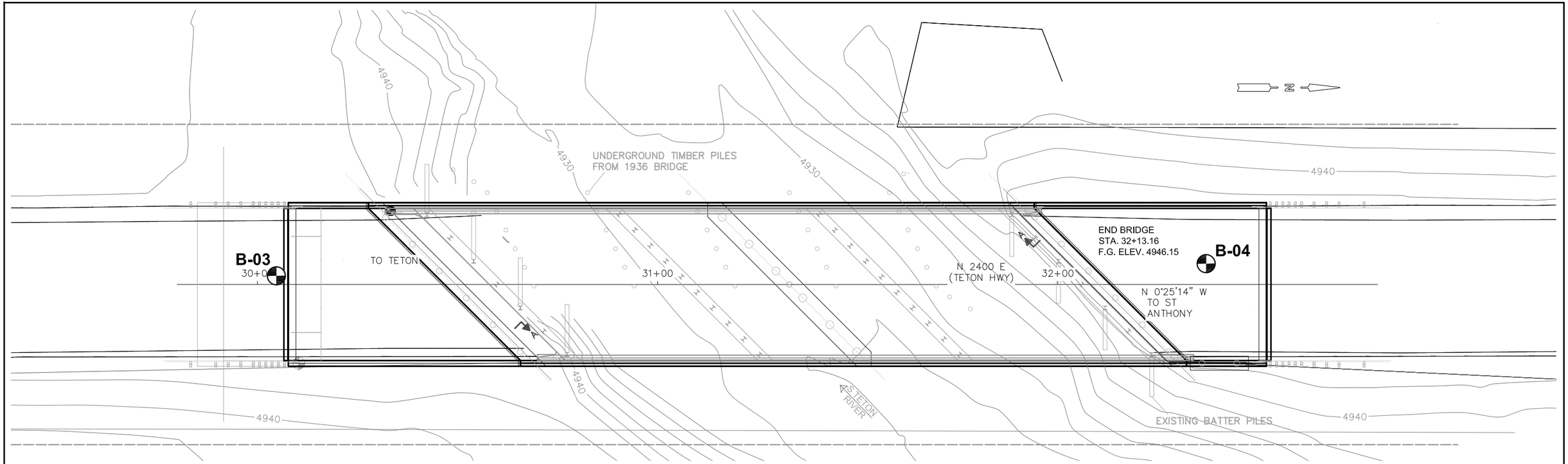
APPENDIX B – FOUNDATION INVESTIGATION PLAT

Appendix B

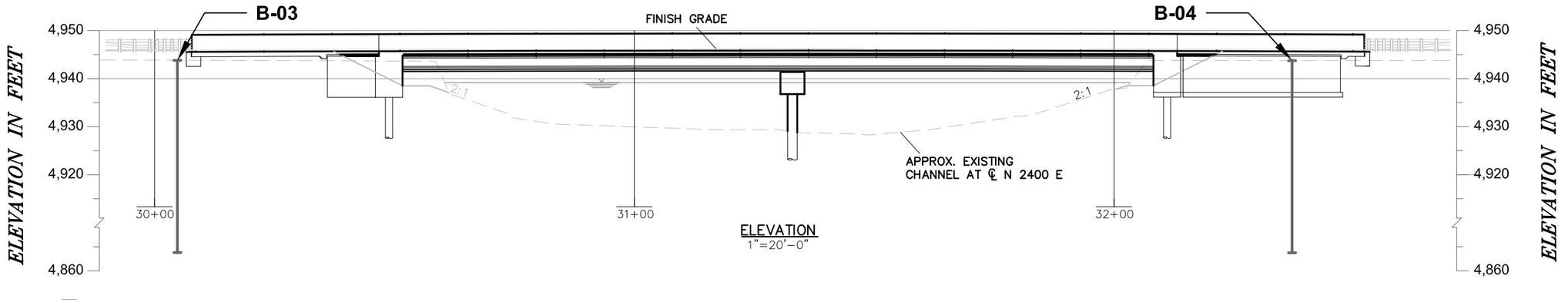
Foundation Investigation Plat

CONTENTS

- Foundation Investigation Plat



PLAN
1"=20'-0"



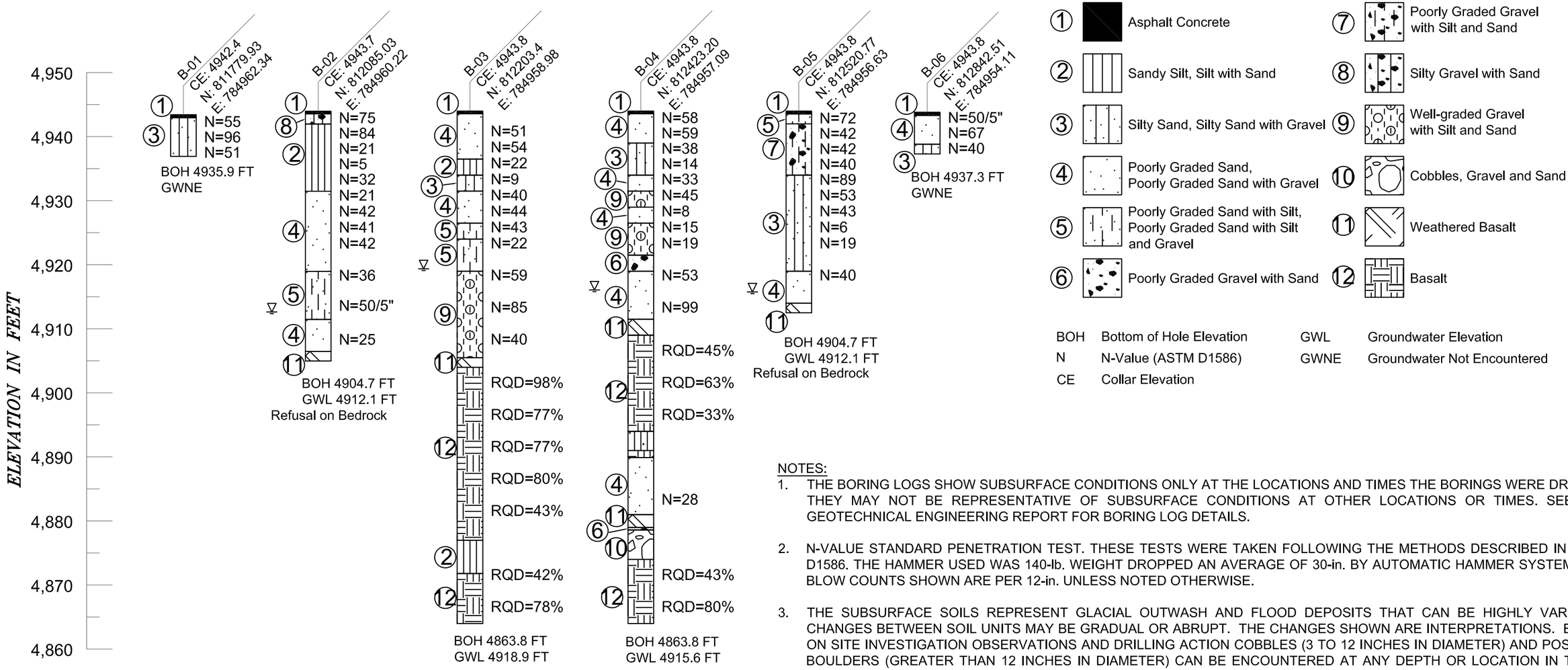
ELEVATION
1"=20'-0"

ORIGINAL SIGNED BY:

DATE SIGNED:

REVISIONS				DESIGNED	SCALES SHOWN ARE FOR 11" X 17" PRINTS ONLY	IDAHO TRANSPORTATION DEPARTMENT	ENGLISH	FOUNDATION INVESTIGATION PLAT (1 of 2)	BRIDGE PLANS	
NO.	DATE	BY	DESCRIPTION	DESIGN CHECKED	CADD FILE NAME	YOUR Safety→YOUR Mobility→YOUR Economic Opportunity	PROJECT NO.		BRIDGE KEY NO. 21035	
				DETAILED A. MADSEN	112645 FIP PREP 13-Mar-25.dwg	SHANNON & WILSON	A024(341)	SOUTH TETON RIVER BRIDGE FREMONT COUNTY, ID	COUNTY FREMONT	KEY NO. 24341
				DWG. CHECKED	DRAWING DATE: March 2025				BRIDGE DWG. NO.	SHEET 1 OF 2
				CORRECTIONS						

112645 - Fence PREP 9-Dec-24.dwg
12/10/2024 10:44 AM
ACM



LEGEND

1	Asphalt Concrete	7	Poorly Graded Gravel with Silt and Sand
2	Sandy Silt, Silt with Sand	8	Silty Gravel with Sand
3	Silty Sand, Silty Sand with Gravel	9	Well-graded Gravel with Silt and Sand
4	Poorly Graded Sand, Poorly Graded Sand with Gravel	10	Cobbles, Gravel and Sand
5	Poorly Graded Sand with Silt, Poorly Graded Sand with Silt and Gravel	11	Weathered Basalt
6	Poorly Graded Gravel with Sand	12	Basalt

BOH	Bottom of Hole Elevation	GWL	Groundwater Elevation
N	N-Value (ASTM D1586)	GWNE	Groundwater Not Encountered
CE	Collar Elevation		

NOTES:

1. THE BORING LOGS SHOW SUBSURFACE CONDITIONS ONLY AT THE LOCATIONS AND TIMES THE BORINGS WERE DRILLED. THEY MAY NOT BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS OR TIMES. SEE THE GEOTECHNICAL ENGINEERING REPORT FOR BORING LOG DETAILS.
2. N-VALUE STANDARD PENETRATION TEST. THESE TESTS WERE TAKEN FOLLOWING THE METHODS DESCRIBED IN ASTM D1586. THE HAMMER USED WAS 140-lb. WEIGHT DROPPED AN AVERAGE OF 30-in. BY AUTOMATIC HAMMER SYSTEM. THE BLOW COUNTS SHOWN ARE PER 12-in. UNLESS NOTED OTHERWISE.
3. THE SUBSURFACE SOILS REPRESENT GLACIAL OUTWASH AND FLOOD DEPOSITS THAT CAN BE HIGHLY VARIABLE. CHANGES BETWEEN SOIL UNITS MAY BE GRADUAL OR ABRUPT. THE CHANGES SHOWN ARE INTERPRETATIONS. BASED ON SITE INVESTIGATION OBSERVATIONS AND DRILLING ACTION COBBLES (3 TO 12 INCHES IN DIAMETER) AND POSSIBLY BOULDERS (GREATER THAN 12 INCHES IN DIAMETER) CAN BE ENCOUNTERED AT ANY DEPTH OR LOCATION IN THESE TYPES OF GLACIAL OUTWASH AND FLOOD DEPOSITS.
4. BEDROCK WAS MEDIUM STRONG (R3) TO VERY STRONG (R5) WITH COMPRESSIVE STRENGTHS FROM 4,702 PSI TO 18,586 PSI.
5. THE BORING LOGS, AND GEOTECHNICAL ENGINEERING REPORT FOR THE PROJECT ARE AVAILABLE AT THE LHTAC OFFICE.

ORIGINAL SIGNED BY:

DATE SIGNED:

REVISIONS				DESIGNED	SCALES SHOWN ARE FOR 11" X 17" PRINTS ONLY	IDAHO TRANSPORTATION DEPARTMENT	ENGLISH	FOUNDATION INVESTIGATION PLAT (2 of 2)	BRIDGE PLANS	
NO.	DATE	BY	DESCRIPTION	DESIGN CHECKED	CADD FILE NAME	YOUR Safety→YOUR Mobility→YOUR Economic Opportunity	PROJECT NO.		BRIDGE KEY NO. 21035	
				DETAILED A. MADSEN	112645 FIP PREP 13-Mar-25.dwg	SHANNON & WILSON	A024(341)	SOUTH TETON RIVER BRIDGE FREMONT COUNTY, ID	COUNTY FREMONT	KEY NO. 24341
				DWG. CHECKED	DRAWING DATE: March 2025				BRIDGE DWG. NO.	SHEET 2 OF 2
				CORRECTIONS						

112645 - Fence PREP 9-Dec-24.dwg

12/10/2024 10:44 AM

ACM

APPENDIX C – SUBSURFACE EXPLORATIONS

Appendix C

Subsurface Explorations

CONTENTS

- Exploration Location Plan
- Logs of Exploration



LEGEND

B-01 Boring Designation and Approximate Location

TP-01 SE Hand Dug Test Pit Designation and Approximate Location

50 200
0 100 300
SCALE IN FEET

N

South Fork Teton River Bridge
Fremont County, Idaho

EXPLORATION LOCATION MAP

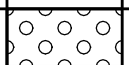
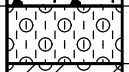
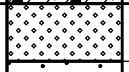
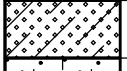
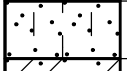
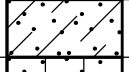
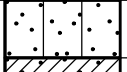
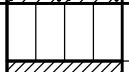
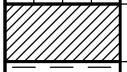
November 13, 2024 71-112645

SHANNON & WILSON **FIG. 2**

112645 - Exp Loc Map PREP 30 Sept 24.dwg
11/13/2024 2:10 PM
ACM

SHANNON & WILSON

ASTM D2488 Field Classification Legend

MAJOR DIVISION			GRAPHIC LOG	GROUP SYMBOL
COARSE GRAINED SOILS CONTAINS MORE THAN 50% FINES	**GRAVEL AND GRAVELLY SOILS** MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH ~ 5% FINES		GW
				GP
		GRAVEL WITH BETWEEN 5% AND 15% FINES		GW-GM
				GW-GC
				GP-GM
				GP-GC
		GRAVEL WITH ≥ 15% FINES		GM
				GC
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SAND WITH ~ 5% FINES		SW
				SP
		SAND WITH BETWEEN 5% AND 15% FINES		SW-SM
				SW-SC
				SP-SM
				SP-SC
		SAND WITH ≥ 15% FINES		SM
				SC
FINE GRAINED SOILS CONTAINS MORE THAN 50% FINES	SILT AND CLAY	LIQUID LIMIT LESS THAN 50		ML
				CL
				OL
		LIQUID LIMIT GREATER THAN 50		MH
				CH
				OH
HIGHLY ORGANIC SOILS				PT

STANDARD PENETRATION RESISTANCE				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	COMPRESSIVE STRENGTH (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.50
MEDIUM DENSE	10 - 30	FIRM	4 - 8	0.50 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

ABBREVIATIONS

AU	Auger cuttings
BK	Bulk sample
GS	Grab sample
NR	No Recovery
OSS	Oversized split-spoon sample with an outside diameter of 3 inches. Blow counts are not corrected to N-values unless specifically noted on the logs.
PT	Piston tube sample with an inside diameter of 2.88 inches, unless otherwise noted on the logs (reference ASTM D 1587)
CR	Core barrel sample (reference ASTM D 2113)
SS	Standard split-spoon sample (reference ASTM D 1586), driven by a hydraulic hammer in pavements
ST	Thin-walled (Shelby) tube sample with an inside diameter of 2.88 inches, unless otherwise noted on the logs (reference ASTM D 1587)
OST	Thin-walled (Shelby) tube sample with an inside diameter of 3.88 inches, used for resilient modulus testing.
VS	Vane shear test
PP	Pocket penetrometer
TV	Torvane
MC	Moisture content (%)
LL	Liquid limit (%)
PL	Plastic limit (%)
PI	Plasticity index
▽	Water level reading at time of drilling
▼	Water level reading __ hrs after drilling

NOTES:

1) Sample descriptions are based on visual field and laboratory observations using classification methods of ASTM D2488. Where laboratory data are available, classifications are in accordance with ASTM D2487.

2) Solid lines between soil descriptions indicate change in interpreted geologic unit. Dashed lines indicate an assumed stratigraphic change within the unit.

3) Fines are material passing the U.S. Std. #200 Sieve.



