Western Technologies Inc.

GEOTECHNICAL EVALUATION REPORT

PROPOSED GRS-IBS BRIDGE
Route 6460 over Laguna Creek
Dennehotso, Arizona
WT Reference No. 3127JS001

PREPARED FOR:
Dibble Engineering
7500 North Dreamy Draw Drive, Suite No. 200
Phoenix, Arizona 85020-4996

May 8, 2017

Registered Professional Engineer (Civil)
CERTIFICATE NO.
32310
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DE LA ROCHA
Date Signed 5/8/17
ARIZONA, U.S.A.
Expires 3/31/19

Roger K. Southworth, P.E.
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The Quality People
Since 1955

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May 8, 2017

Dibble Engineering
7500 North Dreamy Draw Drive, Suite 200
Phoenix, Arizona 85020-4996

Attn: Mr. Drew Spear, P.E.

Re: Geotechnical Evaluation
Proposed GRS-IBS Bridge
Route N6460 over Laguna Creek
Dennehotso, Arizona

Job No. 3127JS001

Western Technologies Inc. (WT) has completed the geotechnical evaluation for the design of the bridge over Laguna Creek. The results of our study, including the boring location diagram, boring logs, laboratory test results, and the geotechnical recommendations are attached.

We have appreciated being of service to you in the geotechnical engineering phase of this project and are prepared to assist you during the construction phases as well. Please do not hesitate to contact us if the design conditions change or if you have any questions concerning this report.

Sincerely,
WESTERN TECHNOLOGIES INC.

Roger K. Southworth, P.E.
Managing Director

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GEOTECHNICAL EVALUATION
PROPOSED GRS-IBS BRIDGE
ROUTE 6460 OVER LAGUNA CREEK
DENNEHOTSO, ARIZONA

JOB NO. 3127JS001

1.0 PURPOSE

This report contains the results of our geotechnical evaluation for the proposed bridge over Laguna Creek. The purpose of these services is to provide information and recommendations regarding bridge design and construction. The results of the field exploration, laboratory tests, design calculation output sheets, and abutment design drawings are presented in the Appendix.

2.0 PROJECT DESCRIPTION

The project will consist of constructing a bridge over Laguna Creek in Dennehotso, Arizona. The approximate bridge location is shown on the attached Site Location Diagram (Plate 1). The bridge will consist of a Geosynthetic Reinforced Soil - Integrated Bridge System (GRS-IBS) structure with a total span of 100 feet. The bridge plan and profile is shown on the attached Plate 2. The design details for the bridge are presented in greater detail in Section 5.2 of this report.

3.0 SCOPE OF SERVICES

3.1 Field Exploration

Four borings were drilled at the abutment locations for this project. The borings were advanced to depth of 25 to 38 feet. The borings were drilled at the approximate locations indicated on the attached Boring Location Diagram (Plate 3).

A WT graduate engineer monitored the drilling operations and prepared a field log for each boring. These logs contain visual classifications of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples.

The final boring logs, included in Appendix A, represent our interpretation of the field logs and may include modifications based on laboratory observations of the recovered samples. The final logs describe the materials encountered, their thicknesses, and the depths at which samples were obtained.

The Unified Soil Classification System was used to classify the soil. The soil classification symbols appear on the boring logs and are briefly described in Appendix A.

3.2 Laboratory Testing

Laboratory tests were performed on representative samples to aid in material classification and to estimate the pertinent engineering properties of the soil. Testing was performed in general accordance with applicable ASTM methodologies. The following tests were performed and the results are presented in Appendix B.

- Dry Unit Weight
- Water Content
- Percent Passing the No. 200 Sieve
- Liquid and Plastic Limits
- Compression
- Direct Shear
- Sulfates, Chlorides, and pH

The laboratory test results were used in the development of the recommendations contained in this report.

3.3 Analyses and Report

Analyses were performed and this report was prepared for the exclusive purpose of providing geotechnical engineering information and recommendations. The scope of services for this project does not include, either specifically or by implication, any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the client is concerned about the potential for such contamination, other studies should be undertaken. We are available to discuss the scope of such studies with you.

This geotechnical engineering report includes a description of the project, a discussion of the field exploration and laboratory testing programs, a discussion of the subsurface conditions, and design recommendations as required to satisfy the purpose previously described.

4.0 SITE CONDITIONS

4.1 Surface

The bridge will be located in an undeveloped area east of an unimproved low-water crossing over Laguna Creek. The base of the low-water crossing is sandstone bedrock. The ground surface in the area of the bridge slopes down toward Laguna Creek. The banks of Laguna Creek are near vertical and are approximately 10 feet high. Groundcover generally consist of desert grasses and brush.

4.2 Subsurface

Very loose to medium dense silty sand was encountered in the borings to depths of about 9 to 13 feet. The silty sand was underlain by sandstone bedrock that extended to the boring termination depths. The upper portion of the sandstone was weathered and therefore it was not possible to core the rock until the boring was advanced several feet into the sandstone.

4.3 Groundwater

Groundwater was encountered in the borings at depths of about 9 to 13 feet during drilling. The level of the groundwater table will fluctuate seasonally with variations in the amount of precipitation, evaporation, and the water level in Laguna Creek. The observations made during this investigation must be interpreted carefully because they are short-term and do not constitute a groundwater study.

5.0 RECOMMENDATIONS

5.1 General

The recommendations contained in this report are based on our understanding of the project criteria described in Section 2.0, **Project Description**, and the assumption that the subsurface conditions are those disclosed by the test borings. Others may change the plans, final elevations, number and type of structures, foundation loads, and floor levels during design or construction. Substantially different subsurface conditions from those described herein may be encountered or become known. Any changes in the project criteria or subsurface conditions shall be brought to our attention in writing.

5.2 General GRS-IBS Information

The Geosynthetic Reinforces Soil - Integrated Bridge System (GRS-IBS) provides an economical solution to accelerated bridge construction. It is a fast, cost-effective method of bridge support that blends the roadway into the superstructure to create a jointless interface between the bridge and the approach. It consists of three main components: 1) the reinforced soil foundation (RSF), 2) the abutment, and 3) the integrated approach.

The RSF is composed of granular fill that is compacted and encapsulated with a geotextile fabric. It provides embedment and increases the bearing width and capacity of the GRS abutment. It also prevents water from infiltrating underneath and into the GRS mass from a river or stream crossing. The abutment uses alternating layers of compacted fill and closely spaced geosynthetic reinforcement to provide support for the bridge, which is placed directly on the GRS abutment without a joint and without cast-in-place (CIP) concrete. GRS is also used to construct the integrated approach to transition to the superstructure. This bridge system therefore alleviates the “bump at the bridge” problem caused by differential settlement between bridge abutments and approach roadways.

This geotechnical design of the GRS-IBS for the Dennehotso Bridge is based on the following FHWA publications:

- FHWA-HRT-11-027 “Geosynthetic Reinforced Soil Integrated Bridge System Synthesis Report” (January 2011)
- FHWA-HRT-11-026 “Geosynthetic Reinforced Soil Integrated Bridge System – Interim Implementation Guide.” (January 2011)
- GRS-IBS Design Spreadsheet 4-11-13.xlsx

Some of the current limits on GRS-IBS are for the span of the bridge with maximum spans in the range of 100 ft up to 140 ft; maximum height of the abutments of about 30 ft, and bearing stress on the abutments less than or equal to 4,000 pounds per square foot (psf). It is recommended that if the bearing stresses on the abutments are greater than 4,000 psf, the performance criteria must be checked against the applicable stress-strain curve resulting from a performance test. The performance criteria for GRS-IBS consists of a tolerable vertical strain of 0.5 percent and lateral strain of 1 percent.

GRS-IBS abutment capacities are dependent on a combination of the strength of the fill and the strength of the reinforcement when built in accordance with the two rules of GRS construction: 1) sufficient compaction (95 percent of maximum dry unit weight, according to AASHTO T99) of high-quality granular fill and 2) closely spaced layers of reinforcement (12 inches or less).

The geotechnical design of GRS-IBS includes checking the internal and external stability of the structure. The external stability analysis checks for the direct sliding, bearing capacity, global stability and overturning. The internal stability analysis checks for vertical capacity either by the empirical method or by the analytical method, deformations (both vertical and lateral deformations) and the required reinforcement strength.

5.3 Bridge Details

Based on the proposed bridge details provided by Client, the following geometric information was obtained for the geotechnical analysis:

- Span Length = 100 feet
- Maximum Wall Height = 25 feet
- Base Width of Wall = 15 feet
- Clear Spacing = 8 inches

The maximum wall height was based upon raising the site grades approximately 10 feet in order to develop the finish site grades and bearing the abutments 2 feet into the sandstone. The sandstone was encountered at depths of approximately 9 to 13 feet at the boring locations. A total wall height of 25 feet was therefore used for design.

The clear spacing is the distance between the geogrid reinforcement. The GRS-IBS requires that the reinforcement layer be placed between the blocks in the face of the wall. The height of standard CMU block is 8 inches. Therefore a clear spacing of 8 inches was used for design.

5.4 Bridge Loading

The structural loads that were assumed for bridge design are presented in the following table. We should be contacted if the actual loading is different than assumed herein.

TABLE 1 - BRIDGE LOADING

Maximum Applied Structural Loads		
Location	**Service I**	**Strength I**
Abutment 1	696 Kips	976 Kips
Abutment 2	696 Kips	976 Kips

The FHWA design methodology uses the Dead Load and Live Load as input parameters. It was assumed that the Dead Load would be about 75 percent of the Total Loads presented in Table 1.

This translates into a Dead Load of 522 kips per abutment under the Service I condition. The Live Loads were estimated from the bridge's geometric information (Approach Roadway Live Load) and Bridge Live Load, which is based on applying the HL-93 LL model. A Live Load was automatically computed by the design excel spreadsheet.

5.5 Soil and Reinforcement Parameters

Three different soil zones must be considered in the analysis of GRS-IBS structures: (1) reinforced soil zone, (2) retained soil zone (the zone right behind the reinforced soil zone), and (3) foundation soil zone. The following properties were used in the analysis/design of the GRS-IBS structure:

TABLE 2 - DESIGN SOIL PROPERTIES

Soil Property	Reinforced Soil	Retained Soil	Foundation Soil
Unit Weight	110 pcf	100 pcf	110 pcf
Cohesion	0 psf	0 psf	0 psf
Angle of Internal Friction	45°	32°	45°

The retained soil properties correspond to the properties of the native material determined in the laboratory by the direct shear test. The properties for the Reinforced Soil and Foundation Soil zones must satisfy the requirements of one of the two rules of a successful GRS construction, that is, to provide sufficient compaction (95 percent of maximum dry unit weight, according to AASHTO T99) of a high-quality granular fill.

The global stability analysis was performed using commercial software *v3.0 Reinforced Soil Stability Analysis* developed by ADAMA Engineering, Inc. The analysis was based upon the GRS-IBS being supported by the sandstone formation. The following design parameters were conservatively assumed for the sandstone in the ReSSA analysis:

- Unit Weight: 125 pcf
- Cohesion: 4,000 psf

The GRS-IBS design methodology requires the reinforcement elements to consist of geosynthetic material with an ultimate strength of at least 4,800 lbs/ft, and a strength at 2% of deformation of at least 1,370 lbs/ft. Geosynthetic materials used in all in-service GRS-IBSs structures have been a biaxial, woven polypropylene (PP) geotextiles. These material properties were used in the analysis.

5.6 Load-Resistance Factors

The load and resistance factors used in the analysis and design of the GRS-IBS structure were the default values presented in the FHWA Excel spreadsheet which are based on AASHTO LRFD Bridge Design Specifications Manual, 2010. The following load factors were used in the analysis:

TABLE 3 - LOAD FACTORS

Type of Load	Load Factor	
	Maximum	Minimum
Dead Load	1.25	0.90
Horizontal Active Earth Pressure	1.50	0.90
Vertical Earth Pressure	1.35	1.00
Earth Surcharge	1.50	0.75
Live Load Surcharge	1.75	----

The following resistance factors were used in the analysis/design:

TABLE 4 - RESISTANCE FACTORS

Resistance	Factor
Capacity Resistance	0.45
Reinforcement Resistance	0.40
Soil-Sliding Resistance	1.00
Bearing Capacity Resistance	0.65

5.7 Analysis and Design

The analysis and design was performed utilizing FHWA Excel Spreadsheet GRS-IBS Design Spreadsheet 4-11-13.xlsx using the information presented above.

The initial analysis and design considered a Bridge Beam Seat Width (Bearing Seat) of 6 ft, a Reinforcement Spacing of 8 inches, and an angle of internal friction of 38˚ but the analysis indicated that this configuration FAILED on the Ultimate Capacity check. It was also cautioned that the applied vertical stress should be limited it 4,000 psf; the analysis resulted on an applied vertical stress of 4,507 psf. A final flag was issued in the analysis indicating that a bearing bed reinforcement was needed. The bearing bed reinforcement are the short length reinforcement layers placed in between the primary reinforcement layers under the Bearing Seat of the box girders for the entire depth of the abutment or adding 37 short length reinforcement layers.