Rock Core Photo 1: B-03, CR-13 to 14; 40.0'-50.0'



- Rock Core Photo 2: B-03, CR-15 to 16; 50.0'-59.7'



Rock Core Photo 3: B-03, CR-16 to 18; 59.7'-70.0'



Rock Core Photo 4: B-03, CR-19 to 20; 70.0'-80.0'



Rock Core Photo 5: B-04, CR-36 to 37; 35.0'-45.0'



Rock Core Photo 6: B-04, CR-38 to 40; 45.0'-59.8'



Rock Core Photo 7: B-04, CR-42 to 46; 61.5'-75.6'



Rock Core Photo 8: B-04, CR-46; 75.6'-80.0'

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

METHOD: Hollow Stem Auger
DATE LOGGED: 9/17/2024
LOGGED BY: Austin Madsen, EI, GIT

COLLAR ELEVATION: 4942.4 ft
NORTHING: 811779.93 by Surveyors
EASTING: 784962.34

SHANNON & WILSON

BORING NO. B-01

GROUNDWATER:

Groundwater not encountered on 9/17/2024

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY %	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
						Asphalt Concrete - AC thickness = 5.5"				Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings.
		SS-21	89	12-16-39		Silty Sand with Gravel (SM) - (FILL) 50% fine to coarse, subangular to subrounded sand; 36% fine to coarse, hard, angular to subangular gravel to 1-1/2"; 14% nonplastic fines; moist; brown.				
4940		SS-22	78	17-48-48					5	
	5	SS-23	89	19-21-30						Boring located approximately 3' east of fogline and 450' south of bridge.

Bottom of Boring at 6.5 ft on 9/17/2024.

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

METHOD: Hollow-Stem Auger/ HQ Diamond Core
DATE LOGGED: 9/19/2024
LOGGED BY: Austin Madsen, EI, GIT

BORING NO. B-02

GROUNDWATER:

▽ **AT TIME OF DRILLING 31.6 ft. bgs. 10:30:00 AM.**

COLLAR ELEVATION: 4943.7 ft
NORTHING: 812085.03 by Surveyors
EASTING: 784960.22

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY %	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
						Asphalt Concrete - AC thickness = 5"				Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings.
		SS-50	89	17-31-44		Silty Gravel with Sand (GM) - (FILL) 48% fine to coarse, hard, subangular to rounded gravel to 2"; 37% fine to coarse, subangular sand; 15% nonplastic fines; moist; brown.				
4940		SS-51	78	23-40-44						
	5					Sandy Silt (ML) - (NATIVE) 55% fines; 41% fine to coarse, subangular to subrounded sand; 4% fine to coarse, hard, subrounded to rounded gravel to 3/4"; brown; weakly cemented; strong HCl reaction.				Boring located approximately 7' east of fogline and 150' south of bridge. Bulk sample from 2' to 4'.
		SS-52	61	8-14-7						
4935		SS-53	56	4-2-3			NV	NP	14	
	10	SS-54	61	4-14-18						
4930		SS-55	39	5-9-12		Poorly Graded Sand with Gravel (SP) - About 80% fine to coarse, subangular to angular sand; about 15% fine to coarse, hard, subangular to subrounded gravel to 3" (based on cuttings); about 5% nonplastic fines; moist; brown; strong HCl reaction on sand.				
	15	SS-56	56	24-24-18						
4925		SS-57	67	8-18-23						
	20	SS-58	50	16-20-22						
4920										
	25	SS-59	72	13-18-18		Poorly Graded Sand with Silt and Gravel (SP-SM) - About 60% fine to coarse, subangular to subrounded sand; about 30% fine to coarse, hard, angular to subrounded gravel to 3" (based on cuttings); about 10% nonplastic fines; moist; brown with white streaks in fines; strong HCl reaction on white streaks.				
4915										
	30	SS-60	82	29-50/5"	▽					
4910						Poorly Graded Sand (SP) - About 95% fine to coarse, subangular to subrounded sand; about 5% fine, hard, subrounded to rounded gravel; trace nonplastic fines; wet; brown.				Driller indicated very smooth drilling from 32.5' to 37.5'.

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

GROUNDWATER:

AT TIME OF DRILLING 31.6 ft. bgs. 10:30:00 AM.

BORING NO. B-02

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY %	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
		SS-61	100	2-6-19						
4905						Weathered Basalt -				Hard grinding from 37.5' to 39'.
						Bottom of Boring at 39.0 ft on 9/19/2024.				Auger refusal at 39' on bedrock.

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

BORING NO. B-03

METHOD: Hollow-Stem Auger/ HQ Diamond Core
DATE LOGGED: 9/17/2024
LOGGED BY: Austin Madsen, EI, GIT

GROUNDWATER:

▽ **AT TIME OF DRILLING 24.9 ft. bgs. 10:00:00 AM.**

COLLAR ELEVATION: 4943.8 ft
NORTHING: 812203.42 by Surveyors
EASTING: 784958.98

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
						Asphalt Concrete - AC thickness = 5"				Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings. Boring located approximately 5' east of fogline and 15' south of bridge.
						Poorly Graded Sand with Gravel (SP) - (FILL) About 50% fine to coarse, subrounded sand; about 45% fine to coarse, hard, angular to subangular gravel to 1-1/2"; about 5% nonplastic fines; moist; brown.				
4940		SS-01	72	22-22-29						
	5									
		SS-02	78	13-26-28						
		SS-03	72	3-11-11		Silt with Sand (ML) - (NATIVE) About 80% nonplastic to low plasticity fines; about 15% fine to medium sand; about 5% fine, hard, subrounded gravel to 3/8"; moist; brown.				
4935										
	10	SS-04	100	1-2-7		Silty Sand (SM) - 50% fines; 40% fine to medium sand; 10% fine, hard, subrounded gravel to 2" based on cuttings; brown.	NV	NP	19	
		SS-05	72	5-19-21		Poorly Graded Sand with Gravel (SP) - About 70% fine to coarse, subangular to subrounded sand; about 25% fine, hard, subrounded to rounded gravel to 2" based on cuttings; moist; brown.				
4930										
	15	SS-06	61	6-23-21		Poorly Graded Sand with Gravel (SP) - About 55% fine to coarse, subangular to subrounded sand; about 40% fine to coarse, hard, angular to subrounded gravel to 2" (based on cuttings); about 5% nonplastic fines; moist; brown.				
		SS-07	67	10-19-24		Poorly Graded Sand with Silt and Gravel (SP-SM) - About 50% fine to coarse, subangular sand; about 40% fine to coarse, hard, subangular to rounded gravel to 2" (based on cuttings); about 10% nonplastic fines; moist; brown.			10	
4925										
	20	SS-08	72	4-10-12		Poorly Graded Sand with Silt (SP-SM) - 86% fine to medium sand; 8% nonplastic fines; 6% fine, hard, subrounded to rounded gravel to 3" (based on cuttings); brown.				
4920						▽				
	25	SS-09	78	10-25-34		Well-Graded Gravel with Silt and Sand (GW-GM) - 51% fine, hard, angular to subrounded gravel to 3" (based on cuttings); 40% fine to coarse, subangular to subrounded sand; 9% nonplastic fines; brown.				
4915										
	30	SS-10	61	12-35-50					12	
4910										

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

GROUNDWATER:

AT TIME OF DRILLING 24.9 ft. bgs. 10:00:00 AM.

BORING NO. B-03

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
		SS-11	50	11-22-18						
4905						Weathered Basalt -				
	40	CR-13	100 (98)			Basalt - Gray; fine grained; moderately vesicular; smooth-stepped; moderate spacing; single horizontal fracture with a single near vertical fracture; fresh.				Compressive Strength = 4,702 psi
4900										
	45	CR-14	100 (77)			Gray; fine grained; slightly vesicular; smooth-stepped; close to moderate spacing; 0 to 40 degree fractures; fresh.				Compressive Strength = 13,597 psi
4895										
	50	CR-15	100 (77)			Gray; fine grained; slightly vesicular; smooth-stepped; close to moderate spacing; near horizontal fractures; fresh.				Compressive Strength = 14,847 psi
4890										
	55	CR-16	100 (80)			Gray shifting to red at 59'; fine grained; slightly vesicular; smooth-stepped; close to moderate spacing; near 45 degree fractures until 59' then highly fractured red basalt; fresh.				Compressive Strength = 18,586 psi
4885										
	60	CR-17	100 (43)			Reddish gray; fine grained; moderately to highly vesicular; very close spacing; top 2' near horizontal fractures with a zone of rubble, bottom 3' near vertical and 45 degrees angle fractures; white silty infilling in fractures (no HCl reaction); fresh to slightly weathered.				
4880										
	65	CR-18	67 (15)			Gray; fine grained; moderately vesicular; very close spacing; low angle fractures with spots of rubble; silty infilling in fractures and voids; fresh.				
4875						Sandy Silt (ML) - About 60% nonplastic fines; about 40% fine to medium sand: wet; red-brown.				
	70	CR-19	77			Basalt -				

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

GROUNDWATER:

AT TIME OF DRILLING 24.9 ft. bgs. 10:00:00 AM.

BORING NO. B-03

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
4870			(42)			Gray; fine grained; highly vesicular; smooth-stepped; very close to close spacing; 0 to 20 degree fractures with silty infilling; fresh.				Top 14" of CR-19 potentially washed out due to drilling fluid.
	75									
		CR-20	100 (78)			Gray; fine grained; highly vesicular; smooth-stepped; close to moderate spacing; near horizontal fractures; fresh.				
4865										
	80									

Bottom of Boring at 80.0 ft on 9/17/2024.

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

METHOD: Hollow-Stem Auger/ HQ Diamond Core
DATE LOGGED: 9/18/2024
LOGGED BY: Austin Madsen, EI, GIT

BORING NO. B-04

GROUNDWATER:

▽ **AT TIME OF DRILLING 28.2 ft. bgs. 9:00:00 AM.**

COLLAR ELEVATION: 4943.8 ft
NORTHING: 812423.20 by Surveyors
EASTING: 784957.09

ELEV. (ft)	DEPTH (ft)	TYPE - No.	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
							Asphalt Concrete - AC thickness = 5"				Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings.
			SS-24	67	10-27-31		Poorly Graded Sand with Gravel (SP) - (FILL) About 75% fine to coarse, subangular sand; about 20% fine to coarse, hard, angular to rounded gravel to 3/4"; about 5% nonplastic fines; moist; brown.				
4940			SS-25	72	8-24-35						
	5		SS-26	89	14-20-18		Silty Sand with Gravel (SM) - (NATIVE) 49% fine to coarse, subangular sand; 35% fine to coarse, hard, subangular to subrounded gravel to 1"; 16% nonplastic fines; brown.			7	Boring located approximately 3' east of fogline and 30' north of bridge.
4935			SS-27	0	15-9-5						
	10		SS-28	67	15-17-16		Poorly Graded Sand with Gravel (SP) - About 65% fine to coarse, angular to subangular sand; about 30% fine to coarse, hard, angular to subrounded gravel to 1"; about 5% nonplastic fines; moist; brown.				
			SS-29	72	17-25-20		Well-Graded Gravel with Silt and Sand (GW-GM) - 57% fine to coarse, hard, subrounded gravel to 1-1/2"; 35% fine to coarse, angular to subangular sand; 8% nonplastic fines; brown.			4	
4930	15		SS-30	50	5-4-4						
			SS-31	56	5-7-8		Poorly Graded Sand (SP) - About 85% fine to coarse, subangular to subrounded sand; about 10% fine, hard, subangular to subrounded gravel to 1/2"; about 5% nonplastic fines; moist; brown.			7	
4925	20		SS-32	56	5-10-9		Well-Graded Gravel with Silt and Sand (GW-GM) - 58% fine to coarse, hard, subangular to subrounded gravel to 3" (based on cuttings); 34% fine to coarse, subangular sand; 8% nonplastic fines; moist; brown.				
4920							Poorly Graded Gravel with Sand (GP) - About 65% fine to coarse, hard, angular to subrounded gravel to 3" (based on cuttings); about 30% medium to coarse, angular to subangular sand; about 5% nonplastic fines; moist; brown.				
	25		SS-33	39	8-24-29		▽ Poorly Graded Sand with Gravel (SP) - About 80% fine to coarse, subangular sand; about 15% fine, hard, subangular to subrounded gravel to 3/8"; about 5% nonplastic fines; wet; brown.				
4915											
	30		SS-34	67	20-49-50						
4910							Weathered Basalt -				Hard auger grinding starting at 32.5'

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

GROUNDWATER:

▽ **AT TIME OF DRILLING 28.2 ft. bgs. 9:00:00 AM.**

BORING NO. B-04

ELEV. (ft)	DEPTH (ft)	TYPE - No.	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
4905	40		CR-36	93 (45)			Basalt - Dark gray; fine grained; moderately vesicular; rough-stepped; close to moderate spacing; near horizontal fractures with zones of rubble; red staining in joints; slightly weathered.				Splitspoon and Auger refusal at 35', switched to rock coring. Compressive Strength = 13,117 psi
4900	45		CR-37	100 (63)			Dark reddish gray; fine grained; moderately vesicular; smooth-stepped; near horizontal fractures with near vertical fractures with silty infilling; slightly weathered.				Compressive Strength = 7,453 psi
4895	50		CR-38	73 (33)			Dark gray with reddish voids; fine grained; moderately vesicular from 45' to 48', slightly vesicular from 48' to 50'; low angle fractures with two 70 degree fractures; slightly weathered.				Mislatch on coring intertube, pieces of rock core lost. Driller indicated coring through fines from 47.0' to 47.7', potential silty layer.
4890	55		CR-39	37 (8)			Silty Sand (SM - washed out)				Top 3' of CR-39 washed out, driller indicated coring through a soil layer for this length.
4885	60		CR-40	100 (0)			Basalt - Dark gray; fine grained; moderately vesicular; rough-stepped; very close to close spacing; near horizontal fractures with silty infilling in fractures and vesicules; fresh to slightly weathered. Poorly Graded Sand (SP) - About 100% fine to medium sand; brown to red-brown; wet; weak to strong HCl reactions.				
			SS-41	100	3-8-20						
4880	65		CR-42	95 (0)			Basalt rubble transitioning to rounded gravel, then to sand.				
4875	70	SS-43	CR-44	60 (0)			Poorly Graded Gravel with Sand (GP) - About 80% hard, fine, angular to subangular gravel to 1/2"; about 15% fine to coarse, angular to subrounded sand; about 5% nonplastic fines; wet; black. Washed out cobbles, gravel, and sand. Basalt cobbles have voids with silty infilling. Subrounded to rounded gravel and sand, appears to be a stream deposit.				
			CR-45	70			Basalt - Dark gray; fine grained; highly vesicular; rough-stepped; close spacing; near horizontal fractures with silty and sandy infilling in voids and				

SHANNON & WILSON

GROUNDWATER:

▽ AT TIME OF DRILLING 28.2 ft. bgs. 9:00:00 AM.

BORING NO. B-04

ELEV. (ft)	DEPTH (ft)	TYPE - No.	TYPE - No.	RECOVERY % (RQD)	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
				(43)			joints; fresh to slightly weathered.				
4870	75						Similar to above.				
			CR-46	93 (80)							
4865	80						Bottom of Boring at 80.0 ft on 9/18/2024.				

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

SHANNON & WILSON

BORING NO. B-05

METHOD: Hollow-Stem Auger/ HQ Diamond Core
DATE LOGGED: 9/19/2024
LOGGED BY: Austin Madsen, EI, GIT

GROUNDWATER:

▽ **AT TIME OF DRILLING 28.5 ft. bgs. 11:30:00 AM.**

COLLAR ELEVATION: 4943.8 ft
NORTHING: 812520.77 by Surveyors
EASTING: 784956.63

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY %	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
						Asphalt Concrete - AC thickness = 5.5"			4	Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings.
		GB-63 SS-62	72	29-30-42		Poorly Graded Sand with Silt and Gravel (SP-SM) - (FILL) 50% fine to coarse, subangular to subrounded sand; 43% fine to coarse, hard, subrounded gravel to 1"; 7% nonplastic fines; brown.			3	
		GB-64								
4940		SS-65	83	11-17-25						Boring located approximately 4' east of fogline and 130' north of bridge. Bulk sample from 2' to 4'.
	5	SS-66	44	17-16-26		Poorly Graded Gravel with Silt and Sand (GP-GM) - 53% fine to coarse, hard, subangular to rounded gravel to 2"; 37% fine to coarse, subangular to subrounded sand; 10% nonplastic fines; brown; weak to strong HCl reactions.				
4935		SS-67	56	12-19-21						
	10	SS-68	89	30-50-39		Silty Sand with Gravel (SM) - (NATIVE) 58% fine to coarse, subangular to subrounded sand; 26% fine to coarse, hard, subangular to subrounded gravel to 1"; 16% nonplastic fines; moist; brown; strong HCl reaction.			5	
4930		SS-69	67	12-23-30						
	15	SS-70	67	21-21-22						
		SS-71	33	3-2-4		Silty Sand with Gravel (SM) - 49% fine to coarse, subangular to subrounded sand; 30% fine to coarse, hard, subangular gravel to 3" (based on cuttings); 21% nonplastic fines; brown.			15	
4925										
	20	SS-72	56	1-3-16						
4920										
	25	SS-73	44	11-19-21		Poorly Graded Sand with Gravel (SP) - About 80% fine to coarse, subangular sand; about 15% fine, hard, angular to subangular gravel to 3" (based on cuttings); wet; brown; weak HCl reaction.				
4915					▽					
	30	SS-74	100	50		Weathered Basalt -				Hard auger grinding starting at 30'.
						Bottom of Boring at 31.5 ft on 9/19/2024.				Auger refusal on bedrock at 31.5'.

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

METHOD: Hollow Stem Auger
DATE LOGGED: 9/18/2024
LOGGED BY: Austin Madsen, EI, GIT

COLLAR ELEVATION: 4943.8 ft
NORTHING: 812842.51 by Surveyors
EASTING: 784954.11

SHANNON & WILSON

BORING NO. B-06

GROUNDWATER:

Groundwater not encountered on 9/18/2024

ELEV. (ft)	DEPTH (ft)	TYPE - No.	RECOVERY %	BLOW COUNTS	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	LAB DATA PI (%)	LAB DATA MC (%)	REMARKS
						Asphalt Concrete - AC thickness = 6"				Driller: Haz-Tech Drill Rig: CME 75 Drilling Fluid: Water Backfill: Bentonite chips and cuttings. Boring located approximately 3' east of fogline and 450' north of bridge.
		SS-47	73	25-50/5"		Poorly Graded Sand with Gravel (SP) - (FILL) About 60% fine to coarse, subangular to subrounded sand; about 35% fine to coarse, hard, subangular gravel to 1"; about 5% nonplastic fines; moist; brown.				
4940		SS-48	67	19-20-47						
	5	SS-49	94	4-26-14		Silty Sand with Gravel (SM) - (NATIVE) 44% fine to coarse, subangular to subrounded sand; 34% fines; 22% fine to coarse, hard, subrounded gravel to 1"; brown.	NV	NP	10	
						Bottom of Boring at 6.5 ft on 9/18/2024.				

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

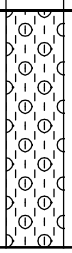
METHOD: Shovel
DATE LOGGED: 9/18/2024
LOGGED BY: Austin Madsen, EI, GIT

SHANNON & WILSON

TEST PIT NO. TP-01 SE

GROUNDWATER:

Groundwater not encountered on 9/18/2024

DEPTH (ft)	TYPE - No.	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	PI (%)	MC (%)
0			Well-Graded Gravel with Silt and Sand (GW-GM) - 64% coarse, hard, rounded gravel to 3"; 28% fine to coarse, subangular to subrounded sand; 8% fines; dark brown.	NV	NP	4
1			Bottom of Test Pit at 1.0 ft on 9/18/2024.			

PROJECT: Teton River Bridge
LOCATION: Teton, Idaho

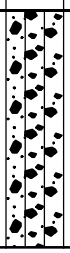
METHOD: Shovel
DATE LOGGED: 9/18/2024
LOGGED BY: Austin Madsen, EI, GIT

SHANNON & WILSON

TEST PIT NO. TP-02 NW

GROUNDWATER:

Groundwater not encountered on 9/18/2024

DEPTH (ft)	TYPE - No.	LITHOLOGY	MATERIAL DESCRIPTION (Stratification lines represent approximate boundaries between materials)	LAB DATA LL (%)	PI (%)	MC (%)
			Silty Gravel with Sand (GM) - 59% fine to coarse, hard, subrounded to rounded gravel to 2-1/2"; 24% fine to coarse, subangular to subrounded sand; 17% fines; dark brown.	NV	NP	9
1			Bottom of Test Pit at 1.0 ft on 9/18/2024.			

APPENDIX D – LABORATORY TESTING

Appendix D

Laboratory Testing

CONTENTS

- Laboratory Test Data

SHANNON & WILSON

Project Information

Report to: Forsgren
Project: Teton River Bridge
Report Date: 10/23/2024
File No.: 112645

Material Information

Date Sampled: 9/22/2024
Sampled By: Shannon & Wilson
Date Received: 9/28/2024
Date Tested: 10/1 - 10/23/2024

SUMMARY OF LABORATORY RESULTS

Lab Number	Borehole	Sample Type	Depth (ft)	Water Content (%)	% Passing #200 Sieve	Liquid Limit (%)	Plasticity Index	Soil Type	Remarks
24-1113	B-01	SS-22/SS-23	2.5'-6.5'	5.2	14.2	--	--	SM	--
24-1103	B-02	Bulk	2.0'-4.0'	--	15.4	--	--	GM	--
24-1114	B-02	SS-53	7.5'-9.0'	14.3	55.4	NV	NP	ML	--
24-1115	B-03	SS-04	10.0'-11.5'	19.0	49.9	NV	NP	SM	--
24-1116	B-03	SS-08	20.0'-21.5'	9.6	8.2	--	--	SP-SM	--
24-1117	B-03	SS-10	30.0'-31.5'	12.2	8.9	--	--	GW-GM	--
24-1118	B-04	SS-26	5.0'-6.5'	6.6	15.9	--	--	SM	--
24-1119	B-04	SS-29	12.5'-14.0'	3.8	7.8	--	--	GW-GM	--
24-1120	B-04	SS-31/32	17.5'-21.5'	6.5	8.0	--	--	GW-GM	--
24-1100	B-05	GB-63	0.5'-1.5'	3.7	6.9	--	--	SP-SM	--
24-1101	B-05	GB-64	1.5'-2.5'	3.4	4.8	--	--	GW	--
24-1104	B-05	Bulk	2.0'-4.0'	--	9.8	--	--	GP-GM	--
24-1121	B-05	SS-68	10.0'-11.5'	4.7	15.5	--	--	SM	--
24-1122	B-05	SS-71/72	17.5'-21.5'	15.4	21.0	--	--	SM	--
24-1123	B-06	SS-49	5.0'-6.5'	10.2	34.3	NV	NP	SM	--
24-1105	TP-01 SE	Bulk	0.0'-0.0'	3.6	7.9	NV	NP	GW-GM	--
24-1106	TP-02 NW	Bulk	0.0'-0.0'	8.9	16.7	NV	NP	GM	--

Prepared By: Roxanne Boucher

QC:TT

SHANNON & WILSON

Project Information

Report to: Forsgren
Project: Teton River Bridge
Report Date: 10/23/2024
File No.: 112645

Material Information

Date Sampled: 9/22/2024
Sampled By: Shannon & Wilson
Date Received: 9/28/2024
Date Tested: 10/1 - 10/23/2024

Sand Equivalent Test
AASHTO T-176

Lab Number	Location	Sample Type	Depth (ft)	Sand Equivalent	Remarks
24-1103	B-02	Bulk	2.0'-4.0'	25	--
24-1100	B-05	GB-63	0.5'-1.5'	38	--
24-1101	B-05	GB-64	1.5'-2.5'	38	--
24-1104	B-05	Bulk	2.0'-4.0'	33	--

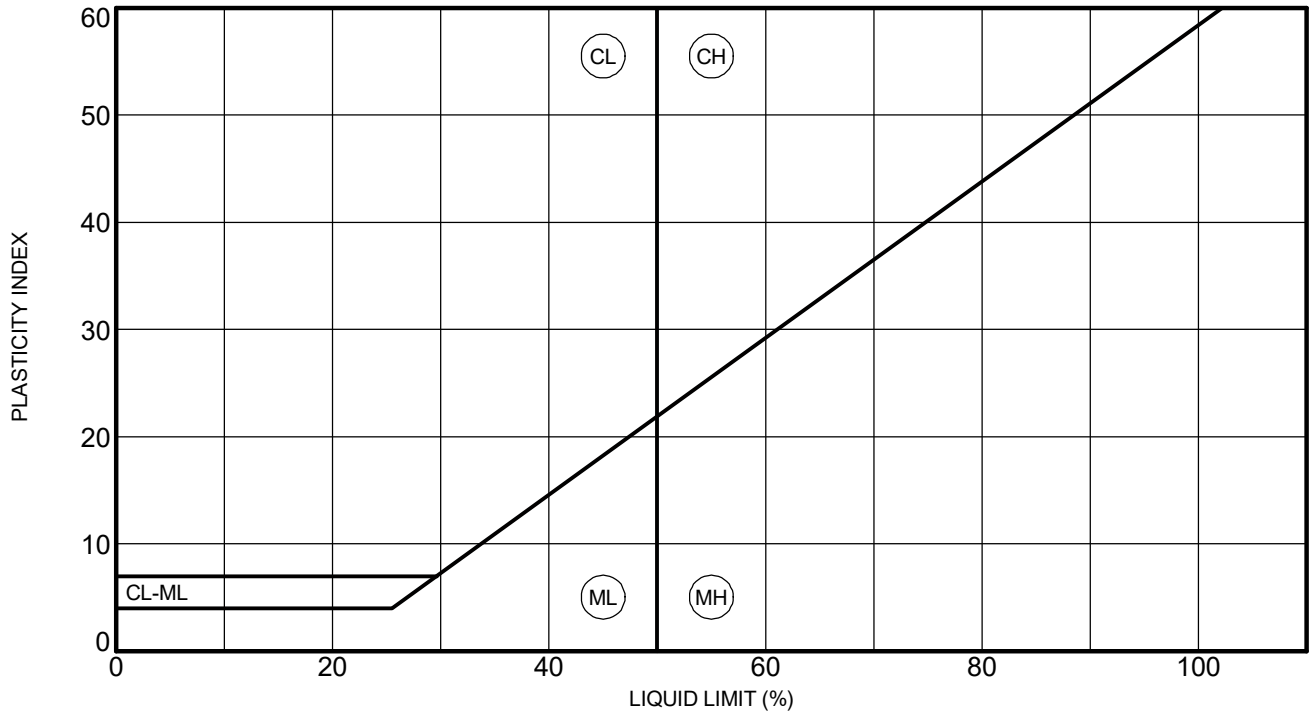
Prepared By: Roxanne Boucher

QC:TT

ATTERBERG LIMITS REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



Specimen Identification			MC	LL	PL	PI	Fines	Classification
B-02	SS-53	7.5	14.3	NV	NV	NP	55	SANDY SILT (ML)
B-03	SS-04	10.0	19.0	NV	NV	NP	50	SILTY SAND (SM)
B-06	SS-49	5.0	10.2	NV	NV	NP	34	SILTY SAND with GRAVEL (SM)
TP-01 SE	Bulk	0.0	3.6	NV	NV	NP	8	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)
TP-02 NW	Bulk	0.0	8.9	NV	NV	NP	17	SILTY GRAVEL with SAND (GM)
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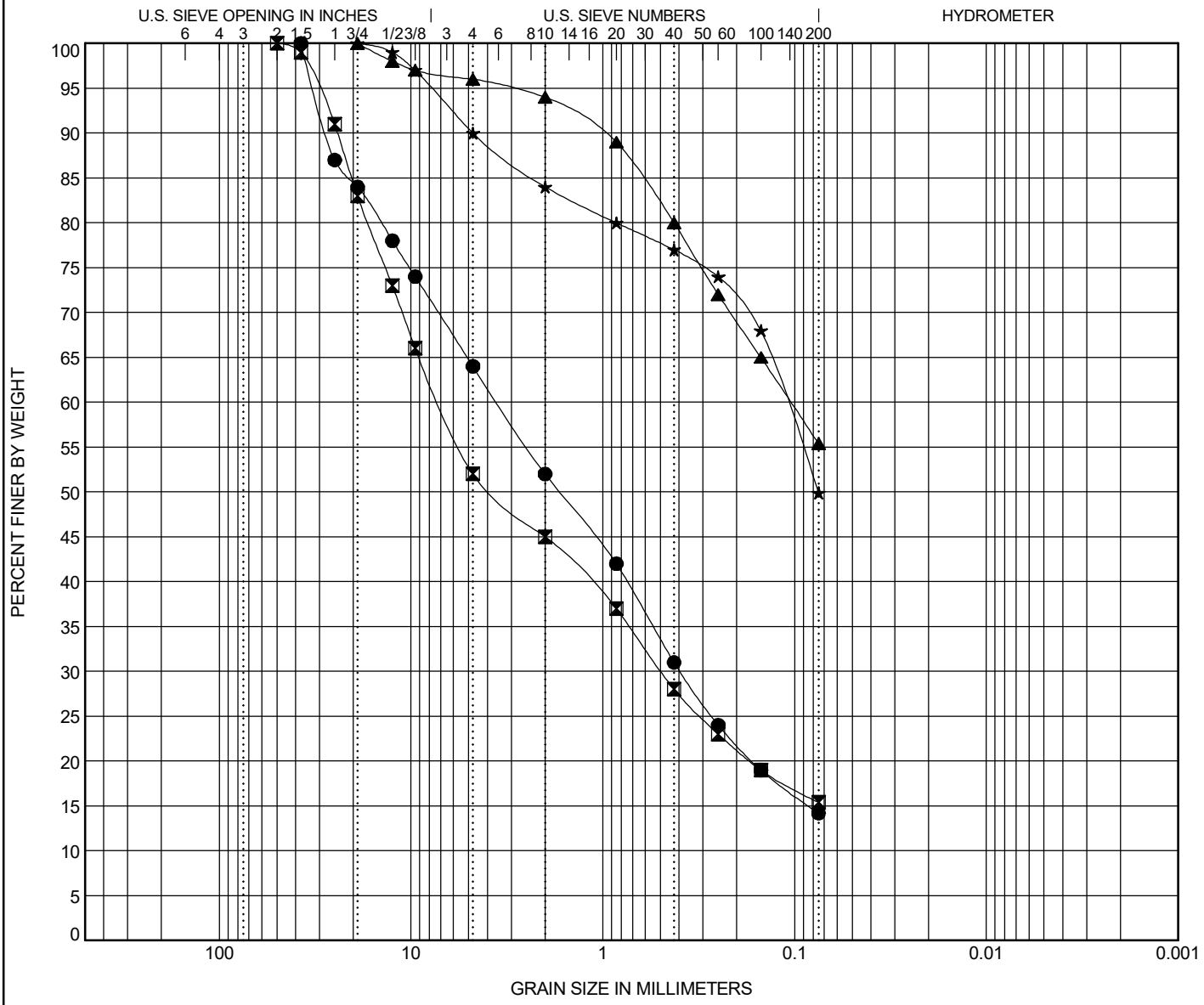
QC:TT

Prepared By: Roxanne Boucher

PARTICLE-SIZE DISTRIBUTION REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification				Classification	D10	D30	D60
●	B-01	SS-22/SS-23	2.5	SILTY SAND with GRAVEL (SM)	--	0.394	3.56
⊠	B-02	Bulk	2.0	SILTY GRAVEL with SAND (GM)	--	0.496	7.058
▲	B-02	SS-53	7.5	SANDY SILT (ML)	--	--	0.105
★	B-03	SS-04	10.0	SILTY SAND (SM)	--	--	0.11

Specimen Identification				%Gravel	%Sand	%Fines	D15	D50	D85	Cc	Cu	MC	LL	PI
●	B-01	SS-22/SS-23	2.5	36.0	49.8	14.2	0.084	1.685	20.82	--	--	5.2	--	--
⊠	B-02	Bulk	2.0	48.0	36.6	15.4	--	3.71	20.349	--	--	--	--	--
▲	B-02	SS-53	7.5	4.0	40.6	55.4	--	--	0.625	--	--	14.3	NV	NP
★	B-03	SS-04	10.0	10.0	40.1	49.9	--	0.075	2.31	--	--	19.0	NV	NP