In order to improve the design the length of the Beam Seat Width was increased to 8 ft and the angle of internal friction of the material within the reinforced zones was increased to 45˚. This configuration reduced the Applied Vertical Stress to 3,750 psf and indicating that Bearing Bed Reinforcements were needed up to a depth of 3 feet or adding 5 short length intermediate reinforcement layers. The results of the Internal Stability analysis/design indicates that the performance criteria is OK.

The results of the global stability analysis performed using the ReSSA Version 3.0 software by ADAMA Engineering Inc. yielded a Factor of Safety of 3.6 which is a much greater value than the minimum required of 1.5. The output sheets of this analysis are presented in the appendices.

The output sheets for the GRS-IBS Design Spreadsheet 4-11-13.xlsx are presented in the appendices.

5.8 Recommendations

Based on the results of the analysis/design using the FHWA methodology, the base of the wall should be 15 feet wide, the beam seat should be 8 feet wide, and the reinforcement spacing should be 8 inches. The addition of Bearing Bed Reinforcement within the upper 3 feet or 5 reinforcement layers in between the primary reinforcing layers is required. The recommended angle of internal friction of the material within the reinforced zone should be 45˚.

5.3 Seismic Considerations

Based on a study completed for the Arizona Department of Transportation (1992), the maximum anticipated horizontal accelerations of bedrock for the site are 0.02 and 0.05. These values assume a 90 percent probability of non-exceedance within 50 and 250 years, respectively.

6.0 EARTHWORK

6.1 General

The conclusions contained in this report for the proposed construction are contingent upon compliance with recommendations presented in this section. Any excavating, trenching, or disturbance which occurs after completion of the earthwork must be backfilled, compacted, and tested in accordance with the recommendations contained herein. It is not reasonable to rely upon our conclusions and recommendations if any unobserved and untested trenching, grading or backfilling occurs.

6.2 Site Clearing

Site clearing may involve removal of existing structure, base course, earth embankment, temporary drainage structures, utility lines, guard rail fences and some other small features. Areas disturbed by the removal of these items should be excavated down to dense, undisturbed soil, and backfilled with native materials compacted to the appropriate densities indicated below. All exposed surfaces after clearing should be free of mounds and depressions which could prevent uniform compaction.

6.3 Excavation

The excavations for the GRS-IBS structure should conform to Section 203-5.03(A) of ADOT Standard Specifications and OSHA Construction Standards for Excavations.

We anticipate that excavations in the overburden material for the proposed construction can be accomplished with conventional equipment. Once the underlying bedrock is encountered, heavy duty, specialized equipment such as hoe rams or jack hammers, possibly together with drilling and blasting, may be required to achieve the required foundation depth.

6.4 Temporary Slopes on Soils (back of reinforced soil zone)

The overburden soils in this area consist mostly of very loose to medium dense (low blow counts and consequently low shear strength) sands, Silty SAND. They classify as Type C soils according to OSHA and the maximum allowable slopes for cuts up to 20 ft high is 1 1/2H:1V.

6.5 Materials

Based on the tests performed on samples from native material, it is recommended that this material not be used as backfill material in the reinforced zones.

Based on FHWA-HRT-12-051 “Sample Guide Specifications for Construction of Geosynthetic Reinforced Soil-Integrated Bridge System (GRS-IBS)” it is recommended that select material (from borrow sources) conforming to the following gradation requirements be used as backfill material in the reinforced zones.

TABLE 5 - REINFORCED ZONE FILL REQUIREMENTS

Description	Values	
	Well-Graded Material	**Open-Graded Material**
Maximum Grain Size (inches)	0.5 - 2	0.5 – 2
Percent Passing the No. 200 Sieve	≤ 12	≤ 5

- Plasticity Index (PI) ....6 Max.
- Angle of Internal Friction .... 45° Min.
- Backfill material for the reinforced soil zones shall be substantially free of shale or any other poor durability particles.
- Backfill material for the reinforced soil zones shall have a magnesium sulfate loss of less than 30 percent after four cycles or a sodium sulfate soundness loss of less than 15 percent after five cycles.

If imported material is required to backfill within the retained soil zone, we recommend the follow gradation:

- Gradation (ASTM C136):

percent finer by weight

6"..........100
4".......... 70-100
No. 4 Sieve.......... 50-100
No. 200 Sieve.......... 50 (max)

- Maximum expansive potential(%)*..........1.5
- Maximum soluble sulfates(%) 0.10

* Measured on a sample compacted to approximately 95 percent of the ASTM D698 maximum dry density at about 3 percent below optimum water content. The sample is confined under a 100 psf surcharge and submerged.

Geosynthetics material may be manufactured from polypropylene, high-density polyethylene, or polyester. It can be either uniaxial or biaxial. When a uniaxial type is used, higher-strength axis must be placed perpendicular to the wall face. It must have a minimum ultimate tensile strength of 4,800 lbs/ft and a reinforcement strength at 2% strain greater than the unfactored required reinforcement strength (1,370 lbs/ft).

6.6 Placement and Compaction

a. Place and compact fill in horizontal lifts, using equipment and procedures that will produce recommended water contents and densities throughout the lift. Hand-held or hand-guided equipment should be used to compact backfill material within 3 feet of the facing members.

b. Uncompacted fill lifts, other than reinforced zone backfill, should not exceed 10 inches. For the reinforced soil zone backfill, uncompacted fill lifts should not exceed 6 inches or the required reinforcement spacing by design.

c. Materials should be compacted a minimum of 95% of the maximum dry density as determined in accordance with the requirements of Arizona Test Method 225 or ASTM Test Method D698. The top five (5) feet of the abutment shall be compacted to 100% of the maximum dry density as determined in accordance with the requirements of Arizona Test Method 225 or ASTM Test Method D698.

d. Placement and compaction of backfill should generally comply with Sections 203-5.03(B)(3) and 203-5.03(B)(4) of the ADOT Standard Specifications with some appropriate modifications for the placement and compaction of backfill material for the MSE walls.

e. Jetting should not be allowed as a method of soil densification.

6.7 Compliance

The retained backfill around and behind the reinforced zones, within the reinforced zone of the GRS-IBSs should be tested to verify that the material is adequately compacted. The testing should generally comply with appropriate ASTM or AASHTO procedures.

6.0 LIMITATIONS

This report has been prepared assuming the project criteria described in Section 2.0. If changes in the project criteria occur, or if different subsurface conditions are encountered or become known, the conclusions and recommendations presented herein shall become invalid. In any such event, contact WT to assess the effect that such variations may have on our conclusions and recommendations. If WT is not retained for the construction observation and testing services to determine compliance with this report, our professional responsibility is accordingly limited.

The recommendations presented are based entirely upon data derived from a limited number of samples obtained from widely spaced borings. The attached logs are indicators of subsurface conditions only at the specific locations and times noted. This report assumes the uniformity of the geology and soil structure between borings, however variations can and often do exist. Whenever any deviation, difference or change is encountered or becomes known, WT should be contacted.

This report is for the exclusive benefit of our client alone. There are no intended third-party beneficiaries of our contract with the client or this report, and nothing contained in the contract or this report shall create any express or implied contractual or any other relationship with, or claim or cause of action for, any third party against WT.

This report is valid until the earlier of one year from the date of issuance, a change in circumstances, or discovered variations. After expiration, no person or entity shall have any right to rely on this report without the express written authorization of WT.

7.0 CLOSURE

We prepared this report as an aid to the designers of the proposed project. The comments, statements, recommendations and conclusions set forth in this report reflect the opinions of the authors. These opinions are based upon data obtained at the boring locations. Work on your project was performed in accordance with generally accepted standards and practices utilized by professionals providing similar services in this locality. No other warranty, express or implied, is made.

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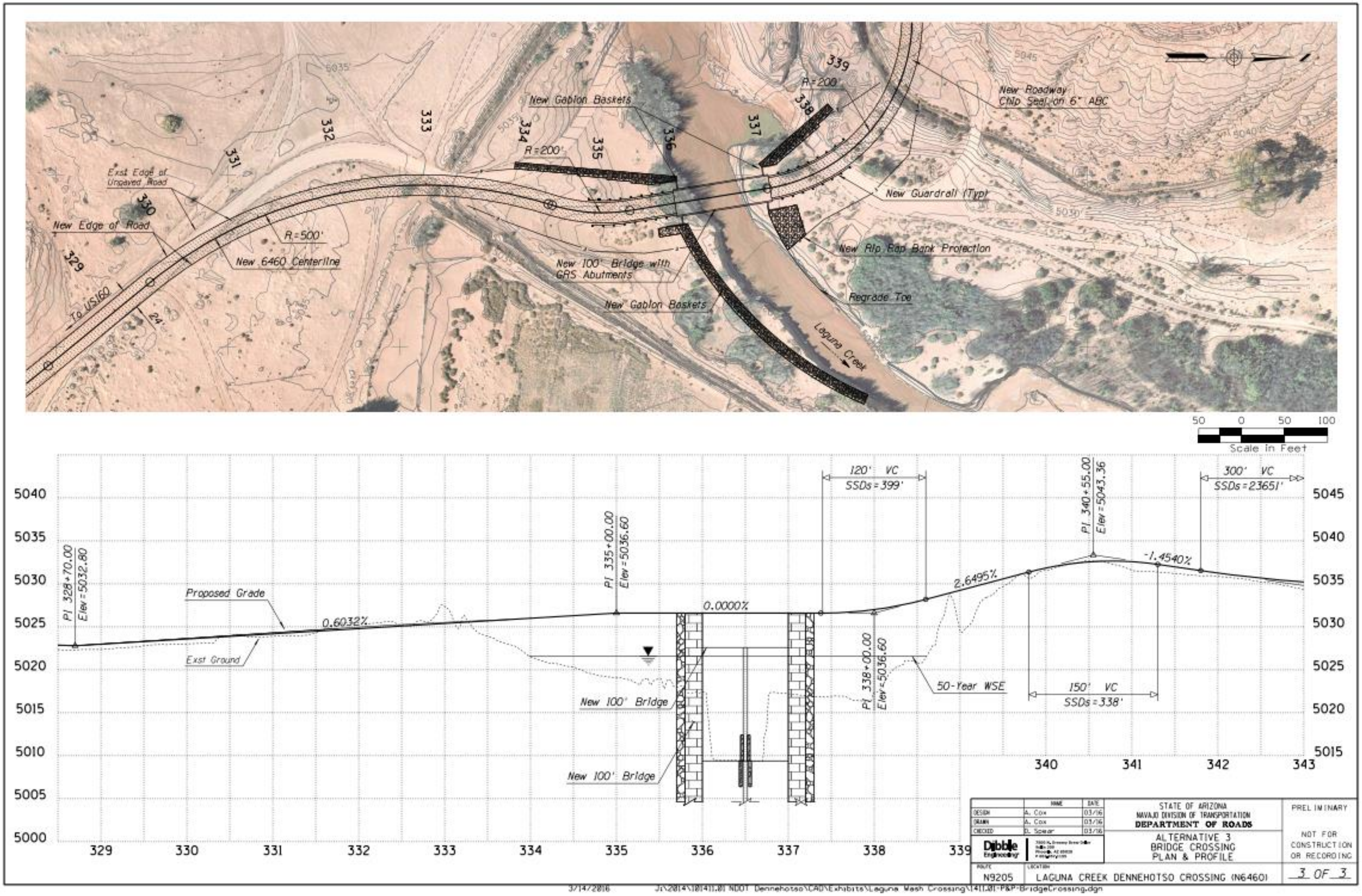
PROJECT: PROPOSED LAGUNA CREEK DENNEHOTSO CROSSING