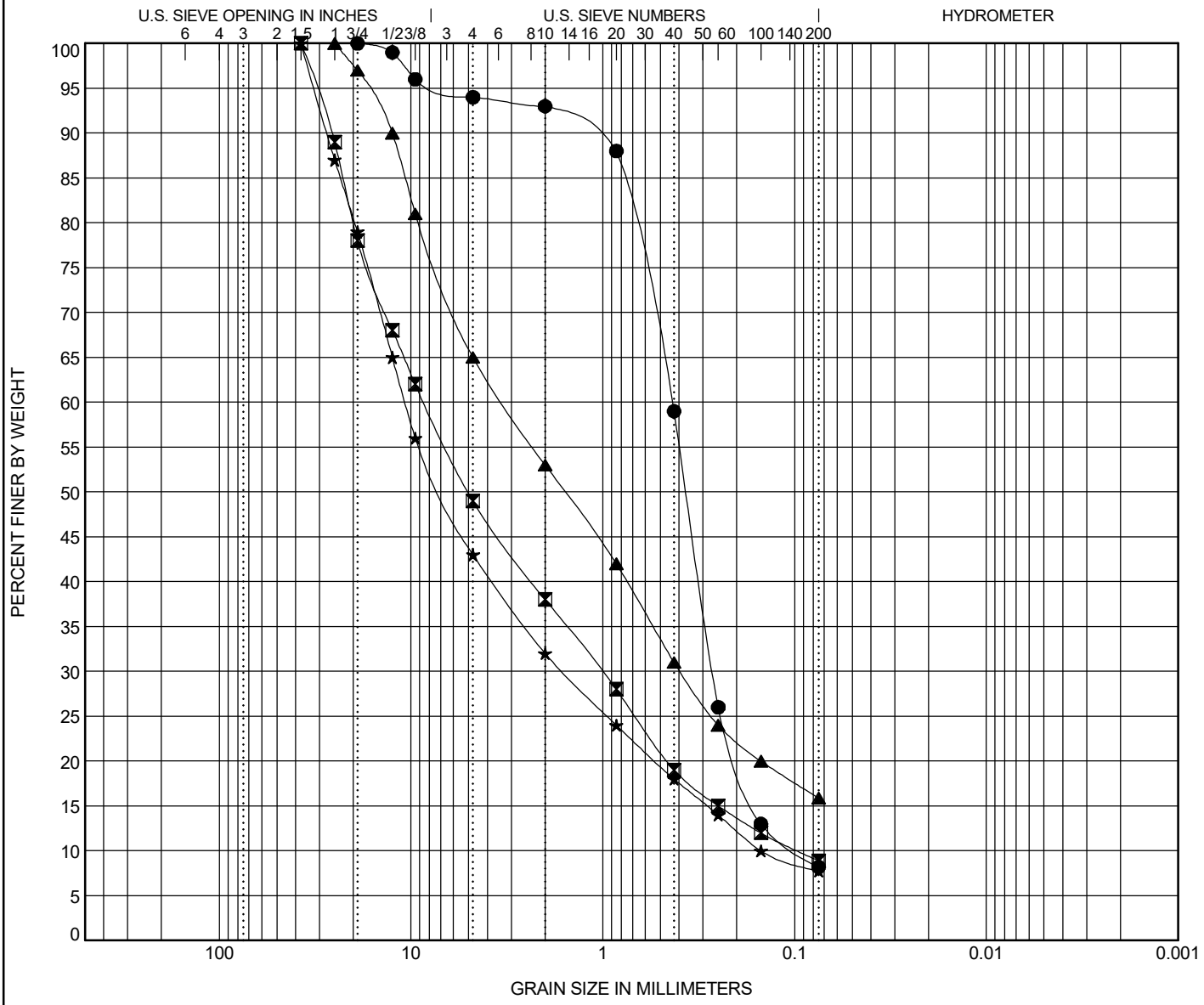
Prepared By: Travis Thomsen

Shannon & Wilson

PARTICLE-SIZE DISTRIBUTION REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification			Classification	D10	D30	D60
●	B-03	SS-08	20.0	POORLY GRADED SAND with SILT (SP-SM)	0.097	0.267	0.435
⊠	B-03	SS-10	30.0	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	0.096	1.009	8.539
▲	B-04	SS-26	5.0	SILTY SAND with GRAVEL (SM)	--	0.394	3.313
★	B-04	SS-29	12.5	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	0.15	1.615	10.732

	Specimen Identification			%Gravel	%Sand	%Fines	D15	D50	D85	Cc	Cu	MC	LL	PI
●	B-03	SS-08	20.0	6.0	85.8	8.2	0.162	0.368	0.791	1.68	4.48	9.6	--	--
⊠	B-03	SS-10	30.0	51.0	40.1	8.9	0.25	5.01	22.626	1.24	89.03	12.2	--	--
▲	B-04	SS-26	5.0	35.0	49.1	15.9	--	1.584	10.732	--	--	6.6	--	--
★	B-04	SS-29	12.5	57.0	35.2	7.8	0.285	6.899	23.342	1.62	71.55	3.8	--	--

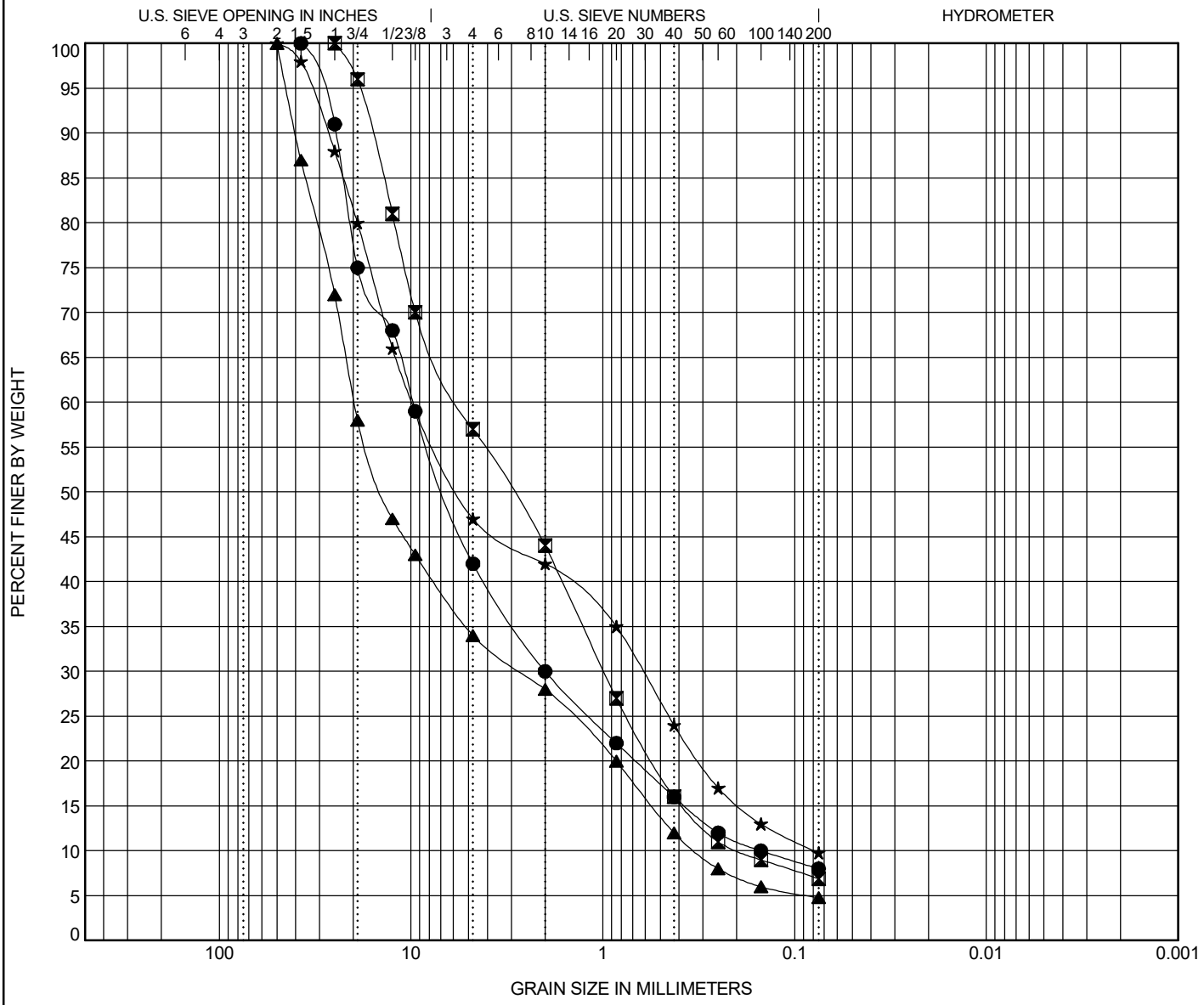
Prepared By: Travis Thomsen

Shannon & Wilson

PARTICLE-SIZE DISTRIBUTION REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification			Classification	D10	D30	D60
●	B-04	SS-31/32	17.5	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	0.15	2	9.794
⊠	B-05	GB-63	0.5	POORLY GRADED SAND with SILT and GRAVEL (SP-SM)	0.194	0.989	5.574
▲	B-05	GB-64	1.5	WELL-GRADED GRAVEL with SAND (GW)	0.326	2.668	19.76
★	B-05	Bulk	2.0	POORLY GRADED GRAVEL with SILT and SAND (GP-GM)	0.078	0.62	9.88

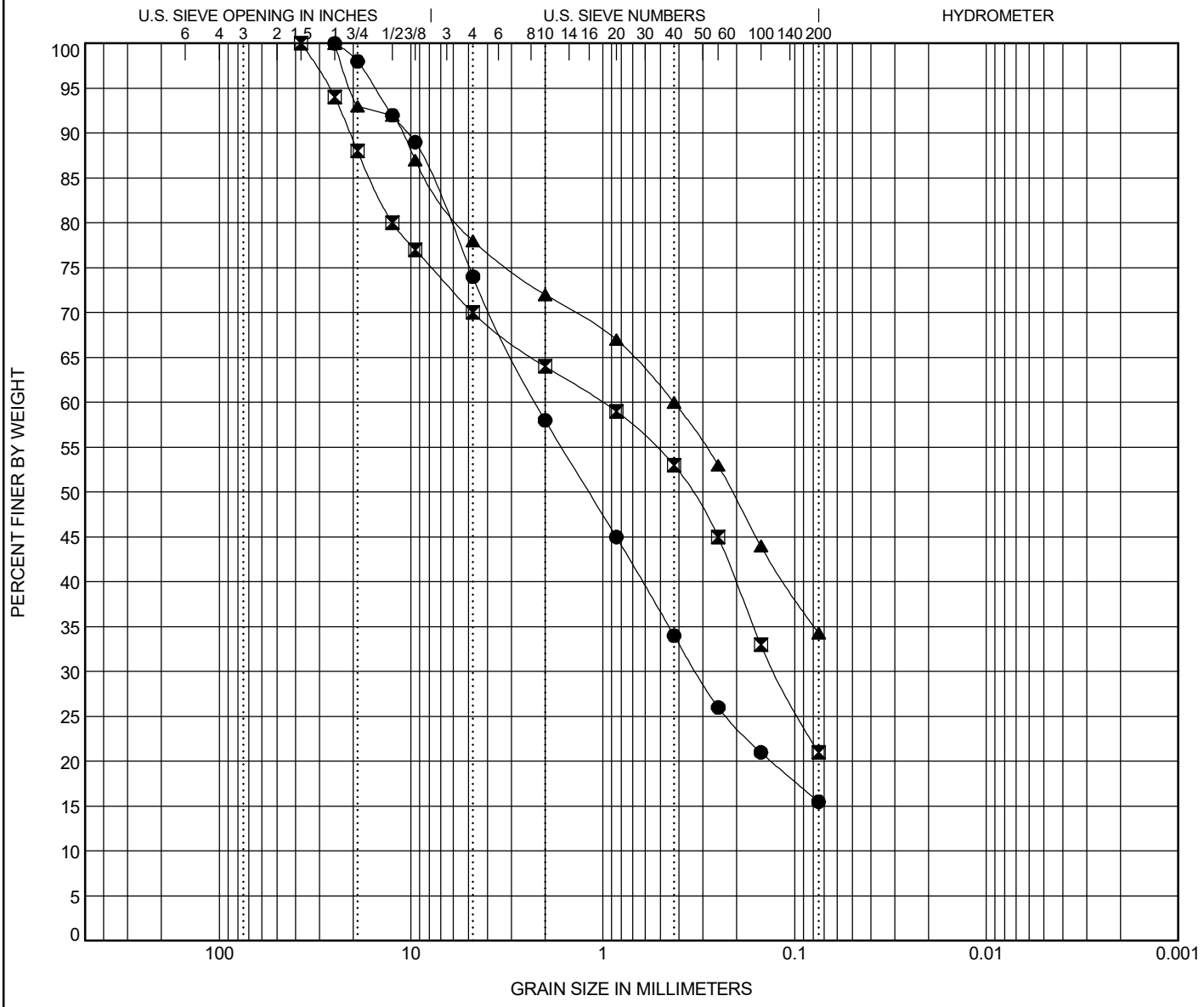
	Specimen Identification			%Gravel	%Sand	%Fines	D15	D50	D85	Cc	Cu	MC	LL	PI
●	B-04	SS-31/32	17.5	58.0	34.0	8.0	0.372	6.582	22.555	2.72	65.29	6.5	--	--
⊠	B-05	GB-63	0.5	43.0	50.1	6.9	0.382	2.981	13.977	0.91	28.78	3.7	--	--
▲	B-05	GB-64	1.5	66.0	29.2	4.8	0.551	14.012	35.527	1.11	60.62	3.4	--	--
★	B-05	Bulk	2.0	53.0	37.2	9.8	0.194	5.649	22.555	0.50	126.15	--	--	--

Prepared By: Travis Thomsen

PARTICLE-SIZE DISTRIBUTION REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification			Classification	D10	D30	D60
●	B-05	SS-68	10.0	SILTY SAND with GRAVEL (SM)	--	0.326	2.228
⊠	B-05	SS-71/72	17.5	SILTY SAND with GRAVEL (SM)	--	0.126	1.009
▲	B-06	SS-49	5.0	SILTY SAND with GRAVEL (SM)	--	--	0.425

	Specimen Identification			%Gravel	%Sand	%Fines	D15	D50	D85	Cc	Cu	MC	LL	PI
●	B-05	SS-68	10.0	26.0	58.5	15.5	--	1.181	7.897	--	--	4.7	--	--
⊠	B-05	SS-71/72	17.5	30.0	49.0	21.0	--	0.348	16.239	--	--	15.4	--	--
▲	B-06	SS-49	5.0	22.0	43.7	34.3	--	0.211	8.144	--	--	10.2	NV	NP

Prepared By: Travis Thomsen

SHANNON & WILSON

Project Information

Report to: Forsgren
Project: Teton River Bridge
Report Date: 10/7/2024
File No.: 112645

Material Information

Date Sampled: 9/22/2024
Sampled By: Shannon & Wilson
Date Received: 9/28/2024
Date Tested: 10/1/2024

SUMMARY OF LABORATORY RESULTS

Lab Number	Borehole	Sample Type	Depth (ft)	Water Content (%)	% Passing #200 Sieve	Liquid Limit (%)	Plasticity Index	Soil Type	Remarks
24-1105	TP-01 SE	Bulk	0.0'-1.0'	3.6	7.9	NV	NP	GW-GM	--
24-1106	TP-02 NW	Bulk	0.0'-1.0'	8.9	16.7	NV	NP	GM	--

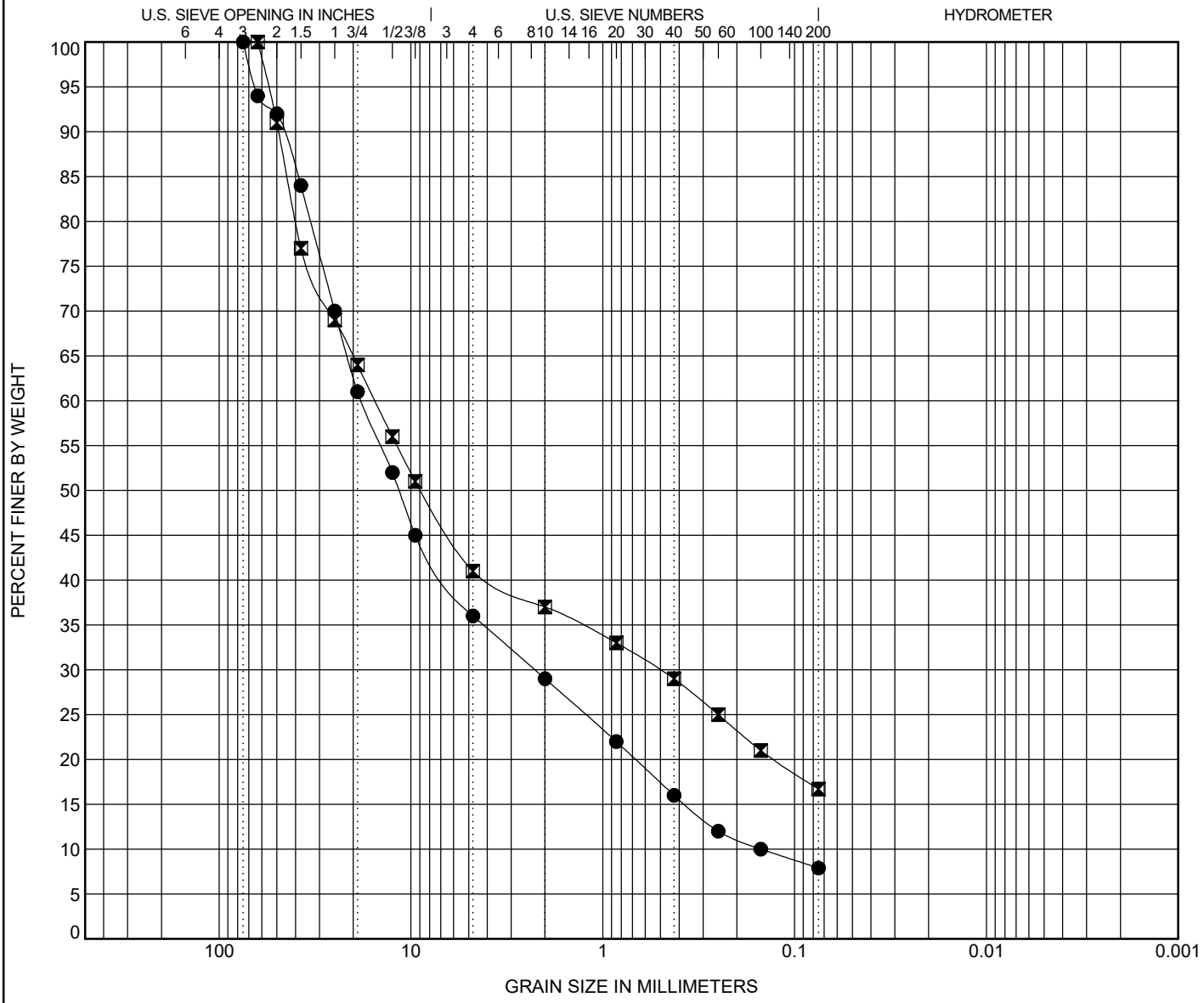
Prepared By: Aliona Chernish

QC:TT

PARTICLE-SIZE DISTRIBUTION REPORT

SHANNON & WILSON

CLIENT: Forsgren
FILE NUMBER: 112645
PROJECT NAME: Teton River Bridge
PROJECT LOCATION: Teton, Idaho



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	D10	D30	D60
●	TP-01 SE Bulk 1.0	WELL-GRADED GRAVEL with SILT and SAND (GW-GM)	0.15	2.263	18.136
⊠	TP-02 NW Bulk 1.0	SILTY GRAVEL with SAND (GM)	--	0.505	15.411

	Specimen Identification	%Gravel	%Sand	%Fines	D15	D50	D85	Cc	Cu	MC	LL	PI
●	TP-01 SE Bulk 0.0	64.0	28.1	7.9	0.372	11.557	38.873	1.88	120.91	3.6	NV	NP
⊠	TP-02 NW Bulk 0.0	59.0	24.3	16.7	--	8.864	44.2	--	--	8.9	NV	NP

QC:TT

Prepared By: Aliona Chernish

Shannon & Wilson
5260 Chinden Blvd.
Boise, Idaho 83714
Phone:(208) 658-8700

SHANNON & WILSON

Report To: Forsgren
Project: South Fork Teton Bridge
Project No.: 112645
Sample ID: B-2, Bulk; 2.0'-4.0'
Soil Description: Silty Gravel with Sand (GM)

Report Date: 10/23/2024
Date Sampled: 9/22/2024
Date Received: 9/28/2024
Tested By: AC
Lab Number: 24-1103

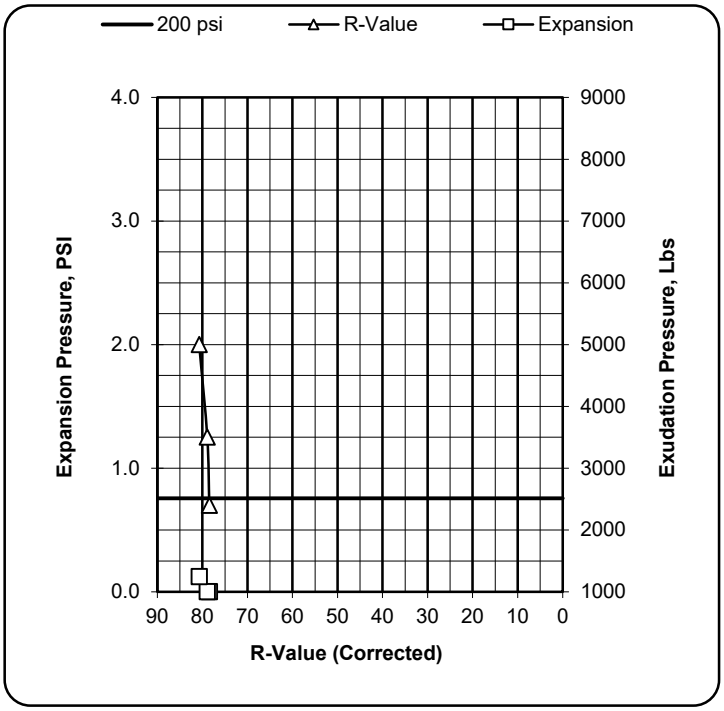
R-VALUE

IDAHO T-8

	Point 1	Point 2	Point 3
Drainage Description	Free Drain	Free Drain	Free Drain
Dry Density, PCF	124.6	123.0	122.3
Moisture Content, %	8.4	7.6	7.1
Exudation, PSI	191	279	398
R-Value (Corrected)	78	79	81
Expansion, PSI	0.00	0.00	0.12

R-Value @ 200 PSI Exudation Pressure

78



Gradation: AASHTO T-11, T-27		
Screen Sizes	% Passing As Received	% Passing As Tested
4"		
3"		
2"	100	
1"	91	
3/4"	83	100
1/2"	73	88
3/8"	66	80
No. 4	52	63
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

* This report covers only material as represented by this sample and does not necessarily cover all soils from this layer or source.

Reviewed By: *Travis Thomsen*

Shannon & Wilson
5260 Chinden Blvd.
Boise, Idaho 83714
Phone:(208) 658-8700

SHANNON & WILSON

Report To: Forsgren
Project: South Fork Teton Bridge
Project No.: 112645
Sample ID: B-5, Bulk; 2.0'-4.0'
Soil Description: Poorly Graded Gravel with Silt and Sand (GP-GM)

Report Date: 10/23/2024
Date Sampled: 9/22/2024
Date Received: 9/28/2024
Tested By: AC
Lab Number: 24-1104

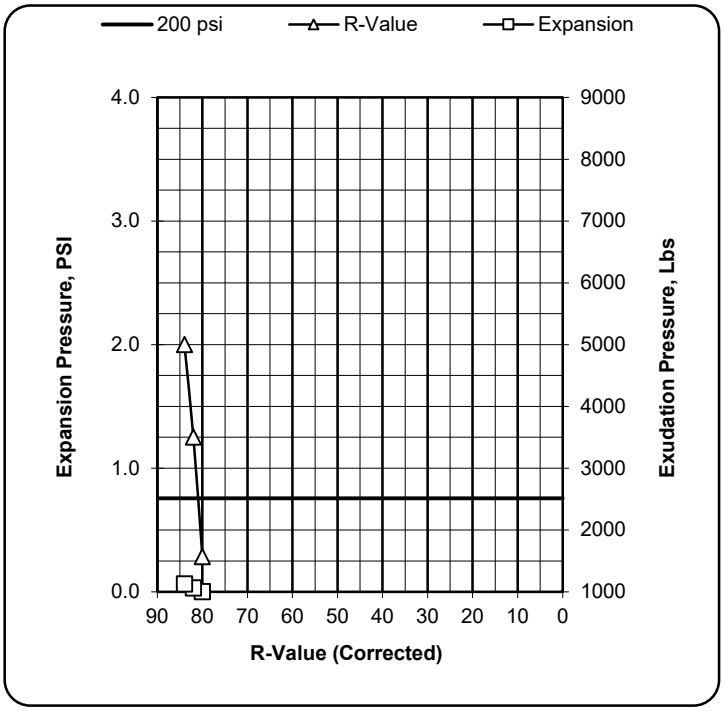
R-VALUE

IDAHO T-8

	Point 1	Point 2	Point 3
Drainage Description	Moderate	Free Drain	Free Drain
Dry Density, PCF	125.0	124.5	123.9
Moisture Content, %	9.0	8.5	8.0
Exudation, PSI	125	279	398
R-Value (Corrected)	80	82	84
Expansion, PSI	0.00	0.03	0.06

R-Value @ 200 PSI Exudation Pressure

81



Gradation: AASHTO T-11, T-27		
Screen Sizes	% Passing As Received	% Passing As Tested
4"		
3"		
2"	100	
1"	88	
3/4"	80	100
1/2"	66	83
3/8"	59	74
No. 4	47	59
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

* This report covers only material as represented by this sample and does not necessarily cover all soils from this layer or source.

Reviewed By: *Travis Thomsen*

Shannon & Wilson
5260 Chinden Blvd.
Boise, Idaho 83714
Phone:(208) 658-8700

SHANNON & WILSON

Report to: Forsgren
Project: South Fork Teton Bridge
Report Date: 10/25/2024
Project No.: 112645

Material Information

Date Sampled: 9/22/2024
Sampled By: S&W
Date Received: 9/28/2024
Date Tested: 10/25/2024

Compressive Strength of Intact Rock Cores ASTM D7012 Method C

Lab Number	Sample ID	Sample Depth, ft	Average Diameter, in	Average Height, in	Load, lbs	Compressive Strength, psi
24-1107	B-3	42.5'	2.406	4.733	21,368	**4,702**
24-1108	B-3	47.0'	2.407	5.095	61,846	**13,597**
24-1109	B-3	52.5'	2.408	5.060	67,613	**14,847**
24-1110	B-3	57.0'	2.411	4.975	84,818	**18,586**
24-1111	B-4	36.5'	2.413	5.259	59,960	**13,117**
24-1112	B-4	42.5'	2.416	4.905	34,154	**7,453**

Reviewed By: *Travis Thomsen*

Shannon & Wilson
5260 Chinden Blvd.
Boise, Idaho 83714
Phone:(208) 658-8700

Report to: Forsgren
Project: South Fork Teton Bridge
Report Date: 10/23/2024
File No.: 112645

Material Information

Date Sampled: 9/22/2024
Sampled By: Shannon & Wilson
Date Received: 9/28/2024
Date Tested: 10/23/2024

Test Results

Soil Resistivity and pH for Corrosion Testing

AASHTO T-288, T-289

Lab Number	Sample ID	Soil Type	Minimum Resistivity Ohm-cm	pH	@ Temp, F°
24-1103	B-2, Bulk; 2.0' - 4.0'	GM	4,030	8.3	67.7
24-1104	B-5, Bulk; 2.0' - 4.0'	GP-GM	6,890	8.0	68.0

Reviewed By: *Travis Thomsen*

Analytical Laboratories Inc

1804 N. 33rd Street
Boise, Idaho 83703
Phone (208) 342-5515

www.analyticallaboratories.com
Lab Federal ID# ID00020

Laboratory Analysis Report

PWS#: 112645

Report To: ROXY BOUCHER
SHANNON & WILSON
5260 CHINDEN BLVD
BOISE, ID 83714
Phone: (208) 658-8700 Copy:
e-mail: roxy.boucher@shanwil.com
Date Received: 10/23/2024
Collection Date/Time: 09/22/2024 0:00

Lab/Sample Number: 2424723-01
Sample Location: SOUTH FORK TETON BRIDGE - B-2, BULK; 2.0' - 4.0'
Date Printed: 11/04/2024 15:07
Collector's Name:
Transported By:
Temp C Received at Lab:

Field Measurements

pH: Total Chlorine mg/L: DO mg/L:
Temp C: Free Chlorine mg/L: Flow g/min:

Analyte	Result	Units	MRL	MDL	MCL	Analyzed	Analyst	Method	Notes
Inorganics									
Sulfate, SO4	23	mg/kg	10	2		11/1/24 23:42	NC	EPA 300.0	
Chloride, Cl	39	mg/kg	10	2		11/1/24 23:42	NC	EPA 300.0	

Analytical Laboratories Inc

1804 N. 33rd Street
Boise, Idaho 83703
Phone (208) 342-5515

www.analyticallaboratories.com
Lab Federal ID# ID00020

Laboratory Analysis Report

PWS#: 112645

Report To: ROXY BOUCHER
SHANNON & WILSON
5260 CHINDEN BLVD
BOISE, ID 83714
Phone: (208) 658-8700 Copy:
e-mail: roxy.boucher@shanwil.com
Date Received: 10/23/2024
Collection Date/Time: 09/22/2024 0:00

Lab/Sample Number: 2424723-02
Sample Location: SOUTH FORK TETON BRIDGE - B-5, BULK; 2.0' - 4.0'
Date Printed: 11/04/2024 15:07
Collector's Name:
Transported By:
Temp C Received at Lab:

Field Measurements

pH: Total Chlorine mg/L: DO mg/L:
Temp C: Free Chlorine mg/L: Flow g/min:

Analyte	Result	Units	MRL	MDL	MCL	Analyzed	Analyst	Method	Notes
Inorganics									
Sulfate, SO4	10	mg/kg	10	2		11/1/24 23:56	NC	EPA 300.0	
Chloride, Cl	32	mg/kg	10	2		11/1/24 23:56	NC	EPA 300.0	

Authorized Signature,

JAMES HIBBS, Client Manager

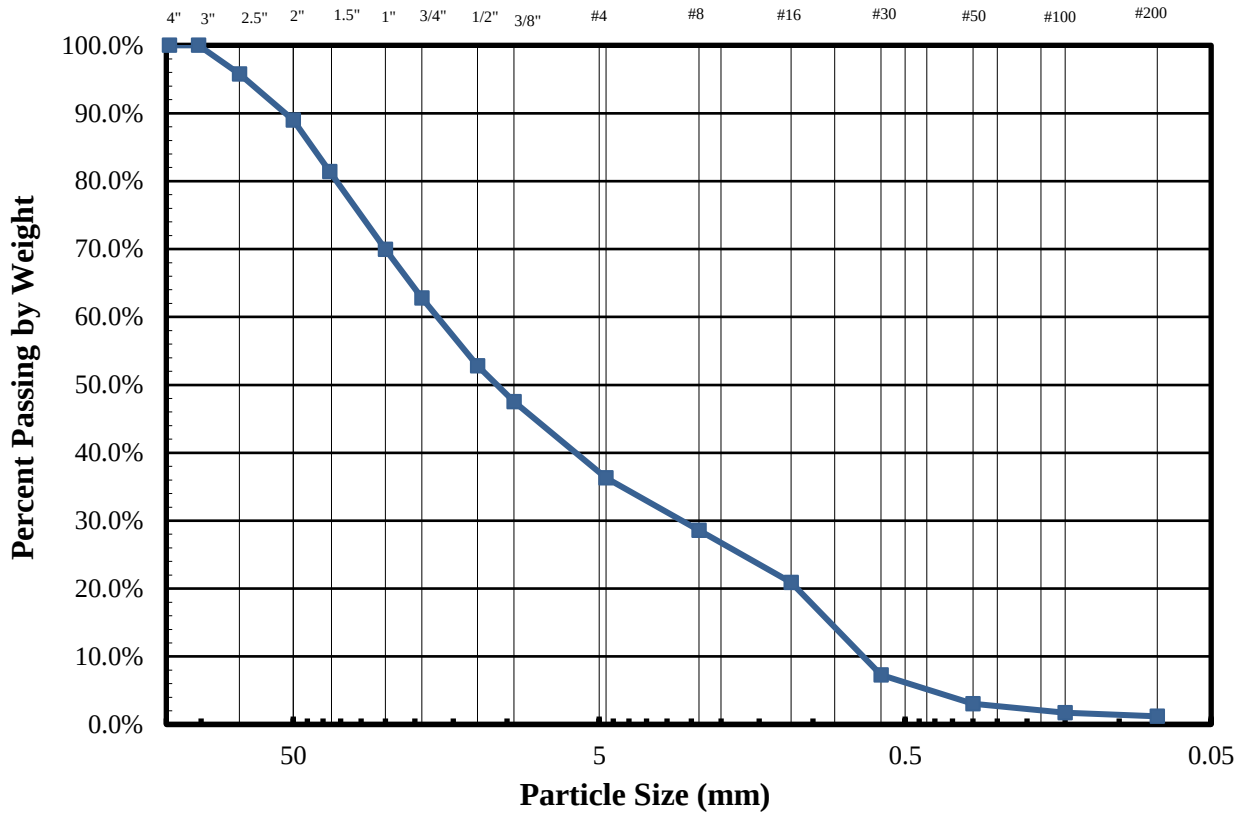
This report shall not be reproduced except in full, without the written approval of the laboratory
The results reported relate only to the samples indicated.