JOB NO.: 3127JS001

PLATE: 1

SITE LOCATION DIAGRAM



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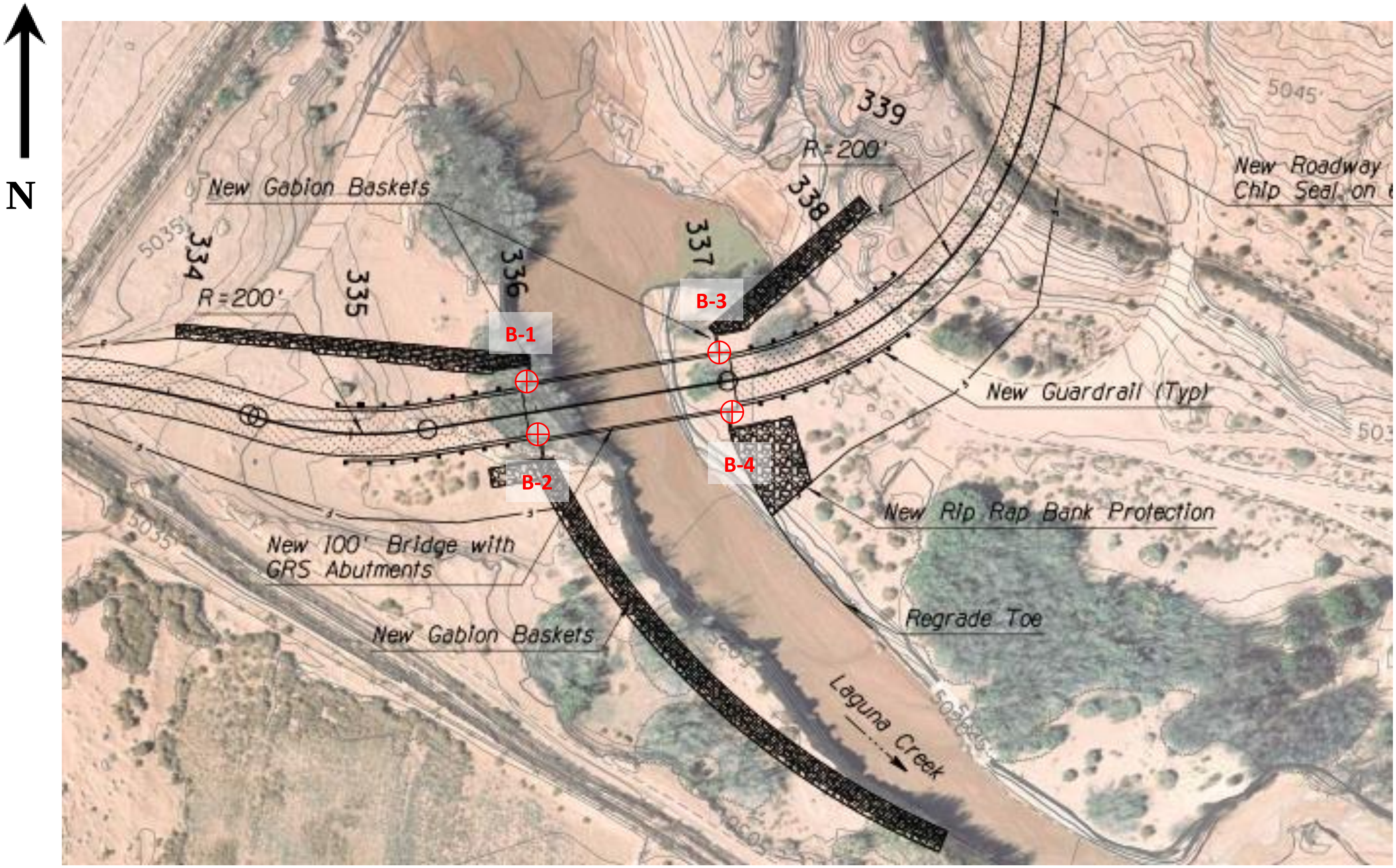
PROJECT: PROPOSED LAGUNA CREEK DENNEHOTSO CROSSING

JOB NO.: 3127JS001

BRIDGE PLAN AND PROFILE

PLATE: 2



⊕ APPROXIMATE BORING LOCATION

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PROJECT: PROPOSED LAGUNA CREEK DENNEHOTSO CROSSING
JOB NO.: 3127JS001

PLATE: 3

BORING LOCATION DIAGRAM

APPENDIX A
BORING LOGS

Allowable Soil Bearing Capacity	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
Backfill	A specified material placed and compacted in a confined area.
Base Course	A layer of specified aggregate material placed on a subgrade or subbase.
Base Course Grade	Top of base course.
Bench	A horizontal surface in a sloped deposit.
Caisson/Drilled Shaft	A concrete foundation element cast in a circular excavation which may have an enlarged base (or belled caisson).
Concrete Slabs-On-Grade	A concrete surface layer cast directly upon base course, subbase or subgrade.
Crushed Rock Base Course	A base course composed of crushed rock of a specified gradation.
Differential Settlement	Unequal settlement between or within foundation elements of a structure.
Engineered Fill	Specified soil or aggregate material placed and compacted to specified density and/or moisture conditions under observations of a representative of a soil engineer.
Existing Fill	Materials deposited through the action of man prior to exploration of the site.
Existing Grade	The ground surface at the time of field exploration.
Expansive Potential	The potential of a soil to expand (increase in volume) due to absorption of moisture.
Fill	Materials deposited by the actions of man.
Finished Grade	The final grade created as a part of the project.
Gravel Base Course	A base course composed of naturally occurring gravel with a specified gradation.
Heave	Upward movement.
Native Grade	The naturally occurring ground surface.
Native Soil	Naturally occurring on-site soil.
Rock	A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting or other methods of extraordinary force for excavation.
Sand and Gravel Base Course	A base course of sand and gravel of a specified gradation.
Sand Base Course	A base course composed primarily of sand of a specified gradation.
Scarify	To mechanically loosen soil or break down existing soil structure.
Settlement	Downward movement.
Soil	Any unconsolidated material composed of discrete solid particles, derived from the physical and/or chemical disintegration of vegetable or mineral matter, which can be separated by gentle mechanical means such as agitation in water.
Strip	To remove from present location.
Subbase	A layer of specified material placed to form a layer between the subgrade and base course.
Subbase Grade	Top of subbase.
Subgrade	Prepared native soil surface.

COARSE-GRAINED SOILS

LESS THAN 50% FINES

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
GW	WELL-GRADED GRAVEL OR WELL-GRADED GRAVEL WITH SAND, LESS THAN 5% FINES	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE
GP	POORLY-GRADED GRAVEL OR POORLY-GRADED GRAVEL WITH SAND, LESS THAN 5% FINES	
GM	SILTY GRAVEL OR SILTY GRAVEL WITH SAND, MORE THAN 12% FINES	
GC	CLAYEY GRAVEL OR CLAYEY GRAVEL WITH SAND, MORE THAN 12% FINES	
SW	WELL-GRADED SAND OR WELL-GRADED SAND WITH GRAVEL, LESS THAN 5% FINES	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE
SP	POORLY-GRADED SAND OR POORLY-GRADED SAND WITH GRAVEL, LESS THAN 5% FINES	
SM	SILTY SAND OR SILTY SAND WITH GRAVEL, MORE THAN 12% FINES	
SC	CLAYEY SAND OR CLAYEY SAND WITH GRAVEL, MORE THAN 12% FINES	

NOTE: Coarse-grained soils receive dual symbols if they contain 5% to 12% fines (e.g., SW-SM, GP-GC).

FINE-GRAINED SOILS

MORE THAN 50% FINES

GROUP SYMBOLS	DESCRIPTION	MAJOR DIVISIONS
ML	SILT, SILT WITH SAND OR GRAVEL, SANDY SILT, OR GRAVELLY SILT	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50
CL	LEAN CLAY OF LOW TO MEDIUM PLASTICITY, SANDY CLAY, OR GRAVELLY CLAY	
OL	ORGANIC SILT OR ORGANIC CLAY OF LOW TO MEDIUM PLASTICITY	
MH	ELASTIC SILT, SANDY ELASTIC SILT, OR GRAVELLY ELASTIC SILT	SILTS AND CLAYS LIQUID LIMIT MORE THAN 50
CH	FAT CLAY OF HIGH PLASTICITY, SANDY FAT CLAY, OR GRAVELLY FAT CLAY	
OH	ORGANIC SILT OR ORGANIC CLAY OF HIGH PLASTICITY	
PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	HIGHLY ORGANIC SOILS

NOTE: Fine-grained soils may receive dual classification based upon plasticity characteristics (e.g. CL-ML).

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	Above 12 in.
COBBLES	3 in. – 12 in.
GRAVEL	No. 4 – 3 in.
Coarse	¾ in. – 3 in.
Fine	No. 4 – ¾ in.
SAND	No. 200 – No. 4
Coarse	No. 10 – No. 4
Medium	No. 40 – No. 10
Fine	No. 200 – No. 40
Fines (Silt or Clay)	Below No. 200

NOTE: Only sizes smaller than three inches are used to classify soils

CONSISTENCY

CLAYS & SILTS	BLOWS PER FOOT
VERY SOFT	0 – 2
SOFT	3 – 4
FIRM	5 – 8
STIFF	9 – 15
VERY STIFF	16 – 30
HARD	OVER 30

RELATIVE DENSITY

SANDS & GRAVELS	BLOWS PER FOOT
VERY LOOSE	0 – 4
LOOSE	5 – 10
MEDIUM DENSE	11 – 30
DENSE	31 – 50
VERY DENSE	OVER 50

NOTE: Number of blows using 140-pound hammer falling 30 inches to drive a 2-inch-OD (1⅜-inch ID) split-barrel sampler (ASTM D1586).

PLASTICITY OF FINE GRAINED SOILS

PLASTICITY INDEX	TERM
0	NON-PLASTIC
1 – 7	LOW
8 – 20	MEDIUM
Over 20	HIGH

DEFINITION OF WATER CONTENT

DRY
SLIGHTLY DAMP
DAMP
MOIST
WET
SATURATED

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METHOD OF CLASSIFICATION

PLATE

A-2

The number shown in **"BORING NO."** refers to the approximate location of the same number indicated on the "Boring Location Diagram" as positioned in the field by pacing or measurement from property lines and/or existing features, or through the use of Global Positioning System (GPS) devices. The accuracy of GPS devices is somewhat variable.

"DRILLING TYPE" refers to the exploratory equipment used in the boring wherein **HSA = hollow stem auger**, and the dimension presented is the outside diameter of the HSA used.

"N" in "BLOW COUNTS" refers to a 2-inch outside diameter split-barrel sampler driven into the ground with a 140 pound drop-hammer dropped 30 inches repeatedly until a penetration of 18 inches is achieved or until refusal. The number of blows, or "blow count", of the hammer is recorded for each of three 6-inch increments totaling 18 inches. The number of blows required for advancing the sampler for the last 12 inches (2^{nd} and 3^{rd} increments) is defined as the Standard Penetration Test (SPT) "**N**"-Value. Refusal to penetration is considered more than 50 blows per 6 inches. (Ref. ASTM D1586).

"R" in "BLOW COUNTS" refers to a 3-inch outside diameter ring-lined split barrel sampler driven into the ground with a 140 pound drop-hammer dropped 30 inches repeatedly until a penetration of 12 inch is achieved or until refusal. The number of blows required to advance the sampler 12 inches is defined as the "**R**" blow count. The "**R**" blow count requires an engineered conversion to an equivalent SPT N-Value. Refusal to penetration is considered more than 50 blows per foot. (Ref. ASTM D3550).

"CS" in "BLOWS/FT." refers to a 2½-in. outside diameter California style split-barrel sampler, lined with brass sleeves, driven into the ground with a 140-pound hammer dropped 30 inches repeatedly until a penetration of 18 inches is achieved or until refusal. The number of blows of the hammer is recorded for each of the three 6-inch increments totaling 18 inches. The number of blows required for advancing the sampler for the last 12 inches (2^{nd} and 3^{rd} increments) is defined as the "CS" blow count. The "CS" blow count requires an engineered conversion to an equivalent SPT N-Value. Refusal to penetration is considered more than 50 blows for a 6-inch increment. (Ref. ASTM D 3550)

"SAMPLE TYPE" refers to the form of sample recovery, in which **N** = Split-barrel sample, **R** = Ring-lined sample, **"CS"** = California style split-barrel sample, **G** = Grab sample, **B** = Bucket sample, **C** = Core sample (ex. diamond bit rock coring).

"DRY DENSITY (LBS/CU FT)" refers to the laboratory-determined dry density in pounds per cubic foot. The symbol **"NR"** indicates that no sample was recovered.

"WATER (MOISTURE) CONTENT" (% of Dry Wt.) refers to the laboratory-determined water content in percent using the standard test method ASTM D2216.

"USCS" refers to the "Unified Soil Classification System" Group Symbol for the soil type as defined by ASTM D2487 and D2488. The soils were classified visually in the field, and where appropriate, classifications were modified by visual examination of samples in the laboratory and/or by appropriate tests.

These notes and boring logs are intended for use in conjunction with the purposes of our services defined in the text. Boring log data should not be construed as part of the construction plans nor as defining construction conditions.

Boring logs depict our interpretations of subsurface conditions at the locations and on the date(s) noted. Variations in subsurface conditions and characteristics may occur between borings. Groundwater levels may fluctuate due to seasonal variations and other factors.

The stratification lines shown on the boring logs represent our interpretation of the approximate boundary between soil or rock types based upon visual field classification at the boring location. The transition between materials is approximate and may be more or less gradual than indicated.