ND - Non Detect
MCL - Maximum Contaminant Level
MDL - Method Detection Limit
MRL - Method Reporting Limit

FORSGREN Associates Inc.
1137 Summers Drive
Rexburg, ID 83440
208-356-9201

Client: LHTAC (Key# 24341)
Project: S. Teton River Bridge

Material Description: River Bed
Sample Location: Onsite from river bed



Sieve Analysis
AASHTO T27

Job No.	02-24-0018
Lab No.	24-116
Received	October 8, 2024
Reported	October 9, 2024

SIZE	PASSING	SPECIFICATION Low	High
4"	100%		
3"	100%		
2.5"	96%		
2"	89%		
1.5"	81%		
1"	70%		
3/4"	63%		
1/2"	53%		
3/8"	48%		
No. 4	36%		
No. 8	29%		
No. 16	21%		
No. 30	7%		
No. 50	3%		
No. 100	2%		
No. 200	1.2%		

Moisture Content
AASHTO T255

16.7%

Sand Equivalent (AASHTO T176) = N/A

Reviewed By:
Eric Gunnarson
Laboratory Manager

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APPENDIX E – LPILE ANALYSES

Appendix E

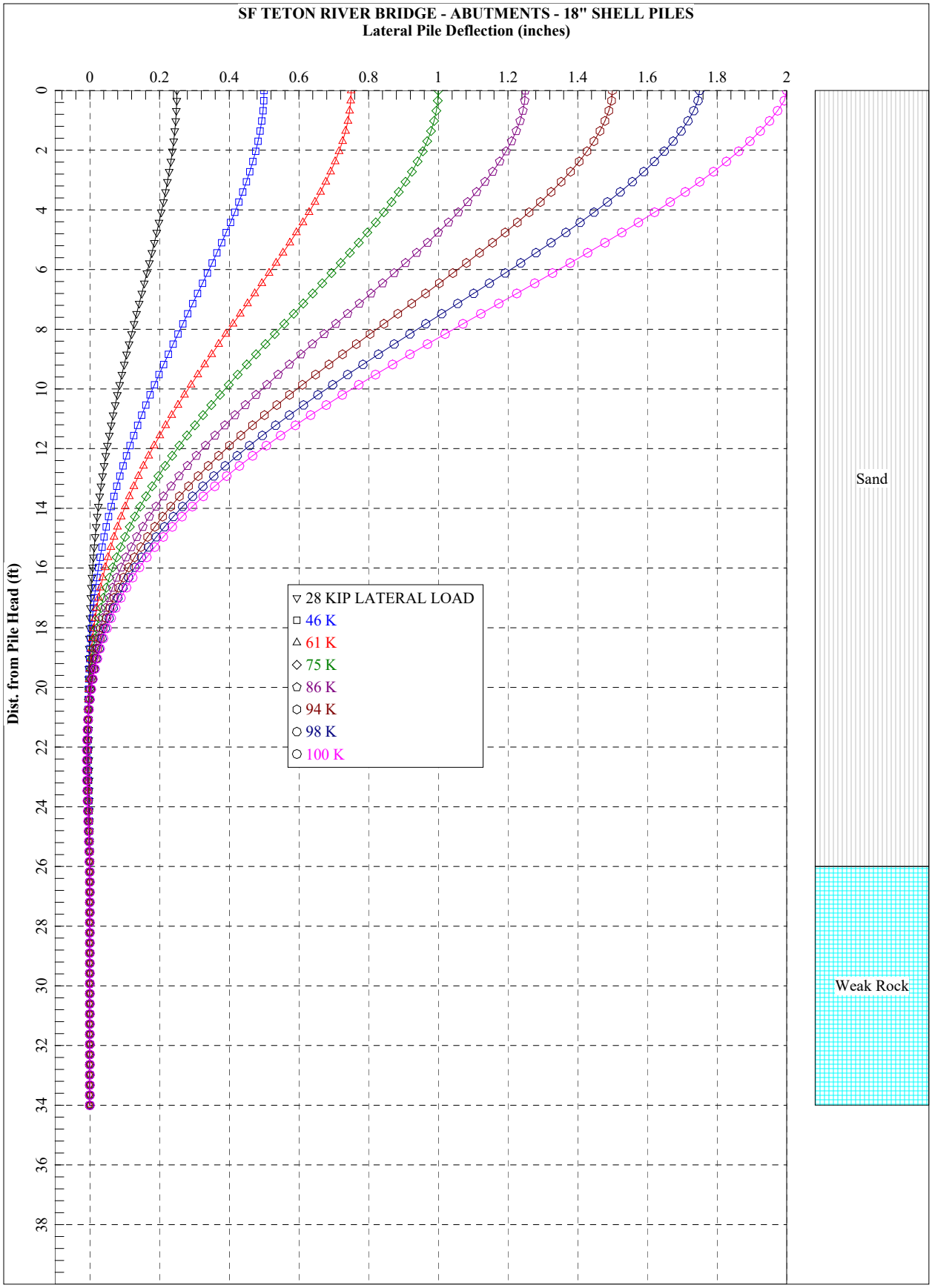
LPILE Analyses

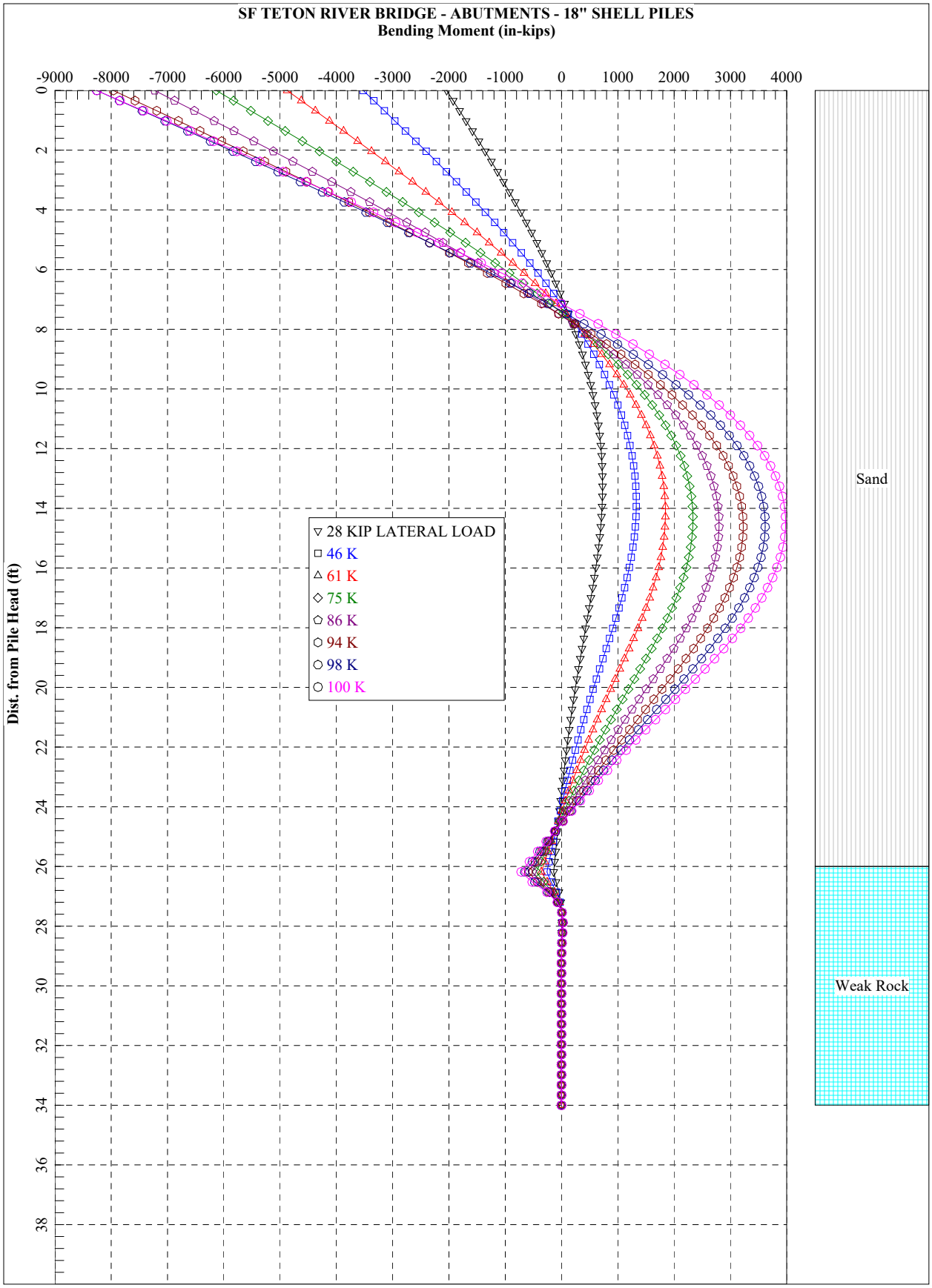
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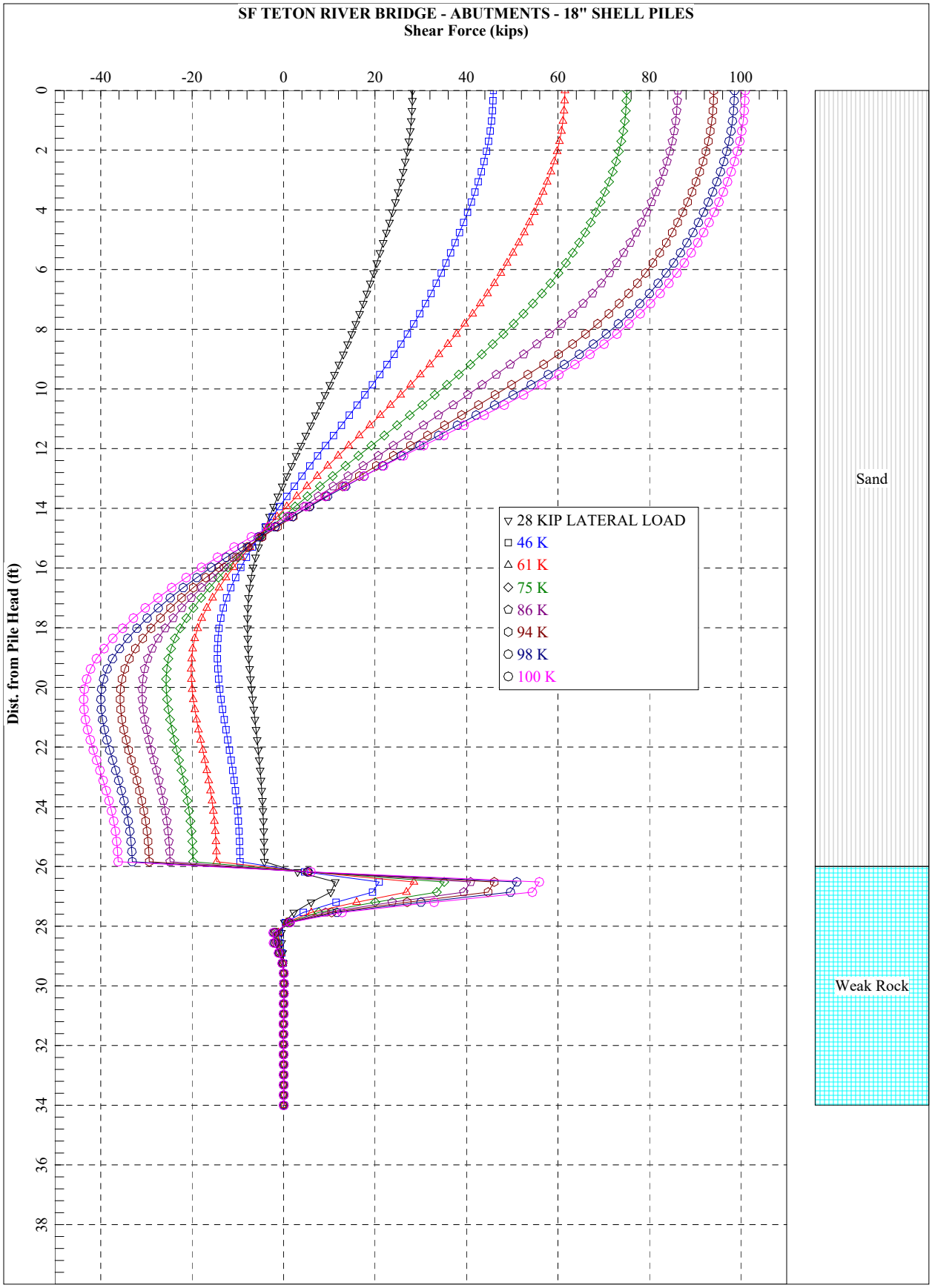
LPILE Analyses
- Abutments
- Center Pier

APPENDIX E – LPILE ANALYSES

ABUTMENTS

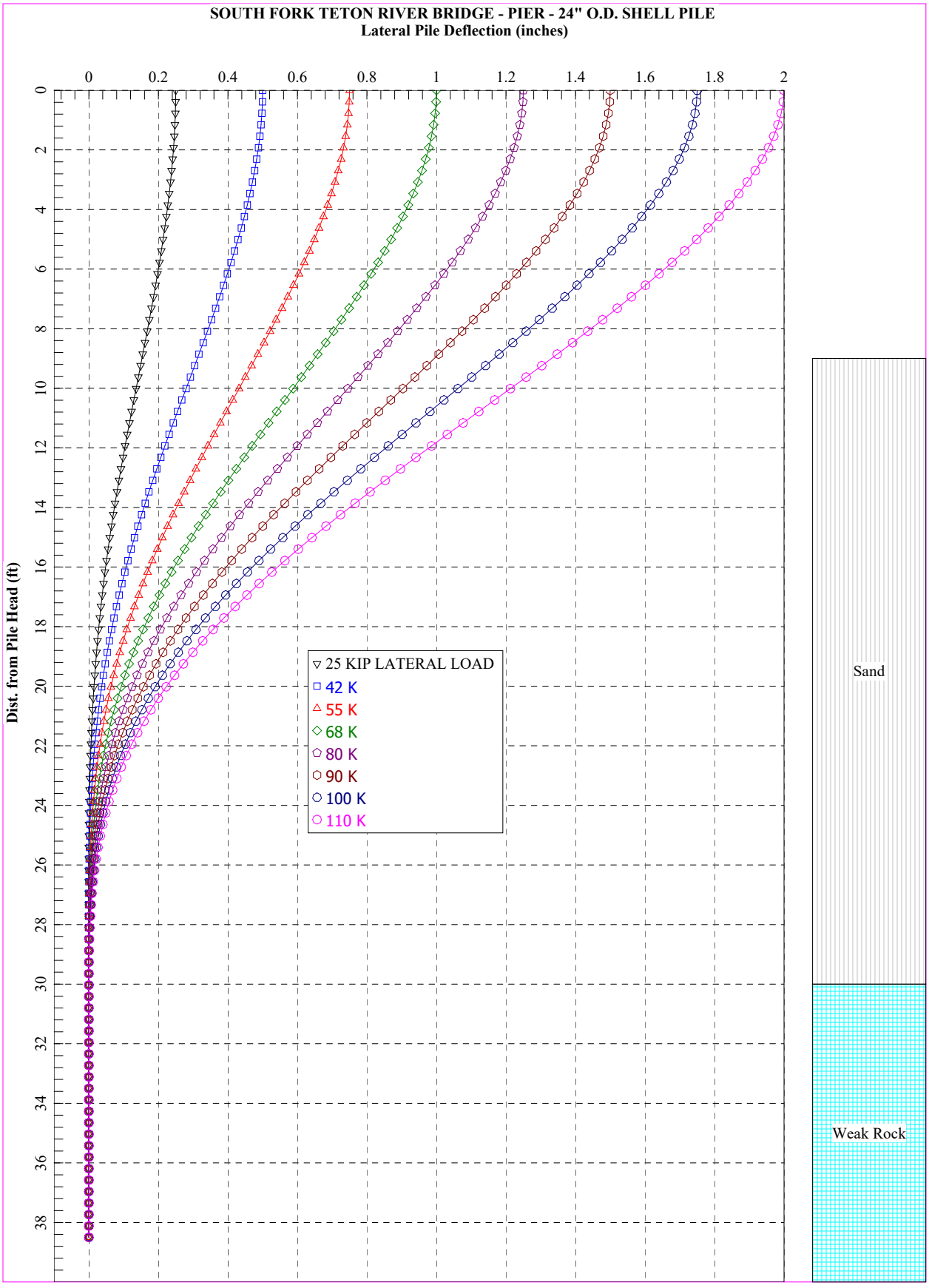

SF TETON RIVER BRIDGE - ABUTMENTS - 18" SHELL PILES
Lateral Pile Deflection (inches)
Dist. from Pile Head (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
28 KIP LATERAL LOAD
46 K
61 K
75 K
86 K
94 K
98 K
100 K
Sand
Weak Rock