DATE DRILLED: **1-17-17**
LOCATION: **See Boring Location Diagram**
ELEVATION: **Not determined**

BORING NO. B-1

EQUIPMENT TYPE: **CME-75**
DRILLING TYPE: **7" HSA**
FIELD ENGINEER: **C. Dumrtru**

WATER CONTENT (%)	POCKET PENETROMETER (tsf)	SAMPLE TYPE	BLOWS/FT.	DEPTH (FEET)	USCS	SOIL DESCRIPTION
		G			SP-SM	POORLY GRADED SAND; with silt, orange-brown, medium dense, damp
		R	27			
6.2		R	6	5		changing to very loose
19.6		R	10		SM	SILTY SAND; orange-brown, loose, moist
						SANDSTONE; orange-brown, soft to moderately hard
		N	50/6"	10		
		N	50/6"	15		
		N	50/4"	20		
		N	50/4"	25		
		C		30		
				35		
						Boring terminated at 38 feet

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

N- STANDARD PENETRATION TEST
R- RING SAMPLE
NR- NO SAMPLE RECOVERY
G- GRAB SAMPLE
B- BUCKET SAMPLE
BN- BLUNT NOSE PENETROMETER

NOTES: **Groundwater encountered at 9 feet during drilling**

WESTERN TECHNOLOGIES INC.

PROJECT: **PROPOSED DENNEHOTSO BRIDGE**
REF. NO.: **3127JS001**

PLATE **A-4**

BORING LOG

DATE DRILLED: **1-17-17**
LOCATION: **See Boring Location Diagram**
ELEVATION: **Not determined**

BORING NO. B-2

EQUIPMENT TYPE: **CME-75**
DRILLING TYPE: **7" HSA**
FIELD ENGINEER: **C. Dumrtru**

WATER CONTENT (%)	POCKET PENETROMETER (tsf)	SAMPLE TYPE	BLOWS/FT.	DEPTH (FEET)	USCS	SOIL DESCRIPTION
		G		0	SM	SILTY SAND; orange-brown, loose to medium dense, damp
2.7		R	16			
3.6		R	14	5		
8.1		R	21			
				9		SANDSTONE; orange-brown, soft to moderately hard
		N	50/4"	10		
		N	50/4"	15		
		N	50/4"	20		
		N C	50/2"	25		
				30		
				35		**Boring terminated at 35 feet**

N- STANDARD PENETRATION TEST
R- RING SAMPLE
NR- NO SAMPLE RECOVERY
G- GRAB SAMPLE
B- BUCKET SAMPLE
BN- BLUNT NOSE PENETROMETER

NOTES: **Groundwater encountered at 9 feet during drilling**

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

WESTERN TECHNOLOGIES INC.

PROJECT: **PROPOSED DENNEHOTSO BRIDGE**
REF. NO.: **3127JS001**

PLATE **A-5**

BORING LOG

DATE DRILLED: **1-18-17**
LOCATION: **See Boring Location Diagram**
ELEVATION: **Not determined**

BORING NO. B-3

EQUIPMENT TYPE: **CME-75**
DRILLING TYPE: **7" HSA**
FIELD ENGINEER: **C. Dumrtru**

WATER CONTENT (%)	POCKET PENETROMETER (tsf)	SAMPLE TYPE	BLOWS/FT.	DEPTH (FEET)	USCS	SOIL DESCRIPTION
		G		0	SP-SM	POORLY GRADED SAND; with silt, orange-brown, loose, damp
0.6		R	16			
1.0		R	12	5		
3.1		R	10			
						changing to medium dense
		N	10	10		
				13		SANDSTONE; orange-brown, soft to moderately hard
		N C	50/1"	15		
				20		
				25		**Boring terminated at 25 feet**
				30		
				35		

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

N- STANDARD PENETRATION TEST
R- RING SAMPLE
NR- NO SAMPLE RECOVERY
G- GRAB SAMPLE
B- BUCKET SAMPLE
BN- BLUNT NOSE PENETROMETER

NOTES: **Groundwater encountered at 13 feet during drilling**

WESTERN TECHNOLOGIES INC.

PROJECT: **PROPOSED DENNEHOTSO BRIDGE**
REF. NO.: **3127JS001**

BORING LOG

PLATE **A-6**

DATE DRILLED: **1-18-17**
LOCATION: **See Boring Location Diagram**
ELEVATION: **Not determined**

BORING NO. B-4

EQUIPMENT TYPE: **CME-75**
DRILLING TYPE: **7" HSA**
FIELD ENGINEER: **C. Dumrtru**

WATER CONTENT (%)	POCKET PENETROMETER (tsf)	SAMPLE TYPE	BLOWS/FT.	DEPTH (FEET)	USCS	SOIL DESCRIPTION
		G		0	SM	SILTY SAND; orange-brown, loose, damp
0.9		R	14	2		
		R	14	5		
5.5		R	12	7		
				9		SANDSTONE; orange-brown, soft to moderately hard
		N	50/4'	10		
		N C	50/2"	15		
				25		**Boring terminated at 25 feet**

THIS SUMMARY APPLIES ONLY AT THIS LOCATION AND AT THE TIME OF LOGGING. CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH TIME. DATA PRESENTED IS A SIMPLIFICATION.

N- STANDARD PENETRATION TEST
R- RING SAMPLE
NR- NO SAMPLE RECOVERY
G- GRAB SAMPLE
B- BUCKET SAMPLE
BN- BLUNT NOSE PENETROMETER

NOTES: **Groundwater encountered at 9 feet during drilling**

WESTERN TECHNOLOGIES INC.

PROJECT: **PROPOSED DENNEHOTSO BRIDGE**
REF. NO.: **3127JS001**

PLATE **A-7**

BORING LOG

APPENDIX B
LABORATORY TEST RESULTS

Boring No.	Depth (ft.)	USCS Class.	Dry Density (pcf)	Water Content (%)	Compression Properties			Plasticity		Percent Passing #200	Remarks
					Surcharge (ksf)	Total Compression (%)		Liquid Limit	Plasticity Index		
						In-Situ	After Saturation				
B-1	5 - 6	SP-SM	97	6.2						7.2	
B-1	7 - 8	SM	102	19.6						14.4	
B-2	2 - 3	SM	96	2.7	0.69	0.8				35.9	1
					1.38	2.7	3.5				2
B-2	5 - 6	SM	101	3.6						29.6	
B-2	7 - 8	SM	108	8.1						16.8	
B-3	2 - 3	SP-SM	105	0.6	0.69	0.9				11.9	1
					1.38	2.2	3.2				2
B-3	5 - 6	SP-SM		1.0						8.1	
B-3	7 - 8	SP-SM	104	3.1						11.3	
B-4	2 - 3	SM		0.9						13.7	
B-4	7 - 8	SM	98	5.5						14.8	

Note: Initial Dry Density and Initial Water Content are in-situ values unless otherwise noted.
NP = Non-Plastic

Remarks

1. Test performed on undisturbed sample
2. Submerged to approximate saturation.

Geotechnical
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PROJECT: PROPOSED GRS-IBS BRIDGE
JOB NO.: 3127JS001

SOIL PROPERTIES

PLATE
B-1

APPENDIX C

GRS-IBS EXCEL SPREADSHEET OUTPUT RESULTS

ASD

PERFORMANCE CRITERIA

Tolerable Vertical Strain	$\varepsilon_{v,tol}$	0.5	%
Tolerable Lateral Strain	$\varepsilon_{h,tol}$	1	%

LAYOUT

Span Length	L_{span}	78	ft
Wall Height	H	15.25	ft
Width of Wall Facing Element	b_{block}	0.64	ft
Length of Individual Wall Facing Element	L_{block}	1.30	ft
Height of Individual Wall Facing Element	H_{block}	0.64	ft
Weight of Individual Facing Element	W_{block}	44	lb
Number of Facing Elements in a Single Column	N_{block}	24	
Base Width of Wall (including wall facing)	B_{total}	6	ft
Base Width of Wall (not including wall facing)	B	5.36	ft
Check Base to Height Ratio ≥ 0.3	**B/H**	**0.35**	**OK**
Set Back (Section 4.3.4, FHWA-HRT-11-026)	a_b	12	in
Clear Space (Section 4.3.4, FHWA-HRT-11-026)	d_e	4	in
Minimum Base Width of Reinforced Soil Foundation (Section 4.3.4, FHWA-HRT-11-026)	B_{RSF}	7.50	ft
Minimum Depth of Reinforced Soil Foundation (Section 4.3.4, FHWA-HRT-11-026)	D_{RSF}	1.5	ft
Minimum Distance of RSF in front of Abutment (Section 4.3.4, FHWA-HRT-11-026)	x_{RSF}	1.50	ft
Reinforcement Spacing	S_v	7.625	in
Number of Reinforcement Layers	N_{Sv}	24	
Secondary Reinforcement Spacing	$S_{v,s}$	3.8125	in

SOIL AND REINFORCEMENT CONDITIONS

Retained Soil Unit Weight	γ_b	125	lb/ft^3
Retained Soil Undrained Shear Strength	c_b	500	lb/ft^2
Retained Soil Effective Cohesion	c'_b	0	lb/ft^2
Retained Soil Friction Angle	ϕ_b	28	deg
Active Earth Pressure Coefficient - Backfill	K_{ab}	0.36	
Reinforced Fill Unit Weight	γ_r	110	lb/ft^3
Maximum Diameter of Reinforced Fill	d_{max}	0.5	in
Reinforced Fill Cohesion	c_r	0	lb/ft^2
Reinforced Fill Friction Angle	ϕ_r	48	deg
Active Earth Pressure Coefficient - Reinforced Fill	K_{ar}	0.15	
Passive Earth Pressure Coefficient - Reinforced Fill	K_{pr}	6.79	
Foundation Soil Unit Weight	γ_f	125	lb/ft^3
Foundation Soil Effective Unit Weight	γ'_f	62.6	lb/ft^3
Foundation Soil Undrained Shear Strength	c_f	4000	lb/ft^2
Foundation Soil Effective Cohesion	c'_f	4000	lb/ft^2
Foundation Soil Friction Angle	ϕ_f	0	deg
Active Earth Pressure Coefficient - Foundation	K_{af}	1.00	
Road Base Unit Weight	γ_{rb}	140	lb/ft^3
Road Base Cohesion	c_{rb}	0	lb/ft^2
Road Base Friction Angle	ϕ_{rb}	40	deg
Active Earth Pressure Coefficient - Road Base	K_{arb}	0.22	

Reinforced Soil Foundation Unit Weight	γ_{rsf}	140	lb/ft^3
Reinforced Soil Foundation Effective Unit Weight	γ'_{rsf}	77.6	lb/ft^3
Reinforced Soil Foundation Friction Angle	ϕ_{rsf}	40	deg
Ultimate Reinforcement Strength	T_f	4800	lb/ft
Reinforcement Strength at 2%	$T_{@\varepsilon=2\%}$	1370	lb/ft

SAFETY FACTORS

Capacity	$FS_{capacity}$	3.5	
Reinforcement Strength	FS_{reinf}	3.5	
Direct Slide	FS_{slide}	1.5	
Bearing Capacity	$FS_{bearing}$	2.5	
Global Stability	FS_{global}	1.5	

LOADING CONDITIONS

Geometry

Equivalent Height for Traffic (Table 3.11.6.4-1 in AASHTO LRFD Bridge Design Specs, 2010)	$H_{t,eq}$	2.48	ft
Height of Bridge Beam	H_{bridge}	3.00	ft
Bridge Seat Width	b	4	ft
Width of Bridge	B_b	34	ft
Width of Traffic and Road Base Surcharge Over Wall	$b_{rb,t}$	0.36	ft

Dead Loads

Total Dead Load	Q_{bridge}	707200	lb
Dead Load per Abutment	$Q_{abutment}$	353600	lb
Bridge Surcharge	q_b	2600	lb/ft^2
Road Base Surcharge	q_{rb}	420.00	lb/ft^2

Weight of GRS Abutment including facing (Eq. 16 - modified, FHWA-HRT-11-026)	W	8991	lb/ft
Weight of RSF	W_{RSF}	873	lb/ft
Weight of Wall Face	W_{face}	811	lb/ft

Live Loads

Approach Roadway Live Load	q_t	310	lb/ft^2
Bridge Live Load (HL-93)	q_{LL}	1400	lb/ft^2

Bearing Stress

Applied Vertical Stress (Eq. 24, FHWA-HRT-11-026)	$V_{applied}$	4000	lb/ft^2

EXTERNAL STABILITY

Direct Slide at Base of Abutment

Driving Forces

Lateral Load from Retained Soil (Eq. 10, FHWA-HRT-11-026)	F_b	5248	lb/ft
Lateral Load from Road Base (Eq. 11, FHWA-HRT-11-026)	F_{rb}	2312	lb/ft
Lateral Load from Traffic Surcharge (Eq. 12, FHWA-HRT-11-026)	F_t	1707	lb/ft
Total Driving Force (Eq. 13, FHWA-HRT-11-026)	F_n	9267	lb/ft

Resisting Forces

Resisting Weight (Eq. 15, FHWA-HRT-11-026)	W_t	20354	lb/ft
Critical Friction Angle (Section 4.3.6.1, FHWA-HRT-11-026)	ϕ_{crit}	38	deg
Sliding Friction (Section 4.3.6.1, FHWA-HRT-11-026)	μ	0.79	
Total Resisting Force (Eq. 14, FHWA-HRT-11-026)	R_n	16132	lb/ft
Factor of Safety for Direct Slide (Eq. 17, FHWA-HRT-11-026)	$FS_{slide,calc}$	1.74	
Direct Slide Check		**OK**	

Direct Slide at Base of RSF

Driving Forces

Lateral Load from Retained Soil above RSF (Eq. 10, FHWA-HRT-11-026)	F_b	5248	lb/ft
Lateral Load from Road Base (Eq. 11, FHWA-HRT-11-026)	F_{rb}	1530	lb/ft
Lateral Load from Traffic Surcharge (Eq. 12, FHWA-HRT-11-026)	F_t	1875	lb/ft
Lateral Load from Retained Fill and Foundation Soil behind RSF	F_f	2930	lb/ft
Total Driving Force (Eq. 13, FHWA-HRT-11-026)	F_n	11582	lb/ft

Resisting Forces (note: passive resistance in front of RSF is ignored)

Resisting Weight (Eq. 15, FHWA-HRT-11-026)	W_t	21227	lb/ft
Critical Friction Angle (Section 4.3.6.1, FHWA-HRT-11-026)	ϕ_{crit}	38	deg
Sliding Friction (Section 4.3.6.1, FHWA-HRT-11-026)	μ	0.78	
Passive Resistance in front of RSF	R_p	6035.21	lb/ft
Assumed Adhesion Resistance of Foundation Soil	R_{af}	20000	lb/ft
Total Resisting Force (Eq. 14, FHWA-HRT-11-026)	R_n	42619	lb/ft

Factor of Safety for Direct Slide (Eq. 17, FHWA-HRT-11-026)	$FS_{slide,calc}$	3.68	
Direct Slide Check		**OK**	

Bearing Capacity

Bearing Pressure

Driving Moments

Traffic		14294.33	
Road Base		19366.51	
Retained Soil		29299.54	
Face		1565.25	
Driving Moments (Equation may vary depending on geometry - check for your conditions)	ΣM_D	64526	ft-lb/ft

Resisting Moments

Weight		9621	
DL		14456	
LL		7784	
Road Base		540	
Traffic		398	
Resisting Moments (Equation may vary depending on geometry - check for your conditions)	ΣM_R	32799	ft-lb/ft
Total Vertical Load (Eq. 19, FHWA-HRT-11-026)	ΣV	26938	lb/ft
Eccentricity (Eq. 20, FHWA-HRT-11-026)	$e_{B,n}$	1.18	ft
Vertical Pressure at the Base (Eq. 18, FHWA-HRT-11-026)	$\sigma_{v,base,n}$	5236.32	lb/ft^2

Bearing Capacity

Bearing Capacity Factors (Table 10.6.3.1.2a-1, AASHTO LRFD Bridge Design Specs, 2010)	N_c	30.10	
	N_γ	22.40	
	N_q	18.40	
Bearing Capacity (Eq. 21, FHWA-HRT-11-026)	q_n	125734.66	lb/ft^2
Factor of Safety for Bearing Capacity (Eq. 22, FHWA-HRT-11-026)	$FS_{bearing,calc}$	24.01	
Bearing Capacity Check		**OK**	

Global Stability

Stability Program Selected		ReSSA	
Global Stability FS	FS_{GS}	1.58	
Global Stability Check		**OK**	

INTERNAL STABILITY

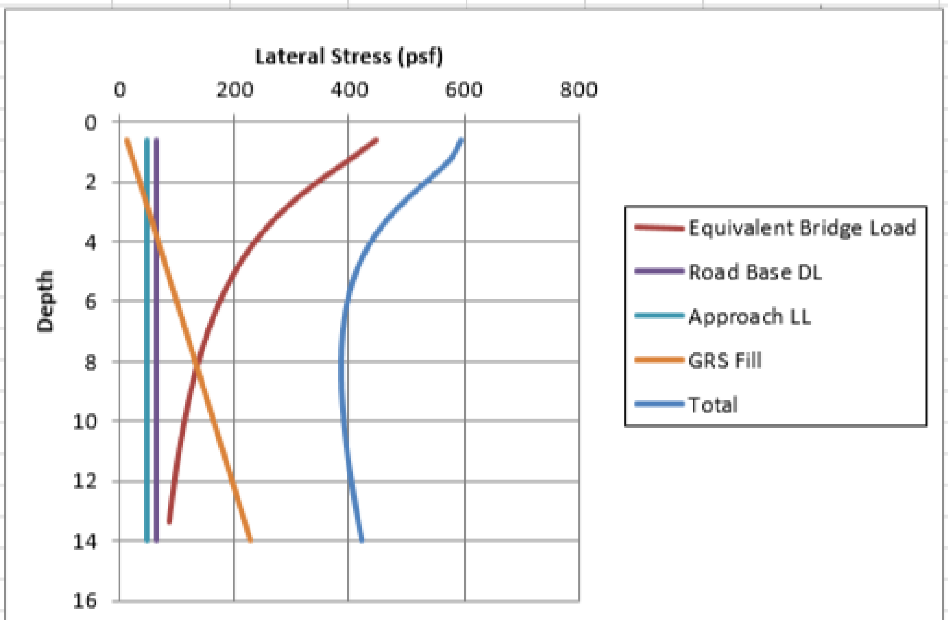
Deformations

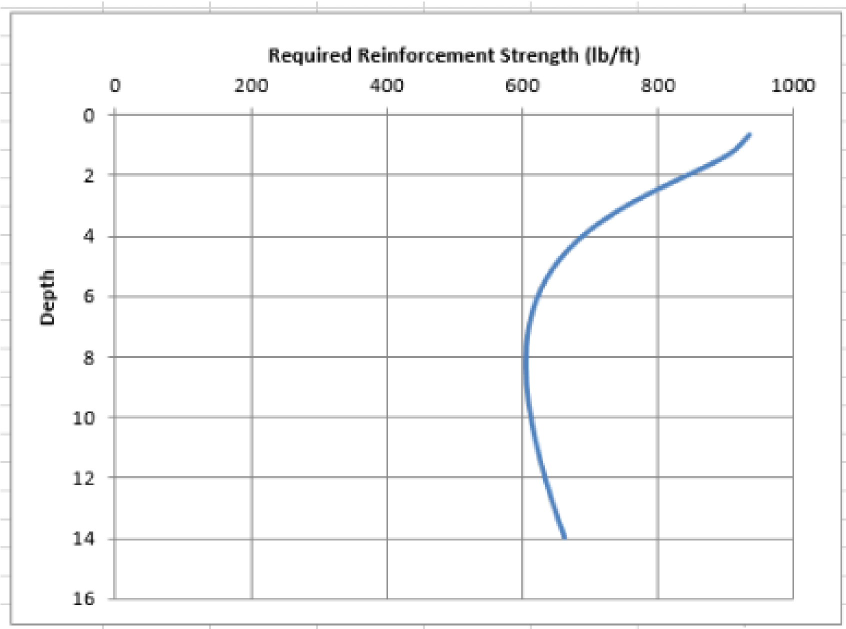
Vertical Strain (From applicable performance test)	ε_v	0.4	%
Vertical Deformation	D_V	0.73	in
Vertical Strain Check		**OK**	

Lateral Strain (Eq. 30, FHWA-HRT-11-026)	ε_L	0.8	%
Lateral Deformation (Eq. 29, FHWA-HRT-11-026)	D_L	0.48	in
Lateral Deformation Check		**OK**	
Ultimate Capacity - Empirical			
Capacity (From applicable performance test)	$q_{ult,emp}$	25000	lb/ft^2
Allowable Load (Eq. 23, FHWA-HRT-11-026)	$V_{allow,emp}$	7143	lb/ft^2
Capacity Check		**OK**	
Ultimate Capacity - Analytical			
Ultimate Capacity (Eq. 25, FHWA-HRT-11-026)	$q_{ult,an}$	20707	lb/ft^2
Allowable Load (Eq. 27, FHWA-HRT-11-026)	$V_{allow,an}$	5916	lb/ft^2
Capacity Check		**OK**	
Reinforcement Strength			
Allowable Reinforcement Strength (Eq. 38, FHWA-HRT-11-026)	T_{allow}	1371	lb/ft
Reinforcement Strength at 2%	$T_{@\varepsilon=2\%}$	1370	lb/ft
Equivalent Bridge Load	q_{bridge}	3270	lb/ft^2
Maximum Required Reinforcement Strength	T_{req}	934	lb/ft
Reinforcement Strength Check		**OK**	
Serviceability Check		**OK**	
Minimum Required Depth of Bearing Bed Reinforcement	z_s	2.86	ft
Minimum Number of Bearing Reinforcement Layers	$N_{Sv,s}$	5	

REQUIRED REINFORCEMENT STRENGTH - ASD

		Dist. from top of wall	Equivalent Bridge Load			Road Base DL and Approach LL		GRS Fill	Total	Req. Strength	Ultimate Check	2% Check
No.	Layer	z (ft)	α	β	$\sigma_{h,bridge}$ (psf)	$\sigma_{h,rb}$ (psf)	$\sigma_{h,t}$ (psf)	$\sigma_{h,W}$ (psf)	$\sigma_{h,total}$ (psf)	T_{req} (lb/ft)	$T_{req} > T_{allow}$	T_{req} >T@2%
1	1	0.6	2.53	-1.26	476	62	46	10	594	934	NO	NO
2	2	1.3	2.01	-1.00	447	62	46	21	575	905	NO	NO
3	3	1.9	1.62	-0.81	401	62	46	31	540	849	NO	NO
4	4	2.5	1.33	-0.67	354	62	46	41	502	790	NO	NO
5	5	3.2	1.12	-0.56	311	62	46	51	470	739	NO	NO
6	6	3.8	0.97	-0.48	274	62	46	62	444	698	NO	NO
7	7	4.4	0.85	-0.42	244	62	46	72	424	667	NO	NO
8	8	5.1	0.75	-0.37	219	62	46	82	409	644	NO	NO
9	9	5.7	0.67	-0.34	199	62	46	93	399	628	NO	NO
10	10	6.4	0.61	-0.30	181	62	46	103	392	617	NO	NO
11	11	7.0	0.56	-0.28	167	62	46	113	387	610	NO	NO
12	12	7.6	0.51	-0.26	154	62	46	124	385	606	NO	NO
13	13	8.3	0.48	-0.24	143	62	46	134	384	605	NO	NO
14	14	8.9	0.44	-0.22	133	62	46	144	385	606	NO	NO
15	15	9.5	0.41	-0.21	125	62	46	154	387	609	NO	NO
16	16	10.2	0.39	-0.19	118	62	46	165	390	614	NO	NO
17	17	10.8	0.37	-0.18	111	62	46	175	394	619	NO	NO
18	18	11.4	0.35	-0.17	105	62	46	185	398	626	NO	NO
19	19	12.1	0.33	-0.16	100	62	46	196	403	634	NO	NO
20	20	12.7	0.31	-0.16	95	62	46	206	409	643	NO	NO
21	21	13.3	0.30	-0.15	91	62	46	216	414	652	NO	NO
22	22	14.0	0.28	-0.14	87	62	46	227	421	662	NO	NO
23	23	14.6	0.27	-0.14	83	62	46	237	427	672	NO	NO
24	24	15.3	0.26	-0.13	80	62	46	247	434	683	NO	NO





LRFD

Inputs

PERFORMANCE CRITERIA

Tolerable Vertical Strain	$\varepsilon_{v,tol}$	0.5	%
Tolerable Lateral Strain	$\varepsilon_{h,tol}$	1	%

LAYOUT

Span Length	L_{span}	100	ft
Wall Height	H	25	ft
Width of wall facing	b_{block}	0.64	ft
Length of Individual Wall Facing Element	L_{block}	1.30	ft
Height of Individual Wall Facing Element	H_{block}	0.64	ft
Weight of Individual Facing Element	W_{block}	44	lb
Number of Facing Elements in a Single Column	N_{block}	39	
Base Width of Wall (including wall facing)	B_{wf}	15	ft
Base Width of Wall (not including wall facing)	B	14.36	ft
Check Base to Height Ratio ≥ 0.3	**B/H**	**0.57**	**OK**
Set Back (Section 4.3.4, FHWA-HRT-11-026)	a_b	8	in
Clear Space (Section 4.3.4, FHWA-HRT-11-026)	d_e	6	in
Minimum Base Width of Reinforced Soil Foundation (Section 4.3.4, FHWA-HRT-11-026)	B_{RSF}	18.75	ft
Minimum Depth of Reinforced Soil Foundation (Section 4.3.4, FHWA-HRT-11-026)	D_{RSF}	3.75	ft
Minimum Distance of RSF in front of Abutment (Section 4.3.4, FHWA-HRT-11-026)	x_{RSF}	3.75	ft
Reinforcement Spacing	S_v	8	in
Number of Reinforcement Layers	N_{Sv}	38	
Secondary Reinforcement Spacing	$S_{v,s}$	4	in