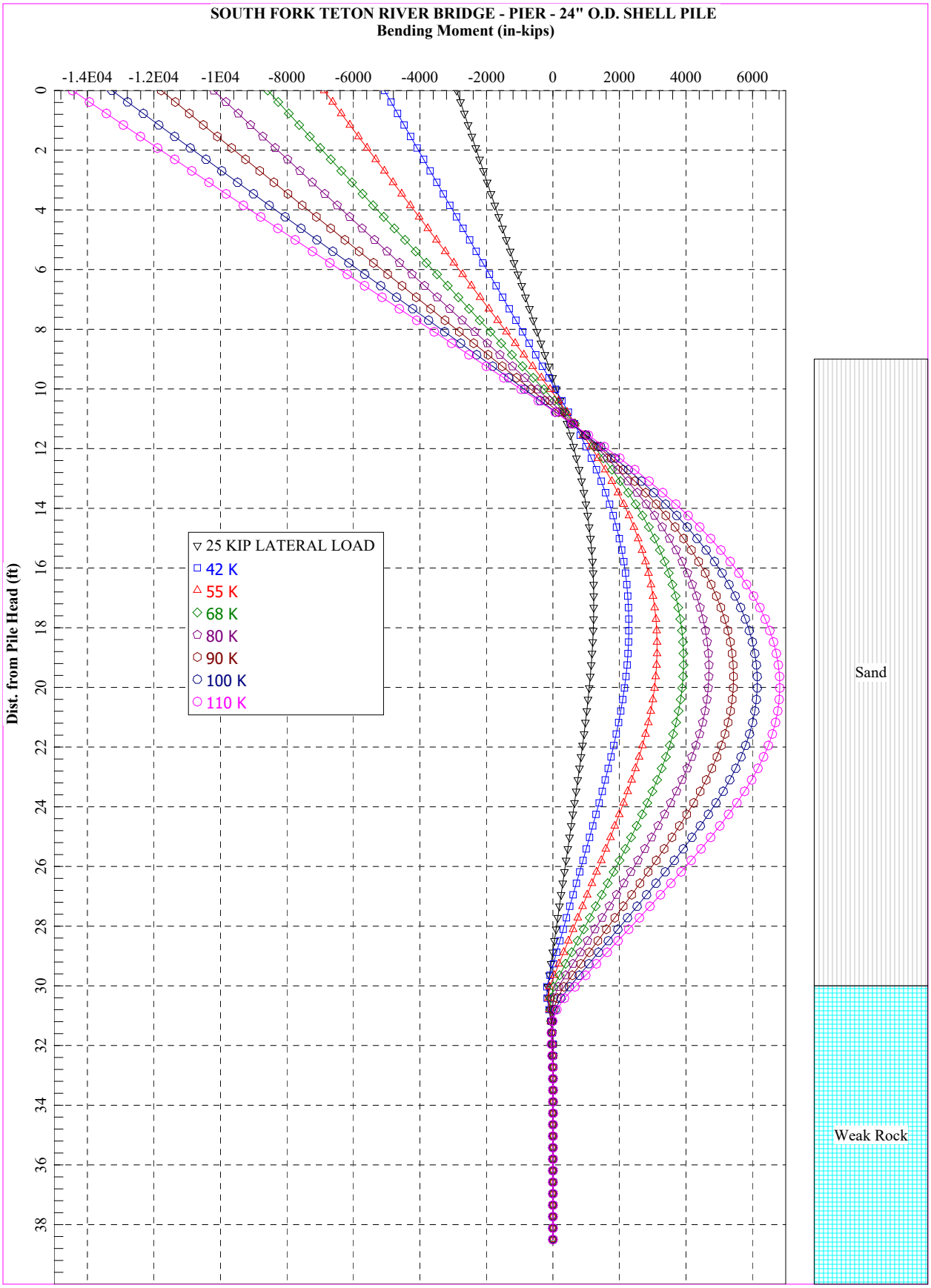

SF TETON RIVER BRIDGE - ABUTMENTS - 18" SHELL PILES
Bending Moment (in-kips)
-9000
-8000
-7000
-6000
-5000
-4000
-3000
-2000
-1000
0
1000
2000
3000
4000
Dist. from Pile Head (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
28 KIP LATERAL LOAD
46 K
61 K
75 K
86 K
94 K
98 K
100 K
Sand
Weak Rock

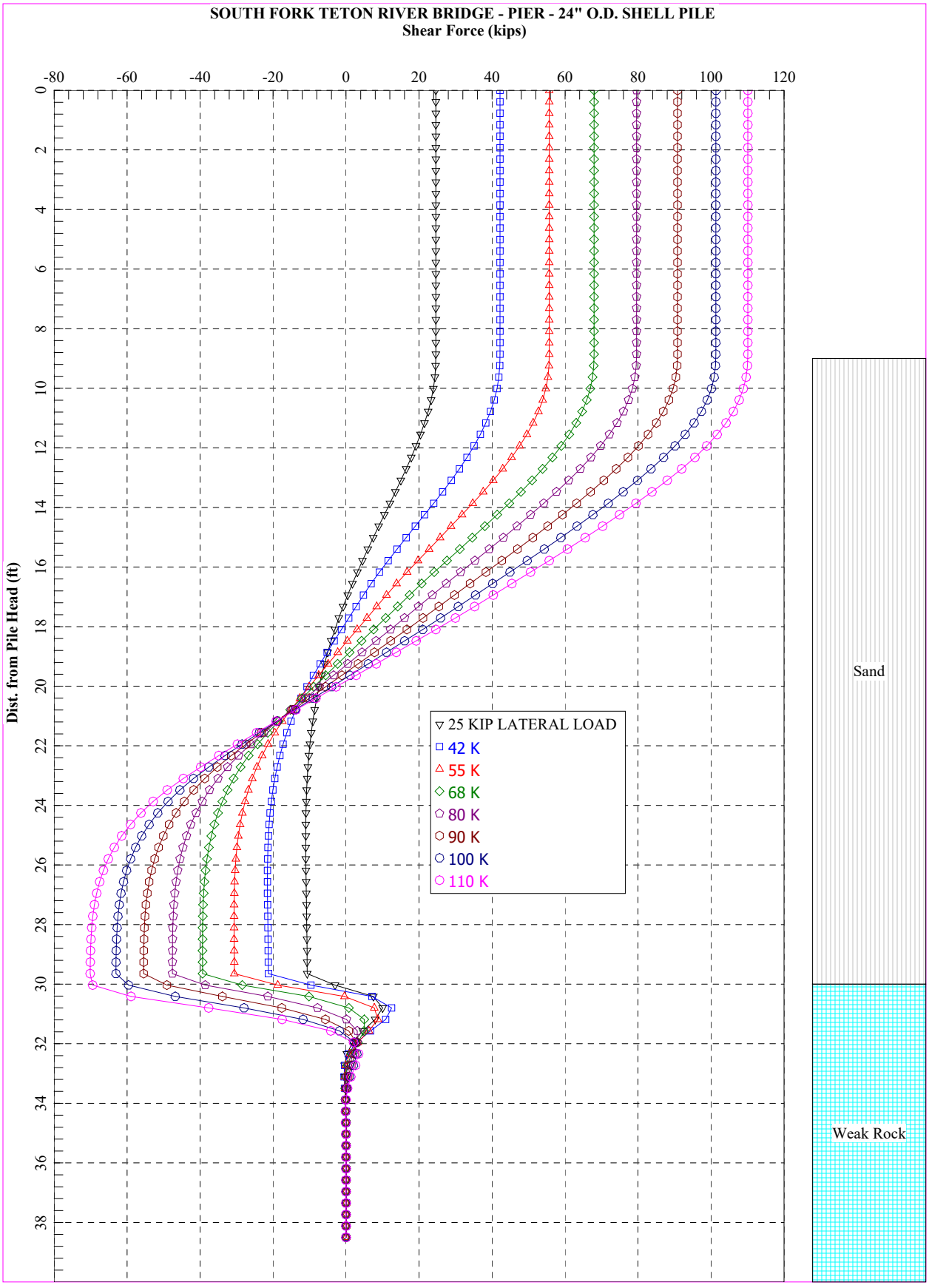

SF TETON RIVER BRIDGE - ABUTMENTS - 18" SHELL PILES
Shear Force (kips)
-40
-20
0
20
40
60
80
100
Dist. from Pile Head (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
28 KIP LATERAL LOAD
46 K
61 K
75 K
86 K
94 K
98 K
100 K
Sand
Weak Rock

APPENDIX E – LPILE ANALYSES

CENTER PIER


SOUTH FORK TETON RIVER BRIDGE - PIER - 24" O.D. SHELL PILE
Lateral Pile Deflection (inches)
Dist. from Pile Head (ft)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
25 KIP LATERAL LOAD
42 K
55 K
68 K
80 K
90 K
100 K
110 K
Sand
Weak Rock


SOUTH FORK TETON RIVER BRIDGE - PIER - 24" O.D. SHELL PILE
Bending Moment (in-kips)
-1.4E04
-1.2E04
-1E04
-8000
-6000
-4000
-2000
0
2000
4000
6000
Dist. from Pile Head (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
25 KIP LATERAL LOAD
42 K
55 K
68 K
80 K
90 K
100 K
110 K
Sand
Weak Rock


SOUTH FORK TETON RIVER BRIDGE - PIER - 24" O.D. SHELL PILE
Shear Force (kips)
-80
-60
-40
-20
0
20
40
60
80
100
120
Dist. from Pile Head (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
25 KIP LATERAL LOAD
42 K
55 K
68 K
80 K
90 K
100 K
110 K
Sand
Weak Rock

APPENDIX F – SLOPE/W STABILITY ANALYSES

Appendix F

SLOPE/W Stability Analyses

CONTENTS

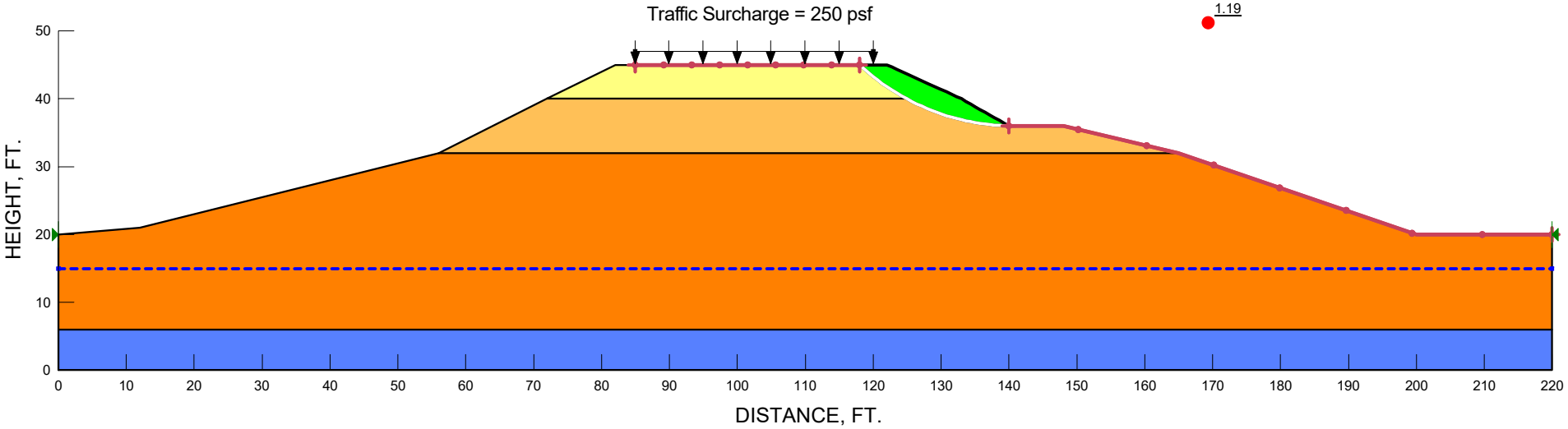
Slope W/ Stability Analyses Results

- Approach Embankment (Station 29+00)
- Approach Embankment (Station 32+50)

APPENDIX F – SLOPE/W STABILITY ANALYSES

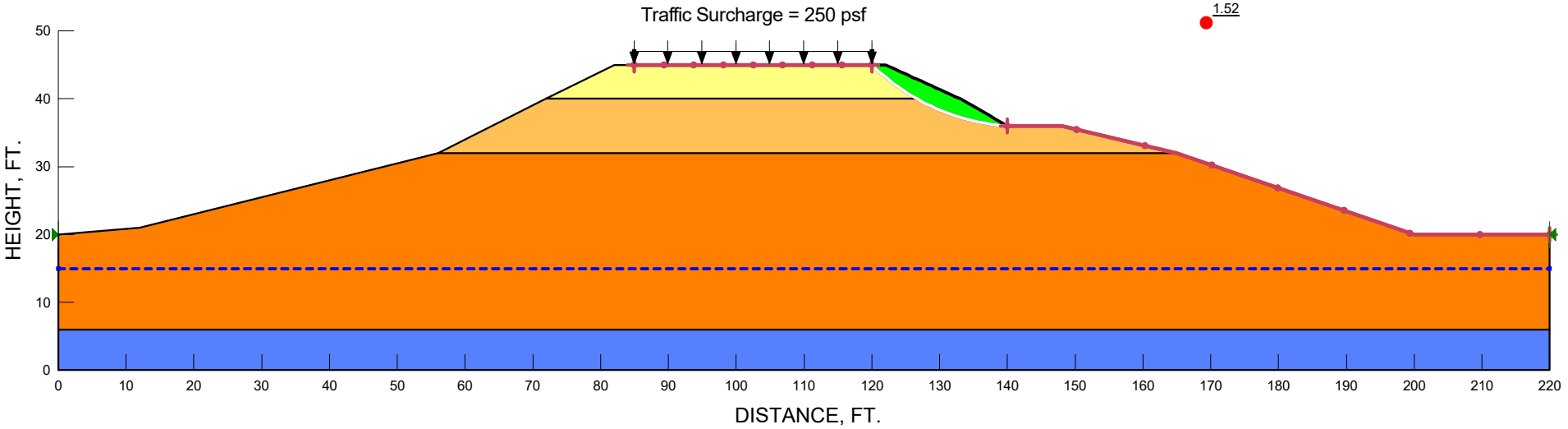
APPROACH EMBANKMENT (STATION 29+00)