SOIL AND REINFORCEMENT CONDITIONS

Retained Soil Unit Weight	γ_b	100	lb/ft^3
Retained Soil Undrained Shear Strength	c_b	0	lb/ft^2
Retained Soil Effective Cohesion	c'_b	0	lb/ft^2
Retained Soil Friction Angle	ϕ_b	32	deg
Active Earth Pressure Coefficient - Backfill	K_{ab}	0.31	
Reinforced Fill Unit Weight	γ_r	125	lb/ft^3
Maximum Diameter of Reinforced Fill	d_{max}	0.5	in
Reinforced Fill Cohesion	c_r	0	lb/ft^2
Reinforced Fill Friction Angle	ϕ_r	45	deg
Active Earth Pressure Coefficient - Reinforced Fill	K_{ar}	0.17	
Passive Earth Pressure Coefficient - Reinforced Fill	K_{pr}	5.83	
Foundation Soil Unit Weight	γ_f	125	lb/ft^3
Foundation Soil Effective Unit Weight	γ'_f	62.6	lb/ft^3
Foundation Soil Undrained Shear Strength	c_f	10000	lb/ft^2
Foundation Soil Effective Cohesion	c'_f	10000	lb/ft^2
Foundation Soil Friction Angle	ϕ_f	0	deg
Active Earth Pressure Coefficient - Foundation	K_{af}	1.00	
Road Base Unit Weight	γ_{rb}	140	lb/ft^3
Road Base Cohesion	c_{rb}	0	lb/ft^2
Road Base Friction Angle	ϕ_{rb}	40	deg
Active Earth Pressure Coefficient - Road Base	K_{arb}	0.22	
Reinforced Soil Foundation Unit Weight	γ_{rsf}	125	lb/ft^3
Reinforced Soil Foundation Effective Unit Weight	γ'_{rsf}	62.6	lb/ft^3
Reinforced Soil Foundation Friction Angle	ϕ_{rsf}	45	deg
Ultimate Reinforcement Strength	T_f	4800	lb/ft
Reinforcement Strength at 2%	$T_{@\varepsilon=2\%}$	1370	lb/ft

LOAD AND RESISTANCE FACTORS

Load Combination (Section 3.4, AASHTO LRFD Bridge Design Specs, 2010)	LC	STRENGTH 1
Dead Load Max (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{DC\ MAX}$	1.25
Dead Load Min (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{DC\ MIN}$	0.9
Horizontal Active Earth Pressure Max (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{EH\ MAX}$	1.5
Horizontal Active Earth Pressure Min (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{EH\ MIN}$	0.9
Vertical Earth Pressure Max (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{EV\ MAX}$	1.35
Vertical Earth Pressure Min (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{EV\ MIN}$	1
Earth Surcharge Max (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{ES\ MAX}$	1.5
Earth Surcharge Min (Table 3.4.1-2, AASHTO LRFD Bridge Design Specs, 2010)	$\gamma_{ES\ MIN}$	0.75
Live Load Surcharge (Table 3.4.1-1, AASHTO LRFD Bridge Design Specs, 2010)	γ_{LS}	1.75
Capacity Resistance (Section C.2.2.1, FHWA-HRT-11-026)	ϕ_{cap}	0.45
Reinforcement Resistance (Section C.2.2.3, FHWA-HRT-026)	ϕ_{reinf}	0.4
Soil-Sliding Resistance (Table 11.5.6-1, AASHTO LRFD Bridge Design Specs, 2010)	ϕ_t	1
Bearing Capacity Resistance (Table 11.5.6-1, AASHTO LRFD Bridge Design Specs, 2010)	ϕ_{bc}	0.65

LOADING CONDITIONS

Geometry

Equivalent Height for Traffic (Table 3.11.6.4-1 in AASHTO LRFD Bridge Design Specs, 2010)	$H_{t,eq}$	3	ft
Height of Bridge Beam	H_{bridge}	3.25	ft
Bridge Seat Width	b	8	ft
Width of Bridge Beam	B_b	28	ft
Width of Traffic and Roadbase Load Behind Wall	$b_{rb,t}$	5.70	ft

Dead Loads

Total Dead Load	Q_{bridge}	1044000	lb
Dead Load per Abutment	$Q_{abutment}$	522000	lb
Bridge Surcharge	q_b	2330	lb/ft^2
Road Base Surcharge	q_{rb}	455.00	lb/ft^2
Weight of GRS Abutment (Eq. 16, FHWA-HRT-11-026)	W	44889	lb/ft
Weight of RSF	W_{RSF}	4402	lb/ft
Weight of Wall Face	W_{face}	1330	lb/ft

Live Loads

Approach Roadway Live Load	q_t	300	lb/ft^2
Bridge Live Load (HL-93)	q_{LL}	1400	lb/ft^2

Bearing Stress

Applied Vertical Stress (Eq. 24, FHWA-HRT-11-026)	$V_{applied}$	3730	lb/ft^2
Factored Applied Vertical Stress (Eq. 82, FHWA-HRT-11-026)	$V_{applied,f}$	5363	lb/ft^2

EXTERNAL STABILITY

Direct Slide at Base of Abutment

Driving Forces

Lateral Load from Retained Soil (Eq. 10, FHWA-HRT-11-026)	F_b	9602	lb/ft
Lateral Load from Road Base (Eq. 11, FHWA-HRT-11-026)	F_{rb}	3495	lb/ft
Lateral Load from Traffic Surcharge (Eq. 12, FHWA-HRT-11-026)	F_t	2304	lb/ft
Total Driving Force (Eq. 13, FHWA-HRT-11-026)	F_n	15401	lb/ft
Factored Driving Force (Eq. 106, FHWA-HRT-11-026)	F_R	23678	lb/ft

Resisting Forces

Resisting Weight (Eq. 72, FHWA-HRT-11-026)	$W_{t,R}$	63612	lb/ft
Critical Friction Angle (Section 4.3.6.1, FHWA-HRT-11-026)	ϕ_{crit}	38	deg
Sliding Friction (Section 4.3.6.1, FHWA-HRT-11-026)	μ	0.78	
Factored Resisting Force (Eq. 71, FHWA-HRT-11-026)	R_R	49699.38	lb/ft
Direct Slide Check		**OK**	

Direct Slide at Base of RSF

Driving Forces

Lateral Load from Retained Soil above RSF (Eq. 10, FHWA-HRT-11-026)	F_b	9602	lb/ft
Lateral Load from Road Base (Eq. 11, FHWA-HRT-11-026)	F_{rb}	2844	lb/ft
Lateral Load from Traffic Surcharge (Eq. 12, FHWA-HRT-11-026)	F_t	2650	lb/ft
Lateral Load from Retained Fill and Foundation Soil behind RSF	F_f	9815	lb/ft
Total Driving Force (Eq. 13, FHWA-HRT-11-026)	F_n	24912	lb/ft
Factored Driving Force (Eq. 106, FHWA-HRT-11-026)	F_R	38030	lb/ft

Resisting Forces

Resisting Weight (Eq. 15, FHWA-HRT-11-026)	$W_{t,R}$	64809	lb/ft
Critical Friction Angle (Section 4.3.6.1, FHWA-HRT-11-026)	ϕ_{crit}	38	deg
Sliding Friction (Section 4.3.6.1, FHWA-HRT-11-026)	μ	0.78	
Factored Passive Resistance in front of RSF	$R_{p,R}$	33948.07	lb/ft
Assumed Adhesion Resistance of Foundation Soil	$R_{af,R}$	93750	lb/ft
Total Factored Resisting Force (Eq. 14, FHWA-HRT-11-026)	R_R	178332	lb/ft
Direct Slide Check		**OK**	

Bearing Capacity

Bearing Pressure

Driving Moments

Traffic		57971.04	
Road Base		75362.35	
Retained Soil		138026.29	
Face		8820.20	
Driving Moments (Equation may vary depending on geometry - check for your conditions)	ΣM_D	280180	ft-lb/ft

Resisting Moments

Weight		132879	
DL		-7525	
LL		-6329	
Road Base		25379	
Traffic		19522	
Resisting Moments (Equation may vary depending on geometry - check for your conditions)	ΣM_R	163926	ft-lb/ft
Total Factored Vertical Load (Eq. 75, FHWA-HRT-11-026)	$\Sigma V_{R,R}$	117988	lb/ft
Eccentricity (Eq. 76, FHWA-HRT-11-026)	$e_{B,R}$	0.99	ft
Vertical Pressure at the Base (Eq. 74, FHWA-HRT-11-026)	$\sigma_{v,base,R}$	7032	lb/ft^2

Bearing Capacity

Bearing Capacity Factors (Table 10.6.3.1.2a-1, AASHTO LRFD Bridge Design Specs, 2010)	N_c	35.50		For phi = 32°
	N_γ	30.2		
	N_q	23.2		
Factored Bearing Capacity (Eq. 77, FHWA-HRT-11-026)	q_R	258405	lb/ft^2	
Bearing Capacity Check		**OK**		

Global Stability

Stability Program Selected		ReSSA		
Global Stability FS	FS_{GS}	3.6		
Global Stability Check		**OK**		

INTERNAL STABILITY

Deformations

Vertical Strain using unfactored loads (From applicable performance test)	ε_v	0.46	%	From Fig. 20 FHWA-HRT-11-026
Vertical Deformation	D_V	1.38	in	
Vertical Strain Check		**OK**		
Lateral Strain (Eq. 30, FHWA-HRT-11-026)	D_L	0.92	%	
Lateral Deformation (Eq. 29, FHWA-HRT-11-026)	D_L	0.96	in	
Lateral Deformation Check		**OK**		

Ultimate Capacity - Empirical

Nominal Capacity (From applicable performance test)	$q_{ult,emp}$	26000	lb/ft^2
Factored Capacity	$q_{ult,emp,f}$	11700	lb/ft^2
Capacity Check		**OK**	
Nominal Capacity (Eq. 81, FHWA-HRT-11-026)	$q_{n,an}$	16211	lb/ft^2
Factored Resistance Capacity	$q_{n,an,f}$	7295	lb/ft^2
Capacity Check		**OK**	

Reinforcement Strength

Factored Reinforcement Capacity (Eq. 93, FHWA-HRT-11-026)	$T_{f,f}$	1920	lb/ft
Reinforcement Strength at 2%	$T_{@\varepsilon=2\%}$	1370	lb/ft
Equivalent Bridge Load (Unfactored)	q_{bridge}	2975	lb/ft^2
Equivalent Bridge Load (Factored)	$q_{bridge,f}$	4155	lb/ft^2
Maximum Factored Required Reinforcement Strength	$T_{req,f}$	1638	lb/ft
Reinforcement Strength Check		**OK**	
Serviceability Check		**FAILED. BEARING BED REINFORCEMENT NEEDED.**	
Minimum Required Depth of Bearing Bed Reinforcement	z_s	3.00	ft
Minimum Number of Bearing Reinforcement Layers	$N_{Sv,s}$	5	

REQUIRED REINFORCEMENT STRENGTH - LRFD

Equivalent Bridge Load				Road Base DL and Approach LL				GRS Fill		Factored	Unfactored	Factored	Ultimate Check	Unfactored	2% Check
α	β	$\sigma_{h,bridge}$ (psf)	$\sigma_{h,bridge,f}$ (psf)	$\sigma_{h,rb}$ (psf)	$\sigma_{h,rb,f}$ (psf)	$\sigma_{h,t}$ (psf)	$\sigma_{h,t,f}$ (psf)	$\sigma_{h,W}$ (psf)	$\sigma_{h,W,f}$ (psf)	$\sigma_{h,total,f}$ (psf)	$\sigma_{h,total}$ (psf)	$T_{req,f}$ (lb/ft)	$T_{req,f} > T_{f,f}$	T_{req} (lb/ft)	$T_{req} > T@2\%$
2.8	-1.4	510	712	78	117	51	90	14	19	938	653	1619	NO	1128	NO
2.5	-1.2	503	703	78	117	51	90	29	39	949	662	1638	NO	1142	NO
2.2	-1.1	490	684	78	117	51	90	43	58	949	662	1638	NO	1143	NO
2.0	-1.0	469	656	78	117	51	90	57	77	940	656	1622	NO	1132	NO
1.8	-0.9	445	621	78	117	51	90	71	97	925	646	1596	NO	1114	NO
1.6	-0.8	418	583	78	117	51	90	86	116	906	633	1564	NO	1093	NO
1.4	-0.7	391	546	78	117	51	90	100	135	888	620	1533	NO	1071	NO
1.3	-0.6	365	510	78	117	51	90	114	154	872	609	1504	NO	1051	NO
1.2	-0.6	341	476	78	117	51	90	129	174	857	599	1479	NO	1034	NO
1.1	-0.5	319	446	78	117	51	90	143	193	846	592	1460	NO	1021	NO
1.0	-0.5	299	417	78	117	51	90	157	212	837	586	1444	NO	1011	NO
0.9	-0.5	281	392	78	117	51	90	172	232	831	582	1434	NO	1004	NO
0.9	-0.4	264	369	78	117	51	90	186	251	827	580	1427	NO	1000	NO
0.8	-0.4	249	348	78	117	51	90	200	270	826	579	1425	NO	999	NO
0.8	-0.4	236	329	78	117	51	90	214	290	826	580	1425	NO	1000	NO
0.7	-0.4	223	312	78	117	51	90	229	309	828	582	1429	NO	1004	NO
0.7	-0.3	212	296	78	117	51	90	243	328	832	585	1435	NO	1009	NO
0.6	-0.3	202	282	78	117	51	90	257	347	837	589	1444	NO	1016	NO
0.6	-0.3	193	269	78	117	51	90	272	367	843	594	1455	NO	1025	NO
0.6	-0.3	184	257	78	117	51	90	286	386	850	600	1468	NO	1035	NO
0.6	-0.3	176	246	78	117	51	90	300	405	859	606	1482	NO	1046	NO
0.5	-0.3	169	236	78	117	51	90	315	425	868	613	1498	NO	1058	NO
0.5	-0.3	162	227	78	117	51	90	329	444	878	621	1515	NO	1071	NO
0.5	-0.2	156	218	78	117	51	90	343	463	888	629	1533	NO	1085	NO
0.5	-0.2	150	210	78	117	51	90	357	483	900	637	1553	NO	1100	NO
0.5	-0.2	145	202	78	117	51	90	372	502	911	646	1573	NO	1115	NO
0.4	-0.2	140	195	78	117	51	90	386	521	924	655	1594	NO	1131	NO
0.4	-0.2	135	189	78	117	51	90	400	540	936	665	1616	NO	1148	NO
0.4	-0.2	131	183	78	117	51	90	415	560	950	675	1639	NO	1165	NO
0.4	-0.2	127	177	78	117	51	90	429	579	963	685	1662	NO	1182	NO
0.4	-0.2	123	171	78	117	51	90	443	598	977	696	1686	NO	1200	NO
0.4	-0.2	119	166	78	117	51	90	458	618	991	706	1711	NO	1219	NO
0.4	-0.2	116	162	78	117	51	90	472	637	1006	717	1736	NO	1237	NO
0.3	-0.2	112	157	78	117	51	90	486	656	1020	728	1761	NO	1256	NO
0.3	-0.2	109	153	78	117	51	90	500	676	1035	739	1787	NO	1276	NO
0.3	-0.2	106	149	78	117	51	90	515	695	1051	751	1813	NO	1295	NO
0.3	-0.2	104	145	78	117	51	90	529	714	1066	762	1840	NO	1315	NO

APPENDIX D
GLOBAL STABILITY ANALYSIS

Geotechnical
Environmental
Inspections
Materials

Western Technologies Inc.
The Quality People
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Dennehotso Bridge: GRS-IBS

PROJECT IDENTIFICATION

Title:	Dennehotso Bridge: GRS-IBS
Project Number:	3127JS001 -
Client:	Dibble Engineering
Designer:	Armando de la Rocha
Station Number:	Abutment 2

Description:

Preliminary Abutment Analysis

Company's information:

Name:	Western Technologies, Inc.
Street:	3737 East Broadway Road Phoenix, AZ 85040
Telephone #:	(602) 437-3737
Fax #:	(602) 470-1341
E-Mail:	armando.dlr@wt-us.com

Original file path and name: C:\Program Files\ADAMA\ReSSA(3.0)\Dennehotso Bridge.MSE
Original date and time of creating this file: 5/2/17

PROGRAM MODE: Analysis of a Simplified Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
REINFORCED SOIL..	110.0	45.0	0.0
RETAINED SOIL...	100.0	32.0	0.0
FOUNDATION SOIL.......................................	125.0	0.0	4000.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Geosynthetic type #1	4800.00	1.20	1.10	1.67	1.00	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Geosynthetic type #1	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.050
Design horizontal seismic coefficient, kh = Am = 0.5 x Ao = 0.025 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - SIMPLE

GEOMETRY

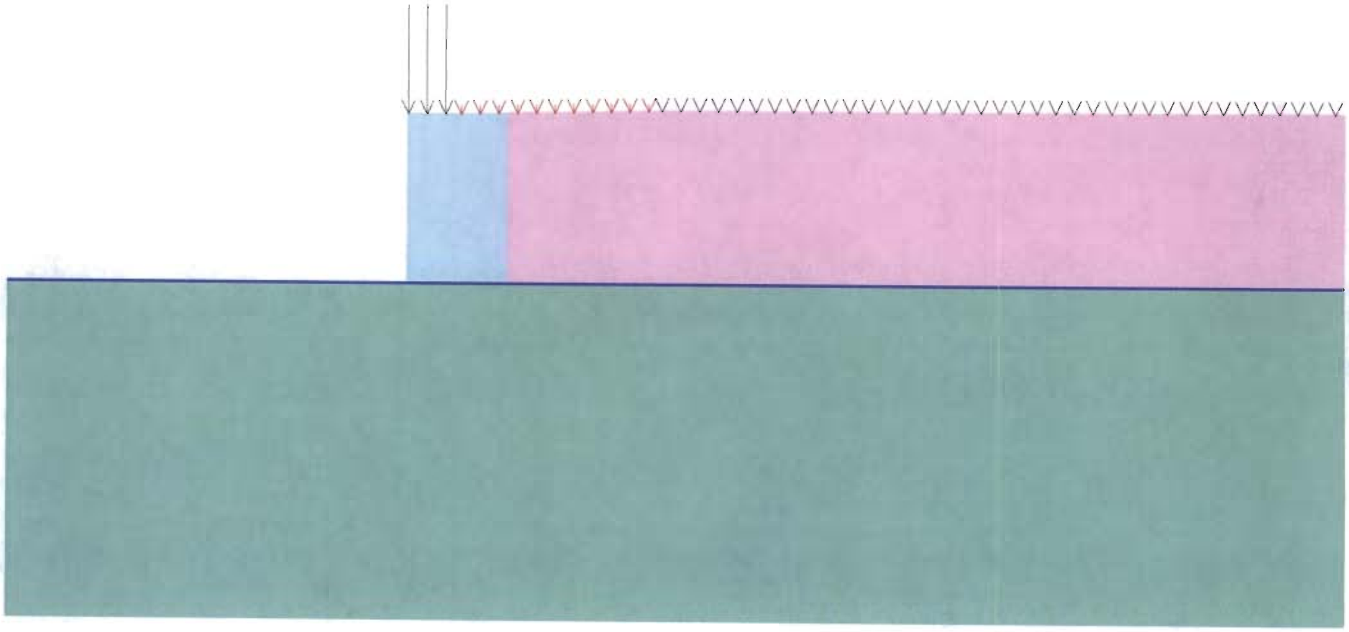
Height of slope, H	25.00	[ft]
Slope angle, i	90.00	[deg.]
Horizontal crest length, A	8.00	[ft]
Horizontal crest length, B	30.00	[ft]
Backslope angle, β	1.00	[deg.]
Sloping angle, α	0.00	[deg.]

UNIFORM SURCHARGE

Surcharge load over A, Q1	3730.00	[lb/ft²]
Surcharge load over backslope B, Q2	300.00	[lb/ft²]
Surcharge load away from backslope, Q3	10.00	[lb/ft²]

WATER GEOMETRY Coordinates of water line in [ft]

#	1	Xw =	0.00	Yw =	0.00
#	2	Xw =	16.40	Yw =	0.00
#	3	Xw =	26.25	Yw =	0.00
#	4	Xw =	32.81	Yw =	0.00



SCALE:

0 5 10 15 20 25 30 [ft]

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	15.00	25.12	9.98	25.04	12.35	33.80	9.08	54.01	
2	21.50	25.25	-25.23	0.15	-8.60	25.25	30.11	5.74	
3	28.00	25.35	-4.15	0.03	1.88	25.46	26.13	3.97	
4	34.50	25.46	-0.75	0.04	3.83	30.83	31.14	3.27	
5	41.00	25.52	-0.75	0.03	4.26	38.76	39.06	3.12	
. 6	47.50	25.52	-0.75	0.02	3.85	49.72	49.91	3.07 .	OK
7	54.00	25.52	-0.75	0.02	3.82	61.73	61.88	3.19	
8	60.50	25.52	-0.75	0.02	4.63	73.39	73.57	3.32	
9	67.00	25.52	-0.75	0.01	4.20	89.60	89.72	3.35	
10	73.50	25.52	-0.75	0.01	4.96	104.20	104.35	3.53	
11	80.00	25.52	-0.75	0.01	3.89	125.85	125.92	3.55	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	-25.45	0.24	34.50	25.46	-5.50	36.68	41.54	4.17	
2	-21.58	0.04	34.50	25.46	-3.64	35.04	39.33	4.03	
3	-18.17	0.08	34.50	25.46	-2.11	34.09	37.61	3.93	
4	-14.88	0.17	34.50	25.46	-0.14	32.25	35.30	3.83	
5	-11.47	0.15	41.00	25.52	1.38	40.51	42.36	3.73	
6	-7.73	0.04	41.00	25.52	0.27	44.08	44.76	3.56	
7	-4.18	0.02	47.50	25.52	2.37	51.87	52.26	3.34	
. 8	-0.75	0.02	47.50	25.52	3.85	49.72	49.91	3.07 .	OK
9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A	#10 - Overhanging Cliff
10	8.06	25.00	34.50	25.46	6.50	872.76	847.76	40.09	
11	9.90	25.07	34.50	25.46	21.75	54.11	31.36	29.84	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 3.07

Critical Circle: Xc = 3.85[ft], Yc = 49.72[ft], R = 49.91[ft]. (Number of slices used = 66)

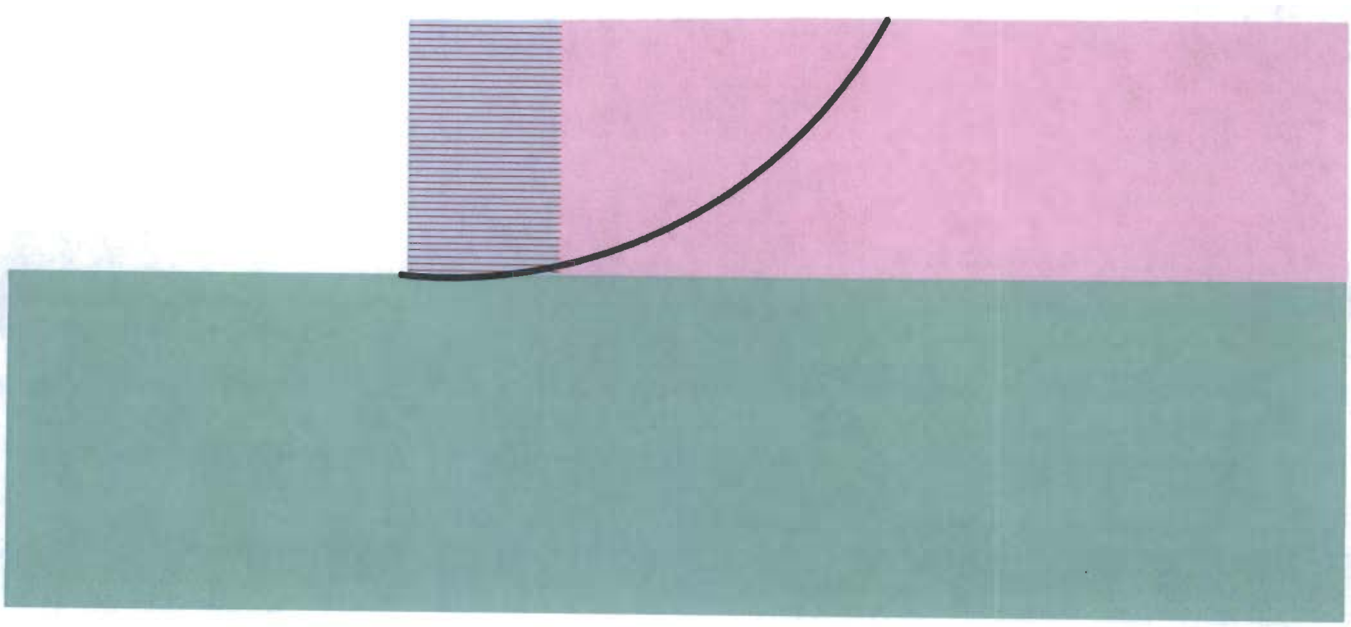
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

NOT CONDUCTED

Three-Part Wedge Stability Analysis

NOT CONDUCTED

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	Geosynthetic type #1	0.50	15.00	1.00	0.00	0.50	15.00	0.50	0.00	0.00
2	1	Geosynthetic type #1	1.17	15.00	1.00	0.00	1.17	15.00	1.17	0.00	0.00
3	1	Geosynthetic type #1	1.84	15.00	1.00	0.00	1.84	15.00	1.84	0.00	0.00
4	1	Geosynthetic type #1	2.51	15.00	1.00	0.00	2.51	15.00	2.51	0.00	0.00
5	1	Geosynthetic type #1	3.18	15.00	1.00	0.00	3.18	15.00	3.18	0.00	0.00
6	1	Geosynthetic type #1	3.85	15.00	1.00	0.00	3.85	15.00	3.85	0.00	0.00
7	1	Geosynthetic type #1	4.52	15.00	1.00	0.00	4.52	15.00	4.52	0.00	0.00
8	1	Geosynthetic type #1	5.19	15.00	1.00	0.00	5.19	15.00	5.19	0.00	0.00
9	1	Geosynthetic type #1	5.86	15.00	1.00	0.00	5.86	15.00	5.86	0.00	0.00
10	1	Geosynthetic type #1	6.53	15.00	1.00	0.00	6.53	15.00	6.53	0.00	0.00
11	1	Geosynthetic type #1	7.20	15.00	1.00	0.00	7.20	15.00	7.20	0.00	0.00
12	1	Geosynthetic type #1	7.87	15.00	1.00	0.00	7.87	15.00	7.87	0.00	0.00
13	1	Geosynthetic type #1	8.54	15.00	1.00	0.00	8.54	15.00	8.54	0.00	0.00
14	1	Geosynthetic type #1	9.21	15.00	1.00	0.00	9.21	15.00	9.21	0.00	0.00
15	1	Geosynthetic type #1	9.88	15.00	1.00	0.00	9.88	15.00	9.88	0.00	0.00
16	1	Geosynthetic type #1	10.55	15.00	1.00	0.00	10.55	15.00	10.55	0.00	0.00
17	1	Geosynthetic type #1	11.22	15.00	1.00	0.00	11.22	15.00	11.22	0.00	0.00
18	1	Geosynthetic type #1	11.89	15.00	1.00	0.00	11.89	15.00	11.89	0.00	0.00
19	1	Geosynthetic type #1	12.56	15.00	1.00	0.00	12.56	15.00	12.56	0.00	0.00
20	1	Geosynthetic type #1	13.23	15.00	1.00	0.00	13.23	15.00	13.23	0.00	0.00
21	1	Geosynthetic type #1	13.90	15.00	1.00	0.00	13.90	15.00	13.90	0.00	0.00
22	1	Geosynthetic type #1	14.57	15.00	1.00	0.00	14.57	15.00	14.57	0.00	0.00
23	1	Geosynthetic type #1	15.24	15.00	1.00	0.00	15.24	15.00	15.24	0.00	0.00
24	1	Geosynthetic type #1	15.91	15.00	1.00	0.00	15.91	15.00	15.91	0.00	0.00
25	1	Geosynthetic type #1	16.58	15.00	1.00	0.00	16.58	15.00	16.58	0.00	0.00
26	1	Geosynthetic type #1	17.25	15.00	1.00	0.00	17.25	15.00	17.25	0.00	0.00
27	1	Geosynthetic type #1	17.92	15.00	1.00	0.00	17.92	15.00	17.92	0.00	0.00
28	1	Geosynthetic type #1	18.59	15.00	1.00	0.00	18.59	15.00	18.59	0.00	0.00
29	1	Geosynthetic type #1	19.26	15.00	1.00	0.00	19.26	15.00	19.26	0.00	0.00
30	1	Geosynthetic type #1	19.93	15.00	1.00	0.00	19.93	15.00	19.93	0.00	0.00
31	1	Geosynthetic type #1	20.60	15.00	1.00	0.00	20.60	15.00	20.60	0.00	0.00
32	1	Geosynthetic type #1	21.27	15.00	1.00	0.00	21.27	15.00	21.27	0.00	0.00
33	1	Geosynthetic type #1	21.94	15.00	1.00	0.00	21.94	15.00	21.94	0.00	0.00
34	1	Geosynthetic type #1	22.61	15.00	1.00	0.00	22.61	15.00	22.61	0.00	0.00
35	1	Geosynthetic type #1	23.28	15.00	1.00	0.00	23.28	15.00	23.28	0.00	0.00
36	1	Geosynthetic type #1	23.95	15.00	1.00	0.00	23.95	15.00	23.95	0.00	0.00
37	1	Geosynthetic type #1	24.62	15.00	1.00	0.00	24.62	15.00	24.62	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	*Designated Name*	*Coverage Ratio*	*Area of reinforcemnt [ft²] / length of slope [ft]*
1	Geosynthetic type #1	1.00	555.00

APPENDIX E
PLAN SHEETS

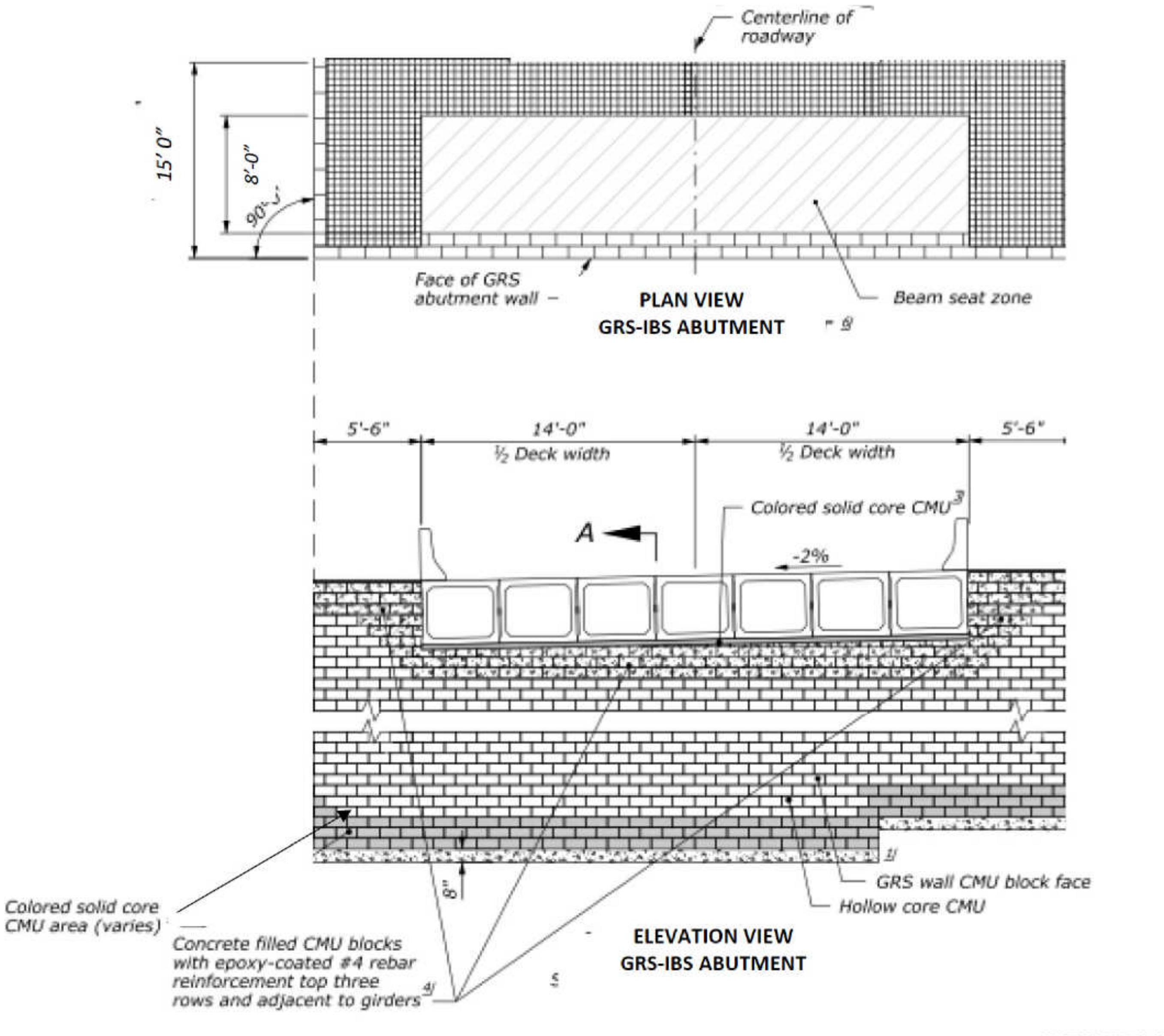


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PLATE 1. **D-1 PLAN VIEW**
Proposed Dennehotso Bridge
Plan and Elevation View

WT Job No. 3127JS001

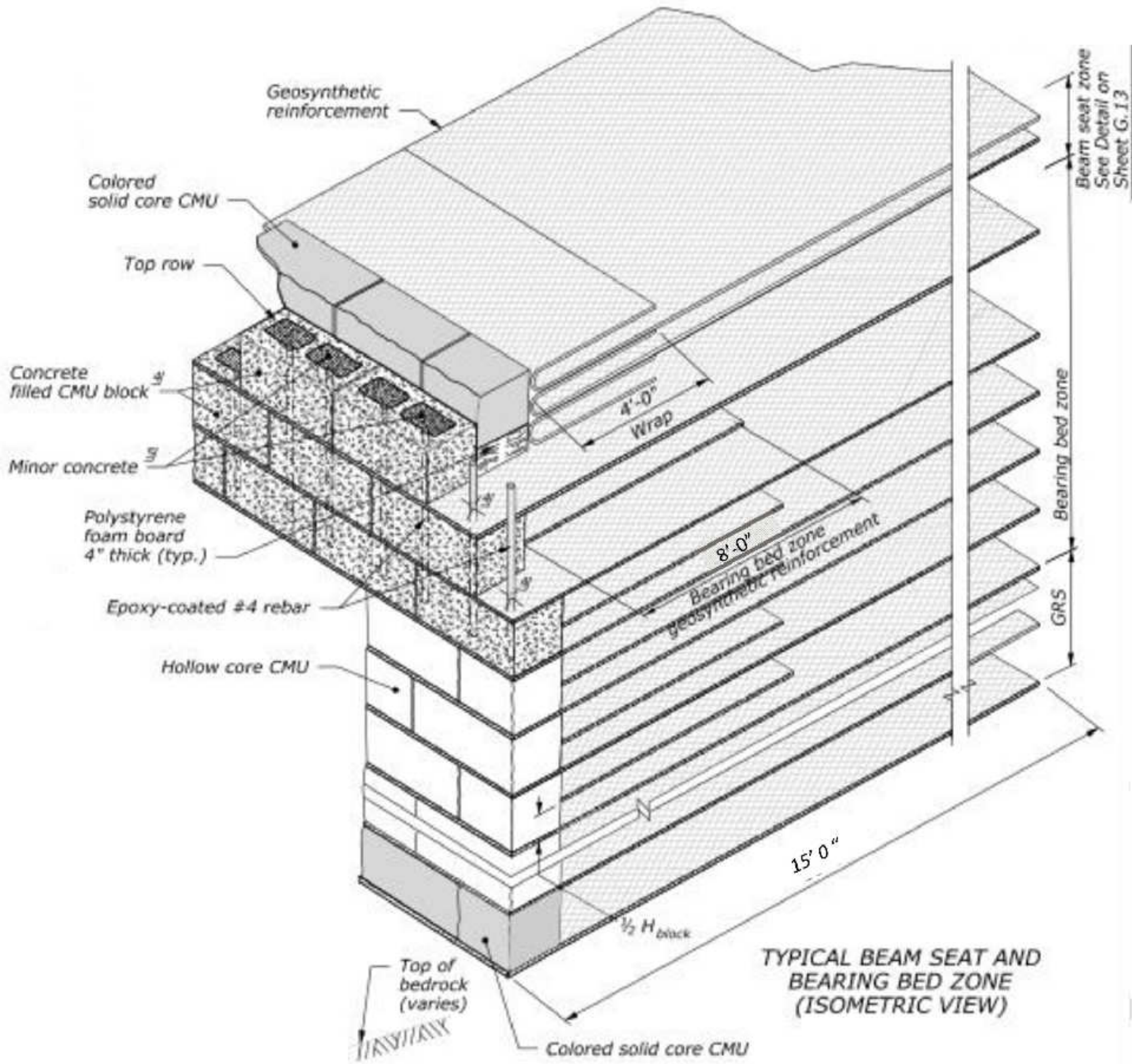

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PLATE 2. D-2 TYPICAL BEAM SEAT
Proposed Dennehotso Bridge

WT Job No. 3127JS001