S.F. TETON RIVER BRIDGE - APPROACH EMBANKMENT AT STATION 29+00 - STABILITY ANALYSIS
Minimum FS = 1.19 (Seismic Condition)



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1 Dense Gravel with Sand	125	0	36
	2 Medium Dense Sandy Silt	115	0	33
	3 Dense Sand with Gravel	125	0	36
	4 Basalt Bedrock			

S.F. TETON RIVER BRIDGE - APPROACH EMBANKMENT AT STA. 29+00 - STABILITY ANALYSIS
Minimum FS = 1.52 (Static Condition)

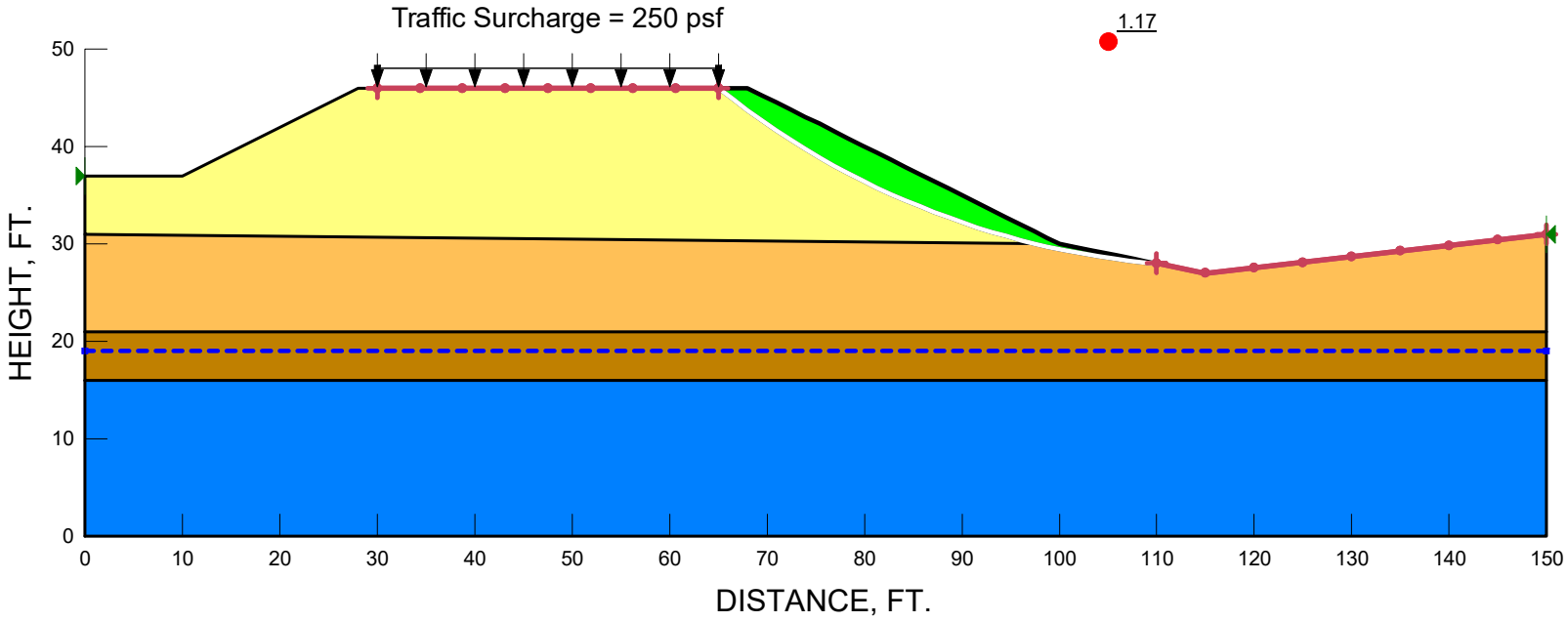


Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1 Dense Gravel with Sand	125	0	36
	2 Medium Dense Sandy Silt	115	0	33
	3 Dense Sand with Gravel	125	0	36
	4 Basalt Bedrock			

APPENDIX F – SLOPE/W STABILITY ANALYSES

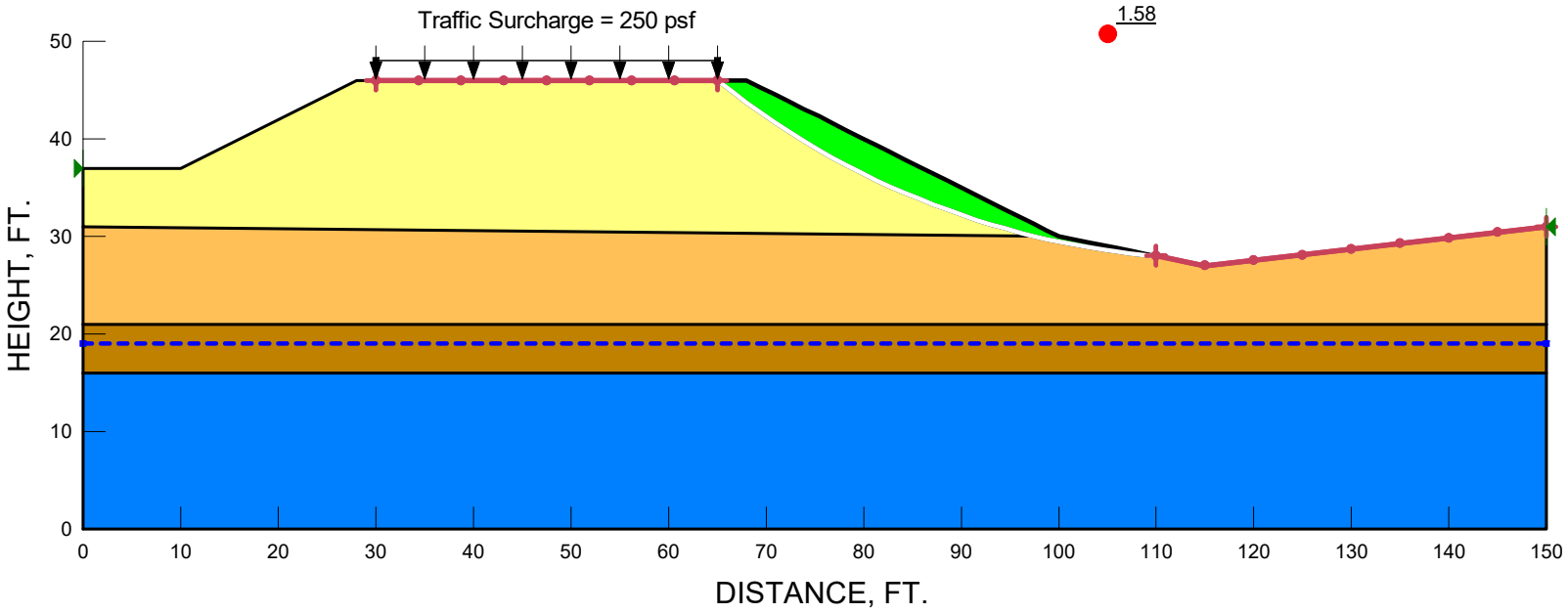
APPROACH EMBANKMENT (STATION 32+50)

S.F. TETON RIVER BRIDGE - APPROACH EMBANKMENT AT STA. 32+50 - STABILITY ANALYSIS
Minimum FS = 1.17 (Seismic Condition)



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1 Dense Gravel and Sand	125	0	36
	2 Medium Dense Sand and Gravel	110	0	32
	3 Dense Sand with Gravel	125	0	36
	4 Basalt Bedrock			

S.F. TETON RIVER BRIDGE - APPROACH EMBANKMENT AT STA. 32+50 - STABILITY ANALYSIS
Minimum FS = 1.58 (Static Condition)



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	1 Dense Sand and Gravel	125	0	36
	2 Medium Dense Sand and Gravel	110	0	32
	3 Dense Sand with Gravel	125	0	36
	4 Basalt Bedrock			

APPENDIX G – FLEXIBLE PAVEMENT DESIGN

Appendix G

Flexible Pavement Design

CONTENTS

- Flexible Pavement Design
  - Traffic Data
  - Traffic Projection Worksheet
  - Design R-Value Worksheet
  - LTTPBIND Output

APPENDIX G – FLEXIBLE PAVEMENT DESIGN

TRAFFIC DATA

Projected Commercial and 18,000 Equivalent Single Axle Loadings (ESALS)

Project Number:	*24(341)*	*Key Number:*	*24341*
County:	*Fremont*	*District:*	*District_6*
Route:	*OH-Off Highway*	*From Measure:*	*7.104*
RouteID:	*05540AOH000*	*To Measure:*	*7.333*
Last year with data:	*2023*		
Starting to Cumulate in:	*2026*	*Requested By:*	*Adam Bass*
Cumulating ESALs up to:	*2048*	*Email:*	*abass@forsgren.com*

					2023: Base Year Values					Projection ADTs			Flexible Pavement ESALs (In 1000s)	
													Both Directions	
RouteID	**From Measure**	**To Measure**	**Description From**	**Description To**	**AADT**	**Passenger AADT**	**CommercialADT**	**Truck Density**	**Projection Year**	**Total (AADT)**	**Passenger (PAADT)**	**Commercial (CAADT)**	**flexible year value**	**flexible cumulative**
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2026	1141	865	276	32	32
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2027	1154	874	281	33	65
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2028	1168	882	286	33	99
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2029	1182	890	291	34	133
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2030	1195	899	296	35	167
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2031	1209	907	302	35	203
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2032	1222	916	307	36	239
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2033	1236	924	312	37	275
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2034	1250	932	317	37	312
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2035	1263	941	322	38	350
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2036	1277	949	328	38	388
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2037	1290	958	333	39	427
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2038	1304	966	338	40	467
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2039	1318	974	343	40	507
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2040	1331	983	348	41	548
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2041	1345	991	354	41	589
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2042	1358	1000	359	42	631
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2043	1372	1008	364	43	674
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2044	1386	1016	369	43	717
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2045	1399	1025	374	44	761
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2046	1413	1033	380	44	805
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2047	1426	1042	385	45	850
05540AOH000	**7.1044159**	7.332861	**MAIN ST (SH-33)**	E 300 N RD	**1100**	840	**260**	1 - Light	2048	1440	1050	390	46	896

APPENDIX G – FLEXIBLE PAVEMENT DESIGN

TRAFFIC PROJECTION WORKSHEET

Traffic Projection Worksheet

Project: **South Teton River Bridge** Date: **November 1, 2024**

Flexible Pavement --> Beginning Year: **2026** Ending Year: **2046** **Analysis Period: 20**

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		Roadway Segment						
		Mainline						
	Lane Information							
	No. of Lanes in Design Direction	**1**						
	% Split of Trucks in Design Direction	**60%**						
	% of Trucks in Design Lane	**100%**						
From ITD's AADT Projection Report	Total Traffic - Both Directions							Flexible
	2026 AADT							
	2046 AADT							
	AADT for the Analysis Period							
	Truck Traffic - Both Directions							
	% Truck Traffic							
	AADTT							
	Truck Density (H, M, L)							
	Design Lane							
	AADTT							
	Traffic Index, TI							
	Flexible Design ESALs							
	Compound Annual Growth Rate							
From ITD's ESAL Projection Report	Flexible ESAL Projections - Both Directions							
	2026 Yearly and Cummulative ESALs	**32,000**						
	2046 Cummulative ESALs	**805,000**						
	Design Lane							
	Flexible Design ESALs (with reliability)	**768,980**						
	Traffic Index, TI	**8.7**						
	Rigid ESAL Projections - Both Directions							Rigid
	Yearly and Cummulative ESALs							
	Cummulative ESALs							
	Design Lane							
	Rigid Design ESALs (with reliability)							

APPENDIX G – FLEXIBLE PAVEMENT DESIGN

DESIGN R-VALUE WORKSHEET

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FLEXIBLE PAVEMENT DESIGN - Idaho R-Value Method

Project Name:	**South Teton River Bridge**
S&W Project No:	**71-112645**
Date:	**1-Nov-2024**
Roadway Segment:	**MP 7.200**
Strategy:	**Flexible Construction**

Layer Parameters

Layer 1:	**HMA:**		**GE 3**
Layer 2:	**Aggregate Base:**	**R= 80**	**GE 2**
Layer 3:	**Granular Subbase:**	**R= 60**	**GE 1**
Layer 4:	**Subgrade:**	**R= 60**	

→ESALs ≈ 768,980 **Traffic Index (T.I.):** 8.7
(Reference Section 510.02 ITD Materials Manual)
Regional Climate Factor: 1.15
(Reference Figure 510.04.01.1 ITD Materials Manual)
Expansion Pressure (psi): 0.45
(Reference Section 510.03.03 ITD Materials Manual)

		Idaho T-8 R-Value	Material Substitution Ratio
Layer 1:	**HMA:**		**1.80**
Layer 2:	**Aggregate Base:**	**80**	**1.00**
Layer 3:	**Granular Subbase:**	**60**	**0.85**
Layer 4:	**Subgrade:**	**60**	

Reduced Design Period Thickness Design

TI for Reduced Design Period:
HMA G_f for Reduced Design Period:
(Reference Section 510.07 ITD Materials Manual)

PAVEMENT STRUCTURE ANALYSIS

Layer Thickness

Layer 1: **HMA:**
Required Thickness: 0.36 feet
Recommended Thickness: 0.35 feet 4.2 Inches

Layer 2: **Aggregate Base:**
Required Thickness: 0.65 feet
Recommended Thickness: 0.65 feet 7.8 Inches

Layer 3: **Granular Subbase:**
Required Thickness: 0.00 feet
Recommended Thickness: 1.05 feet 12.6 Inches

Gravel Equivalency

GE1 Required: 0.64
GE1 Actual: 0.63
GE2 Required 1.28
GE2 Actual 1.28
GE3 Required 1.28
GE3 Actual 2.17

FROST PROTECTION AND SUBGRADE FILTER CRITERIA

Typical Subgrade Frost Group: F4
Design Frost Depth: 36 Inches
Required Ballast for Frost-Susceptible Subgrade: 70 % of Frost Depth

Granular Subbase on Subgrade D_{15}: mm
D_{85}: mm
D_{15}/D_{85}=

Subgrade Type

RECOMMMENDED PAVEMENT STRUCTURE

20-Year Analysis Period

HMA:	0.35	4.0
Aggregate Base:	0.65	8.0
Subgrade:	1.05	12.5
	2.05 feet	**24.5 inches**

Ballast Required for Gravel Equivalency

Required: 1.28 feet
Provided: 2.17 feet OK

Ballast Required to Counter Expansion Pressure

Required: 6 Inches
Provided: 25 Inches OK

Ballast Required for Frost Protection

Required: 25 Inches
Provided: 25 Inches OK

Prep:
QC:

APPENDIX G – FLEXIBLE PAVEMENT DESIGN

LTTPBIND OUTPUT

South Fork Teton River Bridge (KN 24341)

LTPPBind V3.1 PG Binder Selection Report (Date: 11/13/2024)

Parameter	A=2 km	B=21 km	C=39 km	D=40 km	E=41 km
Station ID	ID7644	ID8022	ID9065	ID4457	ID0470
Elevation, m	4570	4598	5732	4393	4886
Degree-Days >10 C	2460	2308	1952	2477	2152
Low Air Temperature, C	-29.6	-30.7	-33.1	-28.5	-29.6
Low Air Temp. Std Dev	4.9	4.7	4.1	4.9	3.8

Input Data

Latitude, Degree	43.8
Yearly Degree-Days>10C	2270
Lowest Yearly Air Temp., Deg. C	-30.3
Low Temp. Std. Dev., Deg. C	4.5
Base HT PG	52

Traffic Adjustments for HT

Desired Reliability, Percent	98
Traffic Loading, Million ESAL	Up to 3 M. ESAL
Traffic Speed	Fast
High Temp. Adjustment	0.0

PG Temperature	HIGH	LOW
PG Temp. at 50% Reliability	50.0	-22.3
PG Temp. at Desired Reliability	53.1	-30.2
Adjustments for Traffic	0	
Adjustments for Depth	0.0	0.0
Adjusted PG Temperature	53.1	-30.2
Selected PG Binder Grade	58	-34

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

IMPORTANT INFORMATION

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise

IMPORTANT INFORMATION

contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland